McGill University Design Project Reports
Design of a Deep Excavation at Oak Island, Nova Scotia
Compiled by Les MacPhie, March 2017

I had the privilege of assigning a Design Project and acting as external advisor to eight groups (one group per year) of fourth year civil engineering students from McGill University, Montreal, Quebec. The Design Project was a full course for the students with the objective of introducing them to the approach used by a consulting engineering firm to produce a practical design and an order of magnitude cost estimate.

Three of the groups were assigned the task of designing a large diameter caisson to investigate conditions at a depth of 190 to 200 feet at the Money Pit, Oak Island, Nova Scotia where drilling programs identified archaeological findings consistent with the presence of underground chambers at that depth. The archaeological findings at 190 to 200 feet depth are described in a two part article by John Wonnacott and Les MacPhie which is posted on the Oak Island Compendium web site. Frequent meetings were held with the students. I was assisted in some of the meetings by the knowledgeable input of D'Arcy O'Connor or John Wonnacott. The Design Projects demonstrate that there are proven technologies to successfully excavate a large diameter caisson to 200 feet. The technologies considered included ground freezing, secant piles and grouting. A deep excavation would find the Chappell vault and the fallen chests from the 1861 collapse in the Money Pit. Also the excavation would determine the nature and content of the underground works at 200 feet depth.

The reports by the students are compiled in this document together with group photos. A copy of the article on archeological findings is also included in this document. The tabulation below gives a summary of the contents of this compiled document.

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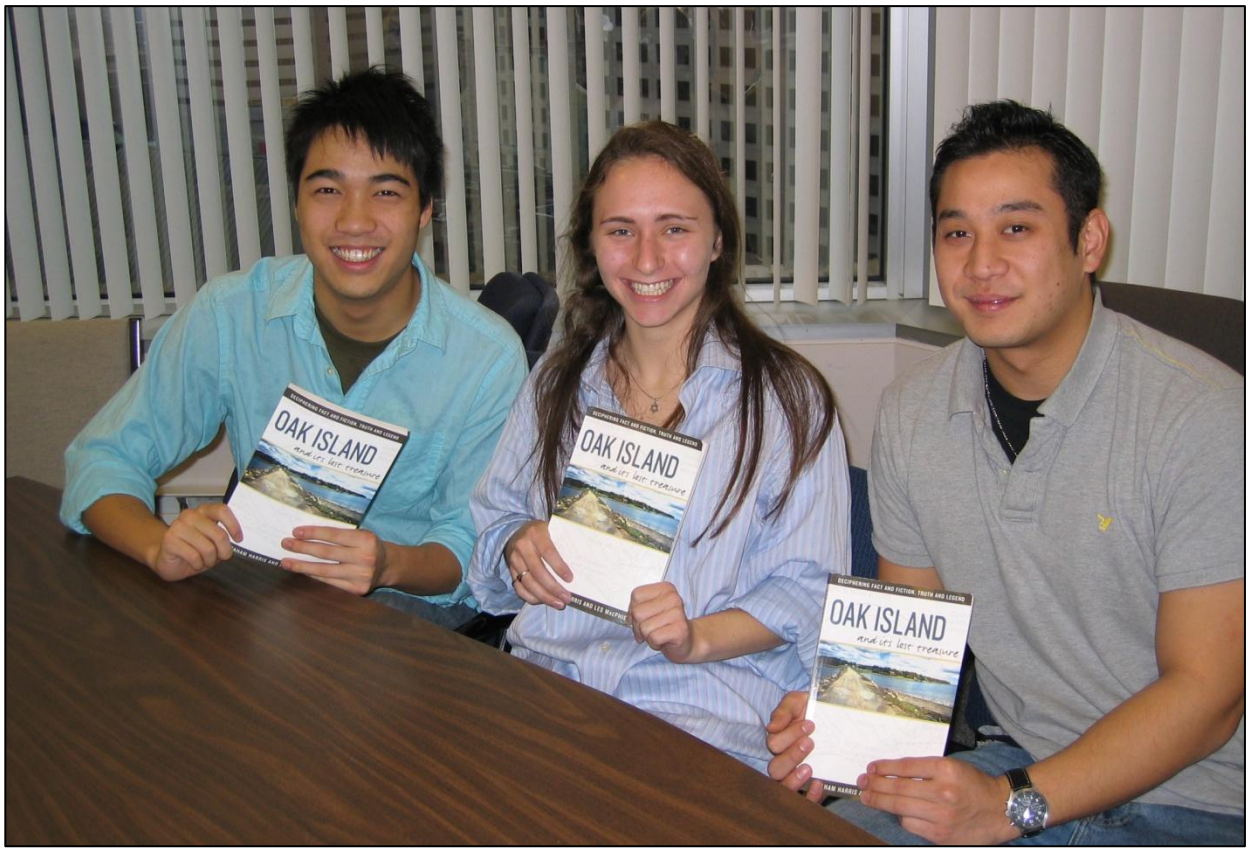
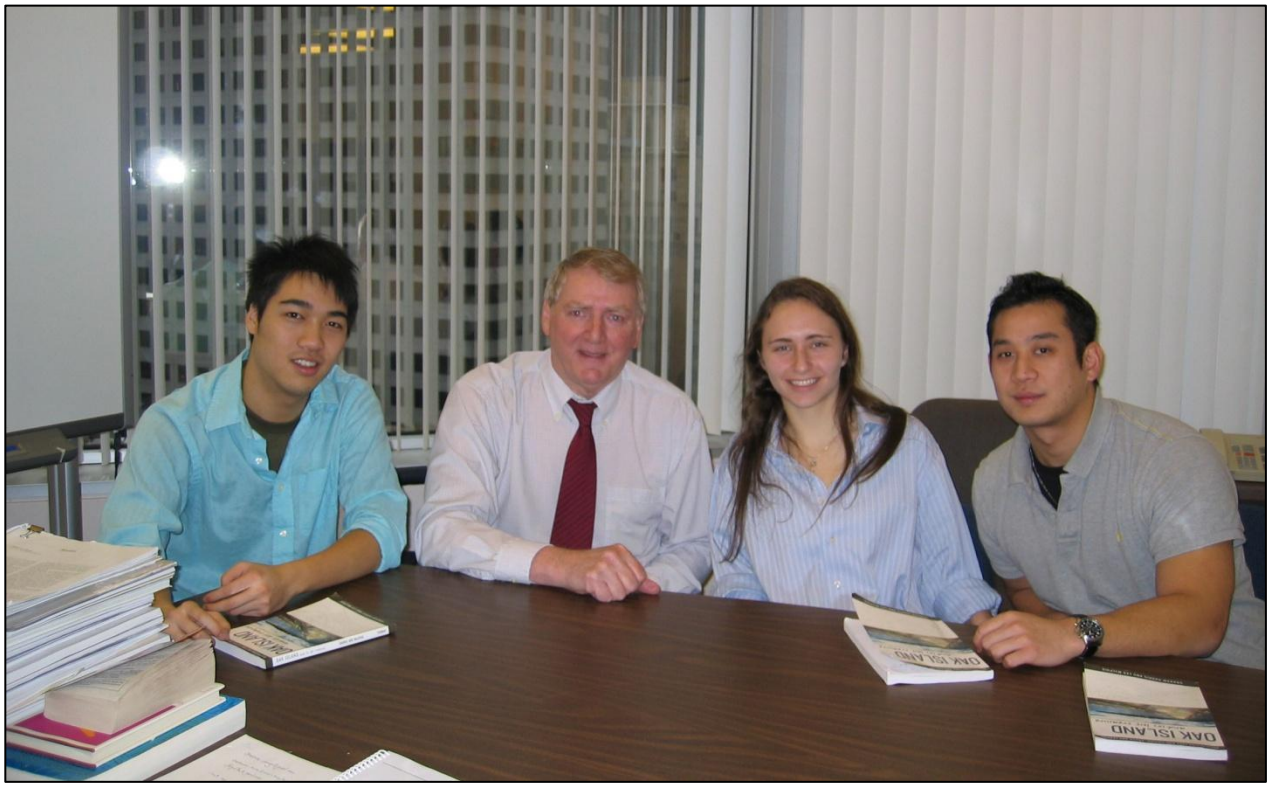


STL Group: L to R - Cheehan Leung, Shoshanna Saxe and Ryan Thé



STL Group with Advisor: L to R - Cheehan Leung, Les, Shoshanna Saxe and Ryan Thé

Topsoil Group: L to R – Christopher Ong Tong, Nathan Ramsey, Robert Wolofsky and Behnam Shayegan

Geovation Group: L to R: - Ming Jia (Peter) Wang, Rebecca Stanzeleit, Claire Meloche, Mariam Abdul Ghani and Nabil Nassor

DESIGN OF A DEEP EXCAVATION USING GROUNDFREEZING TECHNOLOGY

OAK ISLAND, NOVA SCOTIA

Technical Document Prepared for Mr. MacPhie, Prof. Rogers, and Prof. Taylor

PREPARED BY
STL GEOTECHNICAL GROUP
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Abstract

The mystery of the Money Pit has intrigued for more than 200 years. The underground workings, discovered in 1795, are believed to hold pirate treasure. Multiple generations of explorers have tried in vain to excavate to the bottom of the Money Pit. The man-made flood tunnel and the hydraulic connection through the broken anhydrite bedrock have served to flood all excavation attempts.

A 240 ft deep, 70 ft diameter, circular excavation using artificial ground freezing is designed. The frozen earth wall generated by artificial ground freezing serves as both structural support and a complete coffer dam for the excavation. A frozen earth wall ranging in thickness from 1.87 m to 4.60 m is designed to grow over 50 days. Calcium chloride brine at -25°C will be used as the freeze solution. The excavation is designed to be lined in 4 m advancements. The lining system is designed to resist all the external ground and groundwater pressures. It will have thickness ranging from 600mm at the surface to 1000mm at its base. The liner will be reinforced in two layers, in both horizontal and vertical directions to resist ovalisation due to non-uniform loading.

Acknowledgments

Thank you to Les MacPhie of SNC-Lavalin for his invaluable guidance and support throughout the preparation of this design project

Thank you to the following people for their time and guidance:

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- Georges Bibollet of SNC-Lavalin
- Alex Naughts of EcoGrout
- Professor M. Meguid
- Professor Taylor

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1.0 Introduction

The Oak Island mystery has infuriated and mystified treasure seekers for over 200 years. The complicated underground workings have combined with the strong hydraulic connection through the bedrock to prevent the successful excavation of the Money Pit. Artificial ground freezing technology has become more popular in the recent years and has proven to completely obstruct groundwater influx by forming a frozen earth wall, impermeable to water. This report includes the historical background as well as a summary of subsurface conditions from previous collected data followed by the design of the proposed frozen earth wall and lining of the shaft.

2.0 Historical Background

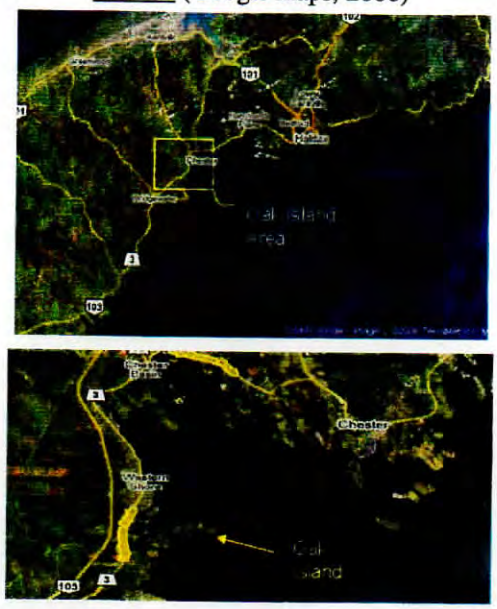
The coloured history of Oak Island and the Money Pit has been written about in many books and articles. A brief summary is given below. A more complete history is provided in "Oak Island and its Lost Treasure" by Graham Harris and Les MacPhie (2005) and "The Mystery of Oak Island" by D'Arcy O'Connor (2004).

2.1 Discovery

Oak Island is a small island located just off the coast of Nova Scotia near the town of Chester.

FIGURE 1: Oak Island

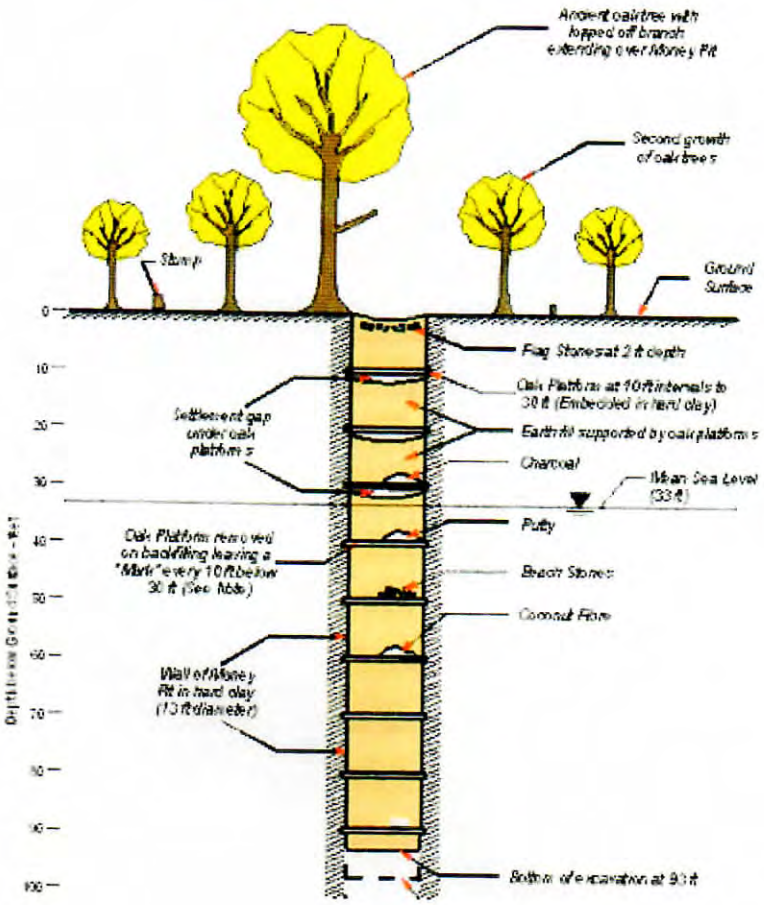
Source: (Google maps, 2006)



In 1795, a young boy named Daniel McGinnis was exploring the island and stumbled upon a depression in the ground. A tackle block hung from an old oak tree directly above the depression. He returned the next day with to friends, John Smith and Anthony Vaughan, to dig for what he suspected was pirate treasure. The boys worked with shovels to dig to a depth of 30 ft. Significantly, the boys found that they were re-excavating a previously dug ~13ft diameter pit and at 10 ft intervals they discovered oak platforms. During the history of the Money Pit, many outrageous claims have been made as to the origin and content of the treasure. These theories range from gold from the Knights Templar to the original works of Shakespeare. Most of these theories have little, if any, historical merit. The theory that appears to be the most likely is that the Money Pit was constructed for the purpose of hiding valuable treasure. (Harris and MacPhie 2005).

FIGURE 2: Cross section of original excavation
As discovered by the Onslow Syndicate

Source: (Harris and MacPhie, 2005)



2.2 Further Exploration

Since the initial discovery of the Money Pit by McGinnis in 1795 many different groups have tried to recover the treasure thought to be buried in its depths. Table 1 shows the chronology of treasure hunters on Oak Island.

TABLE 1: Chronology of treasure hunters on Oak Island

Year	Group
1795	McGinnis, Smith and Vaughan
1804-1805	Onslow Syndicate
1849-1850	Truro Syndicate
1861-1864	Oak Island Association
1866-1867	Halifax Company
1894-1900	Oak Island Treasure Co.
1909	Harry Bowdoin
1931-1934	W. Chappell and F. Blair
1936-1937	Gilbert Hedden
1938-1941	Edwin Hamilton
1955	George Greene
1960-1965	Robert Restall
1965-1966	Robert Dunfield
1966-1969	Tobias and Blankenship
1969-2005	Triton Alliance
2006	Michigan Group

Despite more that 200 years of work at the Money Pit no treasure has ever been recovered.

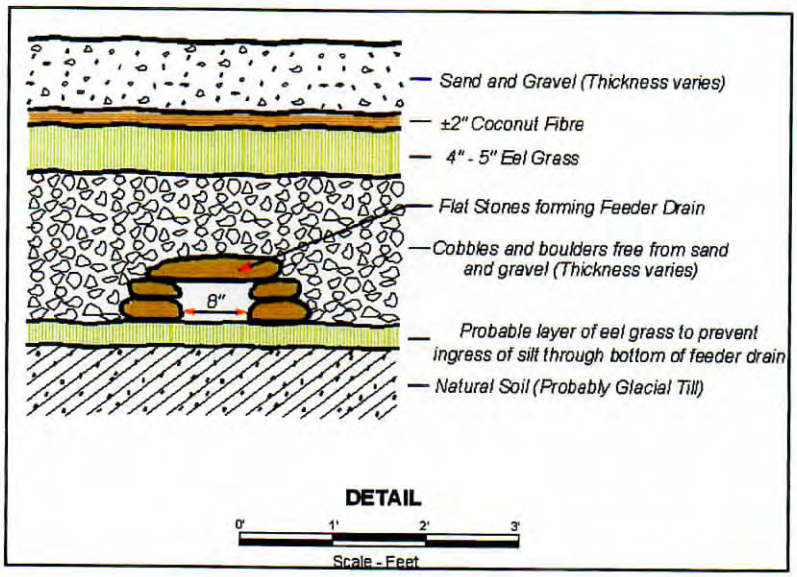
2.3 The Flood Tunnel

All excavations at the Money Pit have been flooded as they reached depths ranging from 90 to 114 ft. For this reason, the existence of a man-made flood tunnel connecting the Money Pit to the sea was suspected. In 1850, in an attempt to locate and destroy the flood tunnel, the Truro syndicate conducted a thorough search of the shore and discovered a filter system. The system consisted of 5 drains covered in layers of beach

rock, decayed eel grass and coconut husk. The drains sloped downward towards a presumed vertical shaft leading to the flood tunnel. This drainage system was the water source for the flood tunnel (Harris and MacPhie, 2005).

FIGURE 3: Configuration of filter drain system

Source: (Harris and MacPhie, 2005)



However, the location and existence of the flood tunnel was only confirmed in 1866, upon its discovery by the Halifax Company. The flood tunnel had dimensions of 2.5 ft wide by 4 ft high, and was filled with smooth beach stones. The flood tunnel connected with the Money Pit at a depth of 111 ft. The ingenious design of the flood tunnel only lent to the attraction of the Money Pit. Those responsible had not only dug through hard glacial till in cramped quarters but had done so without the aid of modern ventilation or lighting (Harris and MacPhie, 2005).

Over the years many attempts were made to destroy the flood tunnel with minimal success: clay was packed into the filter system, a cofferdam was constructed at Smith's cove to block the intake of the tunnel, piles were driven into the suspected location of the tunnel and eventually explosives were used by Oak Island Treasure Company in 1897. Although this work undoubtedly reduced the flow of water through the flood tunnel, it did not alleviate the flooding problems because of the hydraulic connection between the

Money Pit and the sea resulting from the porous nature of the underlying bedrock. This will be elaborated on in the forthcoming discussion of the hydrogeological conditions (Harris and MacPhie, 2005).

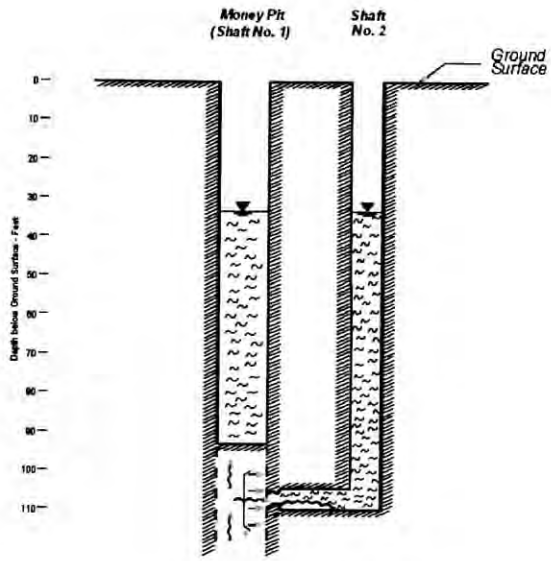
2.4 Previous Methods Used

Since the discovery of the Money Pit by Daniel McGinnis in 1795, many other groups have tried to recover the treasure. All attempts have failed. The following is a discussion of the methods used in attempts to excavate to the bottom of the Money Pit and why they failed.

After the abandonment of the Money Pit by the three boys in 1795 the site lay dormant until the formation of the Onslow Syndicate in 1804. The Onslow Syndicate successfully excavated the Money Pit to a depth of 93 ft. Their hopes of easily finding the buried treasure floundered as the excavation flooded to tide level. Once bailing the water proved unsuccessful they attempted to get at the treasure laterally. They successfully dug to a depth of 110 ft in an adjacent excavation. However, upon proceeding laterally, water broke through the shaft wall and again flooded to tide level. Many different groups have since attempted this method, re-excavate the money pit and after failing, attempt a lateral approach. All have met with failure.

FIGURE 4: Flooding of attempted lateral excavation

Source: (Harris and MacPhie, 2006)



Some groups tried to pump or bail the water out of the Money Pit, either by pumping directly from the Money Pit or by pumping from adjacent shafts. Pumping the Money Pit has had marginal success. In 1860 the Oak Island Association was able to bail to a depth of 118 ft in a shaft adjacent to the Money Pit before massive flooding stalled their efforts. Steam operated pumps were installed in attempts to dewater the excavations but have failed to keep up with the pace of the water influx. The most regrettable result of all the failed pumping has been an increase in the permeability of the broken anhydrite bedrock. As will be explained in section 3.2, anhydrite is highly water soluble and pumping at the Money Pit caused the bedrock to enter solution. For this reason any attempt to pump at the Money Pit has encountered an ever increasing rate of water influx. The more pumping used the greater the permeability of the rock, resulting in more water inflow, requiring more pumping capacity.

Perhaps the most destructive of all the previous attempts at recovering the Money Pit treasure was that of Robert Dunfield. In 1965, Dunfield built a causeway from the mainland to Oak Island in order to move heavy earth moving machinery to the island. He

then dug a 20 ft, 200 ft long trench on the south shore to stop a suspected flood tunnel. Dunfield proceeded to conduct open pit excavation in the Money Pit using a dragline excavator. The pit would eventually become 100 ft wide and 135 ft deep, stopping because heavy rain began to cause slope failures. Not only did the Dunfield excavation not recover any treasure from the Money Pit, it also succeeded in destroying various old shafts and disturbing the soil (Harris and MacPhie, 2005).

2.5 Clues that Point to the Treasure

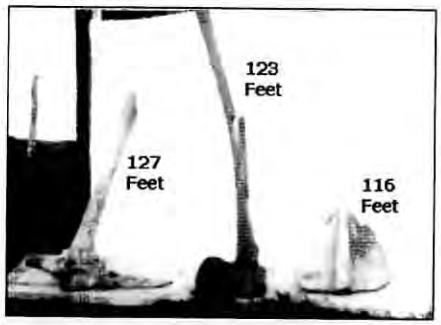
During the 200 + years of exploration and research into the Money Pit a number of important clues have been discovered that point to the existence of buried treasure on the island. Aside from the man-made features of the flood tunnel and the oak platforms in the money pit, there have been interesting recovered artifacts from excavation and drilling. There is also historical evidence that suggest the existence of valuable objects hidden in the vicinity of Oak Island.

During a drilling program conducted in 1897 by William Chappell and Frederick Blair encountered 20 inches of cement, followed by oak wood at a depth of 151 feet. This suggested a cement vault possible containing a wooden chest. Within the supposed chest were what felt like 4 inches of metal in bars and 20 inches of coin or small pieces of metal.

A small piece of parchment with the letter 'm' or 'vi' was recovered from a depth of 155 ft during the same drilling (Harris and MacPhie, 2005).

FIGURE 5: Parchment Recovered from Tip of Drill Bit (left)
FIGURE 6: Anchor Fluke, Pole Pick and Axe Recovered from Money Pit (right)
Source: (Harris and MacPhie, 2006)

Parchment is $^{5}/_{16}$ inches long

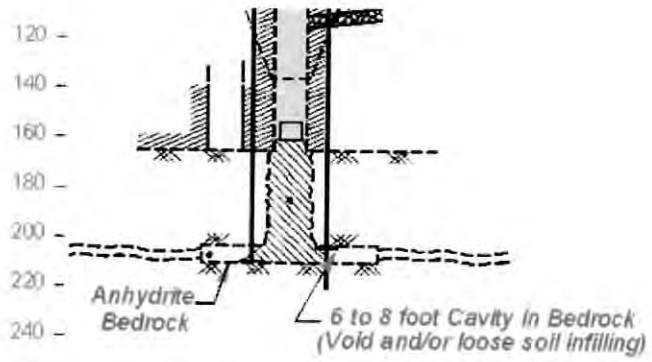


During an excavation by the same William Chappell in 1931, an anchor fluke, poll pick and axe were found at depths of 116, 123 and 127 ft respectively. Since no other searchers had previously reached such depths, it can be safely assumed that these artifacts belonged to the original excavators (Harris and MacPhie, 2005).

Deep drilling to a depth of 210 ft conducted in 1967 by David Tobias and Dan Blankenship found 6 to 8 ft high voids or chambers in the bedrock that were lined locally by wood and clay on the ceiling and ½ inch thick iron plate on the floor. These chambers were imbedded well into the anhydrite bedrock and were much deeper than any past excavation. These findings suggest the presence of several offset chambers originating from the bottom of the Money Pit and extending horizontally outwards. It is suspected that the treasure is cached in these chambers. The wood recovered from the ceiling of the chamber was dated to no later than 1690, indicating that the original excavators must have built these chambers.

FIGURE 7: Offset chambers in anhydrite bedrock

Source: (Harris and MacPhie, 2006)



Also significant is the wording of the Shoreham grant. The Shoreham grant, signed in 1759, gave Oak Island and surrounding areas to British settlers and colonists. The wording in the grant includes a phrase that reserves all gold, silver and lapis lazuli found on the land to the crown of Britain.

> "Do by these Presents give grant and Confirm onto the several Persons hereafter named Seventy Shares & a half of Two hundred Shares or Rights whereof the said Township is to consist with all and all manner of Mines unopened excepting Mines of Gold and Silver, precious Stones and Lapis Lazuli in & upon the said Shares or Rights"
> – Shoreham Grant (Harris and MacPhie, 2006)

Lapis lazuli is a precious stone found mainly in central Asia and China and had never been discovered in Nova Scotia and is not mentioned in other land grants. The fact that it is included in the wording of the grant suggests that senior British and Colonial authorities were aware of the presence of foreign precious stones in the area.

3.0 Site Conditions

3.1 Subsurface Conditions

A number of the groups that have conducted work on Oak Island have carried out subsurface investigations. The work has been done haphazardly and therefore much of the data is incomplete. The most recent and comprehensive geotechnical survey was conducted by Golder Associates in 1970 on behalf of Triton Alliance Limited. The subsurface at the Money Pit has also been subject to prospecting and work dating back over a hundred years. This digging, pumping, and shaft work has sometimes been improperly documented and left unrecorded.

All these reports have shown that the geology of Oak Island consists of a top layer of glacial till that varies from hard firm clayey silt to silt and sandy silt with boulders. These stratum properties occur to a depth of about 160 feet. Below this is about 50 feet of broken anhydrite which can be described as anhydrite bedrock with intermittent cracks and fissures filled with soil, water, or open cavities. Underneath the broken anhydrite is a layer of competent, solid anhydrite bedrock. (Golder Associates, 1971)

FIGURE 8: Permeability with depth

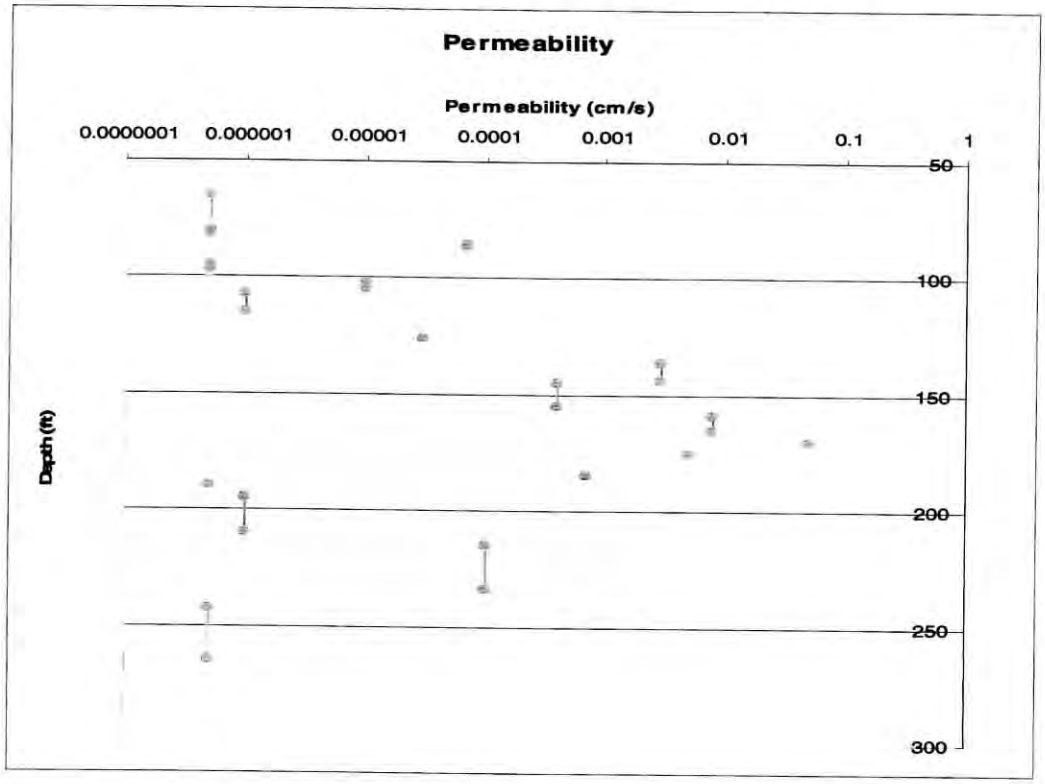


Figure 8 shows the depth in relation to permeability. The data was taken from borehole 102 (appendix A) on Oak Island, near the Money Pit. Figure 8 shows that there is a variable permeability in the glacial till layer, meaning possible seams of sand and sandy silt within a mostly clayey silt stratum. Below 160 ft, permeability begins to have constantly low values with some parts of higher permeability representing fractures or small seams.

FIGURE 9: Water content with depth

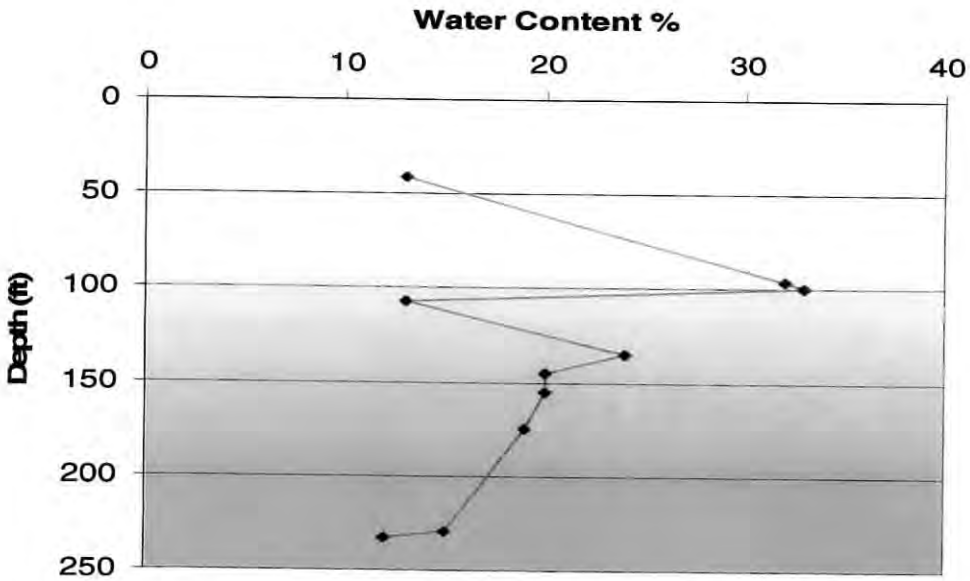


Figure 9 shows the soil water content in relation to the depth. The average water content in the glacial till stratum of Oak Island is 20% whereas the average water content of the broken anhydrite is 15%. Data for this chart was taken from borehole 102 (see appendix A).

3.2 Hydrogeological Conditions

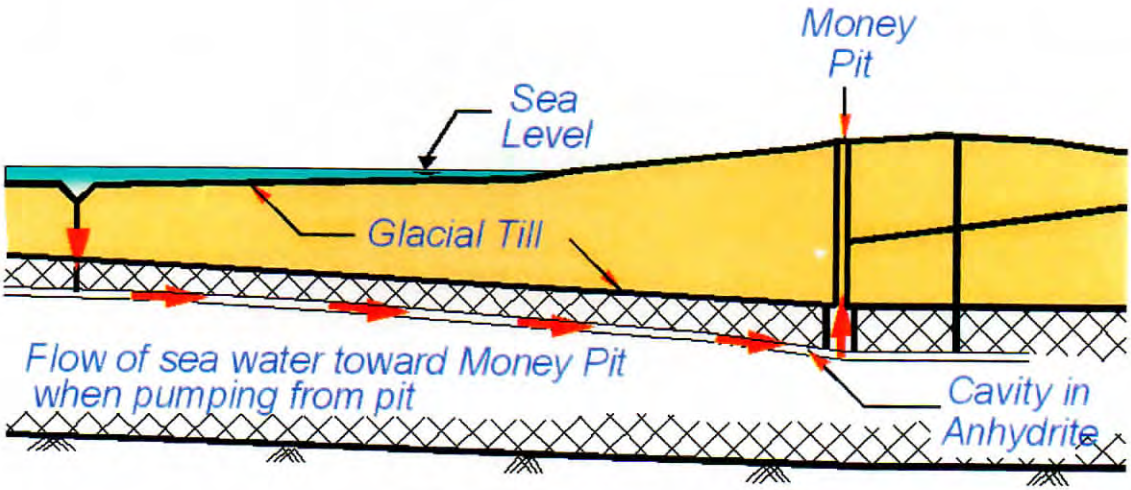
The hydrogeological conditions of the Money Pit and surrounding area can be determined largely from the hydrogeology and hydrography report completed by the Woods Hole Oceanographic Institute in 1996.

Anhydrite is highly soluble, especially in salt water. The seawater that permeates through the Money Pit and several other holes and shafts in the area is flowing in from the broken anhydrite bedrock layer. As mentioned in section 2.4, the previous attempts to pump water out of the Money Pit have only exacerbated the problem of seawater influx. This pumping not only removed the soil that filled the anhydrite voids, but also expanded previous cracks and fissures by dissolving surrounding bedrock. This has led to

a very strong hydraulic connection from the Money Pit to the surrounding sea. Figure 10 shows this hydraulic connection from the sea through the broken anhydrite layer to the Money Pit.

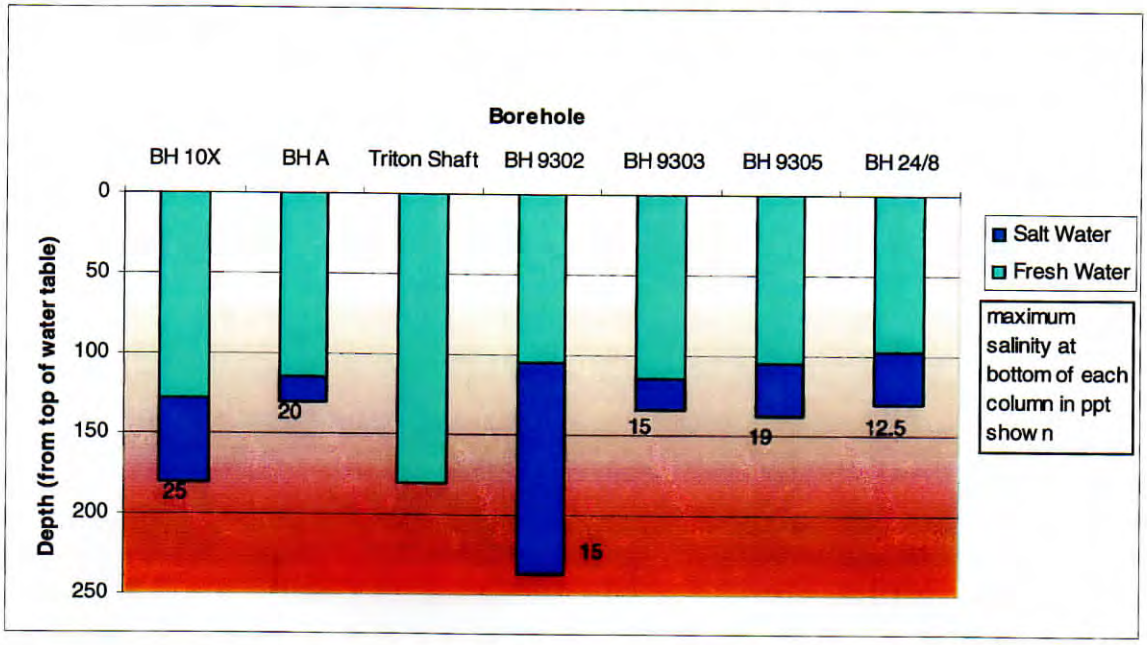
FIGURE 10: Hydraulic connection through broken anhydrite

Source: (Harris and MacPhie, 2005)



As can be seen from figure 11, a freshwater lens sits on top of the saltwater that pervades the island. There is variable water salinity within each borehole or shaft, values ranging from 0 ppt up to 25 ppt. In all boreholes the salinity increases with depth. Figure 11 shows the variable salinity taken at different depths in several boreholes. As shown, the fresh water lens is approximately 100 ft thick (Woods Hole Oceanographic Institution, 1996).

FIGURE 11: Borehole salinity



4.0 Project

4.1 Scope

The purpose of this design project is the design of a deep excavation (240ft) to investigate the possible burial of treasure at the Money Pit on Oak Island, Nova Scotia. The diameter of the excavation has been selected as 70 feet for this design project since this diameter should readily enclose the archaeological features identified by previous explorations. However, a smaller (or perhaps larger) diameter could be considered if further investigation identifies more specific information on the configuration of the archaeological features, particularly those at 200 feet depth.

4.2 Objectives

The excavation designed must be feasible, preserve the archaeological integrity of the site and allow for the future conversion of the excavation to a facility open to the public.

4.3 Challenges

The main obstacle to any excavation of the Money Pit is the hydraulic connection through the broken anhydrite to the sea. Excavations on Oak Island have been repeatedly flooded out for the past 200 years. The saltiness of the water has lead to salt contamination of the soil at the money pit.

4.4 Options Considered

Four different methods were explored as possible solutions. Each were analysed with respect to the previously laid out design objectives. These methods were in-situ grouting to form a water tight grout curtain, installing a sheet piles, well pumping and artificial ground freezing.

4.4.1 Grouting

Grouting is a process where grout, usually composed of clayey cement like material is injected through boreholes into the ground. The grout mushrooms out filling seams and cracks, stopping the influx of water. This technique has been successfully utilized in the past under dams to reduce uplift pressures. While grouting can be very effective at preventing the inflow of water, it requires precise advanced knowledge of the subsurface conditions in order implement effectively. Grout curtains many metres thick are often

required in order to reduce the water seepage to an acceptable value and never results in a completely impermeable barrier (Warner, 2004). Grouting was not selected due to the lack of precise subsurface data, the destruction that would be caused to subsurface artifacts by a multi-metre thick grout curtain, and the improbability of achieving the desire completely impervious barrier.

FIGURE 12: Grouting

Source: (USGS, 2006)

4.4.2 Sheet Piles

Sheet pile walls have been successfully used in groundwater control many times. The basics of this method involve driving a large sheet into the ground in order to stop or reduce ground or surface water flow. This is very reliable provided that the bottom of the sheet pile wall intersects with an impermeable stratum of ground. The drawback of pile walls at the Money Pit is that it is highly destructive to ground soil and may destroy any possible archaeological artifacts. Also, the sheet piles would have to be driven through the anhydrite bedrock, through into the competent bedrock which could prove extremely difficult (Fleming, 1985).

4.4.3 Well Pumping

Well pumping uses boreholes and pumps in order to lower the water table enough so that dry excavation is possible. In this case, a series of boreholes and pumps installed surrounding the Money Pit would potentially lower the water table until the shaft could be excavated and a shaft lining could be installed. This method is easy to install and monitor and can be effective in low permeability soil. (MacPhie, 2006) The conditions below the Money Pit are extremely resistant to pumping; water pumping would strengthen the hydraulic connection between the Money Pit and the sea. This project also requires a slow, careful excavation and prolonged well pumping would be very costly and difficult to maintain. A successful well pumping approach is extremely unlikely at the Money Pit due to the highly soluble anhydrite bedrock. Any pumping would result in increased permeability of the broken anhydrite which would in turn lead to greater inflow of water. The amount of pumping required would constantly increase, requiring ever greater capacity (MacPhie, 2006).

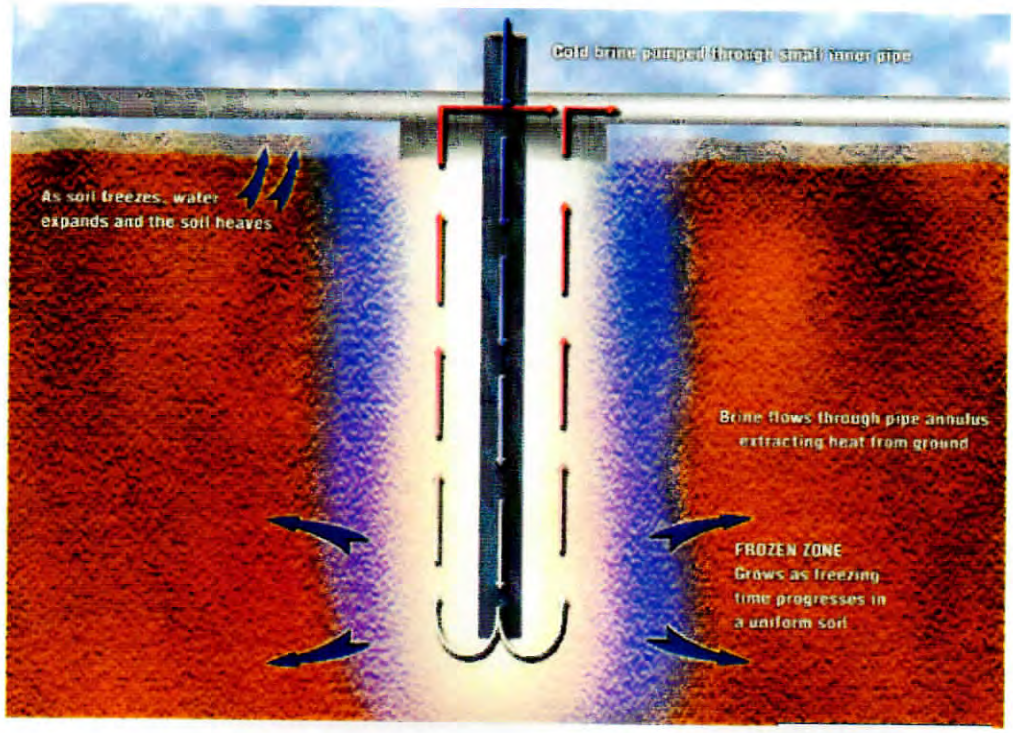
4.4.4 Artificial Ground Freezing

Artificial ground freezing has been selected for the proposed design of a deep excavation on Oak Island. (Do to the high permeability of the broken anhydrite bedrock and the inability of artificial ground freezing to operate successfully in the presence of subsurface water flow, which will be discussed in section 6.3.1; grouting will be used to reduce the permeability in the anhydrite layer.) Artificial ground freezing is a technique by which heat is extracted from the soil to turn the pore water to ice. The ice acts as a bonding agent, fusing together adjacent soil particles or blocks of rock, increasing their strength and creating an impermeable barrier (Andersland and Ladanyi, 2006).

In the construction of a shaft, a tube of soil around the intended excavation is frozen. The frozen soil acts as support for the excavation and a complete cofferdam to prevent inflow of water into the shaft during excavation.

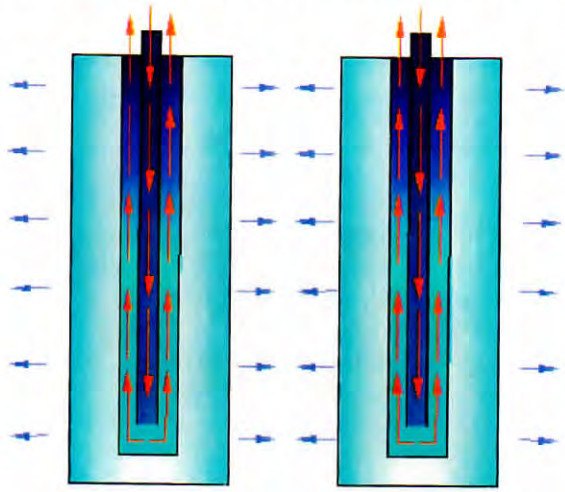
FIGURE 13: Ground freezing technique

Source: (Sopko, 2006)



Freeze tubes are installed into the ground in a ring a round the excavation. One freeze tube consists of two pipes: (1) a smaller diameter internal pipe and (2) a larger diameter external pipe. A cold solution, in this case calcium chloride brine, is pumped down the small diameter pipe and returns up the external pipe. As the cold brine travels the length of the tube it extracts heat from the surrounding soils freezing it. As shown in figure 14, over time, weeks or months, the ring of frozen soil around each freeze tube grows and merges into the frozen soil surrounding adjacent pipes forming a continuous wall (Sopko, 2006).

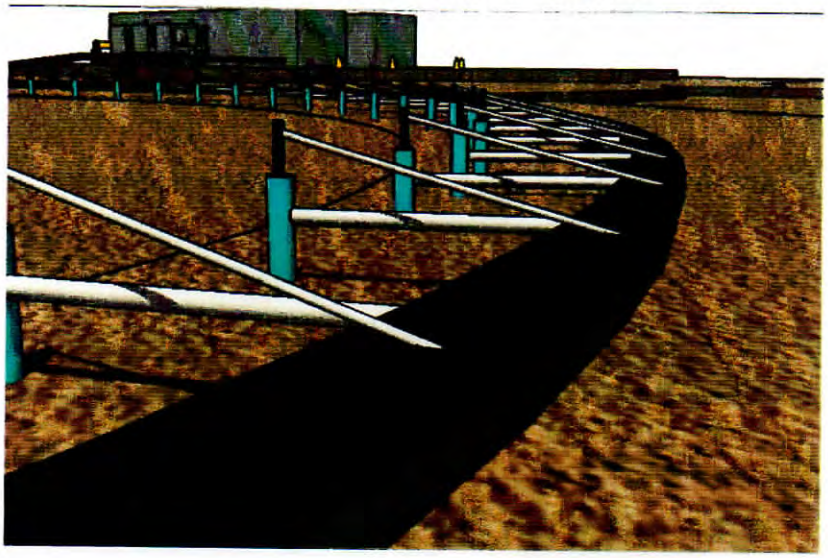
FIGURE 14: Growth of frozen earth wall



Once the frozen earth wall attains the required thickness, excavation can commence (Andersland and Ladanyi, 2004). Ground freezing has been successfully used in mining and civil engineering applications for over 100 years at depths of up to 975 m. The final strength and formation time of the frozen earth wall is effected by the presence of contaminants in the soil, such as salt and subsurface flow. The presence of contaminants depresses the freezing point of water and retards the freezing process. Any subterranean flow has significant negative effects of the development of a frozen earth wall. With a brine system the flow cannot exceed 1-2 m/day. If there is too much flow the frozen earth wall will not develop properly. When constructing in areas with high water flow, grouting must be used to reduce the soil/rock permeability and therefore reduce the flow rate or, a more aggressive ground freezing system, such as cryogenic freezing, must be used. A brine system can be used down to a temperature -40˚C, however the optimal temperature for a brine system is usually -25˚C (Harris, 1995). The brine is cooled in an onsite refrigerant plant from which. The brine is distributed in a series-parallel configuration, with each freeze tube individually connected to the manifold (see Figure 15). The cooled brine travels from the refrigerant plant through the distribution network and once used returns to be re-cooled (Andersland and Ladanyi, 2004).

FIGURE 15: Refrigerant system supply and return manifold

Refrigerant Plant can be seen in background



Artificial ground freezing was selected for this project because it will preserve the historical and archaeological integrity of this site, develop a complete cofferdam preventing inflow of water into the excavation and allow for latter adjustment of the excavation through lateral ground freezing if desired.

For more information on artificial ground freezing consult "Ground Freezing in Practice" by Harris, 1995 and "Frozen Ground Engineering" by Andersland and Ladanyi, 2004.

5.0 External Pressures

The external pressures that will be exerted on the frozen earth wall were calculated under both short and long-term loading.

5.1 Short Term Loading

The short term loading was calculated using Peck's Envelope for clayey soils (Craig, 2004).

$$\begin{aligned} &p_e = 0 \text{ kpa at ground surface} \\ &p_e \rightarrow \text{increases linearly to } 0.25h \\ &p_e = 0.2\gamma h \rightarrow \text{from } 0.25h - 0.50h \\ &p_e \rightarrow \text{decreases linearly to } 0 \text{ kpa at full depth, } h \end{aligned} \qquad \textbf{(5.1-1)}$$

5.2 Long-Term Loading

$$p_g = k\gamma h - 2Cu\sqrt{k} \qquad \textbf{(5.2-1)}$$

$$p_w = \gamma_w h \qquad \textbf{(5.2-2)}$$

$$p_e = p_w + p_g \qquad \textbf{(5.2-3)}$$

In a till matrix pressures due to cohesion are small (Meguid, 2006). For this reason and to be conservative, the cohesion term in the external pressure equation will be ignored.

5.3 External Pressure Envelope

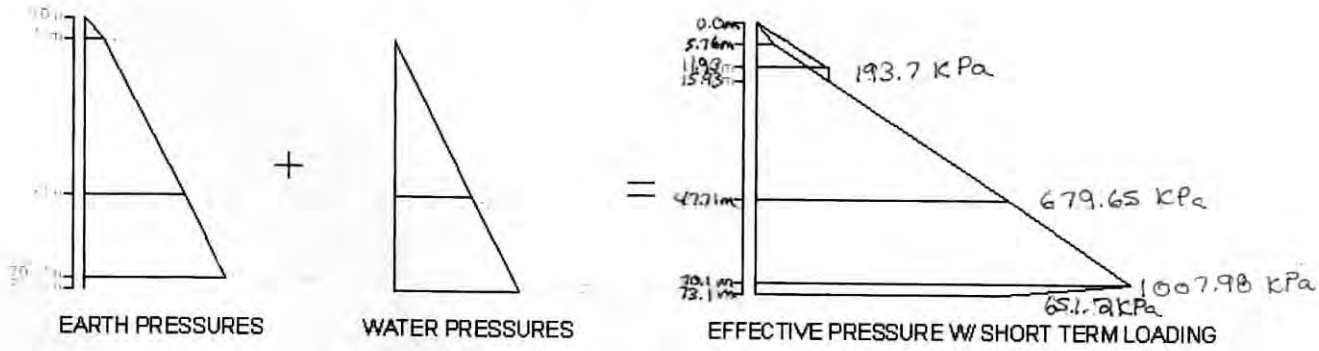
In order to be conservative the larger of the short and long-term loading with depth was used. Short-term loading governs to a depth of 15.43 m after which the long-term loading is consistently larger. Table 2, shows the properties used to calculate the external loads.

TABLE 2: Loading properties of the soil stratum

Stratum	1: Dry till	2: Saturated till	3: Broken Anhydrite	4: Competent Anhydrite	Water
h (m)	5.76	41.95	22.40	3.05	67.4
γ (kN/m^3)	16.70	20.3-9.81=10.49	29.2-9.81=19.39	29.2-9.81=19.39	9.81
k [1]	0.50	0.50	0.25	0	n/a

[1] (MacPhie, 2006)

FIGURE 16: Earth pressure diagram



The external pressures at the base of the till (679.65 kPa) and at the base of the broken anhydrite (1007.98 kPa), the largest loads in each stratum, are the critical points for evaluating the necessary frozen earth wall thickness.

6.0 Design of Frozen Earth Wall

6.1 Site Characteristics

Due to the inability to physically conduct a site investigation and carry out tests in the labs, various texts combined with the subsurface data available from previous investigations at Oak Island, were used to estimate the soil and rock properties shown in Table 3 below. It was necessary to obtain the following properties since it is impossible to perform an artificial ground freezing design without them.

TABLE 3: Soil and rock Properties

Properties	Symbol	Stiff Glacial Till	Anhydrite
Dry Unit Weight (kN/m^3)	γ	16.7[1]	29.2 [2]
Saturated Unit Weight (kN/m^3)	γ_{sat}	20.3 [1]	29.2 [2]
Long term Cohesion (kPa)	c	2000 [5]	5376 [6]
Angle of friction	Φ	20° [4]	0° [5]
Frozen Angle of friction	Φ	0° [5]	0° [5]
Density (kg/m^3)	p		3000 [2]
Water Content	w	20 %[3]	15 %[3]
Coefficient of permeability (cm/sec)	k	10^{-6} [7]	
Frozen Thermal Conductivity (kJ/day.m.°C)	K_1	164.4 [7]	388.8 [8]
Unfrozen Thermal Conductivity (kJ/day.m.°C)	K_2	194.3 [7]	388.8 [8]
Frozen Specific Heat (kJ/m^3.°C)	C_1	1972 [7]	569 [8]
Unfrozen Specific Heat (kJ/m^3.°C)	C_2	2885 [7]	569 [8]
Latent Heat of Fusion (MJ/m^3.°C)	L	208 [7]	150.2 [9]

[1] (Terzaghi et al, 1996)

[2] (Mindat, 2006)

[3] (Golder, 1971)

[4] (Puller, 1996)

[5] (Holubec, 2006)

[6] (CEG, 2006) Unconfined compressive strength of Anhydrite= 120 -1 000 ksf = 11,520 – 96,000 kpa

Taking average value = $\frac{11520+96000}{2}$ = 53760 kpa

Assuming 10% strength due to fractures = 53760(0.10) = 5376 kpa

[7] (LCC, 1997)

[8] (Andersland and Ladanyi, 2004)

[9] From (8) L=ρwL' where L' = latent heat of fusion for water L_{anh}=300*0.15*333.7=150165 kJ/m^3.°C

Note (1): The cohesion values are taken as very conservative and are assumed to have incorporated safety factors of greater than 2. 2000 kPa was taken as the long term cohesion for the till, according to Derek Maishman, a ground freezing engineering expert, a value of up to 10,000 kPa. Similarly, taking 10% strength for the broken anhydrite is very conservative. Due to the lack of detailed geotechnical investigation at the Money Pit it was important to carry out a very conservative design to account for the degree of uncertainty produced by said lack of data.

6.2 Structural Design

The excavation will be lined at 4 m advancements of excavation. Therefore, the frozen soil will never be exposed for long periods of time. For this reason the calculation of the required frozen earth wall thickness was based on elastic design and it was not necessary to perform a creep based analysis.

From Andersland and Ladanyi the following formula for the required ratio of internal radius to external radius of the frozen earth wall obtained.

$$\frac{b}{a} = \exp\left[\frac{p_e}{2c}\right] \qquad \textbf{(6.2-1)}$$

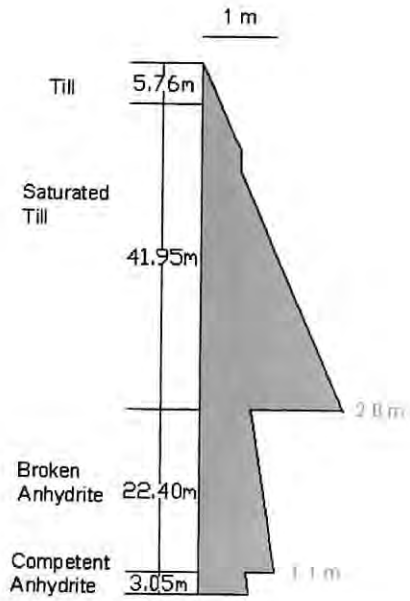
Where:

a = the internal radius (m)
b = the external radius (m)
c = long term cohesion (kPa)
p = the external pressure (kPa)

The required thickness, W, of the frozen earth wall is
$W = b\text{-}a$

6.2.1 Required Thickness

FIGURE 17: Required frozen earth wall thickness with depth



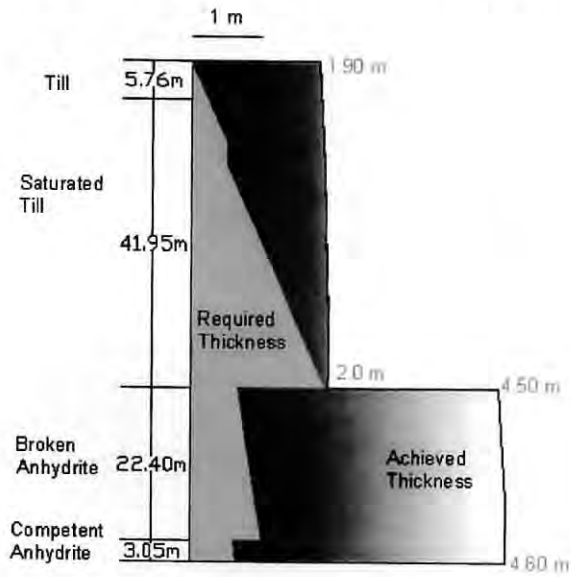
The required frozen earth wall thickness in each stratum was calculated based on the external pressures found in section 5.3 and the strength properties of the stratum. Detailed calculations can be found in appendix B. As can be seen from figure 17 the 2.0 m thickness required at the base of the till stratum is the governing thickness. Even though the loads are larger in the broken anhydrite stratum, the strength of the broken rock is such that the required thickness is only 1.1 m. However, due to difficulties is installing freeze tubes exactly vertical, a minimum desired thickness of 2 m will be used in both layers (Maishman, 2006). It is important for verticality to be maintained during the installation of the freeze tubes so that there is not excessive deviation for the design spacing, *S*. Modern drilling techniques that ensure borehole verticality should be used when installing the freeze tubes,

It is important to note that it is not necessary to apply an additional factor of safety to the calculated required frozen earth wall thickness because it was already accounted for in the determination of the strength properties of each stratum.

6.2.2 Achieved Thickness

As will be discussed in the thermal design below, a 50 days freeze time in required to generate the necessary 2.0 m frozen earth wall thickness in the till stratum. The thickness achieved at 50 days with depth is shown in figure 18.

FIGURE 18: Achieved frozen earth wall thickness



Due to the different thermal properties of the two stratums, the frozen earth wall develops more than twice as fast in the broken anhydrite as in the till. This results in a thickness in the anhydrite which is much larger than the 2.0 m minimum. This extra thickness is not a problem and in fact serves as an advantage; preventing water flow through the anhydrite is the primary requirement of this design, the extra frozen thickness in the anhydrite serves this goal. As can be seen from figure 18, the achieved thickness larger than the required thickness at all points.

6.3 Thermal Design

The time required for the establishment of a frozen earth wall, assuming static hydrostatic conditions, is determined by 3 primary factors (LCC, 2006):

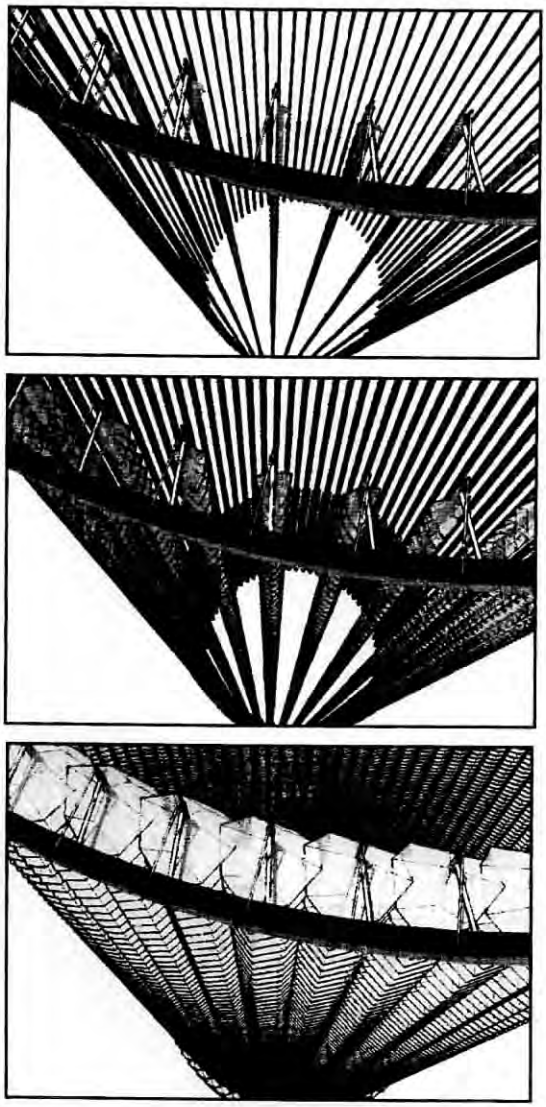
- Thermal properties of the soil
- Spacing of the freeze tubes
- Temperature of the freeze solution

The soil thermal properties are intrinsic to the work site. A freeze tube spacing of 1 m is standard for construction of shafts (Sopko, 2006). Therefore, variability in the thermal design is governed by the applied temperature of the freeze solution.

Bellow is a calculation of the time required to develop the necessary frozen earth wall thickness at the Money Pit.

The total time required for freezing, t_t, is made up of two components: (1) t_I, the time for the ice columns surrounding each individual freeze tube to merge and (2) t_{II}, the time required for the now merged columns to grow as a wall to the required thickness. The frozen earth wall growth in the till strata is governed by the thermal properties of the till. Similarly, the frozen earth wall growth in the anhydrite is governed by the thermal properties of the anhydrite.

FIGURE 19: Growth of frozen earth wall over time



6.3.1 Other Factors Affecting Freezing

The rate of freezing is negatively affected by three factors:

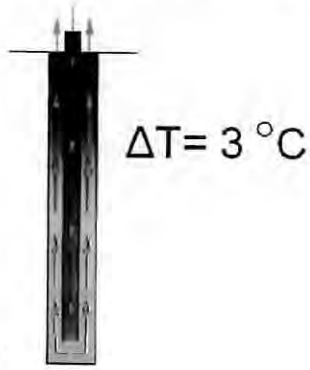
- Soil contaminants
- Warm air in contact with the soil at the ground surface
- The temperature gradient in the freeze solution
- Subsurface flow

Ground pore water at the Money Pit has significant salt contamination. This depresses the freezing point of the soil from 0˚C to -2˚C, retarding the freezing process. This retardation was accounted for in the thermal design, above.

If artificial ground freezing in performed in the summer, the warm air can prevent freezing at shallow depths, preventing the frozen earth wall from forming near the surface. In order to prevent this problem from developing the ground surface is covered with 4 inches of polyurethane foam that acts as an insulator. If artificial ground freezing is performed during the cold winter months this is not a concern (Sopko, 2006).

As the freeze solution travels up through the freeze tube it gets warmer. Meaning, the freeze solution is coldest at the bottom of the freeze tube. The rate of growth of the frozen earth wall will reflect this temperature gradient across the freeze tube. As shown in figure 20, at the beginning of freezing a temperature gradient of 3 ˚C is typical. As the freeze process continues the gradient will reduce to 1-1.5 ˚C (Maishman, 2006). During the frozen earth wall growth the necessary refrigerant capacity is largest, after the completion of the growth of the wall the refrigerant capacity is significantly reduced to maintain the frozen wall (LCC, 2006). To be conservative, a temperature gradient of 3˚C was used in design.

FIGURE 20: Freeze solution temperature gradient



A freeze solution temperature of -25˚C is readily achievable with a calcium chloride freeze solution and is used in this design. Due to the warming of the freeze solution by

the time it reaches the base of the till stratum it will have warmed to -24°C. The temperature of the calcium chloride solution will range from -25°C to -24°C in the anhydrite and from -24°C to -22°C in the till.

Subsurface flow retards the freezing process and can lead to windows in the frozen earth wall by carrying extra heat into the stratum. For this reason subsurface flow must be minimized during freezing. This problem will be further explored in section 6.4.

6.3.2 t_I: Time for wall to close

$$t_I = \frac{(S/2)^2}{4K_1V_s}\left[L_I(2\ln R' - 1) + C_1V_s\right] \qquad \textbf{(6.3-1)}$$

Where:

t_I = time for ice cylinders to meet

K_I= frozen thermal conductivity (kJ/day.m.°C)

$$L_I = L + C_1V_o + 3C_2V_o \qquad \textbf{(6.3-2)}$$

Where:

L = Latent heat of fusion (MJ/m^3.°C)

C_1 = Unfrozen specific heat (kJ/m^3.°C)

C_2 = frozen specific heat (kJ/m^3.°C)

V_s = difference between the freeze pipe surface temperature and the freezing point of water (°C)

V_o = difference between the ambient ground temperature and the freezing point of water (°C)

$$R' = \frac{S/2}{r_o} \qquad \textbf{(6.3-3)}$$

Where:

r_o= freeze pipe radius (m)

S = freeze pipe spacing (m)

6.3.3 t_{II}: Time for thickness to develop

In the time it takes for the ice cylinders surrounding adjoining freeze pipes to merge laterally, the columns will have attained a radial thickness of 0.79S. The time required for the necessary thickness to develop, t_{II}, accounts for the growth time from $W = 0.79S$ to $W = W_{reqd}$.

$$t_{II} = \frac{L_{II} S^2}{8 K_1 V_s}\left(x^2 - 0.62\right) \qquad \textbf{(6.3-4)}$$

Where:

t_{II} = time for ice thickness to develop

S = freeze pipe spacing (m)

K_1= frozen thermal conductivity (kJ/day.m.°C)

V_s= difference between the freeze pipe surface temperature and the freezing point of water (°C)

$$x = \frac{W}{S} \qquad \textbf{(6.3-5)}$$

Where:

W = wall thickness (m)

$$L_{II} = L + \frac{1}{2} C_1 V_s \qquad \textbf{(6.3-6)}$$

Where:

$L =$ Latent heat of fusion (MJ/m^3.°C)

$C_1 =$ Unfrozen specific heat (kJ/m^3.°C)

(Source: Andersland and Ladanyi, 2004)

6.3.4 Freeze Time in Till

For detailed calculations of the required freeze time see appendix B.

The time required to develop a 2 m thick frozen earth wall at the base of the till layer at varying brine temperatures is shown in table 4, below:

TABLE 4: Time required for growth of frozen earth wall in till

T_{pipe} (°C)	t_I (days)	t_{II} (days)	t_t (days)
-30	18.2	21.6	40
-25	22.0	25.8	48
-24	22.9	26.8	50
-23	24.0	28.0	52
-22	25.1	29.3	54
-21	26.4	30.7	57
-20	27.9	32.2	60
-19	29.5	34.0	63
-18	31.2	35.9	67
-17	33.3	38.2	71
-16	35.6	40.7	76
-15	38.3	43.7	82

At -24˚C, a 50 day freeze time is required for the formation of a 2.0 m thickness at the base of the till stratum.

6.3.5 Freeze Time in Anhydrite

The time required to develop a 2.0 m thick frozen earth wall at the base of the anhydrite layer at varying temperatures is shown in table 5, below:

TABLE 5: Time required for growth of frozen earth wall in anhydrite

T_{pipe} (˚C)	t_I (days)	t_{II} (days)	t_t (days)
-30	4.2	6.1	10
-25	5.1	7.4	12
-24	5.3	7.7	13
-23	5.6	8.1	14
-22	5.8	8.5	14
-21	6.1	8.9	15
-20	6.5	9.4	16
-19	6.8	9.9	17
-18	7.3	10.5	18
-17	7.7	11.2	19
-20	8.3	12.0	20
-15	8.9	12.9	22

At the base of the broken anhydrite the sodium chloride solution is -25˚C and requires 12 days to generate the 2.0 m minimum thickness.

As can be clearly seen by a comparison of table 4 and 5 the freezing in the till layer will control the design. The results found here are in keeping with past ground freezing projects that have shown frozen earth walls develop much faster in rock than in clay or silt stratum (Sopko, 2006).

6.3.6 Freezing Results

TABLE 6: Wall thickness @ 50 days in till

T_{pipe} (°C)	t_t (days)	W (m)
-24	50	2.01
-23	50	1.94
-22	50	1.87

TABLE 7: Wall thickness @ 50 days in anhydrite

T_{pipe} (°C)	t_t (days)	W (m)
-25	50	4.60
-24	50	4.49

As previously discussed in section 6.2.2 above, freezing will take place more than twice as fast in the anhydrite layer. Over the required 50 day freeze time a frozen earth wall thickness ranging from 1.87-2.01 m will develop in the till and from 4.49-4.60 m will develop in the anhydrite. The range is due to the warming of the calcium chloride as is travels up the freeze tube. The calcium chloride solution will be warmer at the top of each stratum than it is at the bottom; since the external pressures are smaller at the top of each stratum than at the bottom this does not pose a problem.

6.4 Ground water flow

As mentioned in section 6.3.1, the presence of subsurface water flow has a negative effect on freezing and can lead to windows developing in the frozen earth wall. Uncontrolled flow has resulted in the failure of otherwise well design artificial ground freezing projects (Maishman, 2006). The groundwater velocity must be controlled during

the freezing process and the excavation that follows. For reasons mentioned above, no groundwater pumping should be performed in the vicinity of the Money Pit during while ground freezing is being employed.

The maximum allowed ground water velocity that can be present during ground freezing is given by:

$$u_c = \frac{k_f}{4S\ln\left(\frac{S}{4r_o}\right)}\frac{V_s}{V_o} \quad \text{(m/day)} \qquad \textbf{(6.4-1)}$$

Source: (Andersland and Ladanyi. 2004)

Where:

u_c = critical ground water velocity

r_o= freeze pipe radius (m)

k_f= frozen thermal conductivity (W/m·˚C)

S = freeze pipe spacing (m)

V_s= difference between the freeze pipe surface temperature and the freezing point of water (˚C)

V_o= difference between the ambient ground temperature and the freezing point of water (˚C)

Table 8, shows the critical velocity for varying brine temperatures.

TABLE 8: Critical flow velocity

T_{pipe} (°C)	u_c (m/day)
-30	1.76
-25	1.45
-24	1.38
-23	1.32
-22	1.26
-21	1.19
-20	1.13
-19	1.07
-18	1.01
-17	0.94
-16	0.88
-15	0.82

The warmest temperature reached by the calcium chloride brine is -24°C in the anhydrite stratum, resulting in maximum allowed velocity, u_c = 1.38 m/day. Due to the strong hydraulic connection through the broken anhydrite and the ocean and the resulting tidal response, the groundwater velocity at the Money Pit is assumed to be greater than 1.38 m/day. An exact value of groundwater flow at the site is not currently available; groundwater velocity testing has not yet been performed at Oak Island. In order to reduce the flow to below the critical value, grouting must be used in the anhydrite stratum to reduce the permeability.

6.5 Grouting in broken anhydrite

Ground freezing is ineffective in open voids or voids filled with water. For this reason, any large voids encountered in the broken anhydrite must be grouted before ground freezing can proceed. Artificial ground freezing is very sensitive to subterranean flow. Therefore, grouting must also be used in the anhydrite layer to decrease the permeability of the stratum and reduce the flow rate to below the critical value.

In order to grout under the salty conditions found at the Money Pit a special admixture of cement, fly ash, biopolymer, saltwater, starch and super plasticizer must be used. The development of these special grouts is relatively new and their design is proprietary information. The necessary grout is available from ECO Grouting Specialists Ltd. out of Grand Valley, Ontario (Naughts, 2006).

During drilling for freeze tube installation, upon the discovery of large voids, or excess permeability in the broken anhydrite, indicated by loss of drill water, grouting will be performed. After 3-5 days, the time required for the grout to set, drilling can continue. It is important to note that grouting will not have an appreciable effect of the artificial ground freezing process.

6.6 Monitoring during freezing

During freezing the temperature gradient of the freeze solution, and frozen earth wall thickness need to be monitored on a regular basis. As well, the continuity of the frozen earth wall must be insured before excavation can take place.

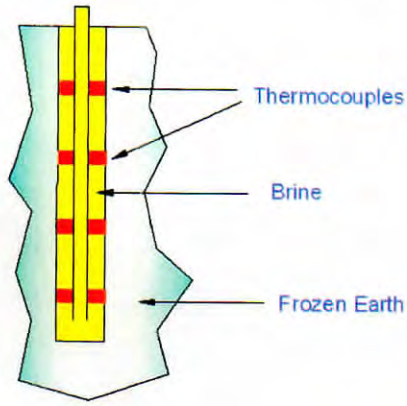
6.6.1 Monitoring the Temperature of the Freeze Solution

The temperature of the calcium chloride brine must be checked as it enters the freeze tube and again as it exits. This can be done using thermal couples. It is important to measure the temperature gradient of the calcium chloride brine to ensure that it is extracting heat as expected and that the design was based on valid assumption. A large deviation in the gradient indicates a problem in the freezing process that requires further investigation. Small deviations are allowable but will effect the calculated time required for freezing, t_f (Harris, 1995).

6.6.2 Monitoring Frozen earth wall thickness

The development of the frozen earth wall can be monitored by ground temperature measurement. During the installation of the freeze tubes, several temperature monitoring pipes are installed. The monitoring pipes are not connected to the distribution manifold but are filled with calcium chloride brine to prevent freezing. Thermocouple sensors floated inside the pipe at specified depths to correspond changes in stratum. The temperature sensor measurements are used during the freeze process to confirm the growth of the frozen earth wall (LCC, 2006).

FIGURE 21: Monitoring frozen earth wall development



6.6.3 Checking for continuity of the frozen earth wall

As the frozen earth wall forms and grows the pressure on the unfrozen internal soil increases. This causes the internal pore water pressure to increase. In order to prevent a catastrophic build up of pressure, a pressure relief hole is installed at the centre of the intended excavation through which water is expelled to the surface. The pressure relief hole is constructed of perforated tubing installed along the depth of the excavation. Along with providing pressure relief, it also acts as an indicator of frozen earth wall continuity. Without continuity the water pressures will not build as water escapes out through windows in the ice column and no water would be expelled up the relief hole. In addition, a pump test must be carried out once the frozen earth wall is complete.

Pumping is carried out in and outside the frozen earth wall in turn and the hydraulic response on the other side is measured. With a continuous wall there should be no appreciable response induced by the pumping (Harris, 1995).

7.0 Lining Design

The purpose of the lining system is to structurally support all earth and groundwater pressures of the surrounding earth acting on the shaft. As well as, stopping the groundwater inflow to allow archeological teams to investigate the site. It has been decided that the scope of this project does not include a secondary lining (final lining) as once the shaft has been excavated, the implementation of a final lining will be decided by the investigatory results as they will indicate the future uses of the shaft. Therefore, a single-layer lining system will be designed in which the initial ground support will function as the permanent ground support system, which is not uncommon in practice. This section includes all considerations and analysis that have been used to design the lining system.

7.1 Codes and Standards

Currently there are no codes implemented in the full design of a shaft lining system but many methods and standards have been taken from the National Building Code of Canada (NBCC) 1995, Canadian Standards Association (CSA) A23.3-04 as well as the US Army Corps of Engineers EM 1110-2-2901 and have been noted when used.

7.2 Lining Process

To provide the safest and most reliable method of primary lining, placement will occur at every 4m advancements of excavation (Sopko, 2002). Although, with the frozen earth

barrier, it is possible to fully excavate to the bottom and then line, there a number of advantages to line while excavating:

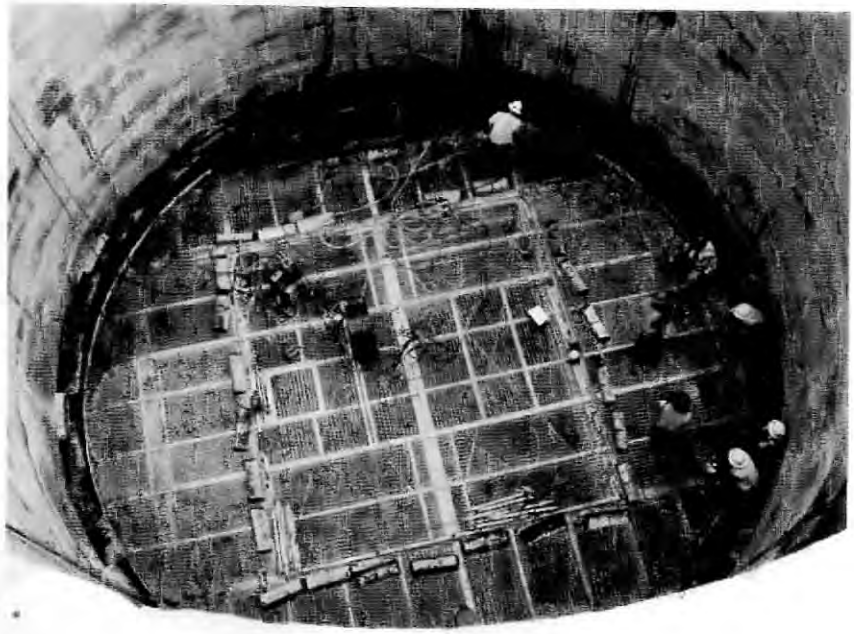
- Insulation is not required if the placement of the primary lining is completed in short time intervals
- Creep deformation will not begin due to the short time period
- If the frozen earth wall fails resulting in flooding, minimal damage will occur in the lined sections

7.3 Lining Material

Selection of the material is a very important step in designing the lining system. Each type of material has its own specific resistance against different loading cases.

FIGURE 22: Installation of concrete shaft lining

Source: (Clean Water, 2006)



7.3.1 Material Types

7.3.1.1 Steel Lining

Steel lining is used when the internal pressures exceed the external ground and groundwater pressures. Steel lining is used to prevent hydro-jacking of the rock (EM 1110-2-2901). If the internal pressures do not exceed the external ones, buckling will occur in steel lining and result in failure (Bibollet, 2006).

7.3.1.2 Un-reinforced Concrete

Un-reinforced concrete is only use if the shaft is not subjected to any internal pressures. This type of lining is usually used if the rock and earth are in equilibrium before installation of the lining. This lining type cannot be subjected to significant soil overburden and/or badly squeezing rock. This lining cannot be used because of the discontinuities and highly disturbed characteristics of soil and rock in the subsurface (EM 1110-2-2901).

7.3.1.3 Reinforced Concrete

Reinforced concrete linings allow resistance to internal pressures or non uniform loading. The magnitude of resistance is dependant on the number of layers of reinforcement but typically one layer of reinforcement is used to resist any unanticipated non uniform loading on the shaft (Bibollet, 2006).

Reinforced concrete is selected due to the anticipated yet minimal non-uniform loading due to discontinuities of the subsurface conditions. Reinforced concrete segments are

standard for a single-layer system regardless of the subsurface conditions. (Bibollet, 2006)

7.3.2 Steel Reinforcement

Reinforcement in the concrete lining will resist any applied tension due moments created by non-uniform loading. CSA A23.3-04 has been used to calculate the properties of the reinforced concrete. A summary of the reinforcement properties are presented below but complete design calculations are in Appendix C

- 20M bars yield an acceptable A_s (7.8.10) with 200mm c/c spacing (N7.8.2, N10.6.2)
- 75mm cover (Bibollet, 2006)
- Reinforcement must be closed with horizontal U-bars at same spacing and diameter as the principal bars

7.3.3 Water & Sulfate Resistance

The lining must resist sulfate damage from the surrounding ground and groundwater. The concrete type must also be of low permeability to retard groundwater from penetrating through the lining.

High sulfate concentrations in groundwater (> 600 mg SO4/L) and in soil (> 3000 mg SO4/kg) require the use of sulfate resistant concrete (Herrenknecht, 2006).

A sulfate resistant concrete is required for this project due to a maximum salinity of 25 PPT (25 g/L) shown in Figure 11.

By substituting 35% fly ash for Portland cement in the concrete, it can cut the overall permeability of the concrete in half while sufficiently resisting the chemical attacks of the high concentration of salt water (Engineering News Record, 1998). Fly ash type concrete

has become more popular over the recent years in the construction of concrete with direct contact with sea water and has great environmentally safe characteristics.

An adequate cover of 75mm should also alleviate any corrosion problems but it is suggested to use epoxy coated re-bars or any other method of protection on the steel reinforcement (Bargaheiser et al, 2006)

7.4 Lining Methods

Lining methods available for the lining of shafts:

- Soldier piles and lagging
- Ring beams and lagging or liner plate
- Pre-cast segmental lining
- Steel Sheet Pile walls
- Diaphragm walls cast inslurry sections
- Secant Pile walls

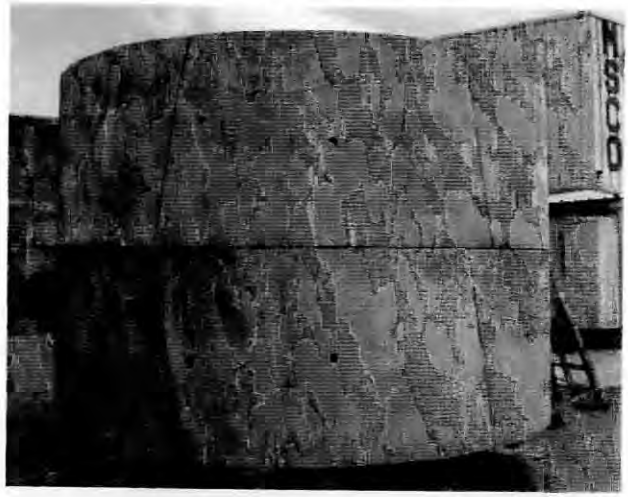
(Department of US Army, 1997)

Installing pre-cast segments proves the most beneficial to the project. Pre-cast segments allow for fast installation and excavation, eliminated lengthy delays for concrete curing time to reach its required compressive strength, and no heat of hydration exerted on the frozen wall to compromise the structural integrity. In addition, it is the most common method of lining for shaft excavation projects using ground freezing technology (Andersland and Ladanyi, 2004).

7.5 Segmental Lining

FIGURE 23: Pre-cast concrete lining segments

Source: (Clean Water, 2006)



A one pass segment lining system needs to comply with all the demands of a final lining, resulting from soil conditions, ground water inflow, and utilization.

Concrete cannot be completely impermeable and therefore will never be able to make a complete water tight seal (Bargaheiser et al, 2006). As stated previously, the addition of fly ash into the concrete will decrease the permeability to an acceptable level. All connections between segments will therefore need to have concrete keys on the vertical and horizontal connections.

After placement, the connections will be grouted to provide a seal of roughly the same permeability as the concrete segments.

7.6 Loading

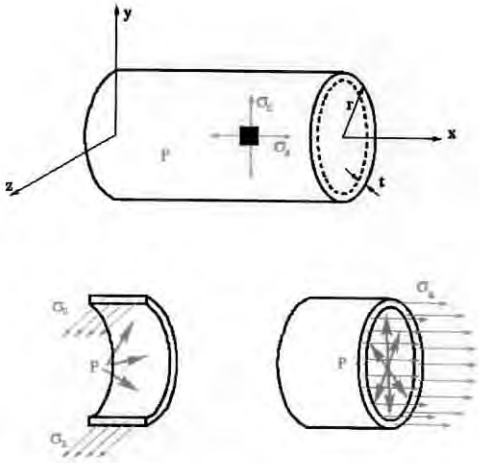
All the external pressures due to the earth and groundwater have been detailed in section 5.3.

7.6.1 Hoop Stress

Circular concrete linings subjected to a uniform external pressure will experience a uniform compressive stress known as hoop stress.

FIGURE 24: Hoop Stress

Source: (Bagley College, 2006)



$$\sigma_h = \frac{pr}{t} \qquad \textbf{(7.6-1)}$$

p = pressure
r = radius
t = thickness of wall

Equation (7.6-1) shows the hoop stress equation for a thin walled cylindrical shape. In practice, a modified version of this equation is used and is shown in section 5.11.7 to design the minimum required thickness of the lining.

7.7 Design of Concrete Liner Thickness

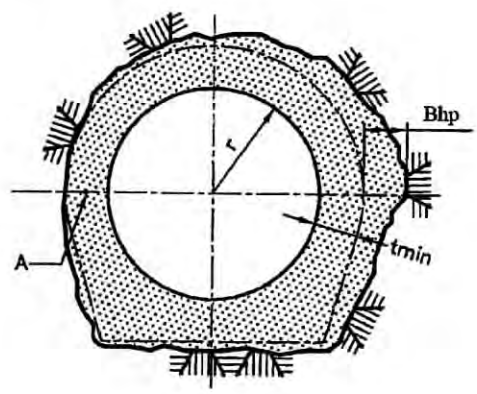
The required minimum thickness of the lining system increases with depth due to the accumulating earth and water pressures. Therefore, intervals of different thicknesses will be used along the depth of the shaft.

FIGURE 25: Liner Thickness

Source: (Bibollet, 2006)

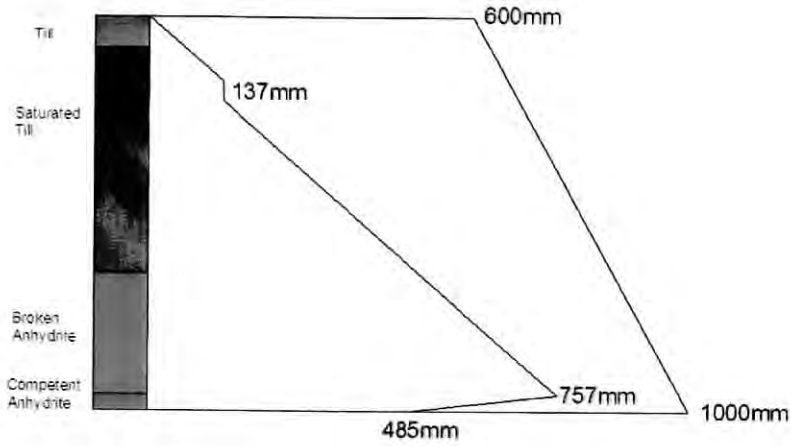
$$t_{\min} = \frac{p\left(r + \frac{Bhp}{2}\right)}{\phi f'_c - p} \qquad \textbf{(7.7-1)}$$

(Bibollet, 2006)



t_{min} = minimum concrete thickness of lining (m)
r = Interior radius (m)
f'_c = 91 day compression resistance (kPa)
p = external hydrostatic pressure (kPa)
Bhp = 0.3 m
ϕ = 0.65 concrete resistance factor (NBCC 1995)

FIGURE 26: Required and actual liner thickness



From previously calculated earth and water pressures, required lining thicknesses corresponding to depths have been evaluated in a table shown in Appendix C.

8.0 Excavation Techniques

Once the growth of the frozen earth wall is complete and its continuity and water tightness confirmed the excavation can begin. Care must be taken during any excavation process at the Money Pit to preserve the archaeological integrity of the site.

Using the road header technique is the most effective for trimming the frozen wall in a circular shaft (Sopko, 2004).

Excavation using a conventional hoe ram and hand operated pneumatic spades is the most common practice while excavating a frozen shaft. This is very labour intensive but should be used along with hand mining methods as there are archaeological concerns ate the site (Sopko, 2002).

Before the commencement of an excavation of the Money Pit a Treasure Trove License must be obtained. A heritage research permit must also be obtained from the Nova Scotia Museum or a letter from the curator that said permit is not required (Nova Scotia Natural Resources, 2004). Upon the discovery of anything of archeological significance the Nova Scotia Museum must be contacted. Before work can continue an archeologist must

be brought on site to evaluate the situation. Due to the innate slowness of government institutions there are two possible alternatives to avoid lengthy work stoppages: (1) have a qualified archeologist on site for the duration of the excavation or (2) have a qualified archeologist on 24 hour retainer who could be quickly summoned onsite in the event of a significant discovery (Iman, 2006).

9.0 Considerations outside scope of project

Presented below are two important issues that must be considered before the final design of a deep excavation at the Money Pit.

9.1 Drilling Program to be conducted

To date there is a lack of comprehensive geotechnical data for the Money Pit. For any artificial ground freezing program, the frozen long term strength and frozen and unfrozen thermal properties of each stratum much be determined. As well, the swelling potential of the anhydrite must be tested for. Swelling anhydrite has been known to cause significant problems in underground construction; if the anhydrite present at the Money Pit has a significant swelling potential it must be accounted for in the final design of the deep excavation (MacPhie, 2006).

A site investigation must be conducted with both shallow borings and borings that extend well bellow the planned depth of the excavation. Soils samples are required to test for both frozen and unfrozen strength in each stratum. Soil type, density and water content must be accurately determined in order to properly estimate the thermal properties of the subsurface. Ground temperatures and permeability must also be determined. (Andersland and Ladanyi, 2004). A detailed subsurface investigation is also required to more accurately determine the extents of the original construction. Once this data is obtained the diameter of the required shaft can be more confidently established.

9.2 Future Development of Tourism Infrastructure

The decision to convert the shaft into a permanent structure will be based on results of the investigation of the Money Pit. The utilization and serviceability requirements of this infrastructure will be completely different than the current project and must be designed accordingly. For example, the shaft lining should be completely impermeable and thus, an impermeable geomembrane must be placed between the current lining and a new cast-in-place final lining (Harris, 1995). Other necessary designs should be calculated in conjunction with all regulations and building codes for an underground structure.

10.0 Conclusions

After meticulous research of the site data and design of the frozen earth wall, artificial ground freezing has proven to be a realistic ground support and groundwater control system for a deep excavation at the Money Pit. The strong hydraulic connection and results from previous attempts at excavation, prove that other conventional methods will not provide the same measure of success. Before a final design for a deep excavation of the Money Pit can be executed, a drilling and soil testing program must be carried out to properly define the geotechnical properties of the subsurface.

Through the execution of this design project we have learned of the mystery of the Money Pit, its history and current condition, how to design a frozen earth wall using artificial ground freezing and how to design the necessary lining. When we took on this project all of the above topics were unfamiliar to us. We learned how to conduct preliminary historical research, gather geotechnical data from reports, design a shaft lining and discovered and applied a new technology, artificial ground freezing. We were very fortunate to function successfully as a team. The success of our team enabled us to equitably divide the work and successfully meet our deadlines.

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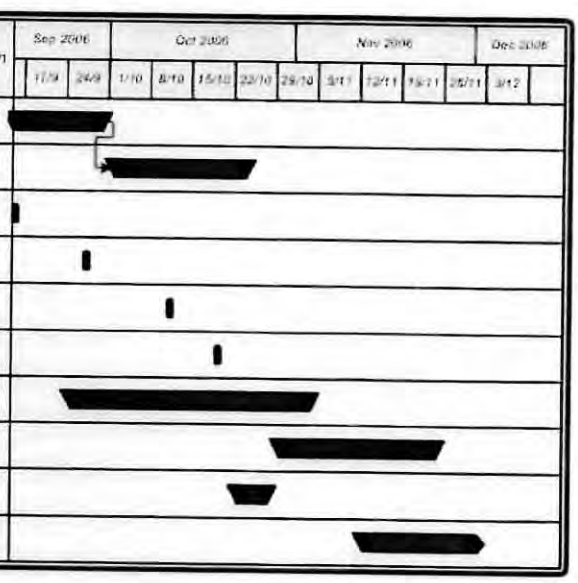
Division of Responsibility

The design team was divided into three main areas. These were freeze ring research and design, shaft lining research and design, and information and data analysis. All team members attended information meetings with professional experts. Works done in all areas were done concurrently with team members constantly providing support for one another. All work was verified through peer review as well as the professional opinion of the external project supervisor. The project timeframe is shown in the Gantt chart below and set deadlines were met. The project team worked together well and with substantial synergy.

FIGURE 27: Gantt Chart

ID	Task	Start	Finish	Duration
1	Preliminary Research	9/15/2006	9/29/2006	2w 1d
2	Method Investigation	10/2/2006	10/23/2006	3w 1d
3	Meeting (Les) Intro	9/15/2006	9/15/2006	1d
4	Meeting (Les) Method Investigation	9/27/2006	9/27/2006	1d
5	Meeting (Les) Grouting/Ground Freezing	10/11/2006	10/11/2006	1d
6	Meeting (Les/Sopko/O'Connor) Ground Freezing Presentation	10/19/2006	10/19/2006	1d
7	Site Investigation/Analysis	9/25/2006	11/3/2006	6w
8	Design	10/30/2006	11/24/2006	4w
9	Midterm Presentation	10/23/2006	10/27/2006	1w
10	Final Presentation and Report	11/13/2006	12/1/2006	3w



Appendix A: Select Borehole records

RECORD OF BOREHOLE 102

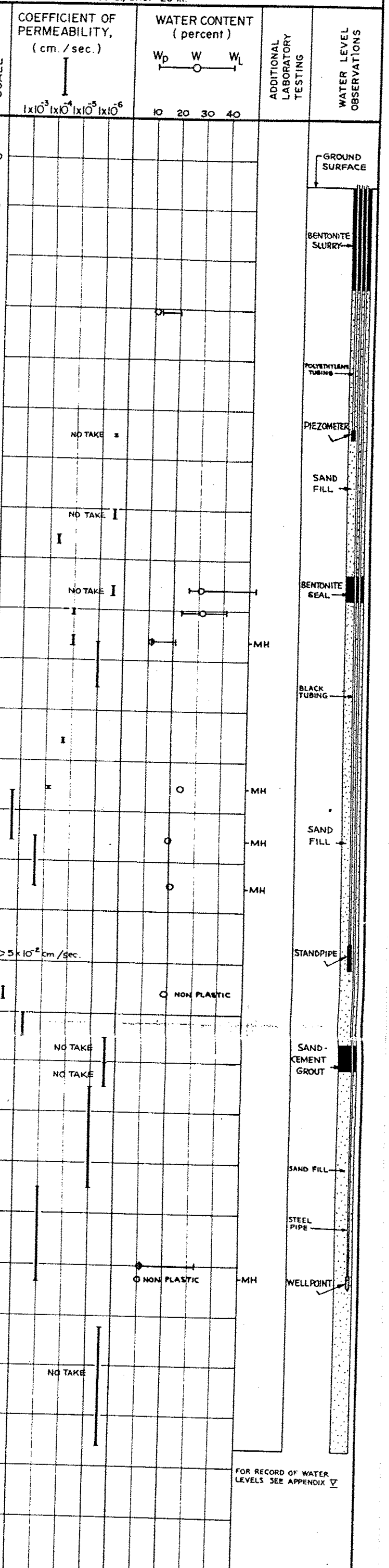
LOCATION See figure 1. BORING DATE APRIL 17 TO 28, 1970 DATUM LOCAL
SAMPLER HAMMER WEIGHT 140 lb., DROP 30 in CASING HAMMER WEIGHT 350 lb., DROP 20 in.

BORING METHOD	ELEV. / DEPTH	DESCRIPTION
	985.0	GROUND SURFACE
H CASING	0.0	COBBLES & BOULDERS IN A LOOSE GREY SILTY SAND MATRIX (FILL)
	3.0	COBBLES AND BOULDERS IN A STIFF BROWN CLAYEY SILT AND SAND MATRIX.
	970.0 / 15.0	HARD GREY CLAYEY SILT TO CLAYEY SILT AND SAND WITH SOME GRAVEL AND OCCASIONAL COBBLES AND BOULDERS (CLAYEY TILL)
NX CASING	905.0 / 80.0	HARD LAYERED GREY SILTY CLAY AND BROWN CLAYEY SILT, TRACE SAND AND TRACE TO SOME GRAVEL THROUGHOUT
	898.0 / 87.0	HARD RED-GREY CLAYEY SILT AND SAND, SOME GRAVEL (TILL)
	893.0 / 92.0	DENSE BROWN SAND, OCCASIONAL COBBLES
	888.0 / 97.0	VERY DENSE BROWN SANDY SILT WITH SOME CLAYEY SILT LAYERS & OCCASIONAL COBBLES BECOMING WITH DEPTH, HARD BROWN LAYERED CLAYEY SILT WITH SOME SILTY SAND LAYERS & OCCAS. THIN SILT PARTINGS
BX CASING	873.0 / 112.0	VERY DENSE BROWN SILT TO SANDY SILT WITH A TRACE TO SOME CLAY, SOME SILTY SAND LAYERS - IRREGULARLY STRATIFIED STRUCTURE WITH SOME GREY LAYERS - OCCASIONAL SHALE AND DOLOMITE GRAVEL - OCCASIONAL GYPSIFEROUS SEAMS
	838.0 / 147.0	VERY LOOSE TO LOOSE BROWN SAND TRACE SILT TO BROWN SAND AND GRAVEL (VERY ANGULAR PARTICLES)
	829.0 / 156.0	COMPACT TO VERY DENSE BROWN TO GREY SANDY SILT TO SILTY SAND TRACE TO SOME CLAY, SOME GRAVEL (SILTY TILL)
BX CORE	822.5 / 162.5	SOUND LIGHT GREY FINE TO MEDIUM GRAINED ANHYDRITE BEDROCK WITH OCCASIONAL THIN FRACTURES HEALED WITH GYPSUM - SOME DARK GREY DOLOMITE INCLUSIONS
	792.0 / 193.0	ANHYDRITE BEDROCK WITH SOLUTION CAVITIES, CAVITIES FILLED WITH BROWN SILTY CLAY TO CLAYEY SILT
AX CASING	771.0 / 214.0	HARD BROWN SILTY CLAY WITH SAND & GRAVEL, SOME SILT ZONES
	767.3 / 218.7	SOUND LIGHT GREY ANHYDRITE BEDROCK
	762.5 / 222.5	SOFT WHITE GYPSUM
AX CORE	758.8 / 226.2	SOUND LIGHT GREY FINE GRAINED ANHYDRITE BEDROCK WITH SOME LIMESTONE AND DOLOMITE INCLUSIONS - OCCASIONAL THIN SEAMS OF WHITE SANDSTONE BELOW ABOUT ELEV. 747.
	733.9 / 251.1	END OF HOLE

ROTARY DRILLING

SAMPLES

NUMBER	TYPE	BLOWS / FOOT	PERCENT CORE RECOVERY	ESTIMATED PERCENT DRILL WATER RETURN
1	2" D.O.	27		
2	NXL CORE	-	40	100
3	"	-	70	100
4	2" D.O.	32		
5	"	23		
6	"	75		
7	NXL CORE	-	20	100
8	2" D.O.	144		
9	NXL CORE	-	40	100
10	2" D.O.	124		
11	"	109		
12	"	52		
13	3" T.P.	Wh		
14	2" D.O.	156		
15	NXL CORE		35	100
16	2" D.O.	175/7"		
17	"	175		
18	"	140/6"		
19	"	130		
20	"	147/10"		
21	"	WR/12" 17		
22	"	WR/24" 65		
23	"	24		
24	BXL RC	-	100	100
25	"	-	100	100
26	"	-	100	90
27	"	-	100	90
28	"	-	100	90
29	"	-	93	90
30	"	-	86	90
31	"	-	30	90
32	"	-	93	90
33	"	-	50	90
34	"	-	35	90
35	"	-	30	90
36	2" D.O.	61		
37	"	80		
38	BXL RC	-	68	80
39	"	-	48	80
40	AXT RC	-	10	100
41	"	-	93	100
42	"	-	100	100
43	"	-	100	100
44	"	-	100	100
45	"	-	100	100



FOR RECORD OF WATER LEVELS SEE APPENDIX V

VERTICAL SCALE (FT.)
5 0 5 10 20

Golder Associates

DRAWN
CHECKED

RECORD OF BOREHOLE 103

LOCATION See figure 1. BORING DATE MAY 1 - 19, 1970 DATUM LOCAL
SAMPLER HAMMER WEIGHT 140 lb., DROP 30 in. CASING HAMMER WEIGHT 350 lb., DROP 20 in.

BORING METHOD	ELEV. / DEPTH	DESCRIPTION	STRATA PLOT	SAMPLES: NUMBER	TYPE	BLOWS / FOOT	PERCENT CORE RECOVERY	ESTIMATED PERCENT DRILL WATER RETURN	ADDITIONAL LABORATORY TESTING	WATER LEVEL OBSERVATIONS
	985.9 / 0.0	TOP OF DRILLING PLATFORM TIMBERS								
		AIR								
	974.9 / 11.0	TIMBERS IN A SOIL MATRIX								
	965.4 / 20.5	WATER								
ROTARY DRILLING, 4" DIA. CASING	865.4	BOTTOM OF SHAFT								
	120.5	VERY LOOSE SILTY SAND TO SANDY SILT								
	123.0 / 855.4	COMPACT TO DENSE BROWN SAND & SILT WITH SOME GRAVEL, TRACE OF CLAY, TIMBER & OCC. PINE NEEDLES		1	2" D.O.	37				
	130.5 / 847.9	DENSE TO VERY DENSE GREY SANDY SILT TO SILTY SAND, SOME GRAVEL, TRACE TO SOME CLAY (SILTY TILL)		2	"	36				
				3	"	47				
				4	"	56				
				5	"	34				
	138.0	ANGULAR, LIGHTLY CEMENTED SANDSTONE FRAGMENTS IN A SANDY SILT, TR. CLAY MATRIX		6	"	74				
NX CASING	140.5 / 841.0	VERY SOFT DARK GREY TO BLACK CLAY WITH SOME SILT AND GRAVEL, TRACE OF SAND, OCCASIONAL TIMBERS		7	"	1				
				8	2" S.T.	WA	100			
				9	NX RC	–	50			
	144.9	LOOSE TO COMPACT BROWN TO GREY SANDY SILT TO SILTY SAND AND GRAVEL, TRACE OF CLAY THROUGHOUT. (SILTY TILL)		10	3" T.P.	PM	0			
				11	"	PM				
				12	2" D.O.	16				
				13	"	14				
				14	"	22				
	829.4			15	"	5				
BX CASING	156.5	SOUND LIGHT GREY ANHYDRITE BEDROCK WITH SOME DOLOMITE AND LIMESTONE INCLUSIONS, OCCASIONAL SEAMS OF CRYSTALLINE GYPSUM UP TO 1/4 IN. THICK.		17	NX, RC	–	100			
				18	NX RC	–	100			
				19	"	–	97			
				20	"	–	20	100		
		4 IN. THICK SEAM OF FRACTURED LIMESTONE AT ABOUT ELEV. 810.0, FRACTURES PARTIALLY HEALED WITH CRYSTALLINE GYPSUM.		21	"	–	100			
	805.4 / 180.5	FRACTURED ANHYDRITE BEDROCK OR SOFT GYPSUM BEDROCK.		22	"	–	30			
	801.4 / 184.5	VERY LOOSE BROWN SANDY SILT WITH SOME CLAY TO VERY SOFT CLAYEY SILT AND SAND, TRACE TO SOME GRAVEL THROUGHOUT – LOOSE GRAVEL LAYER FROM ELEV. 794.9 TO 793.9		23	"	–	0	0		
				24	2" D.O.	WH				
				25	2" S.T.	P.M.				
	791.9			26	2" T.O.	WH				
	194.0	COMPACT TO DENSE BROWN TO GREY SILT WITH SOME SAND, TRACE TO SOME CLAY. GRAVEL LAYER AT ABOUT ELEV. 790		27	2" D.O.	24				
				28	"	220				
	785.9			29	"	49				
				30	"	26				
	200.0 / 780.0	VERY DENSE BROWN TO GREY SILTY SAND, SOME GRAVEL, TRACE CLAY		31	"	66				
				32	"	114				
				33	"	112/9"	72			
BXT CORE	205.3	SOUND LIGHT GREY ANHYDRITE BEDROCK WITH SOME DOLOMITE AND LIMESTONE INCLUSIONS – SOME GYPSUM SEAMS AND INCLUSIONS. – SOME FULLY HEALED AND PARTIALLY HEALED FRACTURES UP TO 1/8 IN. THICK, FRACTURES HEALED WITH CRYSTALLINE GYPSUM. – SLIGHTLY VUGULAR STRUCTURE WITHIN PARTIALLY HEALED FRACTURES		34	BXT RC	–	88			
				35		–				
				36	"	–	93	100		
				37	"	–	100			
				38	"	–	83	25		
				39	"	–	100			
				40	"	–	100			
				41	"	–	84			
				42	"	–	98	0		
				43	"	–	100			
	737.3			44	"	–	100			
	248.6	END OF HOLE								

ELEVATION SCALE: 990, 980, 970, 960, 950, 940, 930, 920, 910, 900, 890, 880, 870, 860, 850, 840, 830, 820, 810, 800, 790, 780, 770, 760, 750, 740, 730, 720

COEFFICIENT OF PERMEABILITY, (cm./sec.): 1×10^{-3} 1×10^{-4} 1×10^{-5} 1×10^{-6}

WATER CONTENT (percent): W_P W W_L — 10 20 30 40

MH

PIECE OF METAL RECOVERED FROM ELEV. 847.9

PIECE OF METAL RECOVERED FROM ELEV. 844.4

WOOD FROM ELEV. 842.9 TO 841.0, TIMBER APPARENTLY MOVED INTO HOLE AFTER SA. 8 OBTAINED.

NO TAKE

NO TAKE

WATER LEVEL OBSERVATIONS: STEEL PIPE, BENTONITE SEAL, CAVED MATERIAL, WELLPOINT, CEMENT SEAL, BLACK TUBING, SAND FILL, STANDPIPE, CEMENT SEAL

FOR RECORD OF WATER LEVELS SEE APPENDIX V

VERTICAL SCALE (FT.) 5 0 5 10 20

Golder Associates

DRAWN M.W.

CHECKED

70

Appendix B: Frozen Earth Wall Calculations

$$\frac{b}{a} = \exp\left[\frac{p_e}{2c}\right]$$

$W = b\text{-}a$

Statum	Distance from surface (m)	Pressure Envelope (kPa)	a (m)	c (kPa)	b (m)	W_{requ} (m)	Temp (˚C)	$W_{act\ @\ 50\ days}$
Unsaturated Glacial Till	0.00	0.00	10.668	2000	10.668	0	-22	1.87
	1.00	16.24	10.668	2000	10.71139	0.04339	-22	1.87
	2.00	32.47	10.668	2000	10.75496	0.086957	-22	1.87
	3.00	48.71	10.668	2000	10.7987	0.130701	-22	1.87
K_0=0.5	4.00	64.95	10.668	2000	10.84262	0.174623	-22	1.87
γ_d=16.70 kN/m3	5.00	81.18	10.668	2000	10.88672	0.218724	-22	1.87
	5.76	93.52	10.668	2000	10.92036	0.252361	-22	1.87
	5.77	93.68	10.668	2000	10.9208	0.252804	-22	1.87
	6.00	97.42	10.668	2000	10.931	0.263004	-22	1.87
	7.00	113.65	10.668	2000	10.97546	0.307464	-22	1.87
	8.00	129.89	10.668	2000	11.02011	0.352105	-22	1.87
	9.00	146.13	10.668	2000	11.06493	0.396928	-22	1.87
	10.00	162.36	10.668	2000	11.10993	0.441933	-22	1.87
	11.00	178.60	10.668	2000	11.15512	0.487121	-22	1.87
	11.93	193.70	10.668	2000	11.19731	0.52931	-22	1.87
	12.00	193.70	10.668	2000	11.19731	0.52931	-22	1.87
	13.00	193.70	10.668	2000	11.19731	0.52931	-23	1.94
	14.00	193.70	10.668	2000	11.19731	0.52931	-23	1.94
	15.00	193.70	10.668	2000	11.19731	0.52931	-23	1.94
	15.43	193.70	10.668	2000	11.19731	0.52931	-23	1.94
Saturated Glacial Till	16.00	202.26	10.668	2000	11.2213	0.553296	-23	1.94
	17.00	217.31	10.668	2000	11.26361	0.59561	-23	1.94
	18.00	232.37	10.668	2000	11.30608	0.638083	-23	1.94
K_0=0.5	19.00	247.42	10.668	2000	11.34872	0.680717	-23	1.94
γ_{sat}=20.3 kN/m3	20.00	262.48	10.668	2000	11.39151	0.723511	-23	1.94
	21.00	277.53	10.668	2000	11.43447	0.766466	-23	1.94

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: | Checked by: | Verified by:

Design Detail:
Required frozen earth wall thickness, and achieved thickness after 50 days

1/3

Statum	Distance from surface	Pressure Envelope	a	c	b	W_{requ}	Temp	$W_{act\ @\ 50\ days}$
[redacted]	22.00	292.59	10.668	2000	11.47758	0.809584	-23	1.94
	23.00	307.64	10.668	2000	11.52086	0.852864	-23	1.94
	24.00	322.70	10.668	2000	11.56431	0.896307	-23	1.94
	25.00	337.75	10.668	2000	11.60791	0.939915	-23	1.94
	26.00	352.81	10.668	2000	11.65169	0.983686	-23	1.94
	27.00	367.86	10.668	2000	11.69562	1.027623	-23	1.94
	28.00	382.92	10.668	2000	11.73973	1.071725	-23	1.94
	29.00	397.97	10.668	2000	11.78399	1.115994	-23	1.94
	30.00	413.03	10.668	2000	11.82843	1.160429	-23	1.94
	31.00	428.08	10.668	2000	11.87303	1.205033	-23	1.94
	32.00	443.14	10.668	2000	11.9178	1.249804	-23	1.94
	33.00	458.19	10.668	2000	11.96274	1.294744	-23	1.94
	34.00	473.25	10.668	2000	12.00785	1.339854	-23	1.94
	35.00	488.30	10.668	2000	12.05313	1.385133	-23	1.94
	36.00	503.36	10.668	2000	12.09858	1.430584	-23	1.94
	37.00	518.41	10.668	2000	12.14421	1.476206	-23	1.94
	38.00	533.47	10.668	2000	12.19	1.522	-24	2.01
	39.00	548.52	10.668	2000	12.23597	1.567966	-24	2.01
	40.00	563.58	10.668	2000	12.28211	1.614106	-24	2.01
	41.00	578.63	10.668	2000	12.32842	1.66042	-24	2.01
	42.00	593.69	10.668	2000	12.37491	1.706908	-24	2.01
	43.00	608.74	10.668	2000	12.42157	1.753572	-24	2.01
	44.00	623.80	10.668	2000	12.46841	1.800412	-24	2.01
	45.00	638.85	10.668	2000	12.51543	1.847428	-24	2.01
	46.00	653.91	10.668	2000	12.56262	1.894622	-24	2.01
	47.00	668.96	10.668	2000	12.60999	1.941994	-24	2.01
	47.71	679.65	10.668	2000	12.64374	1.975736	-24	2.01

Project Title:

Design of a Deep Excavation using Ground Freezing Technology

Project Detail:

Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by:	Checked by:	Verified by:

Design Detail:
Required frozen earth wall thickness, and achieved thickness after 50 days 2/3

Statum	Distance from surface (m)	Pressure Envelope (kPa)	a (m)	c (kPa)	b (m)	W_{requ} (m)	Temp (˚C)	$W_{act\ @\ 50\ days}$
	47.72	679.80	10.668	5376	11.36426	0.696264	-24	4.49
	48.00	683.90	10.668	5376	11.3686	0.700603	-24	4.49
	49.00	698.56	10.668	5376	11.38411	0.716111	-24	4.49
	50.00	713.22	10.668	5376	11.39964	0.731641	-24	4.49
	51.00	727.87	10.668	5376	11.41519	0.747192	-24	4.49
	52.00	742.53	10.668	5376	11.43076	0.762764	-24	4.49
	53.00	757.19	10.668	5376	11.44636	0.778358	-24	4.49
	54.00	771.85	10.668	5376	11.46197	0.793972	-24	4.49
Broken Anhydrite	55.00	786.50	10.668	5376	11.47761	0.809608	-24	4.49
	56.00	801.16	10.668	5376	11.49327	0.825266	-24	4.49
K=0.25	57.00	815.82	10.668	5376	11.50894	0.840944	-24	4.49
γ_{sat}=29.2	58.00	830.48	10.668	5376	11.52464	0.856644	-24	4.49
	59.00	845.13	10.668	5376	11.54037	0.872366	-24	4.49
	60.00	859.79	10.668	5376	11.55611	0.888109	-24	4.49
	61.00	874.45	10.668	5376	11.57187	0.903873	-24	4.49
	62.00	889.11	10.668	5376	11.58766	0.919659	-24	4.49
	63.00	903.76	10.668	5376	11.60347	0.935467	-24	4.49
	64.00	918.42	10.668	5376	11.6193	0.951296	-25	4.60
	65.00	933.08	10.668	5376	11.63515	0.967146	-25	4.60
	66.00	947.74	10.668	5376	11.65102	0.983019	-25	4.60
	67.00	962.39	10.668	5376	11.66691	0.998913	-25	4.60
	68.00	977.05	10.668	5376	11.68283	1.014828	-25	4.60
	69.00	991.71	10.668	5376	11.69877	1.030765	-25	4.60
	70.00	1006.37	10.668	5376	11.71472	1.046725	-25	4.60
	70.11	1007.98	10.668	5376	11.71648	1.048481	-25	4.60
	70.12	631.37	10.668	5376	11.3132	0.645197	-25	4.60
Sound Anhydrite	71.00	640.00	10.668	5376	11.32228	0.654284	-25	4.60
K=0.0	72.00	649.81	10.668	5376	11.33262	0.664619	-25	4.60
γ_{sat}=29.2	73.00	659.62	10.668	5376	11.34296	0.674964	-25	4.60
	73.16	661.19	10.668	5376	11.34462	0.67662	-25	4.60

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: | Checked by: | Verified by:

Design Detail:
Required frozen earth wall thickness, and achieved thickness after 50 days 3/3

$$u_c = \frac{k_f}{4S\ln\left(\frac{S}{4r_o}\right)}\frac{V_s}{V_o}$$

k_f (1) (W/m°C)	S (m)	r_o (m)	T_{ground} (2) (°C)	T_{pipe} (°C)	$T_{freezeground}$ (°C)	V_s (°C)	V_o (°C)	u_c (m/day)
4.5	1	0.038	7.5	-30	-2	28	9.5	1.76
4.5	1	0.038	7.5	-25	-2	23	9.5	1.45
4.5	1	0.038	7.5	-24	-2	22	9.5	1.38
4.5	1	0.038	7.5	-23	-2	21	9.5	1.32
4.5	1	0.038	7.5	-22	-2	20	9.5	1.26
4.5	1	0.038	7.5	-21	-2	19	9.5	1.19
4.5	1	0.038	7.5	-20	-2	18	9.5	1.13
4.5	1	0.038	7.5	-19	-2	17	9.5	1.07
4.5	1	0.038	7.5	-18	-2	16	9.5	1.01
4.5	1	0.038	7.5	-17	-2	15	9.5	0.94
4.5	1	0.038	7.5	-16	-2	14	9.5	0.88
4.5	1	0.038	7.5	-15	-2	13	9.5	0.82

$V_o = |T_{freezeground} - T_{ground}|$

$V_s = |T_{pipe} - T_{freezeground}|$

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by:	Checked by:	Verified by:
Shoshanna Saxe		

Design Detail:
Maximum allowable flow in Broken Anhydrite

$$t_I = \frac{(S/2)^2}{4K_1V_s}\left[L_I\left(2\ln R' - 1\right) + C_1V_s\right]$$

$$L_I = L + C_1V_o + 3C_2V_o$$

$$R' = \frac{S/2}{r_o}$$

TILL

S (m)	r_o (m)	R'	L (MJ/m^3.°C)	C_1 (kJ/m^3.°C)	C_2 kJ/m^3.°C	T_{ground} (°C)	T_{pipe} (°C)	$T_{freezeground}$ (°C)
1	0.038	13.1578947	208	1972	2885	7.5	-30	-2
1	0.038	13.1578947	208	1972	2885	7.5	-25	-2
1	0.038	13.1578947	208	1972	2885	7.5	-24	-2
1	0.038	13.1578947	208	1972	2885	7.5	-23	-2
1	0.038	13.1578947	208	1972	2885	7.5	-22	-2
1	0.038	13.1578947	208	1972	2885	7.5	-21	-2
1	0.038	13.1578947	208	1972	2885	7.5	-20	-2
1	0.038	13.1578947	208	1972	2885	7.5	-19	-2
1	0.038	13.1578947	208	1972	2885	7.5	-18	-2
1	0.038	13.1578947	208	1972	2885	7.5	-17	-2
1	0.038	13.1578947	208	1972	2885	7.5	-16	-2
1	0.038	13.1578947	208	1972	2885	7.5	-15	-2

V_s (°C)	V_o (°C)	L_I	K_1 (kJ/day.m.°C)	t_I (days)
28	9.5	308956.5	164.4	18.18
23	9.5	308956.5	164.4	21.96
22	9.5	308956.5	164.4	22.93
21	9.5	308956.5	164.4	23.98
20	9.5	308956.5	164.4	25.15
19	9.5	308956.5	164.4	26.43
18	9.5	308956.5	164.4	27.86
17	9.5	308956.5	164.4	29.45
16	9.5	308956.5	164.4	31.24
15	9.5	308956.5	164.4	33.28
14	9.5	308956.5	164.4	35.60
13	9.5	308956.5	164.4	38.28

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: Shoshanna Saxe | Checked by: | Verified by:

Design Detail:
Thermal Design, time one, t_I

$$t_{II} = \frac{L_{II} S^2}{8 K_1 V_s}\left(x^2 - 0.62\right) \qquad x = \frac{W}{S} \qquad L_{II} = L + \frac{1}{2} C_1 V_s$$

TILL

S (m)	W (m)	x $=W/S$	L (MJ/m^3.°C)	C_1 (kJ/m^3.°C)
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972
1	2	2	208	1972

T_{pipe} (°C)	$T_{freezeground}$ (°C)	V_s (°C)	L_{II}	K_1 (kJ/day.m.°C)	t_{II} (days)
-30	-2	28	235608	164.4	21.63
-25	-2	23	230678	164.4	25.78
-24	-2	22	229692	164.4	26.83
-23	-2	21	228706	164.4	27.99
-22	-2	20	227720	164.4	29.26
-21	-2	19	226734	164.4	30.67
-20	-2	18	225748	164.4	32.23
-19	-2	17	224762	164.4	33.98
-18	-2	16	223776	164.4	35.94
-17	-2	15	222790	164.4	38.17
-16	-2	14	221804	164.4	40.72
-15	-2	13	220818	164.4	43.65

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by:	Checked by:	Verified by:

Design Detail:
Thermal Design, time two, t_{II}

$$t_I = \frac{(S/2)^2}{4K_1V_s}\left[L_I\left(2\ln R' - 1\right) + C_1V_s\right]$$

$$L_I = L + C_1V_o + 3C_2V_o$$

$$R' = \frac{S/2}{r_o}$$

ANHYDRITE

S (m)	r_o (m)	R'	L (MJ/m^3.°C)	C_1 (kJ/m^3.°C)	C_2 kJ/m^3.°C	T_{ground} (°C)	T_{pipe} (°C)
1	0.038	13.1578947	150.2	569	569	7.5	-30
1	0.038	13.1578947	150.2	569	569	7.5	-25
1	0.038	13.1578947	150.2	569	569	7.5	-24
1	0.038	13.1578947	150.2	569	569	7.5	-23
1	0.038	13.1578947	150.2	569	569	7.5	-22
1	0.038	13.1578947	150.2	569	569	7.5	-21
1	0.038	13.1578947	150.2	569	569	7.5	-20
1	0.038	13.1578947	150.2	569	569	7.5	-19
1	0.038	13.1578947	150.2	569	569	7.5	-18
1	0.038	13.1578947	150.2	569	569	7.5	-17
1	0.038	13.1578947	150.2	569	569	7.5	-16
1	0.038	13.1578947	150.2	569	569	7.5	-15

$T_{freezeground}$ (°C)	V_s (°C)	V_o (°C)	L_I	K_1 (kJ/day.m.°C)	t_I (days)
-2	28	9.5	171822	388.8	4.19
-2	23	9.5	171822	388.8	5.08
-2	22	9.5	171822	388.8	5.31
-2	21	9.5	171822	388.8	5.56
-2	20	9.5	171822	388.8	5.83
-2	19	9.5	171822	388.8	6.13
-2	18	9.5	171822	388.8	6.47
-2	17	9.5	171822	388.8	6.84
-2	16	9.5	171822	388.8	7.26
-2	15	9.5	171822	388.8	7.74
-2	14	9.5	171822	388.8	8.29
-2	13	9.5	171822	388.8	8.92

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: | Checked by: | Verified by:

Design Detail:
Thermal Design, time one, t_I

ANHYDRITE

$$t_{II} = \frac{L_{II} S^2}{8 K_1 V_s}\left(x^2 - 0.62\right) \qquad x = \frac{W}{S} \qquad L_{II} = L + \frac{1}{2} C_1 V_s$$

S (m)	W (m)	x $=W/S$	L (MJ/m^3.°C)	C_1 (kJ/m^3.°C)	T_{pipe} (°C)	$T_{freezeground}$ (°C)
1	2	2	150.2	569	-30	-2
1	2	2	150.2	569	-25	-2
1	2	2	150.2	569	-24	-2
1	2	2	150.2	569	-23	-2
1	2	2	150.2	569	-22	-2
1	2	2	150.2	569	-21	-2
1	2	2	150.2	569	-20	-2
1	2	2	150.2	569	-19	-2
1	2	2	150.2	569	-18	-2
1	2	2	150.2	569	-17	-2
1	2	2	150.2	569	-16	-2
1	2	2	150.2	569	-15	-2

V_s (°C)	L_{II}	K_1 (kJ/day.m.°C)	t_{II} (days)
28	158166	388.8	6.14
23	156743.5	388.8	7.41
22	156459	388.8	7.73
21	156174.5	388.8	8.08
20	155890	388.8	8.47
19	155605.5	388.8	8.90
18	155321	388.8	9.38
17	155036.5	388.8	9.91
16	154752	388.8	10.51
15	154467.5	388.8	11.19
14	154183	388.8	11.97
13	153898.5	388.8	12.86

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by:	Checked by:	Verified by:

Design Detail:
Thermal Design, time two, t_{II}

Calculation of Wall Thickness @ 50 Days: TILL

t_t (days)	S (m)	r_o (m)	R'	L (MJ/m^3.°C)	C_1 (kJ/m^3.°C)	C_2 kJ/m^3.°C	T_{ground} (°C)	T_{pipe} (°C)	$T_{freezeground}$ (°C)
50	1	0.038	13.1578947	208	1972	2885	7.5	-24	-2
50	1	0.038	13.1578947	208	1972	2885	7.5	-23	-2
50	1	0.038	13.1578947	208	1972	2885	7.5	-22	-2

V_s (°C)	V_o (°C)	L_I	L_{II}	K_1 (kJ/day.m.°C)	W (m)
22	9.5	308956.5	229692	164.4	2.01
21	9.5	308956.5	228706	164.4	1.94
20	9.5	308956.5	227720	164.4	1.87

$$t_t = t_1 + t_2$$

$$= \frac{(S/2)^2}{4k_1 V_1}\left[L_1(2\ln R' - 1) + C_1 V_s\right] + \frac{L_{II} S^2}{8 k_1 V_s}\left(\left(\frac{W}{S}\right)^2 - 0.62\right)$$

isolate W

$$W = \left[\sqrt{\frac{8k_1 V_s}{L_{II} S^2}\left[t_1\right] + 0.62}\right] S$$

$$W = \sqrt{\left(\frac{8k_1 V_s}{L_{II} S^2}\left[\frac{(S/2)^2}{4k_1 V_1}\left[L_1(2\ln R' - 1) + C_1 V_s\right]\right]\right) + 0.62} \cdot S$$

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: Shoshanna Saxe | Checked by: | Verified by:

Design Detail:
Thickness of frozen earth wall after 50 days of freezing at -25°C input of calcium chloride brine

Calculation of Wall Thickness @ 50 Days: ANYDRITE

t_t (days)	S (m)	r_o (m)	R'	L (MJ/m^3.°C)	C_1 (kJ/m^3.°C)	C_2 kJ/m^3.°C	T_{ground} (°C)	T_{pipe} (°C)	$T_{freezeground}$ (°C)
50	1	0.038	13.1578947	150.2	569	569	7.5	-25	-2
50	1	0.038	13.1578947	150.2	569	569	7.5	-24	-2

V_s (°C)	V_o (°C)	L_I	L_{II}	K_1 (kJ/day.m.°C)	W (m)
23	9.5	171822	156743.5	388.8	4.60
22	9.5	171822	156459	388.8	4.49

$$w = \sqrt{\left(\frac{8K_1 V_s}{L_{II} S^2}\left[\frac{(S/2)^2}{4K_1 V_1}\left[L_1(2\ln R' - 1) + C_1 V_s\right]\right]\right)} + 0.62 \cdot S$$

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by:	Checked by:	Verified by:
Shoshanna Saxe		

Design Detail:
Thickness of frozen earth wall after 50 days of freezing at -25°C input of calcium chloride brine

Appendix C: Liner Calculations

Short Term Loading

The short term loading was calculated using Peck's Envelope for clayey soils (Craig, 2004).

$p_e = 0$ kpa at ground surface

$p_e \rightarrow$ increases linearly to $0.25h$

$p_e = 0.2\gamma h \rightarrow$ from $0.25h$ - $0.50h$

$p_e \rightarrow$ decreases linearly to 0 kpa at full depth, h

Long-Term Loading

$$p_g = k\gamma h - 2Cu\sqrt{k}$$

$$p_w = \gamma_w h$$

$$p_e = p_w + p_g$$

Stratum	1: Dry till	2: Saturated till	3: Broken Anhydrite	4: Competent Anhydrite	Water
h (m)	5.76	41.95	22.40	3.05	67.4
γ (kN/m^3)	16.70	20.3-9.81=10.49	29.2-9.81=19.39	29.2-9.81=19.39	9.81
k (1)	0.50	0.50	0.25	0	n/a

Project Title: Design of a Deep Excavation using Ground Freezing Technology

Project Detail: Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: JRT | Checked by: | Verified by:

Design Detail: EARTH PRESSURE CALCULATIONS 1 of 2

GEOTECHNICAL ENGINEERING GROUP

STL Geotechnical Engineering Group
Shoshanna Saxe • J. Ryan Thé • Cheehan Leung

Statum	Distance from surface (m)	Water Table (m)	Earth Pressures (kPa)	Water Pressure (kPa)	Effective Pressure (kPa)	Short Term Loading (kPa)	Effective Total Pressure (kPa)
	0.00	-	0.00	-	0.00	0.00	0.00
Unsaturated Glacial Till	1.00	-	8.35	-	8.35	16.24	16.24
	2.00	-	16.70	-	16.70	32.47	32.47
	3.00	-	25.05	-	25.05	48.71	48.71
K_0=0.5	4.00	-	33.40	-	33.40	64.95	64.95
γ_d=16.70 kN/m3	5.00	-	41.75	-	41.75	81.18	81.18
	5.76	-	48.10	-	48.10	93.52	93.52
	5.77	0.01	48.15	0.10	48.25	93.68	93.68
	6.00	0.24	49.35	2.35	51.71	97.42	97.42
	7.00	1.24	54.60	12.16	66.76	113.65	113.65
	8.00	2.24	59.84	21.97	81.82	129.89	129.89
	9.00	3.24	65.09	31.78	96.87	146.13	146.13
	10.00	4.24	70.33	41.59	111.93	162.36	162.36
	11.00	5.24	75.58	51.40	126.98	178.60	178.60
	11.93	6.17	80.45	60.53	140.99	193.70	193.70
	12.00	6.24	80.82	61.21	142.04	-	193.70
	13.00	7.24	86.07	71.02	157.09	-	193.70
	14.00	8.24	91.31	80.83	172.15	-	193.70
	15.00	9.24	96.56	90.64	187.20	-	193.70
	15.43	9.67	98.82	94.86	193.68	-	193.70
Saturated Glacial Till	16.00	10.24	101.80	100.45	202.26	-	202.26
	17.00	11.24	107.05	110.26	217.31	-	217.31
	18.00	12.24	112.29	120.07	232.37	-	232.37
K_0=0.5	19.00	13.24	117.54	129.88	247.42	-	247.42
γ_{sat}=20.3 kN/m3	20.00	14.24	122.78	139.69	262.46	-	262.46
	21.00	15.24	128.03	149.50	277.53	-	277.53
	22.00	16.24	133.27	159.31	292.59	-	292.59
	23.00	17.24	138.52	169.12	307.64	-	307.64
	24.00	18.24	143.76	178.93	322.70	-	322.70
	25.00	19.24	149.01	188.74	337.75	-	337.75
	26.00	20.24	154.25	198.55	352.81	-	352.81
	27.00	21.24	159.50	208.36	367.86	-	367.86
	28.00	22.24	164.74	218.17	382.92	-	382.92
	29.00	23.24	169.99	227.98	397.97	-	397.97
	30.00	24.24	175.23	237.79	413.03	-	413.03
	31.00	25.24	180.48	247.60	428.08	-	428.08
	32.00	26.24	185.72	257.41	443.14	-	443.14
	33.00	27.24	190.97	267.22	458.19	-	458.19
	34.00	28.24	196.21	277.03	473.25	-	473.25
	35.00	29.24	201.46	286.84	488.30	-	488.30
	36.00	30.24	206.70	296.65	503.36	-	503.36
	37.00	31.24	211.95	306.46	518.41	-	518.41
	38.00	32.24	217.19	316.27	533.47	-	533.47
	39.00	33.24	222.44	326.08	548.52	-	548.52
	40.00	34.24	227.68	335.89	563.58	-	563.58
	41.00	35.24	232.93	345.70	578.63	-	578.63
	42.00	36.24	238.17	355.51	593.69	-	593.69
	43.00	37.24	243.42	365.32	608.74	-	608.74
	44.00	38.24	248.66	375.13	623.80	-	623.80
	45.00	39.24	253.91	384.94	638.85	-	638.85
	46.00	40.24	259.15	394.75	653.91	-	653.91
	47.00	41.24	264.40	404.56	668.96	-	668.96
	47.71	41.95	268.12	411.53	679.65	-	679.65
	47.72	41.96	268.17	411.63	679.80	-	679.80
	48.00	42.24	269.53	414.37	683.90	-	683.90
	49.00	43.24	274.37	424.18	698.56	-	698.56
	50.00	44.24	279.22	433.99	713.22	-	713.22
	51.00	45.24	284.07	443.80	727.87	-	727.87
	52.00	46.24	288.92	453.61	742.53	-	742.53
	53.00	47.24	293.76	463.42	757.19	-	757.19
	54.00	48.24	298.61	473.23	771.85	-	771.85
Broken Anhydrite	55.00	49.24	303.46	483.04	786.50	-	786.50
	56.00	50.24	308.31	492.85	801.16	-	801.16
K=0.25	57.00	51.24	313.15	502.66	815.82	-	815.82
γ_{sat}=29.2	58.00	52.24	318.00	512.47	830.48	-	830.48
	59.00	53.24	322.85	522.28	845.13	-	845.13
	60.00	54.24	327.70	532.09	859.79	-	859.79
	61.00	55.24	332.54	541.90	874.45	-	874.45
	62.00	56.24	337.39	551.71	889.11	-	889.11
	63.00	57.24	342.24	561.52	903.76	-	903.76
	64.00	58.24	347.09	571.33	918.42	-	918.42
	65.00	59.24	351.93	581.14	933.08	-	933.08
	66.00	60.24	356.78	590.95	947.74	-	947.74
	67.00	61.24	361.63	600.76	962.39	-	962.39
	68.00	62.24	366.48	610.57	977.05	-	977.05
	69.00	63.24	371.32	620.38	991.71	-	991.71
	70.00	64.24	376.17	630.19	1006.37	-	1006.37
	70.11	64.35	376.70	631.27	1007.96	-	1007.96
	70.12	64.36	0.00	631.37	631.37	-	631.37
Sound Anhydrite	71.00	65.24	0.00	640.00	640.00	-	640.00
K=0.0	72.00	66.24	0.00	649.81	649.81	-	649.81
γ_{sat}=29.2	73.00	67.24	0.00	659.62	659.62	-	659.62
	73.16	67.40	0.00	661.19	661.19	-	661.19

Project Title:

Design of a Deep Excavation using Ground Freezing Technology

Project Detail:

Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: JRT

Checked by:

Verified by:

Design Detail:

EARTH PRESSURE CALCULATIONS 2 of 2

$$t_{\min} = \frac{p\left(r + \frac{Bhp}{2}\right)}{\phi f'_c - p}$$

(ref: personal communication Method by Georges Bibollet, SNC-Lavalin)

t_{min} = minimum concrete thickness of lining (m)
r = Interior radius (m)
f'_c = 91 day compression resistance (kPa)
p = external hydrostatic pressure (kPa)
Bhp = 0.3 m
ϕ = 0.65 concrete resistance factor (NBCC 1995)

Project Title: Design of a Deep Excavation using Ground Freezing Technology		
Project Detail: Search for buried treasure; Location: Oak Island, Nova Scotia		
Designed by: JRT	Checked by:	Verified by:
Design Detail: Required thickness of Lining	1 of 3	

Statum	Distance from surface (m)	Effective Total Pressure (kPa)	Eff. Pressure w/ Factor of Safety (1.3)	tmin (mm)
Unsaturated Glacial Till	0.00	0.00	0.00	0.00
	1.00	16.24	21.11	11.39
	2.00	32.47	42.21	22.81
	3.00	48.71	63.32	34.26
K_0=0.5	4.00	64.95	84.43	45.72
γ_d=16.70 kN/m3	5.00	81.18	105.54	57.21
	5.76	93.52	121.58	65.96
	5.77	93.68	121.79	66.07
	6.00	97.42	126.64	68.72
	7.00	113.65	147.75	80.27
	8.00	129.89	168.86	91.83
	9.00	146.13	189.97	103.43
	10.00	162.36	211.07	115.04
	11.00	178.60	232.18	126.69
	11.93	193.70	251.81	137.54
	12.00	193.70	251.81	137.54
	13.00	193.70	251.81	137.54
	14.00	193.70	251.81	137.54
	15.00	193.70	251.81	137.54
	15.43	193.70	251.81	137.54
Saturated Glacial Till	16.00	202.26	262.94	143.70
	17.00	217.31	282.51	154.55
	18.00	232.37	302.08	165.43
K_0=0.5	19.00	247.42	321.65	176.32
γ_{sat}=20.3 kN/m3	20.00	262.48	341.22	187.24
	21.00	277.53	360.79	198.19
	22.00	292.59	380.37	209.15
	23.00	307.64	399.94	220.14
	24.00	322.70	419.51	231.15
	25.00	337.75	439.08	242.18
	26.00	352.81	458.65	253.23
	27.00	367.86	478.22	264.31
	28.00	382.92	497.79	275.41
	29.00	397.97	517.37	286.53
	30.00	413.03	536.94	297.68
	31.00	428.08	556.51	308.85
	32.00	443.14	576.08	320.04
	33.00	458.19	595.65	331.26
	34.00	473.25	615.22	342.50
	35.00	488.30	634.80	353.76
	36.00	503.36	654.37	365.04
	37.00	518.41	673.94	376.35
	38.00	533.47	693.51	387.69
	39.00	548.52	713.08	399.04
	40.00	563.58	732.65	410.42
	41.00	578.63	752.22	421.83
	42.00	593.69	771.80	433.25
	43.00	608.74	791.37	444.70
	44.00	623.80	810.94	456.18
	45.00	638.85	830.51	467.68
	46.00	653.91	850.08	479.20
	47.00	668.96	869.65	490.75
	47.71	679.66	883.55	498.96

Project Title:

Design of a Deep Excavation using Ground Freezing Technology

Project Detail:

Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: JRT

Checked by:

Verified by:

Design Detail: Required thickness of lining 2 of 3

Broken Anhydrite K=0.25 γ_{sat}=29.2	47.72	679.80	883.73	499.07
	48.00	683.90	889.07	502.23
	49.00	698.56	908.12	513.52
	50.00	713.22	927.18	524.83
	51.00	727.87	946.23	536.17
	52.00	742.53	965.29	547.53
	53.00	757.19	984.34	558.91
	54.00	771.85	1003.40	570.32
	55.00	786.50	1022.45	581.75
	56.00	801.16	1041.51	593.20
	57.00	815.82	1060.56	604.68
	58.00	830.48	1079.62	616.18
	59.00	845.13	1098.67	627.70
	60.00	859.79	1117.73	639.25
	61.00	874.45	1136.78	650.82
	62.00	889.11	1155.84	662.42
	63.00	903.76	1174.89	674.04
	64.00	918.42	1193.95	685.69
	65.00	933.08	1213.00	697.35
	66.00	947.74	1232.06	709.05
	67.00	962.39	1251.11	720.77
	68.00	977.05	1270.17	732.51
	69.00	991.71	1289.22	744.27
	70.00	1006.37	1308.27	756.07
	70.11	1007.98	1310.37	757.37
Sound Anhydrite K=0.0 γ_{sat}=29.2	70.12	631.37	820.78	481.86
	71.00	640.00	832.01	488.56
	72.00	649.81	844.76	476.07
	73.00	659.62	857.51	483.58
	73.16	661.19	859.55	484.79

Maximum value.

Project Title:
Design of a Deep Excavation using Ground Freezing Technology

Project Detail:
Search for buried treasure; Location: Oak Island, Nova Scotia

Designed by: JRT.
Checked by:
Verified by:

Design Detail: Required Thickness of Lining 3 of 3

try 15M @ 200mm c/c spacing.

2 faces x 5 bars x 200mm² = 2000mm² > 1200mm²

∴ ok

* to resist ovalisation due to non-uniform loading 20M bars @ 200 mm c/c spacing is chosen.

cover of 75mm is suggested in practice by Georges Bibelet.

N 10.6.1 states d_c > 50mm does not work well with 10.6.1 ∴ will not use eq 10-6

Reinforcement will be closed on vertical ends with horizontal U bars same diameter & spacing as principal bars.

Project Title: Design of a Deep Excavation using Ground Freezing Technology		
Project Detail: Search for buried treasure; Location: Oak Island, Nova Scotia		
Designed by: JRT	Checked by:	Verified by:
Design Detail: Reinforcement on pre cast segments 1 of 2		

Minimum Reinforcement

$A_s = 0.002 A_g$ (7.8.1)

$A_g = t \times 1m$ sections

$= 600mm^2 \times 1000mm^2 = 600000mm^2$

$A_s = 0.002(600000mm^2) = 1200mm^2$

* in each direction (horiz + vert) as per Georges Biballet for draft lining.

Spacing:

$\leq 200mm$ c/c (N7.8.2), (N10.6.2)

skin reinforcement for crack control

try 10M @ 200mm c/c spacing.

$(2 \text{ faces} \times 5 \text{ bars} \times 100mm^2) = 1000mm^2 < 1200mm^2$

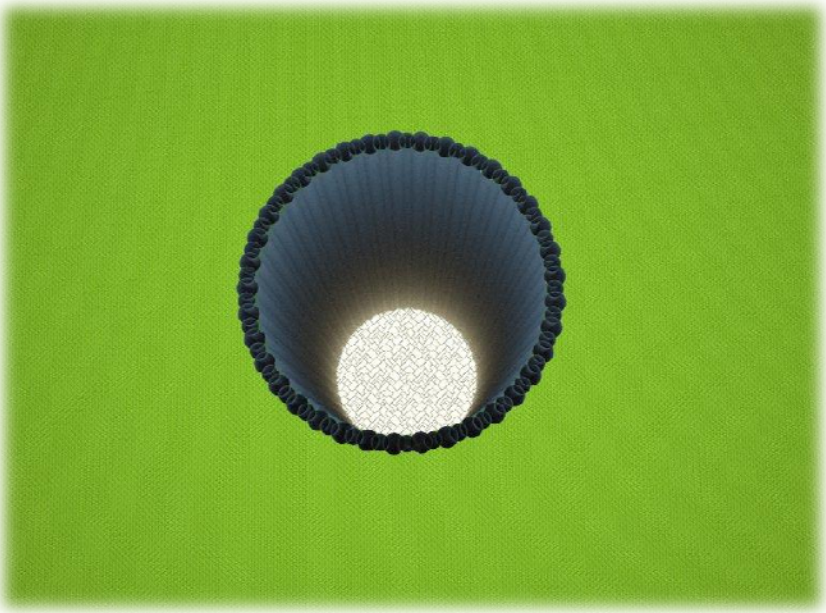
∴ not ok

Project Title: Design of a Deep Excavation using Ground Freezing Technology		
Project Detail: Search for buried treasure; Location: Oak Island, Nova Scotia		
Designed by: JRT	Checked by:	Verified by:
Design Detail: Reinforcement on precast segments		2 of 2

Topsoil SOLUTIONS

December 2008

Design of a Deep Shaft to Explore Underground Workings and Recover Potential Treasure on Oak Island, Nova Scotia



Designed by:

Christopher Ong Tone
Nathan Ramsey
Behnam Shayegan
Robert Wolofsky

Presented to:

Les MacPhie

Professor Colin Rogers

Professor William Taylor

ABSTRACT

Over a span of 200 years, millions of dollars have been spent in attempts to solve the great Mystery of Oak Island and the Money Pit. The evidence of man-made workings beneath the Money Pit has lead explorers to believe that a treasure rests somewhere deep inside the area. All past excavations have been foiled by the inflow of seawater to the Money Pit due to a hydraulic communication between the ocean and the highly fissured broken anhydrite layer at a depth below 150 feet.

In order to improve the constructability of a deep shaft, the soil along the perimeter will be treated with a grouting program to fill the voids, reduce the permeability and stabilize the soil. Low-mobility grout will be applied from a depth of 0 to 220 feet, and pressure grout from 220 to 290 feet.

A circular secant pile shaft has been designed to fully encompass all underground man-made workings, and potential treasure trove beneath the Money Pit. The shaft will have a diameter of 70 feet (21.3 m) and will consist of primary and secondary piles which are driven to a depth of 250 feet (76.2 m). There will be thirty-seven primary piles with a diameter of 4 feet (1.2 m) and thirty-seven secondary piles with a diameter of 5 feet (1.5 m). These piles will have a center-to-center spacing of 3 feet and will overlap and interlock to create an effective water-tight barrier and provide a 1 foot (0.3 m) minimum thickness. This thickness is adequate enough to withstand all lateral earth pressures beneath the surface, thus a permanent liner will not be required.

ACKNOWLEDGEMENTS

The completion of this design project could not have been possible without the technical and historical guidance provided to us by Les MacPhie of SNC-Lavalin. We would like to thank Mr. MacPhie for his dedication and support which has allowed us to produce a project to the best of our abilities.

We would also like to extend our appreciation to Eric Drooff, Andy Anderson, and Alan R. Ringen of Hayward Baker who were kind enough to meet and provide us with expert advice in the field. The solution to use a secant pile wall is based primarily on their recommendations.

We would also like to thank the following people for their time, guidance and recommendations:

- Professor William Taylor
- Professor Colin Rogers
- Professor Mohamed Meguid
- Professor Ferri Hassani
- D'Arcy O'Connor author of "The Secret of Oak Island"
- Danilo D'Aronco of DPHV
- Co-workers of Les MacPhie at SNC-Lavalin
- Cheehan Leung
- Miguel Nunes

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1.0 INTRODUCTION

Oak Island, Nova Scotia is the site of one of the world's most enduring and baffling mysteries. Since 1795, generations of searchers have tried, without success, to discover who created the underground workings on the island, why they were built, what they contain, and when the work was done. The investigations carried out over the years have identified, without a doubt, that original workings exist at a depth of 200 feet on the island (the Money Pit) and that they include an intricate flood system in the tidal zone which is connected to the Money Pit by a 500-foot-long tunnel.

Much research has been done in an attempt to identify the historical characters and incidents connected to the original Oak Island workings. This research has been conducted by many experienced historians, and many theories have been proposed based on circumstantial evidence, but no definitive connection has been made to Oak Island. This is very surprising.

The geological conditions at the site are reasonably well known, the configuration of the original underground workings are also well defined and the approximate time period of the workings is considered to be early 1600s to mid 1700s. However, two areas of research have not been rigorously pursued: the analysis of appropriate investigation techniques to further investigate the nature of the underground workings, and the development of engineering designs to excavate to a depth of about 200 feet to expose the sophisticated underground works. These topics are the subject of this design project.

2.0 HISTORICAL OVERVIEW

The following is a brief overview of the Oak Island Mystery which is sourced entirely from Harris and MacPhie (2005). For a detailed account of the story, reference can be made to the following: *The Secret Treasure of Oak Island* by D'Arcy O'Connor (2004) or *Oak Island and its Lost Treasure (Second edition)* by Graham Harris and Les MacPhie (2005).

In the summer of 1975, a young man named Daniel McGinnis discovered a clearing where newer growth of oak trees was present. At the center of the clearing stood an ancient oak tree from which dangled an old ship's tackle block suspended from a lopped-off limp 16 feet above the ground. The depression under the ground suggested that underground workings had been carried out on the island previously. McGinnis, being an adventurer, returned the following day with two friends, John Smith and Anthony Vaughan.

They boys discovered a previously dug pit of circular diameter of 13 feet. The soil within the pit was very loose and the hard clay walls were indented by marks of tools used by previous diggers. McGinnis and his friends stopped digging at 25 feet without finding anything of significant value.

In 1804, the Onslow Syndicate assembled by Simeon Lynds managed to excavate to a depth of 93 feet with considerable findings (Figure 1). Oak platforms embedded in hard clay were discovered at 10-foot intervals to a depth of 30 feet, in order to provide support for the backfill to the pit. Charcoal was discovered at 30 feet. It is speculated that the charcoal was used as fuel for underground furnaces in order to provide ventilation by inducing convection currents in the tunnel. Putty was discovered at 40 feet. Putty, being an excellent caulking material, was used for sealing and protection from leakages. Coconut fibre was discovered at 60 feet. It was commonly used as packing for ships' cargos. It is assumed that all three materials were used to facilitate the excavation.

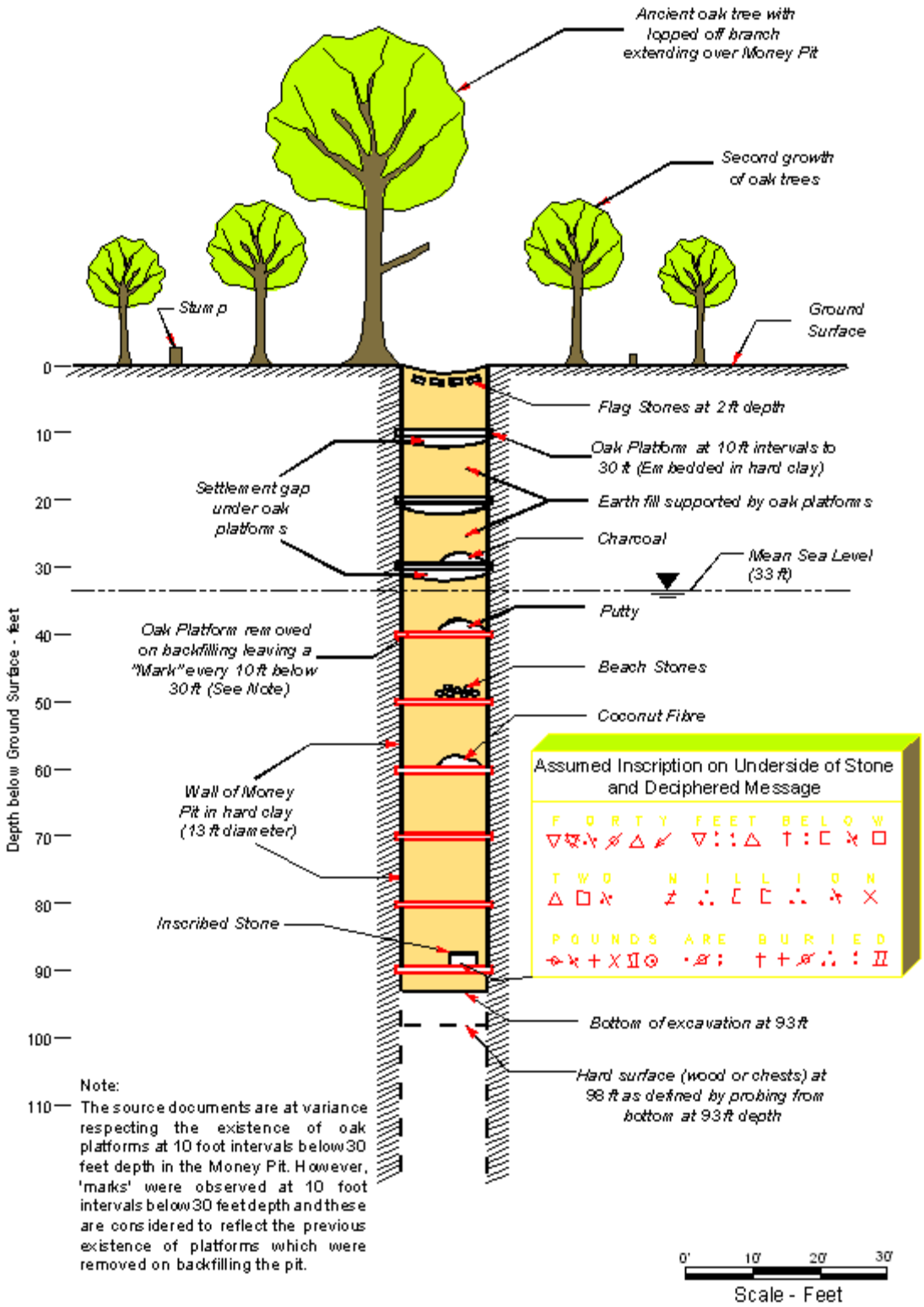

Figure 1 - Section of Money Pit based on findings by Onslow Syndicate in 1804 (Harris and MacPhie, 2005)

At 90 feet, a large stone slab (36 by 15 inches), weighing around 500 pounds was encountered, with supposed markings, the importance of which was disregarded. At a depth of 93 feet, the Onslow Syndicate encountered problems with gushing water through the shaft, labelled then as the 'Money Pit'. The shaft eventually filled with water to the sea level at 33 feet below ground surface.

Forty years passed before the Truro Syndicate, led by Dr. David Barnes Lynds, a former member of the Onslow syndicate, commenced work in 1849. The debris from the original Money Pit had been cleaned out and excavation started at a depth of 112 feet. The auger drilling executed by the Truro Syndicate hit assumed casks and oak chests at 100 feet below ground. These findings provided 'new heart to the digger's endeavors'.

In 1850, the Truro Syndicate constructed a 109-foot shaft, north-west of the Money Pit. They discovered that the water level in the shaft fluctuated in close relation to that of the oceanic tides and the water was found to be considerably saline. In search of further evidence, the Syndicate investigated an area of Smith's Cove covering 145 feet, where they discovered that the soil layer trickled more water than elsewhere on the beach. They excavated and discovered a 2-inch thick coconut layer underlain by 4 inches of decayed eel grass (Figure 2). Subsequently, a cofferdam was constructed to isolate the beach within the tidal zone and aid further investigation. Five well-constructed feeder drains were discovered, each converging towards a common center (Figure 3). According to Harris and MacPhie (2005), coconut and eel grass were placed to prevent the drains from clogging through migration of silt and sand. However, due to the flooding of the cofferdam as a result of unusually high tides, and the soft, water-logged nature of the ground, further investigations by the Syndicate came to an abrupt halt.

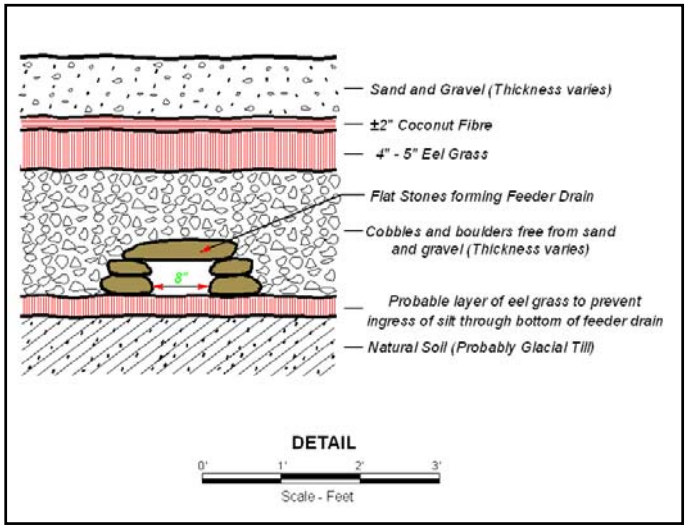

Figure 2 - Filter Bed at Smith's Cove Discovered by the Truro Syndicate in 1850 (Harris and MacPhie, 2005)

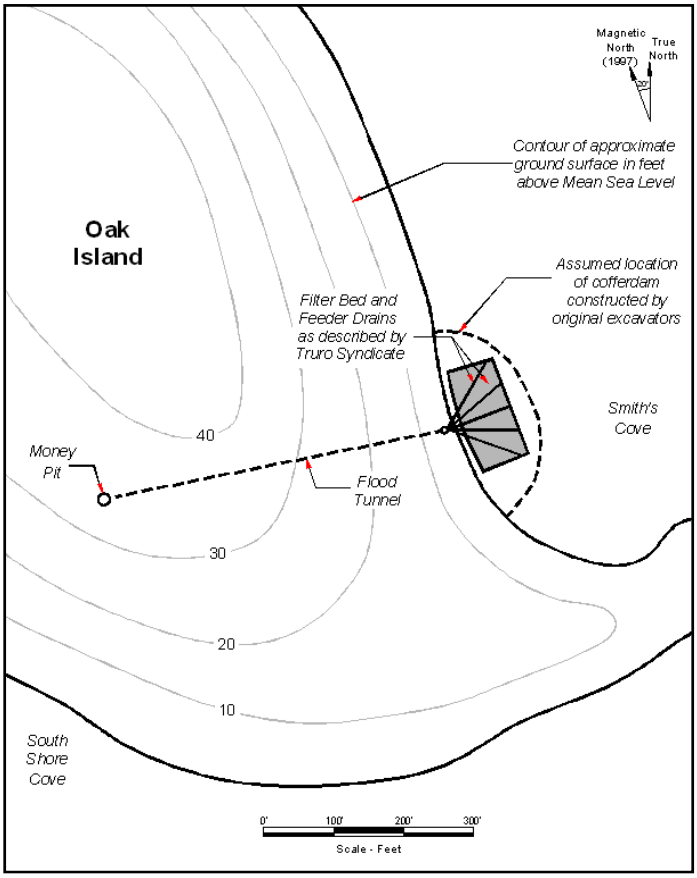

Figure 3 – Location of Filter Bed Discovered at Smith's Cove by the Truro Syndicate in 1850 (Harris and MacPhie, 2005)

From 1850 to 1866, several alliances including the Oak Island Association embarked on several unsuccessful endeavours that led to the collapse of the Money Pit and the loss of one man's life.

In 1866, the Halifax Company conducted an investigation in order to link the "Flood-Tunnel" with the Money Pit and the filter bed at Smith's Cove. They reported the tunnel dimensions as 2 ½ feet wide and 4 feet high, with an upward gradient of 22 ½ degrees at its intersection with the Money Pit. This finding highlights the remarkable endeavour that the miners carried out to execute such a complex system. In 1867, the Halifax Company halted its operations due to a lack of funding.

In 1895, the Oak Island Treasure Company was able to confirm the dimensions of the Flood Tunnel as reported by the Halifax Company in 1866. To further investigate, five borings were placed at 15 foot spacing, 50 feet from Smith's Cove (Figure 4). All borings were excavated to a depth of 80-95 feet with only the central boring striking salt water at a depth of 80 feet.

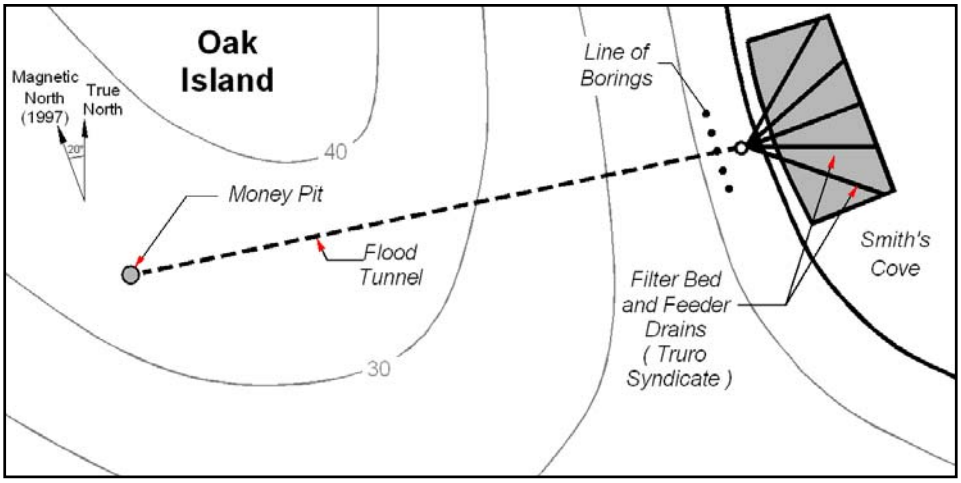

Figure 4 - Line of Borings Excavated to Locate Flood Tunnel
(Harris and MacPhie, 2005)

Subsequently, the central boring was met with rising water to the tide level at 33 feet. This finding was a clear indication that the Flood Tunnel was located directly below the central boring, acting as an intermediary between Smith Cove and the Money Pit. With the aim of cutting the supply to the Money Pit, sticks of dynamite were placed at the central boring location. It was reported that following the explosion, water in the Money Pit began to boil and foam. It

would seem that the Oak Island Treasure Company had solved the great mystery, but their joy was short-lived, as water continued to make its way to the Money Pit and flood the system after only a few hours.

In the same year, William Chappell and Frederick Blair (founder of Oak Island Treasure Company) attempted to determine the level to which the treasure chests had fallen in the 1861 collapse of the Money Pit. With the help of a working platform at 90 foot depth and control of water flow by pumping, they performed drilling and were able to extract a fragment of parchment (5/16th of an inch long). Also, the drilling suggested the presence of metal bars and coins surrounded by a cement vault at a depth of 153 to 160 feet. The searchers concluded that the treasure was encased with cement and that the Money Pit extended past 171 feet, where the drilling struck an iron obstruction, putting an end to the investigation.

In 1899, the Oak Island Treasure Company came across an interesting finding after attaining a depth of 160 feet in Shaft No. 18, located close to the Money Pit. As expected, water began to rush into Shaft No. 18, but this time water in the Money Pit, initially at 70 feet, fell 14 feet, and then rose back to its original level. This was a clear indication that the Money Pit was being fed by a lower source of water, perhaps a second Flood Tunnel.

During the period spanning 1899 to 1930, little activity took place on Oak Island and it was not until 1931 that William Chappell excavated what is now commonly referred to as the "Chappell Shaft". The shaft (located 6 feet from the Money Pit) was extended to 155 feet with the help of pumping wells, to control the water inflow. The following is a list of artifacts and rock formations discovered from 116 to 150 feet (depicted in Figure 5 and 6):

1. Anchor fluke embedded in the wall of a shaft at 116 feet depth
2. Granite boulder at 119 feet
3. Axe at 123 feet
4. Poll pick and remains of an oil lamp at 127 feet
5. Boulders from 150 to 155 feet

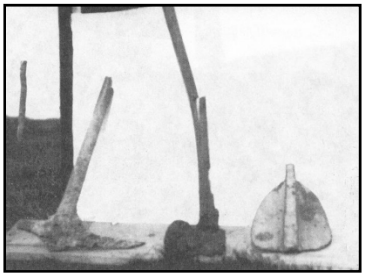

Figure 5 - Poll Pick, Felling Axe and Anchor Fluke found in 1931 in Chappell Shaft (Harris and MacPhie, 2005)

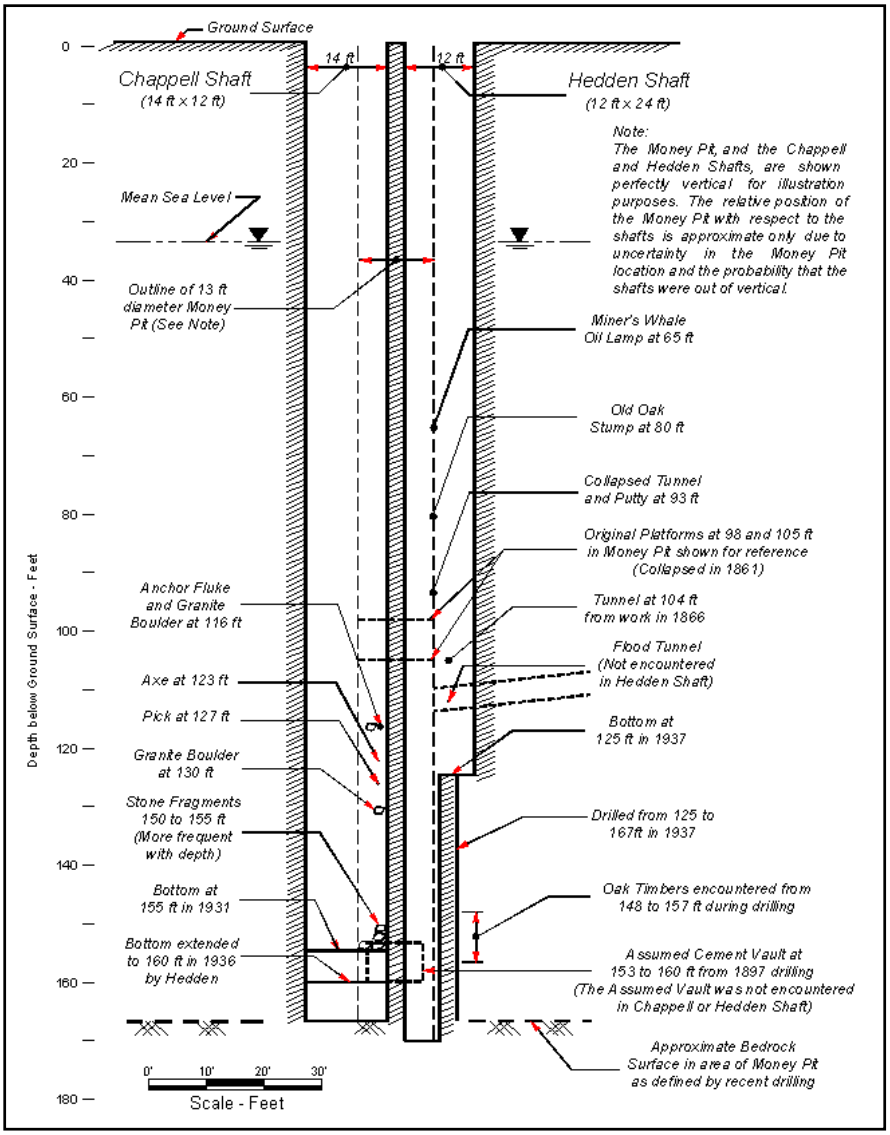

Figure 6 - Section of Chappell and Hedden Shafts Excavated from 1931 to 1941 (Harris and MacPhie, 2005)

The presence of these items could not have been due to the downward collapse of the Money Pit in 1861. The collapse would have caused a random arrangement of the equipment and strata, certainly not what was observed during the excavation in 1931. This occurrence was the work of the original excavators, and the findings were their inventory. In 1933, William Chappell ended his affiliation at Oak Island due to insufficient funding.

The next tragedy at Oak Island occurred in 1962 when Robert Restall, an Ontario native, and his family of 4 succumbed to their death as carbon monoxide fumes from dewatering pumps engulfed their family.

In 1965, a geologist from California, Robert Dunfield, led his team to what was the most destructive recovery program conducted on Oak Island. Dunfield and his team are responsible for the digging of the 'glory hole' at the Money Pit, which would eventually become 100 feet wide at the top and 135 feet deep. The 'glory hole' imposed extensive damage to the Chappell Shaft, causing it to collapse; nothing of significant importance was ever discovered. To this date, the scars of Dunfield's effort are vividly present in the form of a large open excavation at the cave-in pit.

In 1966, the rights of the land at the Money Pit changed hands to David Tobias (a Montreal-based businessman) and Dan Blankenship (a contractor from Florida). In 1967, Tobias sponsored drilling of 45 deep holes using a Becker drill type. This endeavour, along with two others (Warnock Hersey in 1969 and Golder Associates in 1970) is covered in detail in Section 3.3 of this report. In 1969, David Tobias organized the formation of Triton Alliance Limited to continue the search.

Blankenship is the mastermind behind the famous Borehole 10X. In 1969, with the help of a rotary drilling program, he constructed Borehole 10X, 180 feet north-east of the Money Pit. Cavities and loose soils were discovered 50 feet below the bedrock and during the cleaning process, and metal fragments were recovered at a depth of 165 feet. Because of difficulties with dewatering pumps, drivers were sent down along with underwater television cameras, however; due to poor visibility, their efforts proved unsuccessful in finding concrete evidence of man-made workings. In 1976, Blankenship narrowly escaped death following the collapse of the 27-inch diameter casing of Borehole 10X.

Subsequently, with the objective of isolating the filter bed at Smith's Cove, Triton Alliance constructed a cofferdam in 1970. This cofferdam was further offshore than the previous cofferdams and its existence led to many interesting findings that were later associated with the original excavators. A man-made heart shaped stone, a pair of wrought iron scissors, a wooden sled, part of a wrought iron ruler and various iron tools were discovered underlying an elaborate U shaped timber structure. Later that year, the cofferdam gave way to a severe storm and the filter bed and radial drains were not identified.

In 1987, Triton Alliance made plans for an 80-foot diameter lined shaft at the Money Pit. The project, labelled as the 'The Big Dig' proposed to have a shaft extended to 220 feet. Water would be pumped from four pumping stations to impede the water inflow. The plan also included cofferdams at Smith's and South Shore Coves to prevent water from entering the 'Primary' and 'Secondary' Flood tunnels respectively. Much to the disappointment of Tobias, the proposed $10 million 'Big Dig' did not proceed due to lack of funding.

The existence of a Flood Tunnel linking the Money Pit to Smith's Cove and the presence of iron and wood-lined cavities within the bedrock at the Money Pit provide evidence and clues to the massive undertaking at Oak Island. However, after more than 200 years of exploration and recovery attempts by treasure seekers, the existence of treasure is yet to be verified. The Mystery of Oak Island lives on.

3.0 PRELIMINARY INVESTIGATION

3.1 OBJECTIVES

This project phase consists of a borehole exploration program designed specifically to uncover archaeological and geological information of the site subsurface conditions. The initial objective of this program is to verify the presence of man-made workings that can be directly attributed to the activities of the Original Depositors. The ultimate objective of the investigation is to delineate the region of archaeological workings and the location(s) of any artifacts of considerable value. In this regard, the intention is to provide optimal direction in the planning, location and sizing of a deep shaft excavation to facilitate the recovery of artifacts and to identify the historical and archaeological context of the site.

3.2 PAST INVESTIGATIONS

From 1795 to present, numerous excavation and exploration programs have been undertaken in an attempt to solve the Mystery of Oak Island. Prior to 1965, exploration programs extended to a maximum depth of 155 feet below existing surface level. Evidence of man-made workings and artifacts was detected, the most significant being inferred treasure chests (decoy treasure) at depths of 90 and 150 feet by drilling attempts of the mid-to-late 19th century. However, subsequent attempts to recover the chests have been marked by failure (Harris and MacPhie, 2005).

Since 1965, exploration efforts have surpassed the 155 foot horizon, and have uncovered evidence of man-made workings at depths exceeding 190 feet below the existing surface level. It should be noted that since then, no exploration or recovery attempts had reached such depths. It can be argued with considerable certainty that such evidence can be attributed to the activities of the Original Depositors (MacPhie, 2001).

3.3 KEY ARCHAEOLOGICAL FINDINGS

Regarding key archaeological evidence, the most significant data was obtained from recent drilling programs conducted by Becker Drilling Inc. (1967), Warnock Hersey (1969) and Golder

Associates (1970), and a summary of this data is given in MacPhie (2001). Additionally, substantial geotechnical data was obtained from the latter two of these programs. A fourth program worth mentioning is the Oak Island Detection Program which was conducted in 1993-1994. This program provided information on the phenomenon of lateral drift which occurs during drilling and is defined as the lateral deviation of the drill track from a vertical path (MacPhie, 2001). Table 1 provides a summary of the key archaeological findings which resulted from all of the aforementioned programs.

Table 1 - Key Archaeological Findings from Recent Drilling Programs at Money Pit[1]

Borehole ID No.	Description of Key Archaeological Findings	Depth of Findings (ft)
Becker 11	Puddled clay (from 184-200 ft depth). Two oak buds found embedded in clay (at 196 ft depth).	184-200
Becker 13	Puddled clay.	184-200
Becker 14	Puddled clay.	184-200
Becker 17	Puddled clay.	176-198
Becker 21	Puddled clay (from 176-198 ft depth). Piece of slightly crumpled brass foil (at or above 176 ft depth). Stagnant water and possible cavity in rock (from 200-206 ft depths).	176-206
Becker 24	4 inches of wood (at 192 and 193 ft depth with clay between wood layers). 6 ft high water filled cavity in bedrock, inferred as man-made chamber (from 193-199 ft depths)	192-199
Becker 25	7 ft high cavity in bedrock inferred as chamber. ½ thick iron plate at base of cavity (at 198 ft depth).	191-198
Becker 33	Puddled clay (from 190-192 ft depths). Bedrock cavity with soil and lime mortar, inferred as chamber (from 192-198 ft depths). Wood at roof of cavity (at 192 ft depth).	190-198
Becker 35	6-8 inches of very rotten wood (at 179 ft depth). 12 ft high bedrock cavity with old wood, charcoal and clinker.	178-190
Becker 40	Puddled clay.	175-195
Warnock Hersey 9	Wood. Stagnant water and possible 14 ft high disturbed zone.	192-206
Golder 103	Bedrock cavity inferred as chamber. Reworked recent soil.	191-198

[1] MacPhie (2008a)

Based on the above tabulated data and associated borehole records, an interpretive sketch of a section through and around the Money Pit showcasing the key archaeological findings has been rendered (see Figure 7).

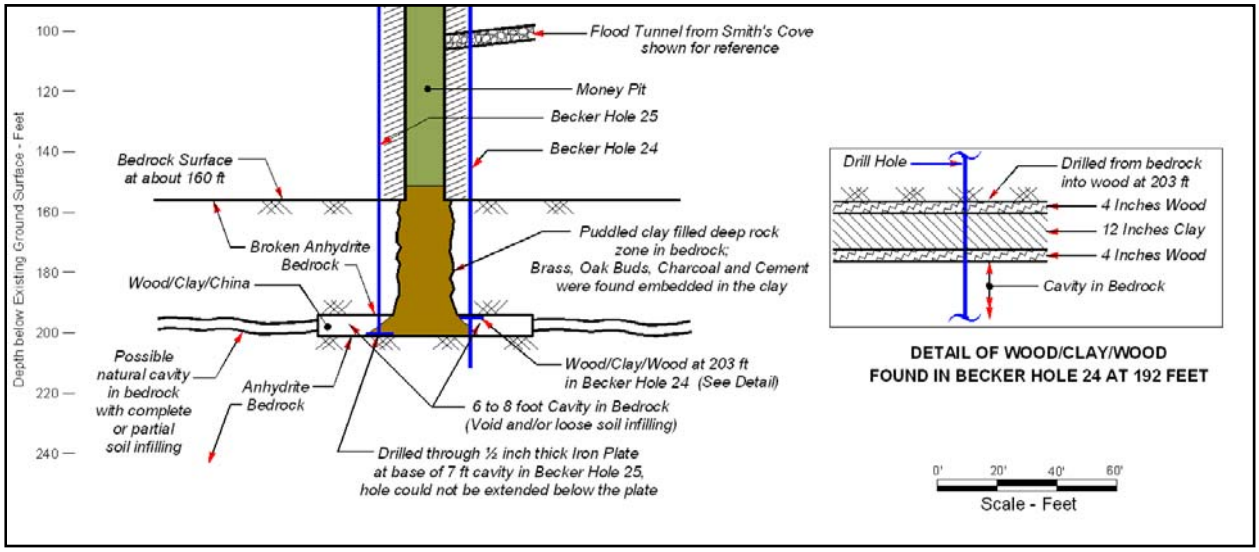

Figure 7 - Section through Money Pit illustrating key findings from drilling efforts (Harris and MacPhie, 2005)

3.3.1 BECKER DRILLING PROGRAM

The Becker Drilling Program, conducted in 1967, provided the first evidence of human activity, deep within the bedrock, between 190 and 200 feet below surface level (MacPhie, 2001). As mentioned above, this evidence can be directly attributed to the Original Depositors and thus proves their existence. The Program consisted of drilling forty-nine boreholes, forty of which were drilled in the vicinity of the Money Pit.

As can be seen from Table 1, ten of these Becker holes uncovered key archaeological features deep in the bedrock including: puddled clay, brass foil, wood, iron plate, bedrock cavities filled with stagnant water or reworked soil.

One significant finding is the localised deep bedrock zone (at approximately 200 foot depth) which was identified by eight boreholes. Since it has been established that in the area surrounding the Money Pit the depth to bedrock is generally 155 ft, these holes have identified a

zone of deep rock overlain primarily by clay (see Figure 1). This clay could be backfill to the man-made excavation by the Original Depositors (MacPhie, 2008a). The term "puddled clay" is used to describe this zone since coarser pebbles were found at regular intervals (MacPhie, 2008a). According to MacPhie (2008a), this deep rock zone is a critical archaeological feature, as it is most explicably the base to a shaft excavation, which was subsequently backfilled with puddled clay to act as a sealant to underground chambers.

It should be noted that any bedrock cavities which contain evidence of man-made workings are believed to originate as a lateral excavation from the base of the deep excavation (MacPhie, 2001) and will henceforth be referred to as chambers.

Two other significant findings were the timber platform found on the roof of two chambers and the iron plate found on the base in Becker holes 24 33, and 25 respectively (see Figure 1). MacPhie (2001) postulates that the iron plate may have formed the cover of a vault residing below the floor of the chamber. He also theorises that the timber roofing (as seen in the Detail in Figure 1) was placed as a precautionary measure by the Original Depositors to prevent the possible collapse of the chamber and to reduce water inflow.

3.3.2 Warnock Hersey Drilling Program

The Warnock Hersey Drilling Program of 1969 (Warnock Hersey, 1969) consisted of ten boreholes, five of which were placed in the vicinity of the Money Pit. Significant findings were encountered in the ninth hole of the program. As reported by MacPhie (2001), from 192 to 196 foot depths, wood chips were recovered in the drilling core barrel, which could have been the result of drilling alongside a wood-lined chamber, or from penetrating a solid wooden slab. Additionally, at a depth of 200 feet, a six-foot high cavity (possibly a chamber) filled with stagnant water and traces of clay were encountered. Deep rock, at 206 feet, was also reported at this location.

It should be noted that the eighth hole of the Program was drilled at the location of Becker hole 24 in an attempt to further explore the cavity with the wooden support previously found at a depth of 192 feet. Borehole results report that the hole was terminated at a depth 200.5 feet without encountering rock or any of the targeted findings from the Becker hole (MacPhie, 2001). It is reported that at the termination depth, soft soil was encountered, which was most probably

the same soil backfill to a deep shaft excavation that was discovered in the Becker Drilling Program.

Deep rock at 200.5, 206 and 212 feet was identified in the second, eighth and ninth holes of the program. The evidence of deep rock is assumed to be pinpointing the same deep shaft bottom which was identified in the Becker Program.

3.3.3 Golder Drilling Program

The Golder Drilling Program, conducted in 1970, involved the drilling of eight holes, two of which were in the immediate vicinity of the Money Pit. One significant archaeological finding resulted from this investigation. Namely, in Golder hole 103, soil recovered from split spoon sampling at depths ranging from 192 to 198 feet was determined to have originated from the surface by pollen count analysis (Ritchie, 1970). This is noteworthy since this borehole, placed 3 feet of the Money Pit, encountered soil in the deep rock zone that, being traced to a surface origin, is most explicably soil backfill laid down by the Original Depositors (MacPhie, 2008a).

3.4 Associated Problems

3.4.1 Lateral Drift

The lateral drift phenomenon is an issue which affects the accurate interpretation of archaeological evidence of past investigations. Save for the five holes of the Oak Island Detection Program, and Warnock Hersey hole no. 8, no lateral drift measurements were performed on past explorations (MacPhie, 2001). This is a cause for concern since it means that the actual tracks of these holes are unknown, and so is the actual position and depths of the associated findings. From the lateral drift measurements made on the aforementioned boreholes, the most extreme case of drift onsite was reported as 15 feet at a depth of 190 feet (MacPhie, 2001); which equates to 0.08 feet lateral drift/foot depth. As MacPhie (2001) further reports, all depth measurements quoted from such borehole records are based on the assumption of perfect linearity in the direction in which the holes were started (except for Becker holes 14, 16, 17, 22, and 23, all holes were commenced vertically).

Such a phenomenon implies that a finding quoted at depth of 200 feet can be positioned anywhere within a 16 foot diameter field in the horizontal plane. Also, due to trigonometric

relationships, a depth measurement of 200 feet subject to the worst-case lateral drift quoted would be equivalent to an actual vertical depth of 199 feet and 5 inches; 7 inches less than the measured value.

It cannot be overstressed that when interpreting past exploratory results and when planning future follow-up investigations, such inaccuracies must be given due consideration.

3.4.2 Ground Elevation Measurements

Ground surface elevations were not measured in the Becker Drilling Program. Consequently, it should be noted that the combined effect of an undulating ground surface and lateral drift could result in depth measurement inaccuracies of up to several feet in the area of the Money Pit (MacPhie, 2001). This has an effect on the interpretation of lateral continuities of archaeological features from the borehole data. In other words, it is possible that the same archaeological feature could be encountered in Becker holes at what appears to be different depths and locations and consequently misinterpreted as separate features.

3.5 Investigation Targets

Based on the key archaeological evidence presented in Section 3.3, several targets are proposed for the Preliminary Investigation Program; the possible cache of treasure or valuable artifacts being the prime target.

3.5.1 Target A: Chamber(s) with Wood Roofing

The wood-roofed chambers identified by Becker holes 24 and 33 from 192 ft to 199 ft depths constitute this target. These holes are spaced 7 feet apart at surface level and are located along the southern edge of the deep rock zone. MacPhie (2001) notes that it is possible that these two boreholes have identified the same chamber as the depth and height of the two findings, which are both timber-roofed, correspond closely. However, although the approximate depths are known, the exact locations in plan are unknown since the lateral drift of these holes was never measured.

3.5.2 TARGET B: CHAMBER WITH IRON PLATE FLOORING

The iron plate intersected by Becker hole 25 is an obvious target for investigation as it would be of interest to determine what lies beneath it. This borehole is located approximately 17 ft to the northwest of Becker hole 24, and it can be only speculated whether the findings in both holes are related. One interesting point which would suggest a relation is that the chambers identified in the holes 24 and 25 (and 33) were all at depths from 190 to 200 ft below surface.

3.5.3 TARGET C: DEEP ROCK ZONE DELINEATION

This target is aimed at further investigating the deep rock zone (rock at 200 ft depth) identified in eleven boreholes from the Becker and Warnock Hersey Drilling Programs. As previously mentioned, this zone is essentially a soil-filled man-made depression below the bedrock surface. A spatial analysis conducted by MacPhie (2001) determined that only the south and west boundaries of the depression have been adequately defined (see *Plan of Past Investigation Holes* in Appendix C). The centroid of the data points is located 10 feet to the north of Becker hole 24, and the available data suggests that the deep rock depression is circular, roughly 16 feet in diameter. This analysis takes into account uncertainties posed by lateral drift and is based on the assumption that Becker holes 14, 17, 21 and 23, which were drilled with a starting inclination, maintained such an inclination throughout.

The complete delineation of the deep rock zone is of great archaeological importance as it would map the extent of the deep shaft excavation of the Original Depositors. It is believed that chambers possibly housing treasures or valuable objects are offset horizontally from this depression. Detailed investigation to the north and east of the inferred circular zone is thus an important target for this investigation.

3.5.4 TARGET D: PUDDLED CLAY

The puddled clay is being targeted because of its significance as an impervious backfill to the deep rock depression (referred to in Target C). The clay was encountered in seven Becker holes, meaning there was a high probability of success of intersecting it in future drilling attempts. The aim is to determine whether the clay is of man-made or geological origin.

3.6 Considered Methods

3.6.1 Cross Hole Geophysical Surveys

Cross hole geophysical surveys are conducted by measuring the propagation velocity of seismic waves which are transmitted between two cased boreholes (CDA, 2008). Seismic velocities depend on the properties of the propagation medium. Therefore, such technology is employed to define the nature and profile of the underground features which reside between the boreholes; whether it is composed of soil deposits, rock formations, or man-made materials (CDA, 2008).

According to CDA (2008), the test apparatus consists of one cased borehole housing an energy source at a particular depth, and another cased hole housing a geophone sensor (see Figure 8). The time it takes for the seismic rays to propagate from the source to the sensor, as well as the length of the ray paths are recorded and used to ascertain the seismic velocities of the rays (which include both compression and shear waves).

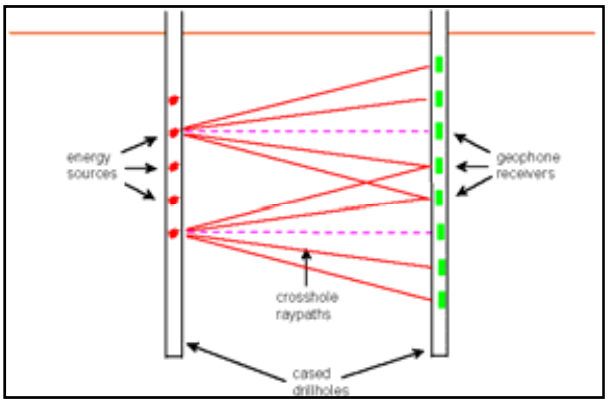

Figure 8 - Schematic of Cross Hole Geophysical Mapping (CDA, 2008)

However, according to MacPhie (2001), this method was employed extensively in the Oak Island Detection Program of 1992-1994 and generated results with insufficient detail for examination of underground features in and around the Money Pit. Therefore this method was not selected for the proposed investigation.

3.6.2 PERCUSSION DRILLING

Also referred to as Churn drilling, percussion drilling operates by the swift and repetitive raising and dropping of a heavy bit to break-up materials and form a slurry which is periodically removed from the bottom of the hole with a bailer or sand pump (CGS, 2006).

Even though it is a relatively economical method for making large-diameter (greater than 2 feet [600 mm]) holes in any material, problems have been reported when drilling under water (Kutzner, 1996). Furthermore, Hunt (2005) notes that rock coring and undisturbed sampling is not possible by this method; slow progress in strong soils and rock is typical, so percussion drilling would be unsuitable for core investigation through the anhydrite bedrock. Moreover, in comparison to solid-bit rotary drilling, percussion drilling leads to increased damage to the borehole (Kutzner, 1996).

3.6.3 AUGER BORING

Auger boring involves the rotational driving of a helical drill shaft into substrata and may involve the periodic removal of ground material or a continuous flight operation (CGS, 2006). For the latter, the helical flight acts as a conveyor to guide the material to the surface, and is the most commonly employed soil boring method employed in North America and Europe at present (Bowles, 1996). The Augers may be solid or hollow-tubed; with the latter providing the advantage of facilitating sampling and in-situ testing through the hollow stem (Bowles, 1996). Continuous-flight augers provide an economic advantage: they can be advanced in one rapid, continuous flight, and do not require the employment of casings for hole stability, as seen in Figure 9 (Hunt, 2005; Bowles, 1996).

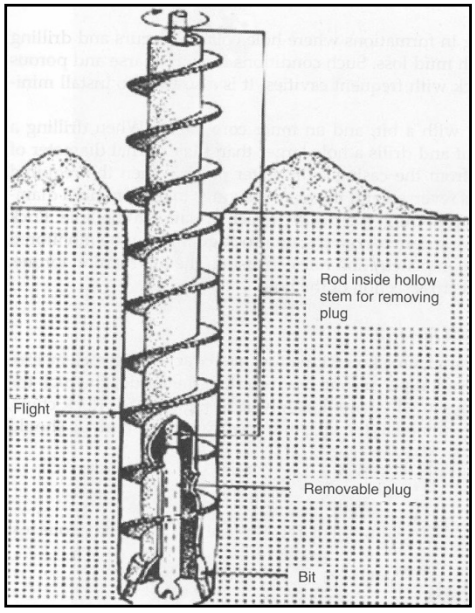

Figure 9 - Continuous Flight Hollow-stem Auger (Hunt, 2005)

However, as noted Hunt (2005), it is not suitable for penetration through boulder or rock formations, or for weak cohesive or granular soils at depths greater than a few feet below the water table. The author further states that advancement through firm cohesive soils or gravel to considerable depths is typically difficult with this method. For these reasons, this method was deemed unsuitable for this project.

3.6.4 ROTARY DRILLING

Rotary drilling is a common method used to advance boreholes in all material types. It operates by advancing a cutting bit which is fastened to the end of a power-driven rotating drill rod (CGS, 2006). Flushing media, either compressed air, water or both, is circulated down inside the rods and upwards through the annular space surrounding the rods to transport the cuttings from the base of the hole to the surface (see Figure 10) (Kutzner, 1996).

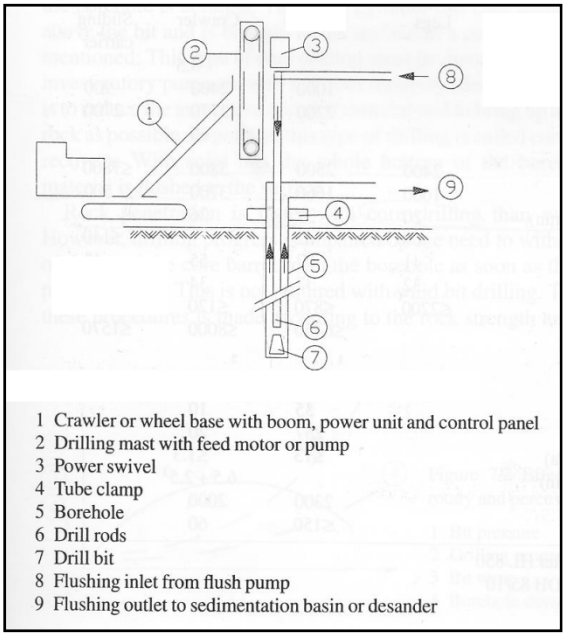

Figure 10 - Components of Rotary Drilling Procedure (Kutzner, 1996)

Rotary drilling consists of solid-bit drilling or core drilling. The latter is most commonly used to penetrate through rock (Kutzner, 1996). With solid-bit drilling, the entire bottom face of the borehole is subject to drilling abrasion and subgrade material is retained in the form of cuttings for material classification. Core drilling, on the other hand, only abrades an outer cylinder on the hole bottom; the core portion is retained within the confines of the hollow tube (Kutzner, 1996).

Rotary drilling can penetrate all types of geologic material at relatively rapid rates and at great depths (Hunt, 2005). Solid-bit drilling is suitable for all cohesive and granular soils, and core drilling equipped with diamond core bits is suitable for obtaining core samples of any rock formation (Kutzner, 1996). The quality of the recovered core depends on the coring system, but it is generally accepted that with double or triple tube samplers, the core samples are nearly intact (Kutzner, 1996). The author further notes that with respect to lateral drift (assuming that skilled personnel and up-to-date equipment are employed), rotary drilling is more accurate than percussion drilling. Specifically, in terms of penetration depths, percussion drilling is reported to drift ≤ 5%, whereas solid-bit and rotary core drilling are subject deviations of ≤ 3% and ≤ 1% respectively.

It is clear that this drilling method (solid-bit and core boring) meet the drilling requirements for the proposed investigation; namely, advancement through the till overburden with controlled lateral deviation, and core sampling (with diamond core bits) through rock to considerable depths (150-200 ft) with heightened sensitivity to definition of the profile penetrated.

3.6.5 WIRE-LINE DRILLING

Wire-line drilling is a special type of core drilling developed to eliminate the need to repeatedly withdraw and re-install the string of rods to recover each barrel of core (Hunt, 2005). Also, the drill rods serve as borehole casing, and thus the advancement of the casing is not a separate operation to the advancement of the drilling apparatus (as is the case for conventional coring procedures) (Hunt, 2005). It is thus an extremely efficient method for deep core drilling.

The drill string and bit are advanced by power rotation as the drill fluid is pumped through it to cool down drill bit and provide lubrication (Hunt, 2005). As the core barrel is advanced and the inner tube is filled with rock core an overshoot is lowered by the wire-line, through the drill rod and fastened to the upper end of the inner core tube (see Figure 7.17). Upon retrieval of the wire-line the inner core tube and core sample is recovered. The inner tube is then lowered using the wire-line winch and it is re-assembled with the core barrel so that coring can be further advanced (Hunt, 2005).

Figure 11 - Wireline Core Barrel
(Kutzner, 1996)

According to Kutzner (1996), this is the most accurate drilling method since lateral deviations are reported as being only ≤ 0.5% of the borehole depth (Kutzner, 1996). CGS (2006) and Kutzner (1996) further note that for boreholes over 100 feet (30 m) in length on land, the added investment costs required for the operation of this method is compensated for by the accelerated drilling progress.

The pinpoint accuracy and efficiency of this method make it ideal for its application in the proposed preliminary investigation. It will be used for situations where specific targets have been intersected and the verticality in follow-up drilling must be ensured (MacPhie, 2008b).

3.7 INVESTIGATION APPROACH

The preliminary investigation is aimed at exploring all of the prescribed targets listed above. It is designed with regards to the nature and inferred location of the underground workings, as well as the subsurface geologic environment. As such, drilling methods and sampling procedures have been selected based on their suitability for this exploration. With the aim of optimizing

resource efficiency, various boring classes (Type I, II and III) are proposed for the investigation (MacPhie, 2008b).

Based on the results of past drilling expeditions (Section 3.3), it is known with considerable certainty that underground workings of interest are located at depths greater than 190 feet (58 m) and up to 220 feet (67 m) below the surface. This corresponds to the lower segment of the broken anhydrite stratum and a portion of the competent bedrock. The glacial till layer, at depths between 0 and 155 feet (0 to 47 m), may contain objects of archaeological significance. However, their locations and depths are not well defined. Therefore, the focus of sampling and core recovery will be in the underlying rock strata, although any incidental hits in the overburden will of course be followed up.

The drilling approach will be to sink borings in an expedient manner down to bedrock at 155 feet (47 m). No coring or sampling is proposed for this overburden region, but a control of lateral drift is required. The methodology for boring in the bedrock will depend on the boring class, for which detailed specifications are provided in Section 3.10.

The rationale for employment of boring classes is based on a progressive findings approach (MacPhie, 2008a). The initial holes (Type I holes) are placed and vertically advanced to intersect the targets specified above. The outcome of the initial borings will inform the decision making process to optimize the chances of success. Namely, if the Type 1 hole fails to intersect the target, a maximum of three Type II holes are to be placed at a 5 foot lateral distance from the position reached by the borehole base at 200 foot depth. It is proposed that the Type II borings be placed at the vertices of an equilateral triangle inscribed in a 10 foot diameter circle, which is centred on the surface projection of the base of the Type I borehole. Type II holes are essentially the same as Type I, but have been given a separate classification in order to simplify the decision making process. However, if the target is intersected by the Type I hole, it is suggested that a maximum of two Type III holes be sunk at a 2 foot lateral distance from the surface projection of the position of the drill bit at target intersection in order to define the extent and nature of the hit. Type III borings call for the most stringent control of later deviation as the position of the specified target has been pinpointed. This methodology gives further direction as to the appropriate course of action for the recovery of a given target (Figure 12). The above program for Type II and III boreholes is a guide to indicate the scope of drilling which may be required.

The actual program beyond the initial few Type I boreholes will of course be dictated by progress findings.

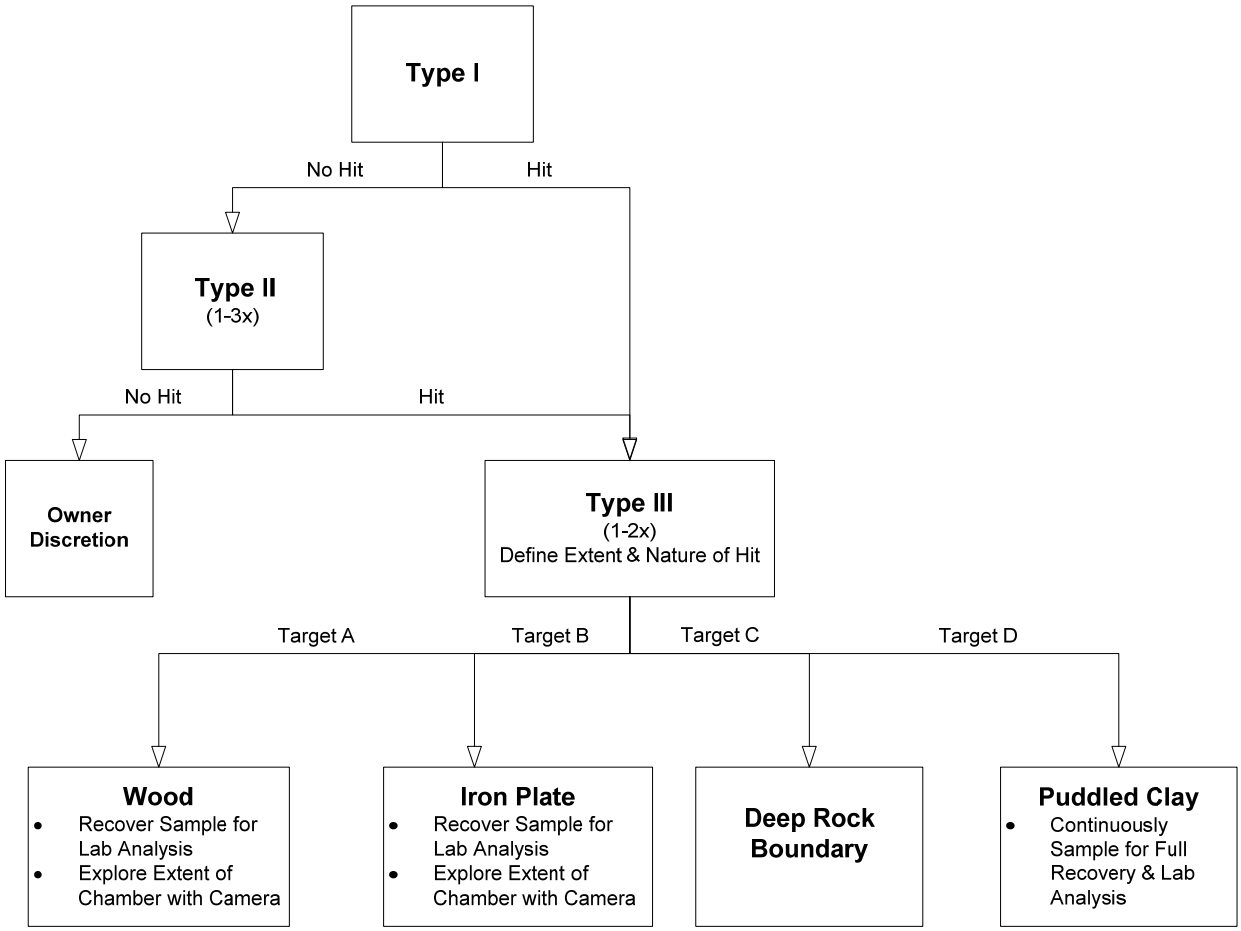

Figure 12 - Schematic of the Proposed Boring Methodology

3.8 INITIAL HOLE LOCATIONS

The initial hole locations have all been selected based on the positions of the identified targets. The locations are not structured in a grid as was advised by CGS (2006) to avoid rigid, preconceived borings patterns. Rather, the pattern of subsequent borehole placement will be guided by the findings of the Type I holes. Considerable discretion must be taken by the Owner (or his representative) to place Type II and III holes to maximise the chances for success.

Plan of Initial Holes for Preliminary Investigation in Appendix C shows a plan of the proposed initial hole placements. It should be noted that the proposed drillings for Targets A and B are to

be placed directly over the hole location of the past boring which previously intersected the target in question with the hope of following the same borehole trajectory. This method should maximise the chances for success.

As can be seen from Figure 1, nine initial borings are proposed. For Target A, borings TA1 and TA2 are proposed to be drilled directly over Becker holes 24 and 33 respectively with the hope of advancing along the same path. For Target B, one boring is proposed, TB1, which will be sunk at the previous location of Becker hole 25. Target C is to be explored with holes TC1, 2 & 3, and Target D with borings TD1, 2 & 3.

3.9 PREPARATORY SURVEY

Before drilling operations begin, a local survey grid must be set-up onsite (MacPhie, 2001). This is a necessary step as the grid will be used to accurately position the hole locations onsite, as specified on plan. Also, it will facilitate the accurate documentation of the positions of all new borings, as well as any associated lateral drift with reference to the established grid system. MacPhie (2001) proposes the use of surface position of Becker hole 24 as the origin on the grid, which will be allocated the arbitrary coordinates of 1000 feet North and 4000m feet East. This same grid system will be used for this preliminary investigation in order to maintain consistency between past investigation documentation and the proposed Program. It should be noted that the documented positions of all past borings, as per MacPhie (2008a), were scaled-off known onsite features and plotted on plan with reference to the stated local grid.

Also, as recommended by MacPhie (2001), all elevations of the proposed borings must be referenced from a Geodetic (mean sea-level) datum which is to be obtained from the closest available Geodetic benchmark.

3.10 DRILLING SPECIFICATIONS

All borings are to be advanced with an initial vertical orientation. Magnitude and direction of lateral drift is to be measured at regular intervals throughout the entire length of each boring. It is suggested that a surface recording gyro guidance system (or equivalent System) be employed to calculate the offset measurements. The system should be capable of detecting lateral

deviations to an accuracy of several inches at a 200 foot (61 m) depth (MacPhie, 2001). At the specified reading interval, boring operations are to be brought to a halt, and the sensor is to be inserted into the drilling rods to the required elevation and the reading performed. Boring operations can only resume once the system has been removed. It is important that the system is not affected by any ferromagnetic anomalies that are likely to be present in the area (MacPhie, 2001).

It is being left to the discretion of the Contractor what precise procedures are adopted to meet the lateral drift drilling requirements. There are several conventional methods used in industry to control drift: using a stiff drill string, or controlling the speed of advancement of the drill string, also known as a controlled advancement (MacPhie, 2008b).

As discussed in Section 3.7, all borings are to be executed in two phases. The first phase is common to all three classes, and will involve a solid-bit rotary drilling from the surface to a depth of 155 feet (47 m). HW casings, which are 4 inches in inner diameter and 4.5 inches in outer diameter (Hunt, 2005) are then installed in the solid bit rotary drilled hole. The hole is then continued with the HW casing and core barrel as discussed later. This large casing size was chosen in order to provide adequate room for telescoping to smaller casing sizes in case obstacles in the broken anhydrite are encountered.

Drilling operations for the second boring phase is specified in Sections 3.10.1 and 3.10.2 below. Borings in this phase are required to traverse from 155 feet (47 m) to a minimum depth of 220 feet (67 m) (unless a direct hit on presumed treasure chests occurs in higher elevations).

3.10.1 Type I and Type II borings

Type I and II borings are designed to function as exploratory drillings. As explained in Section 3.7 they are used either as an initial attempt to intersect a target (Type I) or as follow-up attempts to a 'miss' (or failure to intersect the target) by the Type I hole (Type II).

As an initial recommendation, lateral drift in Type I and II borings should be controlled within 6 feet (1.5 m) at a depth of 200 feet (47 m) (i.e. 3% allowable drift for a given depth).

Triple-tube wire-line rotary core drilling is specified for advancing these boring types from 155-220 feet (47 to 67 m). A coring system with a 2.4-inch (61.1 mm) minimum internal diameter

(HQ3 core barrel with HQ size core bit) must be used as this is a suitable spatial requirement for the borehole camera to be used for down-hole inspection. Triple-tubes are specified in order to ensure high quality rock core recoveries as such systems cause the least disturbance to cores (Kutzner, 1996).

Diamond-infused bits are to be used in order to obtain the necessary sensitivity to subsurface anomalies, as well as the strength and resilience required to advance through any bedrock formations encountered (Kutzner, 1996).

The magnitude and direction of lateral drift is to be measured at every 20-foot advancement of the borehole through the substrata, below 155 feet (47 m).

3.10.2 Type III Borings

Type III borings are used as verification drillings. They are advanced once a target has been intersected and will define and explore the nature of the hit.

Drilling specifications for Type III borings are the same as for Type I and II; the exception being a more stringent requirement for lateral drift control, the establishment of which is necessary in accordance with their function for follow-up exploration of hit.

As per the recommendation by MacPhie (2001), lateral drift in Type III borings is required to be controlled to within 2 feet (0.6 m) at a depth of 200 feet (47 m) (i.e. 1% allowable drift for a given depth).

3.11 Sampling & Exploration Specifications

Sampling and exploration are as critical to the investigation program as the driving of the exploratory boreholes. Depending on the nature of the soil or rock stratum which is being advanced through and on the nature of the target being sought after, specific sampling and exploratory procedures should be employed in order to generate useful results.

3.11.1 Split-Spoon Sampling

Split-spoon (also called split-barrel) samplers are used to obtain representative (undisturbed) samples of granular soils (Hunt, 2005). In order to fit through the HQ core barrel (after removal

of the inner tube), a 2-inch internal diameter, 0.25-inch wall thickness split-spoon sampler should be employed; thus producing 1.5-inch diameter samples with a length of 1.5 to 2 feet (Hunt, 2005).

This tool is to be used for continuous sampling whenever a sand stratum is intersected in the depth range between 155 and 220 feet (47 to 67m). This also applies for the event that a rock cavity with granular fill is encountered.

3.11.2 SHELBY-TUBE SAMPLING

Shelby-tube samplers were developed to obtain undisturbed samples of soft to stiff cohesive soils for laboratory testing and range in diameter from 2 to 6 inches (Hunt, 2005). Governed by the confines of the core barrels being used, a 2 to 2.5-inch diameter sampler must be employed.

For the borings targeting specifically the puddled clay in the deep rock zone, or for any borings that happen to intersect it, continuous sampling must be used throughout the entire duration in order to investigate the entire profile. This is necessary as past investigation programs have discovered that artifacts of archaeological significance are embedded in the puddled clay mass (MacPhie, 2008a).

Continuous sampling should also be employed in the event that a rock cavity with cohesive soil infill is intersected.

3.11.3 ROCK CORES

All rock cores will be recovered in triple-tube core barrels as specified above to ensure minimal core disturbances. Furthermore, all cores are to be retained in core boxes, and wooden spacers must be used to identify the depth of run (Hunt, 2005). This is to ensure that core samples are fully recovered and maintained in good condition for detailed logging and observation.

3.11.4 SPECIAL PROCEDURES

In the event that the 0.5-inch thick iron plate is intercepted (Target B), it must be fully penetrated with drilling using the HQ diamond-core drill bit, to allow further exploration and sampling underneath. The core sample must also be retained for laboratory analysis and dating (see

Section 3.12). The same procedure applies for any pieces of wood intercepted (including Target A).

In the event that bedrock cavities are encountered, which is most probable in depths ranging from 155 to 200 feet, the following steps must be taken (MacPhie, 2001). First, the hole must be 'sounded' in order to determine the nature of the contents of the cavity; be it filled with soil or stagnant water. If the former situation arises, the soil must be sampled as per the procedures specified in Sections 3.11.1 and 3.11.2. Following this, a jet of water sent using the appropriate equipment, must be applied to the base of the hole in order to displace the soil and form an artificial cavity which will facilitate lateral camera inspection. However, if it is determined that the cavity is filled with stagnant water, camera inspection must also be used; but only if turbidity levels of the water permit such inspection. If the water is too turbid, flocculants must be added to the water to induce the aggregation and subsequent sedimentation of suspended particles; thus improving visibility levels.

3.12 Laboratory Testing & Dating

Recovering samples rock, soil, and archaeological artifacts are only the means to an end of analysing and dating the samples.

This section describes testing procedures to be conducted on soil and archaeological artifacts recovered at depths below 155 feet.

It should be noted that any ages which source the deposition of the object prior to Searchers' activities proves beyond a reasonable doubt that the object was placed by the Original Depositors. Dating thus plays a pivotal role in the overall investigation.

3.12.1 Radiocarbon Dating

Radiocarbon dating is a method used to determine the age of carbonaceous materials. It determines the amount of radioactive decay of individual carbon-14 atoms (Plastino, 2001).

All archaeological artifacts that are carbonaceous in composition, recovered from sampling and cores, will be subjected to radiocarbon dating in order to accurately determine the age of the artifact. This applies especially to the wooden roofing (Target A) and oak buds which were

previously found embedded in the puddled clay (Target D). This analysis is to be sub-contracted to specialists in the field.

3.12.2 Metallurgical Testing

The field of metallurgy is focused on the study of the microscopic and macroscopic properties of metallic materials. With the aid of electron microscopes and crystallography, the composition, processing history and physical and mechanical properties of the sample can be determined (Tylecote, 1992).

All ferro-magnetic artifacts and underground workings such as hand tools or plates of iron (Target B) must be retained for metallurgical testing in the laboratory to enable accurate age determination. As for the carbon dating testing, this analysis is to be sub-contracted to specialists in the field.

3.12.3 Mineralogy Analysis

A mineralogy analysis should be conducted on all soil (including puddled clay) which is recovered in the deep rock zone (Target D) and in rock cavities. This is necessary in order to determine the origin, nature and age of the sample. The intention here is to determine if the sample is of man-made or geological origin. If found to be man-made, it is imperative that its age be determined in order to ascertain whether it can be attributed to the workings of the Original Depositors. It is required that an expert in soil science is consulted and sub-contracted to perform the above specified analysis.

4.0 ANALYSIS OF SITE CONDITIONS

4.1 SITE DESCRIPTION

Oak Island is one of more than 300 islands in Mahone Bay, a shallow bay on the Atlantic side of Nova Scotia, and is located 40 miles to the west-southwest of Halifax, Nova Scotia. The island is approximately 0.75 miles in length and 0.5 miles in width, and has an area of about 130 acres (Golder & Associates, 1971).

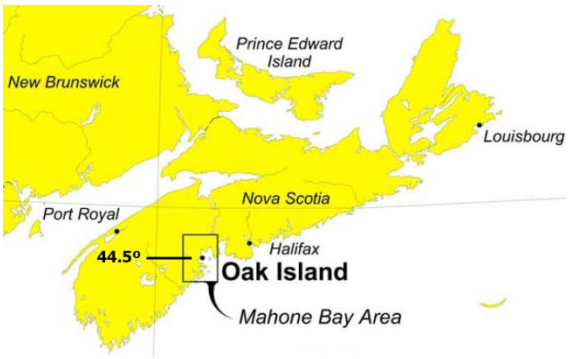

Figure 13 - Map of Nova Scotia
(Harris and MacPhie, 2005)

Figure 14 - Aerial View of Oak Island
(Oak Island Treasure, 2008)

The Money Pit is located 300 feet north of the southern shoreline of the island. In the immediate vicinity of the Pit, the surface is relatively flat, with significant ground removal of approximately 10 feet (Golder & Associates, 1971). To the north and east of the Money Pit, the ground surface rises steeply to the original ground level which slopes upward in a north-easterly direction. To the south of the Money Pit, the ground surface slopes steeply downward to the beach which forms the present shoreline (Golder & Associates, 1971).

4.2 SUBSURFACE GEOLOGY & STRATIGRAPHY

The subsurface profile at the Money Pit consists of approximately 155 feet of glacial till underlain by anhydrite bedrock (Figure 15). The glacial till consists of an upper clayey till low in permeability with frequent boulders, underlain by fine-grained interglacial deposits, followed by a lower till layer which becomes more silty and sandy with depth (MacPhie, 2008a). From roughly 155 to 200 feet, the anhydrite bedrock is highly fissured with frequent soil infillings and is thus referred to as *broken anhydrite* (MacPhie, 2008a). Below 200 feet, the bedrock is generally competent and composed of limestone, gypsum and sandstone of the Windsor Formation which is between 500 to 1000 feet thick (Golder & Associates 1971).

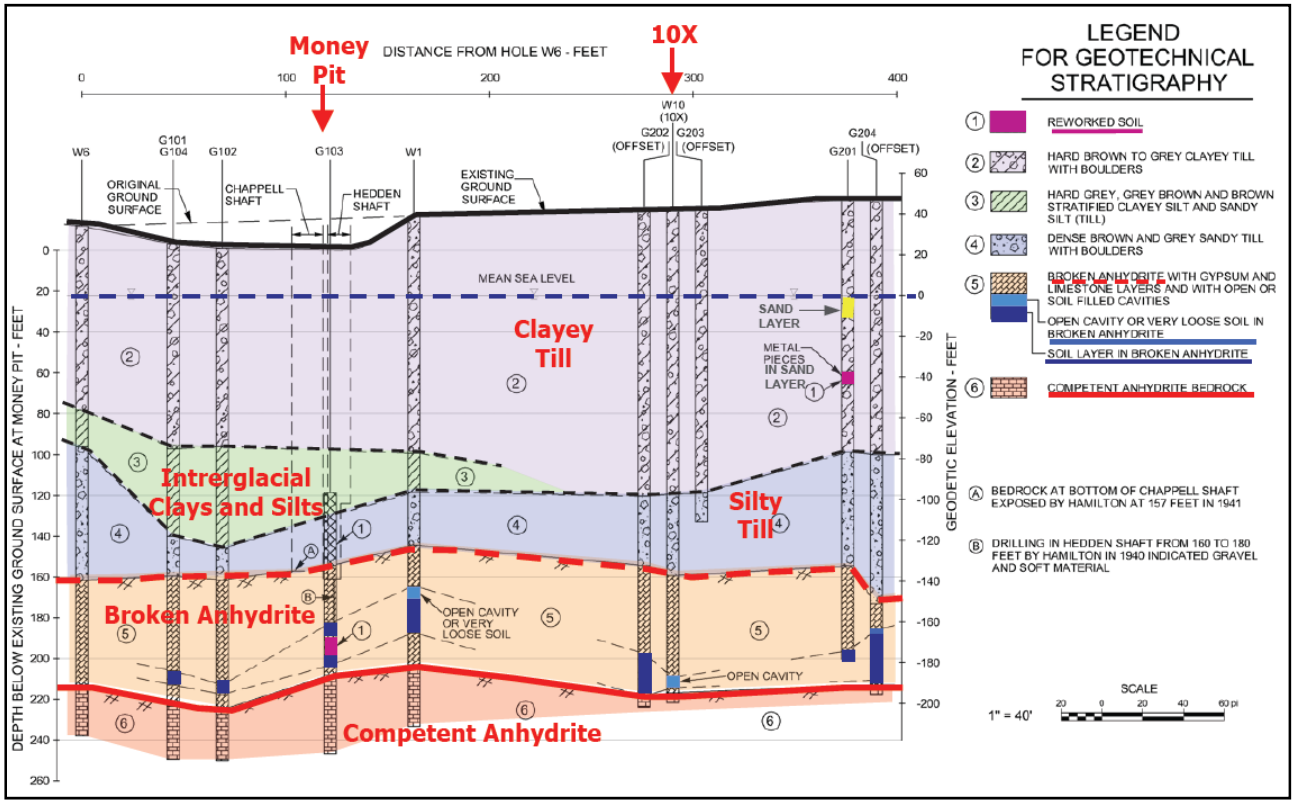

Figure 15 - Subsurface Stratigraphy
(MacPhie, 2008a)

According to Golder & Associates (1971), this complex profile is likely the result glacial deposition occurring 18,000 years ago creating a subsurface profile of either huge anhydrite boulders in a till matrix or broken anhydrite bedrock with frequent soil infillings. Typical of this type of deposition, the coarser material would be deposited at the head of the melt-water stream and the finer material deposited beyond the ice front. As melting progressed the finer grained soils and well graded tills would be deposited in situ and would be pre-consolidated by the weight of the overlying glacier.

The water content, N-values, and permeability of the subsurface strata in the vicinity of the Money Pit are depicted in Figures 16, 17 and 18 below. This data was obtained from the logs for boreholes 101, 102, 103 and 104 of the Golder & Associates Geotechnical Drilling Program conducted in 1971. The positions of these holes are depicted in Appendix C.

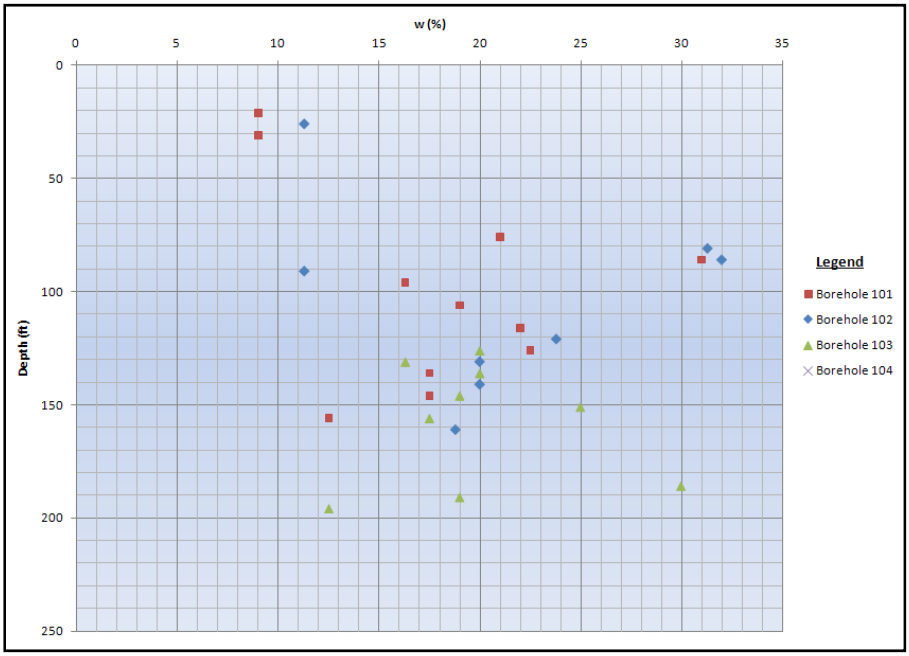

Figure 16 - Water Content with Depth
(Golder & Associates, 1971)

In Figure 16, it is evident that the average moisture content in the glacial till overburden is 20% with moisture contents ranging from 9 to 32%. Below 150 feet, which corresponds to the broken anhydrite stratum, the data points represent the moisture content of soil infillings (rather than the anhydrite rock), which range in moisture from 12.5 to 30%.

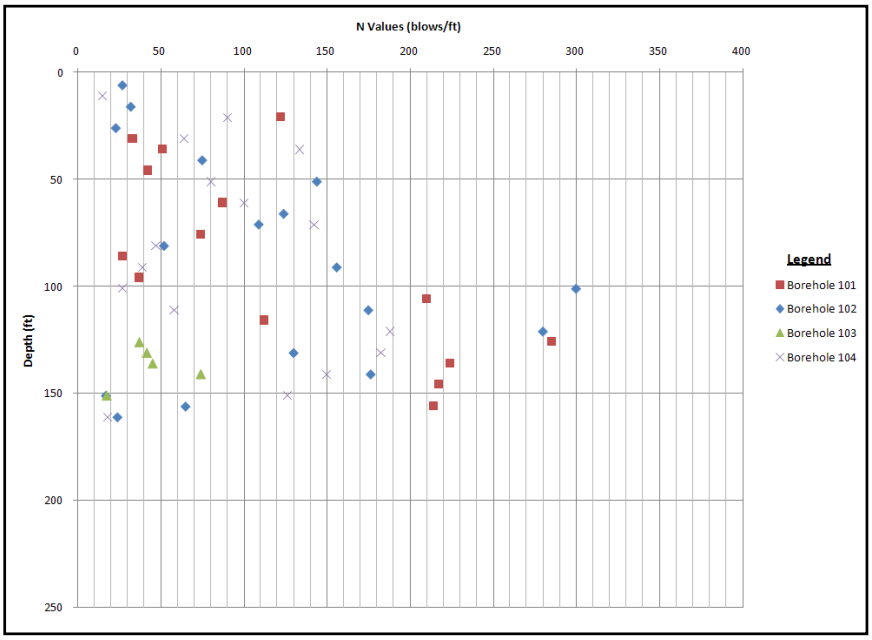

Figure 17 - N Values with Depth

(Golder & Associates, 1971)

Figure 5 illustrates that the average N-values for the glacial till is roughly 140 blows per foot. This implies that the till overburden is very hard in terms of consistency or very dense in terms of relative density (Canadian Geotechnical Society, 2006). The Figure also illustrates that a depth of 100 feet marks a shift in the N-values; overlying soils average at 80 blows per foot and underlying soils average at 180 blows per foot. This is indicative of a general shift from clayey silt strata to more silty and sandy strata.

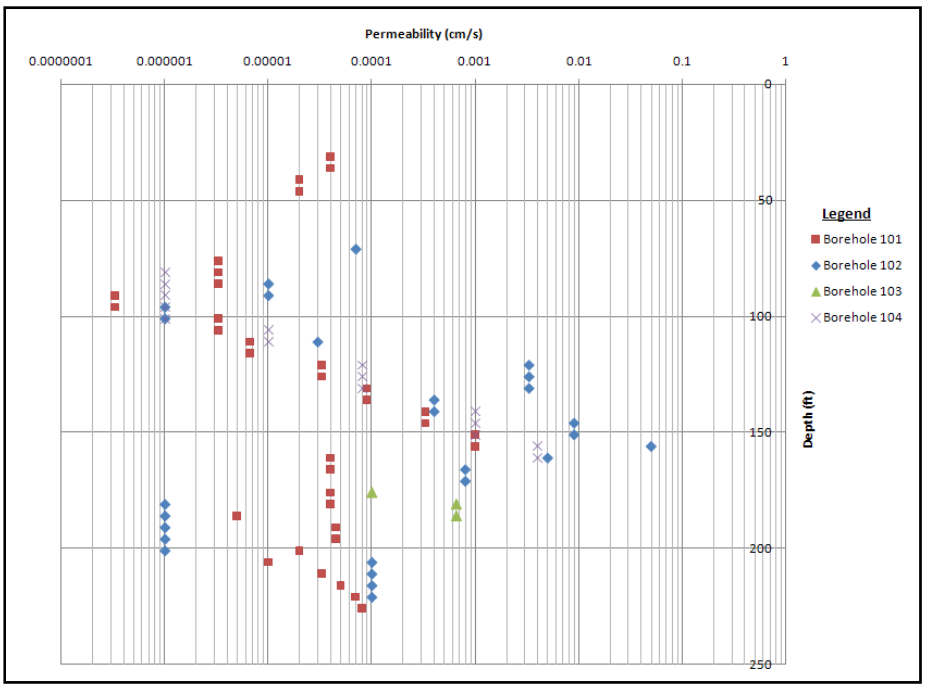

Figure 18 - Permeability with Depth
(Golder & Associates, 1971)

As shown in Figure 18, the permeability of the glacial till ranges from $3x10^{-7}$ to 0.05 cm per second. The lower values occur in the upper 100 feet of the subsurface, indicative of soil matrix primarily composed of low permeability clayey silts. Below 100 feet, permeability increases steadily as the soil matrix becomes more silty and sandy in nature. The broken anhydrite stratum registers an average permeability of $1x10^{-4}$ cm/s; demonstrating the degree to which this bedrock stratum is fissured with frequent soil infillings.

4.3 SUBSURFACE HYDROGEOLOGY

The hydrogeology of the site was determined from a detailed study performed by the Woods Oceanographic Institution in 1995 (Aubrey et al., 1996). The study was composed of a combined hydrogeological, hydrographic, and nearshore morphologic investigation of Oak Island.

In order to estimate the degree of hydraulic connectivity between the boreholes and marine waters, tide gauges were deployed in four areas: the Pier in Chester Harbour (in Mahone Bay), borehole 93-03 and Borehole 10X (located 25 feet and 170 feet to the north-east of the Money Pit respectively) and the Triton Shaft (located 660 feet north of Money Pit). Figure 19 displays a plan of the hydrogeological testing program that was undertaken and includes the positions of the tidal gauges in relation to the Money Pit.

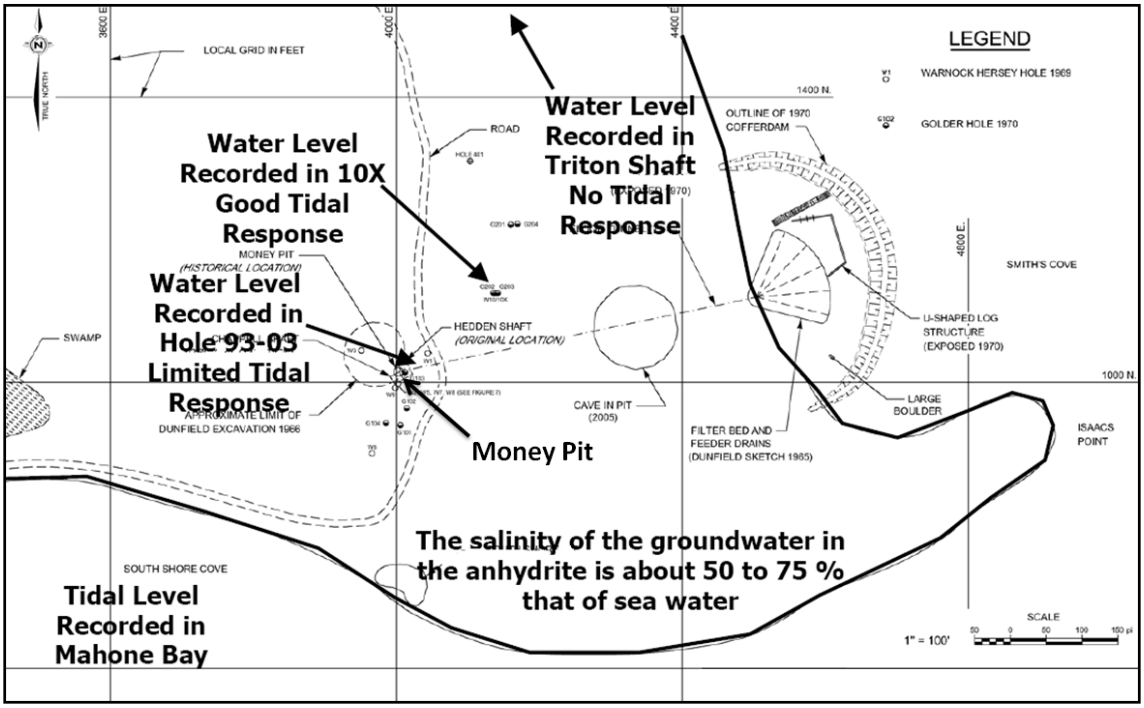

Figure 19 - Plan of the Hydrogeology Testing by the Woods Hole Oceanographic Institution (MacPhie, 2008a)

Tidal gauge results from the study indicate that there is a weak hydraulic connection between borehole 10X and the boreholes in the area of the Money Pit including 24th hole of the Becker Drilling Program, and the five boreholes drilled by the Oak Island Detection Company.

This hydraulic connection was found to occur in the highly pervious broken anhydrite bedrock at a depth of about 200 feet (Aubrey et al., 1996). Additionally, from similarities in the tidal fluctuations at Borehole 10X and Mahone Bay (Figure 20) a hydraulic connection between these sites was discovered.

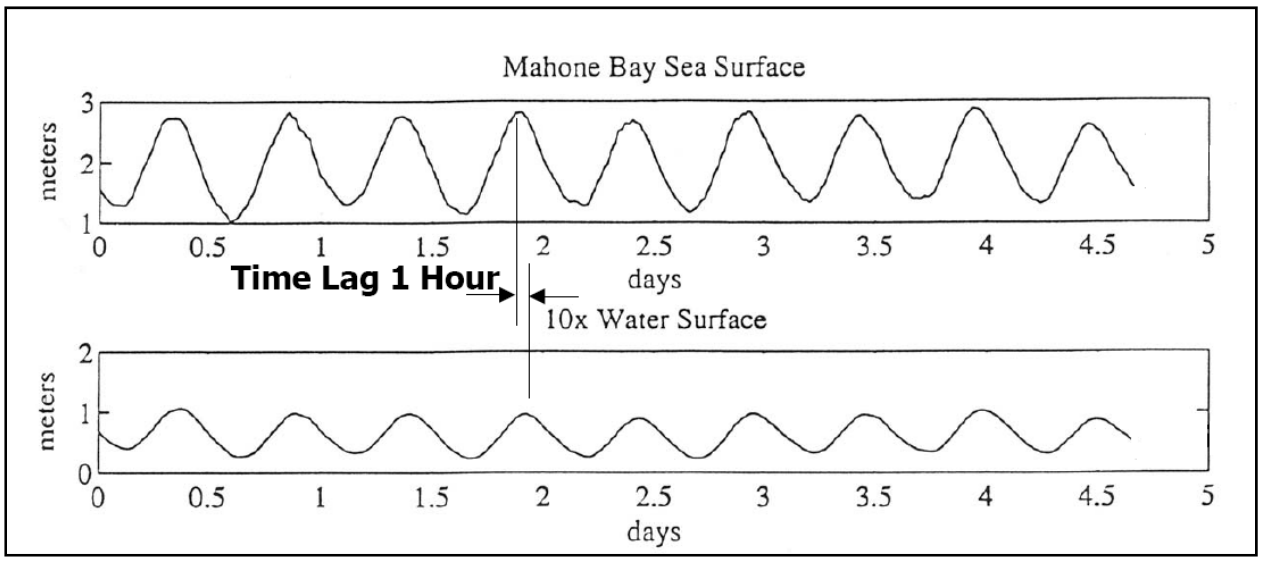

Figure 20 - Tide-Gauge Records from Stations at Mahone Bay and Borehole 10X (Aubrey et al., 1996)

From the time lag of 1 hour and the reduction in amplitude between the tides at these two sites, it can be concluded that the tidal connection is good; indicative of a flow system through a pervious zone in the broken anhydrite layer.

It is thus evident that there is a groundwater flow system in the broken anhydrite connecting the Money Pit with Borehole 10X and Mahone Bay (Figure 21).

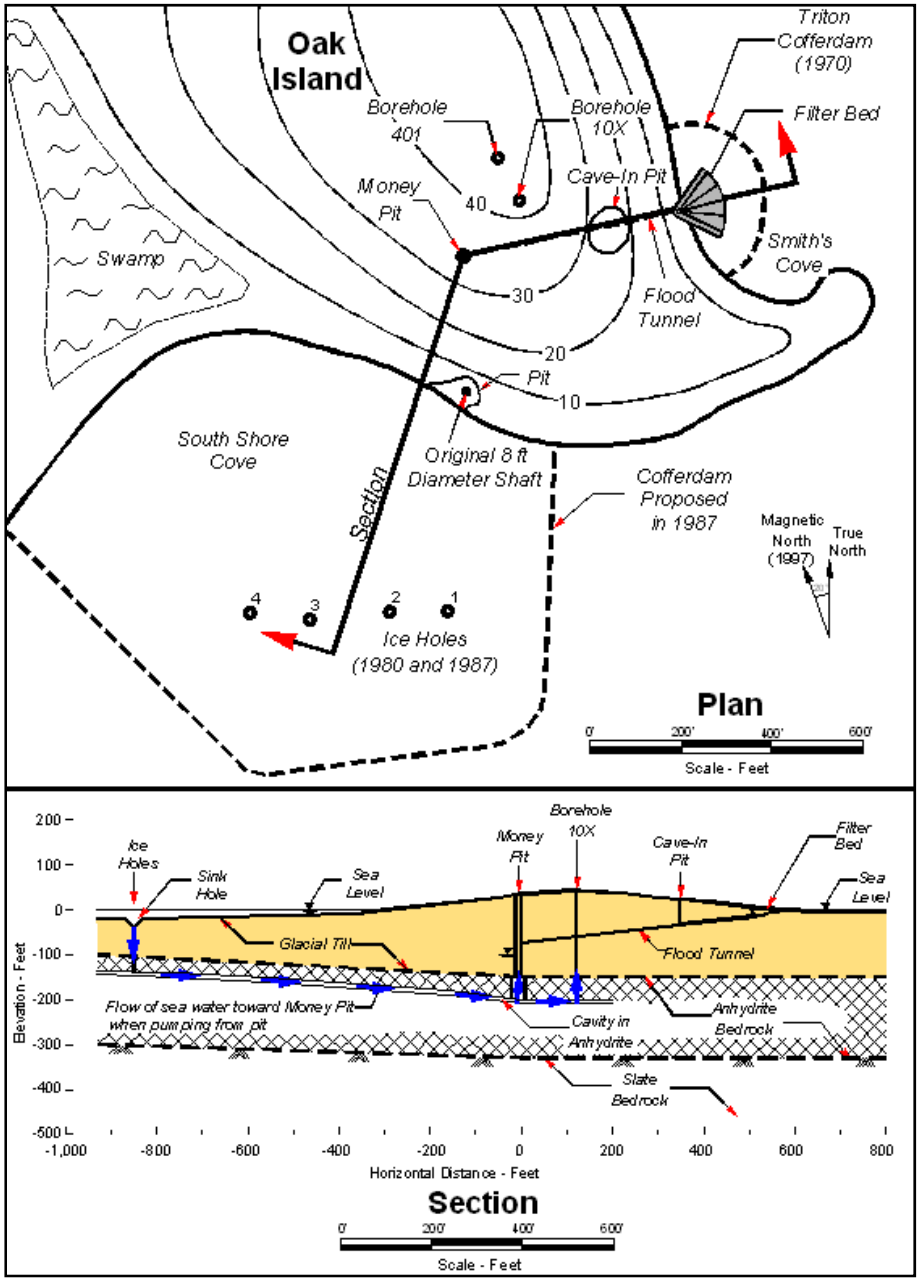

Figure 21 - Flow System through the Broken Anhydrite (Harris and MacPhie, 2005)

According to Harris and MacPhie (2005), the flow system in the anhydrite is activated by dewatering activities at the Money Pit. In the absence of such activities, the groundwater in the anhydrite follows flow reversals in phase with the tidal flow sequences. There is thus a continuous movement of water in the anhydrite.

Aubrey et al. (1996) further note that the salinity of the groundwater in the broken anhydrite ranges between 50 to 75% that of ocean water; a finding that further demonstrates the connection between the two water bodies.

The chemical composition of anhydrite crystals render it fairly soluble in salt water. Dewatering activities at the Money Pit combined with the high saline content of the groundwater, resulting in the dissolution of the walls of the cavities and fissures in the broken anhydrite and a progressive increase in permeability with time.

The above events constitute a negative cycle. As discussed Section 2.0, each successive excavation attempt at about 90 feet was faced with higher rates of incoming water due to an induced increase in permeability of the underlying bedrock. After all these cycles, resorting to dewatering to combat the incoming water would prove fruitless. What is needed is a design that serves as an impermeable barrier to isolate the Money Pit before any excavation takes place.

5.0 LATERAL EARTH PRESSURES

Once the impermeable ring-barrier is constructed and the enclosed earth is dewatered, unbalanced hydrostatic pressures act radially inwards on the walls of the barrier. The surrounding soil imposes an external lateral load on the walls of the barrier as the enclosed earth is excavated.

An analysis of the subsurface pressure distribution must consist of a determination of both short and long-term loadings.

Short-term loadings take effect as excavation proceeds through the glacial till stratum and are local to that particular stratum. According to Terzaghi, Peck, & Mesri (1996), the short-term loading is the 'apparent' earth pressure distribution which is empirically derived. They apply for ground excavations with vertical or near vertical faces, and the distribution shape is dependent on the physical nature of the soil layers that are excavated.

The authors further state that for excavation through stiff-to-hard fissured clays, for which predominantly comprise the till overburden (MacPhie, 2008a), the earth pressure distribution is as follows:

From 0 to 0.25H:	σ_h increases linearly from 0 to $(0.2 – 0.4\gamma H)$
From 0.25H to 0.75H:	$\sigma_h = 0.2 – 0.4\gamma H$
From 0.75H to H:	σ_h decreases linearly from $(0.2 – 0.4\gamma H)$ to 0

The upper bound pressure value of $0.4\gamma H$ was used to generate a more conservative estimate of short term 'apparent' pressures.

The long-term pressures refer to the lateral pressures exerted by virtue of the tendency of soils to transmit vertical in-situ stresses laterally but at a reduced magnitude. Terzaghi, Peck, & Mesri (1996) identify an empirical coefficient, coefficient of earth pressure, which represents the ratio between the horizontal and vertical in situ stresses. Also, a constituent of the long-term loading is the hydrostatic pressures, which exert equal pressures in all directions.

Long-term pressures are calculated by the formulas:

$\sigma_h' = K_o*(q + \gamma'H)$
$\sigma_w = \gamma_w H$
$\sigma_T = \sigma_h' + \sigma_w$

Where σ_h' is the effective in situ stress, q is the surcharge vertical load, γ' is the submerged unit weight of soil, H is the height if the soil layer or water column and K_o is the earth pressure coefficient.

To perform a conservative design, the pressure envelope that is used is defined by the governing pressure distribution; be it short or long-term pressures.

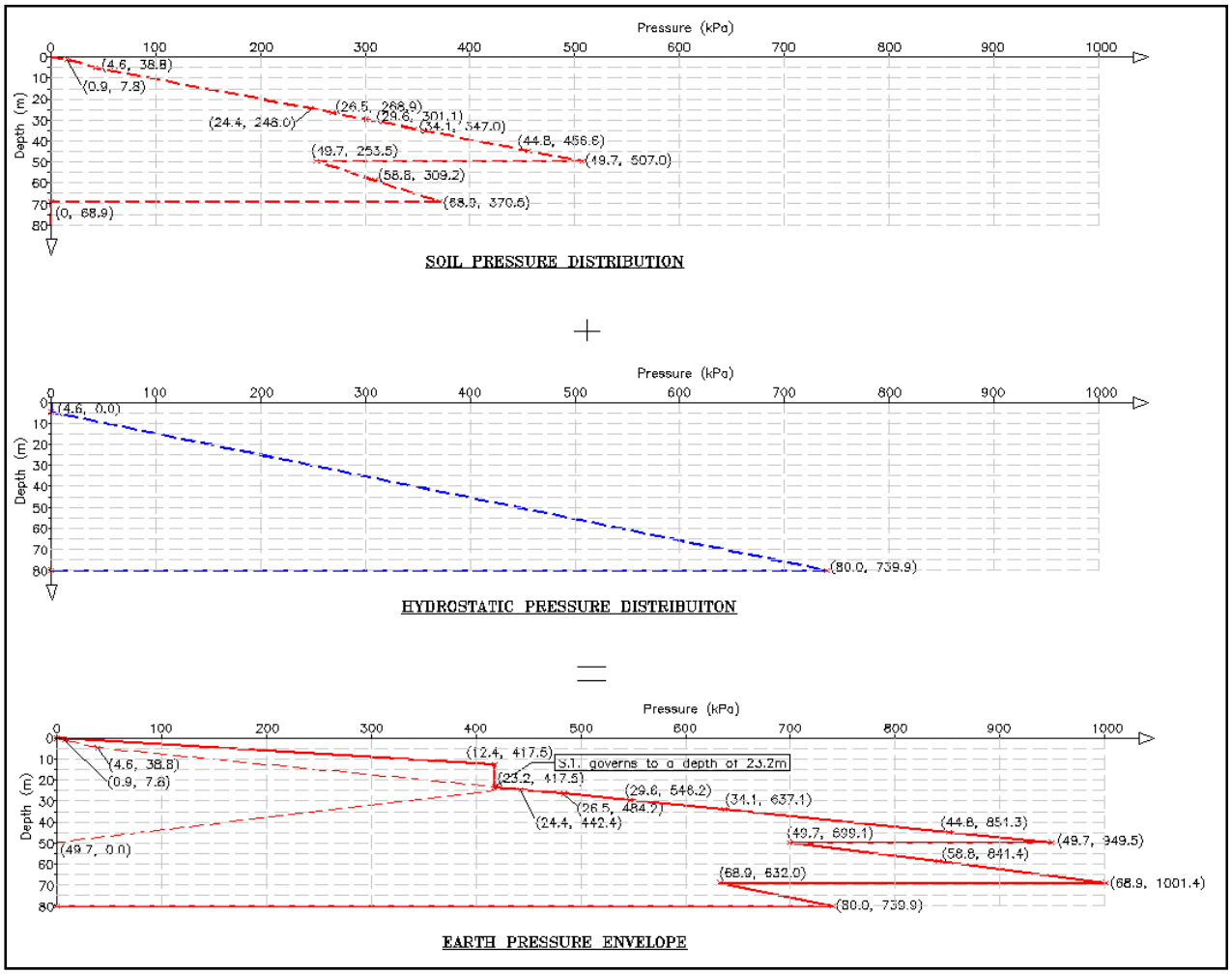

Figure 22 - Earth Pressure Envelope

For a deep excavation of about 220 feet, there are two peaks in the pressure envelop which will govern the structural design of the water barrier: 950 kPa at the base of the glacial till, and 1001.4 kPa at the base of the anhydrite.

6.0 DESIGN OF GROUTING PERIMETER

The two types of grouting technology selected for the Oak Island project are Low Mobility Grouting (LMG) and Pressure Grouting. Prior to excavation, grout must be set in place to fill exploratory voids and thus, ensure a structurally sound stratum for the work to be carried out. This procedure will limit the loss of concrete in the voids during the secant pile phase.

The objective of perimeter grouting in the till overburden and the broken anhydrite is to fill voids and create a subsurface condition which will facilitate the installation of secant piles. The objective of the perimeter grouting in the sound anhydrite, below the ultimate excavation level, is to mitigate the potential for base heave of the excavation due to the possible presence of a near horizontal open water bearing fracture in the sound anhydrite. The type of grouting undertaken depends on the conditions of the underlying stratum identified during the drilling phase.

LMG involves controlled injection of very viscous (low-mobility), mortar like grout, at high pressures, into discrete soil zones. The more conventional application of LMG is for compaction of loose soil. In this case, a properly designed LMG is injected into loose soils, homogeneous grout bulbs are formed that displace, and thus compact the adjacent soil. The application of LMG for the perimeter grouting phase is to fill voids in order to facilitate installation of the secant piles. Also, LMG can reduce permeability and stabilize the soil.

Some of the advantages of LMG are its speed of installation, the ease at which it can be performed in tight access and low headroom conditions, and the wide range of soil conditions to which it is applicable to (Hayward Baker, 2003).

Pressure grouting will be used as a verification grouting phase in the sound anhydrite layer. Pressure grouting is performed by injecting a fluid grout into the ground to fill open joints and fractures. The objective of pressure grouting is to provide protection against hydrostatic base heave of excavation due to the possible presence of water bearing open fractures below the final excavation level.

Some advantages of pressure grouting are its controlled and accurate placement. With pressure grouting, there is great flexibility to increase the efficiency of treatment, both in terms of time

and location. Finally, pressure grouting can be used in limited work spaces where mobility is an issue (Warner, 2004).

The concept of the design is based on a 70-feet diameter grout perimeter that fully encompasses the "treasure". Primary boreholes will be drilled at every 6 feet, up to 290 feet in depth, for a total of 37 boreholes along the entire section of the perimeter. Secondary and tertiary grout holes are to be drilled on an "as needed basis" when it is found that the primary holes have not adequately filled the voids.

The grout design employed is an upstage process that begins with a verification step, where pressure grouting is used to fill any minute cracks within the sound anhydrite bedrock. Figure 23 illustrates this grouting treatment plan.

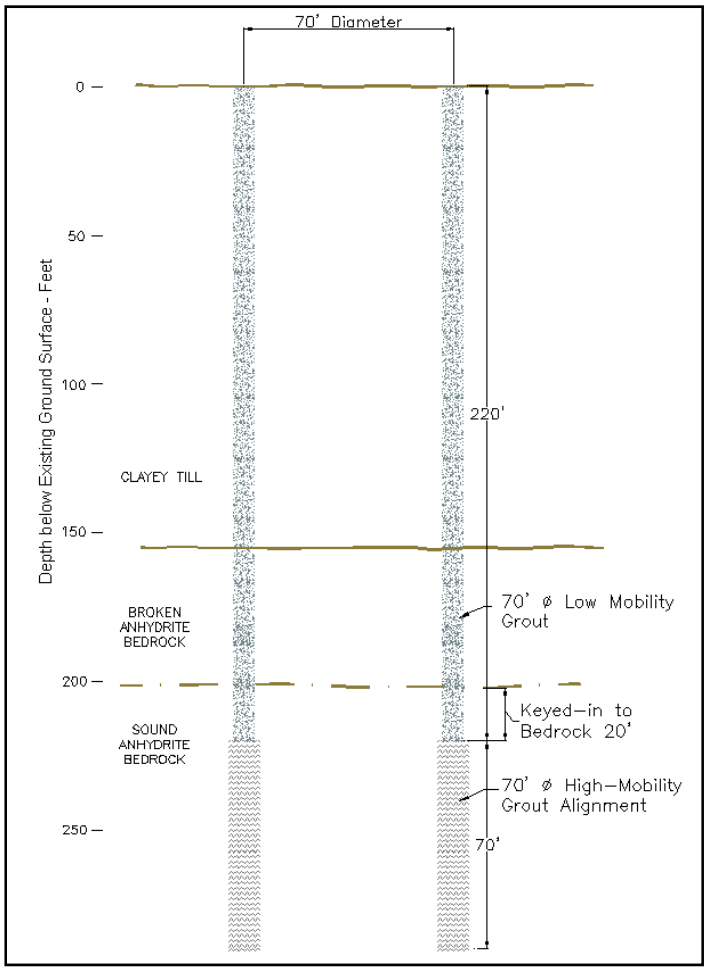

Figure 23 – Section View of Grout Perimeter illustrating Grout Treatment Plan

Subsequently, LMG is used from 220 feet to the surface to fill voids and stabilize the surrounding soil. The majority of the grout treatments will take place from the top of the glacial till stratum to the bottom of the broken anhydrite layer at a depth of 220 feet. During the drilling stage, the location of voids, chambers and cracks have to be identified and kept track of by logging (Hayward Baker, 2008). This practice will provide an accurate depiction of the different scenarios encountered that must be dealt with and allow for the consistent and thorough treatment of voids during the up-stage process.

6.1 SCENARIOS

The following is a listing of the possible scenarios that a 70-foot diameter grout perimeter would encounter. Figure 24 illustrates a profile view along the circumference of the 70 foot diameter perimeter showing the possible scenarios and their relative positions.

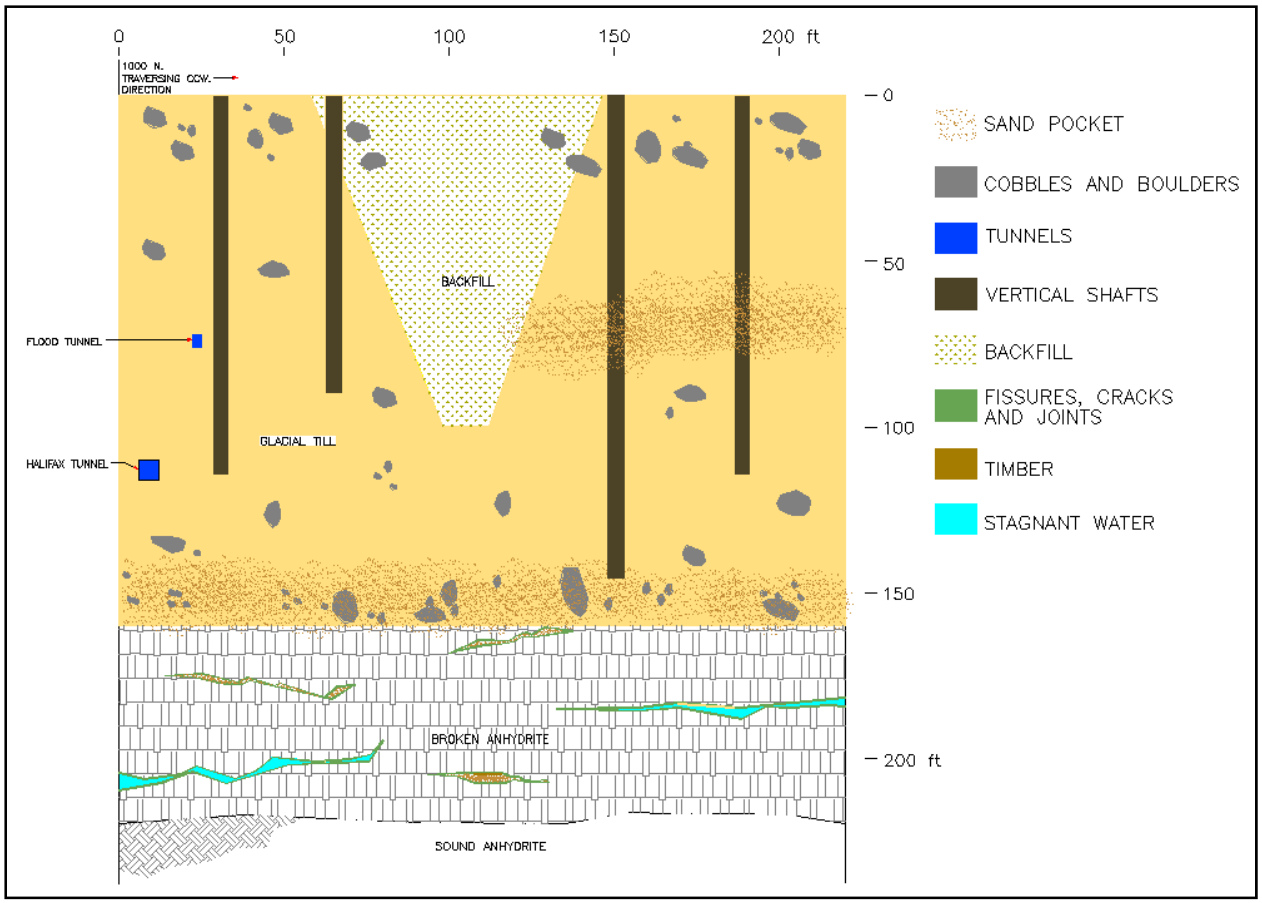

Figure 24 - Grouting Scenarios

6.1.1 TUNNELS

The known tunnels which will be encountered along the perimeter are the Flood and Halifax tunnels. However, it should be noted that it is likely that other tunnels will be encountered due to the multitude of explorers who were known to have constructed tunnels in the area (MacPhie, 2008b). These tunnels provide an easy access for loss of concrete and would complicate the construction of the secant piles if they are not filled. If a tunnel is intercepted during the drilling stage, it is to be treated with LMG.

6.1.2 CAVITIES IN BROKEN ANHYDRITE

The area surrounding the Money Pit full of cavities within the broken anhydrite layer (Harris & MacPhie, 2005). The cavities present in the broken anhydrite layer provide the biggest challenge for excavation and must be treated accordingly. These cavities have been enlarged due to the removal of fine soils and the dissolution of anhydrite by salt water from previous pumping operations.

The large cavities are to be treated with LMG whereas the smaller cavities, which are only a few centimeters thick, are to be disregarded as they cannot be effectively filled with LMG. The proposal to disregard the smaller cavities is based on the fact that the secant pile shaft will be added. Construction of this shaft will be essentially unaffected by the presence of these minute cavities since the casings installed during pile construction provide a temporary watertight barrier (Hayward Baker, 2008).

6.1.3 VERTICAL SHAFTS

Shafts dug by previous explorers and by the Original Depositors are in close proximity to the Money Pit and, where encountered along the grouting perimeter, must be treated with LMG during the grouting phase of the project.

6.1.4 STEEL CASINGS

The steel casings left in place by previous explorers need to be removed. In the event that a steel casing with a very large diameter is encountered and cannot be removed during this stage, an

attempt can be made when drilling the pile since the holes are much larger. If this still cannot be done, the secant pile has to be offset at that location.

6.1.5 OPEN JOINTS IN SOUND ANHYDRITE

Any open joints in the sound anhydrite layer must be pressure grouted to lower the permeability of this layer.

6.2 GROUT PLACEMENT OPERATION

The drilling procedures, proportioning and mixing of grout, and the injection process used in underground grouting operations are similar to those employed when grouting from the surface, except that in the former case, access and workspace are limited. These limitations can be reduced by careful planning of the overall grout operation: selecting the appropriate equipment to be used, and by hiring specialized contractors as well as an experienced construction team (Henn, 1996).

6.2.1 PROPORTIONING AND MIXING

The proportioning and mixing of the grout should be performed as close as possible to the point of injection. Due to large grout intakes and coordination efforts required with other work activities, remote or aboveground proportioning and mixing may be necessary at Oak Island.

6.2.2 WATER TO CEMENT RATIO

Standard practice is to use very thin grouts with water-cement ratios of 5:1 and greater. In theory, these thin mixes would achieve maximum penetration of the foundation materials. Research however, has shown that grout placements using water-cement ratios of 5:1 and higher (i.e., thinner grout) tend to leach, whereas placement using 3:1 or lower ratios (i.e., thicker grout) have been used successfully and produce a higher quality grout once in place (Henn, 1996). Therefore, grout with water-cement ratio of no greater than 3:1 will be used for the pressure grouting phase of the project.

It is important to note that the viscosity of the grout applied should vary between the tunnels and the open fractures (2 cm or more). A lower viscous grout is to be applied in the tunnels as compared to the open fractures.

6.2.3 BENTONITE

The bentonite added to the grout mix must be pre-hydrated before it comes into contact with the cement slurry. To achieve pre-hydration, separate mixing and storage of the bentonite and water mixture is required in a large agitator tank. The amount of bentonite used in the mix is relatively small and is about 2 to 8 % of the weight of the cement. The bentonite can be pre-measured and bagged in plastic sacks weighing 2 to 8 pounds (0.9 to 3.6 kg) when bagged cement is used (Henn, 1996).

6.2.4 REFUSAL CRITERA

Refusal occurs when the grout being infected is no longer taken in by the voids on a specific hole. The refusal criteria used can have a potentially important effect on whether or not a significant reduction in permeability can be achieved. The refusal criterion is given as a minimum amount of grout injected into the hole for a specified period of time (Henn, 1996). It should be provided in the specifications for each grouting method (LMG and Pressure grouting) to be used on the project. A minimum amount of time (at least 10 minutes) must be allowed to pass before injection of the grout is terminated (Kutzner, 1996).

6.2.5 SAFTEY AND ENVIRONMENTAL ISSUES (WASTE DISPOSAL)

Waste is generated by the grouting operations (such as drill cuttings, unused grout and wash-water). Drill cuttings can be removed, along with the other materials, and are not deemed hazardous. Unused grout and wash-water used to clean out the grout lines and equipment, on the other hand, must be treated specially (Henn, 1996).

The unused grout material must be extracted from the underground and brought to the surface. This unused grout material must be pumped into a steel box and allowed to harden. Subsequently, the hardened grout is to be removed from the steel box and disposed of with the other excavated materials or it can be transported to a landfill that accepts used concrete (Henn, 1996).

7.0 CONSIDERED DESIGNS FOR SHAFT CONSTRUCTION

7.1 SHEET-PILE WALLS

Sheet-pile walls are often used for waterfront structures. They are very flexible and are often used where there are unfavourable soil conditions. Once the sheet-pile wall is installed the water flow is reduced and reinforced concrete can be used as a permanent structure (see Figure 25). Grouting is often used to make the structure watertight (Clayton, Milititsky, & Woods, 2001).

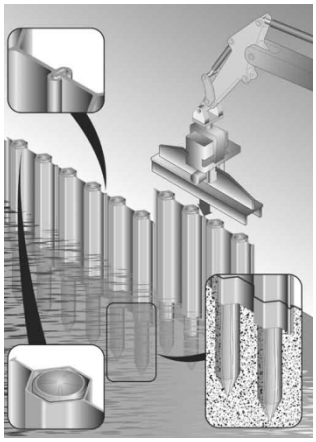

Figure 25 - Sheet-Pile Wall Process
(Clayton, Milititsky, & Woods, 2001)

The main difficulty in installing sheet-pile walls is driving the large sheets through the glacial till with boulders as this is not considered feasible. The sheet piles could not be driven into the broken anhydrite.

7.2 WELL PUMPING

Well pumping begins with the installation of water pumps inside boreholes surrounding the Money Pit. It is used to lower the water table and allow a permanent liner to be installed so an excavation can take place in the dry at the Money Pit site. This method is most effective in low permeability soils (Clayton et al., 2001).

Due to the high permeability of the broken anhydrite and its direct hydraulic communication with the Atlantic Ocean, using well pumping as a method of recovery would essentially involve

the pumping of the ocean: an impossible task. Also, pumping for many months would increase the broken anhydrite's permeability thus resulting in a progressive increase in the inflow of water to be pumped (Harris and MacPhie, 2005).

7.3 CRYOGENIC FREEZING

Cryogenic freezing involves freezing the soil surrounding the Money Pit. As Figure 26 depicts, pipes are placed in particular zones and chilled brine is released through the pipes which will freeze the broken anhydrite layer if the movement of the water beneath the surface is less than 4 inches per hour (Karol, 2003).

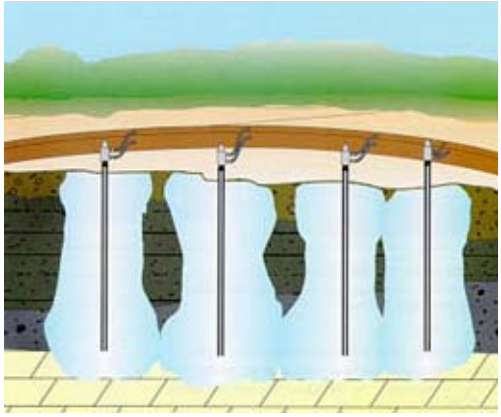

Figure 26 - Cryogenic Freezing Process
(Christensen Layne Company, 2008)

Unfortunately, due to the direct hydraulic communication between the Money Pit and the Atlantic Ocean via the groundwater flow system in the broken anhydrite, the movement is in fact much greater than 4 inches per hour (MacPhie, 2008b). The constant fluctuation of groundwater will make it very difficult to freeze the water. It is also important to consider that salt water, which contains between 50 to 75 percent salinity (Aubrey et al., 1996), will cause the ice to melt because the salt will disrupt the equilibrium of the ice (Senese, 2005).

Another important factor to consider is the constant use of chilled brine. It will be necessary to have a power plant constantly pumping chilled brine into the soil until a permanent liner is installed; an extremely costly endeavour (Hayward Baker, 2008).

7.4 JET GROUTING

Jet grouting is a ground modification system which is sometimes used to create a temporary or permanent stabilization of soft and/or liquefiable soils. It is used in any situation involving the control of groundwater or the excavation of unstable soil. Before jet grouting, it is necessary to have predrilled boreholes (approximately 6 inches in diameter) in order to access the treatment zone. After the boreholes are created, a bottom-up procedure is used by inserting a pipe inside the hole, and erosion is initiated with a high velocity of cutting and replacement of fluids until the grout reaches the top of the borehole (see Figure 27) (Hayward Baker, 2003).

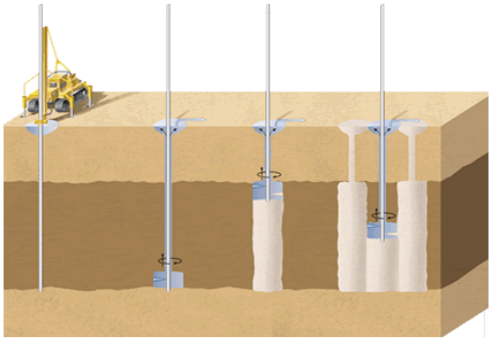

Figure 27 - Jet Grouting Process
(Hayward Baker, 2003)

Jet grouting is a quick and safe method of creating a water barrier which will save both time and money in almost any project. It will be difficult to jet grout in the anhydrite layer since the concrete will not fill the voids adequately and will not guarantee an impermeable barrier (Hayward Baker, 2008).

7.5 DIAPHRAGM WALL

A diaphragm wall is used to provide an impermeable layer in the upper till. PVC pipes are placed vertically inside the diaphragm wall so that the jet grouting or compaction grouting machines can be guided more accurately into the broken anhydrite layer. Figure 29 depicts the diaphragm wall design concept considered.

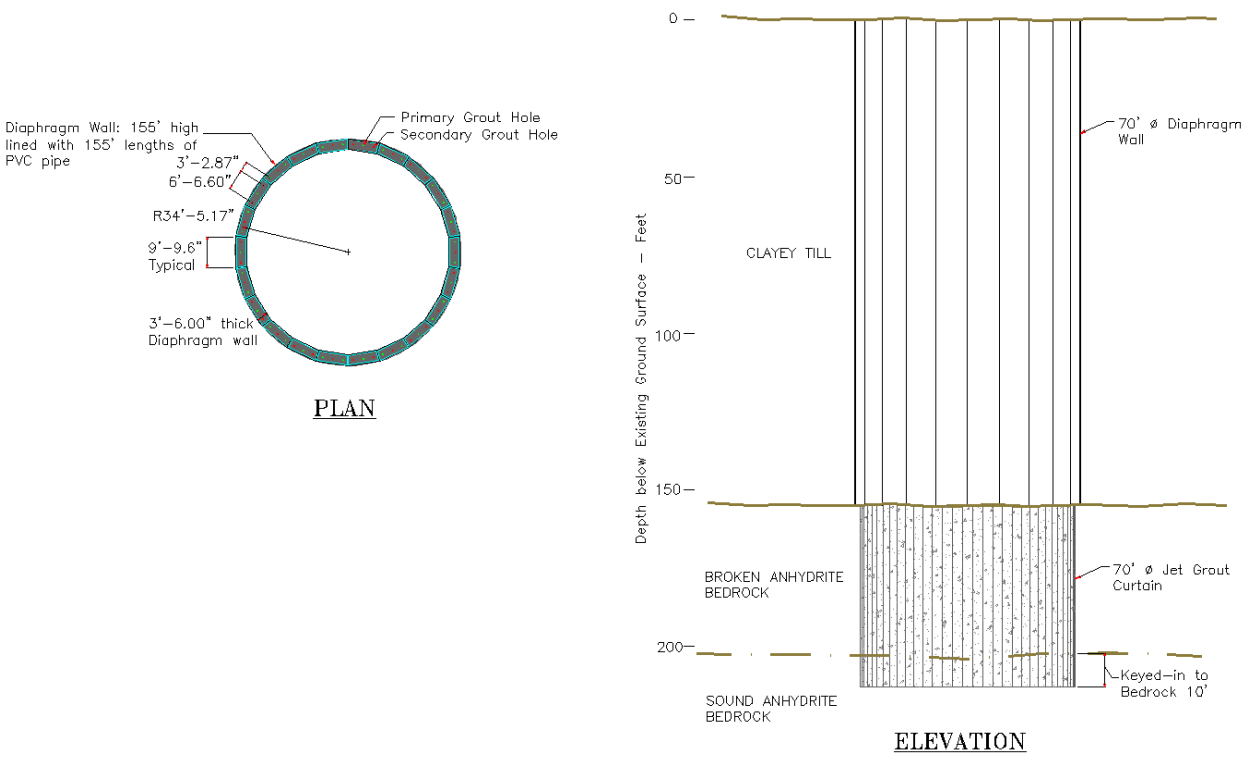

Figure 28 – Diaphragm Wall Design

The diaphragm wall is constructed with the use of a clam shell bucket or with a hydromill. Once the trench is cut, a reinforcing cage with the PVC pipes are lowered into place and concrete is poured in place (Luongo & St-Amour, 2005).

There are several problems with the use of a diaphragm wall. The first major concern is that the use of jet grouting in the broken anhydrite layer would not provide a perfectly water-tight barrier (Hayward Baker, 2008). There is also a weak interface region between the diaphragm wall and the jet grouting columns. Therefore, it does not provide an impermeable perimeter and does not satisfy the needs of this project. It is also worth mentioning that misalignment of the depth of a diaphragm wall panel can cause shadowing regions when using jet or compaction grouting (Luongo & St-Amour, 2005). The second major concern is that the construction of such a wall is not feasible because of the presence of boulders and timber logs found in the soil. The clam-shell bucket is not designed to break through hard materials and the hydro-mill will get jammed if it tries to cut through the timber logs (Hayward Baker, 2008). The diaphragm wall option is thus not the optimal solution for the creation of a water-tight perimeter.

7.6 SECANT PILE WALL

A secant pile wall is constructed by overlapping primary and secondary piles. The primary piles are drilled and constructed first, and only once the primary piles have gained sufficient strength are the secondary piles added. The addition of the secondary piles is done before the primary piles have fully hardened so that a good interface and water tight seal is formed. Each pile is drilled to the final depth and is constructed one at a time. This is to ensure that each pile forms a continuous segment throughout the entire depth (Hayward Baker, 2008).

Secant pile walls are an excellent choice for our project because they provide a water tight barrier through every stratum. Since the concrete is poured all at once, there is no weak interface region in the piles between the silt and broken anhydrite layer, like there would be with a diaphragm wall and jet grout structure.

The major concern with secant pile walls is if the drilling and the piles are misaligned or have deviated (Hayward Baker, 2008). If insufficient overlap between the piles occurs, then points of weakness may occur where the minimum amount of concrete is not provided. These areas might not provide sufficient strength to resist against the soil and water pressures acting locally. A weak spot in the structure can also cause a cataclysmic failure since each pile rests on the others. They provide lateral support to each other. If one of these piles cannot resist the local forces and fails, then the adjacent pile that was braced by it will fail as well, thus causing a domino effect. A solution to overcome this problem is to add lateral bracing rings around the inner circumference of the secant pile shaft while excavating. This would help restrain and brace each pile, allow for better distribution of forces around the shaft, and would lower the unbraced length of a pile if an adjacent pile were to fail.

Another concern with the secant pile shaft is that it is very expensive to construct and can be time consuming as well as labour intensive. It takes almost three to four days to drill and case each pile and then at least another day to place the reinforcement cage and the concrete. In addition to the materials, workers, machinery and transportation required, a secant pile wall can be very costly to build (Hayward Baker, 2008).

However, there are many benefits from constructing a secant pile wall in our given situation. Since the secant pile shaft is designed to resist the soil pressures acting on its surface (as presented in Section 5.0), there is no need for a permanent liner during excavation (Hayward Baker, 2008). The secant pile wall acts as that permanent liner since the concrete is allowed to cure and reach its maximum compressive and shear strengths.

Another benefit for secant pile walls is the fact that it is much more tangible and reliable than compaction and jet grouting. There are many uncertainties with the use of grouting since the soil profile is not entirely known and it is difficult to determine if a perfectly impenetrable grout curtain has been achieved. With secant pile walls, many of these uncertainties are eliminated. It is known during construction exactly how deep a pile is drilled, how much deviation has occurred, what type of soils and other materials were found during the drillings and how much concrete and steel was placed in each pile shaft. There is also less uncertainty about forming a water-tight barrier since each pile overlaps each other and hardens as a whole (Recon, 2005). Furthermore, the curing of the concrete allows for a lower permeability. There are also fewer chances in a secant pile wall for a window occurring and no possible shadow effect can happen with the placement of the concrete.

8.0 FEASIBILITY ASSESSMENT

In order to select the most feasible design for the construction of a water-tight barrier, it is necessary to evaluate all the possible designs for recovery and compare them to one another.

There are many assessment criteria that need to be considered. Some of the assessment criteria are more important than others and for this reason, a weighting factor was applied. This weighting factor is based on the importance of the criteria relative to all other factors. The weighing factors were selected subjectively based on feedback given by MacPhie (2008b). A factor of '3' was given to the most important, a factor of '2' to criteria that was important, and a factor of '1' was given to the criteria of least importance.

The criteria of each considered method will be assigned a numerical value depending on the capability of the method to treat the selected criteria. These numerical values are selected by choosing a number between '1' and '5' depending on the method's qualitative performance; '1' being the lowest and '5' being the highest (Table 2).

Table 2 - Ranking of Feasibility

Qualitative	Numerical
Highest	5
High	4
Average	3
Low	2
Lowest	1

The most important criterion in the assessment is the creation of an *effective water barrier in the broken anhydrite*; a weighting factor of '3' was assigned to it. Because this assessment is critical in our design, the analysis of the sheet-pile wall and well pumping were omitted because these designs do not create an effective water barrier. Cryogenic freezing will be difficult to implement due to the movement of the highly saline water. Jet grouting is also difficult because the voids will not be filled adequately in the broken anhydrite.

Cost is always important to consider when dealing with any project; especially for investors. However, it is not as important as the criteria for the *ability to create an effective barrier in the broken anhydrite* and we have therefore placed *low capital costs* just below it with a weighting

factor of '2'. Cost is also given a numerical value based on estimated ranking of cost as per Table 2. It should be noted that the highest cost is equivalent to a ranking of '1', and the lowest a ranking of '5'.

In this regard the cost for cryogenic freezing and jet grouting are cheaper than the cost for the construction of a diaphragm wall or a secant pile wall. Cryogenic freezing and jet grouting may provide an effective water barrier but does not guarantee its efficiency for the long-term. And although the construction of a secant pile wall is slightly more expensive than the cost of a diaphragm wall, it will not require the additional construction for a permanent liner.

All of the remaining criteria are considered less important relative to the *ability to create an effective water barrier in broken anhydrite* as well as *low capital cost*, but they are considered to have the same level of importance relative to each other. These rankings of feasibility are therefore assigned a weighting factor of '1'. All the remaining assessment criteria are depicted in Table 3.

Table 3 - Feasibility Assessment for Considered Methods of Recovery

			Methods			
Level of Importance	**Assessment Criteria**	***Weighting Factor***	**Cryogenic Freezing[2]**	**Jet Grouting[3]**	**Diaphragm Wall[4]**	**Secant Pile Walls[5,6]**
1	**Ability to create effective water barrier in the broken anhydrite**	***3***	2 Movement of high saline water will make it difficult	2 Will not fill voids adequately	3 Increase of efficiency in jet grouting	5 Piles are sunk and serve as liner and barrier
2	**Low capital costs**	***2***	3 The cost is estimated to be between $3M - $5M	4 The cost is estimated to be between $1.5M - $2M	2 The cost is estimated to be between $5M - $7M	1 The cost is estimated to be between $7M - $9M
3	**Ability to deal successfully with boulders**	***1***	3 Will need to offset freezing if encountered	1 Will create shadow effect	1 Hydro-mill will not be able to effectively cut through boulders	5 Boulders do not hinder pile construction
4	**Water bearing sand layers**	***1***	3 If movement is less than 4 inches per hour will work	3 May require additional treatment if sand is too saturated	4 Will produce impermeable layer in the sand layer	5 Will uniformly constructed through the sand layer
5	**Salinity and movement of groundwater in broken anhydrite**	***1***	0 Movement is greater than 4 inches per hour	2 Flowing water will not allow concrete to cure properly	2 Flowing water will not allow concrete to cure properly	5 Forms impervious barrier to the ingress to the water
6	**Disturbed soil (specifically Dunfield Excavation)**	***1***	5 Will have no problems with freezing	5 Ideal for Jet Grouting	4 Can easily penetrate into disturbed soil	4 Can easily penetrate into disturbed soil
7	**Searchers' tunnels (mainly Halifax tunnel and related timbers)**	***1***	3 After filling the tunnel with sand, will freeze	1 Grout will continue to flow through the tunnel and will not settle	4 Will be treated with pre-grouting but will not have casings	5 Will be treated with pre-grouting and will use casings
8	**Searchers' shafts and related timber**	***1***	3 After filling the tunnel with sand, will freeze	1 Grout will continue to flow through the tunnel and will not settle	1 Hydromill will not be able to effectively cut through timber	5 Can easily drill through shafts and timber
9	**Abandoned steel casings and rods**	***1***	5 Will have no problems with freezing	4 Can be ignored while Jet Grouting	5 Can easily be removed	5 Can easily be removed
10	**Potential for problems during construction**	***1***	3 Depends on boulders and tunnels location	2 Jet Grout does not properly overlap	3 Misalignment of diaphragm wall panels and jamming of hydro-mill	4 Misalignment of drilling
		Total	**37**	**33**	**37**	**55**

[2] (Hayward Baker, 2008; MacPhie, 2008)
[3] (MacPhie, 2008)
[4] (Luongo & St-Amour, 2005)
[5] (Recon, 2005)
[6] (Hayward Baker, 2008)0

Given the above feasibility analysis, in addition to the recommendations from Hayward Baker (2008), the secant pile design option was the design solution which was ultimately adopted for the project. Even though it is the most expensive option, it was proven to be the most effective solution with regards to creating a water controlled perimeter and is the option which is most capable of withstanding the challenges posed onsite (as presented in Table 3)

A secant pile wall is the best way to make a shaft which is impervious to the ingress of water. Additionally, secant pile walls are the most economical method for creating an effective water-tight barrier since the construction of each pile is more reliable in completion compared with the other methods considered (Case Foundation Company, 2007). This is because each pile is constructed as a uniform structure throughout the entire depth. There are no weak interface regions as is the case with a diaphragm wall design option.

The primary piles are positioned first and are allowed to gain sufficient strength. Once the primary piles are in place and have reached a minimum compressive strength, the secondary piles are drilled between the primary piles so that the heavy temporary casing will cut through the primary piles on either side (Recon, 2005).

Construction of each pile is easier to execute since drilling can be done through wood and boulders. The problems encountered using a clam-shell bucket and hydro-mill are no longer an issue with the construction of a secant pile shaft (Hayward Baker, 2008).

The logistics and sequencing of creating a secant pile wall are also much more favorable in producing an impermeable layer. The concrete for each completed pile section is allowed to cure while the other piles are drilled and filled with concrete. Allowing the concrete to cure provides a lower permeability of concrete and also ensures a tight seal between each pile to completely cut off the ingress of water (Kosmatka et al., 2002).

9.0 FINITE ELEMENT ANALYSIS

For the design of our shaft, we used a finite element analysis for a cylindrical pressure vessel. The concept behind the finite element analysis for a cylindrical pressure vessel can be seen in Figure 29 below.

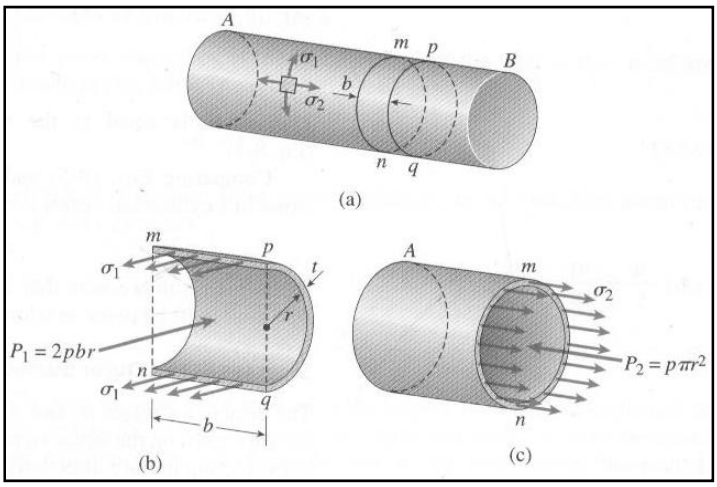

Figure 29 – Cylindrical Pressure Vessel
(Bickford, 1993)

There are three main stresses that need to be considered in a cylindrical pressure vessel. These stresses are the circumferential stress (σ_1), the longitudinal stress (σ_2) and the shear stress (τ_{max}) (Bickford, 1993). Using the finite element analysis, the formula for those three stresses can be seen below:

$$\sigma_1 = \frac{p\,x\,r}{t} \qquad \sigma_2 = \frac{p\,x\,r}{2\,x\,t} = \frac{\sigma_1}{2} \qquad \tau_{max} = \frac{p\,x\,r}{2\,x\,t} = \frac{\sigma_1}{2} = \sigma_2$$

Where p = pressure, r = radius and t = thickness (minimum thickness). The pressure used in the calculations was chosen to be the maximum soil pressure experienced and was found to be 1000 kPa at a depth of 220 ft. The radius used in the calculations was equal to the radius specified in our design, which is equal to 35 ft. The thickness in the calculations was assumed to be 1 m. This thickness was correctly assumed since the minimum thickness provided in our secant pile shaft is equal to 1 m.

Using these values, maximum circumferential, longitudinal and shear stresses are shown below:

$$\sigma_1 = 10.7\text{MPa} \qquad \sigma_2 = 5.4\text{MPa} \qquad \tau_{max} = 5.4\text{MPa}$$

10.0 DESIGN OF SECANT PILE SHAFT

As per recommendations provided by Hayward Baker (2008), the design calls for the use of four-foot primary piles and five-foot secondary piles with a center to center spacing of three feet. The section view of the primary and secondary piles can be seen in Figures E2 and E3, respectively, displayed in Appendix E.

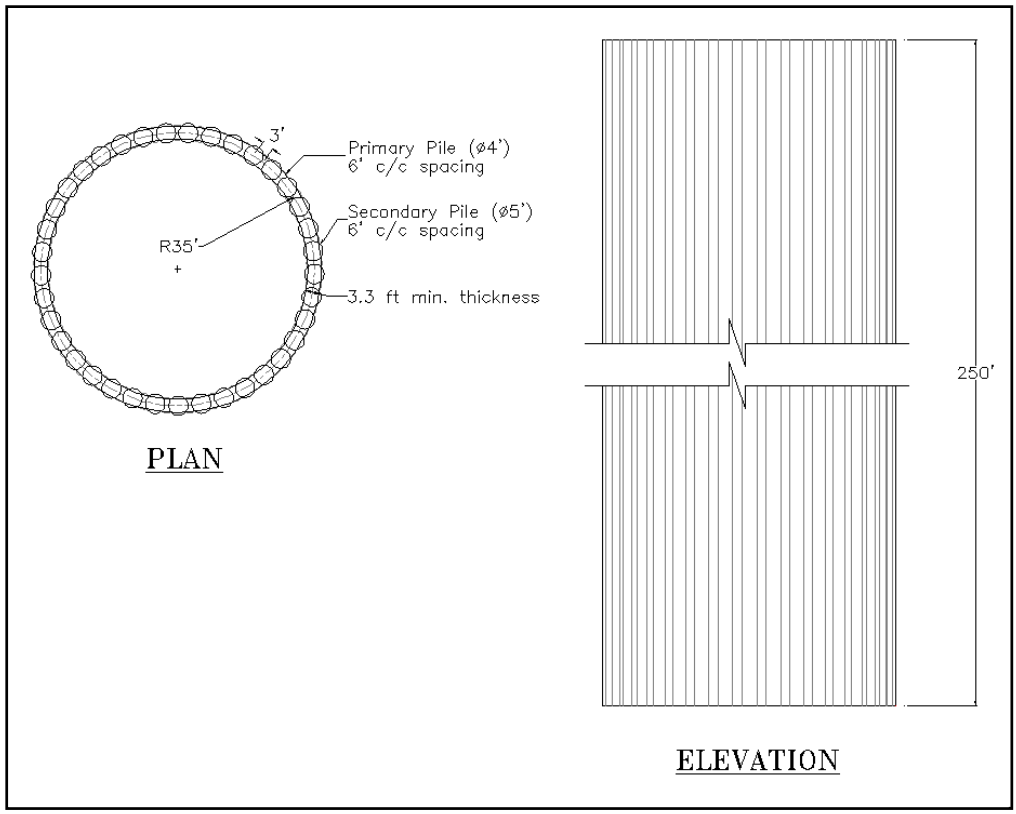

Figure 30 - Plan and Elevation of Secant Pile Shaft

The geometry of this design would provide enough overlap between the piles to produce a tight and sturdy seal. It would also give a minimum thickness equal to 1 m. Therefore, a secant pile shaft with this thickness would provide a durable structure and a secondary liner would not be needed. Lateral bracings must be added during excavation so that the piles do not buckle when the soil is removed. The secant piles shaft will be constructed to a depth of 250 feet. This is to ensure that each pile keys into the sound anhydrite stratum. By embedding the piles 50 feet into

the solid bedrock (which starts at a depth of roughly 200 feet), the structure will be able to provide sufficient resistance against overturning, sliding and base resistance (Meguid, 2008).

10.1 Construction of Secant Pile Shaft

The placement sequencing of the primary and secondary piles is very important to ensure that the constructability of the secant pile shaft is possible, and to also maximize efficiency. The primary piles are placed first, and allowed to gain sufficient strength before the secondary piles are constructed in between (Recon, 2005).

The primary piles are placed at every 6 feet from center to center (Hayward Baker, 2008). The piles are drilled under mud or slurry conditions and casings are placed to stabilize the holes and provide a temporary barrier to block the flow of water (Recon, 2005). While the drilling is being performed, a cage of reinforcing steel consisting of longitudinal and tie bars is constructed onsite. Once drilling has reached the final depth and casings are added, the reinforced steel cage is lowered into place. Concrete is poured immediately after the cage is placed in the right location (Recon, 2005).

Since the secondary piles cut into the primary piles and remove a portion of concrete, the reinforcing steel cage in the primary piles must be placed within a one-foot-wide strip located about the centre-line. This can be seen in Figure E2 of Appendix E. Furthermore, since the secondary piles will not be cut into, the steel reinforcing cage for the secondary piles will be placed along the periphery. This is presented in Figure E3 of Appendix E. Closed loop reinforcement tie bars that are needed to resist the shearing forces shall be used for the primary piles and spiral reinforcement tie bars shall be used for the secondary piles (Recon, 2005).

After a section of primary piles have been placed and the concrete is allowed to set for a minimum of one day, the adjacent secondary piles are installed so that sufficient overlap is produced. The primary piles should not be allowed to fully set before placing the secondary piles because it will be too difficult to remove the portion of concrete necessary to complete the secondary pile. Therefore, the construction of the secant pile shaft will be completed in sections until the whole shaft is completed (Recon, 2005).

Before the steel cage is lowered and concrete is poured, the tip of each pile needs to be approved by a qualified testing agency that is registered and licensed in Nova Scotia. The shaft needs to be inspected and cleared of any materials found inside, including mud, water and debris. Placement of the concrete will be done with the use of a tremie pipe. It is recommended that two tremie pipes be used so that placement of concrete is done at a faster rate. This would eliminate the chances of producing a cold joint in the concrete. Once the concrete pour starts, the casings will be removed, constantly maintaining the top surface of concrete at least 6 feet above the lower end of the casings. The concrete must be placed in each pile in one continuous operation. If the placing of concrete has to be stopped, the surface of the concrete must be approximately level. If the concrete hardens, the surface must be cleaned and flushed with a 1:1 cement sand grout before more concrete can be placed. Pouring of the concrete shall not begin within 1 hour of dusk and the drilled holes cannot be left open overnight. This is to ensure that no debris falls in the open excavation. When water is present, the water level is usually maintained by pumping at a level of 2 inches or less from the bottom of the secant pile. Since pumping is impractical for this project, concrete can be placed through the water by means of a watertight tremie pipe. When concrete is placed under water, the discharge end of the tremie pipe has to be submerged in fresh concrete and the entire tremie shaft must be full of concrete to a point above the water level. When concreting is done under water, the cement content must be increased by 1 sack per cubic yard of concrete (Recon, 2005).

10.2 MATERIALS OF SECANT PILE SHAFT

The materials must be carefully selected since harsh conditions are assumed. This is because of the ingress of highly saline seawater in the vicinity of the secant pile shaft. The groundwater has a salinity of 50 to 75 % and is subject to wetting and drying cycles due to the hydrualic communication of the groundwater in the broken anhydrite and the ocean (Aubrey et al., 1996). Therefore, the materials used must be able to resist sulphate and chloride attacks.

Steel is very susceptible to corrosion caused by chloride ions. Concrete provides some protection to the embedded steel because of its high alkalinity. The high pH protects the steel from corrosion by forming a non-corroding protective oxide film. However, chloride ions can destroy or penetrate this film. Once this passive film is destroyed, an electric cell can be formed

along or between the steel bars and corrosion will begin. One way to prevent chloride ions from reaching the steel is to apply an epoxy or galvanization coating to the reinforcing steel. Therefore, it is recommended that galvanized reinforcing steel be used in the secant pile design (Kosmatka et al., 2002).

Another major concern is sulphate attack of the concrete. Considering that the concrete provides protection for the reinforcing steel, it is very important that the concrete does not deteriorate to maintain the longest amount of protection possible. Sulphate attack is caused by the reaction of sulphates with the hardened cement paste. To protect the concrete from sulphate attack, it is best to use a low-permeability concrete to mitigate the ingress of sulphates into the concrete structure. This is done by providing a low water to cement ratio, i.e. a maximum of 0.4 (Kosmatka et al., 2002). It is also recommended that cements specifically made for sulphate environments be used, such as ASTM Type V cement (Kosmatka et al., 2002). Therefore, the concrete used in the secant pile shaft will be made out of ASTM Type V cement and will have a water to cement ration equal to 0.4. Silica fume (20%) will also be added to the concrete to lower the permeability and decrease the ingress of sulphate and chloride ions (Kosmatka et al., 2002). It has been found that Portland cements with 4 to10 percent tricalcium aluminate content (C_3A) can provide satisfactory protection against seawater sulphate attack as well as corrosion of reinforcement caused by chloride ions (Kosmatka et al., 2002).

Furthermore, the amount of concrete cover should not be less than 2.4 inches (60 mm) in concrete exposed to seawater (Kosmatka et al., 2002). Therefore, the concrete cover provided in our design is equal to 75 mm to ensure that adequate protection is provided against the highly saline seawater experienced.

11.0 EXCAVATION

After the secant pile shaft is placed and the concrete has hardened, the continuity of the structure as well as its water-tightness must be verified. This can be done by placing pumping wells inside the Money Pit area and determining if the water level can be lowered to a depth of 230 feet (70 m). This depth is specified since it is below the target excavation depth of roughly 220 feet. If this level can be maintained for a sufficient amount of time, excavation of the interior section surrounded by the secant pile shaft may begin.

The excavation must be completed with care since there are significant findings at stake. Anything of value found in the Money Pit area could be destroyed during excavation. Therefore, it is recommended that a more labor-intensive hand-excavation be done locally where required (MacPhie, 2008b). This will ensure that no archeological findings are damaged. It is very important to preserve any archeological findings since they can inspire future generations with each discovery. The value gained from preserving and learning from our heritage is invaluable compared with the worth of the actual artifacts found. These discoveries will be written and discussed about in history textbooks for people to learn from for years to come. This is why Nova Scotia has implemented laws to protect any archeological findings.

The Special Places Protection Act of Nova Scotia requires that a Heritage Research Permit from the Heritage Division is approved before disturbing any region where historical artifacts could be found (Government of Nova Scotia, 1989). To obtain a permit, one must demonstrate great skill and interest in archaeology so that important historical sites are carefully explored and that records are kept of each site and of any artifacts found (Nova Scotia Museum, 1996). It is required by the provincial Treasure Trove Act to have a Treasure Trove License if it is expected that treasure will be found (Government of Nova Scotia, 1989). Considering the nature of our project, a Treasure Trove License would be needed; it is issued by the Nova Scotia Department of Natural Resources. Even with such a license, explorers are still obliged to follow the legal requirements of the Special Places Protection Act (Nova Scotia Museum, 1996).

Once a Treasure Trove License and Heritage Research Permit have been acquired, excavation of the Money Pit area can begin. When an artifact or anything of significant archeological interest is discovered, the Nova Scotia Museum must be notified and an archeologist must be on site to

evaluate the findings before work can resume. This is to ensure that all findings are preserved to the greatest possible extent. A qualified archeologist should be onsite, or at least nearby, during excavations to avoid lengthy delays when anything of significance is located.

12.0 USE OF GOOGLE SKETCHUP IN THE GEOPHYSICAL MODEL

In order to accurately portray the location of the many excavations, important findings and other aspects related to the Oak Island mystery, a 3D geophysical model was rendered. The use of the software Google SketchUp was used in order to achieve this model.

Videos are provided on a CD on the back cover. Provided on the CD are videos which describe the history of the Money Pit and soil profile, the borehole investigations and the vertical shafts, and most importantly the design of secant pile walls.

12.1 HISTORY OF OAK ISLAND - FINDINGS FROM PAST EXCAVATIONS

The history of the Money Pit and soil profile enables viewers to see what McGinnis and other treasure seekers saw as they began to excavate at the Money Pit. The video begins with a view of Oak Island and zooms in to the Money Pit (see Figure 31).

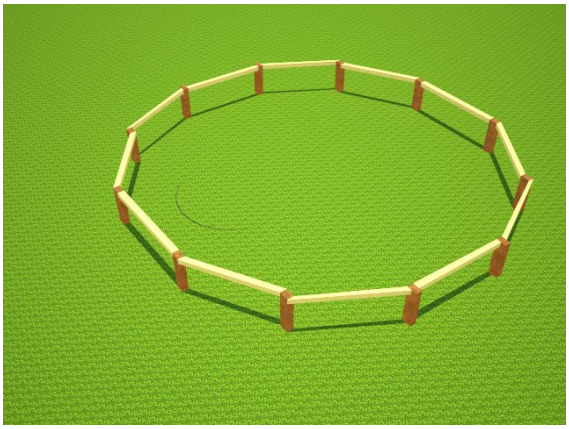

Figure 31 - Money Pit

The video then descends into the Money Pit displaying the soil profile as well as important finding that were discovered during the many excavations over the past 200 years. It includes oak platforms situated at every depth of 10 feet, along with findings of charcoal at 30 feet, putty at 40 ft, beach stones at 50 feet, and coconut fibre at 60 feet (see Figure 32).

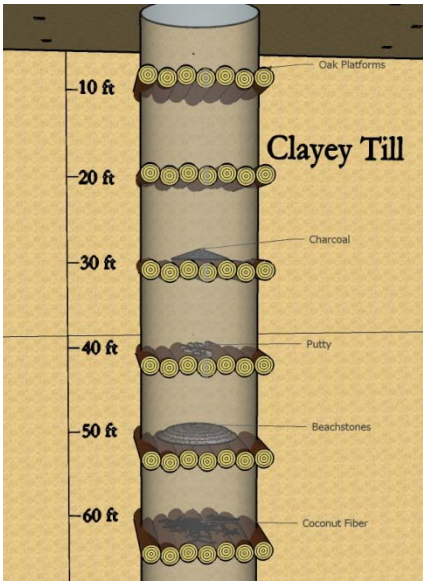

Figure 32 - Findings inside the Money Pit

After descending into the Money Pit, two tunnels are encountered: the original flood tunnel and the Halifax tunnel. The flood tunnel extends for several hundred feet until it merges into the feeder drain/filter bed system at Smith's Cove (see Figure 33).

Figure 33 - Flood Tunnel

After zooming out further at Smith's Cove, the five feeder drains are presented.

The video resumes to a shot inside the Money Pit. This time an inscribed stone is found at a depth of 93 feet which is inscribed with hieroglyphic symbols. It is assumed that the symbols translate to say "*Forty Feet Below Two Million Pounds Lie Beneath*". Shortly after, the Halifax Tunnel is shown and then a lower view shows the fissures and cracks filled with soil and

stagnant water in the broken anhydrite. At the base of which offset chambers are believed to be present (see Figure 34).

Figure 34 - Broken Anhydrite and Offset Chambers

12.2 BOREHOLE INVESTIGATIONS AND VERTICAL SHAFTS

In the past, there have been many boreholes drilled near the proximity of the Money Pit. There were also several vertical shafts which were constructed to investigate new findings and re-investigate the location of old findings.

The video shows the location of the many boreholes and vertical shafts both above and below the surface (Figure 35).

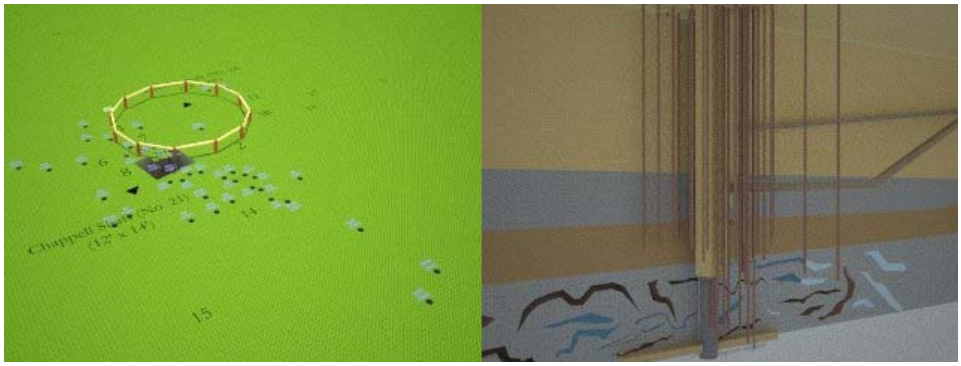

Figure 35 - Boreholes Above and Below Money Pit

12.3 Design of Secant Piles

Although the design of the secant piles does not show exactly how the secant piles are going to be installed, it does provide an easier understanding for the creation of an effective watertight barrier.

The video begins with the primary piles being added around the seventy foot diameter circumference. This is done using 37 primary piles, each having a 4foot diameter. The piles are built as one continuous segment for the entire depth (see Figure 36).

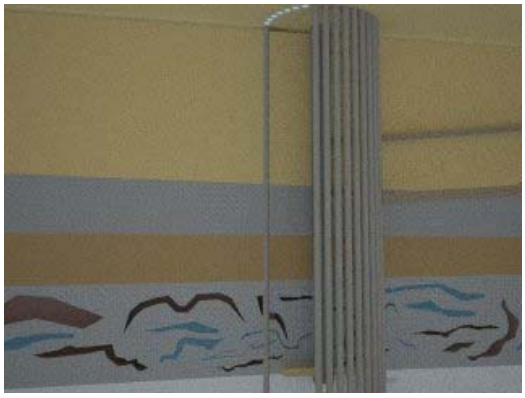

Figure 36 - Primary Piles

Next, secondary piles are added in between the primary where there are 37 piles, each having a 5ft diameter. As the secondary piles are being installed, we can see the primary and secondary piles overlap and interlock with each other which provides an effective water tight barrier so that no water can seep into our shaft (Figure 37).

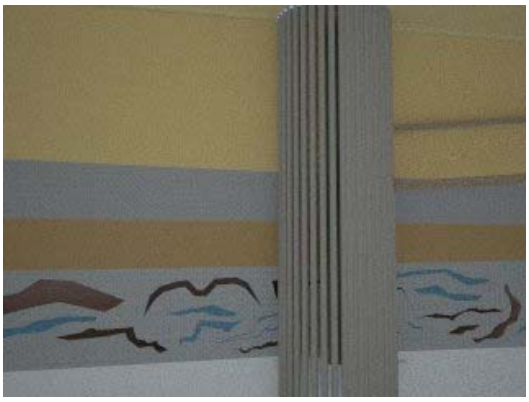

Figure 37 - Secondary Piles

The secant pile shaft is then seen fully encompassing anything within the 70 foot diameter surrounding the Money Pit (Figure 38).

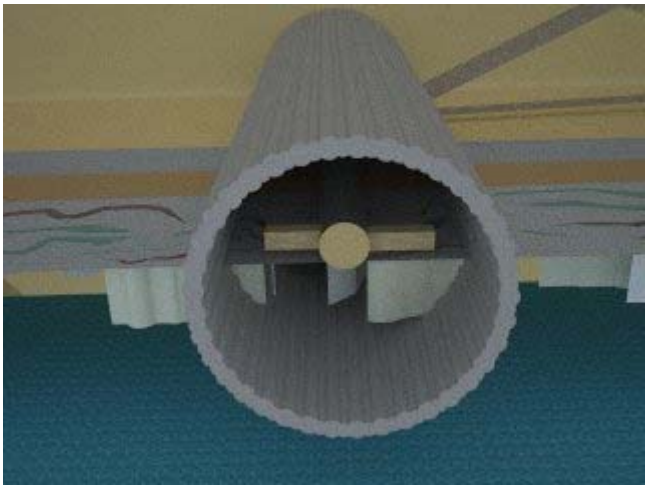

Figure 38 - Encompassing the 70 foot Diameter

Finally, the installation of four dewatering wells equipped with pumps around the periphery of the interior of the shaft is seen so the water pump verification test can take place (Figure 39).

Figure 39 - Water Pumps Installed

13.0 COST ANALYSIS

A cost analysis is important in design work so that the return on investments can be estimated. It is not economically justifiable to invest money on a project if it will not generate any return. Although the actual worth of the treasure is unknown, the historical significance of each finding is priceless. It has been stated that the actual worth of treasure is in the millions (Harris and MacPhie, 2005). However, since no real evidence has been found, this is mostly speculation. Additionally, it should be noted that the secant pile shaft was designed to function as a permanent works, enabling the shaft (and associated findings) to be converted into a heritage tourism site. This function has the potential to generate a great deal of return as a constituent of the Tourism Industry of Nova Scotia.

The total provisional cost estimate (in Canadian Dollars) for the completion of the project has been estimated as $20.3 million. As presented in Section 13.1, 13.2 and 13.3, this is based on the constituent costs of $550,000, $5 million, and $14.8 million for the investigation program, grouting perimeter and secant pile wall phases of the project respectively.

13.1 PRELIMINARY INVESTIGATION PROGRAM

A cost estimate for the preliminary investigation program is provided in Table 4. It is a provisional cost estimate, the unit costs and quantities of which were adopted from MacPhie (2001). The estimate is thus based on the value of the dollar in 2001.

The contract drilling services component is a conservative estimate, based on the scenario that nine Type I holes, six Type II holes, and nine Type III holes will be required; all of which will be drilled to a 220 foot depth. The projected drilling costs are connected to the time required for the boring, including the time required for exploration and testing and the movement of machinery from one hole location to another. It is assumed that on average, the drilling of each hole will take 50 hours. For a drilling operation carried out 10 hours per day, six days per week, the drilling of 24 holes will take 1200 hours, which translates into a five month operation.

Table 4 - Cost Estimate for the Preliminary Investigation Program

Task	Quantity	Unit Rate $CAD	Cost $CAD
Surveying			
Two-man crew	4 days	800/day	3200
Site Preparation			
Site Grading	-	1,000	1,000
Contract Drilling Services			
Drilling-9 Type I holes	1,980	45/foot	89,100
Drilling-6 Type I holes	1,320	45/foot	59,400
Drilling-9 Type III holes	1,980	55/foot	108,900
Camera and Lateral Drift			
Labour and equipment	120 days	1,200/day	144,000
Geotechnical Consultant			
Field Engineer	120 days	700/day	84,000
Logging and report write-up	-	8,000	8,000
Lab testing			
Microscopic examination	-	5,000	5,000
Other	-	5,000	5,000
Security			
Services 24 hours a day,	120 days	350/day	42,000
7 days per week	-		
		Total	**550,000**

13.2 GROUTING PERIMETER

During the meeting with Hayward Baker on November 14th 2008, cost estimation for the grouting phase of works was discussed. They provided a preliminary estimate concerning the treatment required for complete site remediation. As presented in Section 6, the grouting phase calls for a minimum of 37 grout holes (primaries). However, additional grouting series (secondary and tertiary) may be required, resulting in as much as 100 grouting holes. Each hole consists of 220 feet of low mobility grout and 70 feet of pressure grout. Based on this scope of works, the Hayward Baker consultants quoted that it could cost in the range of $4 to 5 million dollars.

13.3 Secant Pile Shaft

The secant pile shaft is designed with 37 primary piles and 37 secondary piles. The price for each secant pile can be found in Table 5 and Table 6 shown below. These prices were obtained from Transport Quebec so they do not necessarily reflect the market prices in Nova Scotia. The quoted prices are based on 2008 figures for similar quantities of the same materials used. Therefore, inflation and the current market can affect the actual price when construction begins.

Table 5 - Cost Estimation for Primary Piles[7]

	# of bars	Size	Weight	Length	Total Mass	Quoted Price	Cost
Long Bar	14	20M	2.355 Kg/m	76 m	2,500 Kg	$2.45/Kg	$6,125
Stirrups	760	20M	2.355 Kg/m	4 m	7,160 Kg	$2.45/Kg	$17,542
Concrete	N/A	1.2 m^2	N/A	76 m	95 m^3	$347.60/$m^3$	$33,022
						Total =	**$56,689**

Table 6 - Cost Estimation for Secondary Piles[8]

	# of bars	Size	Weight	Length	Total Mass	Quoted Price	Cost
Long Bar	20	55M	19.625 Kg/m	76 m	30,000 Kg	$2.45/Kg	$73,500
Stirrups	760	35M	7.850 Kg/m	4.5 m	27,000 Kg	$2.45/Kg	$66,150
Concrete	N/A	2 m^2	N/A	76 m	150 m^3	$347.60/$m^3$	$52,140
						Total =	**$191,790**

It should be noted that the prices for each pile are based on a standard design for the entire structure. Using steel is not efficient since smaller diameter longitudinal bars can be used near the surface where there is not as much pressure (see Figure 22) and greater stirrup spacing can be employed near the surface where the shear stresses are not as large.

[7] Transports Quebec, 2008

[8] Ibid.

Using the costs determined from Tables 5 and 6, it can be shown that the total cost to build 37 primary and secondary piles would cost roughly $9.2 million (in 2008 figures). This includes the cost of materials, transportation and placement. This price is similar to the quote given by the specialists from Hayward Baker. They believed that the cost of a secant pile shaft in our situation would cost roughly $7 to $9 million (Hayward Baker, 2008).

It is speculated that the cost of concrete might be more expensive since the construction of a secant pile shaft is very specialized and difficult. Therefore, an expert contractor in the field of secant pile shaft construction will be needed as well as a specialized working crew. The specialists from Hayward Baker believe that it would cost roughly $15,000 per day to hire a specialized working crew and that it takes roughly 5 days to complete one pile. Using these values, it can be assumed that it would cost almost $5.6 million dollars to complete the 74 piles needed. This is assuming that no problems or delays in construction occur. Hence, the total cost to build a secant pile shaft is roughly $14.8 million.

14.0 FUTURE CONSIDERATIONS

The secant pile shaft was designed to be durable and to last for the long-term. The concrete material specifications (presented in Section 10.2) are based on requirements from the Canadian Standards Association *A23.1-04: Concrete Materials and Methods of Concrete Construction/ Methods of Test and Standard Practices for Concrete* (2006). It is also designed in conjunction with the codes and regulations stipulated by the Canadian Standards Association *A23.3-04: Design of Concrete Structures* (2006). Based on these guidelines, the design life for the structure is expected to be between 50 to 75 years.

Past investigations have proven that without a doubt, man-made workings exists, which can be dated back to the early to mid 17th century at a depth of 200 feet below the surface. Therefore the structure can function as a Heritage Tourism site, regardless of the findings of the excavation.

It should be noted that the 70 foot diameter shaft that the design calls for is based on the assumption that all man-made workings will be completely encompassed by such a diameter. However, it is strongly advised that this diameter is reassessed and optimised based on the results obtained from the preliminary investigation program (as presented Section 3.0).

15.0 CONCLUSIONS

After extensive research on secant pile walls and earth grouting and recommendations by professionals at Hayward Baker, the design of a secant pile shaft can function as an effective watertight barrier and thus facilitate a deep excavation at the Money Pit to fully disclose whatever is housed at 200 foot depths and below. This should close a chapter which has been more than 200 years in the making.

Throughout the execution of this design project we have learned about the Oak Island Mystery; one of the world's most enduring and baffling mysteries. We have also learned how to design a water-tight deep shaft using a secant pile wall and earth grouting. The project required us to overcome a steep learning curve, since all of these technologies were unfamiliar to us. We learned how to conduct historical research, obtain relevant geotechnical data from borehole records, design a water-tight shaft to facilitate a deep excavation and withstand all lateral earth pressures.

Possibly the most important lesson learnt was how to successfully work as a team to meet agreed upon objectives. The Design Project exercise taught us how to manage a project effectively by allocating tasks and by the creation of schedules to meet given deadlines.

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SOURCES OF DATA

Throughout the entire project, sources of data were primarily obtained from our advisor, Les MacPhie, which enabled us to proceed with our design. This includes all site measurements, existing conditions and reports by others. The data that was provided for us are:

Historical Background

The history of the Oak Island Mystery was summurized using the book written by Graham Harris and Les MacPhie entitiled: *Oak Island and its Lost Treasure(2005)*. *The Secret Treasure of Oak Island* by D'Arcy O'Connor was also used to provide historical relevance to our project.

Borehole Investigation

Proposed Phase 1 Investigation, Oak Island, Nova Scotia (2001) by Les MacPhie was needed in order to complete the section for borehole investigation. Geotechnical investigation reports (Golder & Associates, 1971 and Warnock Hersey International Ltd., 1969) were also provided by Les MacPhie. They were utilised to obtain datailed geotechnical and archaeological data from borehole records, as well as onsite conditions.

Analysis

Borehole records were obtained from the Golder & Associates, *Subsurface Investigation: The Oak Island Exploration, Nova Scotia* (1971). This report enabled us to provide a detailed analysis of the soil conditions below the surface of the Money Pit area. The Woods Hole Oceanographic Institution Report (1996) entitled *Oak Island Hydrogeology, Hydrography and Nearshore Morphology: July-August 1995 Field Observations* was also used to analyze the hydrological communication between the sea and Money Pit as well as the nature of the tidal signatures in the boreholes at and around the Pit.

Grouting & Secant Pile Design

Technical data for the design of the secant pile shaft was provided by the *A23.3-04: The Concrete Design Hand Book (2006)*. Standards for material that will be used in the design were provided by the *A23.1-04: The Concrete Design Hand Book (2006)*. The *Mechanics of Solids* by

Bicker was also used for the finite element analysis. Expert advice was provided for the design and geometries of the secant pile shaft during the meeting on November 12th with the consulting specialist of Hayward Baker. These consultants also provided expertise in the field of grouting as well as excellent advice on the logistics of work that needs to be done. Preliminary cost estimates for both the grouting and secant pile design were provided by the consulting specialist of Hayward Baker. Quoted prices for the cost of concrete and steel were found in the Transport Quebec report entitled: *La Direction des contrats et des resources matèrielles* (2008).

Codes and Regulations

All our design and specifications were based strictly on the appropriate codes and regulations. There are three main codes that were used in our project. These handbooks used are:

- Canadian Foundation Engineering Manual, Fourth Edition (2006)

  This manual provided specifications on the drilling strategies and equipment to be employed as well as specifications on the soil analysis performed.

- CSA A23.1-04: *The Concrete Design Hand Book* (2006)

  This manual provided detailed specifications on the material to be used for the design and construction of the secant pile shaft.

- CSA A23.3-04: *The Concrete Design Hand Book* (2006)

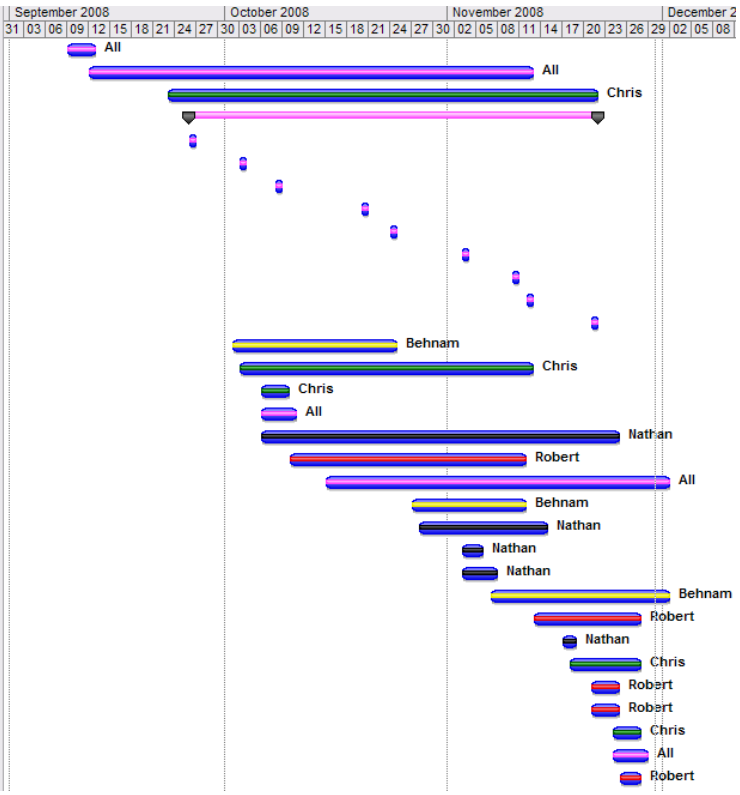
  This manual provided detailed specifications on the strength and design of the secant pile shaft.

DIVISION OF RESPONSIBILITY

Our team has divided our responsibility based on our backgrounds, strengths, and abilities. The work was divided in three sections: history and investigation, research and design, and AutoCAD and 3D modeling. All the team members attended the meeting with our advisor as well as meetings with professionals. All our work was verified and checked by team members. The specific division of work can be seen in the following:

GANTT CHART

		Task Name	Duration	Start	Finish	Resource Names
20	✓	Design Brief	4 days	Tue 09/09/08	Fri 12/09/08	All
19	✓	Research	45 days	Fri 12/09/08	Wed 12/11/08	All
1	✓	3D Geophysical Model	45 days	Tue 23/09/08	Fri 21/11/08	Chris
24	✓	**Advisor Meetings**	**42 days**	**Fri 26/09/08**	**Fri 21/11/08**	**All**
25	✓	Meeting with Les MacPhie	1 day	Fri 26/09/08	Fri 26/09/08	
26	✓	Meeting with Les MacPhie	1 day	Fri 03/10/08	Fri 03/10/08	
27	✓	Meeting with Les MacPhie	1 day	Wed 08/10/08	Wed 08/10/08	
28	✓	Meeting with Les MacPhie	1 day	Mon 20/10/08	Mon 20/10/08	
29	✓	Meeting with Les MacPhie	1 day	Fri 24/10/08	Fri 24/10/08	
30	✓	Meeting with Les MacPhie	1 day	Mon 03/11/08	Mon 03/11/08	
31	✓	Meeting with Les MacPhie	1 day	Mon 10/11/08	Mon 10/11/08	
32	✓	Meeting with Les MacPhie	1 day	Wed 12/11/08	Wed 12/11/08	
33	✓	Meeting with Les MacPhie	1 day	Fri 21/11/08	Fri 21/11/08	
16	✓	Historical Background	18 days?	Thu 02/10/08	Fri 24/10/08	Behnam
3	✓	Considered Methods of Recovery	30 days	Fri 03/10/08	Wed 12/11/08	Chris
5	✓	Soil Parameters	4 days	Mon 06/10/08	Thu 09/10/08	Chris
21	✓	Midterm Presentation	5 days	Mon 06/10/08	Fri 10/10/08	All
10	✓	AutoCAD Drawings	37 days	Mon 06/10/08	Mon 24/11/08	Nathan
11	✓	Grouting	24 days	Fri 10/10/08	Tue 11/11/08	Robert
23	✓	Final Report	34 days	Wed 15/10/08	Mon 01/12/08	All
18	✓	Jet Grouting Consideration	12 days?	Mon 27/10/08	Tue 11/11/08	Behnam
6	✓	Investigation Program	14 days	Tue 28/10/08	Fri 14/11/08	Nathan
9	✓	Soil Parameters	3 days	Mon 03/11/08	Wed 05/11/08	Nathan
7	✓	Soil Pressure Analysis	5 days	Mon 03/11/08	Fri 07/11/08	Nathan
17	✓	Grouting Program	17 days?	Fri 07/11/08	Mon 01/12/08	Behnam
12	✓	Secant Pile Design	11 days	Thu 13/11/08	Thu 27/11/08	Robert
8	✓	Site Conditions	2 days	Mon 17/11/08	Tue 18/11/08	Nathan
4	✓	Feasibility Assessment	8 days	Tue 18/11/08	Thu 27/11/08	Chris
13	✓	Materials	2 days	Fri 21/11/08	Mon 24/11/08	Robert
14	✓	Excavation	2 days	Fri 21/11/08	Mon 24/11/08	Robert
2	✓	Video Editing	4 days	Mon 24/11/08	Thu 27/11/08	Chris
22	✓	Final Presentation	5 days	Mon 24/11/08	Fri 28/11/08	All
15	✓	Cost Estimation	3 days	Tue 25/11/08	Thu 27/11/08	Robert



HOURS

Team Members	Chris	Behnam	Nathan	Rob
Week 1	September 1st - September 5th			
Class	2	2	2	2
Other	1	1	1	1
Total	**3**	**3**	**3**	**3**
Week 2	September 8th - September 12th			
Class	2	2	2	2
Other	6	4	4	2
Total	**8**	**6**	**6**	**4**
Week 3	September 15th - September 19th			
Class	3	3	3	3
Other	4	4	4	4
Total	**7**	**7**	**7**	**7**
Week 4	September 22th - September 26th			
Class	0	2	2	0
Other	8	5	6	5
Total	**8**	**7**	**8**	**5**
Week 5	September 29th - October 3rd			
Class	2	2	2	2
Other	21	10	20	15
Total	**23**	**12**	**22**	**17**
Week 6	October 6th - October 10th			
Class	2	2	2	2
Other	24	13	18	14
Total	**26**	**15**	**20**	**16**
Week 7	October 13th - October 18th			
Class	2	2	2	2
Other	5	5	5	8
Total	**7**	**7**	**7**	**10**
Week 8	October 20th - October 25th			
Class	2	2	2	2
Other	16	3	19	15
Total	**18**	**5**	**21**	**17**
Week 9	October 27th - October 31st			
Class	2	2	2	2
Other	5	5	20	7
Total	**7**	**7**	**22**	**9**

Week 10	November 3rd - November 7th			
Class	2	2	2	2
Other	16	6	21	20
Total	**18**	**8**	**23**	**22**
Week 11	November 10th - November 15th			
Class	2	0	2	2
Other	26	7	26	30
Total	**28**	**7**	**28**	**32**
Week 12	November 17th - November 22nd			
Class	2	0	2	2
Other	65	18	73	42
Total	**67**	**18**	**75**	**44**
Week 13	November 24th - November 28th			
Class	2	2	2	2
Other	63	34	61	39
Total	**65**	**36**	**63**	**41**
Week 14	December 1st - December 2nd			
Class	2	0	2	2
Other	28	9	32	22
Total	**30**	**9**	**34**	**24**
TOTAL	**308**	**140**	**317**	**242**

Note: All dates also include weekends

Appendix A: Photos of Oak Island and the Money Pit

Figure A1 - Aerial View of Oak Island, Nova Scotia
(MacPhie, 2001)

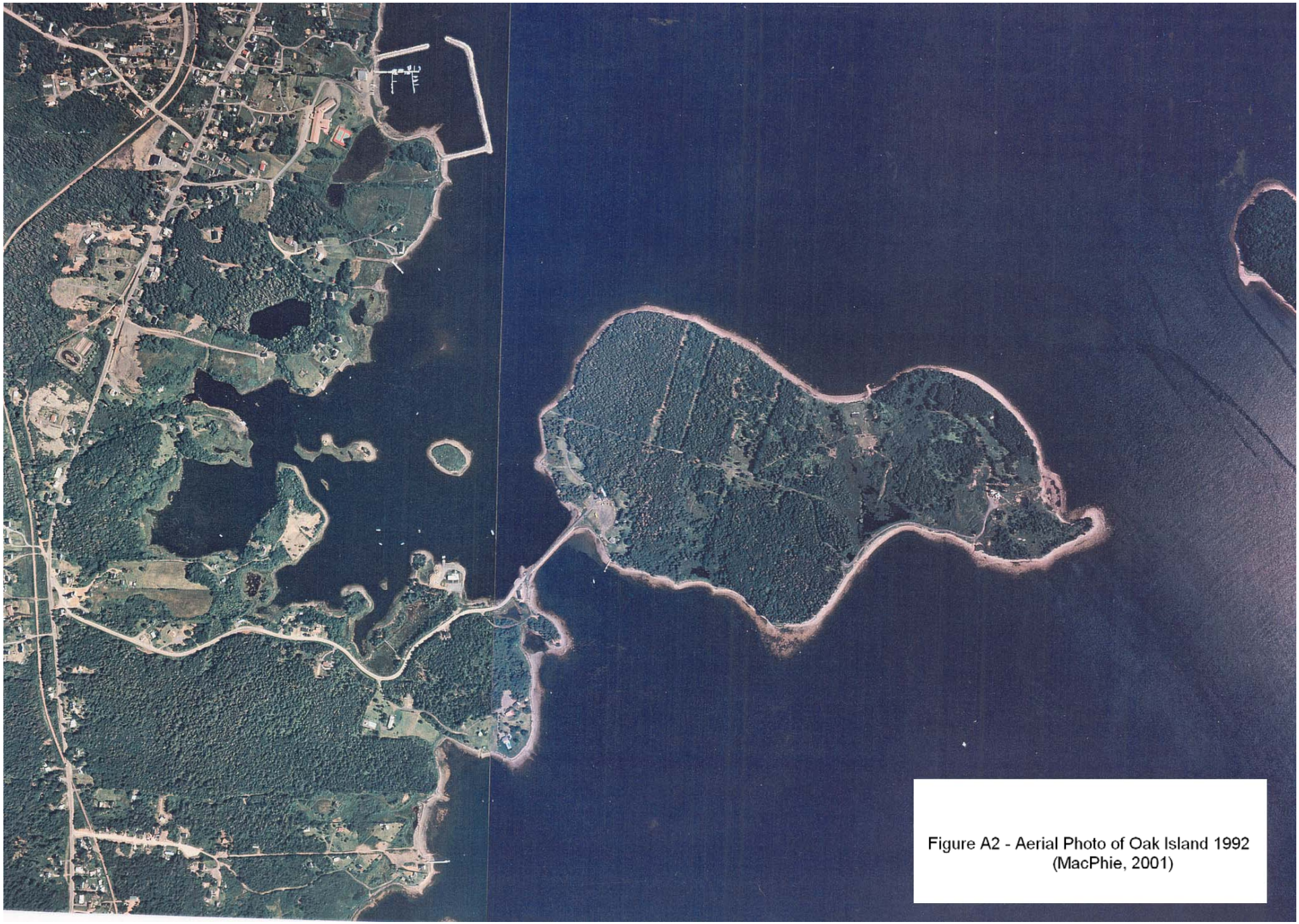


Figure A2 - Aerial Photo of Oak Island 1992
(MacPhie, 2001)

Figure A3 - Aerial Photo of Money Pit Area 1991
(MacPhie, 2001)

Figure A4 – Photo of the Money Pit 1998
(MacPhie, 2001)

Figure A5 – Photo of the Money Pit 1998
(MacPhie, 2001)

Appendix B: Borehole Records

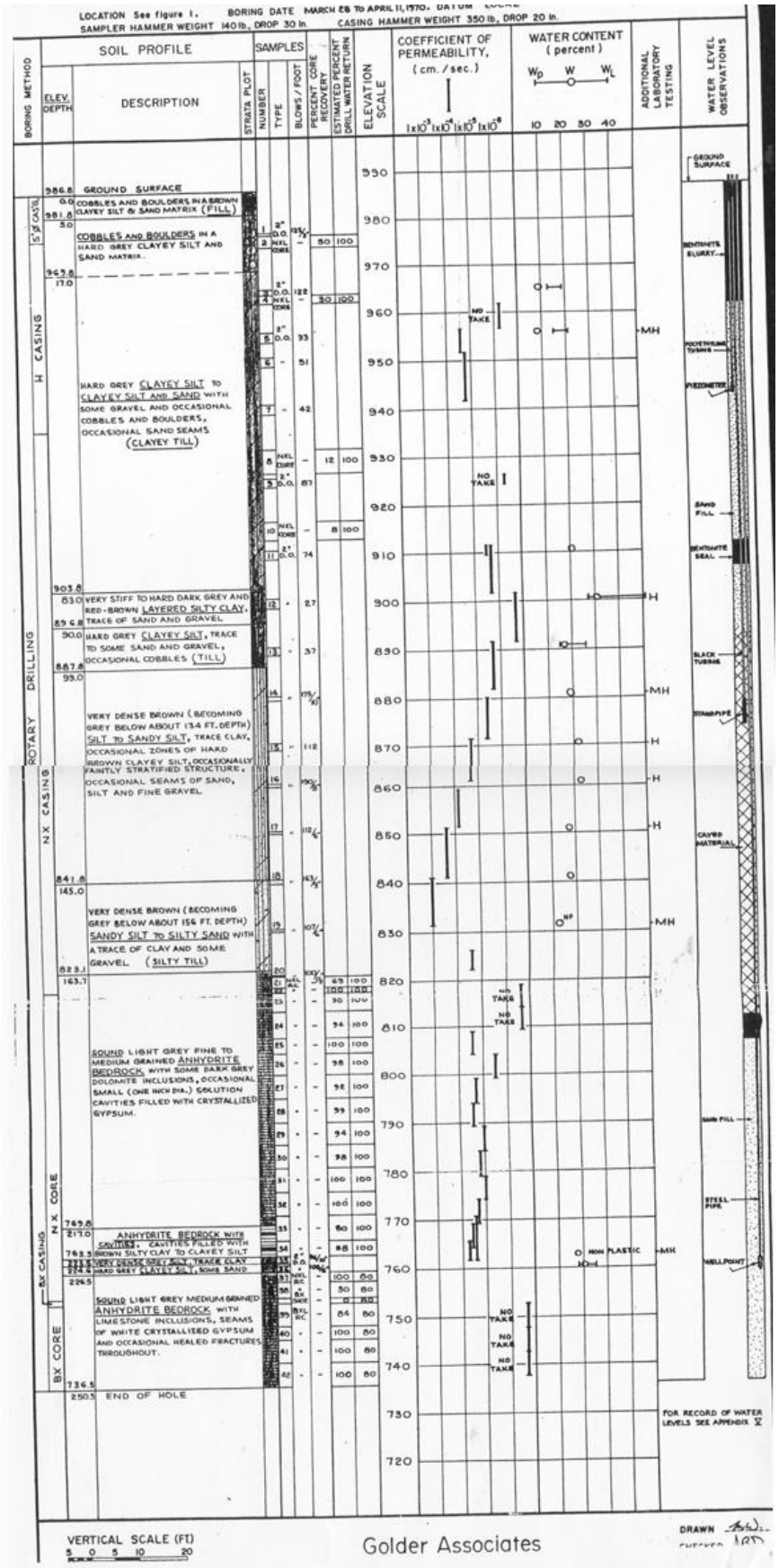


Figure B 1 - Borehole Record 101 (Golder & Associates, 1971)

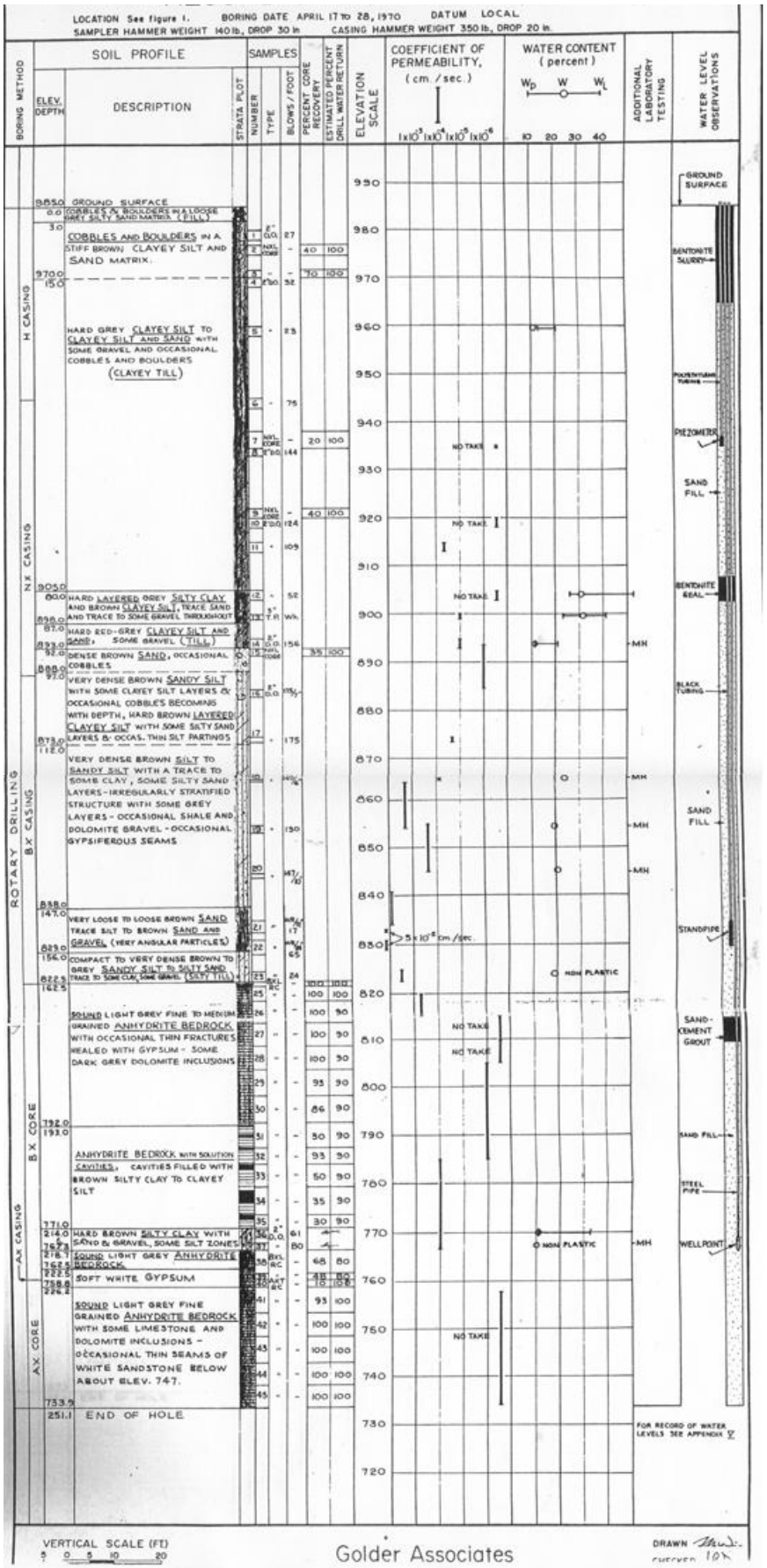


Figure B 2 - Borehole Record 102 (Golder & Associates, 1971)

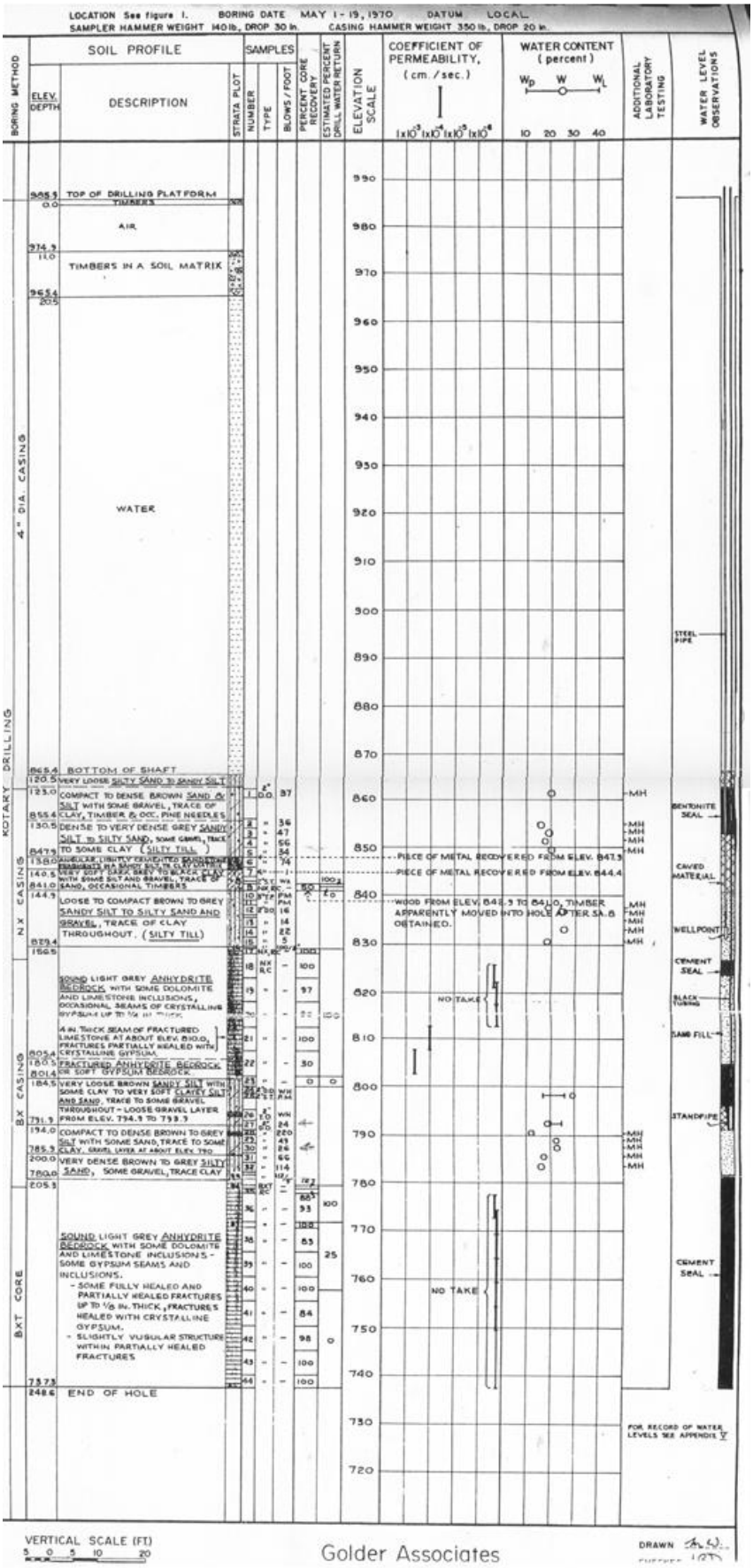


Figure B 3 - Borehole Record 103 (Golder & Associates, 1971)

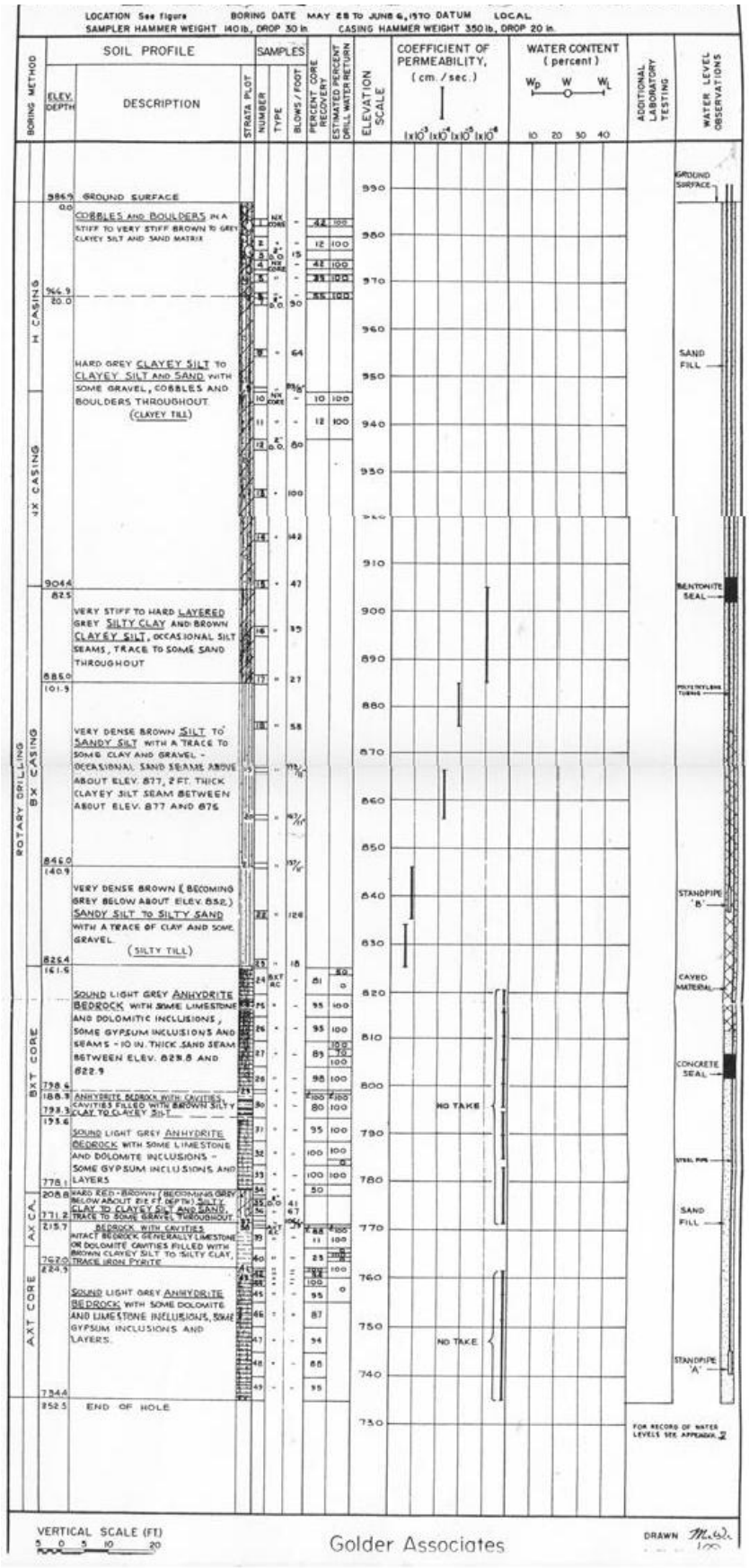


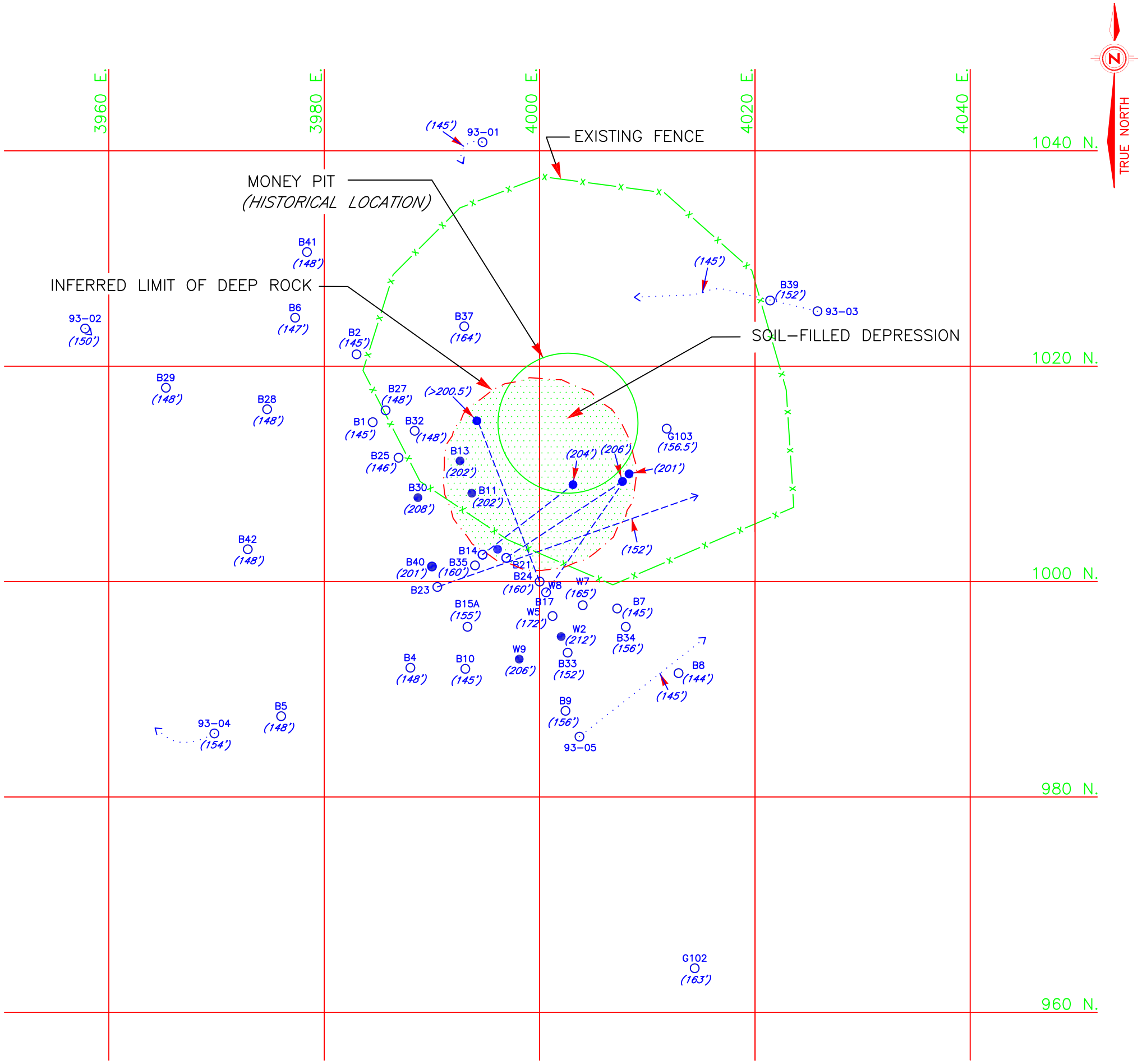
Figure B 4 - Borehole Record 104 (Golder & Associates, 1971)

Appendix C: Investigation Program

See AutoCAD files:

- Appendix C: Plan of Past Investigation Holes

- Appendix C: Plan of Initial Holes for Preliminary Investigation



LEGEND

B	BECKER DRILLING
G	GOLDER ASSOCIATES BOREHOLE
W	WARNOCK HERSEY BOREHOLE
93–	OAK ISLAND DETECTION PROGRAM BOREHOLE
B11 (202')	PAST VERTICAL HOLE WITH DEEP ROCK (GENERALLY 201 TO 208 FEET) *DEPTH TO ROCK (NOTE 2)*
B33 (152')	PAST VERTICAL HOLE WITH SHALLOW ROCK (GENERALLY 145 TO 155 FEET) *DEPTH TO ROCK (NOTE 2)*
B14 (204')	PAST INCLINED HOLE WITH DEEP ROCK *DEPTH TO ROCK (NOTE 2)*
B23 (152')	PAST INCLINED HOLE WITH SHALLOW ROCK *DEPTH TO ROCK (NOTE 2)*
93–03 (145')	PAST DETECTION HOLE WITH SHALLOW ROCK AND MEASURED PATH OF LATERAL DRIFT *DEPTH TO ROCK (NOTE 3)*

NOTES

1. LATERAL DRIFT WAS MEASURED ONLY IN HOLE W8 AND DETECTION HOLES 93–01 TO 93–05. IN THE REMAINING HOLES, THE PLOTTED LOCATIONS AT BEDROCK SUFACE MAY VARY SIGNIFICANTLY DUE TO LATERAL DRIFT DURING DRILLING.

2. THE DEPTH TO ROCK FOR HOLES 93–03 AND 93–05 IS SHOWN ALONG THE MEASURED PATH OF LATERAL DRIFT AT THE ACTUAL PLAN LOCATION WHERE ROCK WAS ENCOUNTERED. IN HOLE W8, ROCK WAS NOT ENCOUNTERED TO THE DEPTH OF 200.5 FEET AT WHICH THE HOLE WAS TERMINATED.

3. THE DEPTH TO ROCK IS WITH RESPECT TO EXISTING GROUND SURFACE WHICH IS ABOUT 10 FEET LOWER THAN ORIGINAL GROUND SURFACE IN THE AREA OF THE MONEY PIT.

SCALE

5 0 5 10 15FT

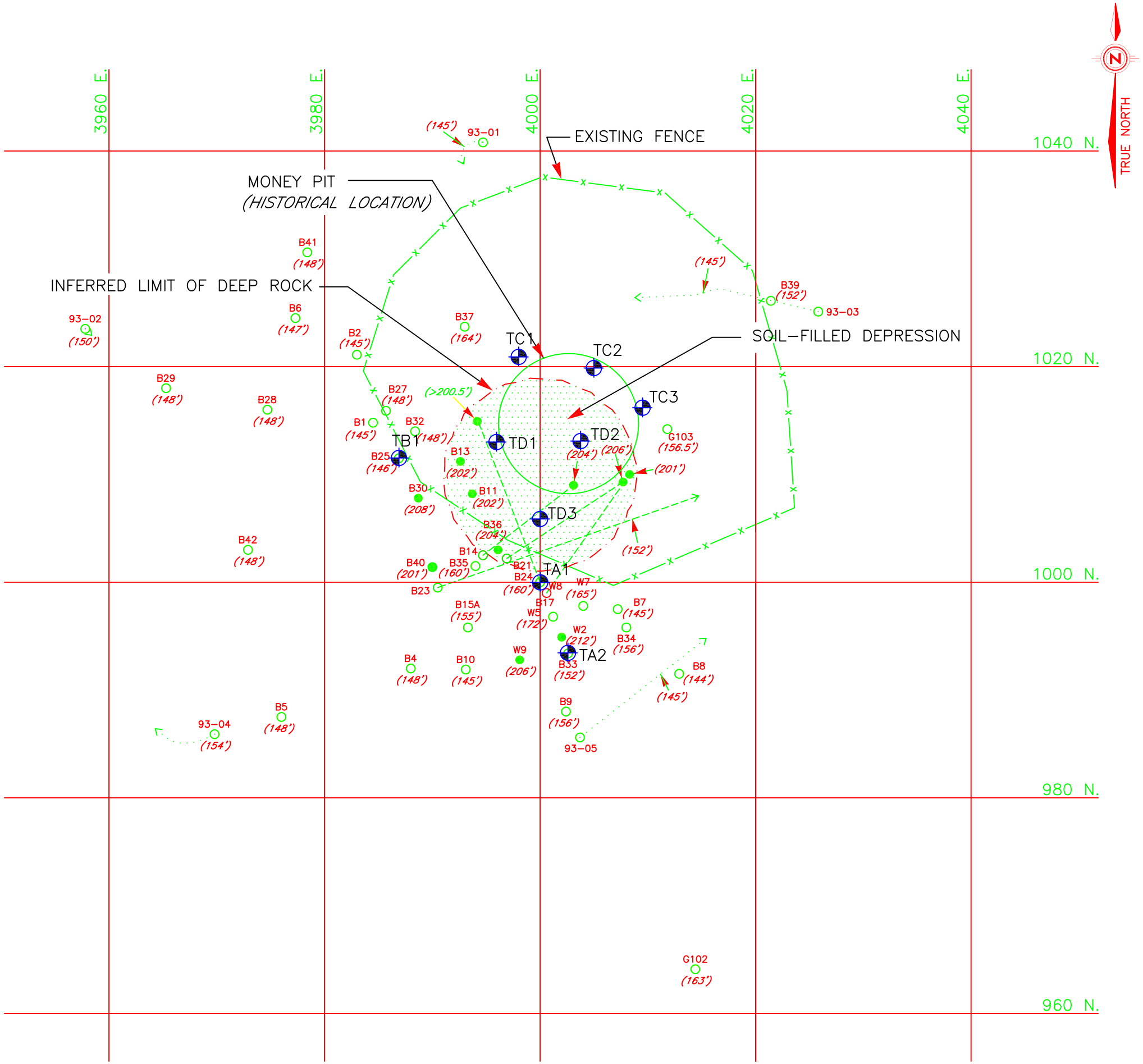
Topsoil SOLUTIONS

Drawing Title:
Plan of Past Investigation Holes

Drawn: N.J.R	Scale: 1" = 10'
Checked:	Date: 28-11-2008

Project:
Design of a Deep Shaft to Explore Underground Workings and Recover Potential Treasure, Oak Island, Nova Scotia

Source: MacPhie (2008a)



LEGEND

B	BECKER DRILLING
G	GOLDER ASSOCIATES BOREHOLE
W	WARNOCK HERSEY BOREHOLE
93–	OAK ISLAND DETECTION PROGRAM BOREHOLE
TA1	PROPOSED TYPE I BOREHOLE *TARGET A – BOREHOLE 1*
B11 (202')	VERTICAL HOLE WITH DEEP ROCK (GENERALLY 201 TO 208 FEET) *DEPTH TO ROCK (NOTE 2)*
B33 (152')	VERTICAL HOLE WITH SHALLOW ROCK (GENERALLY 145 TO 155 FEET) *DEPTH TO ROCK (NOTE 2)*
B14 (204')	INCLINED HOLE WITH DEEP ROCK *DEPTH TO ROCK (NOTE 2)*
B23 (152')	INCLINED HOLE WITH SHALLOW ROCK *DEPTH TO ROCK (NOTE 2)*
93–03 (145')	DETECTION HOLE WITH SHALLOW ROCK AND MEASURED PATH OF LATERAL DRIFT *DEPTH TO ROCK (NOTE 3)*

NOTES

1. LATERAL DRIFT WAS MEASURED ONLY IN HOLE W8 AND DETECTION HOLES 93–01 TO 93–05. IN THE REMAINING HOLES, THE PLOTTED LOCATIONS AT BEDROCK SUFACE MAY VARY SIGNIFICANTLY DUE TO LATERAL DRIFT DURING DRILLING.

2. THE DEPTH TO ROCK FOR HOLES 93–03 AND 93–05 IS SHOWN ALONG THE MEASURED PATH OF LATERAL DRIFT AT THE ACTUAL PLAN LOCATION WHERE ROCK WAS ENCOUNTERED. IN HOLE W8, ROCK WAS NOT ENCOUNTERED TO THE DEPTH OF 200.5 FEET AT WHICH THE HOLE WAS TERMINATED.

3. THE DEPTH TO ROCK IS WITH RESPECT TO EXISTING GROUND SURFACE WHICH IS ABOUT 10 FEET LOWER THAN ORIGINAL GROUND SURFACE IN THE AREA OF THE MONEY PIT.

SCALE
5 0 5 10 15FT

Topsoil SOLUTIONS

Drawing Title:
Plan of Initial Holes for Preliminary Investigation

Drawn: N.J.R	Scale: 1" = 10'
Checked:	Date: 28-11-2008

Project:
Design of a Deep Shaft to Explore Underground Workings and Recover Potential Treasure, Oak Island, Nova Scotia

Basemap: MacPhie (2008a)

Appendix D: Earth Pressures Calculations

ASSUMPTIONS

1) The specific gravity, Gs, for soil and rock is: **2.65**

2) The unit weight of water (γ_w) is **9.81 kN/m^3**

3) Soil above water table is completely dry, and soil below it is saturated.

4) The water table will be assumed to rest at the interface between cobbles and boulders stratum and the Clayey Silt to Clayey silt and Sand Stratum, i.e. 15 ft (4.6 m) below surface, for a conservative design.

5) The void ratio (e) for dry and saturated soil states are equal.

6) The dry strata have an average moisture content (w) of **20%**

7) When 70' (21 m) diameter water barrier centered about Money Pit is constructed, dewatered and excavated, a net subsurface pressure will be induced and act inward on the surface of the permanent barrier structure. This pressure distribution can be in the form of either active or at rest pressure.

However, due to the structural stability of a cylinder, deflection is expected to be within the limits for movement which would be necessary for active pressure conditions to ensue. This is because radial external stresses acting inward on a cylinder are translated into hoop and axial internal stresses and thus deflections are negligible (Terzaghi et al., 1996).

Thus, only at-rest pressures were designed for.

9) In short term pressure calculations, till overburden in its entirety is considered a stiff-hard clay (MacPhie, 2008b)

$\gamma_w = 9.81$ kN/m^3

Table B1: Physical Properties of Soil in Vicinity of Money Pit

Description	Soil Layer	Designation	Height (ft)	Depth (ft)	w (%)	PL (%)	LL (%)	PI (%)	N values	e	γ (kN/m^3)	Ko[1]
Fill	a	Unsaturated Till	3	3	-	-	-	-	263.5	0.5	17.0	0.5
Cobbles and Boulders	b		12	15	-	-	-	-	23.5	0.5	17.0	
Clayey Silt - Clayey Silt and Sand	c*	Saturated Till	65	80	16.3	16.3	28.1	11.8	83.8	0.4	21.1	0.5
Layered Silty Clay	d*		7	87	24.8	25	46.3	21.3	27	0.7	19.6	
Clayey Silt and Till	e*		10	97	16.3	13.2	23.2	10	77.3	0.4	21.1	
Sandy Silt, Layered Clayey Silt	f*		15	112	21.6	-	-	-	168.7	0.6	20.1	
Silt - Sandy Silt	g*		35	147	19.2	-	-	-	142	0.5	20.5	
Sandy Silt - Silty Sand w/ trace of Silty Till	h*		16	163	18.5	-	-	-	135.2	0.5	20.7	
Anhydrite Bedrock w/ thin fractures healed w/ gypsum	i*	Broken Anhydrite	30	193	20.5	18.5	25.5	7	Refusal	0.1[1]	24.4	0.25
Anhydrite Bedrock w/ solution cavities, cavities filled w/ brown silty clay - clayey silt	j*		33	226	14.4	14.5	28.8	14.3	Refusal	0.1[1]	24.4	

Sound Anhydrite Bedrock w/ limestone and dolomite inclusions	k*	Sound Anhydrite	N/A	236					Refusal	0.02	28.0[(2)]	0

*Soil layers with pores saturated with water (beneath water table level)

[(1)] MacPhie, 2008b

[(2)] Bell, 1999

Note: All data, with exception to void ratio e, unit weight γ, and at rest pressure coefficient K_o, were obtained from Golder Borehole Log 102 (Appendix B).

****Sample Calculations****

$e = w*G_s = n/(1-n)$

$\gamma_{dry} = (G_s*\gamma_w)/(1+e)$

$\gamma_{sat} = [(G_s+e)/(1+e)]*\gamma_w$

Where:

e = void ratio

n = porosity

γ_{dry} and γ_{sat} = dry and saturated unit weights, respectively

Clayey Silt - Clayey Silt and Sand Layer

$e = (16.5/100)*2.65 = 0.4$

$\gamma_{sat} = [(2.65+0.4)/(1+0.4)]*9.81 = 21.1\ kN/m^3$

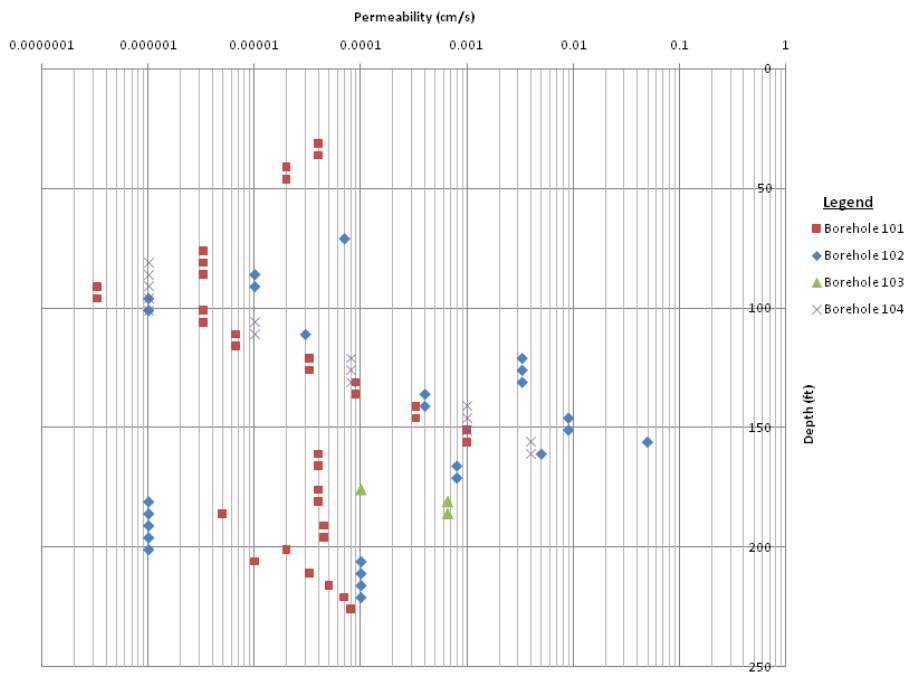
Anhydrite Bedrock w/ thin fractures healed w/ gypsum

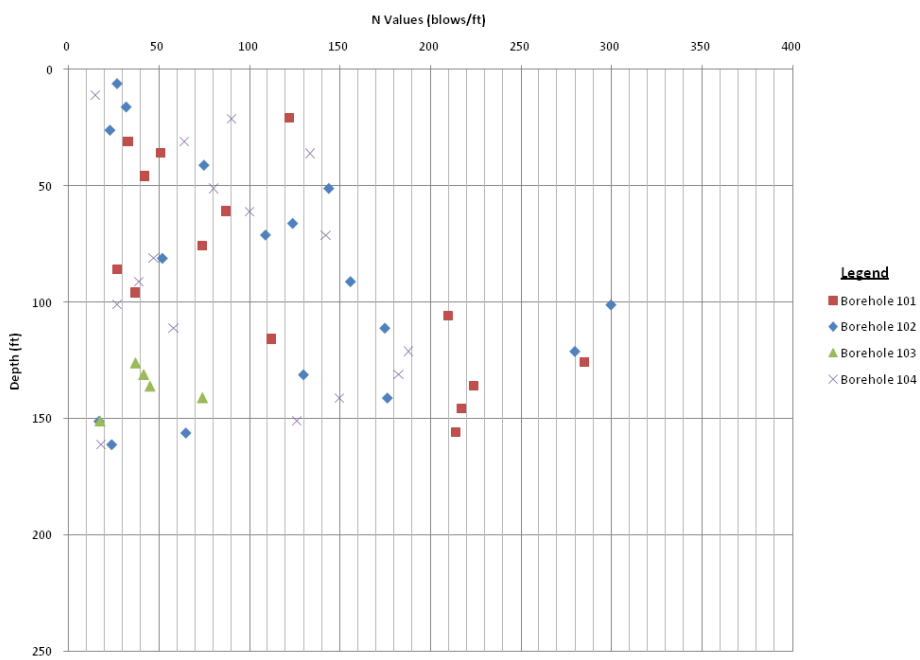
$n = 0.1$[(1)]

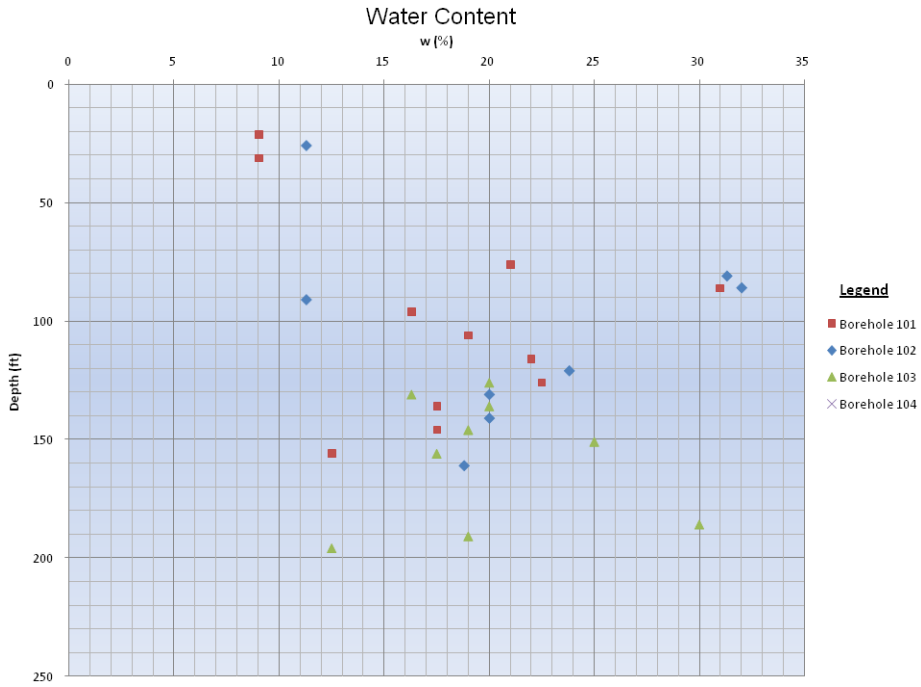
$e = 0.1/(1-0.1) = 0.1$

$\gamma_{sat} = [(2.65+0.1)/(1+0.1)]*9.81 = 24.4\ kN/m^3$

The following graphs were obtained from the borehole records (Appendix B):







SHORT TERM PRESSURES

According to Terzaghi et al. (1996), for ground excavations with vertical faces, the magnitude and distribution of earth pressures must be empirically derived using 'apparent' earth pressure distributions.

Further, the authors state that for excavation through till overburden (stiff-to-hard clay), the earth pressure distribution is as follows:

From 0 – 0.25H: σ_h increases linearly from 0 to (0.2 – 0.4γH)
From 0.25H – 0.75H: $\sigma_h = 0.2 – 0.4\gamma H$
From 0.75H – H: σ_h decreases linearly from (0.2 – 0.4γH) to 0

For the intermediate values, upper bound pressure value of **0.4γH** will be used for a conservative design. For unit weight, greatest value of each of the constituent unit weights (**21.1 kN/m^3**) is adopted for the till layer.

Depth of glacial till = 163 ft = 49.7 m

Depth (m)	Pressure (kPa)
0	0
12.4	417.5
24.9	417.5
49.7	0

LONG TERM PRESSURES

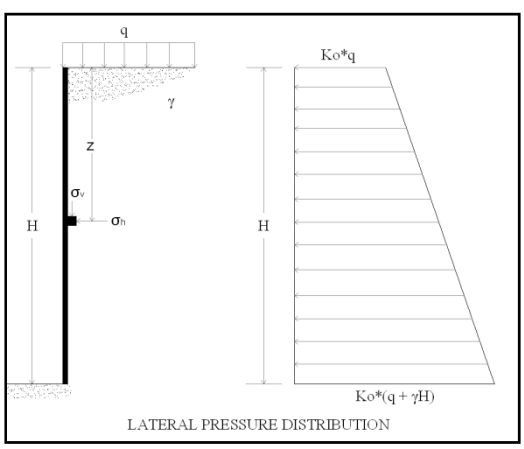
For an at rest pressure configuration by the finite element analysis:

The lateral stress applied for particular depth of soil, H, and surcharge load, q is as follows:

$\sigma_h' = K_o*(q + \gamma'H)$; see Figure 1 below

Where = □' = submerged unit weight = $\gamma - \gamma_w$

Long term pressures are calculated as the summation of the hydrostatic and effective soil pressures for a particular depth.



Finite Element Analysis of Lateral Earth Pressures (Das, 2004)

TOTAL EARTH PRESSURES

Total lateral earth pressures are equal to the short-term or long-term pressures; which ever is greater.

Table B2: Lateral Earth Pressure Distribution Data

Layer Description	Depth (m)	Earth Pressure (kPa)	Water Pressure (kPa)	Long Term Pressure (kPa)	Short Term Pressures (kPa)	Total Pressure (kPa)
Unsaturated Glacial Till $\gamma = 17$ kN/m^3	0.0	0.0	0.0	0.0	0.0	0.0
	0.9	7.8	0.0	7.8	30.7	30.7
	1.0	8.6	0.0	8.6	33.6	33.6
	2.0	17.0	0.0	17.0	67.2	67.2
	3.0	25.4	0.0	25.4	100.8	100.8
	4.0	33.8	0.0	33.8	134.4	134.4
	4.6	38.8	0.0	38.8	153.6	153.6
Saturated Glacial Till $\gamma = 21.1$ kN/m^3	5.0	43.0	4.2	47.2	168.0	168.0
	6.0	53.6	14.0	67.6	201.6	201.6
	7.0	64.2	23.8	88.0	235.2	235.2
	8.0	74.7	33.6	108.3	268.8	268.8
	9.0	85.3	43.4	128.7	302.4	302.4
	10.0	95.9	53.2	149.1	336.0	336.0
	11.0	106.4	63.1	169.5	369.6	369.6
	12.0	117.0	72.9	189.9	403.2	403.2
	13.0	127.6	82.7	210.3	417.5	417.5
	14.0	138.1	92.5	230.6	417.5	417.5
	15.0	148.7	102.3	251.0	417.5	417.5
	16.0	159.2	112.1	271.3	417.5	417.5
	17.0	169.8	121.9	291.7	417.5	417.5
	18.0	180.4	131.7	312.1	417.5	417.5
	19.0	190.9	141.5	332.4	417.5	417.5
	20.0	201.5	151.3	352.8	417.5	417.5
	21.0	212.1	161.2	373.3	417.5	417.5
	22.0	222.6	171.0	393.6	417.5	417.5
	23.0	233.2	180.8	414.0	417.5	417.5
	24.0	243.8	190.6	434.4	417.5	434.4

	24.4	248.0	194.4	442.4	417.5	442.4
	25.0	254.0	200.4	454.4	415.0	454.4
	26.0	263.9	210.2	474.1	398.2	474.1
	26.5	268.9	215.3	484.2	389.5	484.2
	27.0	274.1	220.0	494.1	381.4	494.1
	28.0	284.5	229.8	514.3	364.6	514.3
	29.0	294.9	239.6	534.5	347.8	534.5
	29.6	301.1	245.2	546.2	338.3	546.2
	30.0	305.2	249.4	554.6	331.0	554.6
	31.0	315.4	259.3	574.7	314.2	574.7
	32.0	325.6	269.1	594.7	297.4	594.7
	33.0	335.8	278.9	614.7	280.6	614.7
	34.0	346.0	288.7	634.7	263.8	634.7
	34.1	347.0	290.0	637.1	261.4	637.1
	35.0	355.9	298.5	654.4	247.0	654.4
	36.0	366.1	308.3	674.4	230.2	674.4
	37.0	376.4	318.1	694.5	213.4	694.5
	38.0	386.7	327.9	714.6	196.6	714.6
	39.0	396.9	337.7	734.7	179.8	734.7
	40.0	407.2	347.5	754.8	163.0	754.8
	41.0	417.5	357.4	774.8	146.2	774.8
	42.0	427.8	367.2	794.9	129.4	794.9
	43.0	438.0	377.0	815.0	112.6	815.0
	44.0	448.3	386.8	835.1	95.8	835.1
	44.8	456.6	394.7	851.3	82.2	851.3
	45.0	458.6	396.6	855.2	79.0	855.2
	46.0	468.9	406.4	875.3	62.2	875.3
	47.0	479.2	416.2	895.5	45.4	895.5
	48.0	489.6	426.0	915.6	28.6	915.6
	49.0	499.9	435.8	935.8	11.8	935.8
	49.7	507.0	442.5	949.5	0.0	949.5
Broken Anhydrite $\gamma = 24.4$ kN/m^3	50.0	255.4	445.6	701.1	0.0	701.1
	51.0	261.5	455.5	717.0	0.0	717.0
	52.0	267.6	465.3	732.9	0.0	732.9
	53.0	273.7	475.1	748.8	0.0	748.8
	54.0	279.8	484.9	764.7	0.0	764.7

	55.0	285.9	494.7	780.6	0.0	780.6
	56.0	292.0	504.5	796.5	0.0	796.5
	57.0	298.1	514.3	812.4	0.0	812.4
	58.0	304.2	524.1	828.3	0.0	828.3
	58.8	309.2	532.2	841.4	0.0	841.4
	59.0	310.3	533.9	844.2	0.0	844.2
	60.0	316.4	543.7	860.1	0.0	860.1
	61.0	322.5	553.6	876.0	0.0	876.0
	62.0	328.6	563.4	891.9	0.0	891.9
	63.0	334.6	573.2	907.8	0.0	907.8
	64.0	340.7	583.0	923.7	0.0	923.7
	65.0	346.8	592.8	939.6	0.0	939.6
	66.0	352.9	602.6	955.5	0.0	955.5
	67.0	359.0	612.4	971.4	0.0	971.4
	68.0	365.1	622.2	987.4	0.0	987.4
	68.9	370.5	630.9	1001.4	0.0	1001.4
Competent Anhydrite $\gamma = 24.4$ kN/m^3	69.0	0.0	632.0	632.0	0.0	632.0
	70.0	0.0	641.8	641.8	0.0	641.8
	71.0	0.0	651.7	651.7	0.0	651.7
	72.0	0.0	661.5	661.5	0.0	661.5
	73.0	0.0	671.3	671.3	0.0	671.3
	74.0	0.0	681.1	681.1	0.0	681.1
	75.0	0.0	690.9	690.9	0.0	690.9
	76.0	0.0	700.7	700.7	0.0	700.7
	77.0	0.0	710.5	710.5	0.0	710.5
	78.0	0.0	720.6	720.6	0.0	720.6
	79.0	0.0	730.1	730.1	0.0	730.1
	80.0	0.0	739.9	739.9	0.0	739.9

Note:

Starting at a depth of 24 metres (78.7 ft) below surface, the long-term pressures govern.

Maximum Earth Pressure = **1001.4 kPa**, occurring at a 68.9 metre (226 ft) depth (base of broken anhydrite layer).

<u>Sample Calculations for earth pressures at a 4.6 meter depth**</u>**

<u>Long Term Pressure</u>:

$q = 0.9\ m*17\ kN/m^3 = 15.3\ kPa$

$\sigma_h = (15.3 + 3.7*17)\ kPa * 0.5 = 38.8\ kPa$

<u>Short Term Pressure</u>:

$\sigma_h = (417.5/12.4)\ kPa/m * 4.6\ m = 153.6\ kPa$

<u>Sample Calculations for hydrostatic pressure at a 5.0 meter depth**</u>**

$\sigma_w = \gamma_w * H = 9.81\ kN/m^3 * (5 - 4.6)\ m = 4.2\ kPa$

Note: The water table is at 4.6 m below surface

Appendix E: Secant Pile Shaft Calculations

Design of Secant Pile Shaft

The design for the secant pile shaft was based on the A23.3-04 Concrete Design Handbook. All clauses referenced in this appendix were based on this manual.

Diameter = 70 ft

Height = 250 ft

Primary pile $f'c$ = 45 MPa

Secondary pile $f'c$ = 45 MPa

f_y of Steel = 400 MPa

Primary pile diameter = 4ft

Secondary pile diameter = 5ft

C/C spacing between piles = 3ft

Normal density concrete

Largest Pressure = 1000KPa @ 220 ft

Figure E1 – 3-D View of Secant Pile Shaft

Determination of Stress

Use of a finite element for a cylindrical pressure vessel.

Three (3) stresses to consider:
- σ_1 = Circumferential stress
- σ_2 = Longitudinal stress
- τ_{max} = Shear stress

Circumferential Stress:

$\sigma_1 = \frac{p\ x\ r}{t}$

where p = pressure

r = radius

t = thickness (minimum thickness)

@ Max. Pressure p = 1000 KPa ∴ $\sigma_1 = \frac{1000\frac{kN}{m^2}\ x\ 10.7m}{1.0m}$

r = 10.7 meters $\boxed{\sigma_1 = 10.7\ MPa}$

t = 1.0 meters

Longitudinal Stress:

$\sigma_2 = \frac{p\ x\ r}{2\ x\ t} = \frac{\sigma_1}{2}$

@ Max. Pressure σ_1 = 10.7 MPa ∴ $\sigma_2 = \frac{10.7MPa}{2}$

$\boxed{\sigma_1 = 5.4\ MPa}$

Longitudinal Stress:

$\tau_{max} = \frac{p\ x\ r}{2\ x\ t} = \frac{\sigma_1}{2} = \sigma_2$

@ Max. Pressure $\tau_{max} = \sigma_2 = 5.4$ MPa ∴ $\boxed{\tau_{max} = 5.4\ MPa}$

Design of Piles

Now that the compressive and shear stresses are determined, the resistances of both primary and secondary piles need to be checked.

Compression

$\sigma_1 > \sigma_2 \therefore$ limiting stress is governed by $\sigma_1 = 10.7$ MPa

Primary Piles

$f'c = 45$ MPa

$\alpha_1 = 0.85 - 0.0015 \times f'c = 0.783$ (Cl.10.1.7.)

$\emptyset_c = 0.65$ (Cl.8.4.2.)

$\sigma_{c1} = \emptyset_c \times \alpha_1 \times f'c = 0.65 \times 0.783 \times 45$

$\sigma_{c1} = 22.9\ MPa$

Since $\sigma_{c1} > \sigma_1 \therefore$ **O.K.**

Secondary Piles

$f'c = 45$ MPa

$\alpha_1 = 0.85 - 0.0015 \times f'c = 0.783$ (Cl.10.1.7.)

$\emptyset_c = 0.65$ (Cl.8.4.2.)

$\sigma_{c2} = \emptyset_c \times \alpha_1 \times f'c = 0.65 \times 0.783 \times 45$

$\sigma_{c2} = 22.9\ MPa$

Since $\sigma_{c2} > \sigma_1 \therefore$ **O.K.**

Therefore, it is okay to proceed with the concrete compressive strengths selected.

Design of Primary Piles

The primary piles are assumed to be the joints between the secondary piles and they are confined on two opposite faces. The reinforcing steel cage is placed in a rectangular section with a 1 foot width found about the centerline. This is so that the secondary piles do not drill into the primary piles reinforcing steel cage while being placed.

Try 14 – 20M longitudinal bars

$f'c$ = 45 MPa

Cover = 75 mm (Table 17 from CSA A23.1)

Assume 15M bar ties

Spacing between ties: $s = 100mm$

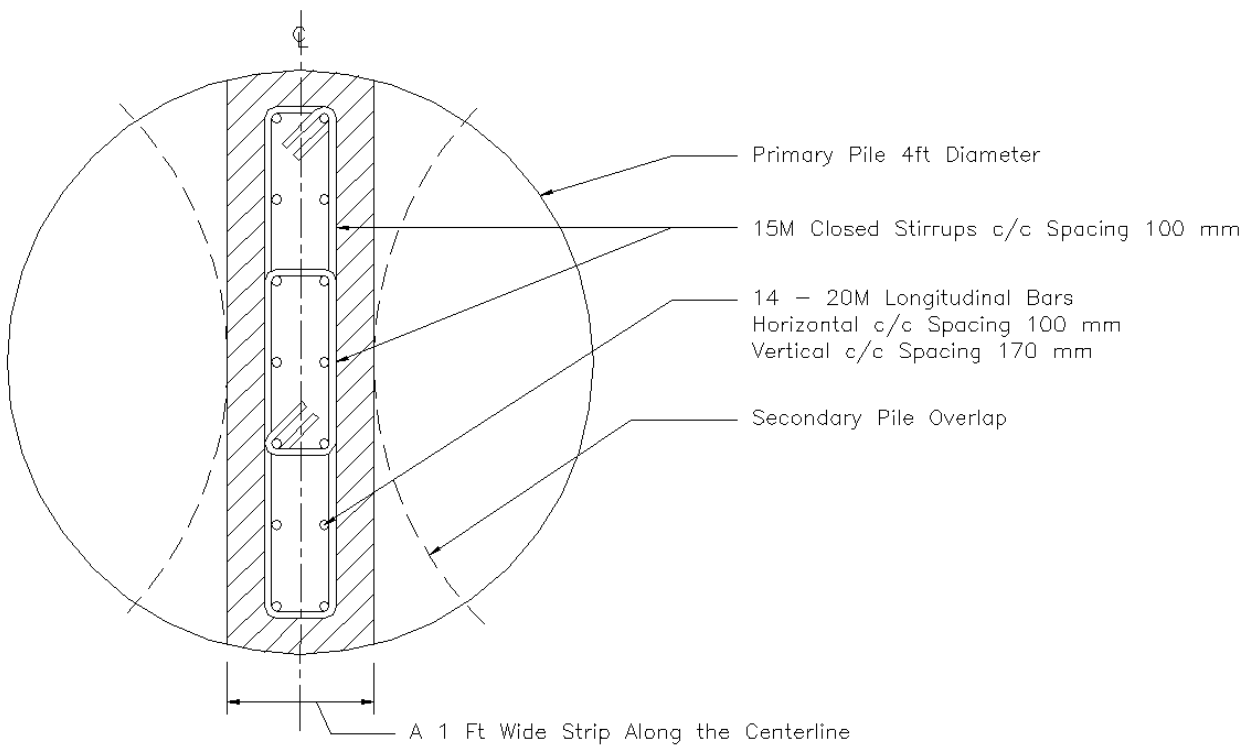


Figure E2 - Section View of Primary Pile

Compression for Primary Piles: (Clause 10)

Circumferential stress:

$\sigma_1 = 10.7$ MPa

Minimum thickness = 3.3ft ≈ 1 meter

For a 1 meter length of the pile, the circumferential force is equal to:

$P_f = \sigma_1 \times A = 10.7 \text{ MPa} \times 1m \times 1m = 10'700\text{kN}$

The factored circumferential stress is equal to:

$P_F = 1.25 \times 10'700 = 13'375\text{kN}$

The resisting compressive force is equal to:

$P_r = \emptyset_c \, \alpha_1 f'cA_g = 0.65 \times 0.783 \times 45\text{MPa} \times 1m \times 1m = 22'900\text{kN}$

Since $P_r \geq P_F$ ∴ **O.K.**

Longitudinal stress for Secondary Piles:

$\sigma_2 = 5.4$ MPa

Area of strip 4ft × 1ft ≈ $0.375m^2$

$P_f = \sigma_2 \times A = 5.4 \text{ MPa} \times 0.375m^2 = 2'025\text{kN}$

The section must also resist the factored dead load of the concrete and steel resting above.

$P_f =$ unit weight of concrete × volume + unit weight of steel × volume

$$P_f = 24\frac{\text{kN}}{\text{m}^3} \times 0.375\text{m}^2 \times 76.2\text{m} + (14 \times 2.355\text{kg/m} \times 76.2\text{m} \times 9.81\text{m/s}^2 + 760 \times 2.3550\text{kg/m} \times 4.0\text{m} \times 9.81\text{m/s}^2) \times \frac{1\text{N}}{1\text{kg} \times {}^{\text{m}}/_{\text{s}^2}} \times 10^{-3}$$

$P_f = 781kN$

The factored circumferential stress is equal to:

$P_F = 1.25 \times (2'025 + 781) = 3'508\text{kN}$

The resisting compressive force is equal to:

$P_{ro} = \emptyset_c \, \alpha_1 f'c \times (A_g - A_s) + \emptyset_s \, f_y A_s$ (Cl.10.10.4)

Assume a rectangular steel cage will be placed in the primary piles.

$P_{ro} = 0.65 \times 0.783 \times 45\text{MPa} \times (1200mm \times 300mm - 14 \times 300mm) + 0.85 \times 400\text{MPa} \times 14 \times 300mm$

$P_{ro} = 9'576kN$

For a tied column:

$P_{rmax} = 0.80\, P_{ro} = 0.8 \times 9'576kN = 7'661kN$

Since $P_{rmax} \geq P_F$ ∴ **O.K.**

<u>Check limits for A_s</u> (Cl.10.9.1. & Cl.10.9.2.)

$0.01A_g \leq A_s \leq 0.04A_g$

$A_s = 14 \times 300\, mm^2 = 4'200\, mm^2$

$A_g = 300 \times 1200\, mm^2 = 360'000\, mm^2$

$\rho = 0.012$ ∴ within max. & min. percent steel limits.

<u>Longitudinal bar spacing</u>

14 – 20M bars used in two columns

Longitudinal bar spacing:

Horizontal = 100 *mm* C/C Vertical = 170 *mm* C/C

Clear spacing:

Horizontal = 80 *mm* Vertical = 150 *mm*

Maximum longitudinal bar spacing in compression members must be less than 500*mm* ∴ **O.K.** (Cl.7.4.1.3)

Clear spacing must be less than or equal to 150*mm* between each laterally supported bar ∴ **NOT O.K.** (Cl.7.6.5.5)

The clear spacing between each longitudinal bar is less than 150 *mm*. However, not every longitudinal bar in the web is next to a laterally supported bar. Stirrups need to be added across the width to satisfy this clause and can be seen in the detailed drawing provided.

<u>Shear stress for Primary Piles:</u> (Clause 11)

τ_{max} = 5.4 MPa

Area that shear stress acts on $\approx 360'000mm^2$

Factor shear stress is equal to:

$V_f = 5.4\text{MPa} \times 0.36m^2 = 1'944kN$

The resisting shear force is equal to:

$V_r = V_c + V_s$ (Cl.11.3.3.) where V_c = Vertical component of concrete

V_s = Shear resistance of ties

Use simplified method: (Cl.11.3.6.3.)

$f_y = 400\text{MPa} \leq 400\text{MPa}$ ∴ **O.K.**

$f'_c = 45\text{MPa} \leq 60\text{MPa}$ ∴ **O.K.**

$\sqrt{f'c} = 6.7\text{MPa} \leq 8\text{MPa}$ ∴ **O.K.**

O.K. to proceed with simplified method for design.

$V_r \geq V_f$

$V_r = V_c + V_s$

$V_c = \emptyset_c \lambda\beta\sqrt{f'c}b_w d_v$ (Cl.11.3.4.)

$V_s = \frac{\emptyset_s A_v f_y d_v cot\theta}{s}$ (Cl.11.3.5.1)

Must determine value of d_v:

where: $\emptyset_c = 0.65$

$\emptyset_c = 0.85$

$\lambda = 1.0$

$\beta = 0.18$ $(A_v \geq A_{vmin})$

$\theta = 35°$

b_w = Least dimension of width

d_v = greater of $0.9d$ or $0.72h$

A_v = Area of stirrup

s = spacing

$\therefore d =$ distance between outermost compression fiber & $centroid\ of\ tension\ steel$

$= 1200 - 75 - 20 - 10 = 1095mm$

$\therefore\ d_v = 0.9d$ or 0.72h (larger or the two)

$= 0.9 \times 1095$ or 0.72×1200

$= 986mm$ or $864mm$

$\therefore\ d_v = 986mm$

$V_c = \ \emptyset_c\, \lambda\beta\sqrt{f'c}b_w d_v$

$= 0.65 \times 1.0 \times 0.18 \times \sqrt{45} \times 300 \times 986 \times 10^{-3} = 232kN$

Since $V_f > V_c \therefore$ ties needed as assumed

<u>Design of ties</u>

Assume 15M closed ties anchored to develop $f_y = 400$MPa are used.

Since 15M bars are used, the stirrups must be anchored by a standard hook around a long. rebar. (Cl.7.1.2)

A standard hook is defined in the CSA A23.1 Manual under Clause 6.6.2.2.

Assume tie spacing is 100mm

$A_v = 2 \times 200 = 400mm^2$

$$V_s = \frac{0.85 \times 400 \times 400 \times 986 \times cot35 \times 10^{-3}}{100} = 1'915kN$$

$V_r = V_c + V_s \geq\ V_f$

$V_r = V_c + V_s = 232kN + 1'915kN = 2'147kN$

Since $V_r > V_f \therefore$ **O.K.**

Maximum V_r (Cl.11.3.3.)

$V_{rmax} = 0.25 \times \emptyset_c \times f'_c \times b_w \times d_v$

$= 0.25 \times 0.65 \times 45 \times 300 \times 986 \times 10^{-3}$

$= 2'163kN$

Since $V_{rmax} > V_f \therefore \mathbf{O.K.}$

Check minimum reinforcement (s = 100mm) (Cl.11.2.8.2.)

$A_v \geq A_{vmin} = 0.06 \times \sqrt{f'_c} \times \frac{b_w S}{f_y}$

$= 0.06 \times \sqrt{45} \times \frac{300 \times 100}{400}$

$= 30\ mm^2$

Since $A_v = 400mm^2 \geq 30mm^2 = A_{vmin}$ $\therefore \mathbf{O.K.}$

The assumption that the minimum amount of ties was used is correct.

Maximum tie spacing

$= 0.125 \times \emptyset_c \times \lambda \times f'_c \times b_w \times d_v$

$= 0.125 \times 1.0 \times 0.65 \times 45 \times 300 \times 986 \times 10^{-3} = 1'082kN$

$V_f \geq 1'082kN$

$\therefore Max\ s = 300mm$ or $0.35d_v = 345mm$

$s \leq 300mm$ Since $s = 100mm \leq 300mm \therefore \mathbf{O.K.}$

Tie spacing must not exceed the smallest of: (Cl.7.6.5.2)

$s \leq 16d_b = 16 \times 20 = 320mm$

$s \leq 48d_t = 48 \times 20 = 960mm$

$s \leq h_{min} = 300mm$ Since $s = 100mm \leq 300mm \therefore \mathbf{O.K.}$

<u>Minimum tie spacing</u> (Cl6.6.5.2. of the CSA A23.1 Manual)

Assume $\alpha_{max} = 28mm$

$s \geq 1.4d_b = 28mm$

$s \geq 1.4\alpha_{max} = 40mm$

$s \geq 30mm$ Since $s = 100mm \geq 40mm \therefore \mathbf{O.K.}$

<u>Minimum longitudinal bar spacing</u> (Cl6.6.5.2. of the CSA A23.1 Manual)

Assume $\alpha_{max} = 28mm$

$s \geq 1.4d_b = 28mm$

$s \geq 1.4\alpha_{max} = 40mm$

$s \geq 30mm$ $\therefore \mathbf{O.K.}$ for horizontal and vertial spacing

Design of Secondary Piles

Try 20 – 55M longitudinal bars

$f'c$ = 45 MPa

Cover = 75 mm (Table 17 from CSA A23.1)

Assume 35M spiral bar ties

Distance between successive turns: $s = 100mm$

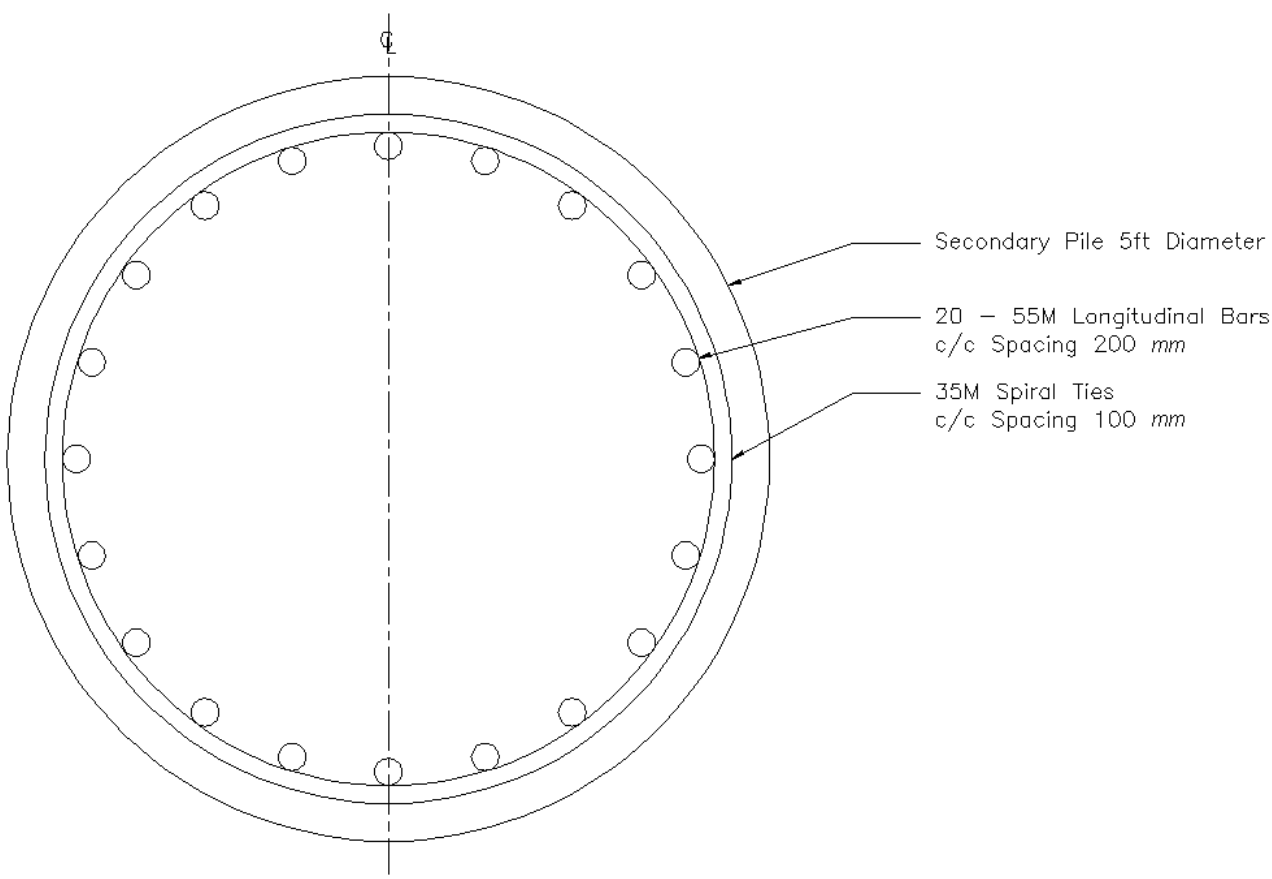


Figure E3 - Section View of Secondary Pile

Compression for Secondary Piles: (Clause 10)

Circumferential stress:

$\sigma_1 = 10.7$ MPa

Minimum thickness = 3.3ft ≈ 1 meter

For a 1 meter length of the pile, the circumferential force is equal to:

$P_f = \sigma_1 \times A = 10.7 \text{ MPa} \times 1m \times 1m = 10'700\text{kN}$

The factored circumferential stress is equal to:

$P_F = 1.25 \times 10'700 = 13'375\text{kN}$

The resisting compressive force is equal to:

$P_r = \emptyset_c \, \alpha_1 f'c A_g = 0.65 \times 0.783 \times 45\text{MPa} \times 1m \times 1m = 22'900\text{kN}$

Since $P_r \geq P_F$ ∴ **O.K.**

Longitudinal stress for Secondary Piles:

$\sigma_2 = 5.4$ MPa

Area of a 6ft segment ≈ $2m^2$

$P_f = \sigma_2 \times A = 5.4 \text{ MPa} \times 2m^2 = 10'800\text{kN}$

The section must also resist the factored dead load of the concrete and steel resting above.

$P_f =$ unit weight of concrete × volume + unit weight of steel × volume

$$P_f = 24\frac{\text{kN}}{\text{m}^3} \times 2\text{m}^2 \times 76.2\text{m} + (20 \times 19.625\text{kg/m} \times 76.2\text{m} \times 9.81\text{m/s}^2 + 760 \times 7.850\text{kg/m} \times 4.5\text{m} \times 9.81\text{m/s}^2) \times \frac{1\text{N}}{1\text{kg} \times \text{m}/\text{s}^2} \times 10^{-3}$$

$P_f = 4'215kN$

The factored circumferential stress is equal to:

$P_F = 1.25 \times (10'800 + 4'215) = 18'769\text{kN}$

The resisting compressive force is equal to:

$P_{ro} = \emptyset_c \, \alpha_1 f'c \times (A_g - A_s) + \emptyset_s \, f_y A_s$ (Cl.10.10.4)

$P_{ro} = 0.65 \times 0.783 \times 45\text{MPa} \times (1824147mm - 20 \times 2500mm) + 0.85 \times 400\text{MPa} \times 20 \times 2500mm$

$P_{ro} = 57'630kN$

For a spiral reinforcement:

$P_{rmax} = 0.85 \, P_{ro} = 0.85 \times 44'850kN = 48'989kN$

Since $P_{rmax} \geq P_F$ ∴ **O.K.**

<u>Short column analysis check:</u>

$\frac{kL}{r} \leq \frac{25 - 10\left({M_1}/{M_2}\right)}{\sqrt{{P_f}/{(f'c \times A_g)}}}$ (Cl.10.15.2.)

where: $k = 2.0 \; for \; cantilever$

$L =$ Unbraced length $(70m)$

$r = 0.25$ diameter for circle

$d =$ diameter $(1.525m)$

$\frac{M_1}{M_2} = 0 \; (Cantilever)$

$P_f = 10'800kN$

$A_g = 1824147mm$

$\frac{kL}{r} = \frac{2.0 \times 70000}{0.25 \times 1524} = 367$

$\frac{25 - 10\left({M_1}/{M_2}\right)}{\sqrt{{P_f}/{(f'c \times A_g)}}} = \frac{25 - 10(0)}{\sqrt{{10'800 \times 10^3}/{(45 \times 1824147)}}} = 68.9$

Since $\frac{kL}{r} \geq 68.9$ ∴ Short column analysis is not correct.

Piles should be laterally braced so that the unbraced length is 10 meters. This would give a $\frac{kL}{r}$ value equal to 52.5. This last check is not entirely true for our case since the secondary piles are fully braced by the primary piles throughout the full depth. Thus, the unbraced length is assumed to be equal to zero. Since the secondary piles are fully braced, the lateral deflections will be restrained. Therefore, the capacity will not be significantly reduced due to lateral deflections as is the case for slender columns.

Check to see if the entire structure can be treated as a short column:

where: $k = 2.0$ for cantilever

$L =$ Unbraced length $(70m)$

$r = 0.25$ diameter for circle

$d =$ diameter $(21.4m)$

$\frac{kL}{r} \leq 68.9$

$\frac{kL}{r} = \frac{2.0 \times 70}{0.25 \times 21.4} = 26.3$

Since $\frac{kL}{r} \leq 68.9$ ∴ **O.K.** to assume a short-column analysis for the entire structure.

<u>Check limits for A_s</u> (Cl.10.9.1. & Cl.10.9.2.)

$0.01A_g \leq A_s \leq 0.04A_g$

$A_s = 20 \times 2500\ mm^2 = 50'000\ mm^2$

$A_g = 1'824'147\ mm^2$

$\rho = 0.027$ ∴ within max. & min. percent steel limits.

<u>Longitudinal bar spacing</u>

20 – 55M bars used

Longitudinal bar spacing = 200*mm* C/C

Clear spacing = 145mm

The maximum longitudinal bar spacing in compression members must be less than 500*mm* ∴ **O.K.** (Cl.7.4.1.3.)

The clear spacing must be less than 150*mm* between each laterally supported bar ∴ **O.K.** (Cl.7.6.5.5.)

Shear stress for Secondary Piles: (Clause 11)

τ_{max} = 5.4 MPa

Area that shear stress acts on $\approx 2m^2$

Factor shear stress is equal to:

$V_f = 5.4\text{MPa} \times 2m^2 = 10'800kN$

The resisting shear force is equal to:

$V_r = V_c + V_s$ (Cl.11.3.3.) where V_c = Vertical component of concrete

V_s = Shear resistance of ties

Use simplified method: (Cl.11.3.6.3.)

$f_y = 400\text{MPa} \leq 400\text{MPa}$ ∴ **O.K.**

$f'_c = 45\text{MPa} \leq 60\text{MPa}$ ∴ **O.K.**

$\sqrt{f'c} = 6.7\text{MPa} \leq 8\text{MPa}$ ∴ **O.K.** **O.K.** to proceed with simplified method for design.

$V_r \geq V_f$

$V_r = V_c + V_s$

$V_c = \emptyset_c \lambda\beta\sqrt{f'c}b_w d_v$ (Cl.11.3.4.)

$V_s = \frac{\emptyset_s A_v f_y d_v cot\theta}{s}$ (Cl.11.3.5.1.)

where: $\emptyset_c = 0.65$

$\emptyset_c = 0.85$

λ = 1.0

β = 0.18 ($A_v \geq A_{vmin}$)

θ = 35°

b_w = diameter (Cl.11.2.10.3)

d_v = greater of $0.9d$ or $0.72h$

A_v = Area of stirrup

s = spacing

Must determine value of d_v:

Assume 11 bars are in tension:

Neglect compressive steel

$F_s = \emptyset_s A_s f_y = 0.85 \times 11 \times 2500 \times 400 = 9'350kN$

$F_c = \emptyset_c \alpha_1 f'c \times A_{cc} = 0.65 \times 0.805 \times 45 \times A_{cc} = 22.9\text{MPa} \times A_{cc}$

$$F_c = F_s \quad \rightarrow \quad 22'900 \frac{kN}{m^2} \times A_{cc} = 9'350kN \quad \therefore A_{cc} \approx 400'000mm^2$$

UsingA_{cc}, it was determined that 7 bars are in the compression region and that 2 bars are on the neutral axis.

Therefore, the assumption that 11 bars in compression is correct.

The centroid of the 11 bars in tension is 404mm from the outer most tension fiber.

$\therefore d =$ distance between outermost compression fiber & $centroid\ of\ tension\ steel$

$= 1524 - 404 = 1120mm$

$\therefore d_v = 0.9d$ or 0.72h (larger or the two)

$= 0.9 \times 1120$ or 0.72×1524

$= 1008mm$ or $1097mm$

$\therefore d_v = 1097mm$

$V_c = \emptyset_c \lambda\beta\sqrt{f'c} b_w d_v$

$= 0.65 \times 1.0 \times 0.18 \times \sqrt{45} \times 1524 \times 1097 \times 10^{-3} = 1'312kN$

Since $V_f > V_c \therefore$ ties needed as assumed

Design of ties

Assume 35M spiral ties anchored to develop $f_y = 400\text{MPa}$ are used.

Spiral ties must conform to (Cl.6.6.3) in the CSA A 23.1 Manual.

Assume tie spacing is 100mm

$A_v = 2 \times 1000 = 2000mm^2$

$$V_s = \frac{0.85 \times 2000 \times 400 \times 1097 \times cot35 \times 10^{-3}}{100} = 10'653kN$$

$V_r = V_c + V_s \geq V_f$

$V_r = V_c + V_s = 1'312kN + 10'653kN = 11'965kN$

Since $V_r > V_f$ $\therefore$ **O.K.**

Maximum V_r (Cl.11.3.3.)

$V_{rmax} = 0.25 \times \emptyset_c \times f'_c \times b_w \times d_v$

$= 0.25 \times 0.65 \times 45 \times 1524 \times 1097 \times 10^{-3}$

$= 12'225kN$

Since $V_{rmax} > V_f$ $\therefore$ **O.K.**

Check minimum reinforcement (s = 100mm) (Cl.11.2.8.2.)

$$A_v \geq A_{vmin} = 0.06 \times \sqrt{f'_c} \times \frac{b_w S}{f_y}$$

$$= 0.06 \times \sqrt{30} \times \frac{1524 \times 100}{400}$$

$= 125\ mm^2$

Since $A_v = 2000mm^2 \geq 125mm^2 = A_{vmin}$ $\therefore$ **O.K.**

The assumption that the minimum amount of ties was used is correct.

Maximum tie spacing

$= 0.125 \times \emptyset_c \times \lambda \times f'_c \times b_w \times d_v$

$= 0.125 \times 1.0 \times 0.65 \times 30 \times 1524 \times 1097 \times 10^{-3} = 4'075kN$

$V_f \geq 4'075kN$

$\therefore Max\ s = 300mm$ or $0.35d_v = 384mm$

$s \leq 300mm$ Since $s = 100mm \leq 300mm \therefore \mathbf{O.K.}$

Spiral reinforcement must have a minimum diameter of 6mm $\therefore \mathbf{O.K.}$ (Cl.7.6.4.2.)

The distance between turns of the spirals shall not exceed 1/6 of the core diameter: (Cl.7.6.4.3.)

Core diameter $= 1'304mm$ Since $s = 100mm \leq \frac{1}{6}(1304) = 217mm \therefore \mathbf{O.K.}$

The clear spacing between successive turns of a spiral shall not exceed $75mm$ nor be less than $25mm$.

Clear spacing = $100mm - 35mm = 65mm \therefore \mathbf{O.K.}$ (Cl.7.6.4.4.)

The ratio of spiral reinforcement must be greater than the value given by: (Cl.10.9.4.)

$$\rho_s = 0.45 \times (\frac{A_g}{A_c} - 1) \times \frac{f'_c}{f_y}$$

$A_g = \pi \times {D_{gross}}^2 = \pi \times 762^2 = 1824147mm^2$

$A_c = \pi \times {D_{core}}^2 = \pi \times 652^2 = 1335504mm^2$

$$\rho_s = 0.45 \times \left(\frac{1824147}{1335504} - 1\right) \times \frac{45}{400} = 0.0185$$

Volume of Steel $= A_s \times \pi \times D_{core} = 1000mm^2 \times 3.14 \times 1304mm = 4'096'637mm^3$

Volume of concrete $= \frac{\pi}{4} \times {D_{gross}}^2 \times s = \frac{\pi}{4} \times (1524mm)^2 \times 100mm = 182'414'693mm^3$

$$\frac{\text{Volume of Steel}}{\text{Volume of Concrete}} = 0.0225 \geq 0.0185 = \rho_s \therefore \mathbf{O.K.}$$

<u>Minimum longitudinal bar spacing:</u> (Cl.6.6.5.2. of the CSA A23.1 Manual)

Assume $\alpha_{max} = 28mm$

$s \geq 1.4d_b = 77mm$

$s \geq 1.4\alpha_{max} = 40mm$

$s \geq 30mm$ Since $s = 145mm \geq 77mm \therefore \mathbf{O.K.}$

<u>Minimum tie spacing</u> (Cl.6.6.5.2. of the CSA A23.1 Manual)

Assume $\alpha_{max} = 28mm$

$s \geq 1.4d_b = 49mm$

$s \geq 1.4\alpha_{max} = 40mm$

$s \geq 30mm$ Since $s = 100mm \geq 49mm \therefore \mathbf{O.K.}$

Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia

McGill University, Montreal, Quebec
CIVE 418 Design Project
Fall 2014

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1.0 Executive Summary

Geovation Engineering has developed a feasibility study report for a proposed deep excavation project intended to solve one of the world's most intriguing mysteries. For more than 200 years, treasure seekers have unsuccessfully attempted to retrieve what they claim to be buried treasure in the Money Pit on Oak Island, Nova Scotia. From the Money Pit's discovery in 1795, to the Truro syndicate in 1845, to present-day drillings, efforts to retrieve what lies deep below the surface have been carried out in vain. Despite a history of fruitless excavations, evidence of man-made workings in the Money Pit has routinely been discovered over the years, motivating generation after generation of treasure seekers to continue launching recovery attempts. The goal of this report is to demonstrate Geovation Engineering's unique, feasible, and safe design, which will hopefully put an end to the 200 year old mystery.

Through comprehensive research and meticulous design, the engineers at Geovation Engineering have developed a feasible solution to the problem of recovering the valuable artifacts present on Oak Island. This report presents the details of a proposed design consisting of safe, environmentally-friendly, and economical geotechnical approaches. The design includes a cylindrical 24 m diameter deep excavation, centered at the alleged location of the original Money Pit, extending to a depth of 50 m. With the help of geotechnical specialists and the use of modern technology, the design primarily rests on the implementation of ground improvement techniques and a steel shaft liner.

The use of two ground improvement techniques, jet grouting and permeation grouting, enabled Geovation Engineering to develop a system to prevent both groundwater and sea water from flooding the proposed excavation, avoiding an issue that has spelled the end for countless past recovery attempts. The ground improvement techniques used in conjunction with a 24 m diameter steel shaft liner will allow Geovation Engineering to successfully excavate to the location of the treasure. The steel shaft liner, installed with steel stiffening rings, will provide sufficient support to resist the lateral pressures at the 50 m depth, ensuring the excavation is safe for individuals accessing the shaft for tourism purposes for up to 30 years after the fact. Geovation Engineering has included in the design the use of dewatering wells and the installation of a concrete slab at the

base of the excavation to counteract the uplift pressures which will be encountered both during and after construction of the steel shaft liner. The dewatering wells will be implemented to decrease the water pressure during the construction of the steel shaft liner. The concrete slab is intended to alleviate the water pressure post-construction of the steel shaft liner.

In addition to the presentation of the technical design, this report includes a 2D representation of a 3D model. The completed 3D model includes additions to the shaft for the purpose of tourism. The progressive 2D representations include a breakdown of the construction sequence, which in turn comprises a detailed description of Geovation Engineering's proposed procedure for the archaeological investigation, the construction schedule associated with the construction sequence, and a detailed cost estimate of the proposed project.

This design project has been undertaken by McGill students three times in the past. In 2006, the first design group suggested a ground freezing technology. In 2007, another design group suggested constructing a cofferdam to block the water. In 2008, a third design group recommended the use of a secant pile shaft to uncover the underground workings. The various designs are well documented in their respective reports. As a result, Geovation Engineering did not consider alternative design methods to expose the underground workings but rather focused on one design approach.

In conclusion, Geovation Engineering has undoubtedly considered all of the technical aspects of retrieving the man-made workings believed to be present on Oak Island, Nova Scotia. Through the help of geotechnical specialists, the use of modern technology, and the implementation of rigorous design procedures, Geovation Engineering has created a solution that aims to settle the 200 year old mystery once and for all.

2.0 Introduction

2.1 Historical Background

The historical context seen below is sourced from the book authored by Harris and MacPhie (2005).

Figure 1: Map of Mahone Bay (Wikipedia, 2014)

The Oak Island Money Pit, located on a remote island in Nova Scotia as seen in Figure 1, has been a source of mystery since 1975 when Daniel McGinnis found a depression in the ground and began digging. Accounts say that as he and his friends started to dig, they encountered a previously dug circular shape of 4 m (13 feet) with the hard clay walls indented by tools used possibly by previous diggers. They continued to dig to 7.5 m (25 feet) before realizing that they required more heavy-duty machinery to continue and stopped. However they discovered man made

workings of oak log platforms at 3 m (10 feet) intervals which gave them hope that the Money Pit was in fact man-made.

Approximately 9 years later, the Onslow Syndicate after learning of the discovery, began their efforts in excavating the Money Pit. The Syndicate excavated to a depth of 28 m (93 feet) and made numerous discoveries; at a depth of 9 m (30 feet) charcoal was discovered. This was significant in that charcoal was used for purifying water on ships hundreds of years ago; furthermore, charcoal was used as gunpowder ingredient and fuel. The charcoal in the Money Pit was believed to be used to create convection currents that allowed the miners to breathe fresh air within the mine. This corresponds to the common ventilation practices during the seventeenth and eighteenth century Europe. Then further down the Money Pit at a depth of 12 m (40 feet), more astonishing discoveries baffled the excavators. A large amount of putty was found and this material was significant in that it was believed to have been used to seal leakage problems within the pit, which then further supports human presence within the Pit prior to the treasure hunt. At about 18 m (60 feet) down the pit, many bushels of coconut fibers were found. The interesting characteristic of coconut fiber is that it was once used as a cushion for packing ship's cargo. This led many to believe that the coconut fibers were present to package the hidden treasure. The natural occurrence of coconut fiber on Oak Island is nearly impossible since coconuts are a tropical product with the nearest tropical tree being approximately 2400 km away. Thus, this became the most convincing evidence that the pit is not of natural occurrence but rather man-made treasure storage.

It was at a depth of 28 m (90 feet) that the most interesting discovery was found. A large stone slab 91 cm x 38 cm (36 in x 15 in) weighing in at approximately 226 kg (500 pounds) was discovered. The symbols on the large stone were deciphered and read "Forty feet below two million pounds are buried." Shortly after the discovery of this stone at a depth of 28 m (93 feet), the excavation was forcibly stopped due to water infiltrating the shaft. Without a method of stopping the inflow of water, the Onslow Syndicate abandoned the project.

Nearly forty years later the hunt for treasure began once again with the Truro Company. The new plan was to avoid the water tunnels. They therefore dug a shaft about 3 m (10 feet) away from the Money Pit and went down to a depth of 33 m (109 feet) before experiencing the same water problems experienced earlier. However in this new shaft, an accidental discovery was made when a worker fell into the shaft - the water was discovered to be saline. Moreover, an observation was made that the water level in the shaft rose and fell in the same manner as the ocean. This then led the company to conclude that man-made Flood Tunnels existed, and that they connected to the ocean. The source of the tunnels was found to be at Smiths Cove, covering 44 m (145 feet) of shoreline. It was here that it was discovered that the soil layer trickled more water than anywhere else. The company then decided to build cofferdams in hopes of containing the water from rushing into the 5 inlets that merge into the Flood Tunnels leading to the Money Pit. However, this proved ineffective as the strong waves and the soft waterlogged nature of the ground made the dam extremely unstable. From the years of 1850 to 1866 numerous shafts have been made in hopes of finding a lateral entrance and avoiding the Flood Tunnels. However, all efforts have failed and each time the water collapsed the shafts.

The Oak Island Association was the next group to attempt the treasure excavation. They dug two parallel shafts about 5.4 m (18 feet) and 7.6 m (25 feet) away from the main Pit. The goal was to enter the Money Pit laterally from these two parallel shafts. Work progressed successfully until the excavators were about a foot away from entering the main Pit from one the parallel shafts; water flooded the lateral tunnel. The other parallel shaft was used and the same result took place. This time two men were sent down to see the cause of the water influx. As they were down there, the tunnel began to collapse and water rushed in; the two men barely escaped with their lives. The Money Pit also collapsed due to the water pressure pushing on the wooden supports that was keeping the Money Pit shaft open. This caused numerous objects to wash up.

The next group to excavate the treasure was the Halifax Company, who did not leave many records. The information that is known today was that they found the dimensions of the Flood Tunnels to be 0.76 m (2.5 feet) wide and 1.2 m (4 feet) high and inclined at an upward gradient of 6.6 degrees. The company had to abandon the project due to the lack of funding.

In 1895, the Oak Island Treasure Company was finally able to confirm the presence of the Flood Tunnels and took measures to block the water flow. They made borings on the expected path between the Money Pit and Smith's cove, where the source is believed to have been. They then used dynamite on these borings in hopes of blocking the path. At first it appeared that they had succeeded since after the explosion, bubbles were forming within the Money Pit. However, after a short-lived moment of joy, the Money Pit was once again filled with water.

That same year, William Chappell and Frederick Blair, the founders of the Oak Island Treasure Company, discovered the fragment of parchment about 0.8 cm long. In addition, through their drilling they hypothesized that there were coins surrounded by a cement vault at a depth of 47 m to 49 m (153 to 160 feet). After hitting an iron obstruction at a depth of 52 m (171 feet), the drilling stopped.

In 1899, the Oak Island Treasure Company discovered the presence of a second Flood Tunnel. After digging a shaft close to the Money Pit to a depth of 49 m (160 feet) the shaft began accumulating water, however, the Money Pit at first started losing water then was replenished. This clearly revealed a second Flood Tunnel coming from a lower source.

After this failure, all operations on the island ceased until William Chappell excavated the Chappell shaft. This shaft is located approximately 1.8 m (6 feet) away from the Money Pit and proved to contain the most discoveries made to date. The artifacts discovered include anchor flukes embedded in the wall of a shaft, a granite

boulder, an axe, poll pick remains of an oil lamp and boulders. Chappell and his associates concluded that these items belonged to the original workers of the Money Pit.

The next significant discovery came after the Triton Alliance constructed a cofferdam in 1970. This revealed numerous interesting findings such as wrought iron scissors, a wooden sled, and various iron tools underlying an elaborate U-shaped timber structure. This cofferdam eventually wore down due to severe weather conditions.

Throughout the next few years the Triton Alliance dug numerous excavations around the island without any success of recovering treasure. Amidst of all the struggles in attempting to uncover the treasure, legal troubles arose and halted the search. The location of the treasure is still very much uncertain at the present time. Currently on the History Channel, a series is being aired covering the daily activities of the Lagina brothers' attempts to recover the treasure of Oak Island. The title of the series is the "The Curse of Oak Island."

2.2 Objectives and Scope of Work

The objective of this project is to provide a solution to uncover the original workings or man-made chambers located in the Money Pit at a depth of 60 m. This is a feasibility study report outlining the preliminary design approach and conceptualizing the completed shaft and its usage after the underground workings are uncovered. Furthermore, this report also provides a cost estimate. A detailed engineering design is required before construction.

The cylindrical steel shaft has a diameter of 24 m and a depth of 50 m. The preliminary design outlines the recommended shaft thickness, the number of required stiffening steel ribs and their dimensions. The bottom of the steel shaft

provides a separate access to the assumed man-made chambers located at 60 m below ground level (10 m below the bottom of the steel shaft).

To deal with the challenging groundwater conditions and potentially more than one Flood Tunnel, jet and permeation grouting of the soil and bedrock is recommended. These ground improvement techniques are based on the soil type and characteristics. The diameter of grouting columns, their depth and overlap is outlined in the design.
A cost estimate based on quotations provided by industry consultants and other experts is prepared. The cost estimate includes the cost of preliminary site investigations required, a steel shaft liner, permeation and jet grouting, and excavation.

A three-dimensional conceptual design is provided for the usage of the shaft after the underground workings are uncovered. The complete Money Pit with wooden platforms every 3 m will be reconstructed according to the findings of Onslow Syndicate in 1804 (Harris & MacPhie, 2005). Furthermore, the original workings at a depth of 60 m will be reinstated. The shaft will then be made accessible to the public for tourism.

The site will have a museum that will be built beside the location of the shaft. The museum will contain any potential artifacts retrieved during excavation. The design of the museum is outside the scope of this project.

2.3 Design Overview

The design can be summarized in Figure 2, showing a cross section through the completed steel shaft.

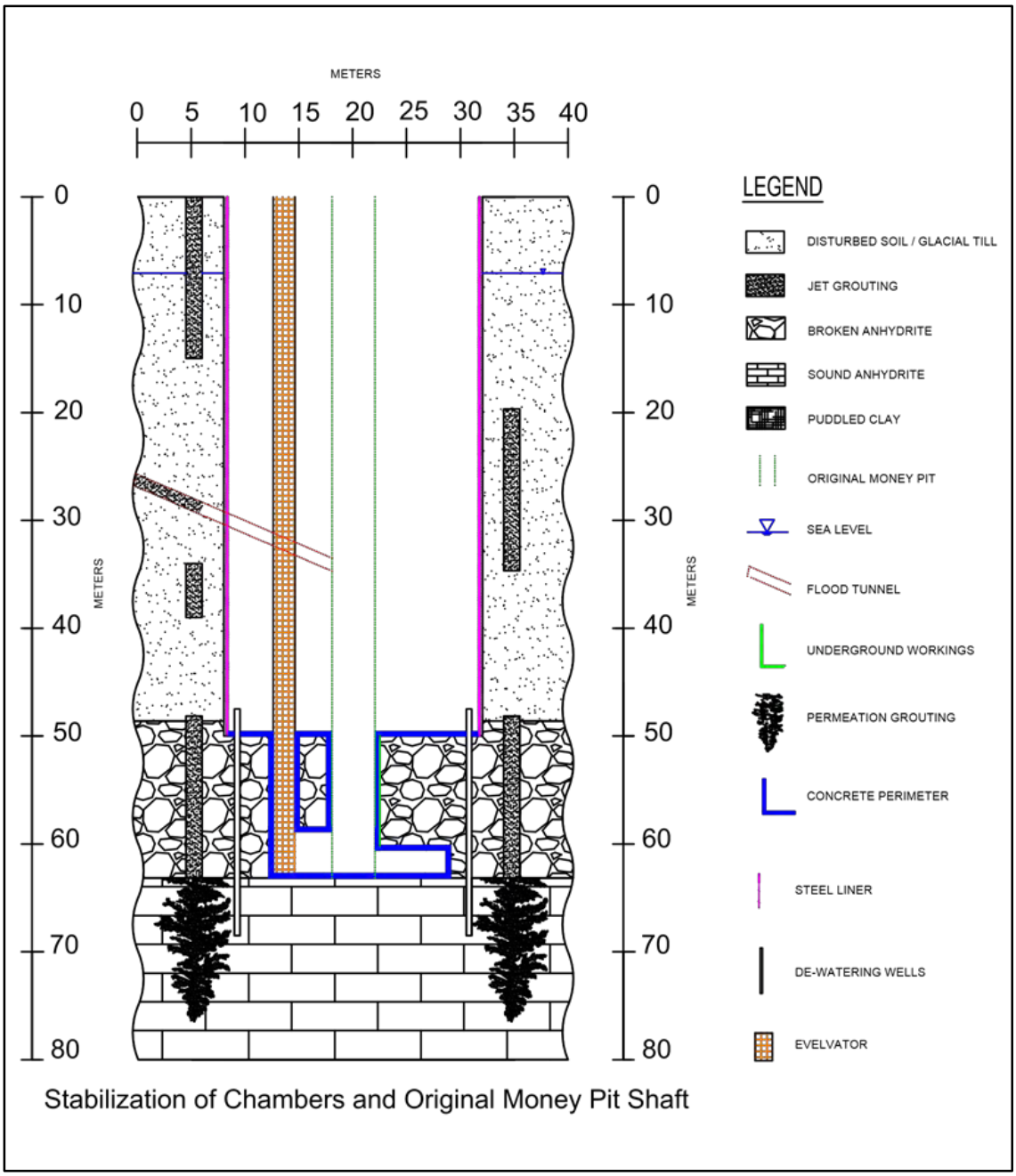

Figure 2: Cross Section of completed steel shaft

The subsurface conditions in Figure 2 above have been simplified into three main soil layers: glacial till, broken anhydrite and sound anhydrite bedrock. The recommended sequence of construction will be discussed in detail in Section 9.0.

The Money Pit, shown in green in Figure 2, has been dug several times. Due to the extensive disturbances from previous explorers, the upper part of the Money Pit is destroyed and there is a depression in the bedrock at the location of the Money Pit. Today, the precise location of the Money Pit is unknown. It is thought that the depression in the bedrock is associated with the Money Pit and therefore, it should be enclosed by the steel shaft. Previous research conducted by Oak Island Exploration Company in (1987) recommends a 24 m diameter steel shaft due to the uncertainty in locating the Money Pit. Furthermore, this shaft diameter will encompass previous shafts dug for the purpose of exploration.
However, our design extends the shaft to a location of 50 m below ground level and provides a separate access into the man-made chambers located 10 m below the bottom of the shaft. This design might allow for a smaller shaft diameter of 15 m instead of 24 m since the shaft does not extend to the depth of the man-made chambers. Therefore, Geovation Engineering recommends preliminary site investigations prior to detailed engineering design to determine the location of the depression in the bedrock. This will allow for a precise determination of the location the Money Pit and a potential reduction in the diameter of the shaft resulting in cost savings. This report will use a diameter of 24 m to allow for the design parameters of the worst-case scenario and consequently, the maximum cost estimate.

Figure 2 also shows the locations of the jet and permeation grouting in the glacial till, broken anhydrite, sound bedrock and Flood Tunnel, which will be explained in detail in Section 6.0 of the report. The man-made chambers will be made watertight by lining them with a thick concrete lining, shown in blue. Dewatering wells are recommended to relieve the uplifting water pressure during construction.

3.0 Site Conditions

Several geotechnical companies over the years have investigated the site conditions at the immediate location of the Money Pit and its surroundings. The borehole records available are 40 boreholes by Becker Drilling 1967, 9 boreholes by Warnock Hersey 1969 and 8 boreholes by Golder Associates 1970/71 (MacPhie, 2008).
Other investigations carried out include:

1. Smith's Cove Excavation in 1970 to confirm the presence of the filter bed system and timber structures located at smith's cove.
2. A geophysical survey of the site by Barringer Geoservice 1988
3. Geophysical studies carried out by Oak Island detection Program 1992/94
4. Hydrogeological studies of the east end of the island and hydrographical studies of the water surrounding the island by Woods Hole Oceanographic Institute 1995/96
5. Bathymetry determination around the east end of the Oak Island by Bedford Institute of Oceanography 1996/98.

3.1 Stratigraphic Profiles

A detailed stratigraphic profile shown in Figure 3 is used to demonstrate the soil layers at the perimeter of the proposed 24 m shaft diameter. The soil profile has been made using borehole records closest to the location of the perimeter. The sea level is 6.7 m below the ground surface and the soil layers are as follows (MacPhie, 2008):

1. Disturbed soil is soil that has been removed and backfilled by the Dunfield excavation that took place in 1965.
2. Hard brown to grey clayey till with boulders. This layer has low permeability with frequent boulders.
3. Hard grey, grey brown and brown stratified clayey silt and sandy silt (Till)
4. Dense brown and grey sandy till with boulders.

5. Broken anhydrite with gypsum and limestone layers encountered at depths of 47 to 61 m. This layer has frequent openings either filled with loose soils or open cavities.
6. Competent anhydrite bedrock

For the purpose of ground improvement design, a simplified stratigraphic profile is used, which is composed of three main soil layers namely: glacial till which includes disturbed soil, broken anhydrite and competent anhydrite bedrock.

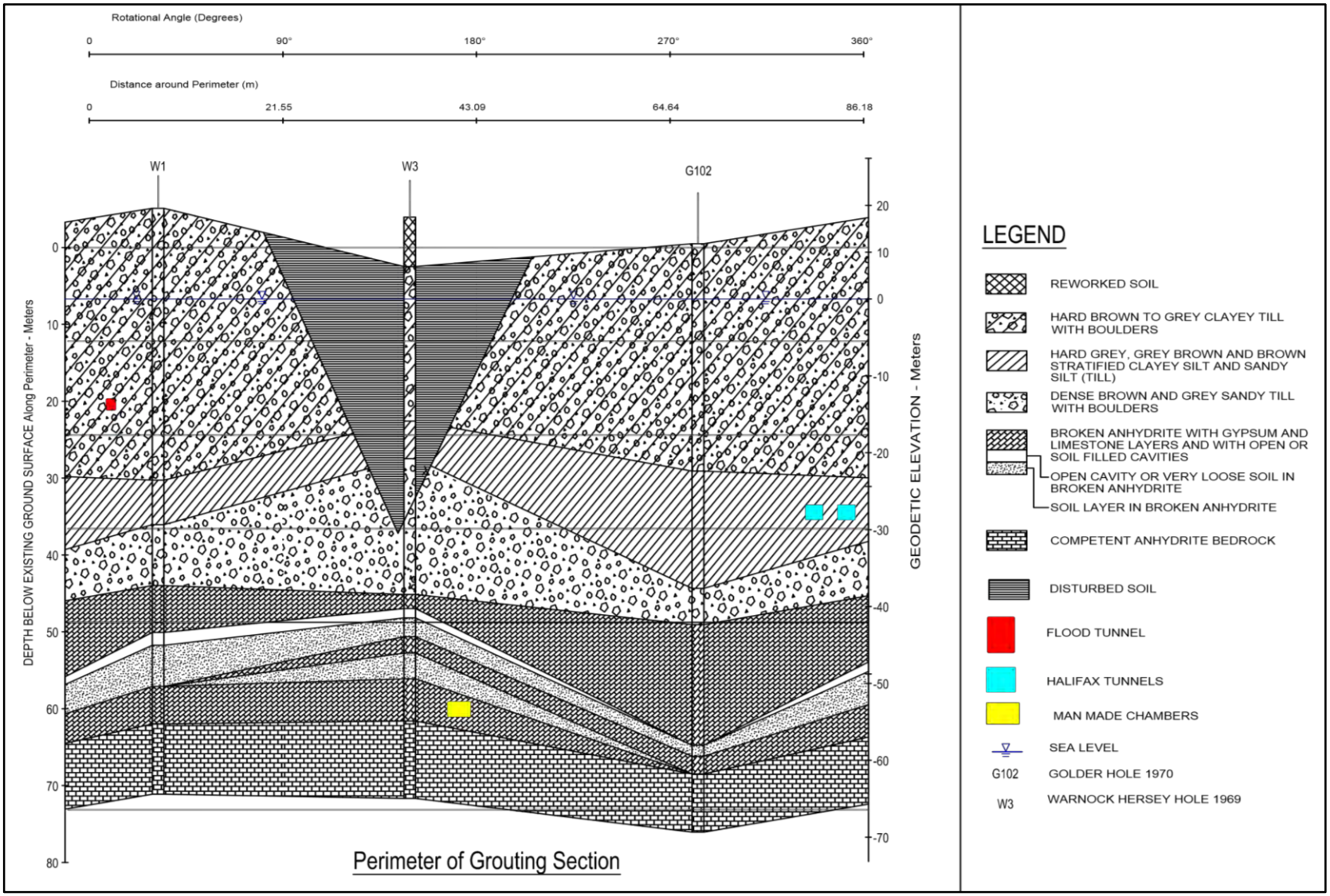

Figure 3: Detailed stratigraphic profile of perimeter

3. 2 Archaeological Evidence

The archaeological findings on Oak Island made through borehole records provided evidence of the presence of man-made workings. According to the technical report compiled by Les MacPhie (2008), the key archaeological features encountered from boreholes can be summarized in the tables from Appendix A. Nine of the forty six holes by Becker Drilling encountered anomalies, which are believed to have archaeological significance. Geophysical studies carried out by the Oak Island detection Program 1992/94 did not reveal any significant archaeological evidence.

3.3 Man-made workings

The man-made workings at the location of the Money Pit include the Halifax Tunnels, Flood Tunnels, and original man-made workings also referred to as chambers. The nature and history of each is explained in the following sections.

3.3.1 Halifax Tunnels

At a depth of approximately 37 m below ground surface, several horizontal tunnels with approximate dimensions of 1.5x2 m, were dug in an attempt to locate the Flood Tunnels (see Figure 4 and Figure 5 below) (Harris & MacPhie, 2005). It is believed that over the years, fresh water has seeped into the abandoned Halifax Tunnels from the groundwater. It is assumed that the tunnels are acting as drainage galleries and are capable of supplying large volumes of groundwater into the excavation, if the seepage is uncontrolled.

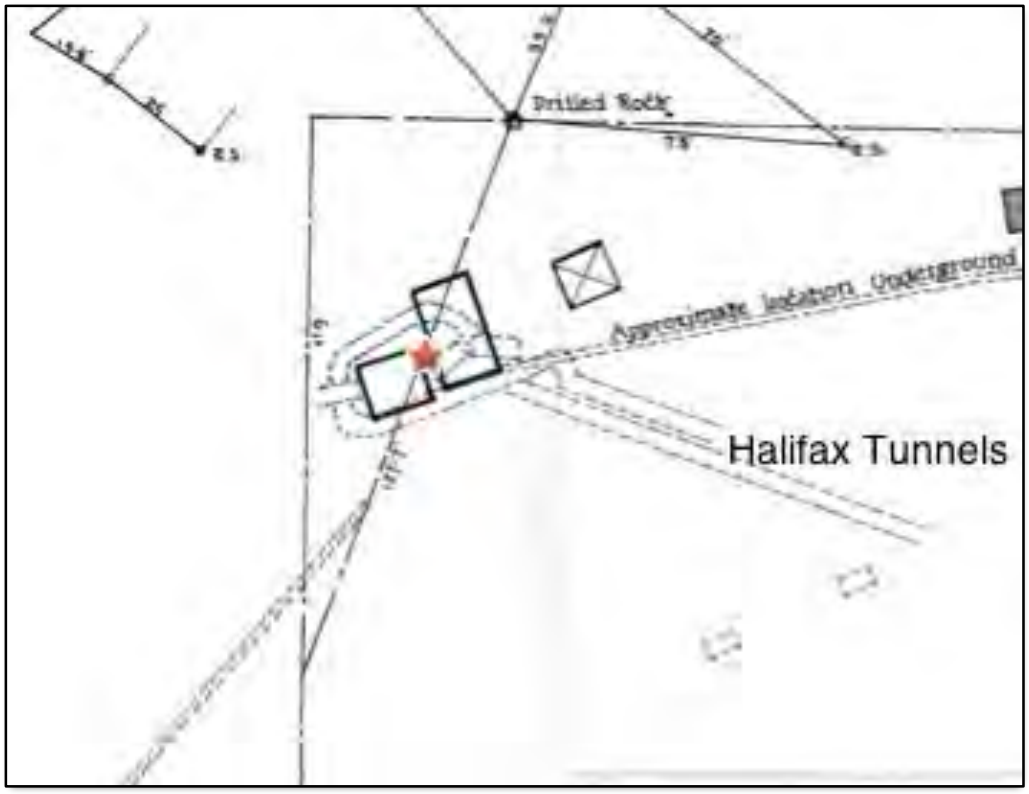

Figure 4: Plan of underground workings on Oak Island. The Halifax Tunnel shown with dotted lines. The red star represents the assumed location of the Money Pit(Bates, 1937)

Figure 5: Figure: Crew members in a drained Halifax Tunnel (Lamb, 2006)

3.3.2 Flood Tunnels

It is believed that the Money Pit shaft is linked with the ocean via Flood Tunnels (possibly more than one). When excavators reach a depth of about 34 m below ground surface, water enters the excavation at a rate so quick that pumps cannot counter the influx of water (see Figure 6). Shafts that intersect the Flood Tunnels get flooded with salt water that rises to levels corresponding to tide levels. Attempts to pump out the water have been unsuccessful (Harris & MacPhie, 2005).

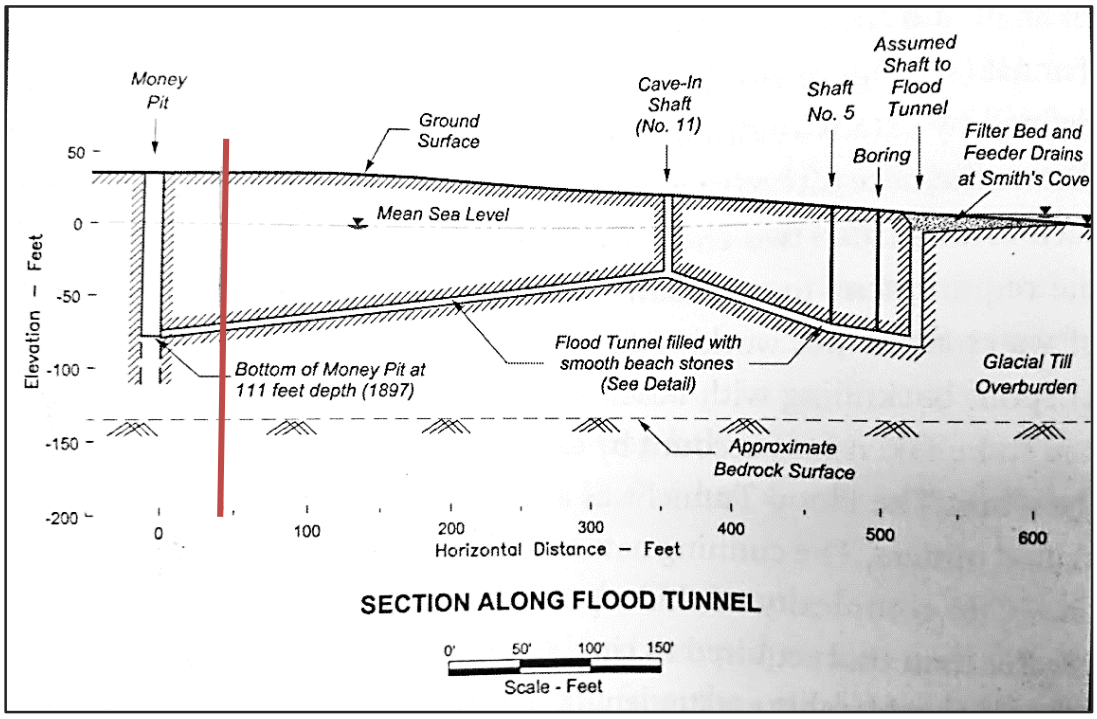

Figure 6: Section along Flood Tunnel. The line in red represents the approximate location where the grout will intersect the Flood Tunnel but is not representative of depth or diameter of grouting. (Harris & MacPhie, 2005)

The water entering the Money Pit is believed to be entering from a filter bed at Smith's Cove (see Figure 7). Estimated dimensions of the Flood Tunnels are 0.75 m by 1.2 m high with an upwards gradient of 6.6 degrees. It is thought that the Flood Tunnels are packed with beach stones but the packing may not extend throughout the entire length of the tunnel (see Figure 8) (Harris & MacPhie, 2005).

As part of using the shaft for tourism purposes after the underground workings are uncovered, the Flood Tunnel shall be reinstated. Therefore, during construction of

the deep shaft, an adit extending beyond the shaft is necessary to preserve a segment of the Flood Tunnel. However, the design of the adit and the process of preserving a segment of the Flood Tunnel are beyond the scope of this project.

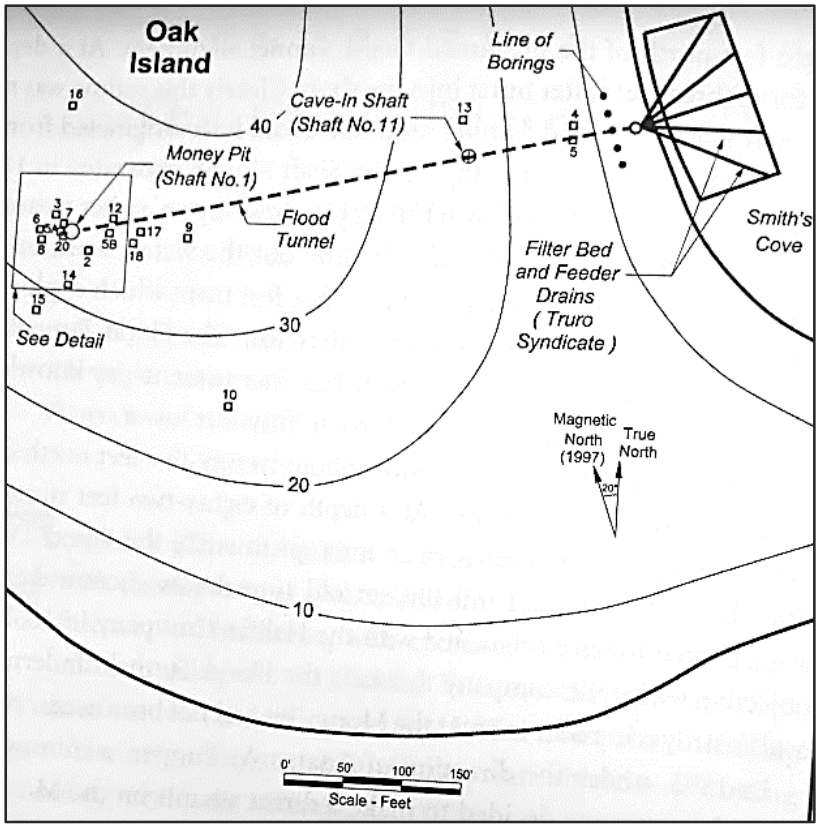

Figure 7: Map of the Money Pit and Smith's Cove (Harris & MacPhie, 2005)

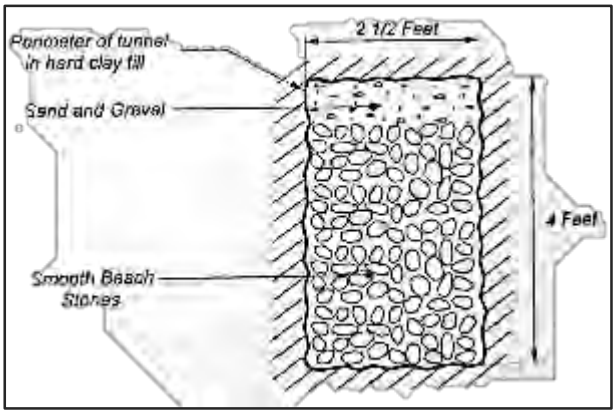

Figure 8: Cross section through the Flood Tunnel (Harris & MacPhie, 2005)

3.3.3 Man-made Chambers

Archaeological evidence from Becker holes B24 and B33 provided significant evidence of the possible presence of man-made chambers. Both boreholes intersected timber, followed by a 1.8 m deep cavity, at the same level, followed by an iron base. This provides evidence for the potential presence of a man-made chamber having a timber roof and an iron base (MacPhie, 2008).

4.0 Conceptual design

Upon successfully retrieving the underground workings and investigating the nature of the contents of the man-made chambers, the constructed shaft will be transformed into a touristic site. The chambers, Flood Tunnel and the Money Pit shaft will be reinstated. A three-dimensional conceptualization of the completed shaft ready for access by tourists is prepared using Google SketchUp. This section is an artistic rendition of the possible design of the shaft and the museum; the engineering design of the museum and access to the shaft is outside the scope of this project.

4.1 Museum and top view of the Shaft

Figure 9 shows the museum situated beside the steel shaft. The museum will contain any artifacts retrieved during excavation. The Money Pit shaft is indicated with a red star and is at the center of the steel shaft. Upon accessing the top of the shaft, there is an elevator and a staircase that lead to the base of the shaft.

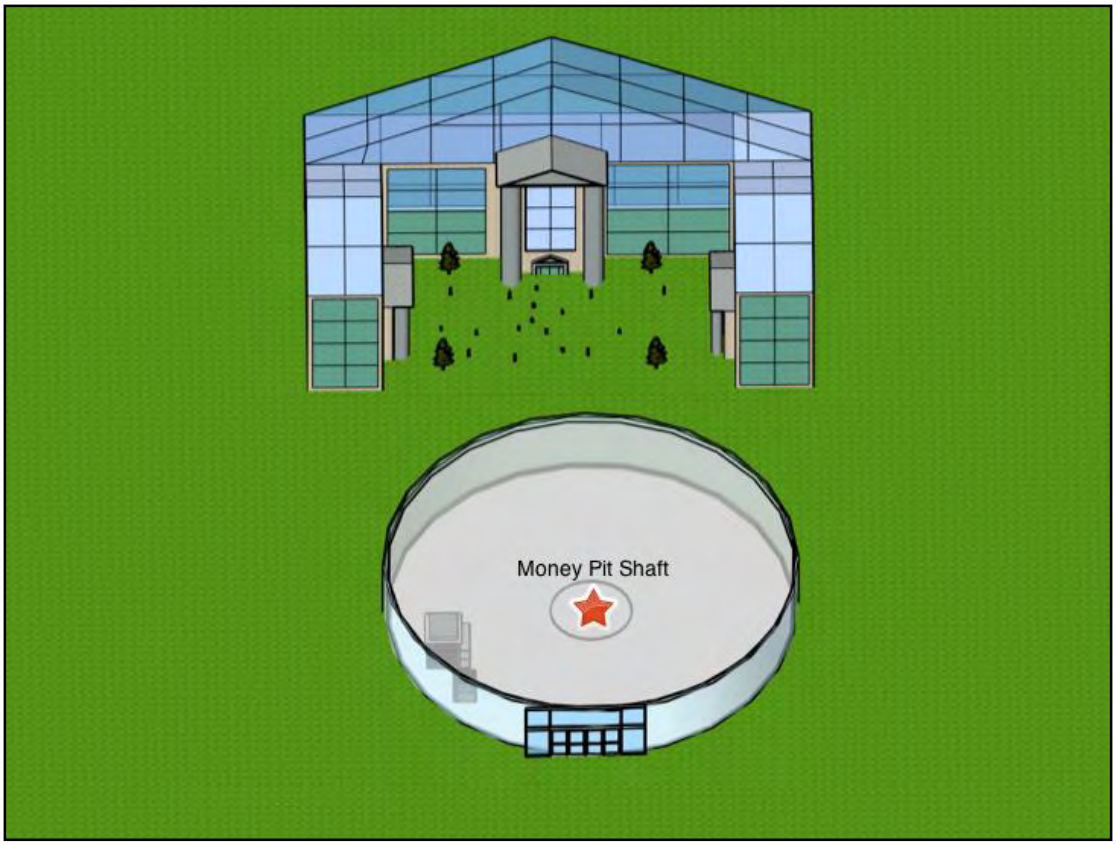

Figure 9: Museum of Money Pit located beside the steel shaft

4.2 Complete Shaft

Figure 10 shows the completed steel shaft. The steel shaft extends 50 m below ground level. The man-made chambers are assumed to be located approximately 10 m below the base of the steel shaft.

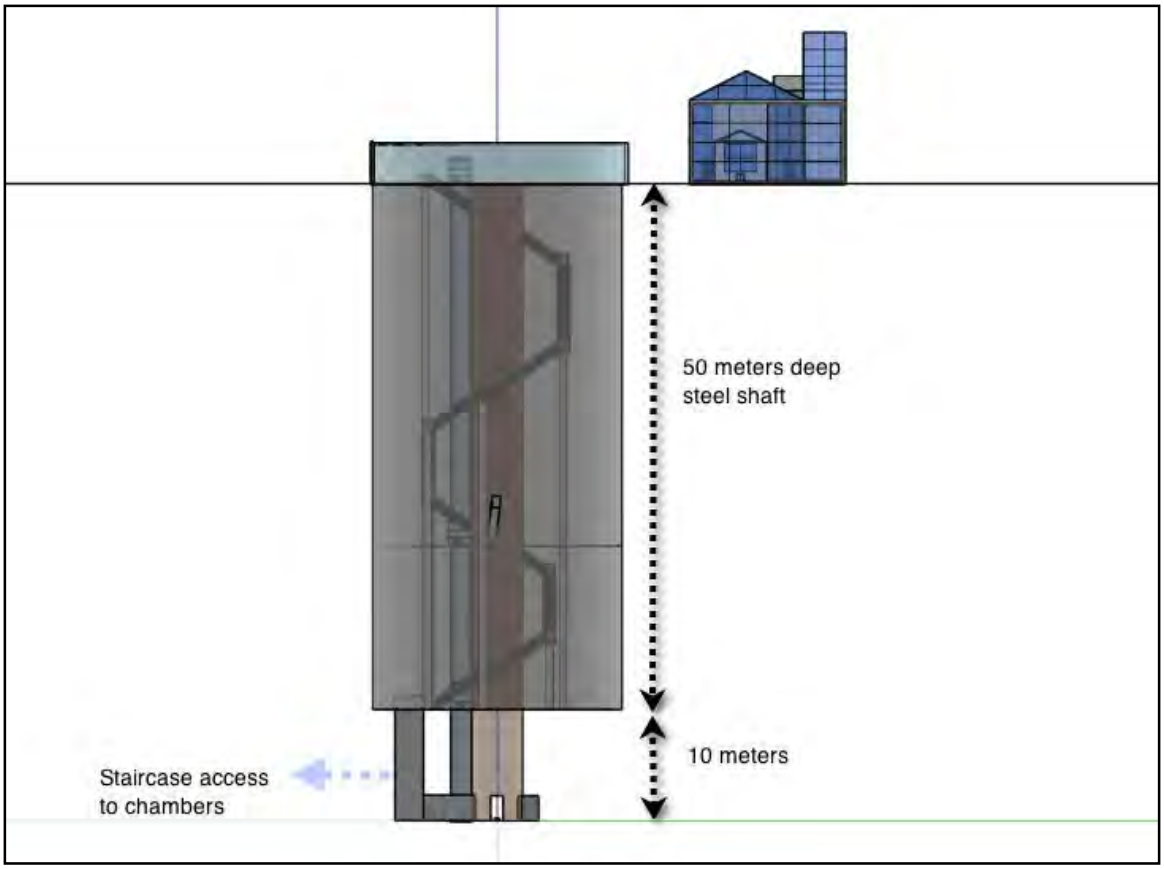

Figure 10: Side view of the completed steel shaft

4.3 Cross Sectional view

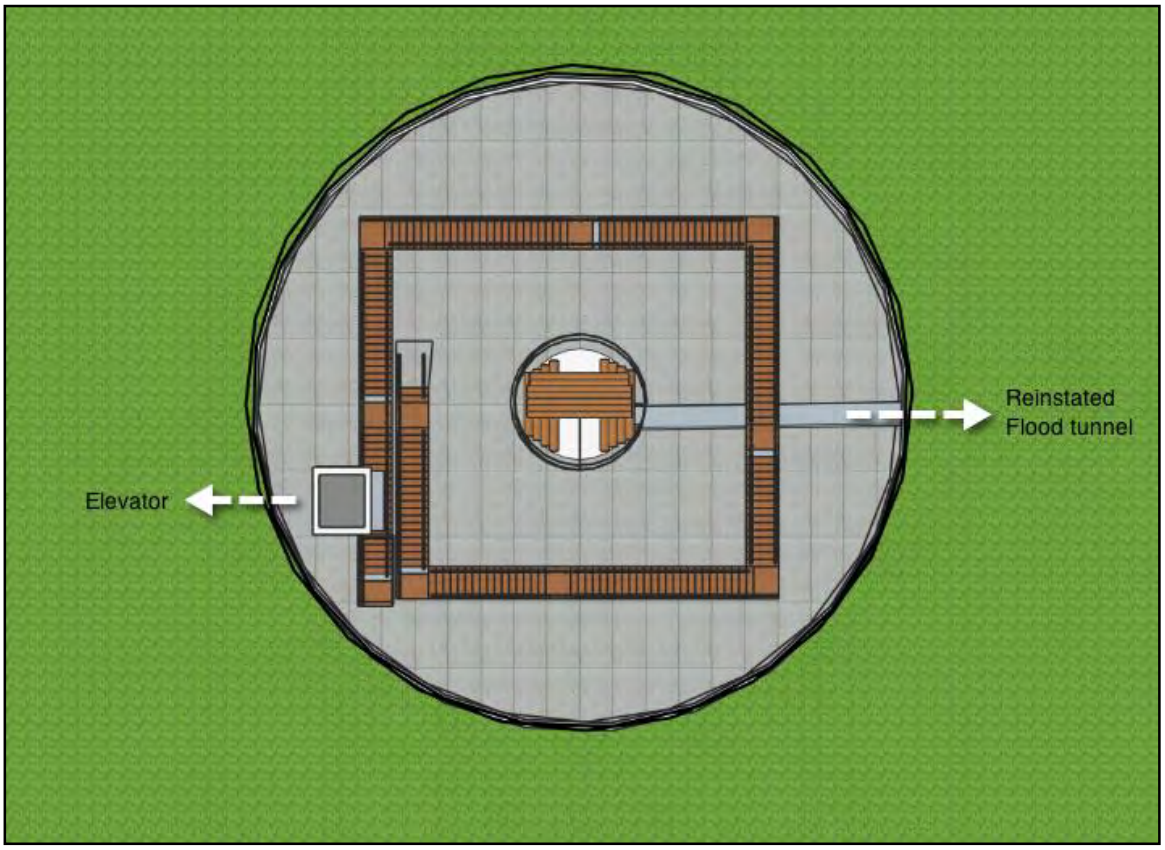

Figure 11: Plan view of steel shaft

Figure 11 shows a plan view of the steel shaft. The Money Pit shaft is located in the center, and has 9 wooden platforms situated every 3 m. A staircase will be constructed to go around the Money Pit shaft.

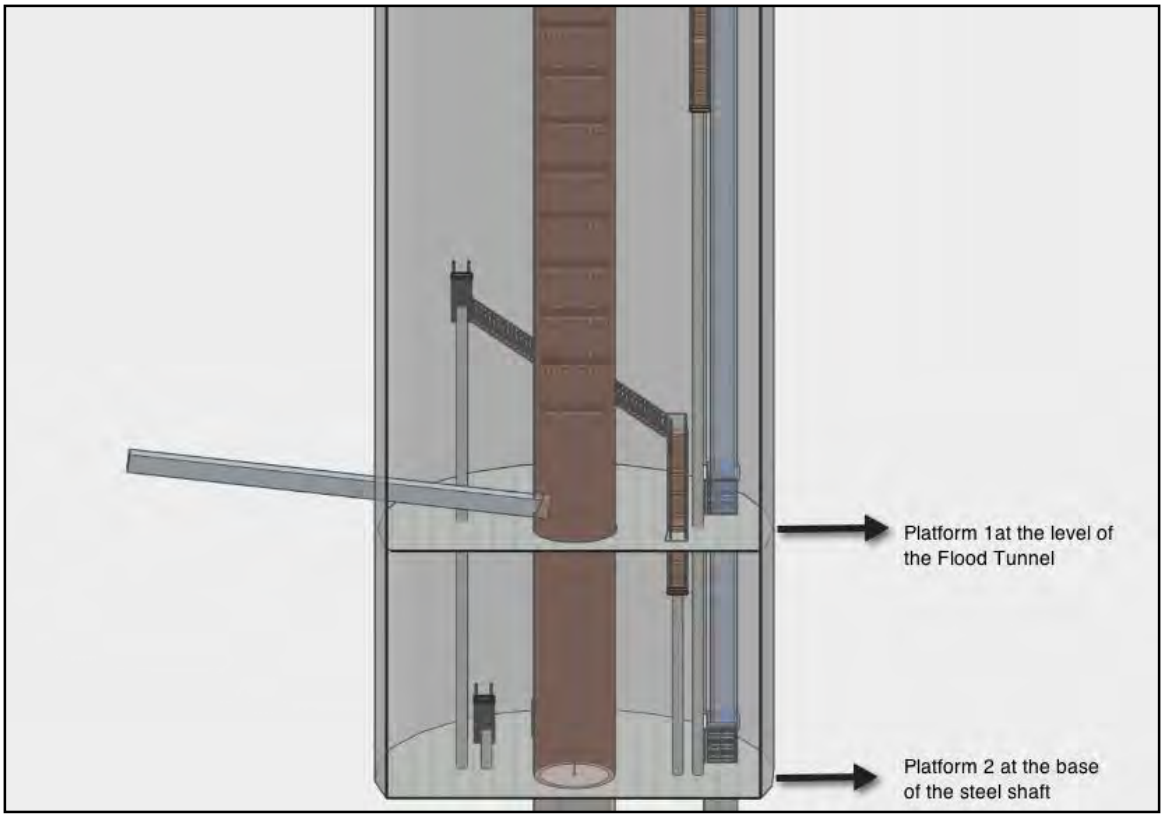

Figure 12: Cross sectional view of completed steel shaft

Figure 12 shows a cross section through the middle of the shaft. The brown center is the reinstated Money Pit. The steel shaft has two concrete platforms as shown in Figure 12, platforms 1 and 2. Geovation Engineering recommends the use of a glass elevator to allow people to view the Money Pit while in the elevator.

The elevator stops at:

- Platform 1, 34 m below ground level
- Platform 2, 50 m below ground level
- Platform3, at the level of the chambers, 60 m below ground level

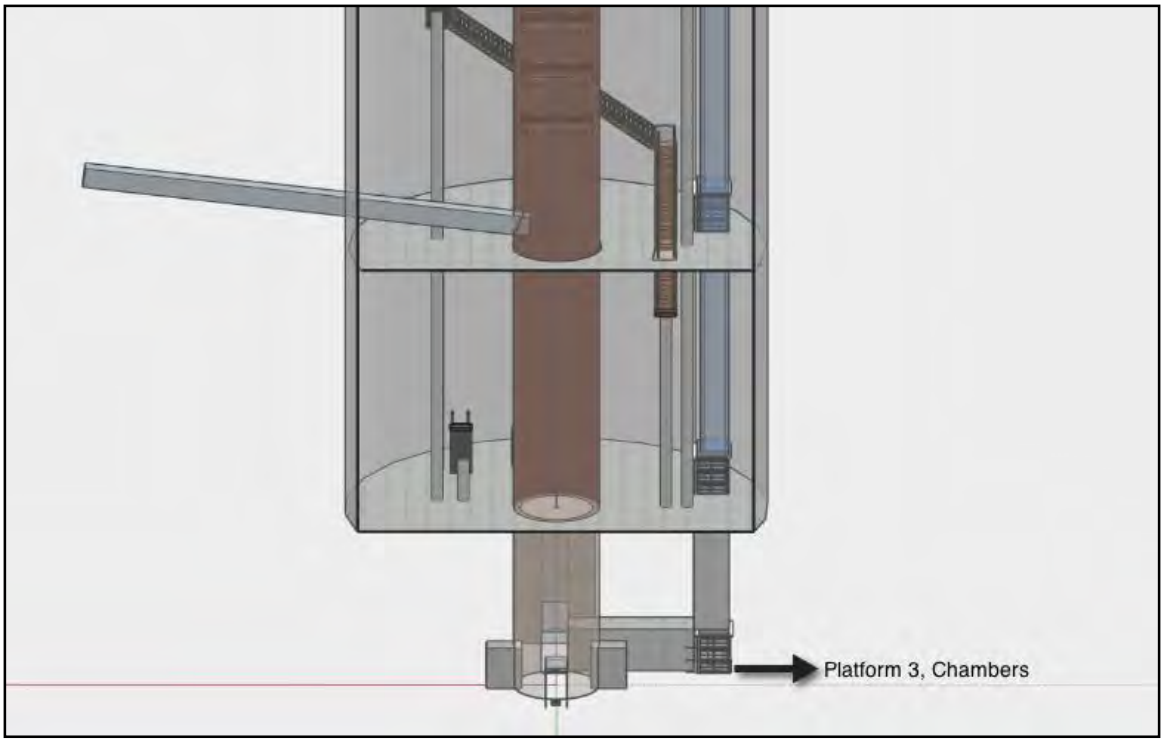

Figure 13: Cross sectional view of bottom of steel shaft

Figure 13 shows the bottom part of the steel shaft. There is a separate staircase leading from the bottom of the steel shaft to platform 3, at the level of the chambers.

4.4 Access to the Chambers

It is assumed that there are three chambers situated perpendicularly to each other. Figure 14 shows the top view of a potential configuration of the chambers, elevator and staircase. The staircase and elevator lead to a reception that in turn provides access to the chambers.

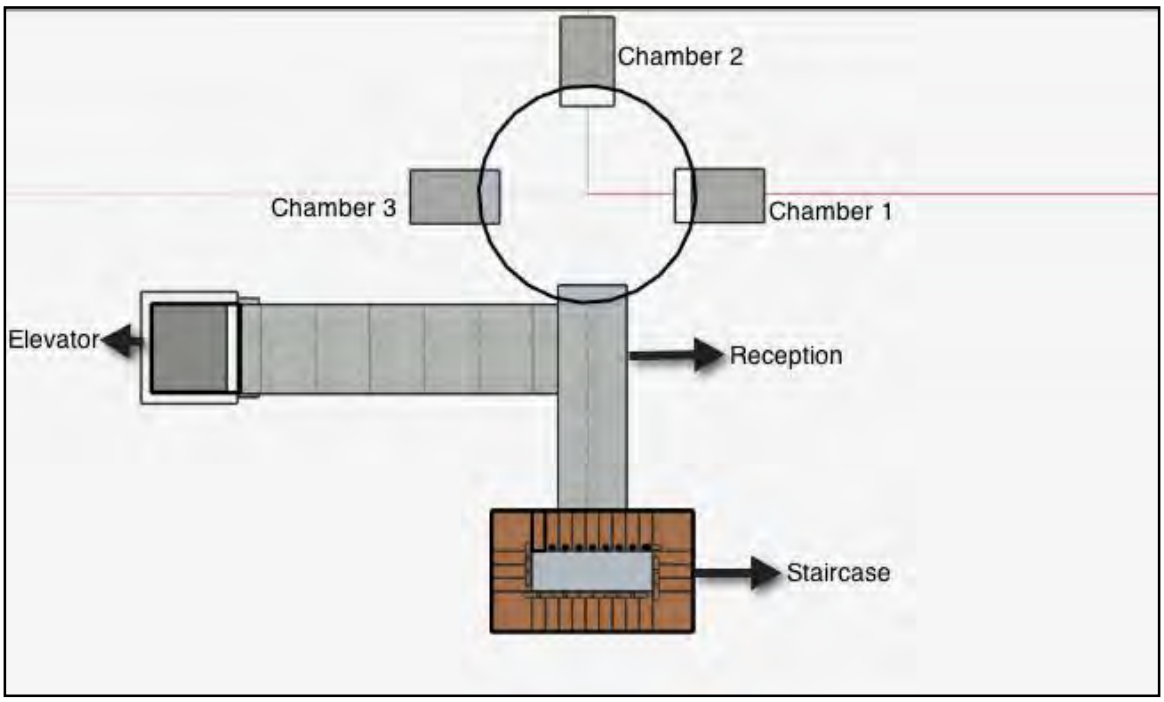

Figure 14: Potential configuration of chambers staircase and elevator

4.5 Ground Improvement visualization

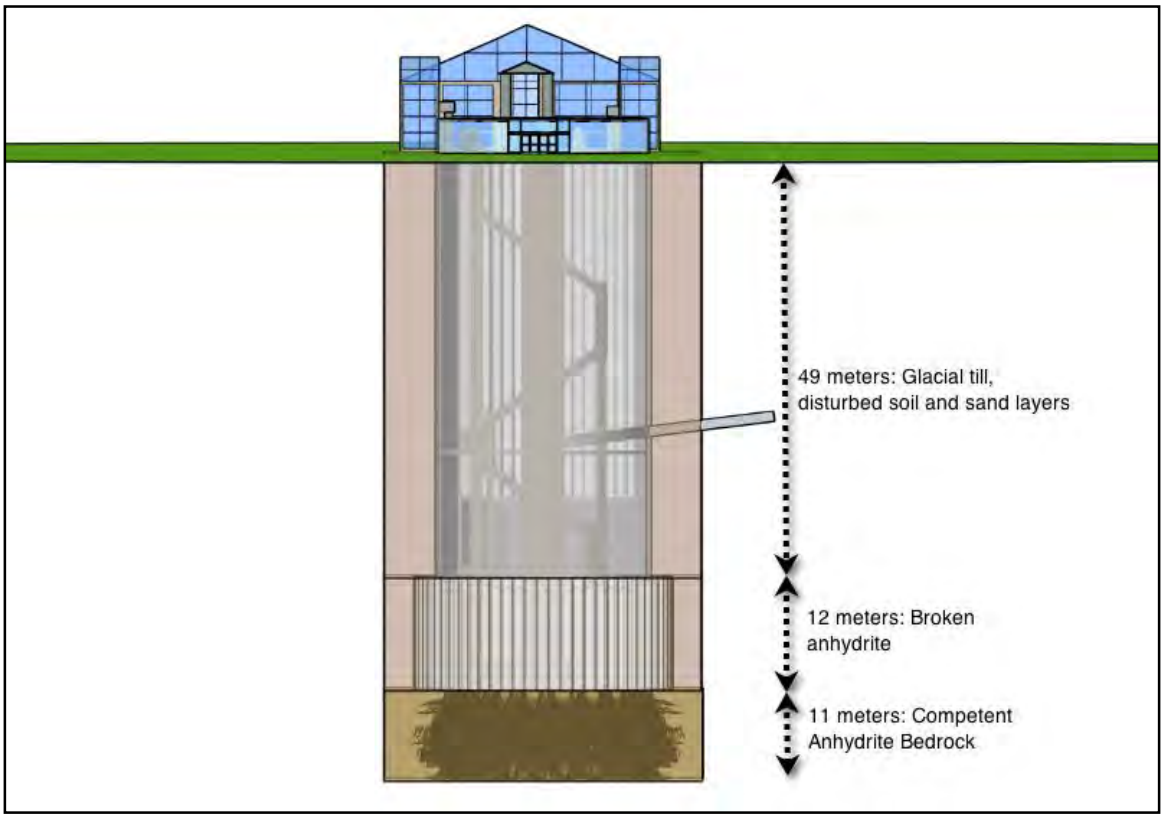

Figure 15: Side view of steel shaft with grouting

Figure 15 shows the permeation-grouting concept used to support the soil during excavation and block groundwater. The top layer is the glacial till layer that has some locations with disturbed soil or sand layers; therefore, only part of the glacial till requires jet grouting. In Figure 15, the jet grout is shown as intersecting cylindrical columns. The second soil layer is the broken anhydrite; this will require jet grouting columns all around the perimeter of the steel shaft. The bottom soil layer is the competent anhydrite bedrock. The permeation grouting in the bedrock fills the fractures within the rock and it therefore does not have a cylindrical shape, but rather a random form. Please refer to Section 6.0 for ground improvement design.

5.0 Preliminary Site Investigations

5.1 Objectives

Prior to implementing the proposed design, Geovation Engineering recommends that extensive preliminary investigations be conducted with the specific purpose of uncovering archaeological and geological information of the site subsurface conditions. Ultimately, the objectives of these investigations include verifying the presence of and delineating the region of man-made workings; as well as any artifacts of considerable value that can be directly associated to the activities of the original depositors. In addition, the goal is to gather enough information to be able to provide optimal direction during the detailed design of a deep excavation in order to facilitate the recovery of artifacts and to identify the historical and archaeological context of the site.

5.2 Investigation Approach

The preliminary site investigation is aimed at exploring the area of the proposed 24 m diameter shaft. Geovation Engineering recommends that the preliminary site investigations be divided into two phases: Phase I and Phase II. The Phase I investigation is a non-intrusive method of investigation while Phase II investigation is an intrusive method of investigation.

5.2.1 Phase I Investigation

A Phase I investigation will include collecting and reviewing data and comparing the data to published information about the site. Phase I investigations are normally carried out at the start of the site investigation process to characterize the site, and dictate the remainder of the site investigation. In addition to reviewing the published information, a detailed inspection of the site termed a walkover survey, is normally undertaken. It is an integral part of the phased approach to site investigation.

The benefits of carrying out a Phase I investigation prior to commencing intrusive works includes the ability to efficiently target project resources, avoid money being wasted on inappropriate intrusive ground investigations, and providing early warning of possible delays or budget implications through previously unknown site characteristics.

5.2.2 Phase II Investigation

A Phase II investigation is an intrusive investigation that includes obtaining samples and carrying out tests from exploratory holes to obtain information about the ground conditions. The exploratory holes may include hand or machine excavated trial pits, mini rig boring, light cable percussive boring or rotary borings. The choice of exploratory hole type heavily depend on the findings of the Phase I investigation and the requirements of the entire investigation.

The exploratory holes enable a program of in-situ testing, laboratory testing and monitoring to be undertaken. The objective of the site investigation is to characterize the ground conditions sufficiently to allow safe and economic designs to be developed, and to reduce, as far as possible, the occurrence and impact of unforeseen conditions. Phase II site investigations are intrusive, which involves collecting and analyzing soil, water, and gas samples (as appropriate) from the site. Every site investigation is site-specific. Multiple aspects must be considered, such as the client and regulators requirements, land-use, area and access, geology, and the relevant standards.

6.0 Ground Improvement design

The geotechnical conditions of Oak Island are complex. Without a ground improvement design the construction of a steel liner could potentially become extremely costly. It is for this reason the use of jet grouting and permeation grouting are recommended in the ground improvement design. These technologies are very versatile. Grouting can be used as earth support, a seepage barrier, earth pressure reduction, underpinning among other functions. Although it may seem like a simple way to solve most geotechnical problems, the use of this technology should be subject to thorough judgment as there exist very few detailed experimental studies on the effects of jet grouting and permeation grouting. The dimension and properties of grouted columns are determined more or less arbitrarily by designers therefore field trials must be performed prior to the execution of the project to verify the treatment effects. That being said, the following design recommendations are preliminary and are subject to change once more site investigations have been performed along with field trials.

6.1 Jet Grouting

6.1.1 Procedure

Jet grouting consists of a small nozzle placed on pipe that is drilled to a maximum desired depth into the soil using a rotating or rotary-percussive direct drilling system. Air, water, grout or foam is used as a flushing mechanism for the drill. The drill bit is mounted at the tip of an instrument called the monitor which is attached to jointed rods provided with single, double or tipple inner conduits. The bit, usually ranging from 120 and 150 mm in diameter, is slightly larger than the conduits, which creates an annular space between the two. This space allows the drill cuttings to surface and gives an indication of the subsurface conditions (Paolo Croce, 2014).

Jetting is then performed by raising and rotating the monitor which is equipped with one or more small diameter nozzles that transform high pressures fluids (grout, air and or water) into high speed jets (see Figure 16). While the jet action remolds the soil, part of the injected fluids and the soil rises to the surface through the annular space between the borehole wall and the jointed rods. This combination of soil and injected fluids is called the spoil. Spoil is the most important quality control indicator on site during the grouting stage. The spoil should be monitored for changes in flow as unexpected changes in spoil return may be indicative of the malfunctioning of the jet. Additionally, it is very important to assure that the that the borehole annulus is not clogged as this will affect the spoil outflow. The spoil will have to be channeled to storage and eventually disposed of. Contractors may consider the option of reusing the spoil to inject new jet grouted columns as a cost savings tactic (Paolo Croce, 2014).

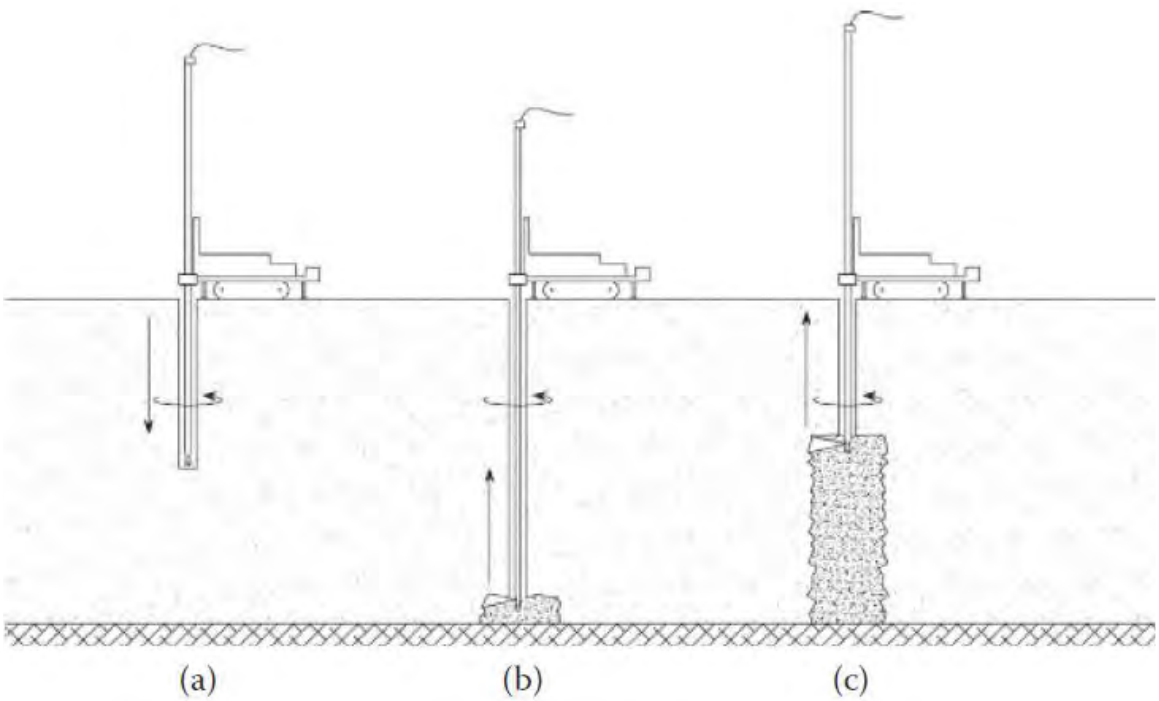

Figure 16: Jet Grouting procedure. a) Represents the drilling portion of jet grouting; (b and c) represents the raising of the monitor and the formation of the jet columns. (Paolo Croce, 2014)

Raising can be carried out continuously or intermittetly. When raising is performed continuously the jet moves along a spiral path. However, due to the variable soil

conditions on Oak Island it is recommended to use intermittent raising with steps of 40-100 mm, as it allows for more than one pass at the same location should it be required (Paolo Croce, 2014).

6.1.2 Jet Grouting Systems

There exists 3 primary jet grouting systems; single fluid system, double fluid system and triple fluid system. The single fluid system silmultaneously remoulds and cements the soil with the use of a single fluid made of water and cement. It is often used for small to medium sized columns and is therefore not an ideal candidate for this project (Paolo Croce, 2014).

The double fluid system is very similar to the single fluid system however the water cement fluid is shrouded by a coaxial jet of air which reduces energy losses. The double system is primarly used for panel walls, underpinning and sealing slabs. The jet columns created by the double fluid system have slightly lower unit weights as compared to the other two systems due to the bubbles that get injected from the air jet (Paolo Croce, 2014).

Finally, in the tripple fluid system the soil remoulding and cement-soil mixing occur in two separate steps. The soil is eroded by an air shrouded water jet that it placed above a second nozzle that delivers the water cement grout mixture. The seperation of the remolding and cementation allows the grout to be delivered at a lower velocity. This triple fluid system is often used for underpinning and cut off walls. It is also effective in cohesive soils and is recommended when creating larger columns. For the purpose of this project Geovation Engineering recommends the use of the tripple fluid system and futur calculations will be done in accordance to this. However futur consulting firms may assess the conditions of the site and their equipement and come to an alternate decision (Paolo Croce, 2014).

6.1.3 Column Overlapping sequencing

In the scope of this project, many jet grouted elements will be created by overlapping jet columns. Geovation Engineering recomends the Primary-Secondary Sequence/ Fresh-in-Hard sequence. Using this method secondary columns are only created after waiting for the primary columns to sufficiently cure as seen in Figure 17 . Tertiary columns may also be used. This method prevents the washout of previously created columns and can help reduce the dimensions of the secondary columns which may have cost saving implications (Paolo Croce, 2014).

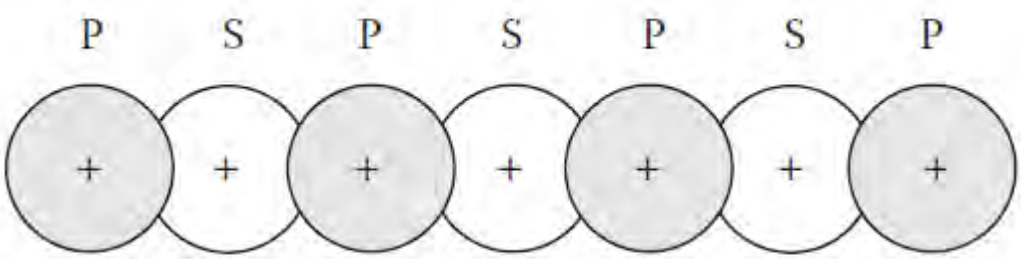

Figure 17: Primary and secondary jet columns

6.2 Permeation Grouting

6.2.1 Procedure

Permeation grouting, although very similar to jet grouting, involves the filling of cracks, joints, and any small defects in rock without the remolding of the existing material. The purpose of this type of grouting is to strengthen and/or prevent water flow through the host material (Warner, 2004).

Permeation grouting is usually performed by using a rotary or percussion rotary rig that creates a borehole in the host material. Before beginning the grouting process the hole must be washed out (Houlsby, 1990).

When the proposed permeation grouting holes are longer than 10 m, it is usually recommended to divide the hole into shorter lengths, called stages, and grout each

of these stages separately. These stages are grouted separately to allow for variation, should one stage require a different amount of grout or pressure to fill the voids. Geovation Engineering recommends intermittent raising with steps of approximately 3 m in length (Houlsby, 1990).

6.2.2 Permeation Grouting Systems

Permeation grouting can be progressed in either a downstage or upstage system. The fastest and most efficient method is upstage grouting and it is the most appropriate method for sound rock. It is similar to jet grouting in the sense that the rig is initially drilled to the maximum desired depth. The washout procedure is performed during this drilling process. As the monitor is raised in intervals, it releases the grout. This system is very dependent on the quality of the rock (Houlsby, 1990).

Should the preliminary investigations prove that the sound anhydrite is prone to severe fractures, the perceived cost savings will decrease significantly. The downstage system should be used in this case. The system differs from the upstage process as the drill does not initially descend to the maximum desired depth. Instead the drilling, the washing and the grouting occurs in stages of approximately 3 m in lengths as mentioned above. The primary disadvantage of the method is that it requires a significant lag time (up to 24 hours) between each stage, to avoid disrupting the previous stage by washout (Warner, 2004).

6.2.3 Column Overlapping sequencing

Similarly to jet grouting, after the grouting of one hole is performed the next hole should not be adjacent to it. The Primary-Secondary Sequence is recommended in order to prevent disruption of previously grouted rock. The primary secondary sequence was selected for the same reasons as the ones mentions in Section 6.1.3. It should be noted that there will be a significant reduction in the grout take from primary to secondary columns.

6.3 Grouting in Broken Anhydrite

6.3.1 Intent

The intent of grouting in the broken anhydrite is to prevent seepage from the groundwater into the area where lie the potential chambers and future elevator shaft. The reduced seepage will ease the stress placed on the dewatering wells during exploration below the steel liner and during construction the permanent structures in the broken anhydrite. In the long-term it will also reduce the water pressure on the permanent underground structures.

Seepage prevention will be attained by overlapping jet columns. The jet grout will span the entire height of broken anhydrite layer in a circular fashion. The diameter of the hollow cylindrical grouted element will be approximately 28.5 m as it must remain at least 2 to 4 m outside the path of steel shaft and potential chambers. Refer to Appendix A for calculation of radius of the cylindrical element.

In literature heavily fractured rock has a permeability ranging from 10^{2} cm/s and 10^{-2} cm/s. This is a high discharge free drainage layer. The jet grouting should reduce discharge and create a poor drainage layer (Henn, 1996). Geovation Engineering suggests reducing the permeability to 10^{-6}cm/s. This would limit the seepage through the grout curtain to only 0.7817m^{3} of water per day (Refer to Appendix A). The following design parameters are intended to achieve this reduced permeability, however they are subject to field trials.

6.3.2 Diameter

The diameter of the jet columns in the broken anhydrite were determined using a deterministic approach as there currently exist very little data relevant to jet grouting calculations. The calculations seen in Appendix A model the broken anhydrite with pockets of loose soil after silty sand and sand and/or gravel using a triple fluid treatment system. The partial factor recognized the lack of available

experimental information, the soils high heterogeneity (due to the pockets of sand) and the massive treatment application. Based off the previously mentioned values, the initial design diameter of the jet columns is 1.5 m (Paolo Croce, 2014).

6.3.3 Spacing

In order for the jet grouted element to adequately reduce seepage the columns must be placed sufficiently close to each other. Based on previous projects that used jet grouted columns to prevent seepage, such as the Diavik Diamond Mine Project, the length of the lens created by two overlapping columns (Se) should be approximately 0.5 m as seen in Figure 18 (Wonnacott, 2014). The corresponding center to center spacing is equivalent to 1.4 m for columns of a diameter of 1.5 m. View Appendix A for detailed calculations.

Figure 18: Length of the lens created by overlapping jet grouted columns.

6.3.4 Sequencing

Based off the spacing and dimensions of the jet columns and diameter of the cylindrical jet grouted curtain it was determined that 64 individual columns are required. Literature suggests that primary and secondary columns should be approximately 6 m to 12 m apart (Army and Air Force, 1970). Therefore, Geovation Engineering suggests placing primary columns every 11.2 m along the arc of the cylindrical shape. Namely, every 8th column will be a primary column. The secondary columns will be placed in the center of the two primary columns and the process will continue as such. Figure 19 illustrates a plan view of the sequencing plan.

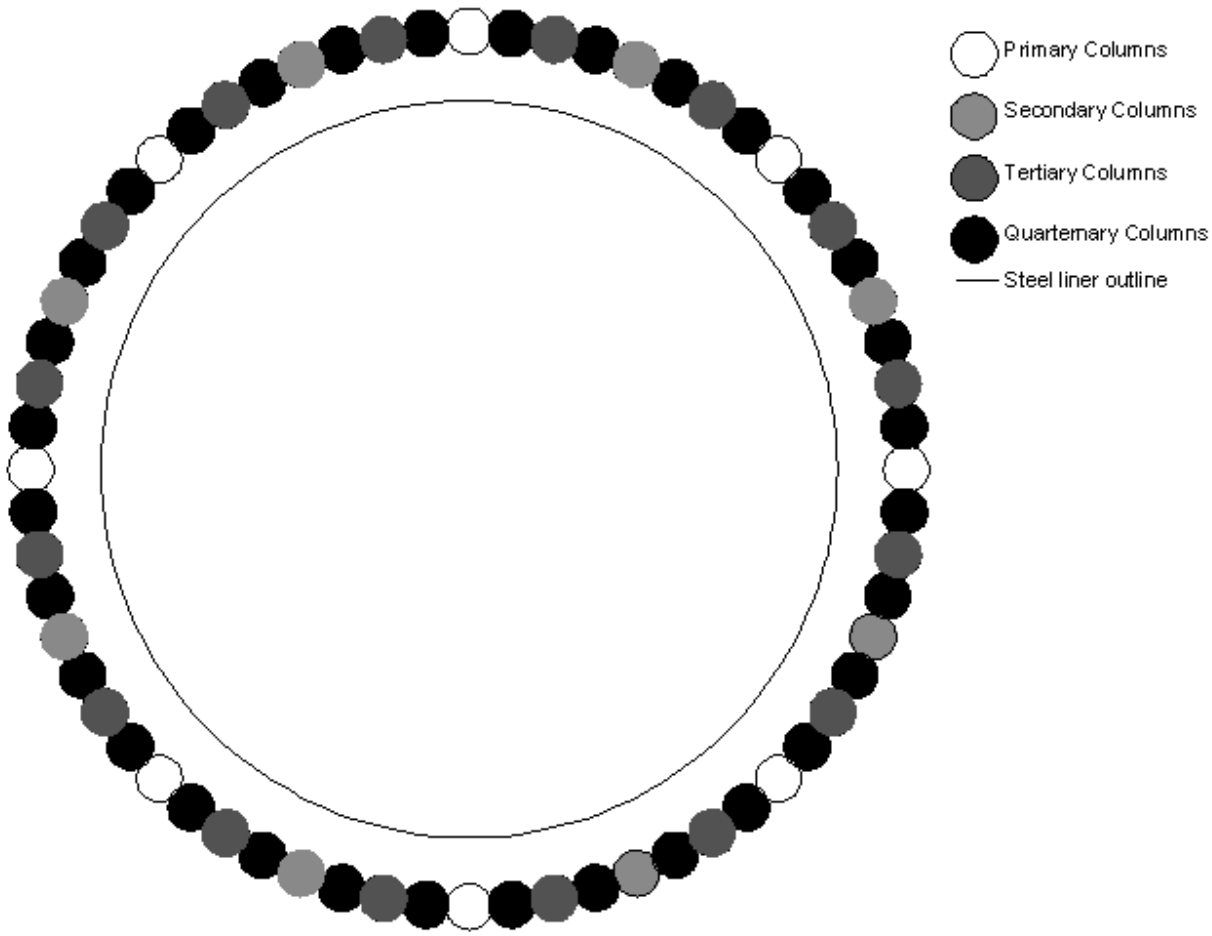

Figure 19: Fresh in hard sequencing for the jet grouting in broken anhydrite

6.3.5 Grout Mixture

The characteristic of upmost importance for the jet grout material in the broken anhydrite is permeability. As seen in Figure 20 permeability and the water cement ratio are intimately related. This is due to the fact that the volume of capillary voids depends on the water cement ratio. The permeability should be low to effectively prevent seepage. This requires a low water cement ratio, however if the water cement ratio is too low the grout will lose its erosion capabilities. Literature

recommends that the w/c ratio remain between 0.6 and 1.3 (Paolo Croce, 2014). Geovation Engineering recommends a w/c ratio of 0.6. Following field trials the water cement value may be adjusted to improve the integrity of the jet columns.

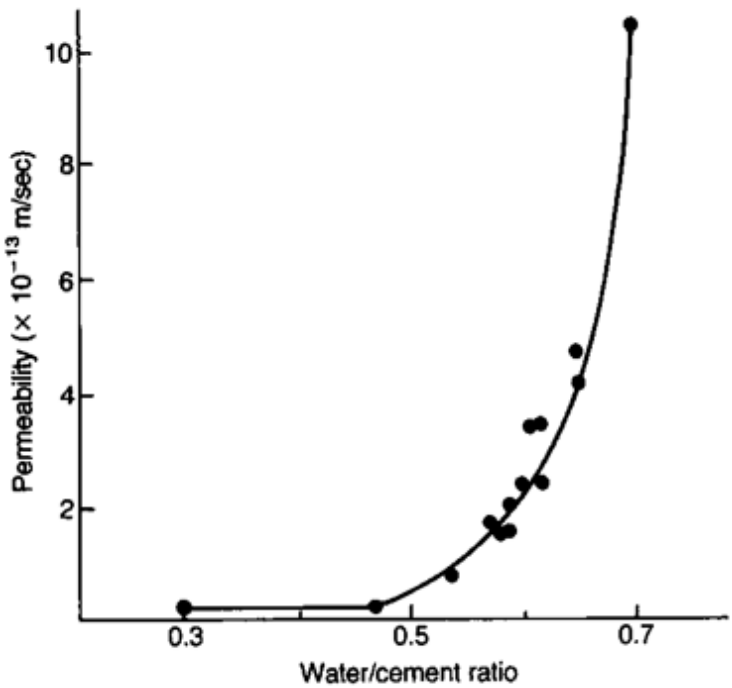

Figure 20: The relationship between the permeability and water/cement ratio of mature cement paste (93% hydrated) (Powers et al., 1954)

The environment on Oak Island is very harsh. The groundwater on Oak Island is sea water which is very high in sulfates, approximately 2700 ppm (Health Canada, 1994). Sulfates are harmful to the grout as they can cause swelling of the interstices (Army and Air Force,1970). The selection of the cement type was based off Table 1. Due to the severe degree of sulfate attack, a type V (ASTM C150)/type 50 (CSA A5) cement which resists sulfates should be used.

Table 1: Selection of the cement type on the basis of sulphate concentration in the soil and in the water

Relative degree of sulphate attack	*Percentage of water-soluble sulphate (as SO_4) in soil samples*	*Sulphate (as SO_4) in water samples (ppm)*	*Cement type*
Negligible	0.00–0.10	0–150	I
Positive	0.10–0.20	150–1500	II
Severe	0.20–2.00	1500–10,000	V[a]
Very severe	2.00 or more	10,000 or more	V+ Pozzolan[b]

Source: Modified from USBR, *Concrete Manual: A Manual for the Control of Concrete Constructions*, 8th ed.: Denver, CO: U.S. Bureau of Reclamation, 627 p., 1981.

[a] Approved Portland-pozzolan cement providing comparable sulphate resistance when used in concrete.

[b] Should-be-approved pozzolan that has been determined by tests to improve sulphate resistance when used in concrete with Type V cement.

The tidal influence caused by groundwater flow reversals at the Money Pit should also be considered during mix design. Although the tidal differential at the Money Pit has only been observed and not measured, tide gauges have been installed in hole 10x and well 9303. Readings from the tide gauges indicate that the tide in 10x has a daily height of 50 cm and the tide in well 9303 has a height of approximately 5 cm (David G. Aubrey, 1996). After consulting with Eco Grouting Specialists Inc. it has been decided that the lateral movement due to the tides will not significantly affect grouting operations. The use of an anti-washout admixture which will mitigate the influence caused by the groundwater (Eric Landry, 2014).

The use of a high early strength cement or of an accelerator were also considered to limit the number of delays during operations. However, due to the significant number of columns that will be created, consultants at Eco Grouting do not foresee the need for accelerators and type early cement for jet grouting applications, and advise against it (Eric Landry, 2014).

6.4 Jet grouting in Sand and Disturbed Soil

6.4.1 Intent

Above the layer of broken anhydrite there exist a layer of glacial till which is a relatively stable soil that is suitable for excavation. However within that layer exist an area of backfill due to the Dunfield Excavation. This soil is not uniform and is softer and looser than the native glacial till. Additionally there exist layers of sand within the glacial till. Sand exhibits dilative behavior and is unstable in vertical cuts. These soils have low internal strengths and are unstable in vertical cuts as they have a greater likelihood of sloughing. Grouting in these layers will provide the necessary strength and stiffness required for the excavation (Ontario, 2014; Winnipeg, 2007).

6.4.2 Parallels with Grouting in Broken Anhydrite

Grouting in the sand and disturbed soil will be done using the same boreholes as the ones used for grouting in broken anhydrite layer. The holes will be spaced 1.4 m away from each other along the arc of the 28.5 m cylinder around the Money Pit. As the monitor rises past the broken anhydrite, if a sand layer or disturbed soil layer is present it will be grouted. The grouting in the glacial till layer will be intermittent.

As the goal of grouting in the sand and disturbed soil is to improve strength, continuity is required. The diameters of the columns should remain the same as the diameter of 1.5 m in the broken anhydrite. Reducing the diameter would reduce the horizontal overlap which is already limited to 0.1m. Due to the risk of misalignment it is not recommended to further reduce the diameter.

Jet grouting in the sand and disturbed soil will be done in the same sequence as grouting in the broken anhydrite: Fresh-in-hard. Figure 19 in section 6.3.4 can be viewed to better understand the sequencing operations.

The initial grout mix will also be very similar to that of the grout in the broken anhydrite. The water ratio should also be relatively low to ensure adequate strength. The initial recommendation is 0.6 w/c ratio. As a large portion of the grout

used for the unstable layers will also be below the groundwater table type V cement is recommended to resist sulfate attacks in conjunction with an anti-washout admixture. For grouting performed above the water table, type I cement may be used without the use of admixtures.

6.5 Jet grouting in Cavities

6.5.1 Intent

Within the subsurface of the Money Pit area there exist many natural and man-made large voids. The man made voids include the Flood Tunnel(s), the Halifax Tunnels and the Money Pit chambers. Natural voids are expected within the broken anhydrite. Based on the 12 geotechnical and archaeological holes that extend into the broken anhydrite, a quarter of the holes encountered cavities varying between 1 to 1.5 m in size. Both the man-made and natural voids, save the Money Pit chambers, would cause large water influxes and could potentially cause overlying structures to settle. Therefore the voids must be sealed to ensure the smooth running of the project at hand. The sealing of voids will be a precarious part of the ground improvement design as it is important to unsure that the chambers do not get grouted if encountered.

6.5.2 Procedure

While drilling during the jet grouting process, geotechnical information will be retrieved and analyzed. If a void is detected, drilling will be stopped immediately and a sonic device will be passed down the void.

If a Money Pit chamber is discovered, the approximate size of the chamber will be determined. The intended path of the cylindrical element created by the jet columns will be relocated outside the location of the chamber in order to minimize any damage.

Should a Halifax Tunnel or an empty Flood Tunnel be discovered, barrier bags, as suggested by consultants at Eco Grouting, will be utilized to seal the voids. Using a

set of barrier bags, a 3 to 9 m plug capable of resisting small cracks will be created. The size will depend of the water pressure present in the tunnel. The sonic device will determine the size and the alignment of the tunnel. Subsequently, a sleeve pipe will be inserted and a barrier bag will be filled with cement grout. For large voids, 3 m diameter bags are available or alternatively 2 smaller diameter bags may be used. A second borehole will be created at the location determined by the sonic device and the second barrier bag will be filled with cement grout. Once the two barrier bags have sufficiently hydrated, cement grout will be injected into the void between the two bags. A relief hole will be created on the alternate end of the injection site to monitor the specific gravity of the grout. When the specific gravity of the discharged material is equivalent to that of the injected grout, the void is filled and the plug has been created (Eric Landry, 2014). Filler may be used for economic reasons. For permanent work, only mineral fillers should be used (Army and Air Force,1970).

If a natural void is detected, the size should be determined. If the void is large enough to create a 9 m plug the procedure for the man-made workings will be used. If the void is smaller, it may suffice to allow the jet grouting monitor to inject a viscous grout for a longer period of time at that location.

6.6 Grouting in Sound Anhydrite

6.6.1 Intent

To prevent water flow through the cracks in the sound anhydrite, 12 m of the competent anhydrite will be permeation grouted. As anhydrite readily dissolves in water, grouting will slow the dissolution by maintain the permeability of the bed rock in the range of 10^{-7} cm/s to 10^{-10}cm/s (Henn, 1996).

Permeation grouting will be performed after the jet grouting. Grouting the broken anhydrite prior to permeation grouting the sound anhydrite will prevent the grout from migrating into the more pervious zone (broken anhydrite) via preferential paths. The jet grouted columns will create confinement which will force the grout to

migrate into the small pore spaces of the competent anhydrite. This will create an effective seal (Eric Landry, 2014).

6.6.2 Spacing and sequencing

The permeation grout holes should be drilled through the jet grouted columns in order to reduce deviation issues at great depths. If the holes are drilled on the outside of the jet grouted curtain, the chances of hitting grouted anomalies with the drilling equipment will be higher. If these anomalies are encountered during drilling, hole deviation issues may become an issue. Additionally, placing the grout holes directly below the 28.5 m diameter jet grouted element will create a continuous vertical curtain.

The spacing of individual boreholes is dependent on the conditions of the material. The permeability, matrix porosity, prevailing direction of fissures all affect how far the grout will diffuse within the rock. Spacing is normally within a range of 0.6 to 1.2 m (Warner, 2004). For the purpose of cost estimation Geovation Engineering will assume a spacing of approximately 1 m (90 holes along the grout curtain), however, field trials will have to be performed to determine a spacing that will provide adequate water seepage prevention.

Geovation Engineering suggests that every sixth borehole be primary. This approximately every 6 m along the arc of the cylindrical shape. The secondary columns will be placed in the center of the two primary columns and the process will continue as such. Figure 21 illustrates a plan view of the sequencing plan. For demonstration purposes only, permeation grouting is demonstrated as a circular element. However, in reality this is not the case, the grout will only fill fissure in the rock.

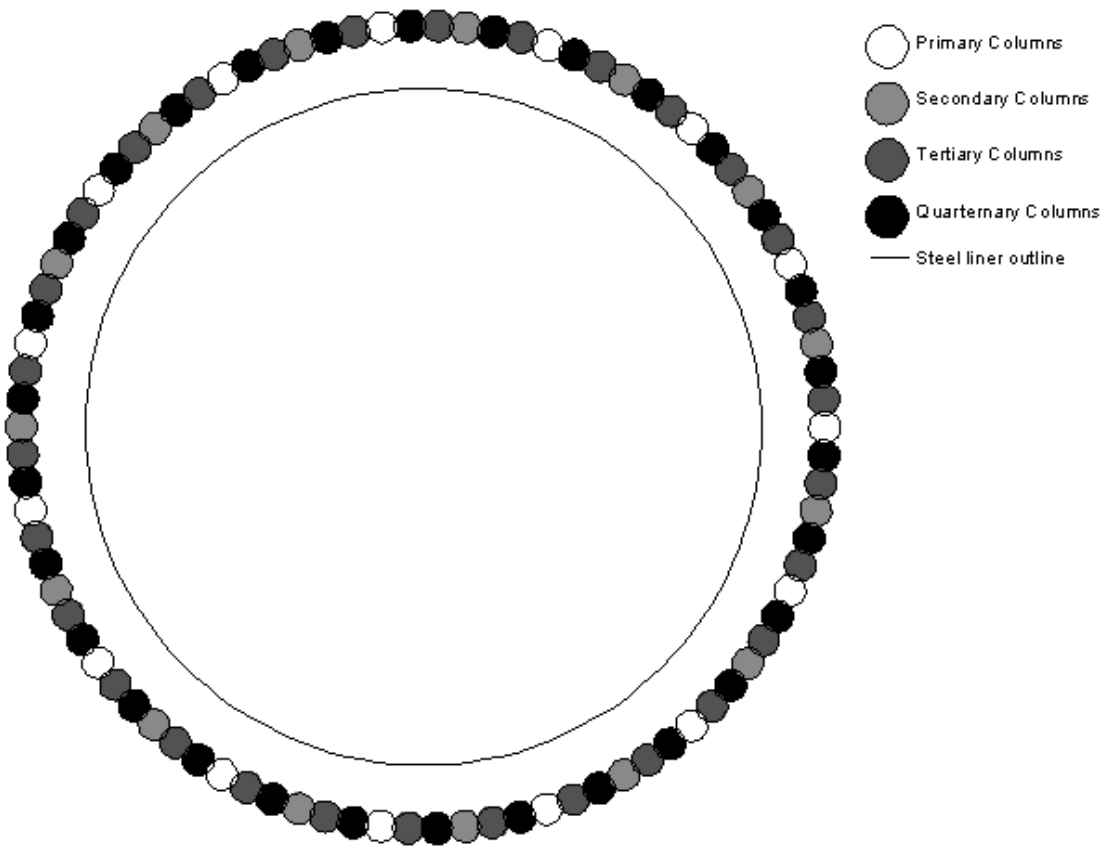

Figure 21:Fresh in hard sequencing for the jet grouting in anhydrite

6.6.3 Grout Mixture

For grouting in sound bedrock, either micro-fine or ultra-fine cement should be used depending on the size of the defects present in the sound anhydrite. Finer

cement must be used as the minimum dimension of the fissures should be at least four times that of the maximum grain size in the grout mixture.

When grouting in a material with low permeability, it is advised to use a higher water cement ratio as it provides better sealing efficiency. Unstable slurries are often used with w/c ratios varying between 0.66 and 3. Geovation Engineering's initial mix design recommends a 1.5 w/c ratio for grouting in sound anhydrite. The use of water suspension of bentonite is recommended to reduce the effect of bleeding as it helps increase inter particle forces and holds cement particles in suspension (Babu, 2010).

If the fissures in the sound anhydrite vary significantly in size, Geovation Engineering recommends the use of the grout thickening technique. Field operations begin with using grout that has a higher w/c ratio. This higher w/c ratio grout penetrates and plugs fracture planes. The grout's w/c ratio is then lowered, to create a thicker grout that penetrates larger openings (Magnus Axelsson, 2007).

6.7 Emergency Grouting

In case a water bearing fault or a man-made cavity has not been correctly dealt with prior to excavation and installation of the steel liner, an emergency grouting plan is in order. To prevent the need of post excavation grouting, which requires more time and incurs more costs, pilot holes and probe holes will be systematically drilled in effort to identify any water bearing faults or holes. If identified in advance the area maybe grouted using one of the methods mentioned above.

However, should a water bearing fault or tunnel go undiscovered until the shaft sinking, a major in-rush of water will occur within the shaft. Current literature suggests the injection from the inside of a hydrophobic polyurethane in conjunction with stable cement based suspension grout. This grout has a high viscosity, and is capable of stopping seepage that is flowing at high velocities or in large volumes.

Additionally this grouting would only interrupt the steel liner installation for approximately 5 days (Naudts, 2003).

7.0 Steel Liner design

The structural design component in our project is the steel shaft liner. This aspect is crucial in restraining the collapse of the excavated shaft from water and soil pressure. As well, a steel shaft liner will enable the shaft to have a longer life span that will eventually enable the Money Pit to become a tourist destination. The steel shaft liner design was obtained by referencing to the text in the SME Mining Engineering Handbook authored by J. de la Vergne and L.O. Cooper (1983), with sample calculations shown in the Appendix B.

7.1 Steel liner maximum thickness

The first step to designing the liner was to determine the maximum pressure being exerted at the deepest point of the shaft; in this case it was 50 m. Thus, with the worst-case scenario in place the design for a maximum thickness of the steel liner can be calculated using the text by J. de la Vergne and L.O. Cooper mentioned above.

The Design Pressure is the sum of the lateral Earth Pressure at 50 m and the Hydrostatic Pressure at the same depth. The lateral earth pressure was calculated by using the values of angle of friction ($\emptyset$), from (geotechnicalinfo.com, 2012) and then inserted into the lateral pressures equation shown in Appendix B to obtain the active pressure coefficient (k_a). With the k_a value, the depth and the unit weight of soil (γ) (geotechnicalinfo.com, 2012) are inserted into the lateral active pressure equation in Appendix B to determine the lateral pressure. The reason for the presence of two different equations is that the soil profile of the Money Pit area is not uniform, thus a separate equation for sandy soil and clays exist. The difference between two soil types is the cohesion factor (c_u), which applies only in the case of cohesive soil. The values of (k_a) changes because of it, with examples shown in Appendix B.

Based on calculations from Appendix B, the lateral earth pressure profile is shown below in Figure 22. The Y-axis is depth of the soil while the X-axis consists of the increasing pressure. The value of lateral earth pressure was obtained from interpolation of known points and the resulting value was 409.5 kPa.

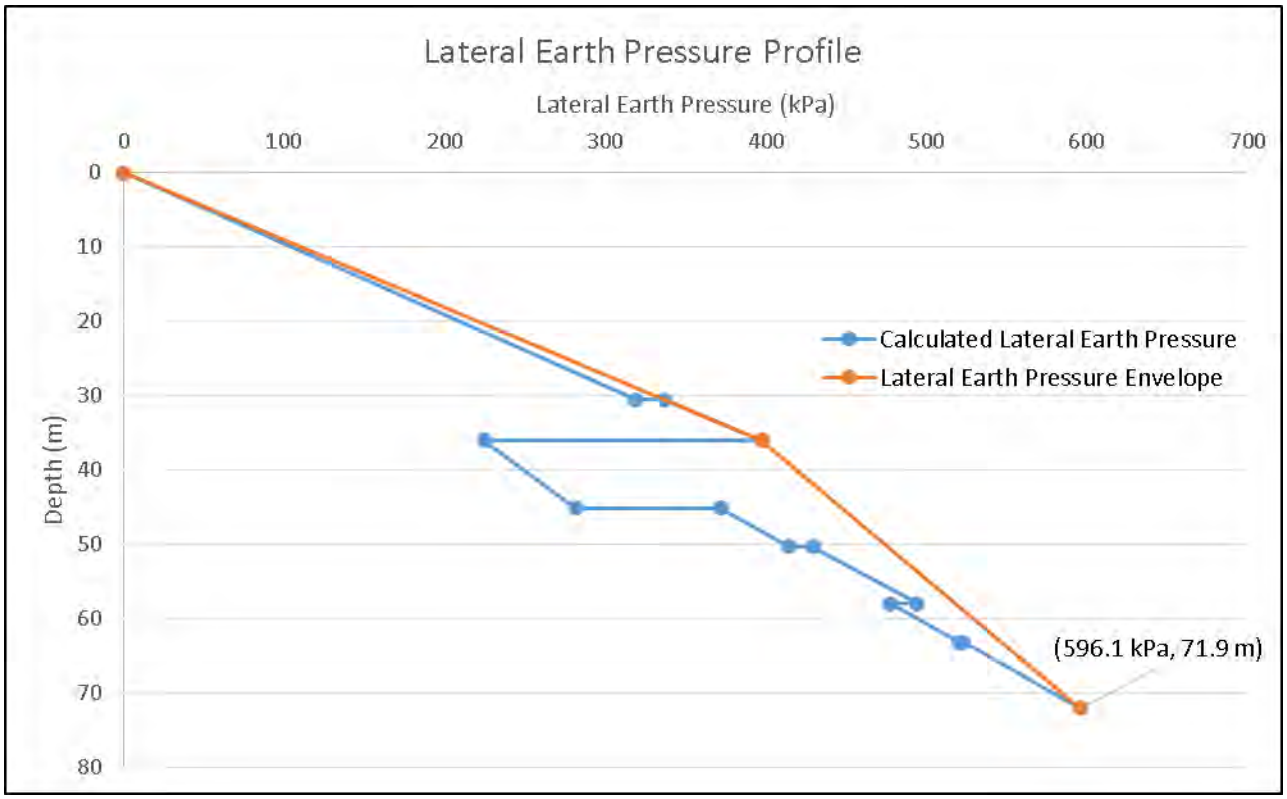

Figure 22:Lateral earth pressure profile

The hydrostatic pressure profile is much simpler than the lateral. With water pressure the distribution is a linear relationship, increasing with depth. The maximum water pressure at the lowest depth of 71.93 m is shown on the graph. The value required for the design of the steel shaft is at the 50-meter mark and this was determined to be 424.8 kPa using the hydrostatic equation shown in Appendix B. As seen on Figure 23 the water pressure does not start at zero, this is due to the fact that the water level is approximately 7 m below ground level.

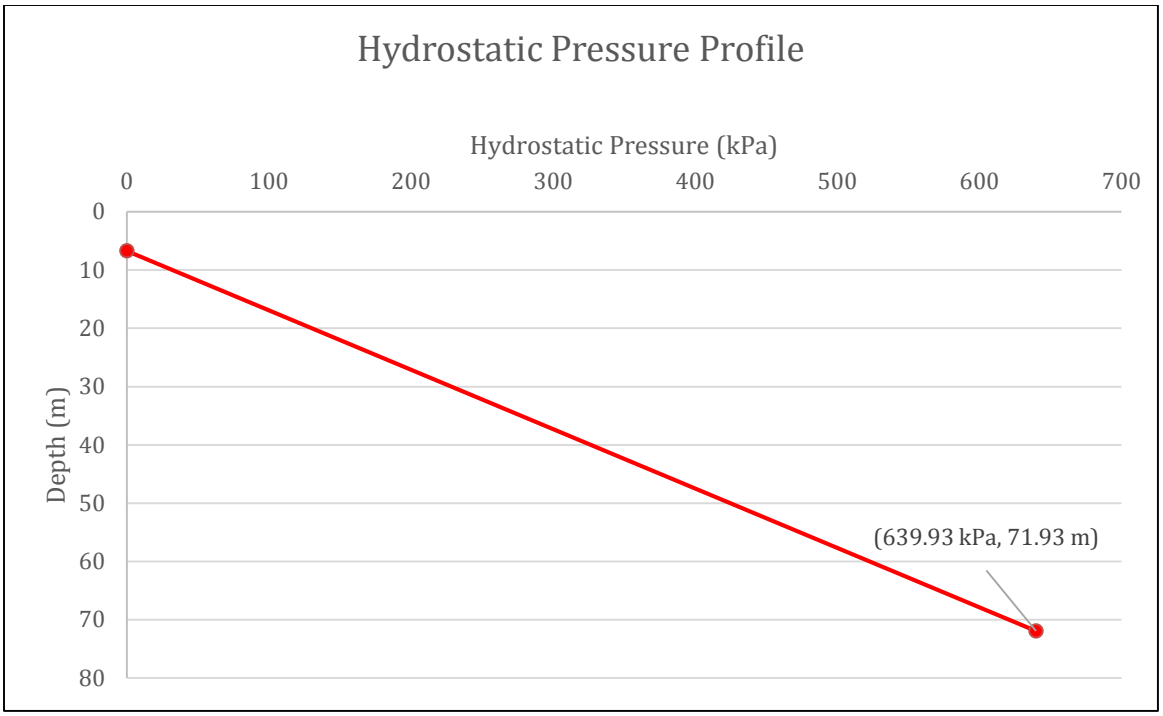

Figure 23: Hydrostatic pressure profile

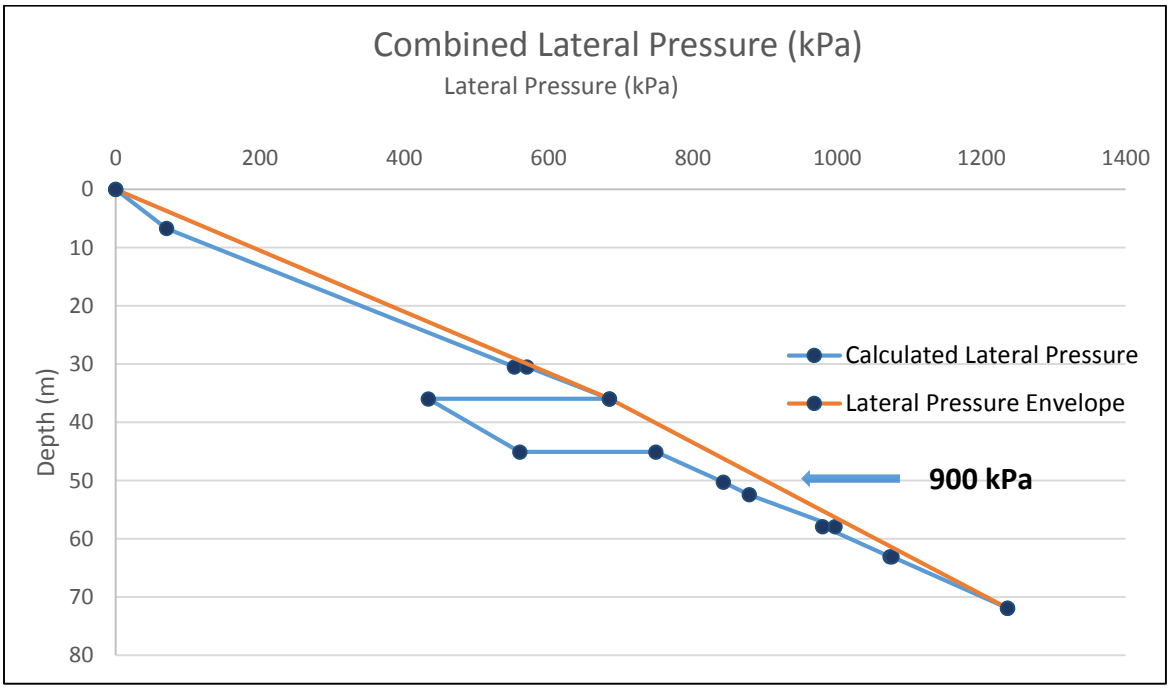

Figure 24: Combined lateral pressure profile

The lateral earth pressure was then combined with the hydrostatic pressure to produce the design pressure, this is shown in Figure 24 and the equation is provided in Appendix B.

7.2 Varying Liner Thickness

In order to be economically viable for this design the thickness of the liner will be reduced as the total lateral pressure decreases. Thus, the higher the liner is situated in the soil profile, the less steel is required to resist lateral pressure. In this design the liner is broken up into three separate thickness sections shown below. Each thickness corresponds to the varying pressures experienced throughout:

From 0 m – 20 m the thickness is 20 mm
From 20 m – 40 m the thickness is 37 mm
From 40 m – 50 m the thickness is 45 mm

The method of calculating the different thicknesses required was obtained from the SME Mining Engineering Handbook and sample calculations as well, maximum lateral pressures experienced at each segment are shown in the Appendix B.

7.3 Properties of Steel Liner

The type of steel used was a grade A-36 with yield strength of 250 MPa. This type was obtained from the text obtained from the SME Mining Engineering Handbook in which the same type of steel was used for a similar steel liner design. The liner must also be galvanized due to the high concentration of chloride ions present, as stated by (Environment Canada, 2011) within the soil profile on Oak Island.

Additionally, in order to further reduce cost the steel, the liner must be corrugated. As shown in Figure 25, the design in the referred text in SME Handbook is similar to the design for this project in terms of corrugation. Corrugated steel has been proven time and again that it is a much more efficient design than smooth walled steel in terms of shaft construction. In this design, the corrugation was not incorporated. However, if the project was to go ahead into the construction phase, a structural

engineer is recommended to modify the design of the steel liner into that of a corrugated steel liner. This was not done since it is outside the scope of this geotechnical project.

Figure 25:Corrugated steel liner similar to that of this project

7.4 Varying Stiffening Ring Size

Due to the large diameter of the shaft, the steel liner alone is not capable of adequately supporting against buckling. Thus, the use of stiffening rings is required. While using the same size stiffening rings throughout will ensure the safety of the structure, it is not viable economically. The size of the stiffening rings are divided into the same three sections like the steel liner thickness:

From a depth of 0 m – 20 m the size of the rings are 400 mm by 180 mm
From a depth of 20 m – 40 m the size of the rings are 400 mm by 300 mm

From a depth of 40 m – 50 m the size of the rings are 400 mm by 320 mm

These values are obtained from inputting the design values of the shaft into the equations provided in the text by the SME Mining Engineering Handbook. Samples calculations and the iterative process used are shown and explained in Appendix B. The shape of the stiffening rings used in this design is rectangular sections as stated in the reference text. However, it must be noted that a more conventional and economical design of an I-Section stiffening ring is more viable for this project. However, with the available text on this topic the design in this project only includes that of rectangular sections.

7.5 Transition depth

The transition depth must be checked before proceeding to design. If the value of the transition depth is higher than that of the design depth then the shell thickness is governed by buckling resistance. The theoretical transition depth was found to be 667 m, which is much higher than the design value of 50 m, thus stiffening rings are required throughout the liner.

8.0 Water Management

8.1 During Excavation

8.1.1 Uplift Prevention

For the first 7 m of the excavation process, uplift will not be of concern. However, once the excavation reaches a depth of approximately 7 m, workers will have reached the water table. The water will hinder the archaeological search and the liner installation. More importantly, due to the cohesive properties of the glacial till, the risk of blowout or heaves will continually increase as excavation approaches the broken anhydrite. The broken anhydrite layer is very pervious and it lies above the impervious sound anhydrite, therefore it will not act as a drainage layer. The downward weight will continually decrease as the excavation progresses, while the upward hydrostatic force acting on the sound anhydrite will remain constant. This force imbalance could be the source of a severe blowout (Limited, 1969). It is for this reason that Geovation Engineering recommends the use of deep well dewatering in conjunction with piezometers.

Deep well dewatering systems are used for medium to long-term dewatering projects where excavation exceeds 4 m in depth. Deep wells as opposed to well point dewatering systems contain submersible pumps placed inside the site boundaries. As the pumps are installed at depth in the wells there is no physical limit on drawdown other than the aquifer response and the performance characteristics of the pumps in use. The deep wells will create an array of widely spaced bored wells capable of pumping large sums of water. Due to the variable soil conditions on Oak Island it is of upmost importance that the wells perform in a range of soil conditions. A filter will be installed to prevent the movement of fine and suspended particles which will in turn maximize yield. Additionally, a liner and screen on the wells provide support for the boreholes and prevent collapse of unstable granular soil. This will keep the site dry for excavation and construction purposes while also providing a stable base (Sykes, 2014).

Geovation Engineering recommends the use of 4 deep wells during the construction phase. To account for the mound created by the groundwater in between the wells the water level in the wells should be 6 m below the surface of the sound anhydrite. The actual depth of the deep wells will depend on the spacing as well as other factors which must be monitored (MacPhie, 2014). Current equations for deep well dewatering systems are based off empirical data and are not very reliable as conditions are variable. Onsite technical expertise on dewatering systems will be required during the construction phase. Monitoring will be done using piezometers and the pumping level will be adjusted accordingly.

8.1.2 Water Influxes

Due to the variable geological conditions on Oak Island, it is possible that the jet grouting does not adequately prevent water infiltration through the excavation zone. As emergency grouting can cause extreme time delays and can incur many additional costs due to the high cost of specialty grout, Geovation Engineering recommends the use of probe holes.

Probing provides valuable insight to contractors on the groundwater conditions. Probe holes are usually drilled 10 to 40 m ahead of the excavation front. As the excavation area is very large, it is advised to use multiple probe holes. Quantities and pressures of water can be detected at depths ahead of the excavation front. This can prevent the use of emergency grouting, as the flow of groundwater can be significantly reduced by less costly methods if the flow is corrected before the soil is excavated. Additionally, inflow of groundwater that is not controlled can have negative effects on the stability of the excavation opening and the final product. Even if the probes do not encounter any problems , the probe holes are a necessary component of the project from an economical and safety perspective (Henn, 1996).

8.2 Post Excavation

In order for the Money Pit to be accessible after the excavation and proper conservation of the archaeological artifacts, the uplift and seepage caused by the groundwater must be controlled. Due to this reason, a concrete slab will be created directly below the steel shaft. The concrete slab will restrict seepage through the pervious broken anhydrite and act as a horizontal water barrier at the base of the excavation. The concrete slab will have to be dimensioned to resist uplift water pressure. The weight of the slab must counterbalance the uplift pressure. However, as the concrete slab will be below the water level it will have to resist large water uplift pressure. It may be required to shape the concrete slab as an inverted arch to take the uplift force through compressive stresses that are then transferred to the steel liner walls. Geovation Engineering also recommends the use of anchors due to the high pressure (Paolo Croce, 2014).

The structural elements below the suggested base slab should also be reinforced to resist seepage. Geovation recommends the use of shotcrete in conjunction with mesh to surround the chambers, the reception area and elevators. Dental concrete should also be considered as it is an effective sealing agent. That being said, the use of a sump pump and pit will be required (MacPhie, 2014; Wonnacott, 2014). It is not a realistic design parameter to create completely waterproof elements out of concrete. The sump pit may be placed under the elevator. Dimensions of sump pit will be determined in the detail design.

9.0 Construction Sequence

Developing a comprehensive construction sequence for the proposed project of a deep excavation, which involves the application of jet grout techniques and the installation of a steel liner, is a fundamental component of this feasibility study report. This is because in order to develop an accurate preliminary cost estimate, an understanding of the sequence of the individual events that are necessary for the implementation of this project and an estimated duration of these individual events is required. It is an important tool, which will aid in the detailed design phase and in the management and execution of this project. In the following sections, Geovation Engineering outlines a recommendation of a proposed construction sequence, which involves the choice of technology, the definition of work tasks, and the identification of any interactions among the different work tasks. A Gantt chart outlining the duration for the individual work tasks can be found in Appendix C.

9.1 Preliminary Investigations

The geological profile at the proposed location of the deep excavation is demonstrated in Figure 26. Prior to implementing the proposed design, Geovation Engineering recommends that extensive preliminary investigations be conducted with the specific purpose of uncovering archaeological and geological information of the site subsurface conditions. As discussed in Section 5**,** ultimately, the objectives of these investigations include verifying the presence of and delineating the region of man-made workings and any artifacts of considerable valuable that can be directly associated to the activities of the Original Depositors. In addition, the goal is to gather enough information to be able to provide optimal direction during the detailed design of a deep excavation in order to facilitate the recovery artifacts and to identify the historical and archaeological context of the site.

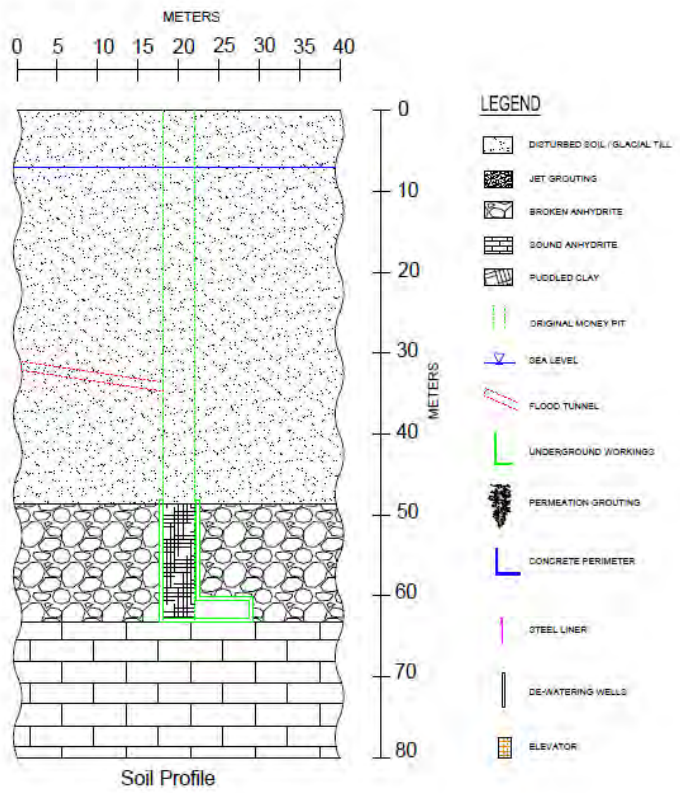

Figure 26: Cross-section of the soil profile through the center of the proposed 24 m diameter shaft

To achieve the goals outlined above, Geovation Engineering recommends that comprehensive archaeological investigations and subsurface explorations be conducted. In the following sections, the processes by which these investigations and explorations will take place will be discussed in detail.

9.1.1 Archaeological Investigation

Prior to excavating on the site of Oak Island, a comprehensive investigation performed by archaeologists is required. This involves preparing a research design, which outlines what questions the archaeologists will try to answer and the techniques they will use to excavate and analyze the data.

Geovation Engineering recommends that almost all of the archaeological effort should be done in conjunction with the deep excavation and the installation of the steel shaft liner, which would require the archaeologists to be on site at all times. Protocols should be set up such that the archaeology does not stall the construction.

Geovation Engineering recommends that the archaeologists review all of the existing records of the site and perform a comprehensive site survey. This site survey will result in the development of a master map of the site. In addition, a report on the site should be completed, which includes giving the site an identification number, and recording information about the vegetation, soil, elevation, and location. The archaeologists should always be aware of the activities that are proceeding on site. In order to ensure this, they should be included in the process of construction planning and they should be constantly informed of the developments during the site investigation, drilling and jet grouting (Wonnacott, 2014).

To minimize the amount of time required for the archaeological investigation of this site, Geovation Engineering recommends that the archaeological investigation of the excavated material be separated into zones. For the zone where the excavated material is known to be backfill, the excavation can proceed as would a typical excavation, the material just needs to be removed and then dumped. Once the excavation gets below the backfilled material, but it is still in overburden, the undisturbed natural soil and the soil that may contain old workings should be separated. The undisturbed natural soil should be removed and dumped. The soil that may contain the old workings should be removed to a special stockpile where it is then run through a screen, while the archaeologists are carefully watching and recording anything of interest (Wonnacott, 2014). When the excavation cuts through the old Searcher's tunnels or Flood Tunnels, the exploration of the tunnels will be completed once the construction of the steel shaft liner has been completed. The archaeologists will wait until the steel shaft liner has been completed and then cut a hole in the steel shaft liner, drill and grout horizontally around the tunnel,

build a short adit, and have the archaeologists carefully excavate and record the results they find in the tunnels. When the excavation for the steel shaft liner reaches the broken anhydrite, and the rock shaft is exposed, the rock shaft should be carefully excavated using smaller excavating equipment, and all the material removed should be placed in a stockpile where it will be carefully searched for artifacts. Finally, when the excavation reaches the lowest point in the rock shaft, and a horizontal tunnel appears to lead to a treasure chamber(s), the archaeologists should carefully excavate the rest of the way recording everything they find as they progress, using a grid system. Careful design and construction planning should allow the archaeologists to explore the treasure chambers while the construction team builds elevators, staircases, and reinstates the Money Pit (Wonnacott, 2014).

Any artifacts and various kinds of samples that have significance to the historical heritage of the site, should be sent to specialists for analysis. Months after the excavation is finished, results of the analyses will be ready. Regardless of where they are stored, artifacts and information should be available to future researchers, as well as for use in displays.

9.1.2 Subsurface Exploration

Subsurface exploration includes obtaining samples and carrying out testing from exploratory holes to obtain information about the ground conditions. The exploratory holes may include hand or machine excavated trial pits, mini rig boring, light cable percussive boring or rotary boring depending on the findings of the Phase I investigation and the requirements of the investigation as discussed in Section 5. Geovation Engineering recommends that the subsurface exploration begins once the preliminary archaeological investigation and the site survey is completed.

The amount of time to complete one sample borehole and the drilling production rate, depend on many variables. Some variables include; the diameter of the drill

hole, the type of drill bit (diamond bit, tricone, churn bit, etc.), and the nature and type of formation of the soil encountered. In addition, part of the time required to drill each hole is dependent on the time it takes to set up the drill rig, such as whether or not it has to be dismantled fully or partly before the rig can be moved from hole to hole. Finally, the required degree of precision for the hole location adds a variable amount of time to the process depending on how accurately the position must be established (Wonnacott, 2014). Typically, for holes of similar diameter, the amount of sampling or testing that must be done as the hole advances has the most impact on the time to complete each hole.

9.2 Site Preparation

Once the preliminary investigations have been completed, the preparation of the site can begin. The first step of the site preparation is to remove all of the vegetation if there exist any on site. Any trees obstructing the construction process will be cut down, and their roots will be uprooted as directed by authority. Secondly, the whole area will be roughly leveled. Any holes on the site will be filled with sand or compacted earth and leveled as required or redirected by the authority. Finally, fixing the position of the site office, the guard and labor shed, and the construction of the access roads for trucks and equipment needs to be completed as part of the site preparation phase.

9.3 Jet Grouting

The jet grouting will be performed after the site has been completely prepared for the excavation of the deep shaft. The configuration of the jet grout curtain is demonstrated in Figure 27. The jet grouting procedure was previously discussed in Section 6.0**.** Jet grouting consists of two parts. The first part is the part where the hole is drilled to the desired depth, and the second part is to release the grout and construct the column. Drilling production rates will vary depending on the nature and type of formation of the soil encountered. The time to drill a hole depends on

many things. Some of the variables include, the diameter of the drill hole, the type of drill bit, and how long it takes to set up the drill rig. If it has to be dismantled fully or partly before the rig can be moved from hole to hole. Finally, the required degree of precision for the hole location adds a variable amount of time to the process depending on how accurately the position must be established.

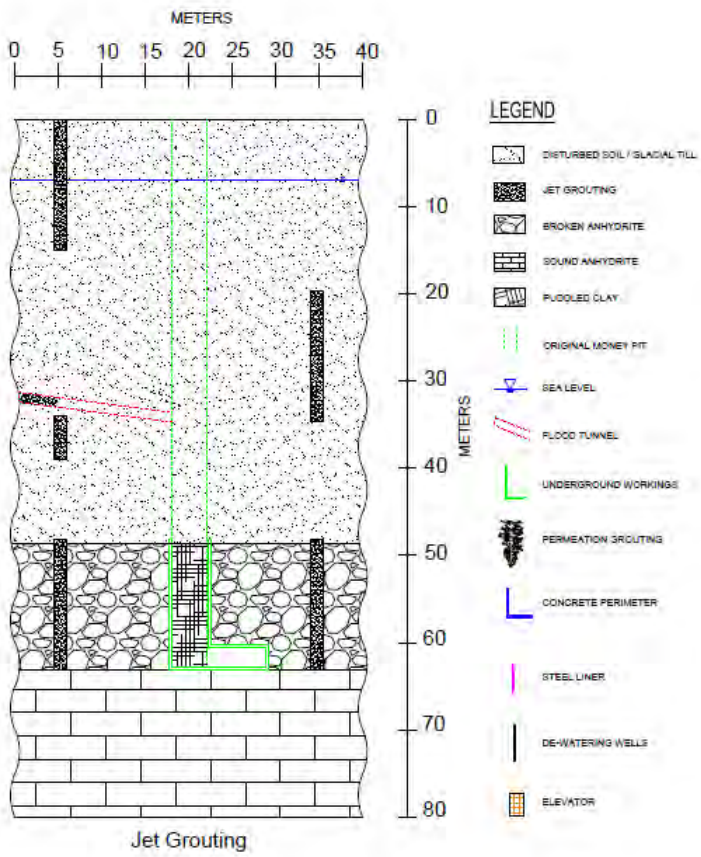

Figure 27:Cross-section of the jet grouting profile in the broken anhydrite and intermittently through the glacial till layers

Since extensive preliminary investigations were performed, which involved extensive subsurface exploration, there is no sample recovery required while the hole is being drilled. Sample recovery is only required during the retrieval and observation of drill cutting as the hole is advanced, so the drilling process is very quick (Wonnacott, 2014). For jet grout holes that are up to 65 m deep, it has been estimated that 2 to 3 hours should be sufficient to drill the hole and survey the hole

to ensure that the correct alignment and vertical angle has been achieved. Geovation Engineering recommends that a minimum of 4 jet grout rigs should be used at the same time depending on the access to the site and the space on site that is available and suitable for the rig. During the construction of the jet grout column, the typical lift rate is in the order of 12 seconds for every 7.6 cm of column length. Using this parameter, coupled with the estimated time to drill the hole, Geovation Engineering provides an estimation of the time it will take to complete one column. Under ideal circumstances, it will take approximately 34 minutes to construct a jet grout column. To account for some contingencies, Geovation Engineering developed the construction schedule by estimating that the average construction of a jet grout column will take 1 hour. This coupled with the conservative estimate of the drill hole taking approximately 3 hours to drill, the time it will take to complete 1 jet grout column, is approximately 4 hours. This value coupled with the ability to use a minimum of 4 rigs at one time decreases the amount of time it will take to complete the jet grouting component of the project. If the crews operating the jet grout rigs are working 10 hours per day, this means that approximately 10 jet grout columns can be completed in 1 working day.

In total, 64 jet grout columns will be constructed around the perimeter of the proposed deep excavation in the broken anhydrite; therefore, 7 working days are required to complete this part of the project. It should be noted that this number does not include the amount of columns required to prevent the seepage of water through the sand layers in the glacial till (disturbed soil) location of the excavation. Determining where grout columns are required in this location will be completed during the preliminary investigations and the construction schedule will have to be modified where appropriate to account for this component of the jet grouting stage.

9.4 Permeation Grouting

Geovation Engineering recommends that the jet grout columns be constructed first and completed. The crews can then proceed with the grouting of the rock formation

(sound anhydrite) below the jet grouted columns as seen in Figure 28. In addition, Geovation Engineering recommends that the permeation grouting be performed from within the shaft.

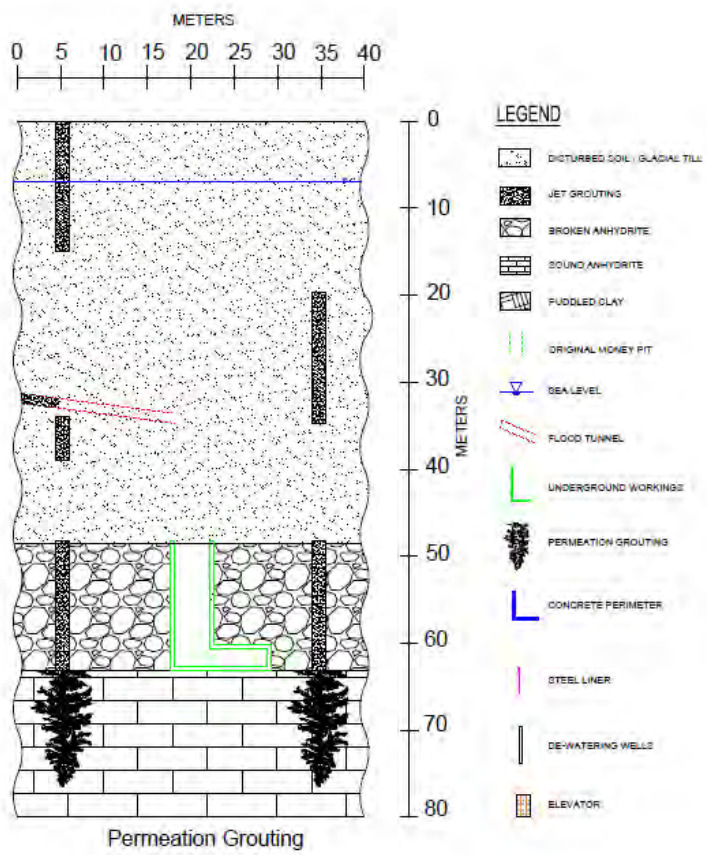

Figure 28: Cross-section of the permeation grouting profile in the broken anhydrite

As discussed in Section 6.2 permeation grouting of the rock formation (sound anhydrite) will be dictated by the volume of grout injected into the formation, which is dictated by the permeability of the formation. Permeation grouting is highly dependent on preliminary investigations and preliminary field tests. Geovation Engineering estimates that 90 holes will have to be drilled to complete the permeation grouting stage, which will take approximately 10 days to complete.

9.5 Stage 1 Excavation and Liner Construction

The excavation of the deep shaft can only begin once the ground improvement process has been completed. The excavation and liner construction process has been divided into two stages. The first stage being the excavation that proceeds until the groundwater level has been reached, which is at a depth of approximately 7 m as seen in Figure 29. Geovation Engineering recommends that the excavation and liner construction proceed in 1 m lifts and that the excavation of the entire shaft is to be documented and photographed by an archaeologist. The process of the archaeological investigation during the excavation was discussed in Section 9.1.1.

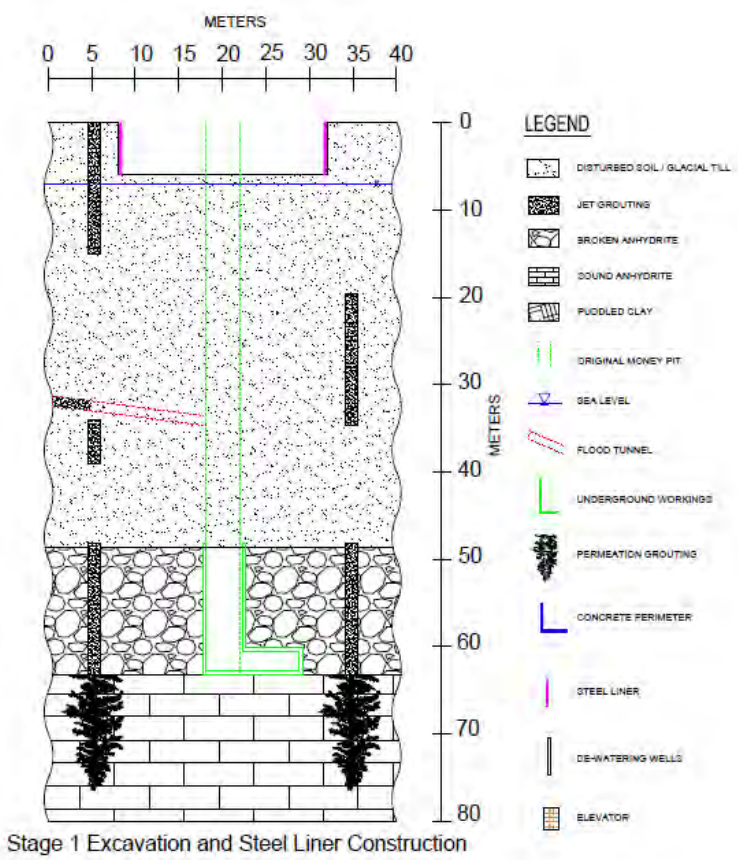

Figure 29: Cross-section of the Stage 1 deep excavation and steel shaft liner construction

Geovation Engineering recommends that one or two excavators be used for the deep excavation. Excavators combine digging and lifting abilities and come in a wide range of sizes. They are ideal for digging and dumping into a dump truck or pile and can accommodate numerous attachments such as pinchers for lifting logs or pipes, a jackhammer for busting up concrete or compacted soil, or a magnet for metal material moving.

9.6 Dewatering and Instrumentation

The dewatering wells and piezometers need to be installed shortly after the excavation reaches below the sea level, once Stage 1 of the excavation and liner construction has been completed as seen in Figure 30. The objectives of the dewatering wells is to relieve the water pressure in the anhydrite bedrock and the objective of the piezometers is to monitor the water pressure in the anhydrite bedrock. Geovation Engineering recommends that 4 dewatering wells be drilled to approximately 67 m inside the steel shaft liner. Dewatering of the partly completed shaft will be augmented by 7.6 cm, high head shaft pumps temporarily located in low areas of the excavated shaft. These will be aided by 5.0 cm trash pumps within the shaft excavation area, that pump water locally to the shaft pumps for removal from the excavation.

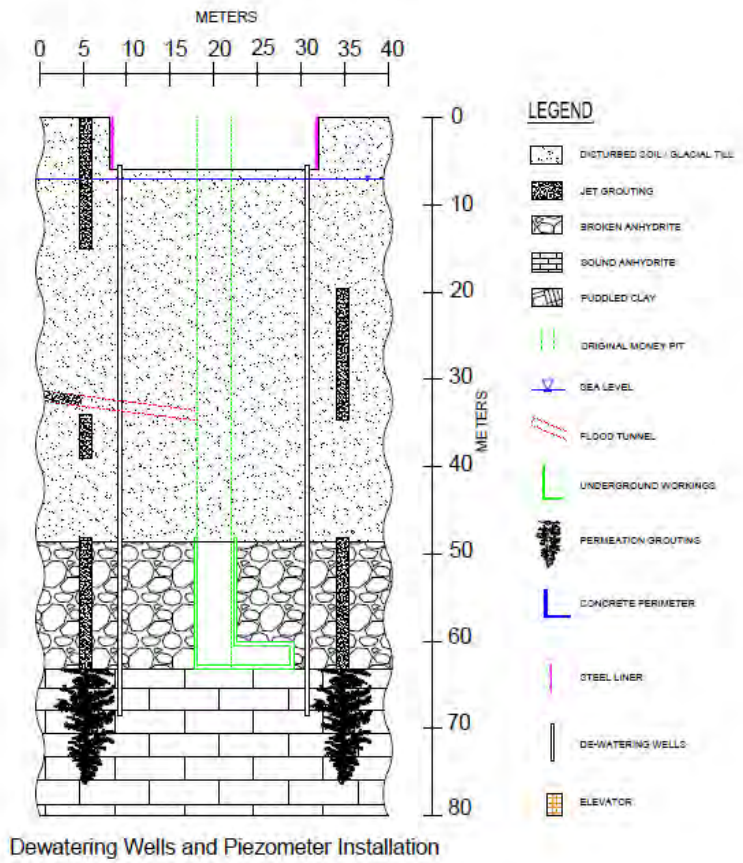

Figure 30:Cross-section of the location of the dewatering wells inside the steel shaft liner

9.7 Stage 2 Excavation and Liner Construction

Stage 2 of the deep excavation and the liner construction occurs once the ground water level has been surpassed and the dewatering and instrumentation stage has been completed. This stage of the excavation proceeds to a depth of 50 m, which is the base of the steel shaft liner as seen in Figure 31. For this stage of the excavation, probing ahead of the working surface is necessary to explore for groundwater, which may be present in unacceptable large quantities or elevated pressures due to holes in the jet grouting curtain. Probing is used to detect changes in the geological structure and type of materials which may exist immediately in front of the advancing excavation. They are typically drilled to some distance, usually 10 to 40 m, ahead of the excavation working surface.

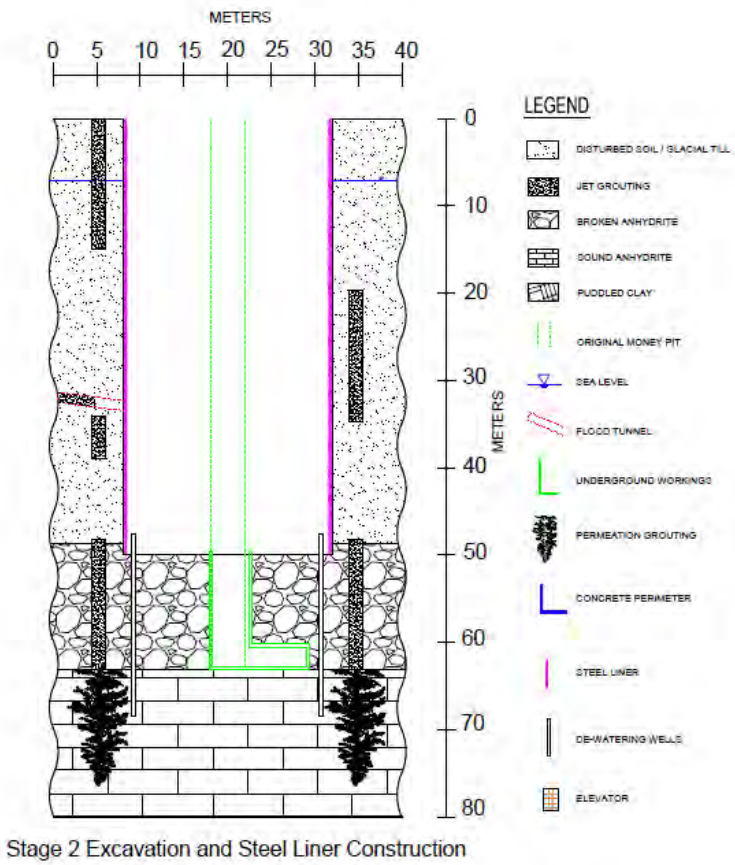

Figure 31: Cross-section of Stage 2 of the deep excavation and steel shaft liner construction

9.8 Bottom Slab

Once the deep excavation and the steel liner installation has been completed, a concrete slab will be poured into the bottom of the excavation in order to prevent long term uplift as seen in Figure 32. In addition, the concrete slab will be the base for the construction of the stairs, the elevator and the reinstatement of the original Money Pit platforms.

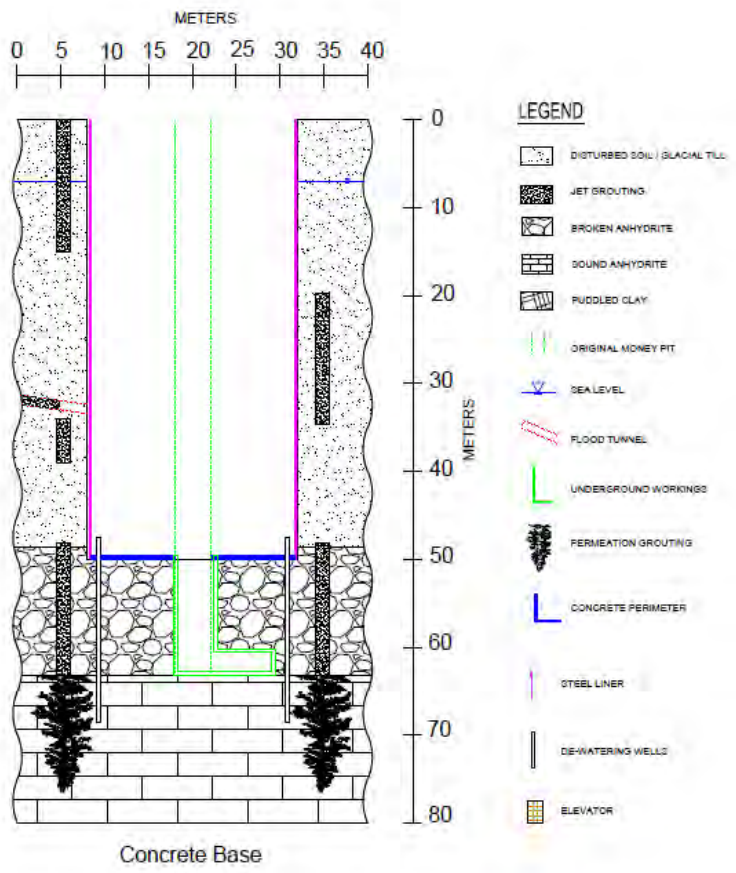

Figure 32: Cross-section of the installation of a concrete slab in the base of the excavation

9.9 Stabilization of the Original Workings

After the concrete slab and the construction of the elevator has been completed, the stairs and the reinstatement of the original Money Pit platforms can begin. Followed by the stabilization of the chambers and the original Money Pit shaft, which are located in the broken anhydrite as seen in Figure 33. Geovation Engineering recommends that this stage be completed by grouting the entire perimeter of these structures. This will be done to ensure the safety of the individuals working in that location and of the individuals visiting for tourism purposes.

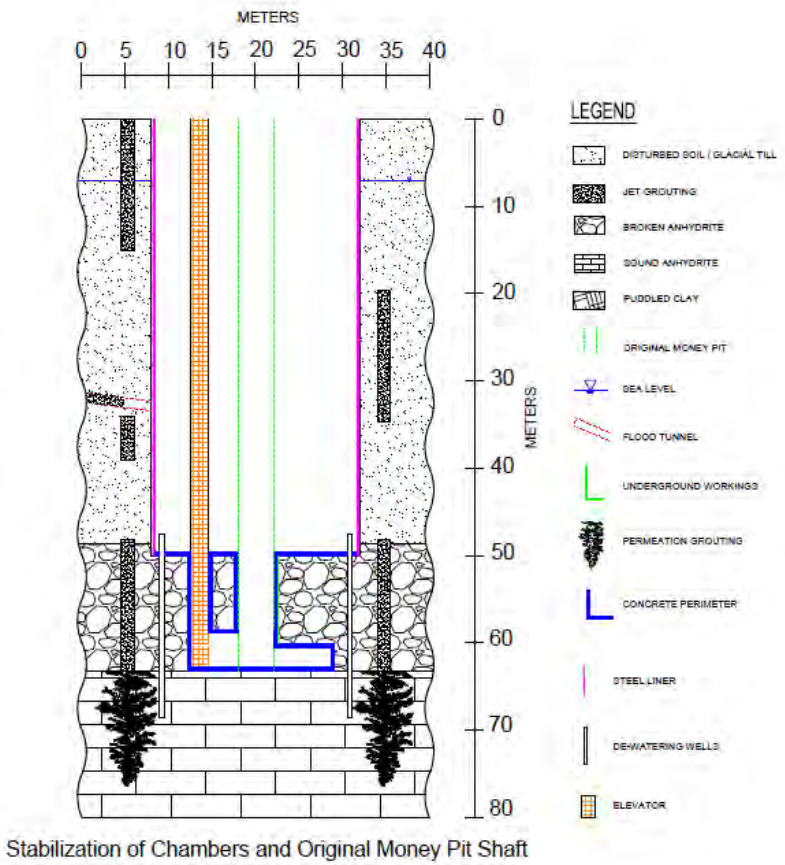

Figure 33: Cross-section of the stabilization of the chambers and the original Money Pit Shaft

10.0 Cost Estimate

Geovation Engineering has developed a preliminary cost estimate for the implementation of the proposed project of a deep excavation intended to preserve the archaeological heritage of Oak Island, Nova Scotia. This cost estimate was created using the Excel and the completed spreadsheet can be found in Appendix C.

The cost estimate includes all components of the construction sequence discussed in Section 9.0**:** the cost of the preliminary investigations, site preparation, jet grouting, permeation grouting, Stage 1 excavation and steel shaft liner installation, dewatering and instrumentation, Stage 2 excavation and steel shaft liner installation, archaeological investigations, maintenance, indirect expenses, and project management. The installation of the staircase, the elevator, the reinstatement of the original Money Pit platforms, and the museum construction are the only components not included in the cost estimate.

The values presented in the cost estimate were determined through research or were provided to Geovation Engineering by industry specialists. Specifically, a quote for the ground improvement component, the jet grouting and the permeation grouting, of the cost estimate was provided by the grouting consultants at ECO Grouting Specialists Inc. In addition, a percentage contingency was also added to the subtotal of each stage. The size of the contingency was determined by the experience of the estimator preparing the various cost components, and the complexity and degree of the unconventionality of the cost components. Due to this report being a feasibility study rather than a detailed engineering design, the percentage contingency ranged between 25% and 45%. In addition, the more relatively unused techniques or materials that the component of the project consisted of, the greater the contingency allowance was.

In summary, the most expensive component of the project is the installation of the steel shaft liner, due to the cost of steel, this ends up being approximately $4,197, 417 CAD. Following the steel shaft liner installation, the next most expensive components of the project are the permeation grouting and the jet grouting stages, which cost approximately $2,660,070 CAD and $2,362,523 CAD, respectively. The total cost of the project is approximately $13,380,490 CAD. However, Geovation Engineering recommends that a more exhaustive cost estimate be developed during detailed design.

11.0 Conclusion

What lies beneath Oak Island has repeatedly entranced treasure seekers into tantalizing quests to solve the 200 year old mystery. From the Money Pit's first discovery in 1795, to the Truro syndicate in 1845, to present-day drillings, efforts to retrieve what lies deep within the subsurface have been unsuccessful. Through comprehensive research and meticulous design, the engineers at Geovation Engineering have developed a feasible solution to one of the world's most bewildering mysteries. The use of ground improvement techniques enabled Geovation Engineering to devise a system to counter the influx of water countlessly hindering previous expeditions. Jet grouting, coupled with the use of a 24 m diameter steel liner, will allow Geovation specialists to excavate without major complications. Furthermore, the corrugated steel shaft liner and stiffening rings will provide sufficient support to resist the lateral pressures at the 50 m depth. To combat the uplift pressures that will be encountered, the combination of permeation grouting and a concrete base have been included in the design. The permeation grouting will alleviate some pressure by diverting water flow, while the weight of the concrete base will counter the seepage uplift forces. Geovation Engineering has undoubtedly considered all technical aspects of retrieving the human workings believed to be present below. However, to ensure all questions to the mystery are answered, archeological investigations will take place in conjunction with the excavation.

This project proved particularly challenging, largely due to the numerous components needed to complete a feasible design. The engineers at Geovation Engineering responsible for creating the aforementioned system encountered various difficulties throughout the design process. For example, the research phase of the project was problematic due to the outdated, and scarce sources of information. Furthermore, learning new concepts such as jet grouting and shaft design, as well as the use of certain software, proved very time consuming. Analyzing data was also challenging, particularly due to the necessary assumptions

needed to complete various preliminary design calculations. Other obstacles included software errors and limitations, relevant design material on steel shafts, and conflicting information regarding site conditions. Nevertheless, despite all hindrances, the engineers at Geovation Engineering not only surmounted these obstacles, but cooperated efficiently to create a unique, feasible, and safe design. The difficulties of the project disguised as a unique opportunity for the engineers to learn how to create attainable schedule deadlines, manage their time efficiently, effectively employ their strengths, and cooperate together to achieve a common goal. Through the help of geotechnical specialists, the use of modern technology, and implementing rigorous design procedures, Geovation Engineering has created a solution that will finally put an end to the 200 year old mystery.

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Sources of Data

Our External advisor, Les MacPhie, Senior Geotechnical Engineer at SNC-Lavalin provided, us with all the Data to learn about the history of Oak Island, and previous attempts to retrieve the treasure. He also provided us with geotechnical reports (by Becker, Warnock Hersey and Golder) and various technical reports written by Mr. MacPhie himself.

Historical Background

All the historical information to learn about the extensive history of Oak Island was obtained from the Book by Harris and MacPhie (2005) Oak Island and its Lost Treasure, and from The Secret Treasure of Oak Island by D'Arcy O' Connor.

Subsurface Conditions

Sources of Sub-surface conditions includes borehole records by three companies:

1. Becker Drilling (1967)
2. Warnock Hersey (1969)
3. Golder Associates (1970/71)

Ground Improvement Design

The ground improvement techniques included a combination of jet grouting and permeation grouting. The primary source of information was from the book by Croce, Flora, and Modoni (2014), *Jet Grouting Technology, Design and Control.* In addition, two industry consultants, Mr. Ward Naudts and Eric Landry, from ECO Grouting assisted us with our design.

Codes and Regulations

Handbook of Steel Construction, Tenth Edition, Canadian Institute of Steel Construction

Canadian Foundation Engineering Manual, Forth Edition (2006)

Division of Responsibility

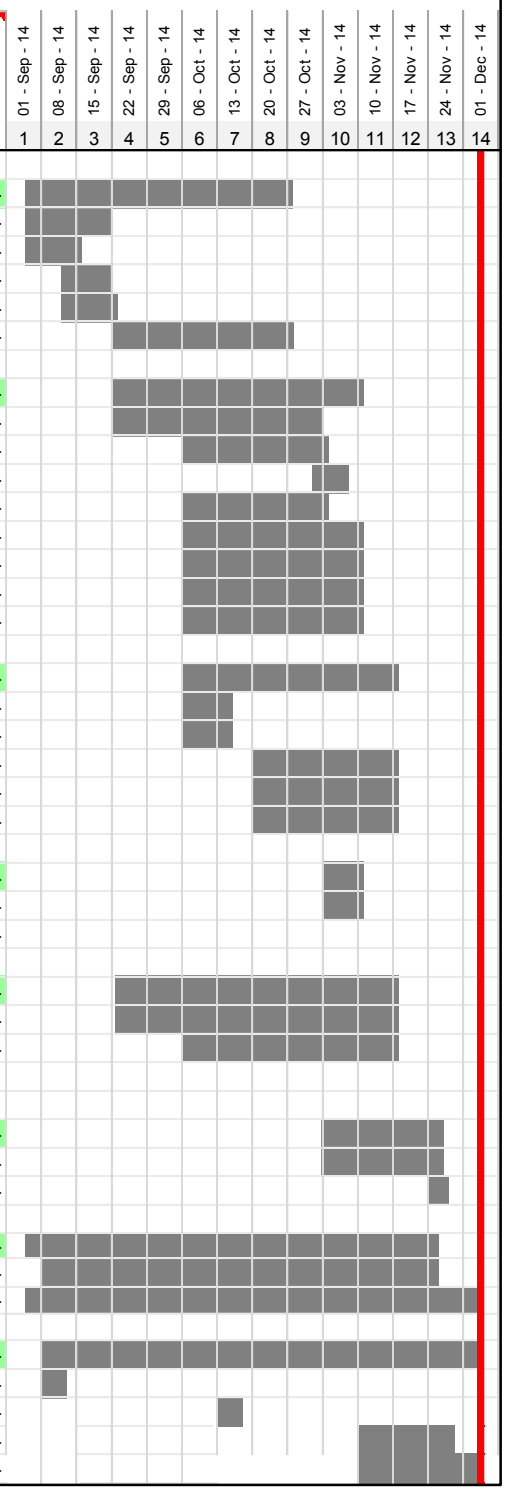
Oak Island - Project Schedule

Geovation Engineering

Project Start Date: 9/5/2014 (Friday)

Today's Date: 12/4/2014 (Thursday)

WBS	Task	Team member(s)	Start	End
1	**Preliminary Analysis**		Fri 9/05/14	Mon 10/27/14
1.1	History of Oak Island	All	Fri 9/05/14	Sun 9/21/14
1.2	Scope of the Project	All	Fri 9/05/14	Mon 9/15/14
1.3	Previous Design Attempts	All	Fri 9/12/14	Sun 9/21/14
1.4	Adequate Excavation Diameter	All	Fri 9/12/14	Mon 9/22/14
1.5	Additional Preliminary Investigations	All	Mon 9/22/14	Mon 10/27/14
2	**Jet Grout Design**		Mon 9/22/14	Mon 11/10/14
2.1	AutoCAD: 2D Soil Profile	Mariam & Nabil	Mon 9/22/14	Sun 11/02/14
2.2	Anhydrite	Rebecca	Mon 10/06/14	Mon 11/03/14
2.3	Sound Anhydrite Bedrock	Claire	Sat 11/01/14	Fri 11/07/14
2.4	Sand	Claire	Mon 10/06/14	Mon 11/03/14
2.5	Disturbed Soil	Mariam	Mon 10/06/14	Mon 11/10/14
2.6	Large Cavities	Rebecca	Mon 10/06/14	Mon 11/10/14
2.7	Techniques	Rebecca	Mon 10/06/14	Mon 11/10/14
2.8	Grout columns Location and Quantity	Rebecca	Mon 10/06/14	Mon 11/10/14
3	**Steel Shaft Liner Design**		Mon 10/06/14	Mon 11/17/14
3.1	Lateral Earth Pressures	Nabil	Mon 10/06/14	Wed 10/15/14
3.2	Hydrostatic Pressures	Nabil	Mon 10/06/14	Wed 10/15/14
3.3	Hoop Stress	Peter	Mon 10/20/14	Mon 11/17/14
3.4	Depth of Individual Sections	Peter	Mon 10/20/14	Mon 11/17/14
3.5	Thickness	Peter & Nabil	Mon 10/20/14	Mon 11/17/14
4	**Uplift**		Mon 11/03/14	Mon 11/10/14
4.1	Blowout Prevention	Rebecca	Mon 11/03/14	Mon 11/10/14
4.2	Long-Term Uplift Prevention	Mariam	Mon 11/03/14	Tue 10/14/14
5	**Cost Estimate**		Tue 9/23/14	Mon 11/17/14
5.1	Spreadsheet	Claire & Peter	Tue 9/23/14	Mon 11/17/14
5.2	Construction Schedule	Claire	Mon 10/06/14	Mon 11/17/14
5.3	Construction Sequence Drawings	Claire & Nabil		
6	**Three-Dimensional Model Design**		Mon 11/03/14	Wed 11/26/14
6.1	3D Model using SketchUp	Mariam	Mon 11/03/14	Wed 11/26/14
6.2	Animation	Mariam	Mon 11/24/14	Thu 11/27/14
7	**Meetings**		Fri 9/05/14	Tue 11/25/14
7.1	Meeting with Les MacPhie and John Wonnacott	All	Mon 9/08/14	Tue 11/25/14
7.2	Group meetings	All	Fri 9/05/14	Thu 12/04/14
8	**Presentations and Reports**		Mon 9/08/14	Thu 12/04/14
8.1	Introductory Presentation	All	Mon 9/08/14	Fri 9/12/14
8.2	Midterm Presentation	All	Mon 10/13/14	Fri 10/17/14
8.3	Final Presentation	All	Mon 11/10/14	Fri 11/28/14
8.4	Final Report	All	Mon 11/10/14	Thu 12/04/14



Hours

	Hours of work per person				
	Mariam	**Rebecca**	**Nabil**	**Claire**	**Peter**
Week 1: September 5th- September 7th	2	2	2	2	2
Week 2: September 8th-September 14th (Introductory Presentation)	21	21	20	20	19
Week 3: September 15th-September 21	10	9	7	10	10
Week 4: September 22nd-September 28th	15	10	12	16	17
Week 5: September 29-October 5th	10	14	19	20	20
Week 6: October 6th-October 12th	22	25	17	20	20
Week 7: October 13th- October 19th (Midterm Presentation)	28	28	26	27	25
Week 8: October 20th-October 26th	14	17	20	19	16
Week 9: October 27th-November 2nd	19	20	20	19	20
Week 10: November 3rd-November 9th	20	16	18	10	15
Week 11: November 10th-November 16th	28	28	25	22	22
Week 12: November 17th-November 23rd	30	32	29	32	28
Week 13: November 24th-November 30th (Final Presentation)	38	38	38	38	40
Week 14: December 1st- December 4th (Final Report write up)	27	29	30	30	29
Total Hours	**284**	**289**	**283**	**285**	**283**

Appendices

Appendix – A: Ground Improvement

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Design Diameter of Jet Columns		
Designed by: Mariam Abdul Ghani	**Checked by:** Rebecca Stanzeleit	**Verified by:** Nabil Nassor

Table 1 - Archaeological Evidence from Becker Drilling 1967 (Source: After (MacPhie, 2008))

Becker Drilling 1967			
Borehole	**Depth (m)**	**Material encountered**	**Interpretation**
B11	48	Wood	-
	56 to 61	Puddled clay	-
	59.7	Two Oak buds embedded in a clay sample	-
B13	56 to 61	Puddled clay contained coarser pebble size at regular intervals of 45 cm	Man-made type of clay
B14	56 to 61	Puddled clay	-
B17	54 to 60	Likely puddled clay	-
B21	53.6	Piece of slightly crumbled brass foil	Brass had been torn from a large piece of brass in the ground
	53.6 to 58.5	Puddled clay	-
	61 to 62.8	Stagnant water and possible cavity	-
B24	58.5	10.16 cm of wood, 30.48 cm of clay, 10.16 cm of wood and then a 1.83 m cavity	-
B25	58.2 to 60.3	2.1 m cavity	-

	60.3	A hard obstruction at the base of the cavity	The sound of the diamond drill on the obstruction suggested that it is iron metal and penetrated a depth of ½ inch.
B33	58 to 58.5	Clay	-
	58.5	Wood	-
	58.5 to 60.3	A cavity containing soil and a crude lime mortar	-
	60.3	Rock	-
B35	55	15.24 to 20.32 cm of wood	-
	55 to 58.5	A partial cavity	-

Table 2 - Archaeological Evidence from Warnock Hersey 1969 (Source: After (MacPhie, 2008))

Warnock Hersey 1969			
Borehole	**Depth (m)**	**Material encountered**	**Interpretation**
W9	58.5 to 60	Wood	-
	61 to 62.8	Cavity	-

Table 3 - Archaeological Evidence from Golder Associates 1970/71 (Source: After (MacPhie, 2008))

Golder Associates 1970/71			
Boreholes	**Depth (m)**	**Material encountered**	**Interpretation**
G103 (through the Hedden Shaft)	58.5 to 60.3	Samples indicated soil consisting of recent soil from the surface.	Suggesting that the zone consists of backfilled soil by the original depositors

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Design Diameter of Jet Columns		
Designed by: Rebecca Stanzeleit	**Checked by:** Mariam Abdul Ghani	**Verified by:** Nabil Nassor

Deterministic Approach to Diameter Design

$$D_d = \frac{D_K}{\gamma_D}$$

$Where$:

$$D_d = Design\ Value\ of\ Column\ Diameter$$
$$D_K = Characteristic\ Value\ of\ Column\ Diameter$$
$$\gamma_D = Partial\ Safety\ Factor\ for\ Diameter\ of\ Column$$

Characteristic Value of Column Diameter Calculation (D_K)

From Table 4 of Appendix A:
Assuming that a triple fluid treatment system will be used.
Modeling the broken anhydrite and loose soil after silty sand the mean diameter of the jet grout column is given by 1.2-2.5m. (Also within the sand and gravel limit of 1.5-3)

$$D_K = 1.2 + \frac{2.5 - 1.2}{2} = 1.8$$

Table 4 – Column Diameter Selection

Treatment system	Mean diameter of columns (m)			
	Moderately stiff clay	Soft silt and clay	Silty sand	Sand and/or gravel
Single fluid	NR*	0.4–0.8	0.6–1.0	0.6–1.2
Double fluid	0.5–1.0	0.6–1.3	1.0–2.0	1.2–2.5
Triple fluid	0.8–1.5	1.0–1.8	1.2–2.5	1.5–3.0

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Design Diameter of Jet columns		
Designed by: Rebecca Stanzeleit	**Checked by:** Mariam Abdul Ghani	**Verified by:** Nabil Nassor

Partial Safety Factor for Diameter of Column Calculation (γ_D)

From Table 5 of Appendix A:
Assuming high soil heterogeneity, poor experimental data and a massive application

$$\gamma_D = 1.20$$

Table 5 – Soil Safety Factor Selection

Application	Available experimental information	γ_D Low soil heterogeneity	Medium soil heterogeneity	High soil heterogeneity
Isolated columns, thin structures	Poor	1.10	1.15	1.25
	Good	1.00	1.05	1.10
Massive treatments	Poor	1.05	1.10	1.20
	Good	1.00	1.00	1.05

Design Value of Column Diameter Calculation (D_d)

$$D_d = \frac{D_K}{\gamma_D} = \frac{1.85}{1.20} = 1.54m \approx 1.50m$$

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Jet Column Spacing		
Designed by: Rebecca Stanzeleit	**Checked by:** Mariam Abdul Ghani	**Verified by:** Nabil Nassor

Jet Column Spacing Calculations

$Know\ parameters$:
$D_d = 1.5m$
$S_e = 0.5m\ (\ from\ past\ projects)$

$$S = \sqrt{D_d^2 - S_e}$$

$Where$:
$S_e = Length\ of\ lens\ created\ by\ two\ overlapping\ colums$
$D_d = Design\ Value\ of\ Column\ Diameter$
$S = Center\ to\ center\ spacing$

$$\therefore S = \sqrt{1.5^2 - 0.5^2} = 1.41\ \approx 1.40m$$

Cylindrical Grouting Curtain Calculations

Minimum distance away from steel liner: $2m$

Minimum radius of grout curtain : $2m + 12m\ =\ 14m$

Perimeter of minimum size of grout curtain: $2\pi r =\ 2\pi(14m) =\ 87.96m$

Number of Jet Columns that can fit into perimeter: $\left(\frac{87.96m}{1.4m/column}\right) = 62.82\ column \approx 64column$

To facilitate the sequencing process, the number of columns is increased to the nearest even number.

Perimeter of a cylinder made up of 64 columns: $64 \times 1.4m = 89.6m$

Diameter of cylinder of 89.6m in perimeter: $\frac{P}{\pi} = \frac{89.6m}{\pi} = 28.52m \approx 28.5\ m$

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Sequencing of Jet Columns		
Designed by: Rebecca Stanzeleit	**Checked by:** Mariam Abdul Ghani	**Verified by:** Nabil Nassor

<u>Jet Column Sequencing Calculations</u>

Know Parameters:

$$P = 89.6m$$
$$S = 1.4m$$
$$C = 64\ columns$$
$$S_P: 6 - 12m/Primary\ column$$

$Where$:

$$P = perimeter\ of\ grout\ curtain$$
$$S = Centre\ to\ centre\ Spacing\ of\ jet\ columns$$
$$C = Number\ of\ Columns\ around\ perimeter$$
$$S_P = Spacing\ between\ primary\ columns$$

$$\frac{S_{Pmax}}{S} = \frac{12}{1.4} = 8.57 \approx 8$$
$$Every\ 8th\ columns\ will\ be\ primary$$
$$S_P = 8 \times 1.4m = 11.2m$$

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Allowable Water Infiltration Through Jet Grouting Curtain		
Designed by: Rebecca Stanzeleit	**Checked by:** Mariam Abdul Ghani	**Verified by:** Nabil Nassor

<u>Allowable Infiltration Rate Calculations</u>

- The permeability coefficient is obtained from the "Practical Guide to Grouting of Underground Structures" (Henn, 1996).
  -Table 3-1 *Typical Permeability Coefficients of Rock and Soil Formations*

- Low discharge, poor drainage
  Use permeability coefficient of 10^{-6} cm/s
  This is equivalent to 0.864 $\frac{L}{m^2 \times day}$

- Surface area of grouted area = $2\pi \times r \times L$
  L= Length of grouted column in the broken anhydrite= 12 m
  r= Radius of the steel shaft =12 m
  Surface Area = $2\pi \times 12 \times 12$
  = 904.8 m^2

- Volume of infiltrated water= 0.8640 $\frac{L}{m^2 \times day}$ ×904.8 m^2
  = 781.7 $\frac{L}{day}$
  =0.7817 $\frac{m^3}{day}$

- Depth of water accumulated in one day = $\frac{Volume\ of\ infiltrated\ water}{Area\ of\ the\ steel\ shaft}$
  =7817 $\frac{m^3}{day}/\pi r^2$
  =7817 $\frac{m^3}{day}/\pi(12)^2$
  =$1.728 \times 10^3 \frac{m}{day}$
  =1.728 $\frac{mm}{day}$

Appendix – B: Steel Liner Design

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Lateral Pressure Calculations		
Designed by: Peter Wang	**Checked by:** Nabil Nassor	**Verified by:** Claire Meloche

Lateral Pressures – Example Calculation

- The lateral pressure calculations are based on Terzaghi's "Theoretical Soil Mechanics." (Terzaghi, 1996).
- Calculations completed for the "dense sand layer" from a depth 36.0m to 45.1m
- Soil Properties:
  - $\emptyset = 29°$
  - $\gamma = 18\ \frac{kN}{m^3}$

- k_a = Coefficient of active pressure
- $\emptyset$ = Friction Angle

Lateral Earth Pressure

Sandy Soils:

$$k_a = \frac{(1 - sin\emptyset)}{(1 + sin\emptyset)}$$

$$= \frac{(1-\sin(29°))}{(1+\sin(29°))}$$

$$= 0.35$$

Lateral Active Pressure (Sandy Soils):

$$P_a = 0.65 k_a \gamma H$$
$$= 0.65\ x\ 0.35\ x\ 18 \frac{kN}{m^3}\ x\ (36.0m)$$
$$= 147.4\ \text{kPa}$$

Hydrostatic Pressure

Hydrostatic Pressure = $(\rho_{water} g \Delta h)$

$= (1000\frac{kg}{m^3})(9.81\frac{m}{s^2})(36.0\text{m} - 6.7\text{m})\left(\frac{1KN}{1000N}\right)$

= 287.4 kPa

Combined Lateral Pressure

Combined Lateral Pressure = Lateral Earth Pressure + Hydrostatic Pressure

= 147.4 kPa + 287.4 kPa

= 434.8 kPa

At bottom of sand layer (45.1m):

Combined Lateral Pressure = Lateral Pressure at Top of Layer + Δ Lateral Earth Pressure + Δ Hydrostatic Pressure

$= 434.8 \text{ kPa} + 0.65\ x\ 0.35\ x\ 18\frac{kN}{m^3}\ x\ (45.1m - 36.0m) + (1000\frac{kg}{m^3})(9.81\frac{m}{s^2})(45.1m - 36.0m)\left(\frac{1KN}{1000N}\right)$

= 561.3 kPa

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Soil and Liner Equations		
Designed by: Nabil Nassor	**Checked by:** Peter Wang	**Verified by:** Claire Meloche

<u>Soil and Liner Equations</u>

(1) Sandy Soils

$k_a = (1 - sin\emptyset)/(1 + sin\emptyset)$

(2) Soft to firm clays

$k_a = 1 - m(4c_u/\gamma H)$

(3)Lateral Active Pressure (Sandy Soils)

$P_a = 0.65k_a\gamma H$

(4) Lateral Active Pressure (Soft to firm clays)

$P_a = 1.0k_a\gamma H$

(5) Hydrostatic Pressure = $(\rho_{water} g\Delta h)$

$= (1000\frac{kg}{m^3})(9.81\frac{m}{s^2})(50\text{m} - 6.7\text{m})\left(\frac{1KN}{1000N}\right)$

= 424.8 kPa

(6) Design Pressure = Lateral Earth Pressure + Hydrostatic Pressure

= 409.5 kPa + 424.8 kPa

= 834.3 kPa → 900 kPa from stress envelope

(7) Case 1: R=12m → $t = (\frac{PR}{1000Y}) = \left(\frac{(900kPa)(12m)}{1000(250Mpa)}\right) = 43.2\text{mm} \approx$ **45mm**

(8) Case 2: R=10m → $t = \left(\frac{(900kPa)(10m)}{1000(250Mpa)}\right) = 36.0\text{mm} \approx 38.0\text{mm}$

(9) Case 3: R=7.5m → $t = \left(\frac{(900kPa)(7.5m)}{1000(250Mpa)}\right) = 27.0\text{mm} \approx 30.0\text{mm}$

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Steel Liner Thickness Calculations		
Designed by: Nabil Nassor	**Checked by:** Peter Wang	**Verified by:** Claire Meloche

Steel Liner Calculations

- The design of the steel liner is based on a reference from the Mining Engineering Handbook authored by J. de la Vergne and L.O. Cooper, with calculations shown in Appendix. The article is titled "Simplified Procedure for the Design of the Full Hydrostatic Steel Mine Shaft Liner (HSL)" (de la Vergne & Cooper, 1984).

Table 1 – Lateral Earth Pressure Calculations

Soil Description	Depth (Feet)	Depth (m)	ø	Ka	γ(KN/m ^3)	Cohesion Factor(kPa)	Lateral Pressure(kpa)
Hard Brown Clayey till with boulders	0	0	0	0	0	0	0
	100	30.48	32	0.55	19	65	319.12
Hard Grey clayey silt and sandy silt	100	30.48	31	0.60	18.50	67	336.76
	118	35.97	31	0.60	18.50	67	397.38
Dense brown and grey sandy till with boulders	118	35.97	29	0.35	18	20	147.40
	148	45.11	29	0.35	18	20	183.13
Broken Anhydrite with gypsum and limestone	148	45.11	36	0.41	20	30	371.90
	165	50.29	36	0.41	20	30	414.62
Soil in broken anhydrite	172	50.29	34.20	0.44	19.50	30	429.46
	190	57.91	34.20	0.44	19.50	30	494.53
Broken Anhydrite with gypsum and limestone	190	57.91	36	0.41	20	30	477.44
	207	63.09	36	0.41	20	30	520.16
Competent anhydrite bedrock	207	63.09	40	0.36	23.20	30	522.88
	236	71.93	40	0.36	23.20	30	596.13

The lateral earth pressures were calculated similarly to the above example calculation. The resulting values are shown in Table 1 of Appendix B. The value of lateral earth pressure needed for the liner design was obtained from interpolation at a 50 meter depth. The obtained value for the maximum lateral earth pressure on the steel liner is 409.5 kPa. The hydrostatic pressure at the same 50 meter mark was calculated similarly to the method shown above; the obtained value is 424.8 kPa. The lateral pressure used for the liner design was obtained by using the stress envelope obtained after plotting the combined lateral and hydrostatic pressures (See Figure 24 in the report).

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Steel Liner Thickness Calculations		
Designed by: Nabil Nassor	**Checked by:** Peter Wang	**Verified by:** Claire Meloche

Liner Thickness Calculations

- For economic and environmental purposes, the liner will be designed with 3 varying thicknesses. The liner thickness transition depths will be 20m and 40m. Using the stress envelope, the design pressures at these depths are 380 kPa, and 746 kPa, respectively. The design pressure at the maximum depth, using the same approach, is 900 kPa.

- $0 - 20\text{m} : t = \left(\frac{PR}{1000Y}\right) = \left(\frac{(380\text{kPa})(12\text{m})}{1000(250\text{Mpa})}\right) = 18.2\text{mm} \approx \mathbf{20\ mm}$
- $20\text{m} - 40\text{m} : t = \left(\frac{PR}{1000Y}\right) = \left(\frac{(746\text{kPa})(12\text{m})}{1000(250\text{Mpa})}\right) = 35.8\text{mm} \approx \mathbf{37\ mm}$
- $40\text{m} - 50\text{m} : t = \left(\frac{PR}{1000Y}\right) = \left(\frac{(900\text{kPa})(12\text{m})}{1000(250\text{Mpa})}\right) = 43.2\text{mm} \approx \mathbf{45\ mm}$

Stiffening Rings Calculations

- There exists a transition depth where for depths shallower than the transition depth, the thickness of the liner shell is governed by buckling resistance. For depths greater than the transition depth, the thickness of the steel liner is governed by compressive resistance.

$$\text{Theoretical Transition Depth} = \frac{\text{Transition Pressure}}{\text{Average Soil Density} + \text{Water Density}}$$

$$= \frac{19764\ \text{kPa}}{19.8\ \frac{\text{kN}}{\text{m},^3} + 9.81\ \frac{\text{kN}}{\text{m},^3}}$$

$$= 667\text{m}$$

- This depth of 667m is much greater than the depth of the steel liner (50m), thus stiffening rings will be required throughout the liner.

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Steel Stiffening Rings Calculations		
Designed by: Nabil Nassor	**Checked by:** Peter Wang	**Verified by:** Rebecca Stanzeleit

Table 2 – Steel Liner Design Variables

Symbol	Dimension	Definition
t	mm	Shell thickness
d	m	Depth
P_h	kPa	Hydrostatic pressure
P	kPa	Design pressure
R	mm	Inside shaft radius
R_o	mm	Outside shaft radius
Y	MPa	Yield stress
P_t	kPa	Transition pressure (Pressure at transition depth)
c	Mm	Length of shell portion contributing to buckling resistance
I	mm^4	Combined moment of inertia of ring and shell portion
b	mm	Width of stiffening ring
h	mm	Height of stiffening ring
L	mm	Ring spacing
E	MPa	Young's modulus of elasticity
$A_1 = c \times t$	mm^2	Effective area of shell acting with stiffening ring
$A_2 = b \times h$	mm^2	Cross sectional area of stiffening ring
B_1	MPa	Theoretical elastic buckling area
$B_2 = k_1 \times B_1$	MPa	Elastic buckling stress
$B_3 = k_1 \times k_2 \times B_1$	MPa	Inelastic buckling stress
k_1	Unit less	Reduction factor for imperfections and non-linearity
k_2	Unit less	Reduction factor for plasticity
P_1	kPa	Allowable pressure on shell

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Steel Stiffening Rings Calculations		
Designed by: Nabil Nassor	**Checked by:** Peter Wang	**Verified by:** Rebecca Stanzeleit

- The size of the stiffening rings required at each liner section was determined through an iterative process. The height of the ring (the dimension in the radial direction) was kept constant at 400mm (obtained through trial and error) and the width of the ring (the dimension in the axial direction) was increased by 20mm. Once an acceptable allowable pressure ($P_1 > P$) was obtained while satisfying all other checks, the corresponding dimensions were chosen.

<u>Example Calculation</u>

- Example calculation completed for the 0-20m portion of the shell liner. In this region, P = 380 kPa. Assumed stiffening ring height of 400mm and stiffening width of 180mm.
- <u>Note:</u> Liner thickness used from earlier calculations (See Appendix B).

$$c = 1.56[t(R + t)]^{\frac{1}{2}}$$

$$= 1.56 * [(20\text{mm}) * (12000\text{mm} + 20\text{mm})]^{\frac{1}{2}}$$

$$= 764.9 \text{ mm}$$

$$A_1 = c \times t$$

$$= 764.9\text{mm} \times 20\text{mm}$$

$$= 15297.5 \text{ mm}^2$$

$$A_2 = b \times h$$

$$= 180\text{mm} \times 400\text{mm}$$

$$= 72000\text{mm}^2$$

$$I = \frac{A_2h^2 + A_1t^2}{3} - \frac{(A_2h - A_1t)^2}{4(A_1 + A_2)}$$

$$= \frac{(72000\text{mm}^2)(400\text{mm})^2 + (15297.5\text{mm}^2)(20mm)^2}{3} - \frac{\left((7200\text{mm}^2)(400\text{mm}) - (15297.5\text{mm}^2)(20\text{mmm})\right)^2}{4(15297.5\text{mm}^2 + 72000\text{mm}^2)}$$

$$= 1.52\text{x}10^9 \text{ mm}^4$$

- Stiffening ring spacing of 1000mm is chosen based on empirical findings and article example

$$B_1 = \frac{3EI}{LR^2t} x \frac{A_1}{(A_1 + A_2)}$$

$$= \frac{3(200000MPa)(1.52x10^9 mm^4)}{(1000mm)(12000mm)^2(20mm)} x \frac{(15297.5mm^2)}{(15297.5mm^2 + 72000mm^2)}$$

$$= 55.38 \text{ MPa}$$

$B_2 = k_1 \text{ x } B_1$
$= 0.8 \text{ x } 55.38 \text{ MPa}$
$= 44.30 \text{ MPa}$

$$Z = \frac{K_1 B_1}{Y}$$

$$= \frac{0.8 \text{ x } 55.38 \text{ MPa}}{275 \text{ MPa}}$$

$$= 0.16$$

- Since Z < 0.55, K_2 will equal 1.0

$K_2 = 1.0$

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Steel Stiffening Rings Calculations		
Designed by: Nabil Nassor	**Checked by:** Peter Wang	**Verified by:** Rebecca Stanzeleit

Allowable pressure:

$$P_1 = k_1 k_2 \frac{3EI}{LR^3}$$

$$= (0.8)(1.0)\frac{3 * (200000\text{MPa})(1.52\text{x}10^9\text{mm}^4)}{(1000\text{mm})(12000\text{mm})^3}$$

$$= 422.2 \text{ kPa}$$

$P_1 > P = 380$ kPa ∴OK

Stiffness Check:

$$\frac{bh^3}{Lt^3} \geq 1$$

$$\frac{(180mm)(400mm)^3}{(1000mm)(20mm)^3} \geq 1$$

$1440 \geq 1$ ∴OK

- Since $P_1 > P$ and the stiffening check is satisfied, the chosen stiffening ring dimensions are sufficient to provide resistance to buckling. Therefore, for the 0-20m section of the steel liner, the stiffening ring thickness will be 400mm x 180mm.
- The design of the stiffening rings through iterative tables for the 0-20m and other two sections is shown in the tables on the subsequent pages. The highlighted columns indicate the appropriate stiffening ring dimensions for the respective liner section. The highlighted row represents the parameter required to assess the stiffening ring's performance (the allowable pressure).

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Table 3 – Segment 1 Stiffening Ring Dimensions

0-20m										
h= height of the ring	(mm)	400	400	400	400	400	400	400	400	400
b= width of the ring	(mm)	80	100	120	140	160	180	200	220	240
c=1.56[t(R+t)]^(1/2)	(mm)	765	765	765	765	765	765	765	765	765
A1	(mm)^2	15298	15298	15298	15298	15298	15298	15298	15298	15298
A2	(mm)^2	32000	40000	48000	56000	64000	72000	80000	88000	96000
I (Combined Moment of Inertia)	(mm)^4	883.60E+6	1.02E+9	1.15E+9	1.28E+9	1.40E+9	1.52E+9	1.63E+9	1.75E+9	1.86E+9
L - Constant	(mm)	1000	1000	1000	1000	1000	1000	1000	1000	1000
L - Old	(mm)	272.72	315.38	355.58	394.15	431.58	468.18	504.17	539.68	574.82
L - Effective	(mm)	920	900	880	860	840	820	800	780	760
B1 (Theoretical Elastic Buckling Strength)	MPa	59.54	58.89	58.01	57.08	56.20	55.38	54.63	53.95	53.33
K1 (Geometry Reduction Factor)		0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B2 (Elastic Buckling Strength)	MPa	47.63	47.11	46.41	45.67	44.96	44.30	43.70	43.16	42.66
Z		0.17	0.17	0.17	0.17	0.16	0.16	0.16	0.16	0.16
K2 (If Z<0.55 then K2=1.0)		1	1	1	1	1	1	1	1	1
B3	MPa	47.63	47.11	46.41	45.67	44.96	44.30	43.70	43.16	42.66
P1	KPa	245.45	283.84	320.03	354.74	388.42	421.37	453.75	485.71	517.34
bh^3/(Lt^3)		640.00	800.00	960.00	1120.00	1280.00	1440.00	1600.00	1760.00	1920.00

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Table 4 – Segment 2 Stiffening Ring Dimensions

20-40m										
h= height of the ring	(mm)	400	400	400	400	400	400	400	400	400
b= width of the ring	(mm)	160	180	200	220	240	260	280	300	320
c=1.56[t(R+t)]^(1/2)	(mm)	1041	1041	1041	1041	1041	1041	1041	1041	1041
A1	(mm)^2	38520	38520	38520	38520	38520	38520	38520	38520	38520
A2	(mm)^2	64000	72000	80000	88000	96000	104000	112000	120000	128000
I (Combined Moment of Inertia)	(mm)^4	2.01E+9	2.16E+9	2.31E+9	2.46E+9	2.60E+9	2.73E+9	2.87E+9	3.00E+9	3.12E+9
L - Constant	(mm)	1000	1000	1000	1000	1000	1000	1000	1000	1000
L - Old	(mm)	619.07	667.43	713.70	758.29	801.49	843.53	884.61	924.86	964.41
L - Effective	(mm)	840	820	800	780	760	740	720	700	680
B1 (Theoretical Elastic Buckling Strength)	MPa	84.87	84.88	84.63	84.24	83.74	83.18	82.60	82.00	81.40
K1 (Geometry Reduction Factor)		0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B2 (Elastic Buckling Strength)	MPa	67.89	67.90	67.71	67.39	66.99	66.55	66.08	65.60	65.12
Z		0.25	0.25	0.25	0.25	0.24	0.24	0.24	0.24	0.24
K2 (If Z<0.55 then K2=1.0)		1	1	1	1	1	1	1	1	1
B3	MPa	67.89	67.90	67.71	67.39	66.99	66.55	66.08	65.60	65.12
P1	KPa	557.16	600.68	642.33	682.46	721.34	759.18	796.15	832.37	867.97
bh^3/(Lt^3)		202.16	227.43	252.70	277.97	303.24	328.51	353.78	379.05	404.32

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Project Detail: Steel Stiffening Rings Calculations		
Designed by: Peter Wang	**Checked by:** Nabil Nassor	**Verified by:** Rebecca Stanzeleit

Table 5 – Segment 3 Stiffening Ring Dimensions

40-50m										
h= height of the ring	(mm)	400	400	400	400	400	400	400	400	400
b= width of the ring	(mm)	180	200	220	240	260	280	300	320	340
c=1.56[t(R+t)]^(1/2)	(mm)	1000	1000	1000	1000	1000	1000	1000	1000	1000
A1	(mm)^2	45000	45000	45000	45000	45000	45000	45000	45000	45000
A2	(mm)^2	72000	80000	88000	96000	104000	112000	120000	128000	136000
I (Combined Moment of Inertia)	(mm)^4	2.34E+9	2.50E+9	2.65E+9	2.80E+9	2.95E+9	3.09E+9	3.23E+9	3.36E+9	3.49E+9
L - Constant	(mm)	1000	1000	1000	1000	1000	1000	1000	1000	1000
L - Old	(mm)	721.77	771.62	819.43	865.55	910.25	953.76	996.23	1037.83	1078.65
L - Effective	(mm)	820	800	780	760	740	720	700	680	660
B1 (Theoretical Elastic Buckling Strength)	MPa	83.28	83.33	83.18	82.87	82.47	82.01	81.51	80.99	80.45
K1 (Geometry Reduction Factor)		0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
B2 (Elastic Buckling Strength)	MPa	66.62	66.67	66.54	66.30	65.98	65.61	65.21	64.79	64.36
Z		0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.24	0.23
K2 (If Z<0.55 then K2=1.0)		1	1	1	1	1	1	1	1	1
B3	MPa	66.62	66.67	66.54	66.30	65.98	65.61	65.21	64.79	64.36
P1	KPa	649.59	694.46	737.49	778.99	819.23	858.38	896.61	934.04	970.79
bh^3/(Lt^3)		126.42	140.47	154.51	168.56	182.61	196.65	210.70	224.75	238.79

Appendix – C: Costs and Scheduling

	Geovation Engineering™	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Cost Estimate		
Designed by: Claire Meloche	**Checked by:** Peter Wang	**Verified by:** Nabil Nassor

Table 1 – Cost Estimate of Proposed Project

Cost Estimate - *Oak Island*

Item #	Description	Unit Rate (C$)	Quantity	Pre-Contingency TOTAL (C$)	TOTAL	% Contingency
	PRELIMINARY INVESTIGATIONS					
	SITE SURVEY					
	Personnel:					
1	Surveyor #1	400/day	5 days	2000		
2	Surveyor #2	400/day	5 days	2000		
	Equipment:					
	Materials:					
3	Drawing	1000/drawing	1 drawing	1000		
	Subtotal, Site Survey			5000	5500	10%
	BOREHOLES					
4	Mob & Demob	3000	1	3000		
5	Type 1 Hole (6)	45/foot	960 feet	43000		
6	Type 2 Hole (2)	55/foot	640 feet	35000		
7	Core Drilling and Sampling	200/hour	250 hours	50000		
8	Casing, pumps, etc.	150/hour	1	15000		
9	Standby for Testing	150/hour	100 hour	15000		

	Personnel:					
10	Field Man #1	300/day	50 days	15000		
11	Field Man #2	300/day	50 days	15000		
12	Field Engineer	700/day	50 days	35000		
13	Project Manager	160/day	50 days	8000		
	Equipment:					
	Materials:					
14	Factual Report	8000/report	1	8000		
	Subtotal, Boreholes			242000	278300	15%
	SITE PREPARATION					
	SITE ROADWORK					
15	Bypass Road	25000	1	25000		
16	Upgrade Existing Roads	20000	1	20000		
17	Construction Road Maintenance	10000	10 months	10000		
18	Culverts & Ditches	5000	1	5000		
19	Site Grading	1000	1	1000		
	Personnel:					
20	Superintendent	1000/day	10 days	10000		
21	Foreman	400/day	10 days	4000		
22	Specialized Labour	350/day	10 days	3500		
23	Specialized Labour	350/day	10 days	3500		
	Equipment:					
24	Dozer, D7	1750/day	10 days	17500		
25	Loader, 6 cy	833/day	10 days	8330		
26	Gravel Truck, Tandem Axle	450/day	10 days	4500		
	Materials:					
	Subtotal, Site Roadwork			112330	129180	15%
	SITE POWER					
27	Nova Scotia Power Installation	15000	1	15000		
28	Substation Feeder Installation	30000	1	30000		

29	Supply Power	30000/month	10 months	300000		
	Personnel:					
	Equipment:					
	Materials:					
	Subtotal, Site Power			345000	379500	10%
	TRAILER INSTALLATION					
	Personnel:					
30	Crane/Backhoe Operator	360/day	7	2520		
31	Loader/Bull Operator	340/day	7	2380		
32	Carpenter	330/day	10	3300		
33	Surface Labour	375/day	10	3750		
34	Electrician	170/day	20	3400		
	Equipment:					
35	First Aid Treatment Trailer	400/month	10 months	4000		
36	Dry House Trailer	600/month	10 months	6000		
37	Engineering Office Trailer	400/month	10 months	4000		
38	Archaeologist Trailer	400/month	10 months	4000		
39	Crane, 15 t	20000/month	10 months	200000		
40	Loader, 6 cy	25000/month	10 months	250000		
	Materials:					
41	Trailer Furniture	1000/month	9	9000		
42	Business Machines	500/month	9	4500		
43	Courier Services	200/month	9	1800		
44	Telephones	1500/month	9	13500		
45	Stationary	250/month	9	2250		
	Subtotal, Trailer Installation			64400	74060	15%
	FENCE INSTALLATION					

46	Fence & Gate at Causeway	100/meter	30 meters	3000		
47	Fence around Site Shaft	75/meter	370 meters	27750		
	Personnel:					
48	Crane/Backhoe Operator	360/day	7	2520		
49	Loader/Bull Operator	340/day	7	2380		
50	Carpenter	330/day	10	3300		
51	Electrician	375/day	10	3750		
52	Surface Labour	170/day	20	3400		
	Equipment:					
53	Crane, 15 t	666/day	7	4662		
54	Loader, 6 cy	833/day	7	5833		
	Materials:					
55	Stone Surface for Site	12/tonne	4500 tonnes	54000		
56	Stone Surface for Laydown	12/tonne	2700 tonnes	32400		
	Subtotal, Fence Installation			142995	157295	10%
	SECURITY					
57	Guard/Search House					
	Personnel:					
58	Gate House Guard					
	Equipment:					
	Materials:					
59	Punch Clocks	600	2	1200		
60	Closed Circuit TV	15000	1	15000		
61	Infrared Grid Line	20000	1	20000		
62	Search Wands	1500	1	1500		
63	Time Clocks	2000	1	2000		
64	Warning Signs	4000	1	4000		
65	Monitoring Stations	10000	1	10000		
66	Sirens & Emergency Lights	2500	1	2500		
67	Radios & Chargers	15000	1	15000		
68	Radio License	500	1	500		

69	Radar Monitor	15000	1	15000		
	Subtotal, Security			86700	95370	10%
	GROUND IMPROVEMENT					
	JET GROUTING (Specialized Subcontractor)					
70	Food & Lodging	150/day	15 days	2250		
	Personnel:					
71	Engineer	1500/day	12 days	18000		
72	Superintendent	1000/day	12 days	12000		
73	Specialized Labour	750/day	12 days	9000		
74	Specialized Labour	750/day	12 days	9000		
75	Specialized Labour	750/day	12 days	9000		
76	Specialized Labour	750/day	12 days	9000		
77	Local Labour	500/day	12 days	6000		
78	Local Labour	500/day	12 days	6000		
79	Local Labour	500/day	12 days	6000		
	Equipment:					
80	Cement Grout Plant	3000/day	12 days	36000		
81	Real-Time Monitoring System	1200/day	12 days	14400		
82	Drilling Rates	75/foot	13440 feet	1008000		
	Materials:					
83	Cement Based Grout	400/cubic meter	1357 cubic meters	542867		
	Subtotal, Jet Grouting			1687517	2362524	40%
	PERMEATION GROUTING (Specialized Subcontractor)					
84	Food & Lodging	150/day	15 days	2250		
	Personnel:					
85	Engineer	1500/day	10 days	15000		
86	Superintendent	1000/day	10 days	10000		
87	Specialized Labour	750/day	10 days	7500		
88	Specialized Labour	750/day	10 days	7500		
89	Specialized Labour	750/day	10 days	7500		
90	Specialized Labour	750/day	10 days	7500		
91	Local Labour	500/day	10 days	5000		
92	Local Labour	500/day	10 days	5000		

93	Local Labour	500/day	10 days	5000		
	Equipment:					
94	Cement Grout Plant	3000/day	10 days	30000		
95	Real-Time Monitoring System	1200/day	10 days	12000		
96	Drilling Rates	75/foot	22680 feet	1701000		
	Materials:					
97	Cement Based Grout	400/cubic meter	212 cubic meters	84800		
	Subtotal, Permeation Grouting			1900050	2660070	40%
	STAGE #1					
	EXCAVATION					
	Personnel:					
98	Superintendent	1000/day	9 days	9000		
99	Foreman	400/day	9 days	3600		
100	Specialized Labour	350/day	9 days	3150		
101	Specialized Labour	350/day	9 days	3150		
102	Specialized Labour	350/day	9 days	3150		
103	Specialized Labour	350/day	9 days	3150		
104	Excavator Operator	340/day	9 days	3060		
105	Lower Deck Men	300/day	9 days	2700		
106	Upper Deck Men	280/day	9 days	2520		
107	Crane Operator	360/day	9 days	3240		
108	Loader/Bull Operator	320/day	9 days	2880		
109	Surveyor	280/day	9 days	2520		
	Equipment:					
110	Crane	1285/day	9 days	11565		
111	Gravel Truck, Tandem Axle	450/day	9 days	4050		
112	Loader, 6cy	833/day	9 days	7497		
113	320 Excavator	885/day	9 days	7965		
	Materials:					
	Subtotal, Excavation			73197	102476	40%
	LINER INSTALLATION					
	Personnel:					

114	Superintendent	1000/day	10 days	10000		
115	Foreman	400/day	10 days	4000		
116	Specialized Labour	350/day	10 days	3500		
117	Specialized Labour	350/day	10 days	3500		
118	Specialized Labour	350/day	10 days	3500		
119	Specialized Labour	350/day	10 days	3500		
120	Crane Operator	360/day	10 days	3600		
121	Bull Operator	320/day	10 days	3200		
	Equipment:					
122	Crane	1285/day	10 days	12850		
123	Loader, 6cy	833/day	10 days	8330		
	Materials:					
124	836 Steel Shell			43347		
125	836 Steel Rings			183717		
	Subtotal, Liner Installation			283044	396262	40%
	DEWATERING & INSTRUMENTATION					
	WELL INSTALLATION & DEWATERING					
	Personnel:					
126	Drill Operator	375/day	20 days	7500		
127	Casing Welder	350/day	20 days	7000		
128	Specialized Labour	300/day	20 days	6000		
	Equipment:					
129	Deepwell Electric Pump	25000/unit	4 units	100000		
130	Trash Pump	4000/unit	3 units	12000		
131	Deepwell Discharge Line	50/meter	550 meters	27500		
132	Deepwell Pump Screens	3500/unit	6 units	21000		
133	Deepwell Casings	100/meter	300 meters	30000		
134	Deepwell Choke Valves	250/unit	4 units	1000		
135	Trash Pump Discharge Line	15/meter	300 meters	4500		
	Materials:					
	Subtotal, Well Installation & Dewatering			216500	259800	20%
	PIEZOMETER INSTALLATION					

	Personnel:					
136	Drill Operator	350/day	20 days	7000		
137	Technician	350/day	20 days	7000		
138	Specialized Labourer	300/day	20 days	6000		
	Equipment:					
139	Piezometers	600/unit	4	2400		
	Materials:					
	Subtotal, Piezometer Installation			22400	26880	20%
	STAGE #2					
	EXCAVATION					
	Personnel:					
140	Superintendent	1000/day	100 days	100000		
141	Foreman	400/day	100 days	40000		
142	Specialized Labour	350/day	100 days	35000		
143	Specialized Labour	350/day	100 days	35000		
144	Specialized Labour	350/day	100 days	35000		
145	Specialized Labour	350/day	100 days	35000		
146	Excavator Operator	340/day	100 days	34000		
147	Lower Deck Men	300/day	100 days	30000		
148	Upper Deck Men	280/day	100 days	28000		
149	Crane Operator	360/day	100 days	36000		
150	Loader/Bull Operator	320/day	100 days	32000		
151	Surveyor	280/day	100 days	28000		
	Equipment:					
152	Crane	1285/day	100 days	128500		
153	Gravel Truck, Tandem Axle	450/day	100 days	45000		
154	Loader, 6cy	833/day	100 days	83300		
155	320 Excavator	885/day	100 days	88500		
	Materials:					
	Subtotal, Excavation			813300	1138620	40%
	LINER INSTALLATION					
	Personnel:					
156	Superintendent	1000/day	100 days	100000		
157	Foreman	400/day	100 days	40000		

158	Specialized Labour	350/day	100 days	35000		
159	Specialized Labour	350/day	100 days	35000		
160	Specialized Labour	350/day	100 days	35000		
161	Specialized Labour	350/day	100 days	35000		
162	Crane Operator	360/day	100 days	36000		
163	Bull Operator	320/day	100 days	32000		
	Equipment:					
164	Crane	1285/day	100 days	128500		
165	Loader, 6cy	833/day	100 days	83300		
	Materials:					
166	836 Steel Shell			470584		
167	836 Steel Rings			1684728		
	Subtotal, Liner Installation			2715112	3801157	40%
	PROBE HOLES					
	Personnel:					
	Equipment:					
	Materials:					
	Subtotal, Probe Holes					
	ARCHAEOLOGICAL INVESTIGATIONS					
	ON-SITE OBSERVATION					
	Personnel:					
168	Archeologist	300/day	314 days	94200		
	Equipment:					
	Materials:					
	Subtotal, On-Site Observation			94200	117750	25%
	EXCAVATE ORIGINAL WORKINGS					
	Personnel:					
169	Loader/Bull Operator	320/day	50 days	16000		
170	Labourer/Student	150/day	50 days	7500		
171	Labourer/Student	150/day	50 days	7500		

	Equipment:					
	Materials:					
	Subtotal, Excavate Original Workings			31000	38750	25%
	STABILIZE ORIGINAL WORKINGS					
	Personnel:					
172	Loader/Bull Operator	320/day	30 days	9600		
173	Labourer/Student	150/day	30 days	4500		
174	Labourer/Student	150/day	30 days	4500		
	Equipment:					
	Materials:					
	Subtotal, Stabilize Original Workings			18600	23250	25%
	ARCHAEOLOGICAL REINSTATEMENT					
	Personnel:					
175	Loader/Bull Operator	320/day	30 days	9600		
176	Labourer/Student	150/day	30 days	4500		
177	Labourer/Student	150/day	30 days	4500		
	Equipment:					
	Materials:					
	Subtotal, Archaeological Reinstatement			18600	23250	25%
	MAINTENANCE					
	MAINTENANCE					
	Personnel:					
178	Mechanic	360/day	165 days	59400		
179	Carpenter	340/day	40 days	13600		
180	Electrician	375/day	30 days	11250		
	Equipment:					
	Materials:					
	Subtotal, Maintenance			84250	101100	20%

	INDIRECT EXPENSES					
	INDIRECT & COMPANY INDIRECT EXPENSES					
181	PL & PD Insurance	40000/unit	1	40000		
182	Vehicle Insurance	1000/month	10 months	10000		
183	Licenses	100/unit	1	100		
184	Contract Bonds	25000/unit	1	25000		
185	Financing Costs	25000/unit	1	25000		
186	Head Office Overheads	150000/unit	1	150000		
187	Miss. Freight	2000/month	9 months	18000		
188	Accommodations & Meals	2000/month	9 months	18000		
189	Travel	2500/month	9 months	22500		
190	Legal Fees	800/month	9 months	7200		
191	Safety PPE, Rain Gear	9000/unit	1	9000		
192	Shaft Phones, Signals	500/month	9 months	4500		
193	Misc. Building Materials	750/month	9 months	6750		
194	Dry House Heat / Supplies	500/month	9 months	4500		
195	Garbage Disposal	200/month	9 months	1800		
196	Portable Toilet, Serviced	200/month	9 months	1800		
197	Misc. Fuel & Lubricants	500/month	9 months	4500		
	Subtotal, Indirect Expenses			348650	400948	15%
	PROJECT MANAGEMENT					
	PROJECT MANAGEMENT					
198	Review Avail Borehole Data	1000/day	4 days	4000		
199	Review Available Text	800/day	3 days	2400		
200	Study Shaft Methods	1500/day	2 days	3000		
201	Select Viable Shaft Method	1500/day	4 days	6000		
202	Prepare Prelim. Design	1500/day	4 days	6000		
203	Analyze Shaft Sinking Costs	1500/day	10 days	15000		
204	Prepare Prelim Report	800/day	4 days	3200		
205	Modify Prelim Report	1200/day	5 days	6000		

206	Establish Site Elevations	2200/day	2 days	4400		
207	BH/Shaft Locations	2200/day	2 days	4400		
208	Review Additional Data	1000/day	4 days	4000		
209	Review Labor Agreements	800/day	3 days	2400		
210	Establish Government Auth.	1600/day	4 days	6400		
211	Procure More Accurate Costs	1200/day	6 days	7200		
212	Obtain Local Quotations	2200/day	2 days	4400		
213	Final Shaft Concept. Design	1200/day	4 days	4800		
214	Finalize Location Drawings	2500/day	5 days	12500		
215	Final Shaft Drawings	1400/day	4 days	5600		
216	Solicit Contractor Input	1500/day	4 days	6000		
217	Negotiate Site Labor Agt.	1400/day	5 days	7000		
218	Obtain Safety Approvals	1350/day	6 days	8100		
219	Select Final Shaft Alternative	1400/day	3 days	4200		
220	Final Shaft Design & Spec.	1000/day	6 days	6000		
221	Prequalify Contractors	1700/day	2 days	3400		
222	Prepare Tender Documents	700/day	8 days	5600		
223	Call Tenders	10000/day	10 days	100000		
224	Negotiate Material prices	1500/day	5 days	7500		
225	Procure All Materials	800/day	10 days	8000		
226	Receive Tenders	600/day	1 days	600		
227	Review Tenders / Recommend	1400/day	5 days	7000		
228	Final Construction Specs.	1600/day	10 days	16000		
229	Final Contractor Negotiation	1400/day	2 days	2800		
230	Award Contract	2200/day	4 days	8800		
231	Project Manager on Site	800/day	200 days	160000		
232	Engineer on Site	500/day	110 days	55000		
233	Inspectors on Site	600/day	200 days	120000		
234	Travel In & Out	1200/trip	24 trips	28800		

235	Travel Room & Board	150/day	310 days	46500		
	Subtotal, Project Management			703000	808450	15%
				TOTAL	**$13,380,490**	

	Geovation Engineering	Mariam Abdul Ghani Claire Meloche Nabil Nassor Rebecca Stanzeleit Peter Wang
Project Title: Feasibility Study of a Deep Excavation to Preserve the Archaeological Heritage of Oak Island, Nova Scotia		
Project Detail: Construction Schedule		
Designed by: Claire Meloche	**Checked by:** Peter Wang	**Verified by:** Nabil Nassor

Construction Schedule

Oak Island, Nova Scotia

Period Highlight: 1

Plan | Actual | % Complete | Actual (beyond plan) | % Complete (beyond plan)

ACTIVITY	PLAN START	PLAN END	PLAN DURATION	ACTUAL START	ACTUAL DURATION	PERCENT COMPLETE
Preliminary Investigations						
Site Survey	1	6	5	0	0	0%
Boreholes	16	76	60	0	0	0%
Site Preparation			0			
Site Roadwork	6	16	10	0	0	0%
Site Power	16	26	10	0	0	0%
Trailer Installation	16	21	5	0	0	0%
Fence Installation	18	27	9	0	0	0%
Security	21	31	10	0	0	0%
Ground Improvement			0			
Jet Grouting	77	87	10	0	0	0%
Permeation Grouting	88	96	8	0	0	0%
Stage #1			0			
Excavation	97	106	9	0	0	0%
Liner Installation	99	115	16	0	0	0%
Dewatering & Instrumentation			0			
Well Installation	107	110	3	0	0	0%
Piezometer Installation	107	110	3	0	0	0%
Dewatering	107	345	238	0	0	0%
Stage #2			0			
Excavation	110	210	100	0	0	0%
Liner Installation	112	215	103	0	0	0%
Probe Holes	110	210	100	0	0	0%
Archaeological Investigation			0			
On-Site Observation	6	345	339	0	0	0%
Excavate Original Workings	215	245	30	0	0	0%
Stabilize Original Workings	246	315	69	0	0	0%
Archaeological Reinstatement	315	345	30	0	0	0%
Archaeological Investigation			0			
Stairs Installation	215	245	30	0	0	0%
Elevator Installation	215	250	35	0	0	0%
Reinstatement of Original Money Pit Platforms	215	250	35	0	0	0%

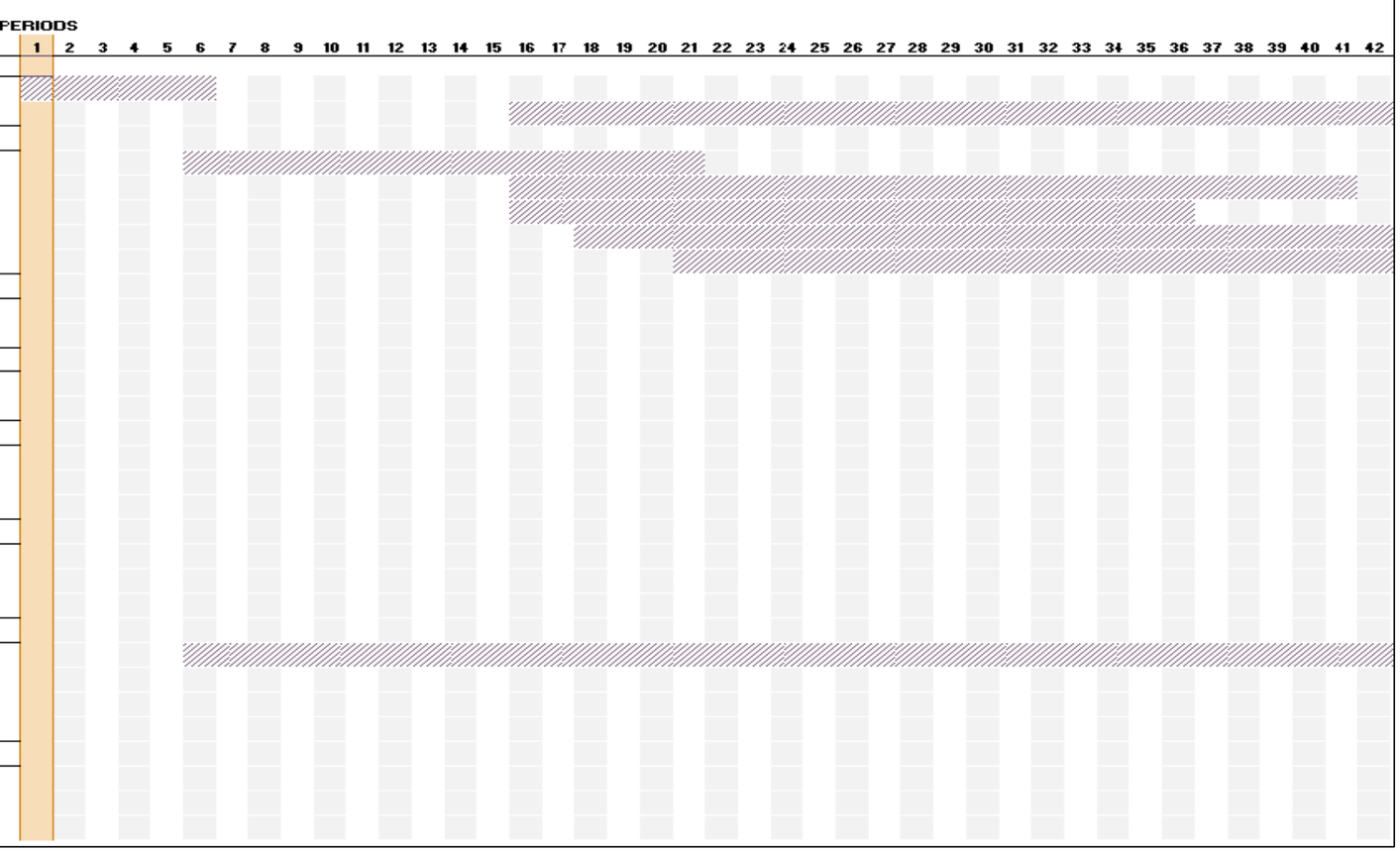

Figure 1 – Concstruction Schedule Gantt Chart

Note: Due to the large size of the original document, only 42 days of the construction schedule is shown in the Appendix. For the complete Gantt Chart see the Excel file also included.

Archaeological Discoveries Deep in the Money Pit

By John Wonnacott and Les MacPhie, May 2016

Part 1 - **The 1967 Becker Program**

"Who or what the heck is a Becker?", you are probably asking. Well it's the name of a drilling company and also a special type of drilling device named after the man who invented the equipment and started the company. In 1967 David Tobias and Dan Blankenship hired the Becker company to investigate the Money Pit area and with 40 drill holes they discovered compelling evidence of man-made workings far deeper than any Searcher had ever explored. What the Becker program found is the most intriguing set of archeological discoveries ever made at Oak Island.

Part 1 of this article will explain the Becker drilling program in the area of the Money Pit, and will present the factual details of what was found. Part 2, which will be published later, will present the interpretation of these findings. The reader will soon see that drill holes do not usually go in a straight line, and for that reason, understanding the configuration of underground objects or constructions encountered in the Becker holes is not nearly as easy as one would first think.

Before anyone can properly assess and interpret those archeological findings, it is important to understand the principal advantage and a critical weakness of the Becker drill. The way the drill works, a pile-driving hammer pounds the drill pipe into the ground, with a drill bit attached to the bottom of the drill pipe. The drill pipe is actually double walled pipe and compressed air is forced down the outer annulus. The compressed air passes by an opening in the drill bit, and drill cuttings are blown back to the surface, travelling inside the inner pipe. The compressed air travels at very high speed, so drill cuttings reach the surface before the drill bit has advanced more than a few inches. This is a tremendous advantage when drilling in search of artifacts, because the driller can know precisely at what depth any sample came from. An illustration to show how the Becker drill works is given on Figure 2 at the end of this document and a photograph of the Becker drill in operation at Oak Island is shown on Figure 2.

Tobias and Blankenship knew all about the advantages of the Becker drill but they did not appreciate a problem that eventually plagued the work and confounded efforts to interpret the discoveries they made. No drill rig can make a perfectly straight hole. That's because no soil or rock is perfectly homogeneous, so when one side of a drill bit face hits soil or rock of a greater hardness, the path of the drill bit wanders off vertical. When drilling in soil that has hard boulders, the drill bit is often deflected from its intended path, and a significant amount of "lateral drift" or deviation occurs. Around the Money Pit, the upper soil contains a lot of hard boulders, and despite the fact that the Becker drill string used double-wall pipe, which is pretty stiff and you'd think resistant to deflection, the drill holes deviated from their planned route. We're not talking about a few inches of deviation here either – in a different drilling program some years later, where borehole deviation was measured, one hole drilled to a similar depth as the Becker holes deviated by 17 feet!

The problem with drill-hole deviation is that the driller does not know how much any particular hole deviates, and he/she does not know in which direction the drill bit has wandered off course. Based

on our best estimate, a lateral drift of about 10 feet at 200 feet depth is considered to represent the upper range of values for most of the Becker holes. However, the lateral drift could readily exceed 10 feet for some holes. So if the maximum deviation is 10 feet, then the drill bit can end up anywhere within a 20 foot diameter circle at a depth of 200 feet. Surprisingly though, even if a 200 foot deep hole deviates by as much as 10 feet, the true depth that the bit achieves will be almost 200 feet (if you do the trigonometry, you'll see such a hole reaches a true depth of about 199.7 feet!). A schematic illustration showing the possible range of locations at a depth of 200 feet for holes with a lateral drift of 10 feet or less is given on Figure 3. Also shown on this figure is a simplified geological profile of subsurface conditions in the area of the Money Pit.

Of course in 1967 there were down-hole tools designed to measure the orientation and amount of borehole deviation, but Tobias and Blankenship did not have that equipment on site during their Becker program. They did not realize at the time that they would need it. Since the Becker drilling technique does not routinely leave any drill casing in a hole after the drilling is completed, when the drill string is extracted, the hole soon collapses and it becomes impossible to measure deviation afterwards.

The Becker program of 1967 produced 40 boreholes in the area of the Money Pit. We know quite accurately where the holes were started or "collared", we know how deep they were drilled, we know whether the holes were started in a vertical or inclined orientation and we know what archeological artifacts were found. But we don't really know, in a lateral sense, where the samples were collected from. If we assume that every hole was drilled in a straight line, we can make some "interpretation" of the findings, but we must keep in mind that probably none of the Becker holes ended up where they were intended to go. Possible combinations and permutations of drilling results will be discussed in Part 2 of this article.

Tobias and Blankenship were hoping to find treasure chests that were assumed to be sitting at a depth of 100 feet based on Searchers' work in 1849 or close to bedrock based on Searchers' work in 1897. Bedrock was expected at about 150 to 160 feet and in 1967 everyone thought the Money Pit ended at bedrock. "So what did the Becker drilling actually discover?" you ask. Well at first, nothing remarkable. Dan Blankenship was supervising the work in the field, and the first borehole (B1) was placed about 15 feet west of the Hedden Shaft. The hole hit bedrock at 145 feet without encountering anything very interesting. The next 9 holes (B2 to B10) were located rather randomly, west and south of the Hedden Shaft, as shown on Figure 4 which also includes the later Becker holes in the area of the Money Pit:

Each of the first 10 holes ran into bedrock somewhere between 145 and 156 feet. But then Borehole 11 went surprisingly deeper, to a full depth of 200 feet before hitting bedrock! Along the way the hole encountered uniform clay – likely puddled clay[1] (based on findings from later Becker holes) – from 184 to 200 feet. Besides the clay, two "oak buds"[2] were found embedded in a clay sample recovered from a depth of 196 feet. Dan Blankenship described the consistency of the clay as "coming out like toothpaste".

Such a simple discovery, but what a profound meaning it had!! The depth of 196 feet was greater than any known Searcher had every explored at the Money Pit. Glacial deposits do not contain oak or maple tree seeds, so the "oak buds" could only have arrived in the clay well after the glacial period, meaning that the oak buds were relatively recent, in geologic time. **To have relatively recent, organic material embedded in sixteen feet of uniform clay could only mean that the clay, which itself was enclosed in a glacial soil deposit, had been placed there by human hands.** A somewhat similar condition of recent[3] material at depth was encountered in 1970.

Borehole 12 was put down in an old shaft located well south of the Money Pit area. This hole hit large boulders and was abandoned at a depth of 136 feet. Borehole 13 was located 4 feet north of B11 and again the hole went very deep before it struck anhydrite bedrock at 200 feet. Clay was encountered from 184 to 200 feet and careful examination of the clay found that it contained coarser pebble sizes at regular intervals of about 18 inches. This was a strong indication that it was a "puddled clay" deposit. Borehole 14 found clay from 184 to 200 feet. Boreholes 15 and 16 had drilling problems and Borehole 15A was put down four feet east of Borehole 15. Then Borehole 17 found more clay – likely puddled clay – from 176 to 198 feet.

Boreholes 18, 19 and 20 were not drilled anywhere close to the Money Pit. Borehole 21 was inclined to the northeast, in an attempt to get below the bottom of the Hedden Shaft. At 176 feet depth, a piece of slightly crumpled thin brass[4] was recovered (see Figure 5 for photograph).

At first sight, the brass had a bright shiny appearance but it quickly turned a dark color (probably due to oxidation upon exposure to the air). It appeared as if the brass had been torn from a larger piece of brass in the ground. A clay layer – likely puddled clay- was encountered from 184 to 192 feet. Stagnant water and evidence of a possible cavity were found from 200 to 206 feet.

It was at about this point in the drilling program that David Tobias made an intuitive decision. Up until that time, the general instruction to drillers was to stop the Becker holes once sound bedrock was found. However after a number of the early holes had continued to more than 200 feet before bedrock was hit, David Tobias decided to have the drill keep going to at least 200 feet in every new hole, even if bedrock was found at a shallower depth. At first this decision did not pay off – the casing broke off in Borehole 22 and the hole was abandoned before anything interesting was found. Borehole 23 found disturbed ground to 160 feet and then anhydrite bedrock until the hole was terminated at 205 feet.

Borehole 24 changed everything for the Oak Island Searchers. The hole was advanced to rock surface at a depth of 160 feet and the hole was advanced through rock from 160 feet to 192 feet with a tricone bit by rotary drilling with air circulation. **After penetrating 32 feet of continuous rock, a sequence of 4 inches of wood, 12 inches of clay, 4 inches of wood and then a 6 foot cavity was found!!** Since the wall of the hole was in continuous rock from the bottom of the Becker casing to 192 feet, it was concluded that the first wood came from 192 feet depth. Even though the tricone bit was used in this section of the hole, the depth is considered to be representative since air circulation was used to advance the bit. A sample of the wood from 192 feet was radio-carbon dated to 1575, plus or minus 85

years[5]. The carbon dating report is shown on Figure 6 (2 pages). The tricone bit dropped through the cavity under the weight o the rods. The hole was then continued below the bottom of the 6 foot cavity, through bedrock from a depth of 199 feet to a final depth of 207 feet.

The next Borehole, B25, produced results as dramatic as B24. This hole, located 17 feet northwest of B24, was advanced to rock surface at a depth of 146 feet and then the hole was continued to 191 feet using the tricone bit. At 191 feet, **after penetrating 45 feet of continuous rock, a 7 foot high cavity was found, that extended to 198 feet!** Again the tricone bit dropped through the cavity under the weight of the rods. A hard obstruction encountered at the base of the cavity could not be penetrated by the tricone bit, so a diamond bit was put on, and a ½ inch thickness was eventually penetrated after 30 minutes of drilling – while the distinctive sound of a diamond bit on metal[6] was constantly being heard. The conclusion drawn was that the floor of the 7 foot high cavity was covered by a ½ inch iron plate.

Holes 26 to 32 were uneventful and then Borehole 33 was drilled, located 7 feet south of B24. Bedrock was struck at 152 feet and the hole was advanced using a tricone bit. After penetrating 32 feet of continuous rock, 2 feet of soft rocky drilling were encountered, followed by 2 feet of hard drilling and then 2 more feet of soft drilling. **Clay was found from 190 to 192 feet and then a layer of wood was hit. The hole then advanced through a partial cavity** containing soil and fragments of what appeared to be crude lime mortar. Rock was encountered again at 198 feet.

Borehole 34 found bedrock at 156 feet and this was drilled with the tricone bit to a depth of 205 feet without finding anything remarkable.

Borehole 35 was located about 6 feet west of B24. The hole was advanced to rock surface at 160 feet and then continued using the tricone bit to a depth of 181 feet. At that depth, 6 to 8 inches of wood was encountered, followed by a partial cavity from 181 to 192 feet where charcoal and clinker were recovered. An attempt was made to advance the Becker casing to the partial cavity, including down-hole blasting, but this was unsuccessful and the hole was terminated. Boreholes 36 to 39 did not discover anything of great interest; however Borehole 40 encountered rock at 167 feet and continuous clay from 175 to 195 feet with bedrock again at 201 feet. Boreholes 41 to 49 were either drilled somewhere further away from the Money Pit, or nothing of great interest was found.

Well that ended the Becker program around the Money Pit. Out of 49 holes attempted, 6 holes were abandoned because of drilling problems and 9 were drilled at other places away from the Money Pit. A total of 10 holes found something of archeological interest as summarized below in Table 1:

Table 1: Summary of Main Archaeological Features Encountered in Becker Holes

Becker Holes with Archeological Features		
Hole	Features	Depth (Feet)
B11	Puddled Clay	184 – 200
	Oak Buds (probably maple seed samaras)	196
B13	Puddled Clay	184 – 200
B14*	Puddled Clay	184 – 200
B17*	Puddled Clay	176 – 198
B21*	Brass Foil	176 or higher
	Puddled Clay	184 – 200
	Stagnant Water, Possible Cavity	200 – 206
B24	Wood/Clay/Wood sequence (wood 400 years old)	192 – 193
	Inferred Chamber	193 – 199
B25	Inferred Chamber	191 – 198
	Iron Plate (one half inch thick)	198
B33	Inferred Chamber, Wood, Lime Mortar	190 – 198
B35	Wood	178
	Charcoal and Clinker	178 – 190
B40	Solid Clay (Possibly Puddled Clay)	175 – 195
*Asterisk indicates inclined hole, other holes started vertical		

Tobias and Blankenship found 400 year old wood, cavities and an iron plate all under what looked like solid bedrock!! What does it all really mean?? The authors will discuss the interpretation of these fascinating discoveries in Part 2.

Footnotes:

1. Puddled clay is a material made from natural clay (or in the Oak Island context from clayey glacial till with larger cobbles and boulders removed) that has been chopped up and mixed with water to make it very soft and workable. When it is mixed to a soupy consistency, and placed in layers, small pebbles and then coarse sand particles settle to the bottom, with finer and finer particles gradually settling on top of the coarser pieces. For centuries puddled clay was used as a water seal.
2. This reference to oak buds has been reported a number of times in the past, but we doubt that the description is accurate. In the first place, “oak bud” is an unknown term, unless it means the tiny tip of an oak twig, where leaves will grow in the next spring. Dan Blankenship’s field notes give a clue to what was really found. Dan reported that “the oak buds being as light as paper could not have gotten there through any normal or natural action”. The oak bud description sounds very much like

“maple seeds” to us – meaning the seed samaras of maple trees, which grow on Oak Island. Wikipedia says: “These seeds occur in distinctive pairs each containing one seed enclosed in a "nutlet" attached to a flattened wing of fibrous, papery tissue”. We think Dan Blankenship simply was mistaken when he called his find “oak buds”.

3. In 1970 and 1971 the Triton Alliance engaged Golder Associates to conduct a geotechnical investigation which included drilling and sampling near the Money Pit. Palynological analysis of pollen grains found in soil samples recovered from borehole G103 (beneath the Hedden Shaft) from greater than 190 feet established the pollen as post-glacial or “recent”, meaning it had been deposited within the past 2000 years or so. Analysis of pollen grains found in soil samples recovered from borehole G102 (located about 50 feet south of G103) from greater than 190 feet depth indicated this pollen to be ancient, consistent with a glacial deposit. The pollen analyses strongly suggested that the deep soil in the vicinity of the Money Pit had been disturbed by human activity.
4. The piece of brass was about 3 inches in size (consistent with the inside diameter of the return drill pipe) and it was apparent that the brass had been distorted by the drill bit. Considering the size and shape of the brass, it is possible that it was dragged downward by the advancing drill bit before it reported to the surface. The stratigraphy from 0 to 176 feet was defined as overburden and disturbed ground, therefore the brass may have come from a higher level than 176 feet. The brass was analysed and reported to contain much higher levels of impurities than found in modern brass. In the opinion of one expert from Stelco, the brass may have been manufactured before the middle of the 19th century.
5. David Tobias told the authors during a face-to-face meeting, that he was on site when B24 was drilled, and he personally witnessed collection of the sample of wood chips that came from the drill, which subsequently were radio-carbon dated. He also said this was one of the main findings that kept him in the game over the years.
6. Dan Blankenship reported in his field notes, that he collected some of the drilling water that returned from downhole while drilling on the presumed metal object. After letting the water settle, Dan used a magnet to collect iron filings from the bottom of the water bucket. From this Dan deduced that the metal penetrated by the drill was iron. Dan also reported verbally that when the drill string was being brought back to the surface, just as the last drill rod (that held the drill bit at its end) was being uncoupled from the second last rod, he and the driller heard the sound of something falling down into the hole. Dan thought that the drill bit had caught a circular sample of the iron, and this fell out of the bit while the last piece of drill rod was being removed. They tried to capture the fallen piece of metal but they were unsuccessful.

References:

Blankenship, Dan, 1967. Three separate typewritten documents describing the results of the Becker drilling program (these documents are included in MacPhie, 2008):

a) Notes on drilling done by Becker starting January 1967 for Holes B1 to B42 and 15A (three pages, 1967).
b) Notes on selected Becker Holes with interesting features for Holes B1, B11, B13, B14, B15A, B16, B17, B21, B24, B25, B35, B39 and B42 (one page, 1967).
c) Letter to The Cementation Company (Canada) Limited, Brampton, Ontario with brief geological description for Holes B1 to B42, B15A and B45 (three pages, 1967).

Ritchie, J.C., 1970. *Report on Palynological Analyses of Four (4) Samples from the Oak Island Exploration.* Dalhousie University, Halifax, Nova Scotia, May 1970.

Stelco (The Steel Company of Canada Limited), Hamilton, Ontario, 1970. Letter Report, including testing of brass sample, to The Oak Island Exploration by Allen B. Dove, Senior Development Metallurgist, August 18, 1970.

MacPhie, Les, 2008. *The Money Pit, Oak Island, Nova Scotia, Summary of Geotechnical and Archaeological Conditions and the 1967 Becker Drilling Results*. Technical Report, January 2008.

Harris, Graham and MacPhie, Les, 2013. *Oak Island and Its Lost Treasure, Third Edition*. Formac Publishing Company Limited, Halifax, Nova Scotia, 2013.

Acknowledgment:

Thanks to Mark Sykes for drafting Figures 3 and 4.

List of Figures

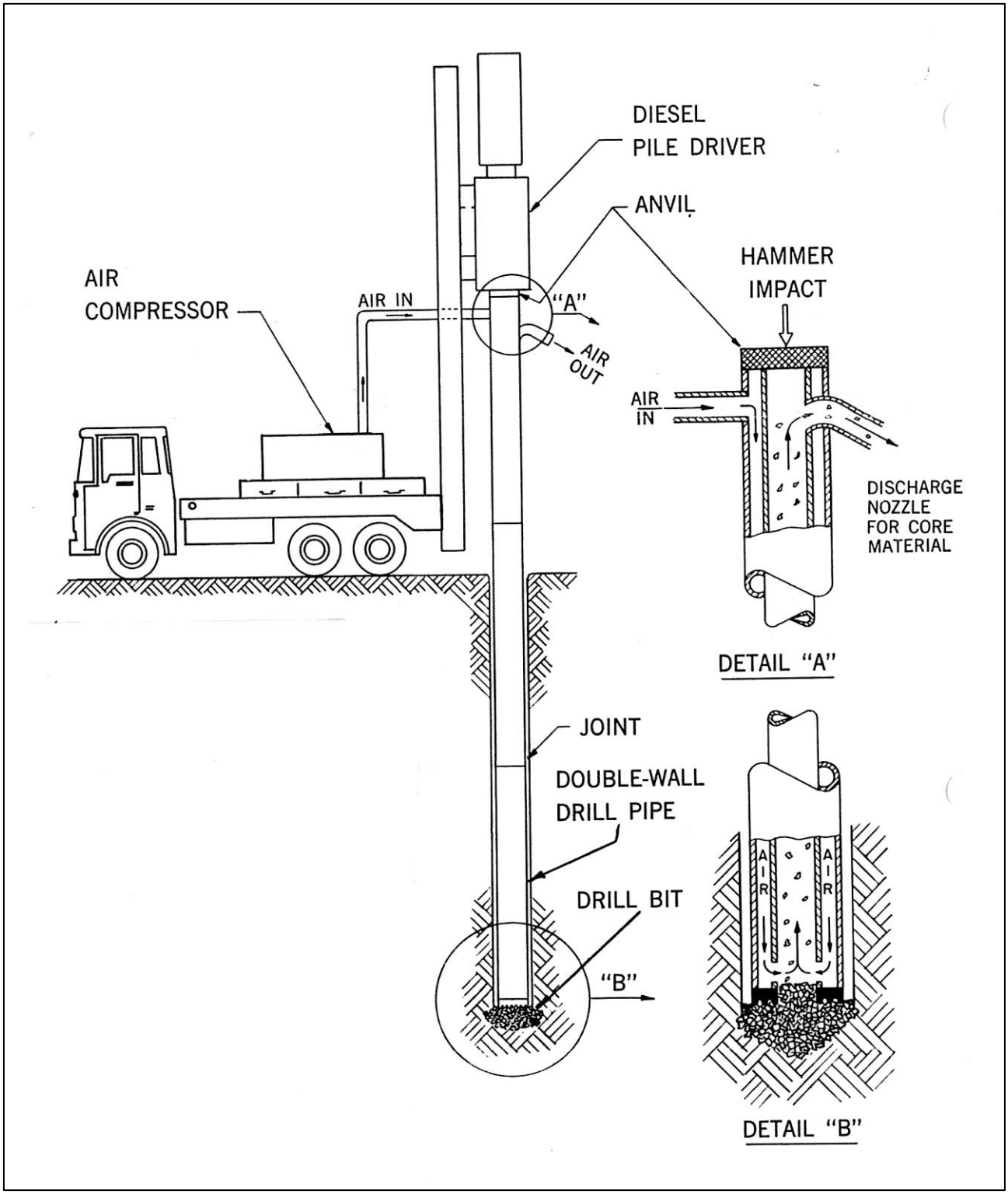

Figure 1: Sketch of Becker Drill

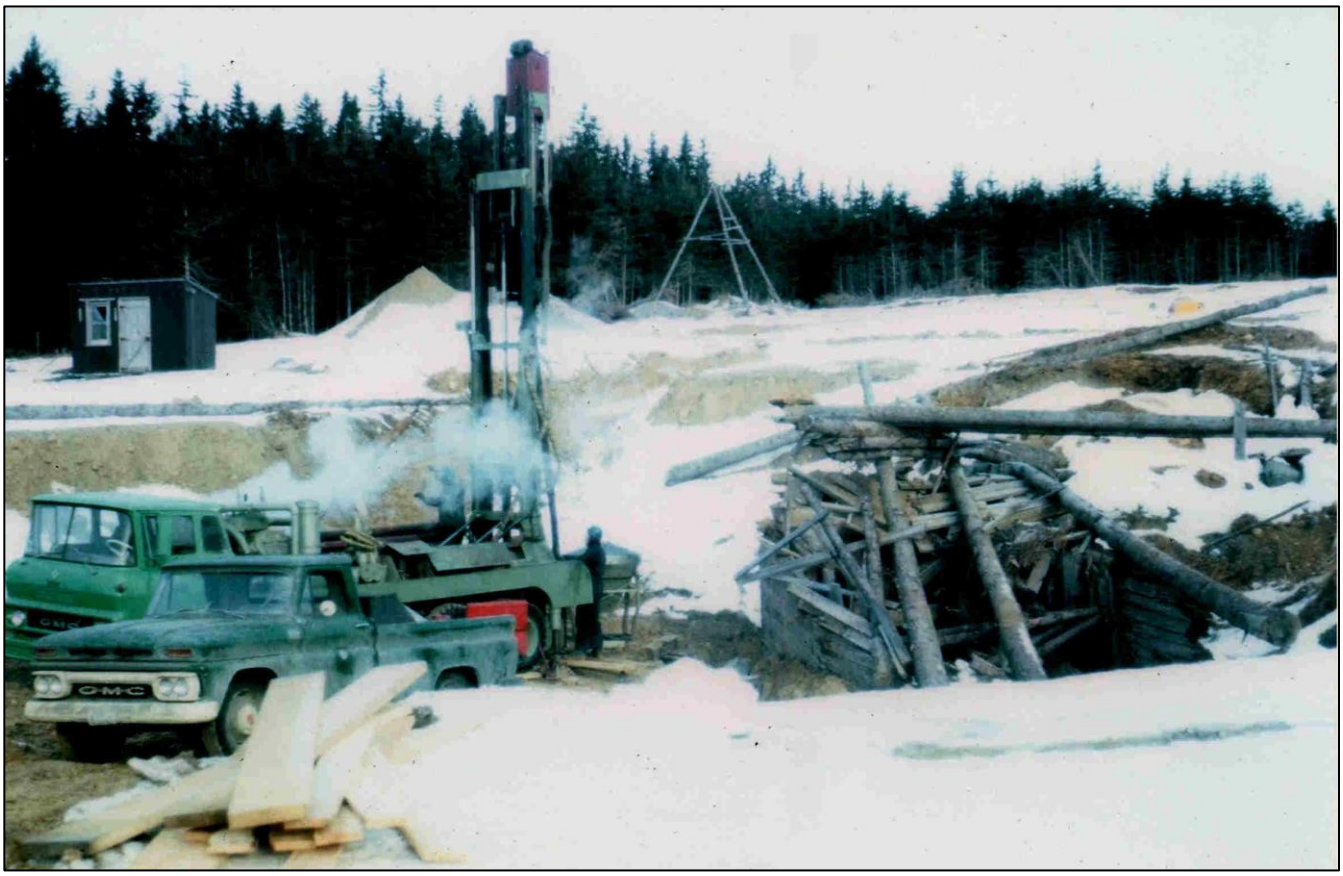

Figure 2: Photograph of Becker Drill beside Remnants of Hedden Shaft

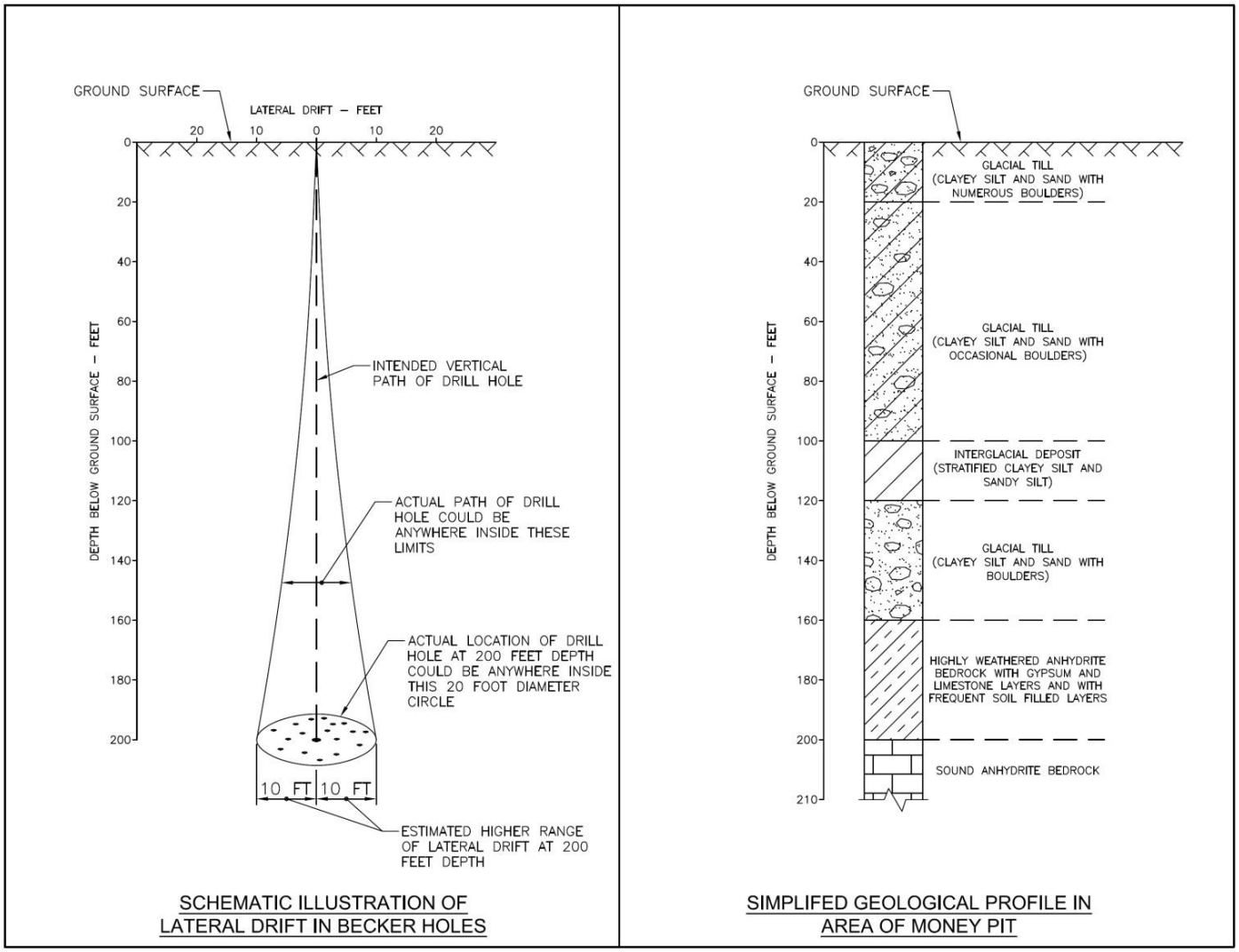

Figure 3: Schematic of Lateral Drift in Becker Holes and Simplified Geological Profile in Area of Money Pit

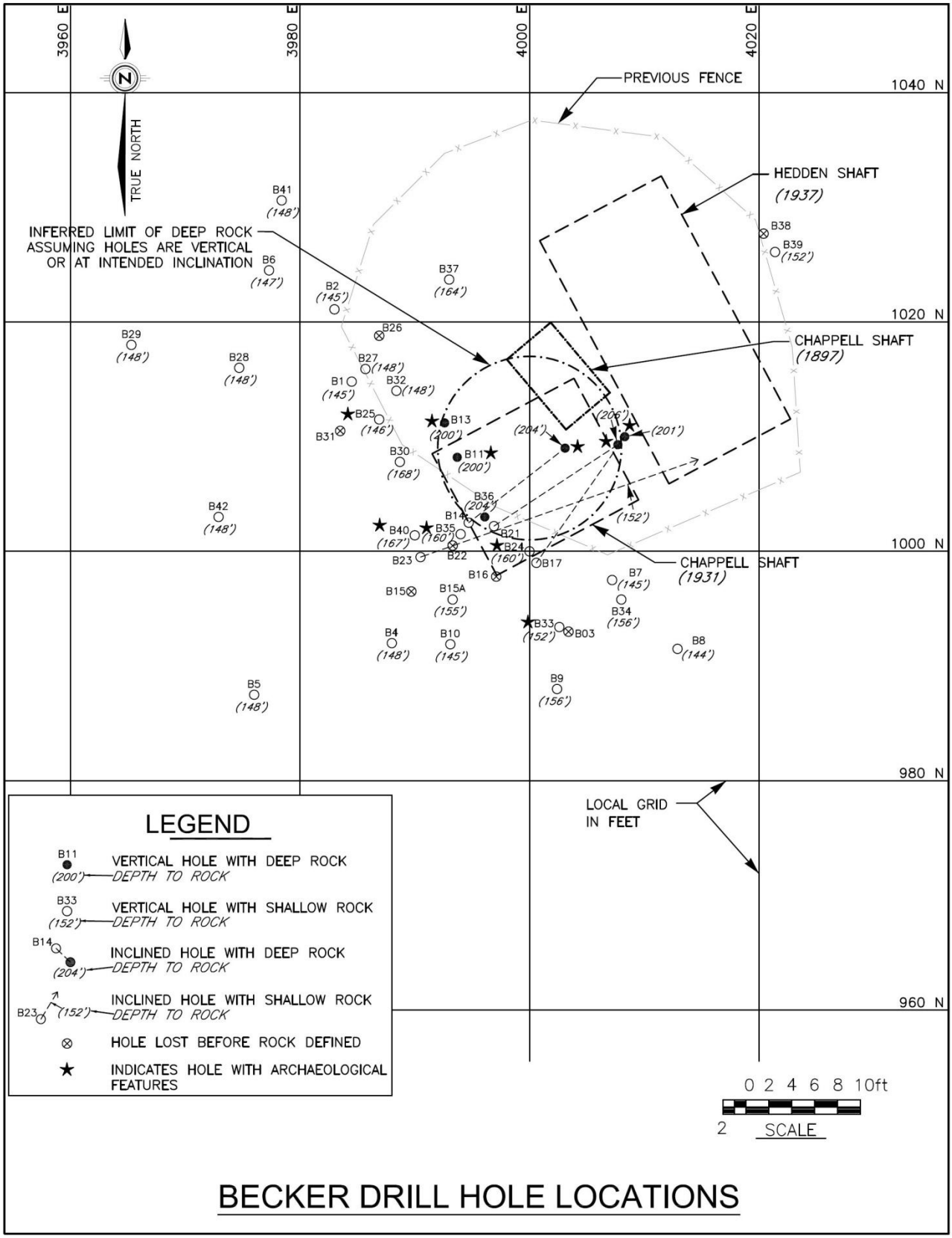

Figure 4: Becker Drill Hole Locations with Identification of Hole Types

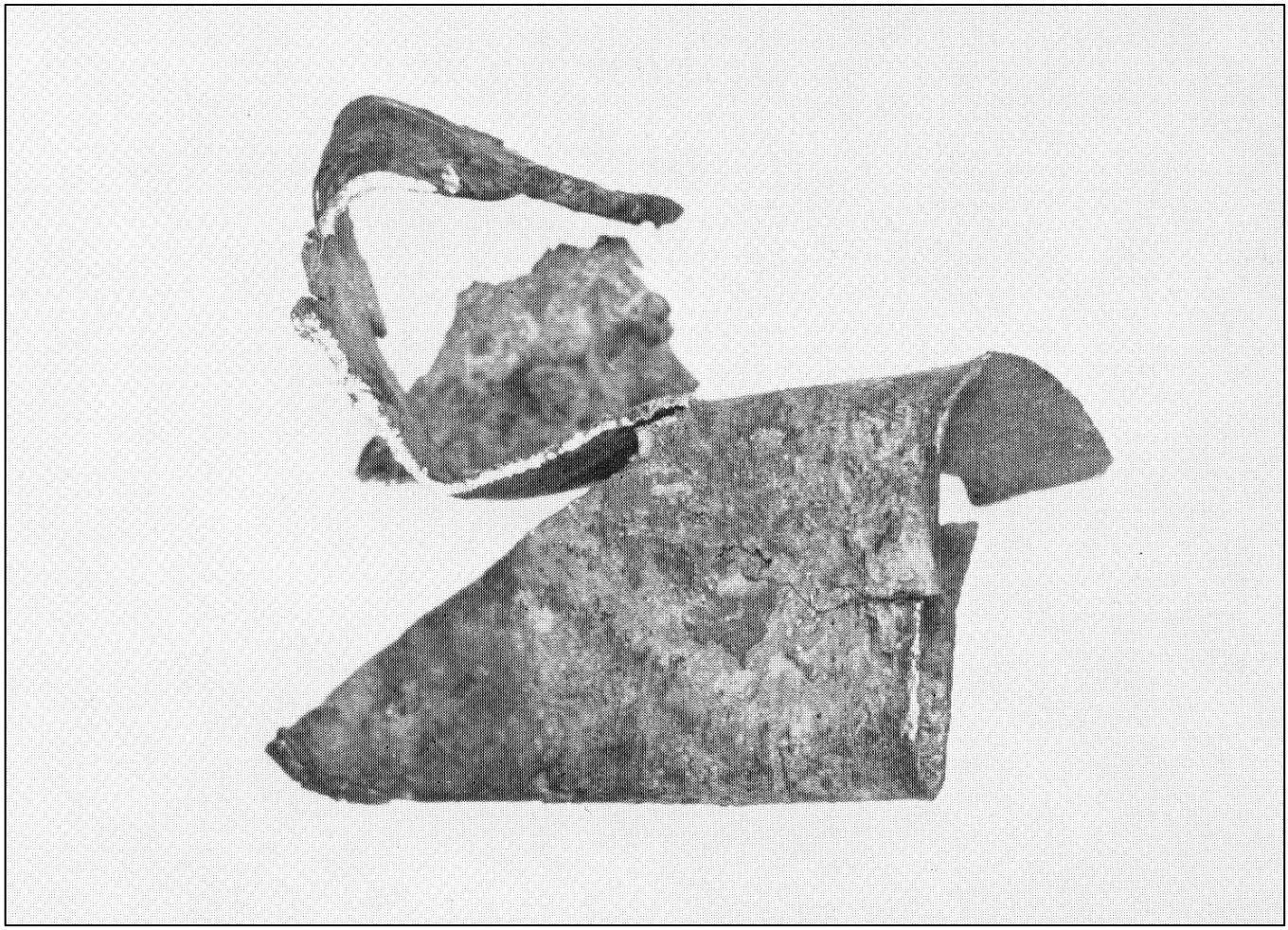

Figure 5: Piece of Brass from 176 Feet Depth or Higher in Becker Hole B21

Geochron Laboratories, Inc.

24 Blackstone Street, Cambridge, Mass. 02139
Telephone TRowbridge 6-3691

3 June 1969

Dr. Michael J. Needham
T'ang Management Ltd.
347 Bay Street
Toronto 1, Ontario

Dear Dr. Needham:

We have now completed the radiocarbon age determination on a sample of wood which you described in your letter of 25 April 1969 and which we discussed later via telephone. As you will recall we decided that we could date only your sample A since samples B and C would undoubtedly be too small for reliable results. You will find our age determination on the enclosed report.

We have determined an age of 375±85 C-14 years B.P. which would correspond approximately to a Christian calendar date of A.D. 1575. Taking into consideration the analytical error associated with the age determination there is a possibility that the wood is very early colonial in age, although it is equally possible that it is just slightly pre-colonial and was emplaced by natural processes. Certainly the wood bears no relationship to glacial deposits since most of these have ages of almost 10,000 years or older.

I realize this date is coming to you slightly later than I had hoped, but I hope it will still prove useful to you in your studies. If you have any questions about the results please do not hesitate to write or call, and will be glad to help you with the interpretation of the results. In the meantime I am enclosing the invoice for this analysis. It has been a pleasure to servie you and I hope we will have the opportunity again in the near future.

Sincerely,
GEOCHRON LABORATORIES, INC.

Harold W. Krueger
Technical Director

HWK/pw
encs.

Figure 6 Page 1 of 2: Carbon Dating Report for Wood Sample from 192 Feet Depth in Becker Hole B24

Geochron Laboratories, Inc.

24 Blackstone Street, Cambridge, Mass. 02139
Telephone TRowbridge 6-3691
617

REPORT OF ANALYTICAL WORK

RADIOCARBON AGE DETERMINATION

Our Sample No. GX-1584

Your Reference: Sample A

Submitted by: Michael J. Needham
T'and Management Ltd.
347 Bay Street
Toronto 1, Ontario

Date Received: 19 May 1969

Date Reported: 30 May 1969

Sample Name: Wood sample A.

AGE = 375 ±85 C-14 years B.P. (A.D. 1575)

Description: Small sample of woody material.

Pretreatment: Selected charcoal fragments were cleaned of foreign material, including rootlets or other contaminating material that could be observed. They were then digested in hot dilute HCl and in hot dilute NaOH to remove chemical contaminants prior to combustions and analysis.

Comment:

Notes: This date is based upon the Libby half life (5570 years) for C^{14}. The error stated as $\pm 1\ \sigma$ as judged by the analytical data alone. Our modern standard is 95% of the activity of N.B.S. Oxalic Acid.

The age is referenced to the year A.D. 1950.

Figure 6 Page 2 of 2: Carbon Dating Report for Wood Sample from 192 Feet Depth in Becker Hole B24

Archaeological Discoveries Deep in the Money Pit

By John Wonnacott and Les MacPhie, June 2016

Part 2 – Interpretation of Findings

1. Introduction

Important archaeological discoveries were made after the Becker drilling program described in Part 1 was completed in 1967. Drilling programs by Warnock Hersey in 1969 and Golder Associates in 1970 also found evidence of original work at depth in the area of the Money Pit as summarized in Section 2 below.

So three different drilling programs, with different crews and different equipment, all found intriguing artifacts close to the original Money Pit at depths greater than the "normal" bedrock surface! What is the real significance of these findings? What can we infer from the hard evidence brought to the surface by the drills some 45 to 50 years ago and what is the most probable reality that lies deep underground? In Part 1 of this article, the authors presented the raw data obtained from the Becker program of 1967. Here in Part 2 we give you the best engineering analysis we can think of, plus our best assessment of what the Depositors actually built at the bottom of the Money Pit. Keep in mind that this discussion relates mainly to the archaeological features at about 190 to 200 feet depth and does not consider other interesting findings higher up. Also we have not ventured into the controversial topic of the various theories which have been put forward to explain the "who" and "what" and "when" of the Oak Island Mystery.

2. Additional Discoveries after the Becker Drilling Program

In 1969 Tobias and Blankenship engaged Warnock Hersey to drill a series of holes. Their report (Warnock Hersey, 1969) indicates that Hole W9 encountered wood chips and highly plastic clay from a depth of 192 to 197.5[1] feet. From 197.5 to 198 feet, clay and a 2 inch thick layer of "red, silty brick-like material" was recovered. Also from 200 to 206 feet, a possible cavity was reported and the drilling return water from this zone was a different color and had a very stagnant odor.

Warnock Hersey Hole W8 (put down at the same location as Becker Hole B24) experienced significant lateral drift to the north based on an accurate down-hole survey in 2015. There was little resistance to drilling from 168 to 200.5 feet and limited sample recovery indicated the presence of loose silty sand. Blankenship (1969) reported that the hole encountered clay from about 178 feet to 204 feet. Our interpretation is that the hole ended at 200.5 feet inclined depth which is equivalent to 199.8 feet vertical depth for the measured

lateral drift of 17 feet to the north (depth rounded to 200 feet). The important implication of this hole is that the top and bottom locations are accurately known and thus it represents a reliable point to identify the configuration of the Deep Rock area at 200 feet depth.

In 1970 Golder Associates were engaged to conduct a subsurface investigation including Borehole G103 which was drilled through the bottom of the old Hedden shaft. Based on pollen count analysis (Ritchie, 1970) of soil samples recovered from over 190 feet depth (Sample 27 at 194 feet depth and Sample 30 at 199 feet depth) this hole contained recent soil from the surface in comparison to pollen count analysis identifying ancient glacial soil (embedded in the weathered anhydrite) at an equivalent depth in Hole G102 located 50 feet to the south of G103. Also Hole G103 encountered a zone of very loose soil from 184.5 to 194 feet depth underlying about 30 feet of sound bedrock.

For convenient reference the summary table of archaeological findings included in Part 1 is reproduced below and the additional archaeological findings are included.

Table 1: Summary of Main Archaeological Features in Holes

Holes with Archeological Features		
Hole	Features	Depth (Feet)
B11	Puddled Clay	184 – 200
	Oak Buds (probably maple seed samaras)	196
B13	Puddled Clay	184 – 200
B14*	Puddled Clay	184 – 200
B17*	Puddled Clay	176 – 198
B21*	Brass Foil	176 or higher
	Puddled Clay	184 – 200
	Stagnant Water, Possible Cavity	200 – 206
B24	Wood/Clay/Wood sequence (wood 400 years old)	192 – 193
	Inferred Chamber	193 – 199
B25	Inferred Chamber	191 – 198
	Iron Plate (one half inch thick)	198
B33	Inferred Chamber, Wood, Lime Mortar	190 – 198
B35	Wood	178
	Charcoal and Clinker	178 – 190
B40	Solid Clay (Possibly Puddled Clay)	175 – 195
W8	Very Loose Silt or Possibly Puddled Clay in Deep Rock Area	168 – 200
W9	Wood Chips, Clay, Brick-Like Material	192 – 198
	Possible Cavity and Stagnant Water	200 – 206
G103	Very Loose Soil Zone	184.5 – 194
	Recent Soil from Surface, Inferred Chamber	194 and 199
*Asterisk indicates inclined hole, other holes started vertical		

3. Location of Key Historical Shafts and Archaeological Boreholes in the Money Pit Area

There have been so many holes drilled since the first Searchers started investigating, that if all the boreholes close to the Money Pit were plotted, the result would be such a cluttered drawing that nothing very useful could be done with it. So Figure 7 shows only key historical shafts and boreholes with archeological findings in the area around the Money Pit (note that figure numbering continues from Part 1).

The first thing that needs to be said, is that we are confident that the Hedden, 1897 Chappel and 1931 Chappel shafts shown on Figure 7 are correct with reference to each other, and their orientation with respect to True North is correct[2]. The locations of the Becker, Warnock Hersey and Golder holes with respect to each other are considered to be accurate. It is possible that the relative locations of the shafts on the one hand and the boreholes on the other hand may not be correct with the shafts possibly being several feet further south with respect to the boreholes. However, our best assessment is that the shaft and hole locations shown on Figure 7 are correct and this is the configuration used for interpretation of the archaeological findings.

4. Lateral Drift of Boreholes

We mentioned in Part 1 of this article, that all boreholes are subject to lateral drift – but unfortunately (except for Hole W8) none of the holes that found archeological artifacts were measured for this possible problem[3]. The many possibilities for the alignment of adjacent holes make it difficult to interpret the configuration of archaeological features. To illustrate this point, Figure 8 provides a number of possible variations of adjacent holes which are six feet apart and vertical or which have 10 feet of lateral drift at a depth of 200 feet. As discussed in Part 1, it is considered that 10 feet of lateral drift at 200 feet depth is likely in the upper range of values, although 10 feet is certainly not the maximum lateral drift possible.

5. Convention for Reporting Depths in Boreholes and Shafts

After the massive excavations by Robert Dunfield in the area of the Money Pit in 1965/66 (Dunfield, 1966), the ground surface was lower by about 10 feet in comparison to the "original ground surface." Therefore, the Becker, Warnock Hersey and Golder drilling programs were carried out from this new ground surface which is referred to as the "existing ground surface." However, depths in the Chappell and Hedden shafts are with reference to the original ground surface whereas depths in the Becker, Warnock Hersey and Golder boreholes are with reference to the existing ground surface. The convention adopted for this article is to refer depths to existing ground surface. Therefore depths in the historical records related to the two Chappell shafts and the Hedden shaft have been reduced by 10 feet.

Figure 9 is a photo of the Money Pit area taken about 1967/68 after the Dunfield excavation was backfilled and after the site was graded. The Hedden shaft is visible but the 1931 Chappell shaft, which collapsed during the Dunfield excavation, is completely buried. This photo illustrates site conditions after the Becker drilling program but before the Warnock Hersey and Golder drilling programs. The site conditions illustrated in the photo indicate that depths below existing ground level reported in boreholes are with reference to a datum surface which is reasonably level but which may vary by several feet between widely spaced holes.

6. Discussion of Main Archaeological Discoveries

In view of the potential for complicated underground configurations of adjacent holes, the only rational approach for discussion of main archaeological discoveries is to start out with the assumption that each borehole was drilled in a straight line at the intended vertical or inclined orientation, keeping in mind that each hole could have wandered off-line as illustrated in Figure 8. There are at least four main discoveries that can be identified from a first examination of the archeological data:

1. Wood of modern age (versus glacial age) was found below bedrock in Hole B24, which is highly indicative of human activity.
2. Very stagnant-smelling water was found below bedrock, which indicates rotting organic matter, and this is considered to indicate human activity.
3. The depth to bedrock in the general area of the Money Pit is between 145 and 160 feet, however, there is a cluster of holes where bedrock is not found until about 200 feet. We refer to this unusual feature as the Deep Rock area.
4. Most of the Deep Rock holes had a thick layer of puddled clay just before the bottom of the hole.

We think these discoveries are linked logically, as we will explain a bit later – but first let's discuss each of these in more detail.

There are only two ways that we can think of, to explain how modern wood could end up beneath a significant depth of bedrock, which itself is covered by about 160 feet of glacial till. The first scenario would involve development of a large natural cavity in the bedrock and then, well after the last glaciation, development of a sinkhole by "rat holing" from the cavity up through the thick glacial till layer to the surface. This event would then have to be followed by transfer of surface wood to the bottom of the sinkhole with lateral transfer of the wood into the cavity so it is overlain by some 30 to 40 feet of bedrock. We believe this first scenario to be practically impossible, particularly since there is no evidence of massive sinkholes on the Island. The second scenario is that Oak Island Depositors dug a shaft vertically down through 160 feet

of glacial till and hit bedrock consisting of weak, weathered anhydrite. As the weathered anhydrite would have been structurally weak, the Depositors continued excavating vertically into the bedrock until they were in sound rock, and then they excavated one or more lateral chambers. If the Depositors were concerned about stability, they would have shored the roof of their chamber(s) with wood that was thick enough to be structurally competent.

The condition of the weathered anhydrite during the Depositors' work was completely different from present day conditions. Initially any significant fractures in the weathered anhydrite would be completely filled with low permeability soil thus allowing excavation with only limited and manageable water inflow. In its present condition, the weathered anhydrite is highly pervious due to removal of soil from fractures during long intervals of Searchers' pumping over the years.

Samples of the wood found below the bedrock surface in Hole B24 were radio carbon dated and an age of about 400 years was obtained. We now know that radio carbon dating for samples less than about 500 years old is not very reliable[4]. However, the results are positively definitive in that the wood had to come from a tree which lived no more than about 800 years ago and thus well after the last ice age. So our inescapable conclusion is that the wood found in these holes is of archeological interest and it was placed in chambers created by whoever dug the original Money Pit.

We think the smell of stagnant water from below bedrock comes from submerged organic matter that has decomposed in a low dissolved-oxygen environment. The same argument that concludes that wood found at this depth must have been placed as a result of human activity, strongly suggests that whatever organic matter caused the stagnant smell, must have originally been placed in the local area by human activity. It is interesting to note that the drilling wash water return (when drilling below the bedrock surface) did not generally have a stagnant smell – the odor was only reported in connection with several subterranean cavities. So all the deep groundwater does not have a stagnant smell, it only occurs in a few deep cavities in the bedrock. We consider that where very stagnant water has been found in cavities or chambers below the bedrock surface, those locations are of archeological significance.

Enough boreholes have been drilled in the general area of the Money Pit that we know the bedrock surface does not naturally vary by more than 10 or 15 feet in elevation except at the Deep Rock area. So either the Deep Rock area is a natural depression in the bedrock surface or it is a man-made feature. We really can't be absolutely sure without excavating and examining the area, but drilling records indicate that the sides of the Deep Rock area seem to be about vertical. Most natural bedrock depressions have sloping sides. We also observe that sound anhydrite bedrock occurs below the bottom of the Deep Rock area, whereas the top of

anhydrite bedrock is weathered everywhere in the area, other than at the Deep Rock area – and this suggests that the Deep Rock area has not has enough time to begin to show signs of weathering. So it appears that the Deep Rock zone was excavated vertically through 35 to 50 feet of weathered anhydrite. Everything we know about the Deep Rock area is consistent with the concept that it is man-made. We have selected a diameter of 16 feet for the Deep Rock area and realize that the diameter could be somewhat larger (if some of the boreholes were subject to lateral drift).

We discussed the characteristics of "puddled clay" in Part 1 of this article. Puddled clay was used for centuries to seal areas from water intrusion. It is very interesting that puddled clay was found in the Deep Rock and Money Pit areas and nowhere else in the immediate area. Since we believe these features are man-made, we consider that the puddled clay was placed by Depositors as part of their underground construction.

We have concluded that the wood found below bedrock was part of a roof shoring system for structural support of chambers excavated in the rock. We also believe that the Depositors were concerned about water seeping into their chamber(s) from above, particularly while they were working, so they would have sealed the roof with clay after shoring the roof, and they would have needed a second layer of wood to hold the clay in place. This accounts for the "wood/clay/wood/ 6 foot space" sequence found in Hole B24. Perhaps the side walls of the chambers are also lined with clay, but we have no data to support or contradict this notion. Either way, seepage water would have been directed around the chamber(s), collecting on the chamber floor, where side drains could have conducted it to sumps where it would have been pumped, or lifted in containers, to the surface. When the Depositors had finished building their chambers and depositing their treasure, we believe that they would have installed a timber bulkhead at the entrance of each chamber and then filled the bottom of the Deep Rock area with puddled clay. Based on current engineering principles, the reason for using puddled clay in this fashion is not readily evident to us. It is likely that the Depositors had reasons for using puddled clay which are not consistent with modern practice in underground engineering.

7. Discussion of the Configuration of the Underground Workings

Because wood was found at several quite widely-spaced locations, we conclude that there is more than one lateral chamber in the bedrock. For example, Boreholes B25 and B33 are 24 feet apart, and a single underground chamber 24 feet wide would require massive geotechnical supports – so this suggests that B25 and B33 encountered separate chambers extending out from the base of the Deep Rock area. However, there is an unlikely scenario where these two holes each drifted 10 feet toward each other in which case the holes would be 4 feet apart at 200 feet depth and thus could be in the same chamber. We consider that this scenario is highly improbable.

We don't know how many chambers there are, but we have settled on three chambers as being a likely configuration and we have numbered them Chambers 1, 2 and 3. Chamber 1 is in the area of Holes B24, B33 and W9, all located to the south of the Deep Rock area and all having archaeological features at about the same depth. Chamber 2 is associated with Hole B25 where an iron plate was encountered at 198 feet depth. Chamber 3 is associated with Borehole G103 where recent soil from the surface was encountered. It is noted that, since G103 was drilled through the bottom of the 115-foot deep Hedden shaft, there would not have been much lateral drift in this hole. It is of interest to note that G 103 is about 24 feet from both B25 and B33 and that B25 and B33 are about 24 feet apart.

There is an interesting demonstration of lateral drift and how elusive the chambers are when trying to intersect them with a borehole. Warnock Hersey Hole W5 (not shown on Figure 7) was put down half way between Holes B24 and B33, both of which were assumed to intersect Chamber 1, with the objective of verifying the continuity of the chamber. However no indication of a chamber was found in this hole which extended to a depth of 210 feet. Also Warnock Hersey Hole W7 (not shown on Figure 7) was put down 3 feet east/northeast of Hole W5. No archaeological features were found in this hole which extended to a depth of 218 feet. However, both Holes W5 and W7 encountered significant soil inclusions in the weathered anhydrite.

We think the Depositors tried to construct chambers with a 6 foot roof height, as that would have been a convenient height for mining – and where the chambers were measured to be 7 feet high, those would be places where more of the roof came down than intended, during the mining. We don't know the length of the chambers but Holes B 33, B25 and G103 at Chambers 1, 2 and 3 respectively are only some 5 to 7 feet (at surface) beyond the inferred limits of the Deep rock area. We have assumed for illustration purposes that the chambers extend about 20 feet beyond the limit of the Deep rock area. Figure 10 shows a plan view of the chambers, Figure 11 shows Section A-A through Chamber 1, Figure 12 shows Section B-B through Chambers 2 and 3 and Figure 13 illustrates our concept of a typical cross section through a chamber. Figure 12 shows a projection of the original Money Pit to Section A-A and the implications of this configuration are discussed in Section 8.

As explained in the preceding text, the presence of puddled clay, and the use of clay between layers of wood in the shoring for chamber roofs, indicates that the Depositors had concerns about ground water seeping downward into their workings. During construction of the underground chambers, this seepage water would have been an issue for the Depositors to deal with, and the conventional way to manage seepage water like this, is to excavate one or more sumps at low points in the underground workings, and install some form of pumping at those places. We think the ½ inch iron plate that was found in Hole B25 may possibly have been

used to cover a sump hole which would have been excavated in the floor of one of the chambers. The plate would have been pulled back for pumping, and replaced when people needed to walk across the chamber floor.

So in our opinion, all of the archeological features found below the bedrock surface are consistent with construction of deep chambers which very likely would have been excavated as storage places for objects of great value. In other words, we think the archeological discoveries mentioned in this article strongly support the idea that the Money Pit was constructed to store some form of treasure or highly valued artifacts.

8. A Remaining Problem

After developing our concept for the configuration of chambers at 200 feet depth, we are left with one very puzzling and troublesome observation. The center of the Deep Rock area is about 18 feet away (to the southeast) from the center of the original Money Pit. Most previous publications (including Harris and MacPhie, 2013 and Triton Alliance, 1988) assumed that the original Money Pit was directly over the Deep Rock area. Our best estimate of where the Money Pit was located is shown on Figure 14. The supporting evidence for the correct Money Pit location is discussed below.

The correct position for the original Money Pit has always been a subject of some uncertainty. In the first place, we have no record of any reliable survey having been done, which tied the original Money Pit to other still-existing reference points. During the 1800's many attempts were made to excavate at the Money Pit, and when these initial attempts to recover treasure were flooded out by water rapidly rising in the original workings, a series of new shafts were dug near the original Money Pit. These new shafts eventually either collapsed or were later backfilled so that by the 1930's no evidence of the original Money Pit was visible at original ground surface. One good reference to the position of the original Money Pit is based on the work of William Chappel[5] in 1897. In this regard, his records are the oldest reliable accounts on the subject, and thus should contain the fewest historical inaccuracies. M. R. Chappell, in his manuscript (Chappell M. R., 1973, Page 24) recorded information on the location of the 1897 William Chappell shaft as follows: *"At this time the Money Pit which was not cribbed and therefore not too safe was used as the pumping pit, work and drilling being done in a new timbered shaft about five foot by eight foot south of and close to the Money Pit."* It is noted on Figure 14 that the perimeter of the Money Pit is only 4 feet from the 1897 shaft and that the shaft is positioned at a right angle to the Money Pit perimeter as would be expected. Since constructing a shaft would require a lot of effort and a sizeable financial outlay, we assume William Chappell consulted old records and spoke to older eye-witnesses to be as sure as possible that his new shaft was located close to the old Money Pit. Chappell probably

had direct access to people who worked on Oak Island in the 1850's, and those early Searchers almost certainly knew where the original Money Pit had been located.

Additional evidence of the original Money Pit location was recorded during the work by Dunfield (1965) where he reported that *"At 8:00 am, November 4, 1965, we reached a depth of 22 feet and we have observed 1/2 to 2/3 of the original money pit well defined immediately north of the Chappell Shaft."* M. R. Chappell also reported (Chappell M. R., 1973, Page 82) on the position of the original Money Pit exposed by Dunfield as follows: *"the thirteen foot circular uncribbed shaft appeared about fifteen feet north of the Chappell shaft and ten feet west of the north end of Hedden shaft."* The dimensioned position reported by M. R. Chappell was used to plot the location of the original Money Pit shown on Figure 14 with the dimensions assumed to be to the center of the Money Pit. It is clear that this location is also consistent with the other two descriptive locations of the Money Pit.

Our thinking of the way the Money Pit was constructed, is that the Depositors dug a vertical shaft through the glacial till, and when they encountered the anhydrite bedrock, with weathered zones and soil-filled cavities near the surface, they just kept excavating through the rock until they got to sound rock conditions, and at that depth they started excavating lateral chambers. We always thought that the Deep Rock area was the downward extension of the Money Pit – so how could the bottom of the Money Pit be so far away laterally, from the Deep Rock area?

We have been puzzling over this problem for the past few weeks. One obvious answer to the problem could be that there was a series of surveying or recording errors in the positions of the boreholes or the Hedden/Chappel shafts. There indeed were several errors made in some of the old surveys, but we just cannot find any errors that are anywhere close to being large enough to explain away the problem.

Another possibility is that the boreholes that ran into the Deep Rock area all drifted laterally to the north/northwest. Any one hole could have deviated by 18 feet, but we just cannot accept that all the holes that hit Deep Rock had the maximum lateral drift, all in the same direction!! Theoretically it would be possible, but the probability of that happening is astronomically tiny.

We are now mulling over the possibility that there were two vertical components to the Money Pit – another possible device, something never considered by Searchers until now – created by the Depositors to throw off future Searchers, if they ever dug as deep as the bedrock surface. Perhaps the original Money Pit was dug exactly where we believe it was dug, down to bedrock, and then perhaps the Depositors built a short lateral tunnel with temporary shoring that they would have removed after depositing their treasure, and from there they excavated

vertically into the bedrock, creating the Deep Rock area and the underground chambers in rock. There are other possible scenarios for connecting the original Money Pit and the Deep Rock area. However, this issue has not been addressed in detail since it is beyond the scope of this Part 2 article.

We know the idea of having an offset shaft in the bedrock, as described above, may not be readily accepted. This is why we have taken so long to write the second part of this article – we have been struggling to find a solution - but we just cannot find another explanation for the 18 foot discrepancy between the Money pit location and the Deep Rock area location. So we are opening up this point for discussion by thoughtful Readers. We have gone through a lot of historical records and surveys that confirm the location of the shafts and boreholes, and we can provide the details if there is enough technical interest.

We are asking Readers to suggest alternative solutions to this remaining problem. There can be little doubt that the archeological discoveries of the Becker, Warnock Hersey and Golder drilling programs demonstrate that there are man-made workings deep below the normal bedrock level at the Money Pit. So what exactly did the Depositors build?? There is a way to find out.

9. Large Diameter Shaft to 200 Feet Depth

The concept of constructing a large diameter shaft to 200 feet depth to explore the chambers at that depth was considered by Triton Alliance in 1988. The project is described in their promotional document (Triton Alliance, 1988) but the project did not proceed due to lack of financing. However, this approach would give detailed information on the configuration of the chambers at 200 feet depth, would determine the contents (if any) of the chambers and would in all likelihood solve the Oak Island mystery. In addition, issues related to chests at 90 feet and a vault (with parchment?) at 140 feet should be resolved, because those probable artifacts would be within the area to be excavated. Excavation within a large diameter shaft would have to be carried out with particular care to preserve the archaeological heritage of the Money Pit, the Deep Rock area and the related chambers, as required by current legislation.

Several technologies for design and construction of such a shaft, even in the difficult ground conditions at the Money Pit, are readily available and there is successful precedent for the use of such technologies. The authors have participated as external advisors in three different conceptual designs for a large diameter shaft at the Money Pit as part of a Design Project program for fourth year engineering students at McGill University, Montreal, Quebec. The reports on these Design Projects (McGill University 2006, 2008 and 2014) include only conceptual designs and order of magnitude cost estimates. The technologies considered are ground freezing (perimeter freeze ring), secant piles (continuous ring of interconnected

concrete piles) and grouting (jet grouting and conventional pressure grouting). We consider the ground freezing design to be the preferred approach.

One of the interesting questions for such a design is to select the diameter of the shaft. This depends to a large extent on the actual length of the chambers. Assuming that the chambers are 20 feet long and allowing a distance of 7 feet between the end of the chambers and the wall of a shaft centered on the Deep Rock area, the diameter of the shaft should be 70 feet. The plan limits of a 70 foot diameter shaft are shown on Figure 15. It is noted that there is a clearance of about 10 feet between the wall of the shaft and the closest edge of the original Money Pit. In final deliberations for shaft design, consideration could be given to a larger diameter or to moving the shaft some 5 feet northwest (from that shown on Figure 15) so that it is centered more on the combined Money Pit and Deep Rock areas. If additional drilling more accurately identifies the configuration of the chambers, consideration could also be given to constructing a shaft with a diameter smaller than 70 feet. On the other hand, Triton Alliance proposed an 80 foot diameter shaft to 200 feet. Figure 16 shows a schematic cross section of the 80 foot diameter shaft proposed by Triton Alliance in 1988.

We think that a large diameter shaft to 200 feet would not only reveal many of the secrets of Oak Island's Money Pit, its construction would generate tremendous world-wide interest. Imagine a weekly television show that presented all the artifacts, old Searcher's structures and construction issues encountered since the last episode. Until someone builds such a shaft, the idea will give us all something to daydream about!

10. Conclusion

We hope through Parts 1 and 2 of this article, that we have demonstrated strong evidence that there are man-made structures deep in the Money Pit area. These structures are far deeper than any Searcher has ever been known to have explored. We can understand an overall construction sequence in which the deep underground workings were first constructed, and then the flood tunnel (possibly more than one) was built to prevent Searchers from recovering whatever was placed in the expected underground chambers. But the possibility that some unknown Searcher could have overcome the man-made water problems and other technical challenges, dug down to 200 feet and created these underground workings – and kept all of their efforts a secret to this day – is beyond what we are prepared to believe. So we conclude that the archeological artifacts buried deep in the Money Pit area are the work of whoever first created the Money Pit. We believe there is original construction down there, and very possibly treasure of great value.

Footnotes:

1. The Warnock Hersey drilling program consisted of casing the hole when they were in overburden, coring with a diamond bit and core barrel when they were in rock, and sometimes drilling ahead with a tricone bit when the drilling conditions became too difficult for the core barrel to advance. So they were not able to obtain a continuous record of samples in some places. The zone from 192 to 200 feet in Hole W9 was one of those places. The drilling notes for this hole indicate that the drillers thought that perhaps the drill was on wood at a depth of 192 feet in this zone, and that is what impeded the core barrel. We do not know precisely where in this zone the wood was encountered, but we know that wood chips came back in the wash water in this zone, and the drillers thought they had drilled through at least 4 feet of solid wood.
2. The location and true north orientation of the 1931 Chappell shaft and the 1937 Hedden shaft were defined by the Roper survey in 1937.
3. Borehole W8 (one of the Warnock Hersey holes) had a lateral drift measurement of about 17 feet to the north. There is no reason to assume this hole had the greatest lateral drift of any borehole, so we must assume that at least some other holes had lateral drifts of 17 feet or more.
4. Because new research has found that the atmospheric concentration of C_{14} was not constant throughout the past 500 years, so when the C_{14} concentration in the sample being tested is measured, and the known decay rate of radioactive C_{14} is used to calculate the age of the sample, multiple ages result because there are multiple possibilities for the initial concentration of C_{14}.
5. William Chappell was the father of Mel R. Chappell, who constructed a new "Chappell Shaft" in 1931. When M. R. Chappell was laying out the position for his shaft, William Chappell was on the site to help his son. During the early excavation of the 1931 Chappell Shaft, William Chappell was on site and was able to identify parts of the old 1897 shaft that the new excavation was uncovering. Therefore the position of the two Chappel shafts with respect to each other is quite precise.
6. The depths of 150 feet and 161 feet are 160 feet and 171 feet respectively in the Chappell affidavit.

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(Note: The initial report was followed by Warnock Heresy Letter dated August 27, 1969 transmitting the logs for Boreholes W5 and W7 and by Warnock Hersey Letter dated November 5, 1969 transmitting the descriptive results of Boreholes W8, W9 and W10.)

Acknowledgements

We would like to acknowledge Oak Island Tours for providing detailed information on the lateral drift measurements in Hole W8.

Also we wish to acknowledge the excellent work by Mark Sykes in drafting most of the figures for this article.

List of Figures

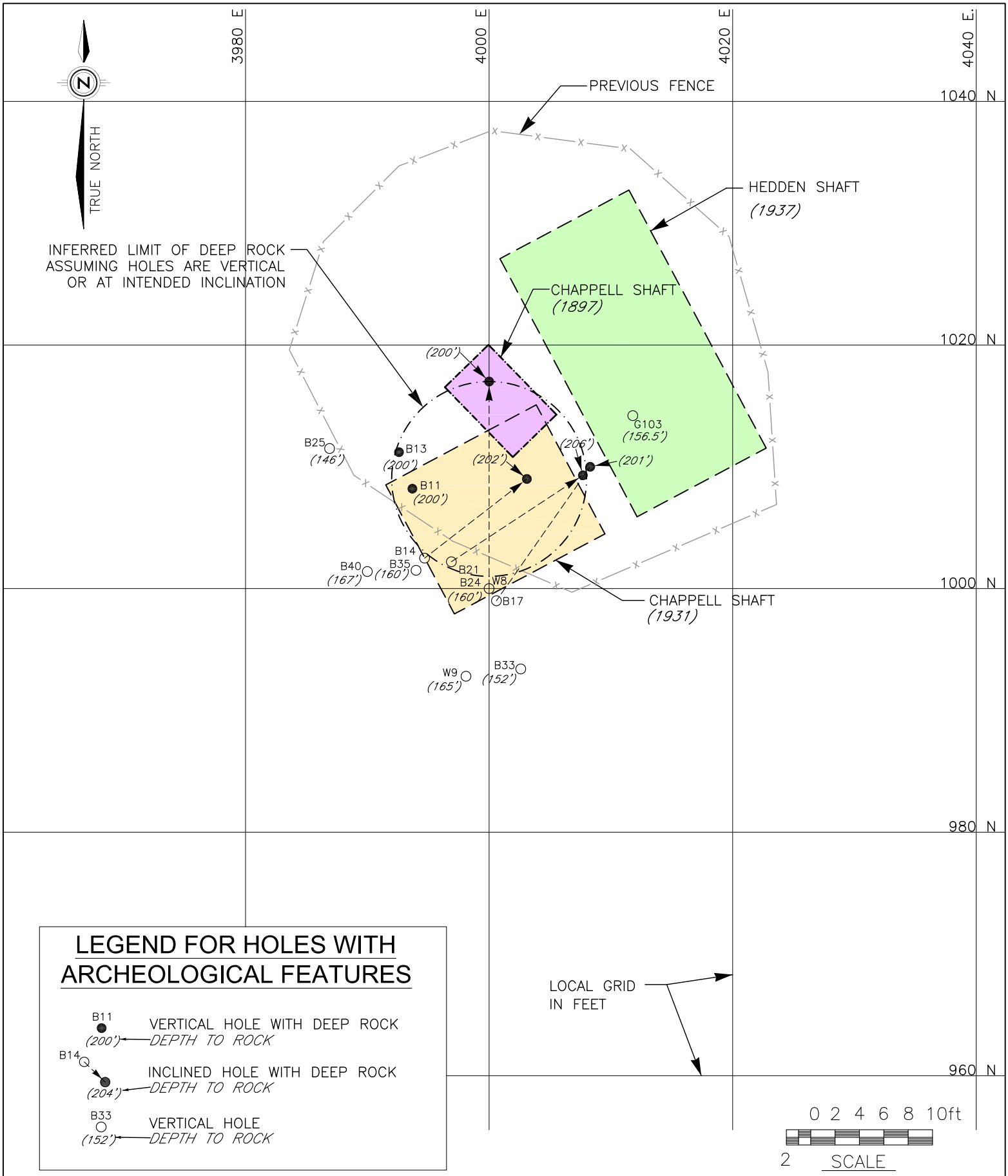

FIGURE 7: PLAN OF HOLES WITH ARCHEOLOGICAL FEATURES

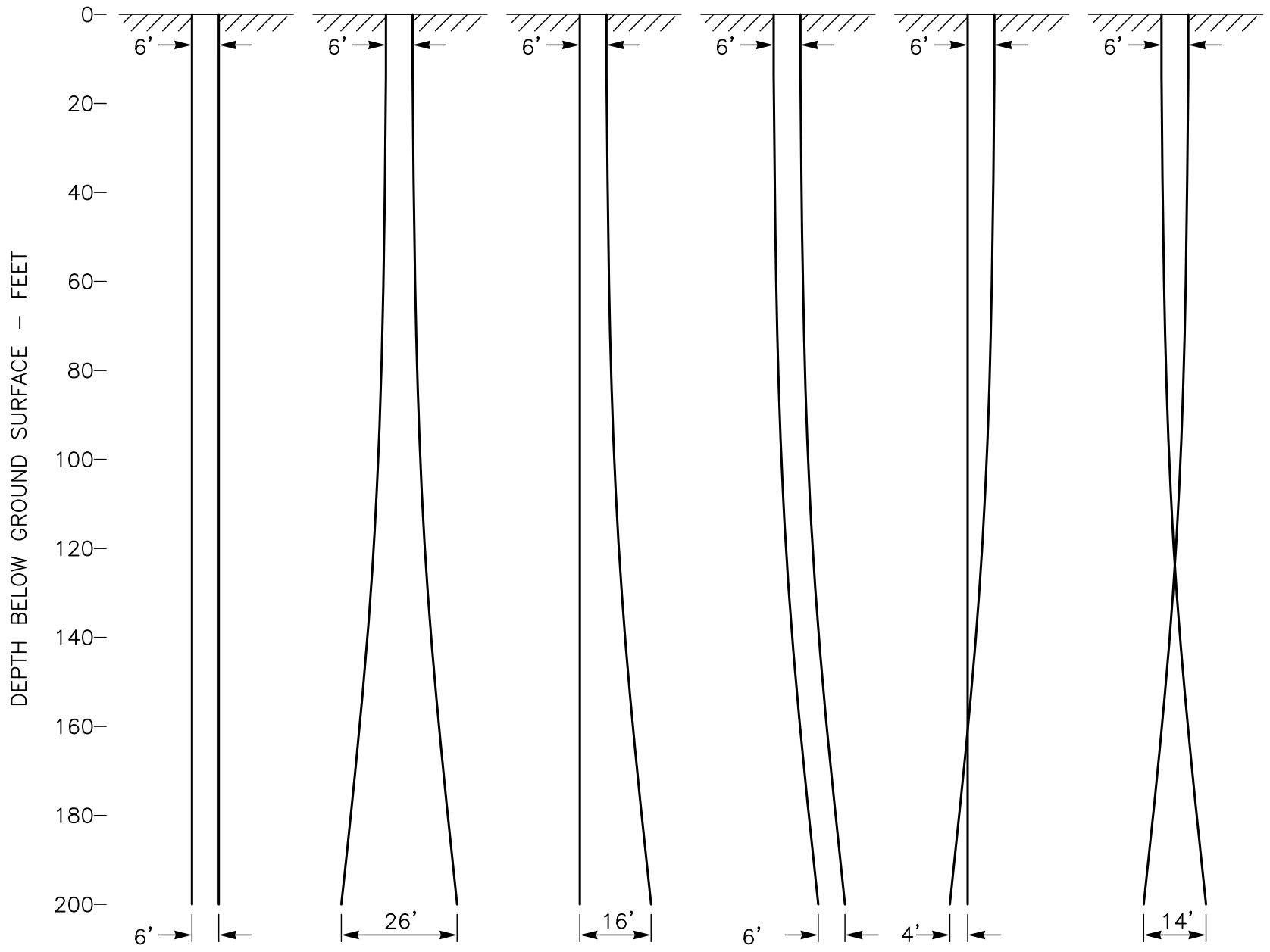

FIGURE 8: POSSIBLE CONFIGURATIONS IN PROFILE OF TWO HOLES 200 FEET DEEP AND 6 FEET APART AT SURFACE (HOLES ARE VERTICAL OR HAVE 10 FEET OF LATERAL DRIFT AT 200 FEET DEPTH)

Figure 9: Photo of Hedden Shaft Looking West toward Swamp - circa 1967/68

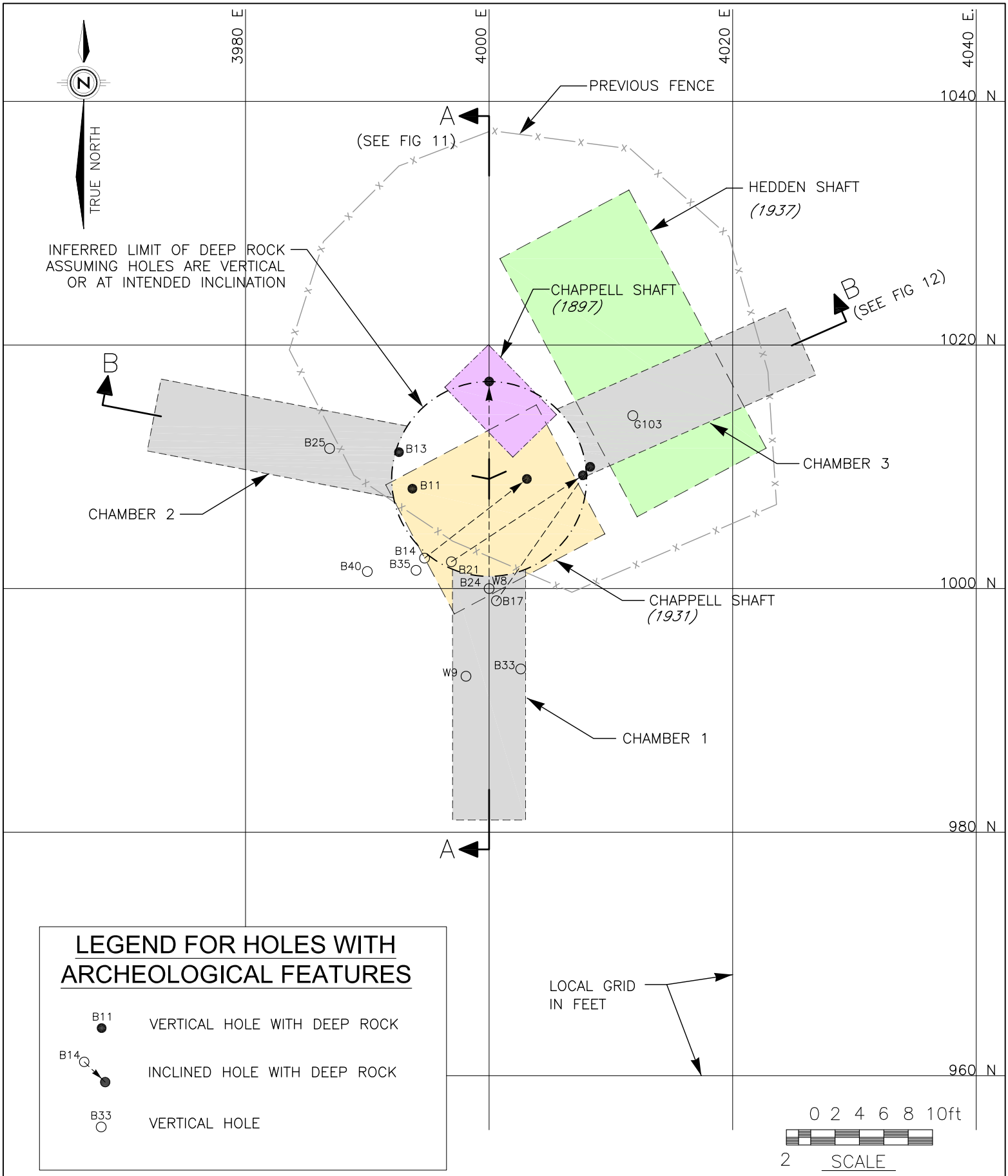

FIGURE 10: PLAN SHOWING POSSIBLE CONFIGURATION OF CHAMBERS AT 200 FEET DEPTH

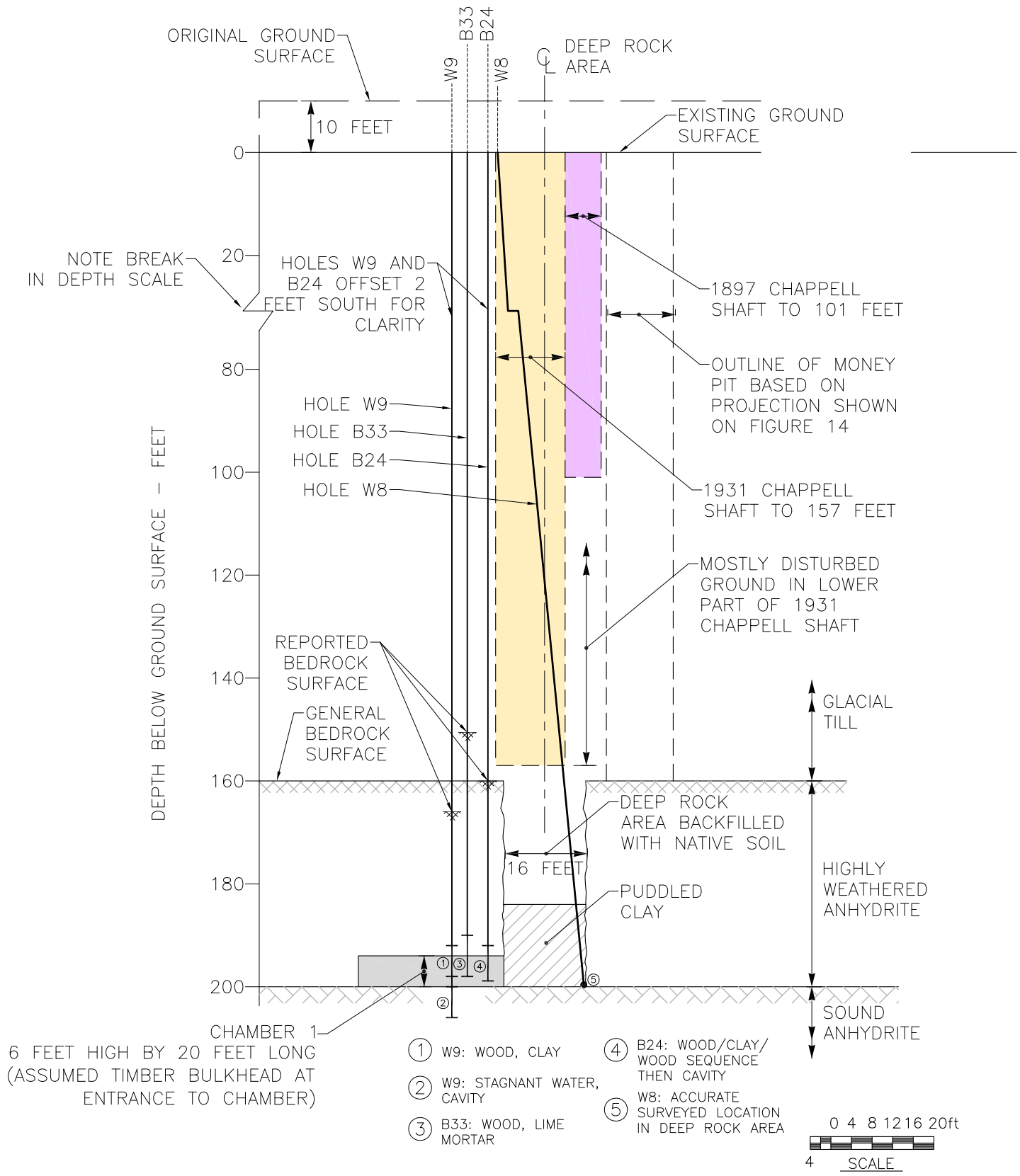

FIGURE 11: SECTION A-A THROUGH DEEP ROCK AREA AND CHAMBER 1 (LOOKING WEST)

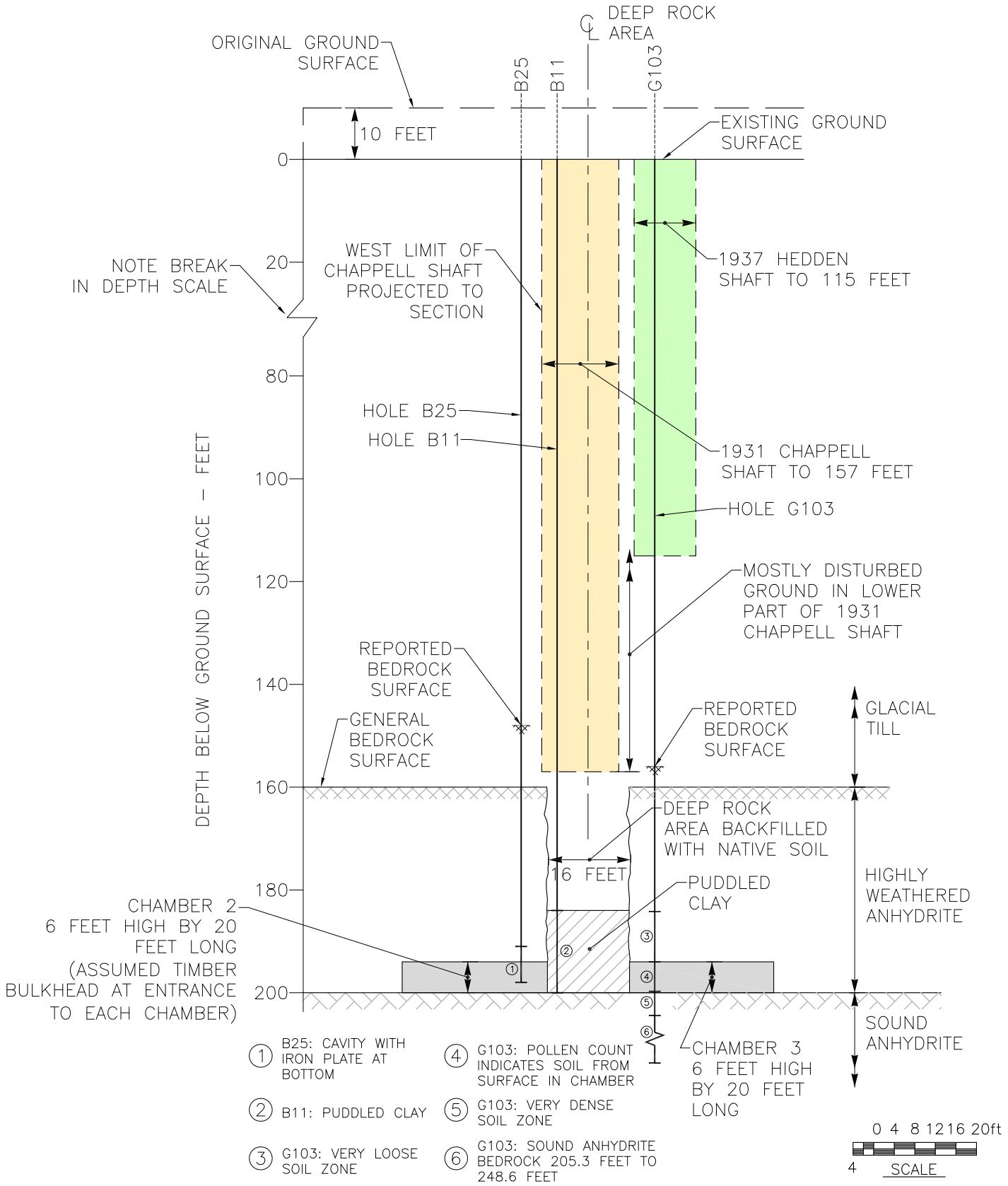

FIGURE 12: SECTION B-B THROUGH DEEP ROCK AREA AND CHAMBERS 2 AND 3 (LOOKING NORTH)

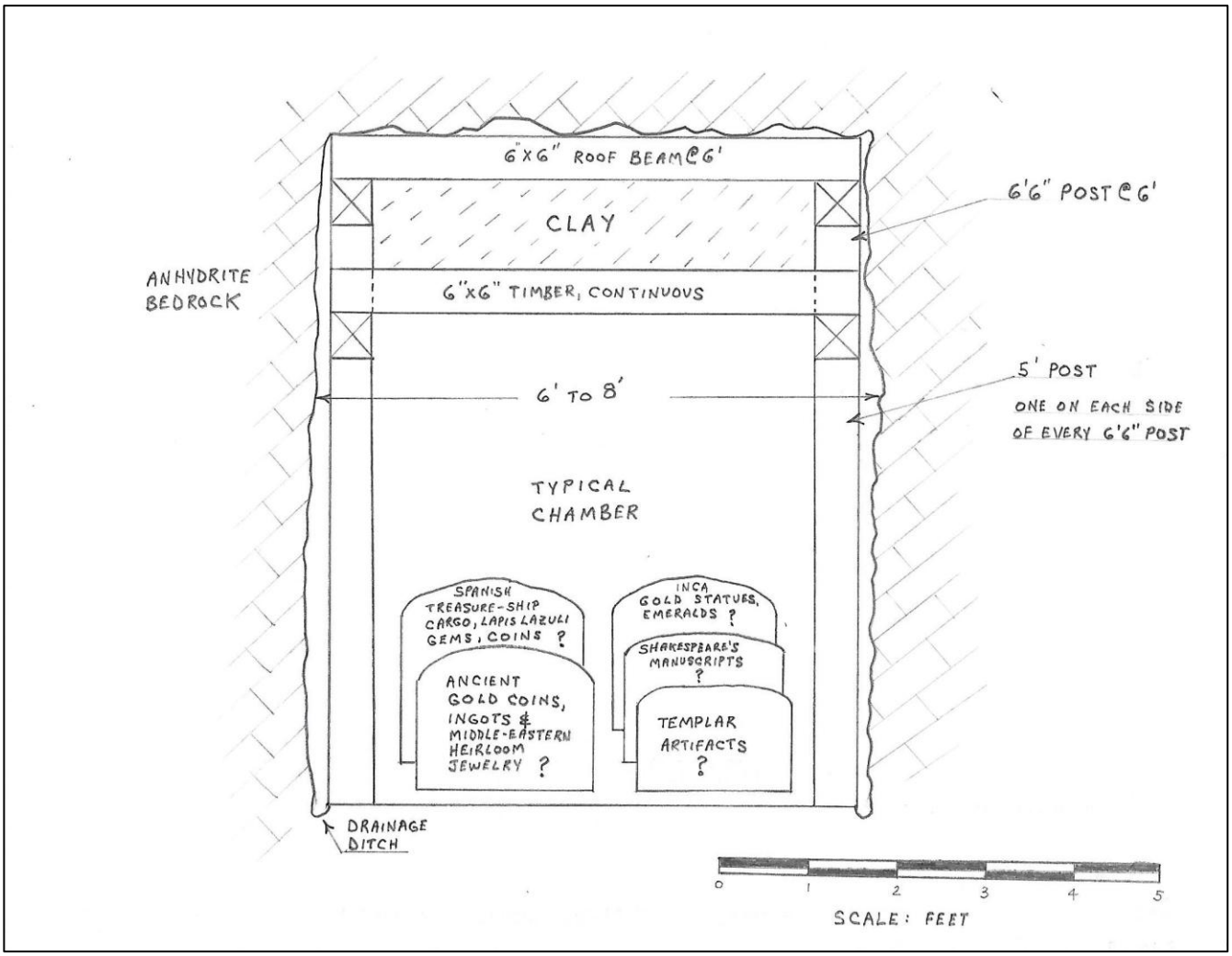

Figure 13: Authors' Concept of a Typical Cross Section through a Chamber

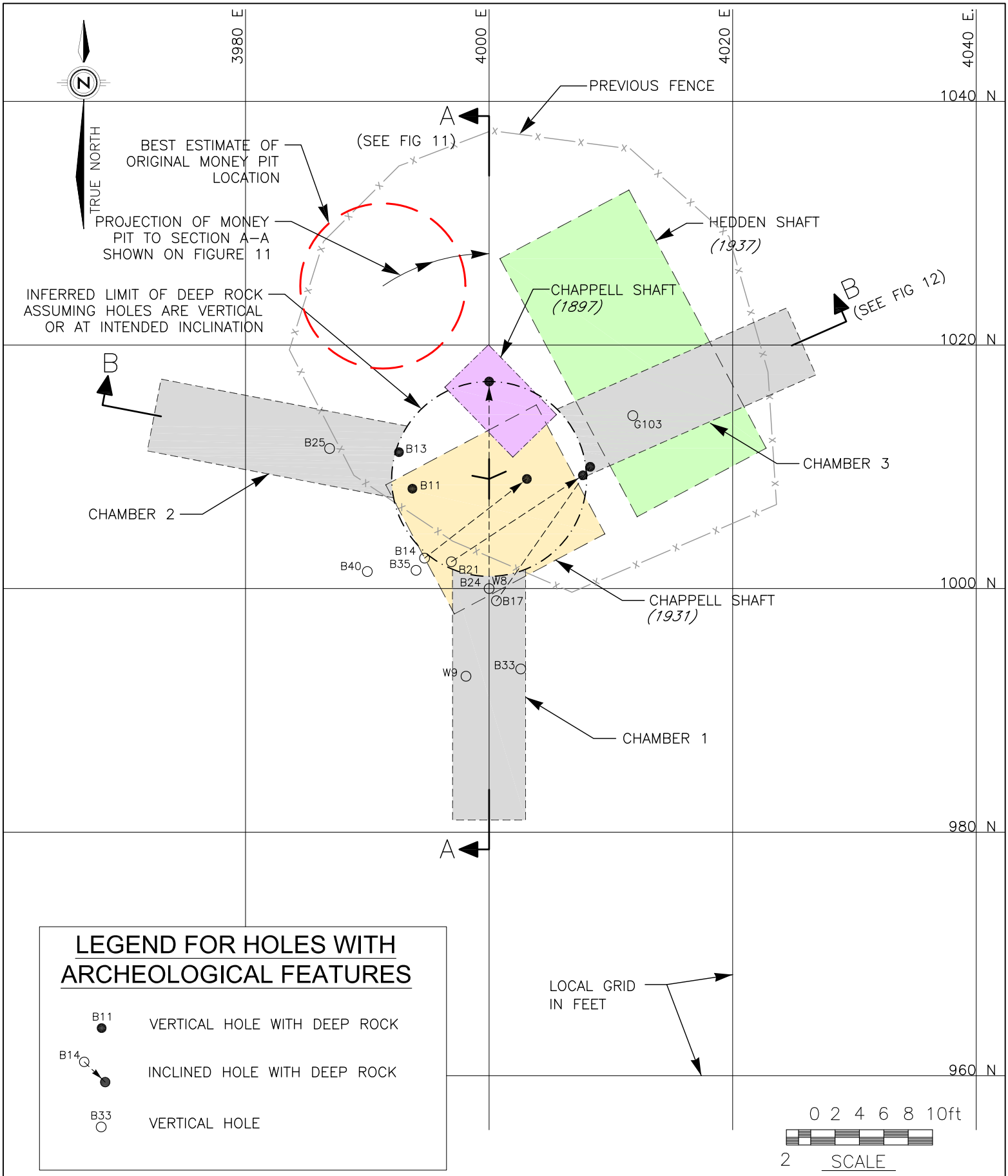

FIGURE 14: PLAN SHOWING MONEY PIT RELATIVE TO DEEP ROCK AREA AND CHAMBERS

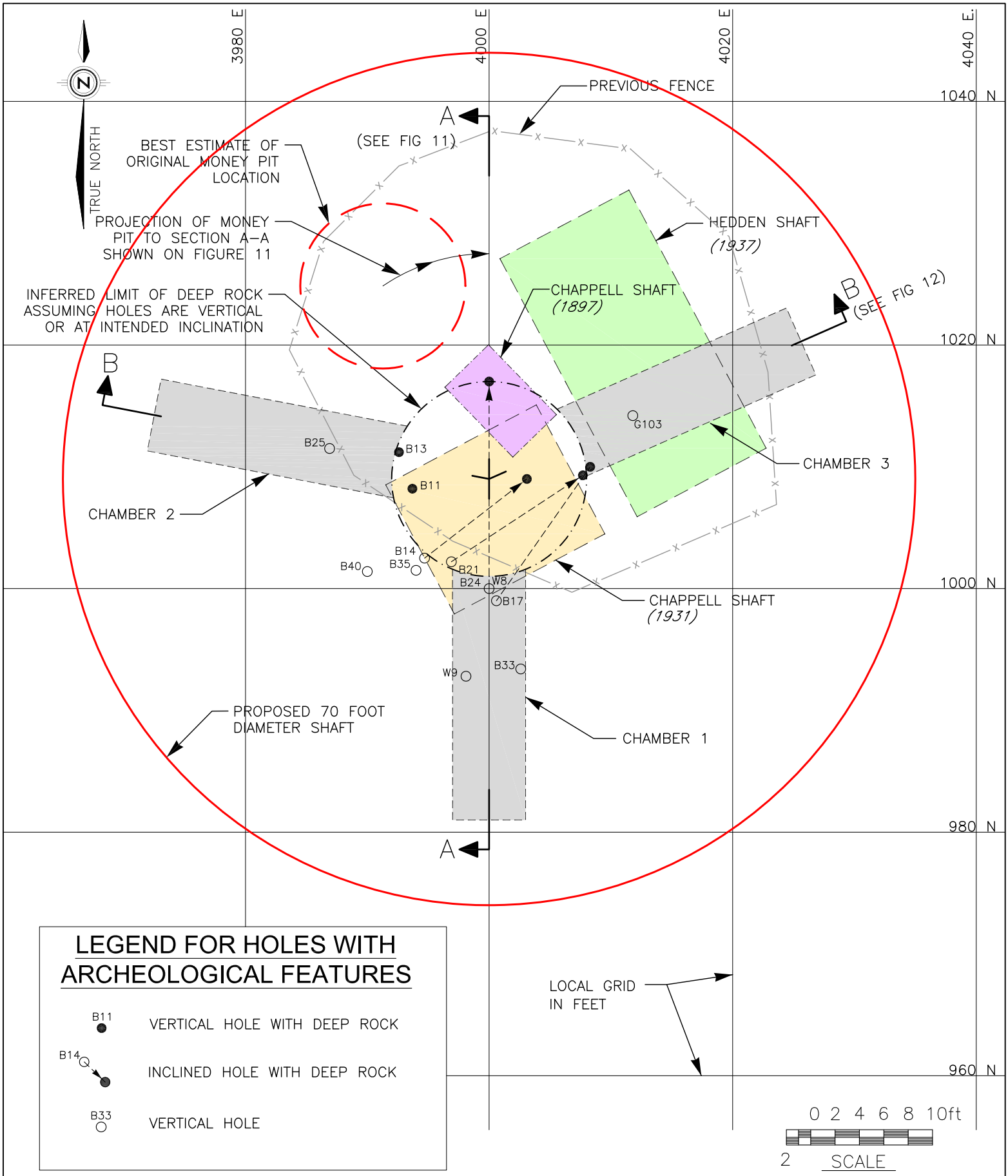

FIGURE 15: PLAN SHOWING PROPOSED LOCATION OF 70 FOOT DIAMETER SHAFT TO 200 FEET DEPTH

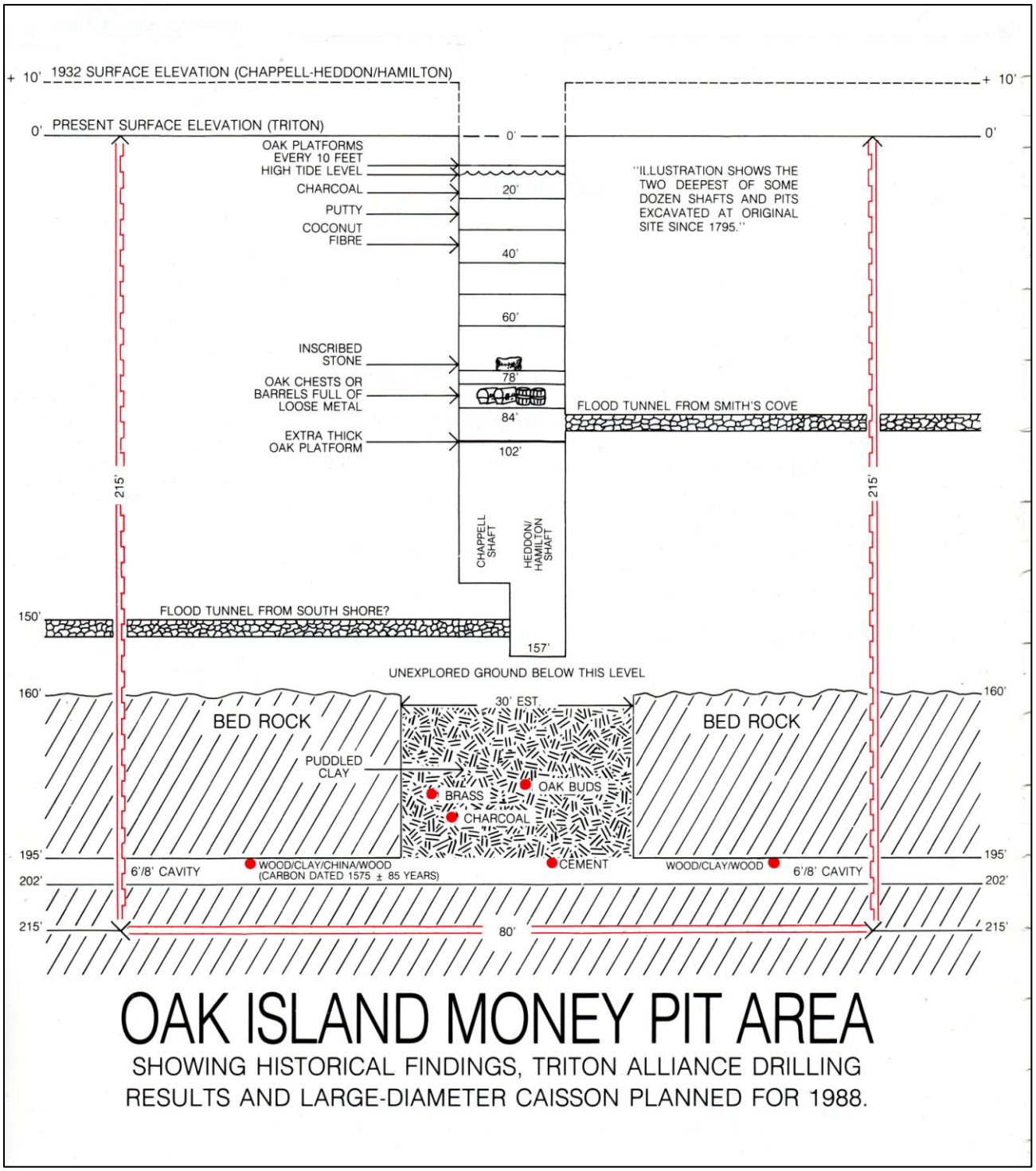

Figure 16: Schematic Section of 80 Foot Diameter Shaft Proposed by Triton Alliance in 1988