ERINDALE SUBWAY STATION

CME525

WILLIAM HALLORAN
MAHZABIN KARIM
INSHA MERCHANT
PARSA BONDAR SAHEBI

Contents

Work Breakdown

1. **William**

- Research and report for
  - Introduction
    - Project descriptions
    - Soil classification
    - Loading
    - Constraints and assumptions
  - Station Design
    - Station position
    - Station design
  - Cavern design
    - Tunnel dimensions
    - Tunnel shape
    - Tunnel influence on surface structures
    - Cavern lining
      - Shotcrete
      - Compressible material
      - Waterproof membrane
      - Cast-in-place final lining
- Plaxis
  - Tunnel shape comparison
- AutoCAD drawings for the station design
  - Plan for platform, mid-level, and top level
  - Elevations
  - Section

2. **Insha**

- Executive summary
- Citations
- Research and report for
  - Design of station and position
  - Cavern design and reinforcement
  - Cavern influence on surface structures
  - Existing infrastructure and as-built dimensions
- Plaxis
  - Model set-up
    - Properties assumptions and research
      - Soil
      - Existing infrastructure
    - Staging
  - Ko iterations and shape comparison
  - Reinforcement (rock-bolt) and spacing iterations
- AutoCAD drawings for
  - plan view for laydown and TBM route alteration
  - Distances from surrounding infrastructure

3. Mahzabin

- Research and report for
    - Design of station and position
    - Shaft Design (entire section 7 of report)
        - Shaft Shape (rectangular vs circular)
        - Shaft Support System (secant piles and shotcrete)
        - Model Set Up
        - Comparison between secant piles vs tiebacks option
    - SEM Design
        - Model Setup and Staging (6.3)
    - Construction Timeline (entire section 8 of report)
        - All research and calculations from pre-construction to the end
        - Gantt chart for timeline
    - Plaxis model inserts in Appendix E and F
- Plaxis
    - Shaft modelling of rectangular face and circular face
    - Shaft modelling with tiebacks and shotcrete
        - Asymmetric model and section view
    - Shaft modelling with secant piles and shotcrete
        - Asymmetric model and section view
        - SEM excavation modelling (Construction stages modeling for each step)
- AutoCAD drawings for
    - Mobilization layout (figure 36)

4. Parsa

- Research and report for
    - Design of station and position
    - Shaft Design
        - Shaft Shape (rectangular vs circular)
        - Shaft Support System (secant piles and shotcrete)
        - Model Set Up
    - SEM Design (entire section 6 of report)
        - SEM Face sequencing determination
        - SEM Depth sequencing determination and AutoCAD design
    - Cost Estimates (entire section 9 of report)
        - All the calculation and research regarding cost calculations
        - Final project cost summary
    - Appendix J cost calculations
- Plaxis
    - Shaft modelling of rectangular face and circular face
    - Shaft modelling with tiebacks and shotcrete
        - Asymmetric model and section view
    - Shaft modelling with secant piles and shotcrete
        - Asymmetric model and section view
    - SEM excavation modelling
        - Confinement analysis implementation for 2D model
        - Construction stages modeling for each step

1. Executive Summary

The design of Erindale subway station as part of the five-station proposed subway project in Mississauga considers a variety of risks and hazards within the planning, design, pre-construction, and construction stages. The site conditions and existing land-use of the area heavily influence the planning and decision-making, including existing infrastructure and associated loadings, as well as soil and water table conditions. The project is expected to take 582 days starting from practical pre-construction tasks until the turnover to the owner post commissioning and testing. Erindale station is expected to require 748 million dollars of funding. The major considerations and processes involved are mentioned below.

The stratigraphy of the site involves a layer of glacial till at the surface and Georgian Bay shale continuing past the depth of cavern excavation. Risks involved include high horizontal stresses as well as short- and long-term swelling. Additionally, the water table level is close to the surface at the site location due to the stormwater pond nearby, requiring extensive dewatering. Existing infrastructure considered includes a multi-story midsize apartment building to the east, residential housing, and a sewer to the west. Deformations of the infrastructure past allowable levels will require pre-work and strengthening.

The tunnel boring machine path was altered to allow for the footprint of the station to exist within the owned property area due to the constraint in working space and surrounding infrastructure. Necessary easements for this change in route are required. The access shaft will be placed in the center of the 150m station footprint, allowing the access shaft to include a concourse level leading up to the station entrance at ground level. The access shaft is to be circular in shape, reinforced with secant piles due to the increased efficiency in the distribution of stresses generated. Sequential excavation methods are to be used to excavate the access shaft and cavern tunnel, using shotcrete as the initial support layer. The cavern construction will be implemented in four stages to ensure an acceptable amount of deformation is induced.

Based on the high horizontal stresses present, the most optimal shape of cavern was determined to be an elliptical shape. Reinforcements of the cavern include hot-rolled steel rock bolts and cast-in-place concrete liner.

Based on extensive modeling in Plaxis 2D, it was determined that there will be no significant deformations expected to occur on the nearby infrastructure.

Essential construction timeline was determined based on critical path analysis. The acquiring of permits and easements were included in the timeline. mobilization, traffic management, dewatering, installation of secant piles, excavation of shaft and reinforcing was considered, with a special focus on the sequential excavation method. Productivity rates were considered to calculate the duration of the project along with the associated costs. Conservative assumptions were considered where there was lack of credible information regarding schedule duration and task costs.

After the final liner of the cavern is placed, the station will be prepared for the arrival of the TBM. A semi-ring concrete support system will be implemented to move the TBM through the station. Subsequently, the final aspect of the station's construction will continue to ensure the station has been built according to the best standards.

As such, hazards and risks were considered in the design, planning and construction of the station at Erindale and its components.

2. Introduction

Mississauga is a small city bordering Toronto, making up the western portion of the Greater Toronto Area (GTA). Due to its relation to other densely populated areas, Mississauga sees over 160,000 people commuting within and outside of the community daily. Currently, roughly 75% of this commuting is done by driving, resulting in high commute times and busy roads [1].

Due to these factors, a subway line was proposed, connecting Mississauga to other areas in the GTA. The subway line will be made up of 5 stations including Clarkson, Sheridan, UTM, Erindale and Square One. Each of the stations will be located as a key junction allowing better movement of people. The introduction of this line would better connect the region to each other and to Go-Stations. Furthermore, reducing the high reliance on driving, fostering safer travel, cleaner air, and lower emissions [2].

The focus of this report will be to present a feasible and comprehensive solution for the Erindale subway station design. This station will be directly next to the current Erindale Go-Station, which connects to the Kitchener line running East/West into the core of Toronto.

3. Project Description

The purpose of this document is to present a feasible subway design for integration into the current transportation network of Mississauga.

3.1. Site Characterization

The proposed subway station will be located within the current footprint of the Erindale Go-Station (Figure 1). This will allow the station to directly connect to the current Go network. This will also allow easier access to the excavation and permits due to the site already being owned and operated by GO. This will also allow the proposed subway station to be easily connected to the current Go station.

The proposed TBM route was routed directly underneath the parking lot of the current Go station. This allowed the parking lot to be used as the initial shaft location, reducing impacts associated with a building removal needed with another site type. The essential property easements are to be acquired prior to the planning and construction. Consideration was made after and before the tunnel for a 30m tangent length from the station.

Figure 1 – Altered TBM Route

3.2. Soil Classification

The basis of the soil classification will be done on a borehole taken from the proposed site. The borehole data is directly at the site. Additional borehole data is available for a nearby site. This data will be used for reference and comparison to confirm the validity of the borehole taken on site.

From the borehole provided, the soil was found to be primarily made of Georgian Bay shale, with some glacial till along the surface. The surrounding boreholes support this, as they are primarily made up of Georgian Bay shale. In addition to this breakdown, the surrounding borehole data includes a rock quality description (RQD) for the cores collected. Since these boreholes were close to the proposed site, they were assumed to be of similar RQD rating. On this basis, of the 38 recorded boreholes, the average RQD rating was 79.1 (Appendix A). Using this rating, and the Terzaghi rock quality designation, the soil can be found to be moderately blocky and seamy (Figure 2). This rock classification will be used in determining the proper rock support methods later in the report.

Rock condition	RQD	Rock load height Hp(m)	Remarks
1. Hard and intact	95 ~ 100	0	Light lining required only if spalling or popping occurs
2. Hard stratified or schistose	90 ~ 99	0 ~ 0.5B	Light support, mainly for protection against spalls Load may change erratically from point to point
3. Massive, moderately jointed	85 ~ 95	0 ~ 0.25B	-
4. Moderately blocky and seamy	75 ~ 85	0.25B ~ 0.20(B+Ht)	Types 4, 5, and 6 reduced by about 50% from Terzaghi value because water table has little effect on rock load(Terzaghi, 1946: Brekke 1968)
5. Very blocky and seamy	30 ~ 75	(0.20 ~ 0.60)(B+Ht)	
6. completely crushed but chemically intact	3 ~ 30	(0.60 ~ 1.10)(B+Ht)	
6a. Sand and gravel	0 ~ 3	(1.10 ~ 1.40)(B+Ht)	
7. Squeezing rock, moderate depth	NA	(1.10 ~ 2.10)(B+Ht)	Heavy side pressure invert struts required Circular ribs are recommended
8. Squeezing rock, great depth	NA	(2.10 ~ 4.50)(B+Ht)	-
9. Swelling rock	NA	Up to 250ft irrespective of value of (B+Ht)	Circular ribs required In extreme cases, use yielding support

Figure 2 – Terzaghi rock quality designations [3].

3.2.1.Surface Characteristics

The proposed site is adjacent to several important infrastructures that must be considered (Figure 3). Within 100m of the station, residential houses, apartment complexes and a parking garage are all present. There is also surface parking around the top of the station. Additionally, a sewer is present around the edge of the surface parking lot. Each aspect will be considered based on the loading they apply on the tunnel and the deformation the tunnel applies on them.

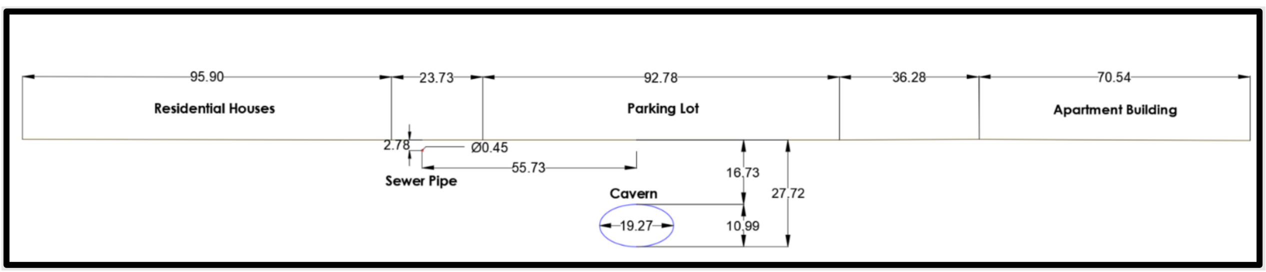


Figure 3 – Distance from surrounding infrastructure.

Another aspect of surface consideration is the presence of a large pond directly next to the excavation. This will cause large amounts of water to ingress into the tunnel and will be a major consideration during excavation and final design of the tunnel.

3.3. Loading

Due to the proposed subway station's location, the impact of surrounding buildings must be considered when looking at potential loads.

3.3.1. Apartment Building

The proposed subway station will be directly next to the current Erindale Go Station. As a result, the already built 9 storey apartment building that is close to the site must be considered with respect to surface loads. These loads will be considered and factored using the National Building Code of Canada [4] and the Ontario Building Code [5] considerations for the design of parking garages. The load was calculated assuming a percentage of the structure filled with concrete. This approach is overly conservative in its approach, assuming more concrete is there than typically expected. As a result, the final calculated load is larger than would typically be expected. This load is 206 kN/m^2 that will act on the ground from this building (Appendix B). This load will be assumed to apply this distributed load across the entire base of the structure.

3.3.2. Go Train

Due to the integration with the current Erindale Go Station, the subway station will border the current Go line. With this, specific loads from the train will be considered in the subway station's design. These loads were created by looking at average weight of a Go train and will be taken as 61 kN/m (Appendix B). The load will be assumed to apply this load along the entire length of subway track near the station.

3.3.3. Residential Houses

The location of the proposed subway station will board houses along one of its sides. These houses are all two-story low density housing. As a result, the impact of them will be small, with loads of 13.2 kN/m^2 (Appendix B). This load will be assumed to be the same for each house and apply the same distributed load across each of the house's footprints.

3.3.4. Parking Lot Loads

Due to the presence of a parking lot above the subway station. A load will be applied representing the dead weight of the cars above. This load will be taken as 363 kN/m^2 across the entire footprint of the parking lot above (Appendix B).

3.4. Constraints & Assumptions

Due to the nature of the project, there are numerous complex constraints that each will be throughout the length of the project.

3.4.1. Geotechnical Baseline Report

Since the project does not have a field aspect or team that can do site testing, there was no geotechnical baseline report (GBR) produced. This constrains the project to work with data for the soil and site conditions based solely on the single provided borehole and data for projects in the surrounding area. Without a GBR, many elements of the design must be estimated. With this, the overall accuracy of the design must be acknowledged in relation to the true conditions present beneath the surface.

The major assumption with this constraint is that the research and provided borehole data will provide a semi realistic picture of the soil conditions present. Another big assumption is that the soil characteristics that are found through research will represent the soil present.

3.4.2. Borehole Data

As mentioned, the project is constrained to limited borehole data. The usable borehole data came from borehole five (BH5) which was taken at a station beyond the site. This borehole presented a make-up comprised of Glacial till and Georgian bay shale (Figure 4). Borehole four (BH4) was taken at a station prior to the site and contained a layer of sand on top of the till and shale.

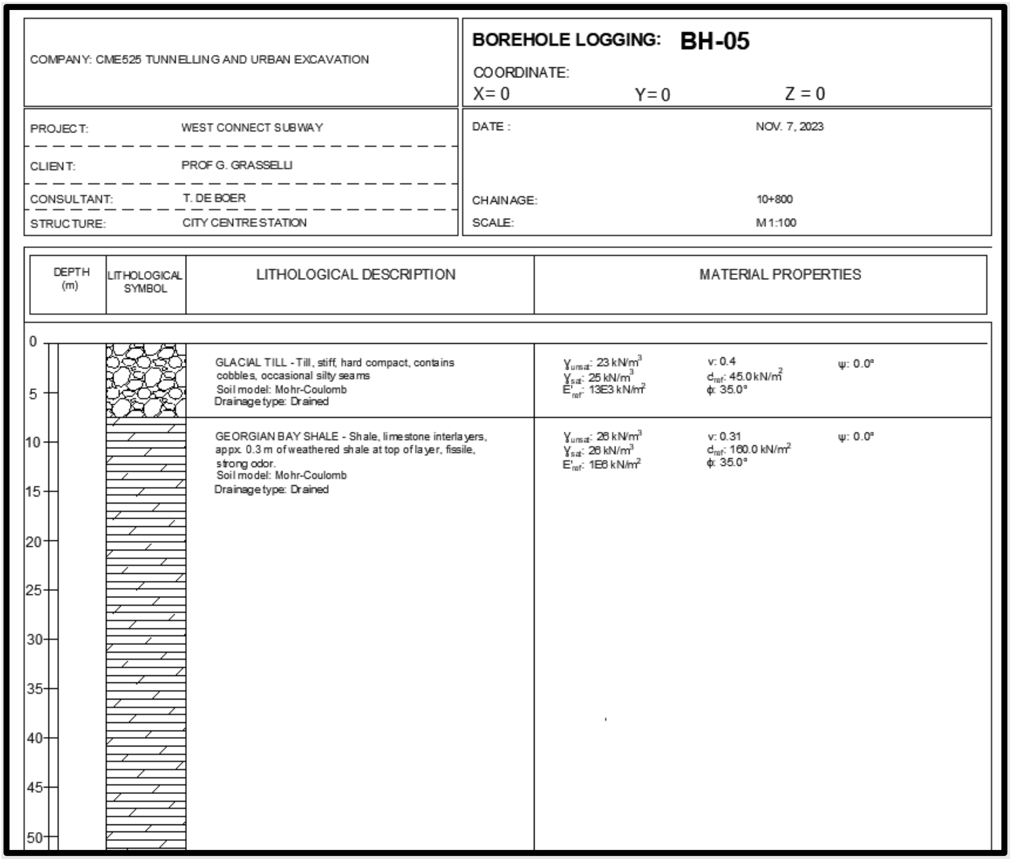


Figure 4 – Borehole 5 data.

Instead of interpolating between the two boreholes, the major assumption surrounding the borehole data was that BH5 would be sufficient in capturing the soil beneath the station. Given that the proposed site location is far away from BH4, and is in a developed area, where the shallow topsoil would typically be stripped away, the layer of sand BH4 presents would not be present at BH5. Additionally, there is a large elevation change between the

3.4.3. Glacial Till and Georgian Bay Shale Properties

The data presented by BH5 allowed curtained properties of the two soil layers to be quantified. However, the project was constrained by the properties not presented in the borehole that would be needed for analysis. The general assumption was that the unknown properties could be filled in using other examples of them. Although properties change between soil locations, a general idea of values for the unknown values would be assumed to be valid for use in analysis.

Overall till and Shale properties were taken to be the generally known values.

3.4.4. Infrastructure

3.4.4.1. Sewer

The existing housing neighborhood close to the station has a sewer running along the road. The sewer type was not specified, ie storm, sanitary, or mixed. As a result, both scenarios were considered to determine approximate depth. The City of Mississauga identifies this pipe as a concrete sewer with a diameter of 450 mm [6]. If it is a storm sewer of this diameter, depth of invert can be at a maximum of 3 m below the road surface [7]. If it is a sanitary sewer of this diameter, depth of obvert should be at a minimum of 2.5m below the road [8]. To meet both requirements, the pipe is modeled in plaxis with the obvert at 3m below ground and the resulting invert at 2.55m (obvert depth minus diameter), satisfying both storm and sanitary sewer requirements.

3.4.4.2. Residential Apartment Building

To model the large apartment complex, foundation piles were assumed to be present every 3 meters. The properties were taken to be the following: the building applies a unit weight of 203 kN/m/m on the ground surface.

3.4.4.3. Residential Houses

For the low-density residential buildings, spread footings can be used when soil conditions are ideal. There is uncertainty in design choice, but modeling piles in Plaxis may result in higher deformations which will lead to more conservative planning. Therefore, piles were used and a depth of 10m [9]due to general economical placement of building foundation depth [10].

3.4.5. Underground Infrastructure

As mentioned above, the site is surrounded by buildings and other infrastructure. The big constraint with this infrastructure is in how it is expressed below grade. Estimates can be made on the surface and its impact on the station. However, without access to the proper documentation about the utilities present below ground, the depth of foundations, historical underground structures of the area, and any other underground elements, the overall accuracy of the below grade conditions will be reduced.

The big assumption with this constraint is that the estimations for below grade conditions as stated in 3.3.4, will provide a good basis for their consideration in their design.

3.4.6. Water Table

The proposed site does have water table information. The water level is roughly 10m below the surface (Figure 5). However, there is a large pond directly next to the proposed site. As a result, the water table will be assumed to be at surface level for the entirety of the design. Water ingress will be dealt primarily with dewatering pumps. With this, it will be assumed that the water table drop associated with this level of dewatering will have minimal effects on surfaces structures.

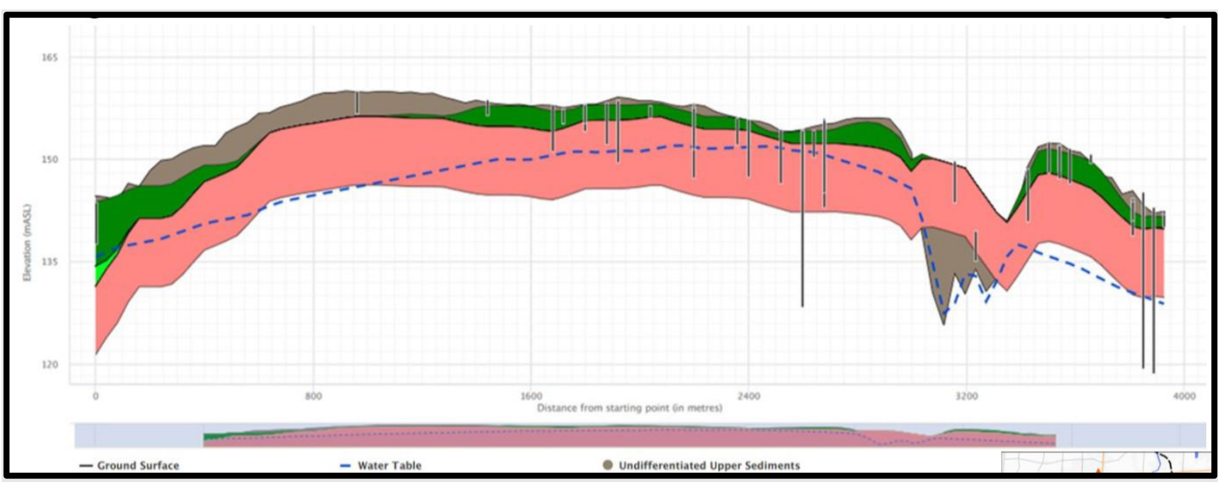


Figure 5 – Water table details.

3.4.7. Building Deformations

As previously mentioned, the proposed site is surrounded by a large parking structure, apartment structure, residential houses, roadways, and a Go train line. As a result, deformation will be kept within an acceptable range for each. Deformation control will be the biggest constraint for later analysis due to its impact on safety. Due to potential structural failure from large cracks in the foundation, deformation control is an important constraint.

It will be assumed for the basis of design, that any deformations below these limits will be acceptable and above these limits proper deformation mitigation methods will be provided. The limit will be 6.66 mm for differential settlements and 25mm for total settlements (Table 1). Differential settlements refer to the difference in maximum and minimum deformations along foundation edges. Total settlements refer to total sinking that takes place at the foundations.

Table 1 – Deformation Limits.

Deformation Element	Value
Differential Settlements	6.66 mm [11]
Total Settlements	25 mm [12]

3.4.8. Deformations of Go Train Rails

The proposed station will be built within the current footprint of the already existing Erindale Go station. With this, there will be a TBM tunnel dug directly underneath the current Go transit rail systems. As a result, the major constraint will be for the design of the subway station to not impact the train tracks in the form of deformations. https://nrc-publications.canada.ca/eng/view/ft/?id=515340b5-f4e0-4798-be69-692e4ec423e8

It will be assumed that deformation less than the deformation limit of 25mm, will be acceptable for the track. Any deformations below this will not affect operational safety of the track. Inversely, for any deformations above this threshold specific solutions will be presented.

3.4.9.Swelling Loads

As shown in the borehole data, the location of the proposed station is made up primarily of Georgian Bay shale. This is a major constraint, as this soil is susceptible to large amounts of swelling. This swelling is present in both the horizontal and vertical directions of up to 0.34%/log cycle horizontally, and 1.4%/ log cycle vertically [13]. As such, the swelling has the potential to produce large squeezing stresses around the tunnel if not properly addressed.

To address this, as discussed in lining design, a compressible material will be present. The assumption around this is that the material will be able to take the entire swelling load and that no additional loading will be necessary to model.

3.4.10. Impact on Go Network

As mentioned in 3.3.7, the construction of the station will be directly next to the current Erindale Go station. As a result, construction must be constraint to comply with Metrolinx work permit requirements for work done near tracks.

Special considerations will take place to ensure that construction effects will not prevent regularly scheduled service from running in and out of the station. It will also be assumed that since the project will be done on Metrolinx land, they will be the eventual owner of the project. As a result, surrounding integration between the two stations in the form of foot bridges and building connections will be available with no extra considerations for third party permits.

3.4.11. TBM vs shaft timing

A large initial constraint of both design and construction of the tunnel was the presence of TBM tunnels. With a TBM tunnel already present, the excavation would have to remove the TBM liners and excavate from the tunnels. Under these considerations, both of the TBM tunnels would be completely excavated and all deformations would have taken place.

However, this tunnel was initially assumed to not have been dug. Instead, all excavation will take place from the shaft down to the bottom of the depth of the tunnel lining. Both footprints of the tunnel will be removed during this method. Once fully excavated, the TBMs will enter the empty passage. Special considerations will be noted on how to move the TBM through the empty cavern. Additional reinforcement will be added around the cavern face around where the TBM will drill through.

4. Overall Design

The overall design of the station will be based upon land available along with design considerations as stated in the Metrolinx design guides. The basis for the design will be to create an underground cavern for the platform, and a vertical portion for the actual station.

4.1. Station Position Based on Access Shaft Position

The station will be located directly below the access shaft for several reasons, and the shaft position itself was determined by a variety of considerations, including the path of the TBM. The team chose to re-route the TBM to allow for simplicity and ease of construction (Figure 6).

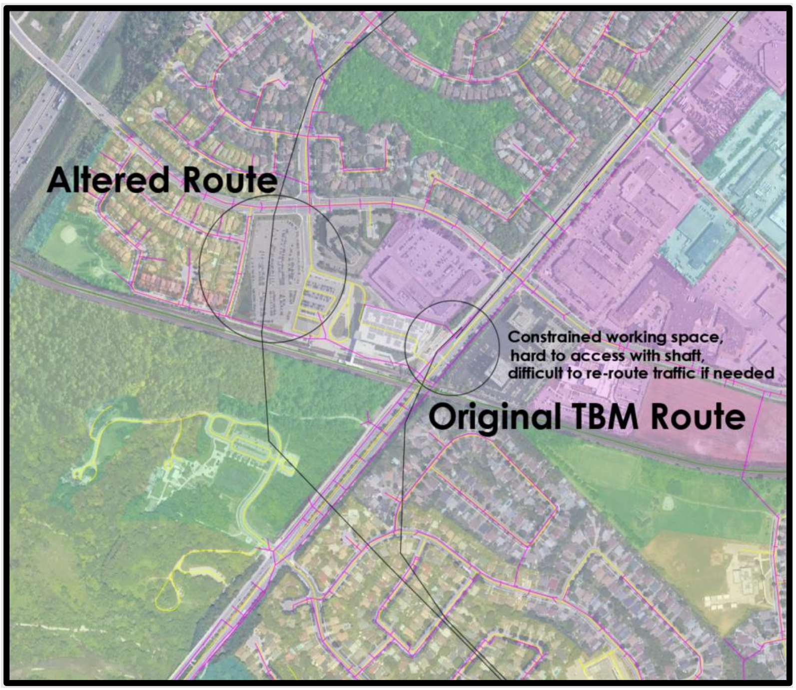


Figure 6 – Original versus altered TBM route

To begin, constructing an access shaft from owned property (Go parking lot) to the main road would have been challenging. The multistory Go parking lot building foundation may have been negatively impacted and added to the existing complexity of creating a suitable size access shaft. Additionally, creating station entrances may have required the shutting down of the main road for some time. This road is difficult to re-route given the geography (river and green space nearby) and may have blocked off access to vital areas, adversely impacting Go bus routes and the local residents. The working space on this road is also very limited and coordination of resources and equipment may have been difficult. To prevent such complexities, the team decided that obtaining easements and re-routing the TBM would be a better option. This would allow for the station and shaft footprint to overlap in the owned parking lot. This would not only simplify the cavern design but allow for the re-purposing of the access shaft as a concourse level. This allows for the cavern and SEM excavation itself to be a single floor with stairs leading into the shaft.

Moving forward, the shaft will refer to the large vertical excavation taken to the underground platform. The cavern will refer to the large underground horizontal portion of the station. The overall subway station will be designed to fit within the excavation shaft. As such, the excavation shaft will be heavily dependent on space requirements as set out in the station design. The biggest of these considerations were based on fire exit requirement set by OBC [5]. These state that stairs at the platform level should be available every 25m from any one egress point. This aspect combined with the 5m clearance required in front of stairs created a shaft that was roughly 34.1m in diameter.

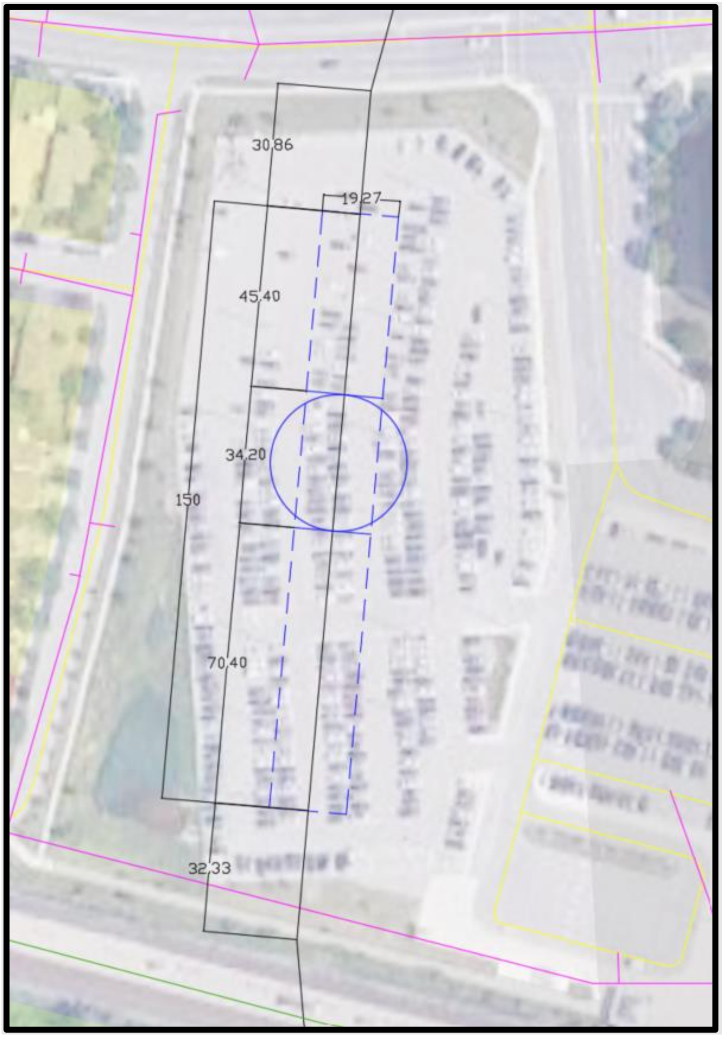


Figure 7 – Shaft and station location

Two options were considered for where to locate the shaft within the footprint of the cavern. Since the cavern is much larger than the shaft, the shaft was capable of being placed along the entire length of the cavern. However, placing the shaft at the end and center of the cavern was considered initially. The final design was chosen for construction related reasons and based on the minimum distances of 25m as stated above. With both considerations, a more central shaft was chosen, that was roughly 70.40m from the lower side (Figure 7).

4.2. Station Design

The design will be made up of 6 key elements [14] which include:

1) Entrance
2) Transaction zone/ Fare Gates

3) Paid access zone
4) Vertical circulation
5) Platform
6) Back of house

4.2.1.Entrance [14,15]

The upper portion of the subway will be based around three entrances. These entrances will each have 3m of clearance into the station. Along this top floor, there will be payment kiosks for ticket purchases. These kiosks will provide a queuing clear distance of 1400mm in front of them. Additionally, a personal assistance intercom (PAI) will be provided at this level. This intercom can be used for assistance from transit personnel. A 2000mm x 2000mm clear space will extend out from this intercom. A large part of this floor will be open to the floor below. This will improve aesthetic criteria and will improve overall air circulation from the platform level.

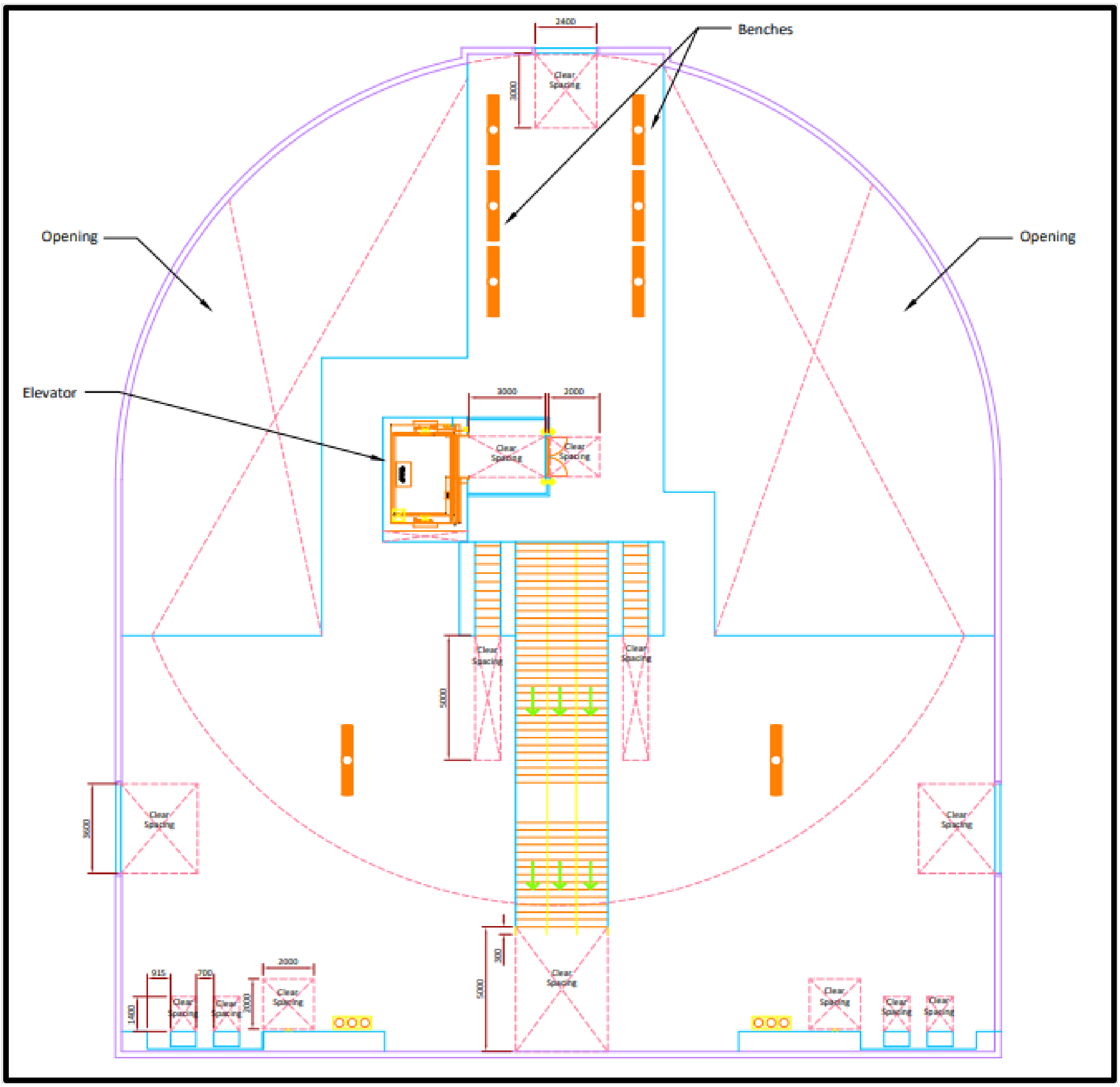


Figure 8 – EL. 0.000m: Top floor layout.

The entrance will connect to another floor above the ground level, directly connecting the station to the Go train platform via a pedestrian bridge. The bridge will allow riders to have an enclosed connection between the subway and Go train. At the ground level entrance, the elevator and an additional staircase will lead up to this. A bridge will connect this floor to the Go platform via a set of stairs and another elevator on the platform side.

4.2.2. Transaction Zone and Fare Gates

The transaction zone will be on the middle floor primarily. On this floor there will be six fare gates of 800mm width present. This width is larger than the minimum 600mm required of the faire gates. There will be one additional gate with a 1500mm width for both accessibility and surge flows. In front of the fare gates, a 5000mm waiting zone will be present. This minimum space is to allow for queuing space on both sides of the fare gates.

Along the edge of the fare gates, an ambassador’s office will be present. This office will be used for service needs and will house subway station workers during the day. As such, additional space will be provided for a backroom that will be used as a kitchen, and break room. To meet accessibility requirements, 1500mm minimum distance within the office will be provided for ease of movement within the office. Outside of the office, a clear space of 2000mm by 2000mm will be provided to allow customers to communicate with workers within the office. This space will be provided on the unpaid side of the station to ensure needs surrounding fare payment can be met.

Since the elevator will take users from the top floor and users will not enter through the main fair gates, a secondary fare gate will be added at the top floor services only the elevator. The appropriate size and clear spacings will be applied to this gate.

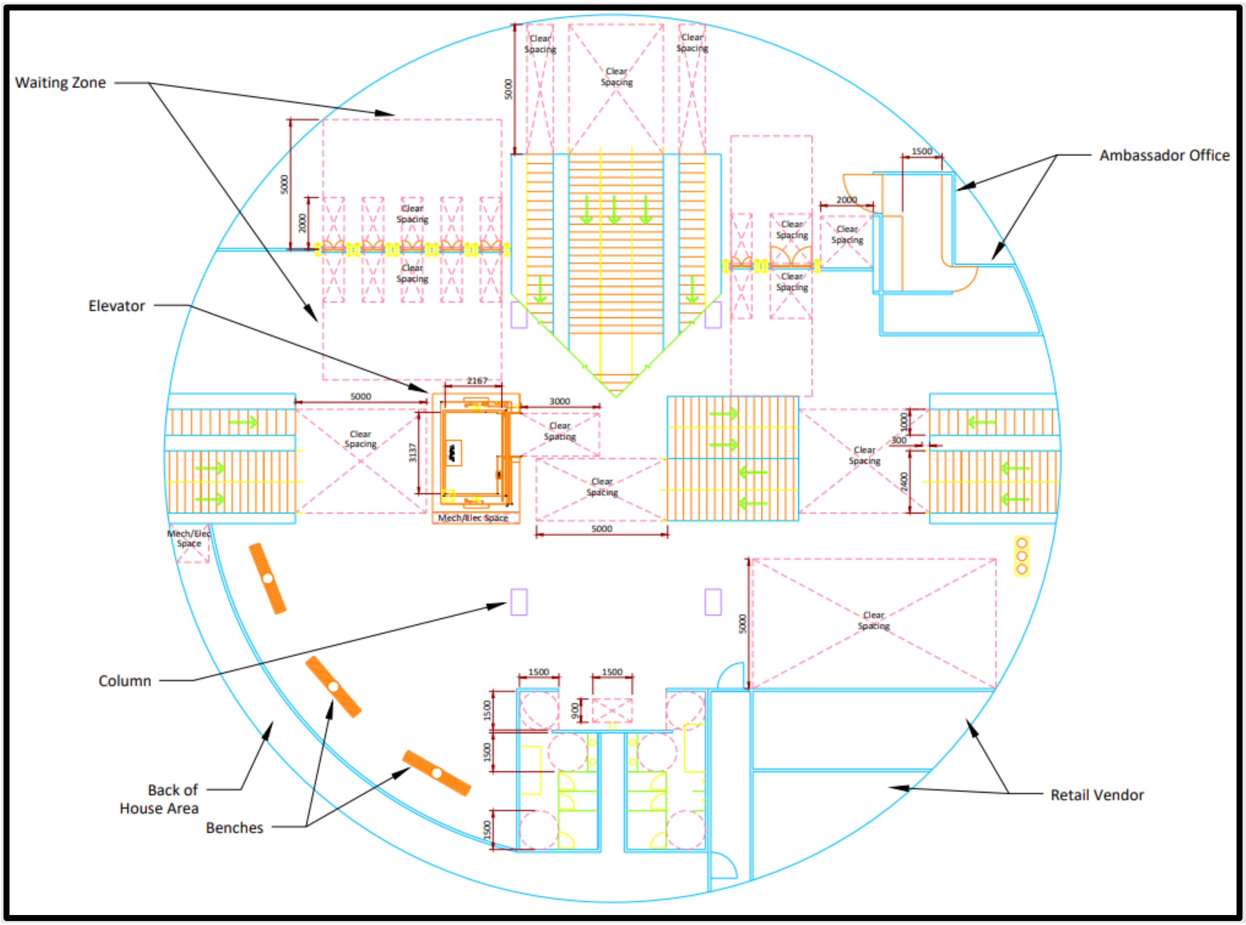


Figure 9 – El. -13.887m: Middle floor layout.

4.2.3. Paid Access Zone

Paid access zones will give users access to the transit network along with station amenities. These amenities will include a retail area and bathrooms. The retail area will have a clear area away from vertical circulation areas of 5000mm. At this distance, the retail is away from all specified elements, including platform edges, edge of stairs or escalators, and ambassador hubs. The retail will also have minimum requirements for shop size, and storage of 3050mm x 3050mm, and 2440mm x 2440mm respectively. The footprint of the proposed retail zone will exceed both minimum areas. Additionally, a garbage can will be provided within 3000mm of the retail space.

A set of male and female bathrooms will be provided. These bathrooms will provide minimum space of 1500mm clear width at the doors and along the width of the bathroom. Within the bathroom, a changing station will be provided, and will be assumed to be folded up, to allow for proper clear distances. A clear distance of 900mm will be provided between the two-bathroom walls, to ensure plumbing can be accessed and maintained easily. The bathrooms will have a drinking fountain outside of their footprint that will have a 900mm x 1500mm minimum clear distance directly in front of it.

4.2.4. Vertical Circulation

Vertical circulation will take place in 3 ways. The first of these will be direct stair access. From the surface 4 flights of stairs will take users from the surface station to the second floor. Once passed through, 3 more flights of stairs will take users to the platform. These stairs will be designed using the minimum clearance of 2400mm between the edges of the stairs. For the entire duration of the stairs, along the edge and middle of the stair, handrails will be provided at spacing of 550mm and 915mm from the ground. These handrails will extend 300mm past the boundary of the stairs. These are required for stairs exceeding 2200mm in size. The actual size of the stairs used will be based on average values as stated in table 2.

Table 2 – Stair parameters.

Stair Characteristic	Value
Height	155mm
Depth	300mm
Stair Width	2400mm
Edge Tred	50mm

Although there are many flights of stairs, the maximum distance vertically that stairs will go will be 3.65m. This value is based on the maximum distance available before a landing must be installed. Landings will be provided between stair flights and will be of a minimum size of 1600mm. Where stairs end and begin, 5000mm approach will be provided to allow for surge capacity of the stair.

Alongside the stairs, escalators will be provided from surface level down to the platform level. The size of the escalator used will be based on values as stated in table 3. There will be a clear width of 1000mm to allow for safe usage.

Table 3 – Stair parameters.

Stair Characteristic	Value
Height	155mm
Depth	300mm
Clear Width	1000mm

Like the stairs, a surge distance of 5000mm will be provided at the beginning and ending of the escalator. The escalator will be used in both directions and will be changed manually by a trained person. The directional change will be made to reflect the direction of flow. Handrails will be provided along the edge of the escalator at a height of 900mm from the bottom of the escalator.

In addition to both methods, an elevator will be provided in the center of the station that will allow users to be taken down to platform level. This elevator will be installed to accommodate accessibility requirements throughout the station. The elevator will have a clear width within the elevation of 2000mm x 2000mm minimum. As mentioned, the entrance to this elevator will have a personal fare gate. This allows the elevator to service each floor and meet accessibility requirements. The elevator will have a surge distance of 3000mm in front of its doors at each level.

Vertical circulation will also be provided at the platform level by a fire exit. This will not be for regular use, but instead under fire conditions. This fire exit will consist of a ring of stairs that takes users from the platform level to the surface level.

4.2.5. Platform

The platform will be created as an island in between two subway cars. This was done to reduce space within the cavern and consolidate passageways. From this island, egress will take place along the length of the tunnel with stairs being available ever 25m from any one egress point. As a result, there will be 3 widely used vertical circulation methods and one available fire exit from the platform. The overall length of the platform will be 150m. This will allow the typical length of a subway which is 139.14m [16], to stop at the platform. There will be additional space at the front and end of the train to allow for some mismatch between stoppage and the location of the platform.

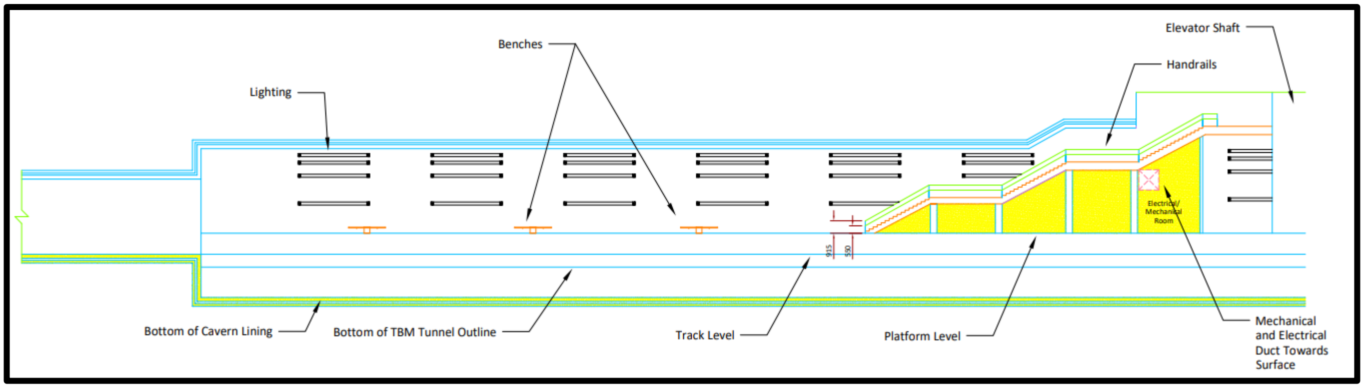


Figure 10 – Platform elevation.

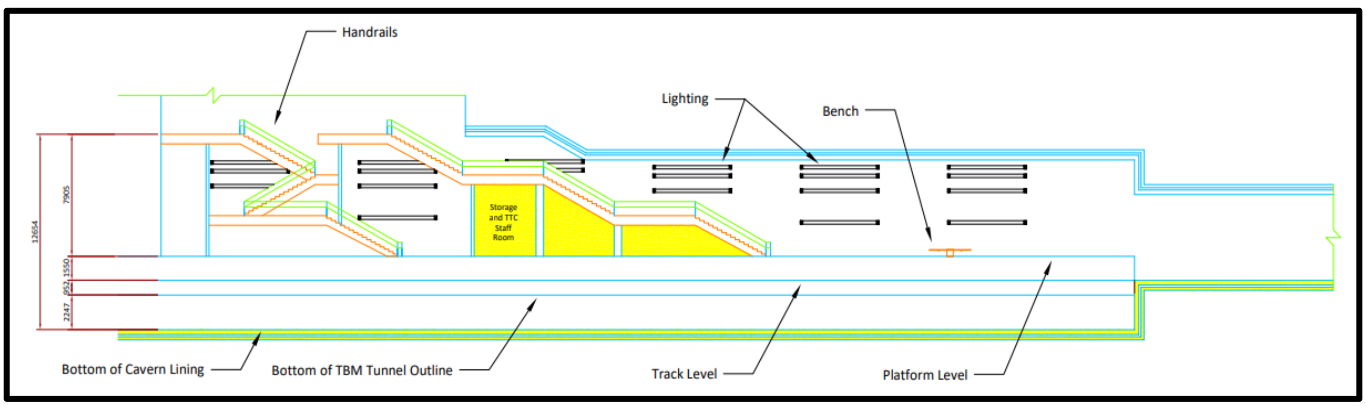


Figure 11 – Platform elevation.

As shown in Figure 10 and Figure 11, the platform has many elements of elevation. The elevation at the platform level is broken up into 3 parts. First the platform, which is located 7.905m below the middle floor, and 21.792 m below the surface. The track level is located 1.5m below this. The invert is then filled with concrete until the bottom of the lining at a final elevation of 26.541m below the surface.

The platform was primarily designed using minimum clearances from the Metrolinx Design Manuals and Ontario Building Code. Using elements such as platform edge, stair width, escalator width, required gaps and other miscellaneous items, a minimum width of 10m was calculated and used as a preliminary design width for analysis (table 4). This platform plus the distance to the edge of the TBM tunnel will make up the width of the shaft. A clear distance of 2500mm will be kept between the platform edge and any island walls or items.

Table 4 – Minimum widths of specific elements.

Element	Minimum Width (mm)
Platform Edge x 2	2500
Stairs	2400
Escalator	1000
Gap between Stairs and Escalator	600
Railings, separations, miscellaneous factors	1000*
Total Minimum Platform Width	10000
Minimum width of island platform	6400

* Minimum width was based on approximate widths for items not explicitly listed.

At the platform level, a designated waiting area will be provided that will hold benches every 1250mm. This area will allow users to wait safely for trains.

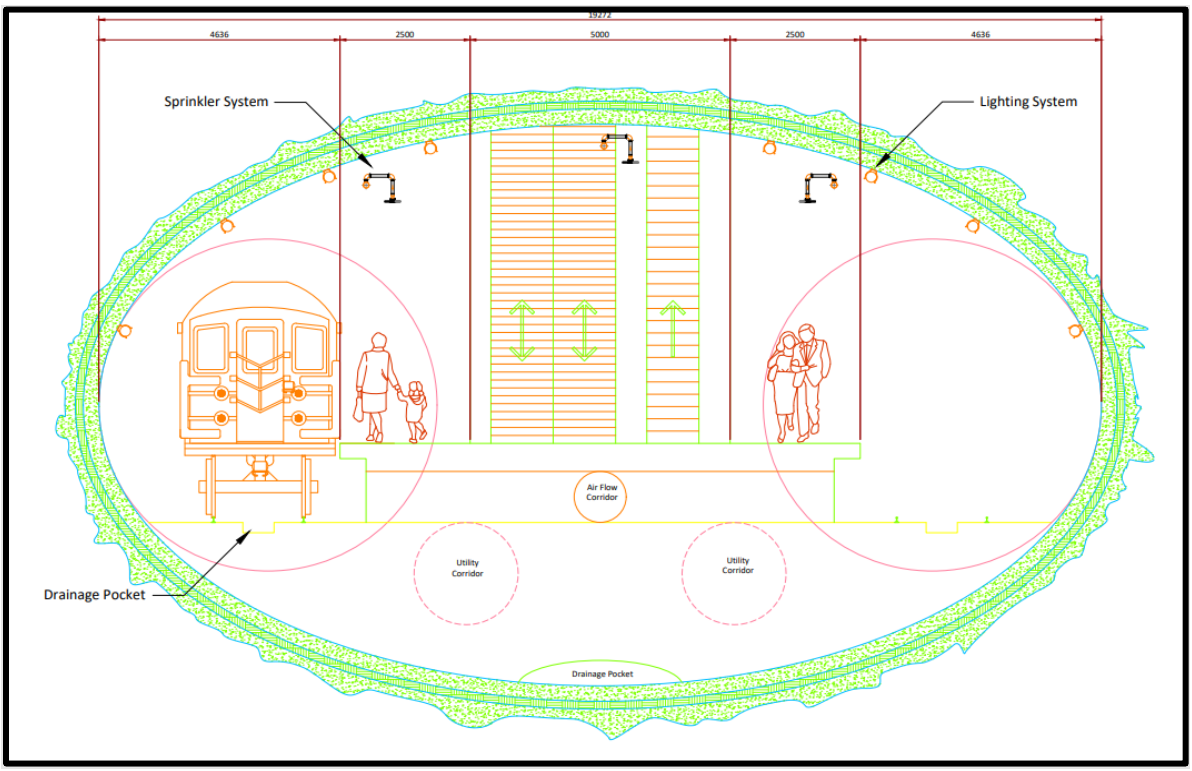


Figure 12 – Shaft Section.

4.2.6. Back of House

Due to the complex nature of the subway, there are many electrical and mechanical considerations in their relation to the overall shaft. At platform level there are many aspects that will be included.

The first of these is drainage along the bottom of the tunnel. There will be drainage areas at the invert that will collect pooled water. This water will come in from the waterproof membrane and collect along the bottom. Any leaks in the membrane will be collected here. Using sump pumps, the water will then be pumped to the shaft through the mechanical and electrical room passages provided. These drains will also be present underneath the subway trains to collect water in a similar way.

Due to the large area of clear concrete underneath the platform, utility tunnels will be included. These tunnels will allow electrical and mechanical infrastructure to run under the tunnel. Additionally, underneath the tunnel there will be air flow passages. This air flow will circulate air from the tunnel into the mechanical ducts where air can be scrubbed or removed from the site. This consideration was included to address air quality within the tunnel.

Lighting will be provided at each level. The lighting will line the top of the tunnel and should be designed to illuminate the entire underground portions fully. This lighting will be used at both the platform level and on the middle and top floors. This lighting will run the entire length of the platform. The lighting design will change when entering the subway tunnels to accommodate TTC personnel optimal lighting ranges.

Electrical and mechanical considerations will be addressed in the electrical and mechanical room present at the platform level. This space will be assumed to sufficiently accommodate all the needs of both systems. The wires and mechanical equipment exist above the platform edge at least 2500mm up and will run up towards the middle floor empty space (Figure 13). Along the bottom edge of the middle floor, appropriate space has been provided to facilitate the movement of these items from the platform room to the surface (Figure 9). It will be assumed the electrical and mechanical equipment will come up just below the surface and fed into buried infrastructure already in place.

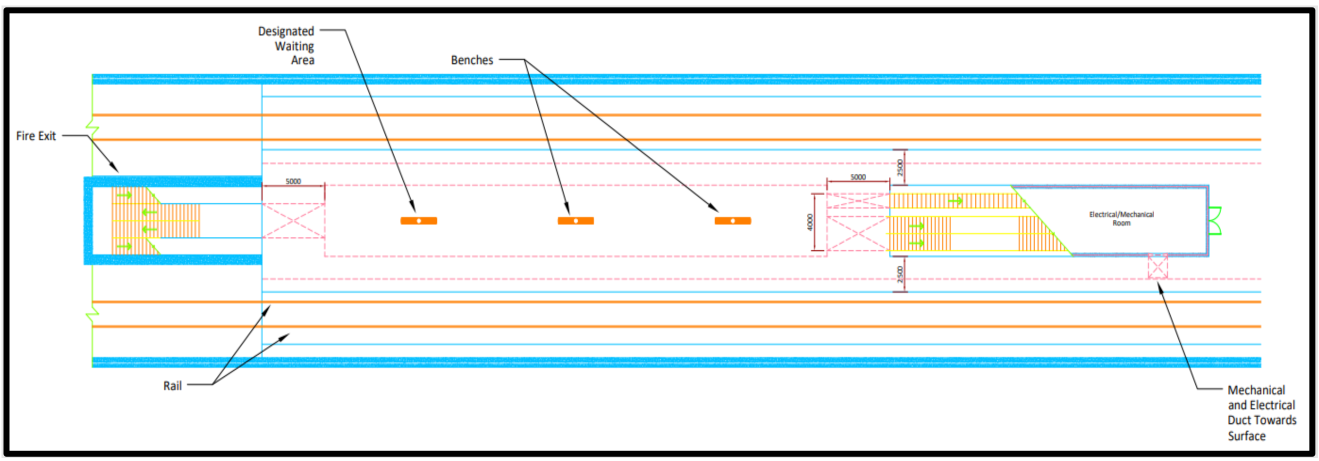


Figure 13 – Plan view of platform.

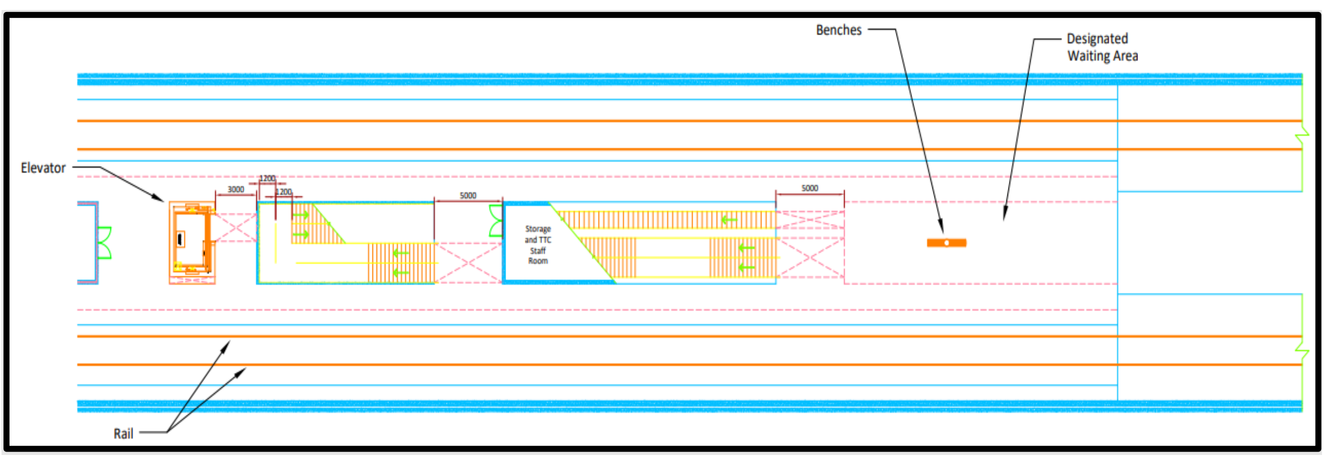


Figure 14 – Plan view of platform.

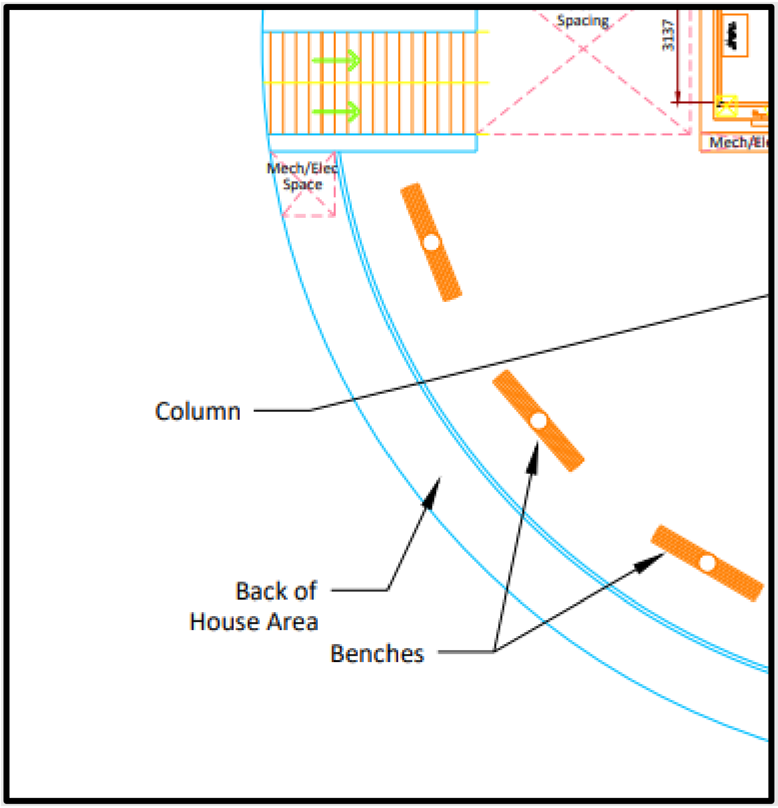


Figure 15 – View of mechanical and electrical space on main floor.

To address fire requirements two aspects are considered. The first as stated in the platform design, maximum egress points were considered for stairs. These egress points are designated by the fire code, to prevent passengers from travelling more than 25m in case of a fire. The second is the inclusion of sprinkler systems throughout the station. These sprinklers will be provided at regular intervals to ensure the entire station is fed with water in the event of a fire. These will be included at both the platform level and on the middle and top floor of the station.

5. Cavern Design

Based on the shaft and station design considerations, the cavern was determined to have a single level at which the trains arrive, as opposed to a second concourse level built into the cavern. The basis for analysis was done using Plaxis 2-D. This program was used to model how the excavation of the shaft would interact with the surrounding soil. Using the outputs from Plaxis, the size of the tunnel many elements of design were determined. This was done by modelling alternatives and comparing them to determine the optimal shape, reinforcement, etc.

5.1. Tunnel Dimensions

Tunnel dimensions were determined initially based on the TBM profiles provided (Figure 16). The TBM tunnels were measured. The first aspect of tunnel dimensions was determining the width of the platform. As mentioned in 4.2.2.5, the platform was determined to be 10m wide. There will also be a 0.076m gap between the edge of the platform and the train. From this a half a train will be taken at 1.493m, with the total train width at 2.986m [17]. The distance from the edge of the tarin to the wall of the cavern will be 3.067m This leads to a final width for half the cavern of 9.636m. The total width will then be 19.272m

Table 5 – Tunnel dimensions.

Element	Distance
½ Platform	5m
Gap	0.076m
½ Train Width	1.493m
Width	3.067m
½ Cavern Width	9.363m
Full Cavern Width	19.272m

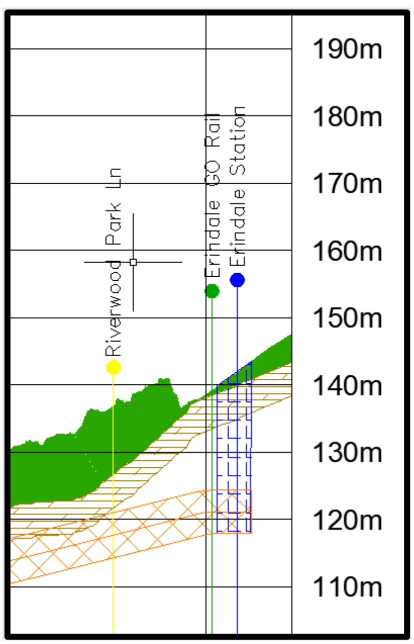


Figure 16 – Initial profile.

The height of the excavation was based on the initial profile which showed the TBM center dug 22.223m below the surface. Using the width, a shape was determined initially for the tunnel. After the shape analysis it was determined to be 5.2m. The total depth was determined as 5.497m below the center of the TBM tunnel. This puts the total depth at 27.72m below the surface.

5.2. Tunnel Shape

The initial analysis was conducted on two different cavern shapes. One shape was more elliptical, while another was more circular (Figure 17 & 18). Both shapes were analyzed using Plaxis, to check which would provide the best stress distribution around the tunnel. The width of the two shapes was not changed, however, the height was changed to reflect different tangent angles.

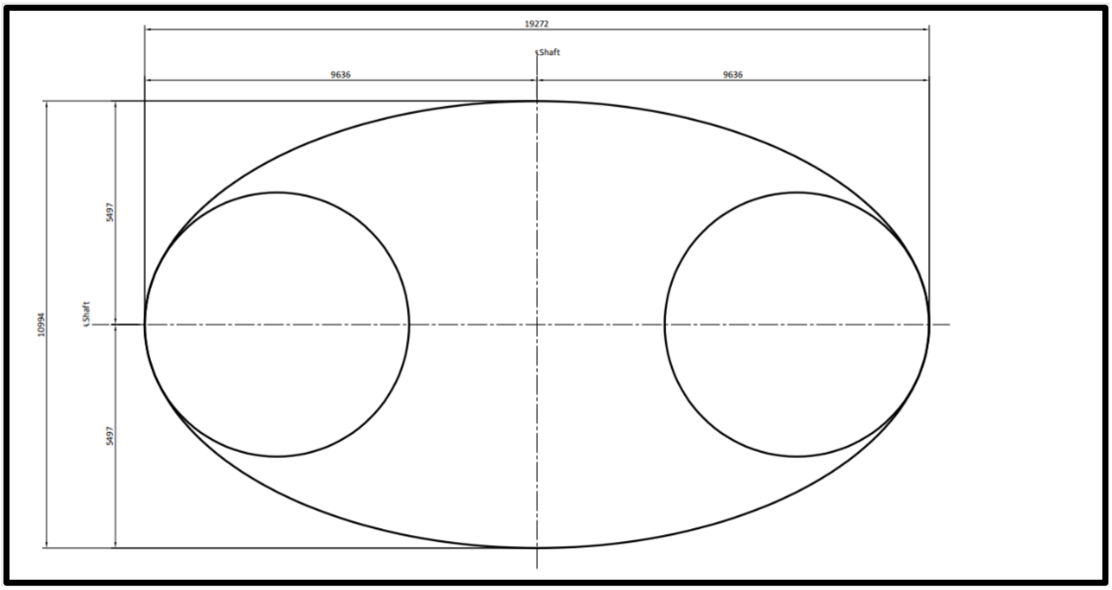


Figure 17 – Tunnel shaft alternative 1.

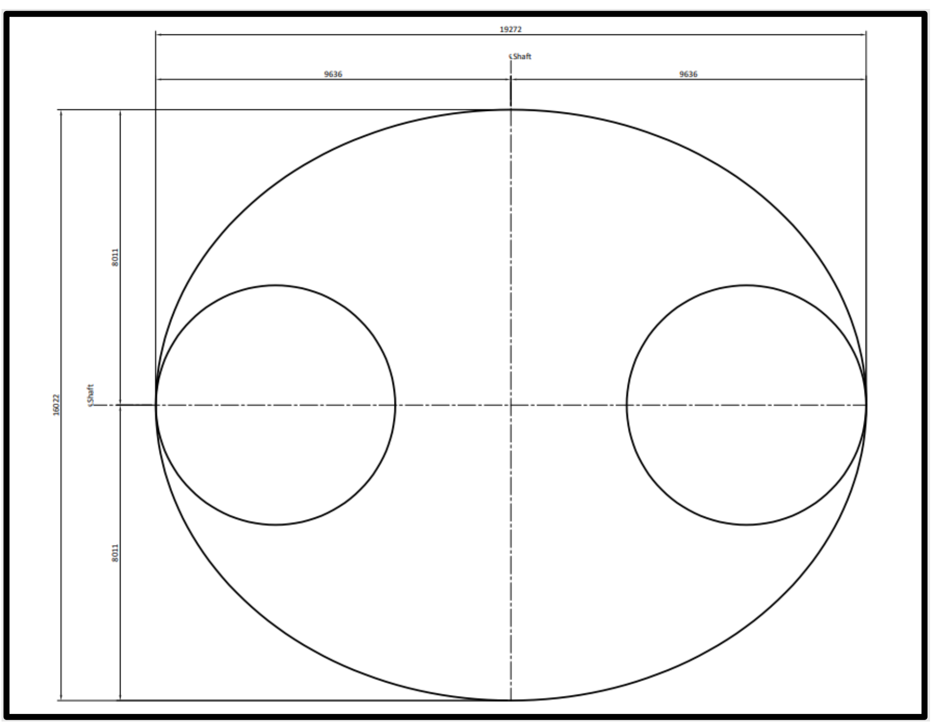


Figure 18 – Tunnel shaft alternative 2.

Modeling in Plaxis was conducted to determine the impact of the geology and existing infrastructure on the two different cavern shapes.

5.2.1. Model Set Up

As mentioned in the project description, several existing infrastructure element properties were assumed based on research and common practice to model the impact of these elements on the cavern. For instance, foundations are modelled below frost depth for large buildings and residential buildings are assumed to use shallow spread foundations. See the table below for the inputs entered into Plaxis:

Table 6 – Infrastructure properties.

Element	Material	Properties						
		Unit Weight		Axial Stiffness (KN/m)	Bending Stiffness (KNm2/m)	Stiffness (KN/m2)	Poisson's Ratio	Axial Skin Resistance
Sewer	**Concrete Properties [18]**	8.4	KN/m/m	1.40E+07	1.43E+05	-	0.15	-
Building	**Piles [9]**	7	KN/m3	-	-	1.00E+10	0	Linear, Tmin 1, Tmax 100
	Plate	203	KN/m/m	1.00E+10	1.00E+10	-	0	-
Houses	**Plate**	13.2	KN/m/m	1.00E+10	1.00E+10	-	0	-
Cavern Liner	**Concrete Cast in Place Liner**	22.56	KN/m/m	4.40E+08	7.96E+09	-	0.15	-

The base model in Plaxis is as follows:

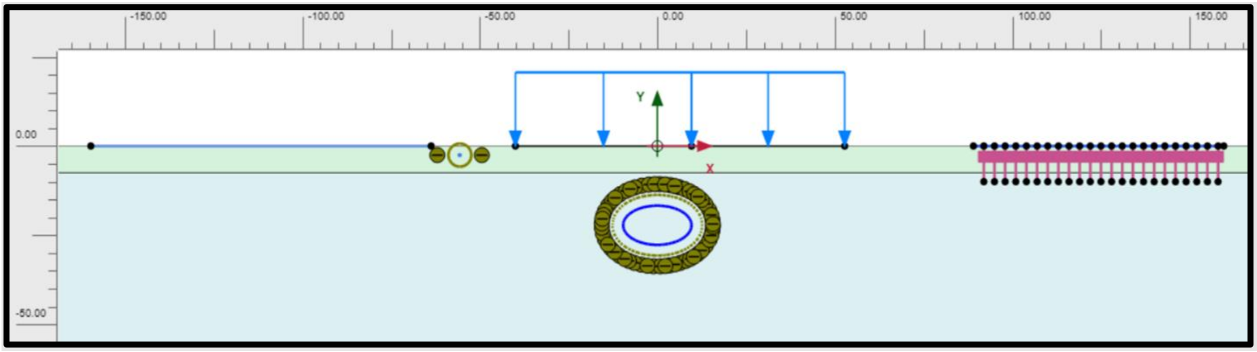


Figure 19 – Layout of Plaxis model.

5.2.2. Staging

The goal of modeling the loads and cavern in Plaxis was to determine the basic feasibility of the shape at the depth required and to get a better understanding of the behavior of the cavern and impact on surrounding infrastructure. In order to simplify the modeling, a basic staging method was conducted.

1. Stage 1: Initial, no elements activated
2. Stage 2: Existing infrastructure activated
3. Stage 3: Cavern excavated and liner activated in one step to model the primary support (as opposed to the shotcrete that would be applied during SEM construction)

Although the final stage is not representative of the way the shale is excavated, the model provides a basic understanding in order to conduct the analyses mentioned below and determine reinforcements needed.

5.2.3.Shape Analysis

Based on the analysis of the Plaxis results key findings can be witnessed. The findings are related to two key aspects of design. The two aspects are the “Ko” ratio, which represents the ratio of horizontal to vertical stress, and the overall shape of design. As shown in table 7, as “Ko” increases representing increasing horizontal loads, a more ellipse shape is better at handling the loads. Due to the smaller height, the horizontal loads are better equipped to be translated around the face of the tunnel. Also, since horizontal loads are not applied to a large area in the horizontal direction, less moment and shear forces act on it. Analytically, this point can be seen in the lower shear and moment for the more ellipse shape at Ko =3, than the more circular shape.

This element of the tunnel is important as Georgian Bay shale, which the tunnel sits in, has much higher horizontal values than vertical. This means that the “Ko” is much larger for tunnels in this soil condition than in other soils. In extreme cases, horizontal loads have been found to be 9 times that of the vertical [19]. As a result, the more ellipse shape is the better choice for the tunnel shape when compared to a more circular shape.

Although extremes can reach up to 9, “Ko” values will be evaluated at Ko = 3 moving forward to represent more accurate values for this area, based on examples of other Ko and ratios of horizontal to vertical stress [19].

Table 7 – Breakdown of shear, moments, and deflections across different Ko (Appendix C).

		More Ellipse Shape (Alt 1)		**More Circular Shape (Alt 2)**	
Element		K = 2	K = 3	K = 2	K = 3
Shear kN	Min (kN)	-1731	-1150	-1172	-2469
	Max (kN)	1817	1208	1164	2461
Moment	Min (kN*m)	-5736	-2848	-3216	-8952
	Max (kN*m)	5459	3297	4593	9890
Deflection (mm)		3.671	3.655	5.083	5.083

5.3. Tunnels Influence on Surface Structures

The model was initially considered with the impact of a large building near the tunnel. This building applied a large load on the ground, due to its size. However, after analyzing displacement and stress graphs, the influence of this building on our design can be seen to have no influence. The building is sufficiently away from the tunnel, allowing it to have a negligible effect on the tunnel’s construction (Figure 20 & 21). Due to the high loads imposed by this building, visual inspection of stresses and deflections around the tunnel proved difficult. Moving forward, the loads from this building will be removed to allow for better analysis of the tunnel surface.

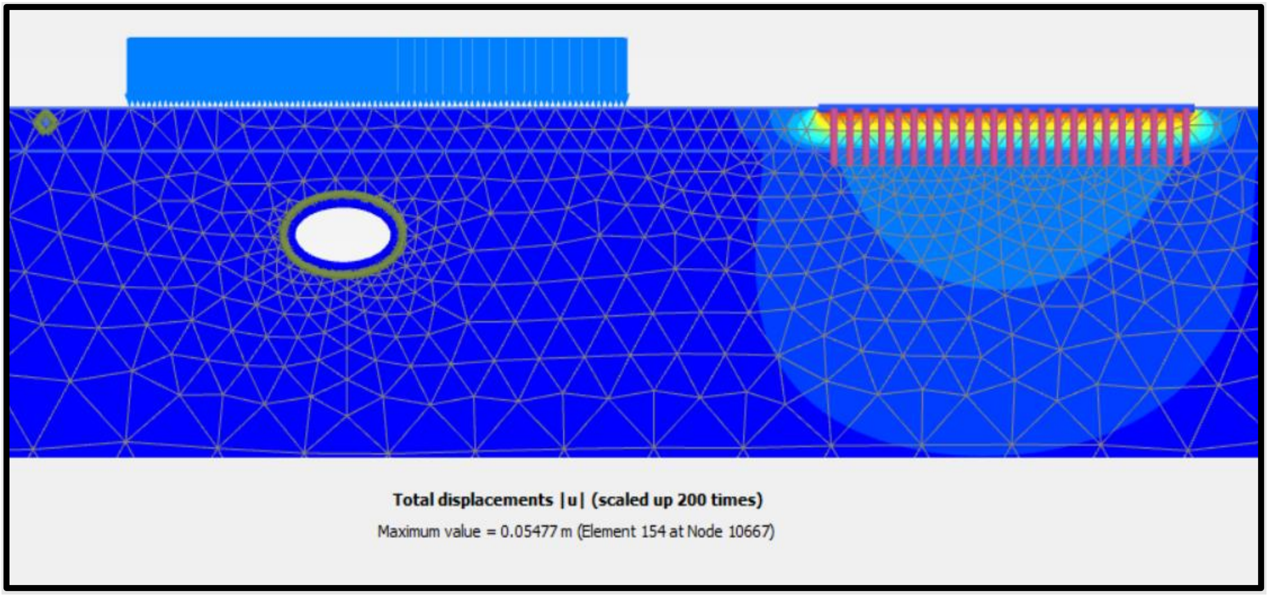


Figure 20 – Displacement graph.

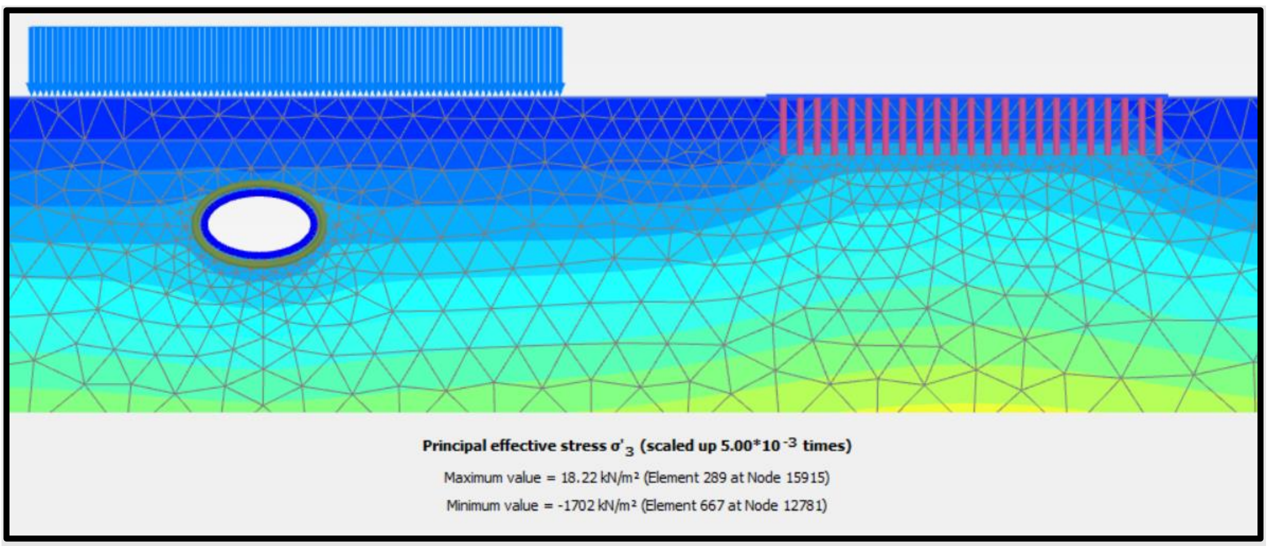


Figure 21 – Principal stress graph.

Once the considerations for the building foundation were removed a better idea of surface deformations could be seen in relation to the sewer, residential areas, and overall surface. Seen in Figure 22, the overall impact on the tunnel has little effect on the surface. Due to the depth of the tunneling, the influence of the tunnel on surrounding elements is very small. As a result, surrounding elements are kept within the acceptable deformation limits.

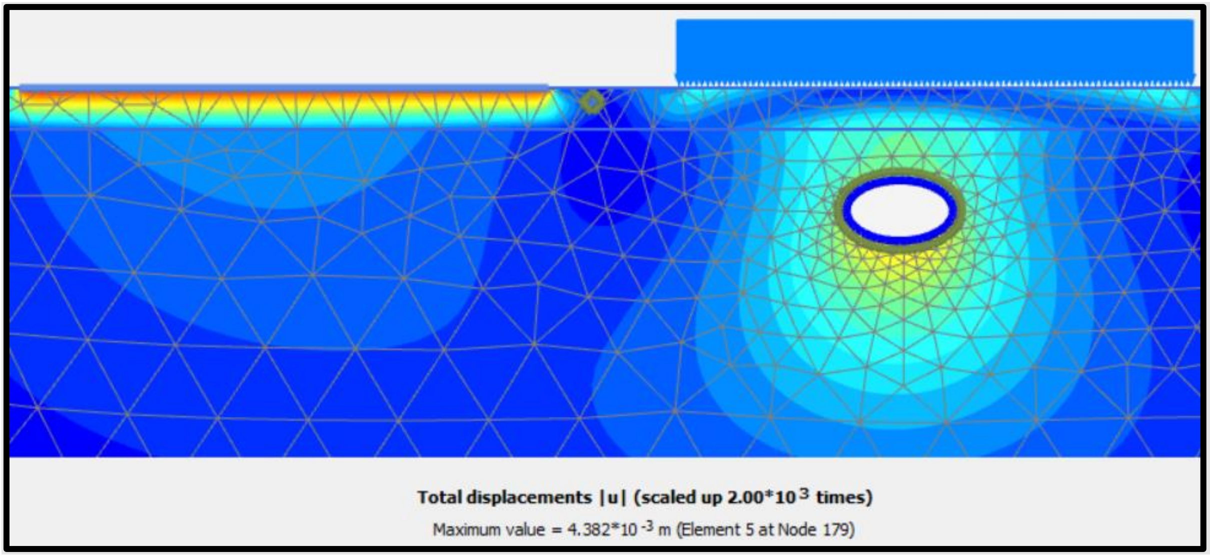


Figure 22 – Principal stress graph.

5.4. Cavern Lining

The shaft lining will make up the bulk of the support for the underground structure. The overall shaft lining will be made up of 5 major elements. These include the initial rock supports, layer of shotcrete, a compressible material, a waterproof membrane, and a final cast in place liner.

5.4.1. Rock Support and Reinforcement [20]

The initial support will be steel rebar bolts drilled into the rock. This will create zone of knit together rock that increase the overall strength of the rock face after excavation. This initial support is important for the initial excavation strength of the rock and in tandem with shotcrete will provide the bulk of the initial support. The specific rock support of steel rebar was chosen for its low cost and high strength as a support considering the jointed shale in this excavation zone. This option will involve the drilling of a borehole into the face, placing the rebar, grouting, and then tensioning the system.

The rebar rock anchors used were based on DYWIDAG hot-rolled steel thread bars. Two different bar diameters were used along the top of the tunnel vs the sides of the tunnel. This was done to reflect the larger moments seen along the tunnel crown and invert, vs the sides of the tunnel. A summary of the input values can be seen in table 8.

Table 8 – Rock anchor properties breakdown.

Property	Value
Unit Weight	7850 kg/ m^3 = 77 kN/m^3
Length	5.9m (max), 11.9m (max)[21]
Diameter	15mm, 20mm
Young's Modulus	205,000 MPa
Axial Skin Resistance	Taken from tutorial (min 1, max 100)
Max Bolt Length (Invert, Crown)*	5.47m
Max Bolt Length (Sides)**	9.34m

***Max Bolt Length (Inert, Crown) = 0.5*H (H = height of tunnel) [22]*
***Max Bolt Length (Sides) = 0.5*B (B = width of tunnel)*

The spacing of the anchors will be based on the equation spacing (s) = (3 to 4)*e. Where "e" is the max spacing of joints for the respective soil. The soil layer of Georgian Bay shale has major, and minor average spacing of joints of 1.05m and 2.8m [23] respectively. This is based on test data for shale samples pulled from the southern Ontario area. Using these ranges, this yields a minimum spacing of 3.15m on the lower end and 11.2m on the upper end [22].

5.4.1.1. Analysis in Plaxis

Seven cases were modeled in Plaxis to determine the optimum spacing and length of the steel bolts.

Based on the table 8 above, two embedded beam materials were defined, one each for the wall bolts and roof bolts given that some properties differ (such as diameter). To begin, the maximum reinforcement lengths were used as specified above, 5m for walls and 9m for the roof, at a minimum spacing of around 3m. This is Case 1, the most conservative of the cases, providing the most support possible. See the table below summarizing the cases explored and the conclusions drawn from them. See appendix D for screenshots of Plaxis outputs.

Table 9 – Reinforcement iterations.

Case #	Reinforcement Location	Spacing (m)	Roof Length	Wall Length
Case 1	Roof, Bottom, Right and Left Walls	Minimum, approx 3.5	5	9
Case 2	Roof, Right and Left Walls	Middle, approx 5.3	3	5
Case 3	Roof, Right and Left Walls	Middle, approx 4.3	1	2
Case 4	Roof, Right and Left Walls	Middle, approx 4.3	1	3
Case 5	Roof, Right and Left Walls	Middle, approx 4.3	2	3
Case 6	Roof, Right and Left Walls	Middle, approx 5.3	1	5
Case 7	Roof, Right and Left Walls	Middle, approx 5.3	1	4
Case 8	Roof, Right and Left Walls	Middle, approx 5.3	1	3
Case 9	Roof, Right and Left Walls	Closer to minimum, varies	1	4

The highlighted cases were promising, and case 7, highlighted green, was the most effective and economical.

The cases were explored in order and thus, lessons learned from each case were as follows:

Table 10 – Reinforcement iterations results.

At End of Case #	Observations and Actions Taken
Case 1	Remove reinforcements from base of cavern, no load pick-up observed. Loads are generally small and bar length not appearing to be utilized. Shorten lengths and increase spacing, ie make less conservative
Case2	Length can still be reduced; no impact of roof can be seen but keep for contingency given high horizontal loads
Case 3	Bending moment not picked up, big reduction, increase the length of the wall reinforcements
Case 4	The effect on bending got worse, try to increase roof length.
Case 5	No effect on increasing roof length, revert to case 2 but reduce length of roof reinforcement

Case 6	Same effect as case 2 observed. Keep this case, as it is more economical. Next, try reducing length of wall reinforcement
Case 7	Really good results compared to other cases. Continue reducing wall length in case we can get a more optimal load pick-up
Case 8	Worse, try to double the number of wall reinforcements.
Case 9	Worse results, revert back to case 7.

The final cavern reinforcement comes out to be the following on Plaxis:

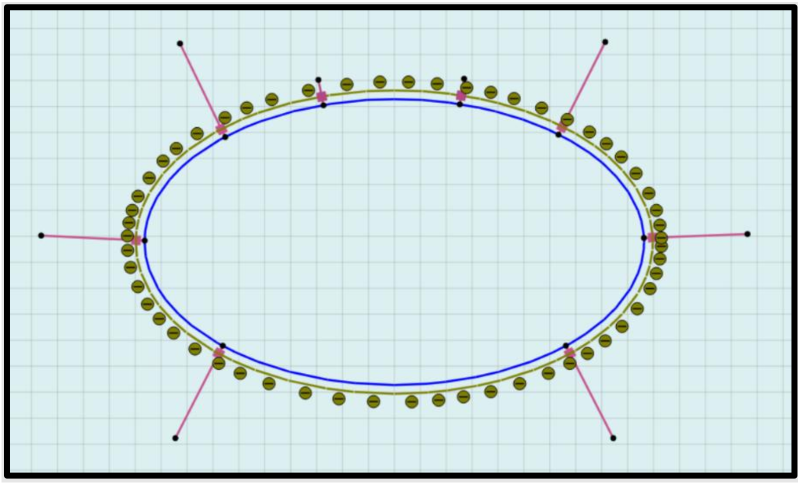


Figure 23 – Final reinforcement layout.

5.4.2. Shotcrete

Once the rock supports are placed, shotcrete will be used as the initial support of the tunnel immediately after excavation. Shotcrete is used as it can provide immediate support to the ground. This aspect is important for such a large space of unreinforced ground. The shotcrete will be used in tandem with steel rebar bolts. As such, will be designed to support the excavation completely before the final lining is installed.

Shotcrete thickness will depend on the type of rock/ soil present surrounding the excavation. As previously mentioned, the ground soil was classified as moderately blocky and seamy soil. As such, the ground will be assumed to fall between type II and type IV rock type. Within this range, shotcrete will be between 150mm and 200mm thick [20]. To be conservative, a 200mm thick shotcrete layer will be used as the initial support. The thickness as stated by the federal highway commission will be based on the soil quality designated in section 3.1.1.

Overall shotcrete will be used structurally to provide initial safety when excavating. The thickness designated will ensure the tunnel is adequately supported, however still thin enough to deform and not crack. At this thickness, a lattice of reinforcement will be necessary to support the shotcrete (Figure 24).

5.4.3.Compressible Material

Following shotcrete, a compressible material will be included in the lining. This material will be used to counteract the expected swell. Due to the location of the excavation, the primary rock type will be Georgian Bay shale. This shale has a large swell potential, which can cause large squeeze stresses on the tunnel lining [13]. To prevent these swelling stresses, a layer of compressible material will be placed in between the shotcrete and waterproof membrane. Compressible materials can range in size from roughly 150mm [24] – 300mm [25] while still being effective. Due to the smaller size of the cavern, and low depths, the lower end of this range will be taken at 150mm. The material will be a compressive concrete mix. This mix will also serve to connect the space between the grout lining and the final lining. The concrete mix will be pumped into the space between the two layers using vents along the final linings surface.

5.4.4. Waterproof membrane.

Following the layer of compressible material, a waterproof membrane will be installed. This membrane ensures groundwater flows will be diverted away from the crown of the tunnel, leaking onto subway internals. The water will instead be diverted along the outside of the membrane to water removal systems along the invert of the tunnel. This layer will be roughly 3 mm thick.

5.4.5. Cast-in-Place Final Lining

Finally, after the waterproof membrane is installed, the final cast in place liner is installed. Cast-in-place lining will be used instead of alternatives such as precast segmental linings, steel plate linings, or shotcrete linings. This is due to cast-in-place linings flexibility in shaft size and support type. Since concrete is poured on site, the irregular shape of the tunnel can be supported with this lining type. Additionally, cast-in-place tunnel lining provides a durable and low maintenance way to support the tunnel. The overall thickness of the precast lining will be 300mm. This thickness will meet the practical minimum of 250mm. The tunnel lining will also be cast in 10m long sections and connected with a small key connection to prevent surface cracks. Reinforcement will be present along the liner to prevent tension loading in the concrete lining. This reinforcement will be a rebar lattice along the length of the tunnel. Reinforcement within the concrete lining will be critical to the overall strength of the lining. Due to the presence of a waterproof membrane, the effect of water in terms of corrosion will be mitigated. Additionally, steel reinforcement will be galvanized, to prevent any water that does reach the rebar from causing corrosion to occur.

Due to the use of a cast-in-place liner, special considerations will be made when casting. Air vents will be included along the length of the pour to ensure concrete is poured consistently along the length of the pour. This will ensure no pockets will form in between the cast-in-place liner and the compressive material/shotcrete layer. As such, air vent should be spaced at regular intervals when pouring.

The final thickness will end up being 653mm, going back from the tunnel face (Figure 24).

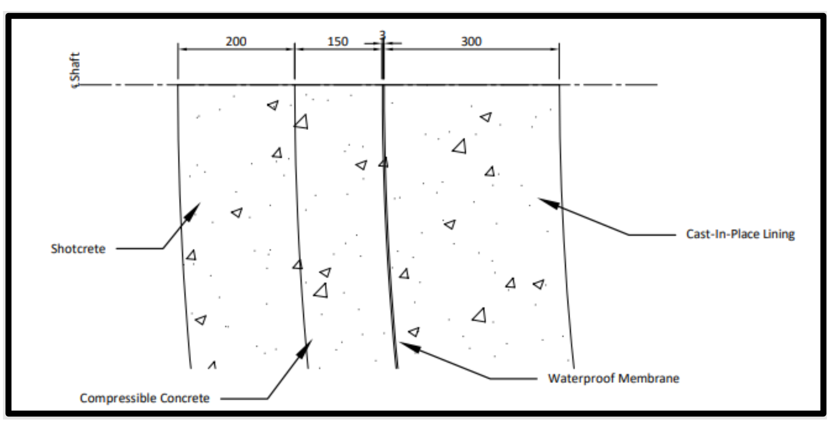


Figure 24 – Tunnel Lining Diagram.

6. Cavern Construction Modeling

6.1. Construction Method – NATM

To construct the cavern, we need to use a method which allows the construction of a larger opening without inducing a lot of deformation. This led us to choose NATM method to do the construction in a sequence. After picking how we wanted to excavate the cavern we needed to design the steps and sequencing phases. These steps are elaborated below.

6.2. SEM face sequencing

The face sequencing was separated into four distinct steps. Initially we divided the cavern into two distinct areas so we could create the bench. The bench was picked to ensure machinery can reach the top of cavern for excavation as the cavern size is relatively large. Figure 25 illustrates this distinction.

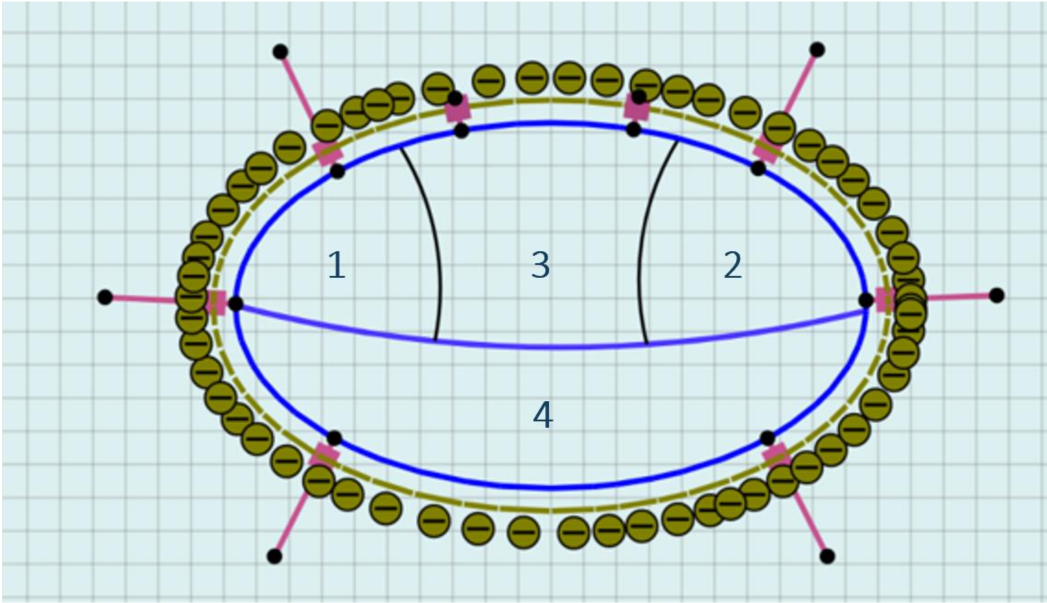


Figure 25 – Face sequencing stages.

Subsequently we divided the top part of the cavern into three different areas to reduce the excavation area. The aim was to have a design which results in a stable construction period while maximizing the amount of excavation we can have to reduce the project timeline. The following Figure shows the final face sequencing for the cavern. Each step will be done in order of their assigned number. We will also sequence the excavation of each step relative to each other by doing a depth analysis which is shown below. This face sequencing was tested through a Plaxis model to ensure the stability of cavern through the construction. The model will also be explained further below.

6.3. SEM depth sequencing

Based on the provided borehole data the friction angle for the rock mass is 35 degrees. Using this angle of friction, we will calculate the depth sequencing for the SEM method. By testing different depths, we converged to 10m. This shows us based on our friction angle how deep we can safely excavate to ensure we are not affecting the whole face. The figure below illustrates this result. Due to this 10m will be our gap between excavation of step 3 and 4.

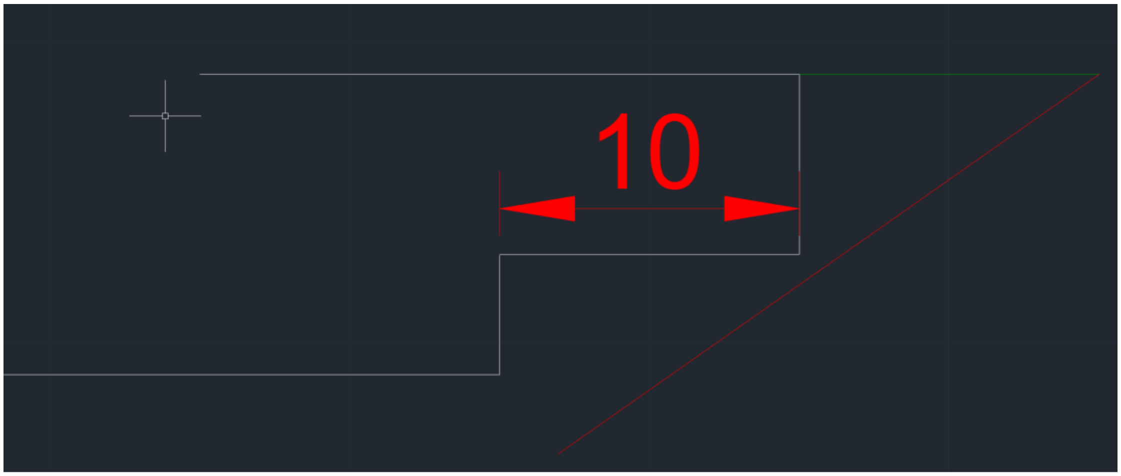


Figure 26 – Depth sequencing stages.

6.4. Model Setup and Staging

The same material properties discussed earlier for the cavern including the final liner and steel rebar bolts were used in this model. A temporary liner was added in which carried the properties of unit weight equal to 23.54 kN/m/m, EA equal to 645.9E6 kN/m and EI equal to 93.34E9 kN m2/m (Appendix K).

The goal of modeling the SEM face sequencing in Plaxis was to get a better understanding of the behavior of the cavern while being excavated and impact on surrounding infrastructure. To simplify the modeling, a basic staging method was conducted.

1. Stage 1: Initial, no elements activated.
2. Stage 2: Section 1 deconfinement set to 15% and is deactivated.
3. Stage 3: Section 1 temporary liner and steel rebar bolts activated.
4. Stage 4: Section 2 deconfinement set to 15% and is deactivated.
5. Stage 5: Section 2 temporary liner and steel rebar bolts activated.
6. Stage 6: Section 3 deconfinement set to 100% and is deactivated. Change deconfinement of section 1 and 2 to 100% as well.
7. Stage 7: Section 3 temporary liner and steel rebar bolts activated. The temporary liner along the bench is activated as well. Now the top half is excavated.
8. Stage 8: Section 4 deconfinement set to 100% and is deactivated.
9. Stage 9: Section 4 temporary liner and steel rebar bolts activated. Deactivated the temporary liner at the bench.
10. Stage 10: Activate the cavern liner along the entire circumference as the cavern.

6.5. Final SEM Step design

The final Plaxis results after having run the staging steps provided in 6.4 showed there will be no significant impact on the surrounding buildings. The modeling output shows that the high deformations of the apartment building exist prior to the excavation of the tunnel. This means that the deformations are not due to the tunnel excavation. The building element was deactivated as a result. The deformations found on other structures were not concerning. Additional images showing the bending moment and shear forces present can be found in Appendix E.

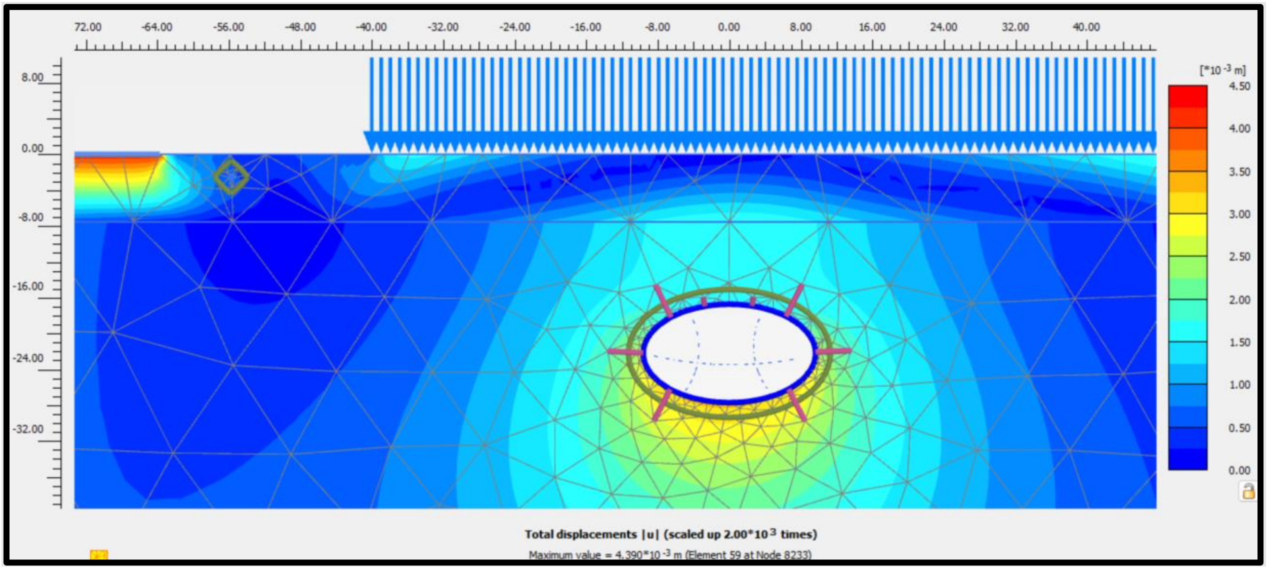


Figure 28 – Building element deactivated, showing deformations on other existing infrastructure.

7. Shaft Design

7.1. Shaft Dimensions

The final shaft design consists of a cylindrical Figure which carries a diameter of 34.2m and reaches a depth of 27.71m.

If a shaft is more than 6m deep or connects to a tunnel that is greater than 15m, the minimum inside dimensions of the shaft must be 2.4m for cylindrical and the minimum transverse sectional area must be minimum 5.7 square meters (O. Reg. 213/91, s. 278) [26]. Our dimensions satisfy the requirements, so we are ok to proceed.

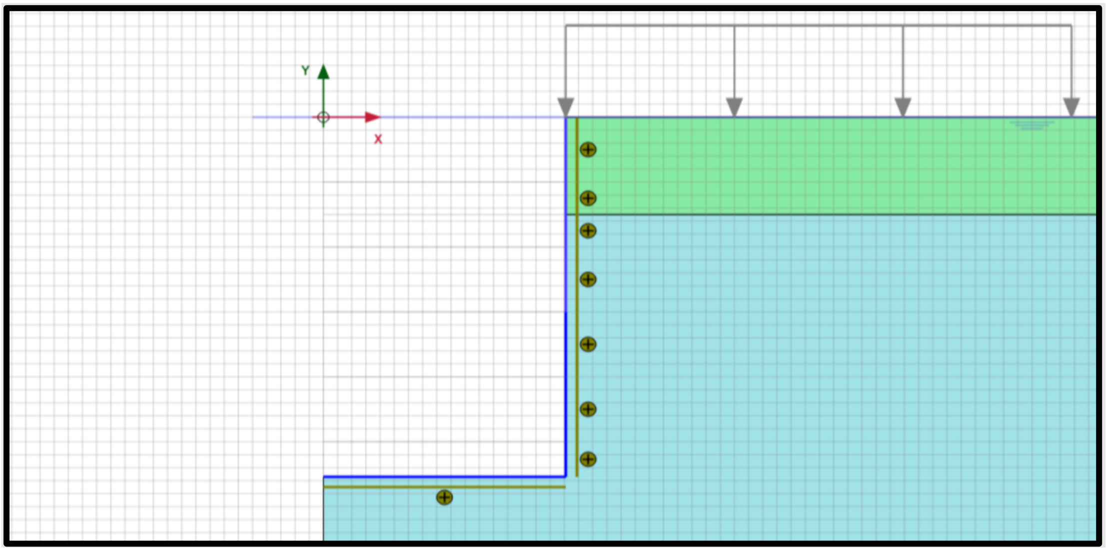


Figure 29 - Asymmetric model of the circular shaft at a depth of 27.71m and having a radius of 17.1m.

7.2. Shaft Shape

It was decided to go with a circular shaft over rectangular or square shape. Circular shafts are more structurally stable, and the earth loads that are applied on the shaft puts its support in ring compression [27]. Due to dependency of compression to take the lateral loads they do not require any internal bracing and the structures reinforcement can be reduced [27]. A square or rectangular shaft carries a greater concentration of stress at the corners which makes it more difficult to be self-supporting [28].

7.3. Shaft Support System

Two alternatives were assessed when determining what support structures should be used for the shaft. The first option explored the use of both tiebacks and shotcrete simultaneously. The second option had secant piles that were drilled to a depth of 15m and then shotcrete applied from the 15m mark to the final depth of 27.71m. The final design that we proceeded with was the second option of secant piles accompanied by shotcrete which will be further explored below.

7.4. Model Set Up

In the initial stage of designing, both alternatives were modelled on Plaxis to determine what the bending moments, shear forces, and displacements were. There were two sets of models made for each of the alternative in which one was an asymmetric model and the other was a continuation of our cavern model looking into the section profile.

7.4.1.Final Proposed Design: Secant Piles and Shotcrete

The design we are proceeding with will be discussed first starting with the material properties used in the Plaxis model and the calculation results.

7.4.1.1. Secant Piles and Shotcrete Material Properties

The secant piles used were drilled to a depth of 15m and carried a diameter of 600mm [29]. The shotcrete used has a thickness of 200mm, which was identified earlier to be the most conservative thickness [20]. The properties used are summarized in the table below with supporting results found in Appendix F.

Table 11 – The material properties of the secant piles and shotcrete used.

	Secant Piles	Shotcrete
Unit Weight (kN/m/m)	23.54	23.54
EA (kN/m)	1914888685	645894993
EI (kN m2/m)	2.70315E+11	93335055959
Diameter (mm)	600 [29]	X
Thickness (mm)	X	200 [20]

7.4.1.2. Final Plaxis Models of Secant Piles and Shotcrete

Below the final models of the asymmetric models and the section models of our proposed design are presented. Other illustrations demonstrating the bending moments, shear stresses, displacements etc. can be found in Appendix F.

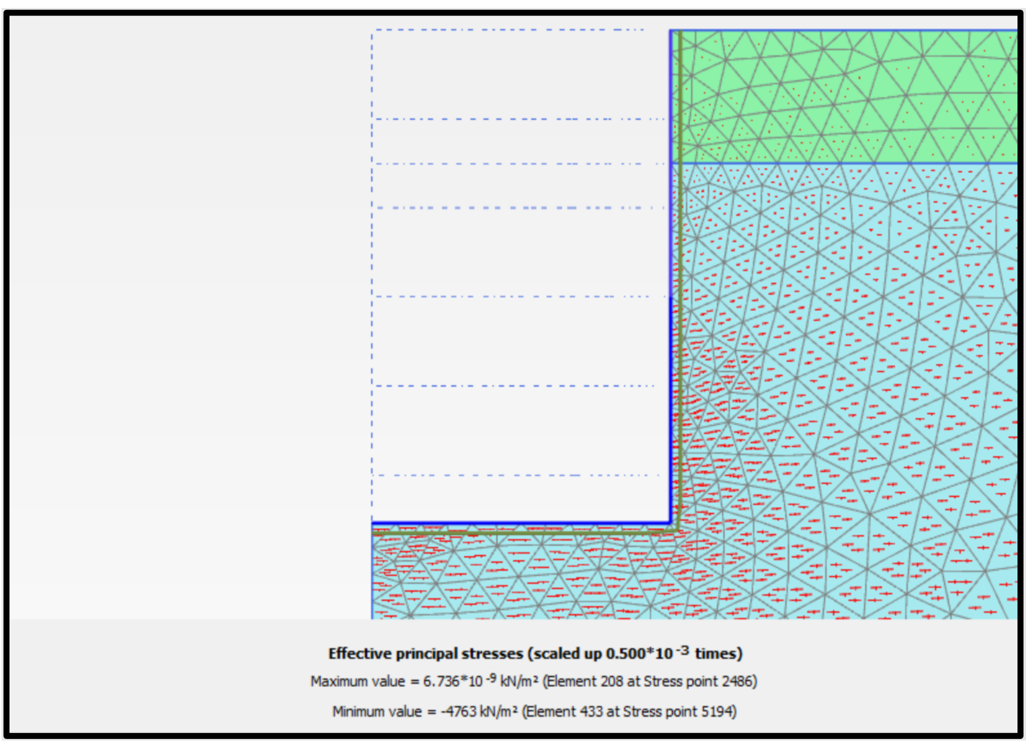


Figure 30 – The asymmetric model showing the effective stresses present around the secant piles and shotcrete of the shaft.

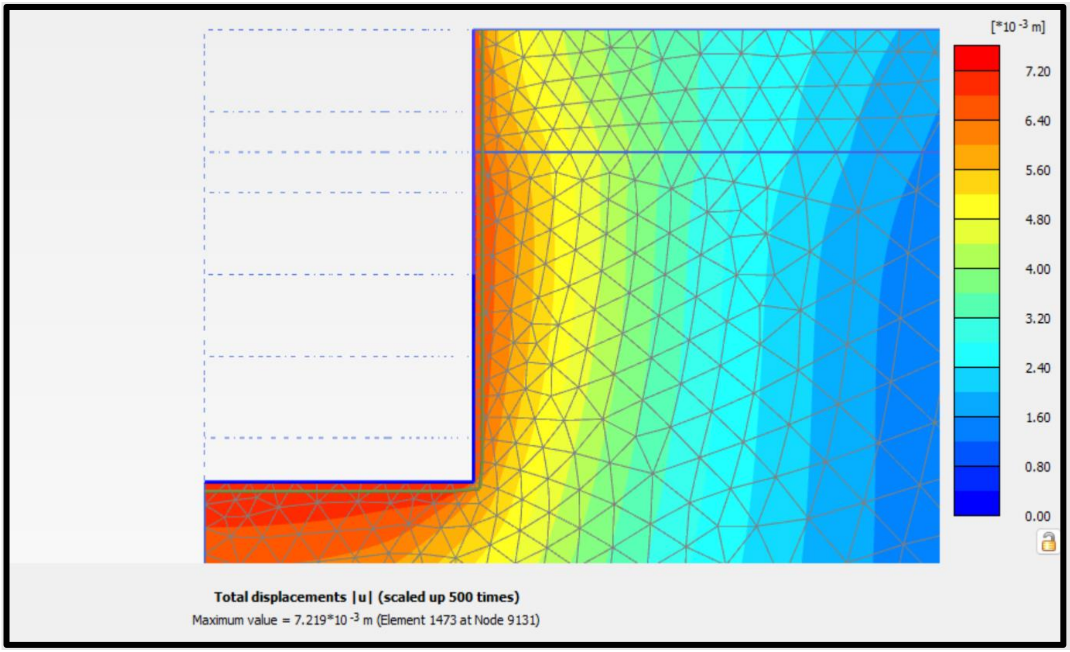


Figure 31 – The asymmetric model shows a total displacement of 7.219E-3m.

7.4.2. Alternative Design: Tiebacks with Shotcrete

The design we explored as an option but did not proceed with included the use of tiebacks and shotcrete which will be explored below.

7.4.2.1. Tiebacks and Shotcrete Material Properties

When modeling this alternative, the length of tiebacks used was 12m. We started with 6m and iterated to 12m to attain better results. They were angled at 65 degrees from the wall of the shaft and were spaced every 3m apart from each other.

Table 12 – The material properties of the tiebacks and the shotcrete used.

Property	Anchors	Embedded Beams	Shotcrete
Unit Weight (kN/m/m)	X	X	23.54
EA (kN)	500E3	X	645894993
E (kN/m2)	X	7.07E6	X
EI (kN m2/m)	2.70315E+11	X	93335055959
Diameter (mm)	600 [29]	300mm	X
Thickness (mm)	X	X	200 [20]
T (axial skin resistance, start max – kN/m)	X	400	X
T (axial skin resistance, end max – kN/m)		400	X
L spacing (m)	3	3	X

7.4.2.2. Plaxis Models of Tiebacks and Shotcrete

Below is the final model of the second alternative. Other illustrations demonstrating the bending moments, shear stresses, displacements etc. can be found in Appendix F.

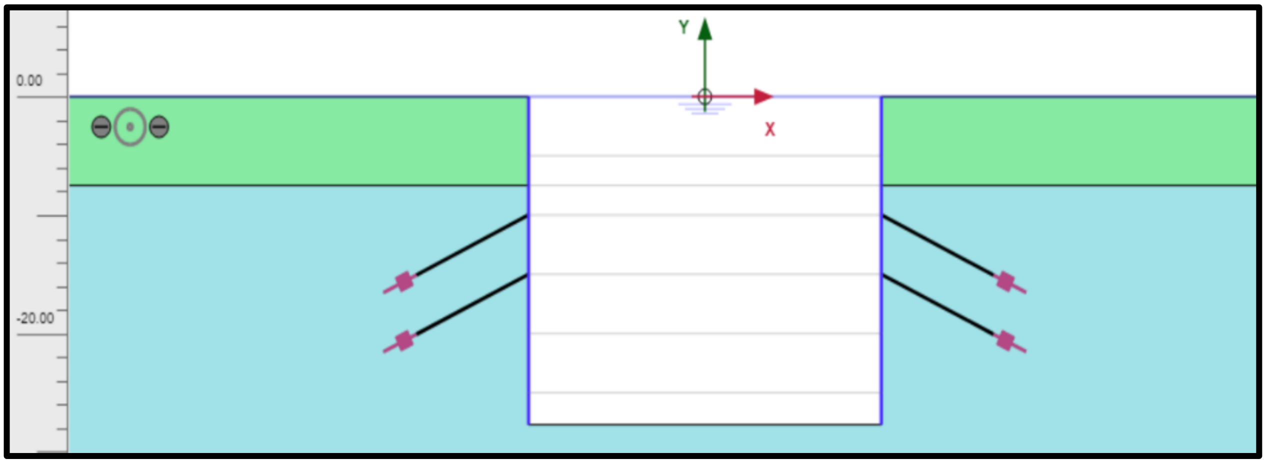


Figure 32- Model of the circular shaft with tiebacks and shotcrete.

7.4.2.3. Comparison between tiebacks/shotcrete and secant piles/shotcrete

The following table summarizes the results attained from the two alternatives.

Table 13 – Plaxis results for alternative 1 with the use of tiebacks and shotcrete and alternative 2 where secant piles and shotcrete were used.

	Tiebacks and Shotcrete	**Secant Piles and Shotcrete**
Total displacement	0.02057 m	0.03400 m
Shear force (max)	1147 kN/m	1106 kN m
Shear force (min)	-673.9 kN/m	-648 kN/m
Bending moments (max)	329.8 kN m/m	617.8 kN m/m
Bending moments (min)	-5039 kN m/m	-3959 kN m/m

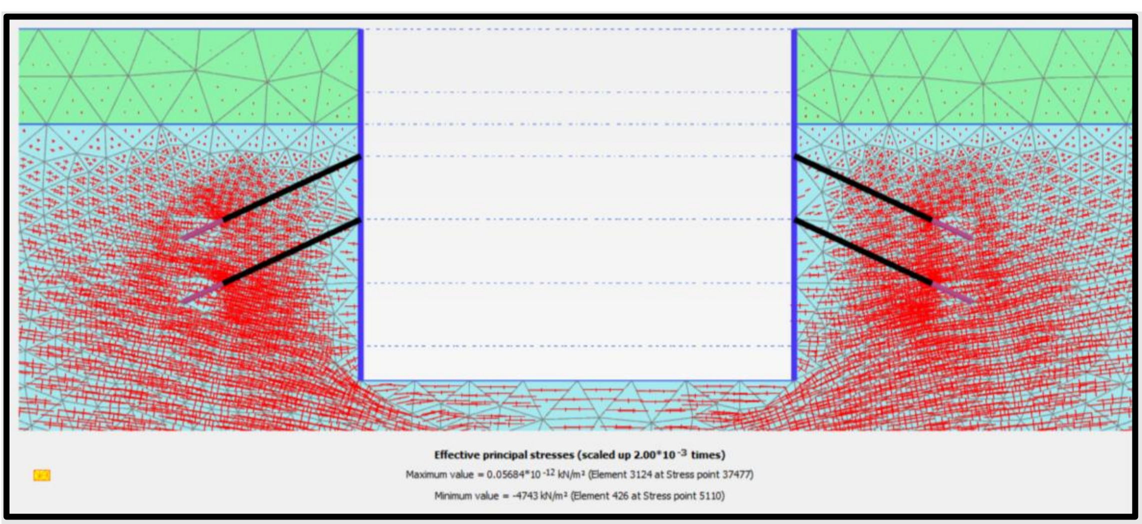


Figure 33 – The effective stresses around the tieback and shotcrete option.

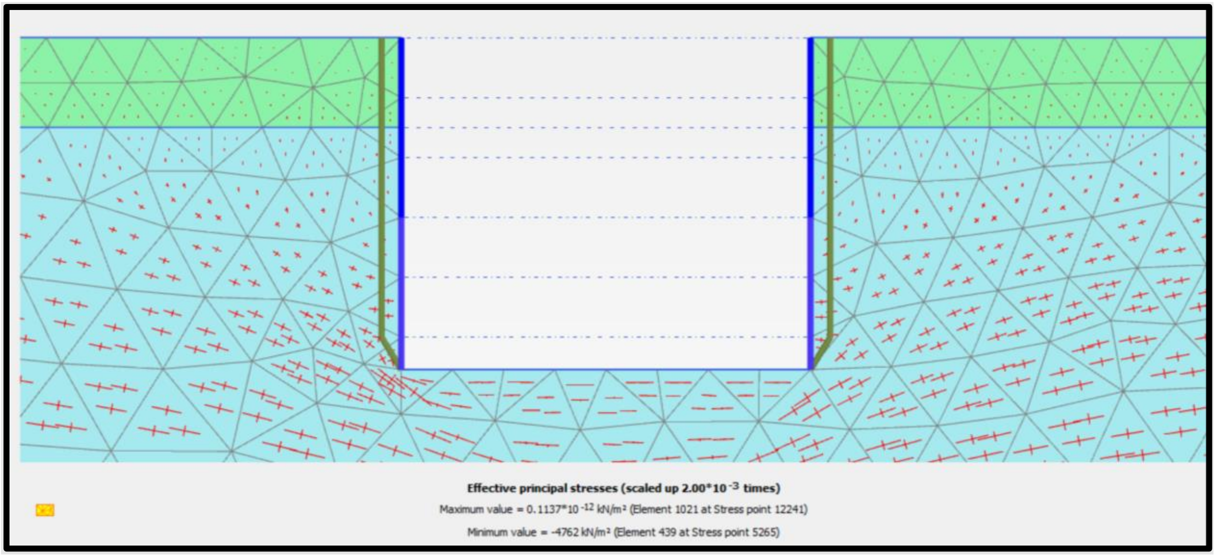


Figure 34 - The effective stress around the secant piles and shotcrete option.

Evidently from the results the bending moments, shear forces, and displacement is lower for the secant piles and shotcrete option. There were several other considerations we had to make to determine which route to take. Secant piles were favored because they were watertight, easy to construct, and are best suited for circular shafts [30]. A key consideration was the fact that our excavation location is near a pond meaning the water table is assumed to be high. Secant piles create a force-locked connection and sufficient water tightness which will ultimately serve as a water mitigation strategy [30]. Another consideration was that by extending the secant piles to 15m we are ensuring to pass the initial soil layer along with the first few meters of Georgian Bay shale. This consideration ensures we are leaving our piles in stable Georgian Bay shale and allows us to use shotcrete for the rest of the access shaft not as a means to support the structure, but rather to cover the exposed rock.

8. Construction Timeline

8.1. Pre-Construction

Within pre-construction there are decisions to be made regarding the best alternative selection, environmental assessment, approvals required, and procurement and finance. In this stage of the construction, it is assumed that a feasibility study has been completed that explores the viability of the project and has considered the economic, financial, legal, and environmental considerations [31]. All the environmental assessments have been completed.

8.2. Permits (30 Days)

The construction will take place 5 days a week, meaning no work will take place on Sundays as construction noise is prohibited by the City of Mississauga on Sundays and statutory holidays [32]. No noise exemption permits are required as we will be operating within a quiet zone outside the hours from 5:00 PM of one day to 7:00 AM the next day [32]. Road occupancy permits will not be required as we will not block off any existing sidewalks or city road, it is not necessary in our case considering the vast amount of parking space available and we want users to have access to the Erindale Go Station even during the construction phase. A stormwater temporary discharge approval will be required as we will be discharging the water collected during excavation to a nearby sewer catch basin [33]. A site plan will need to be approved which will show the waste collection area, sufficient drawings, the traffic management plan, noise reports etc.

The Building Transit Faster Act, 2020 helps fasten the process of completing transit projects by simplifying and increasing efficiency of certain process and eliminating barriers that may interfere with the timeline of the original schedule [34]. Through this act, lands can be subjected to being designated as "transit corridor lands" (TCLs) [34], in which the boundaries are seen in Figure 35. For the residences that fall within the TCL+ 30m permit buffer for Erindale Subway Station, will be contacted need be for inspections which can assist in the development of the transit project [34]. Hence notices will be sent to nearby residences to aid in the development of the transit project concerning vibration and noise [34].

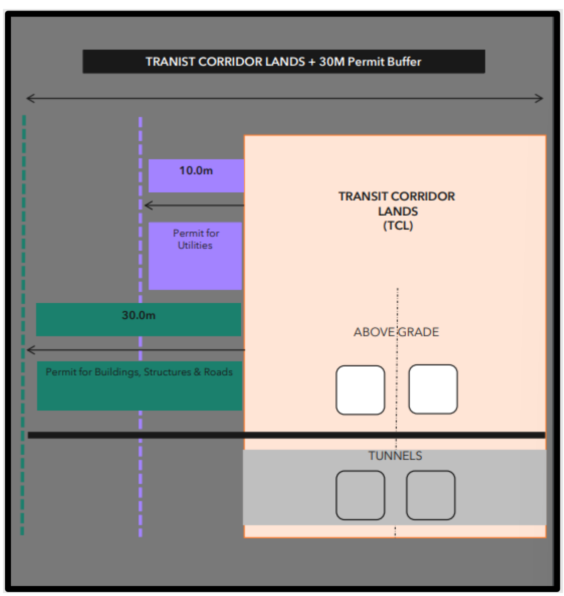


Figure 35 – Boundaries around the TCL [34].

Notices Required on Site:

Employers are required to notify the Ministry of Labor, Immigration, and Training and Skills Development prior to commencing the construction work regarding tunnels and shafts and will need to have a copy of this notice at the worksite for inspection [26]. In accordance with the section 5 of the Regulation for Construction Projects (O. Reg. 213/91), prior to the start of construction every constructor and employer involved will be required to complete the approved registration form (form 1000) and will need to keep it at the site location [26].

8.3. Utility Investigation (5 Days)

While waiting to receive permit approval, the first step will be identifying any locates prior to the start of construction. It will prevent any unnecessary collisions with existing pipes or lines during excavation by informing us of where they are situated. It has been assumed a 5-day duration based on the size of the site.

8.4. Mobilization (7 Days)

Our second step is to clear the site to ensure it is ready for construction. We have allocated 2000 m2 of mobilization area on site. In this phase we will install 2 portable toilets, a trailer acting as a site office with a lunchroom, another trailer acting as a changeroom with a shower, an area designated for equipment parking, employee parking, and stockpiling area. Temporary signs will be installed at this phase as per the Ontario Traffic Manual. As per the law, a heated room needs to be available for the underground workers (O. Reg. 213/91, s. 260 (1)), there needs to be a change room, and for workers employed underground there will need to be a shower in the change room for every 10 people (O. Reg. 213/91, s. 260 (5)) [65]. So, with the installation of the above facilities we will be fulfilling the requirements.

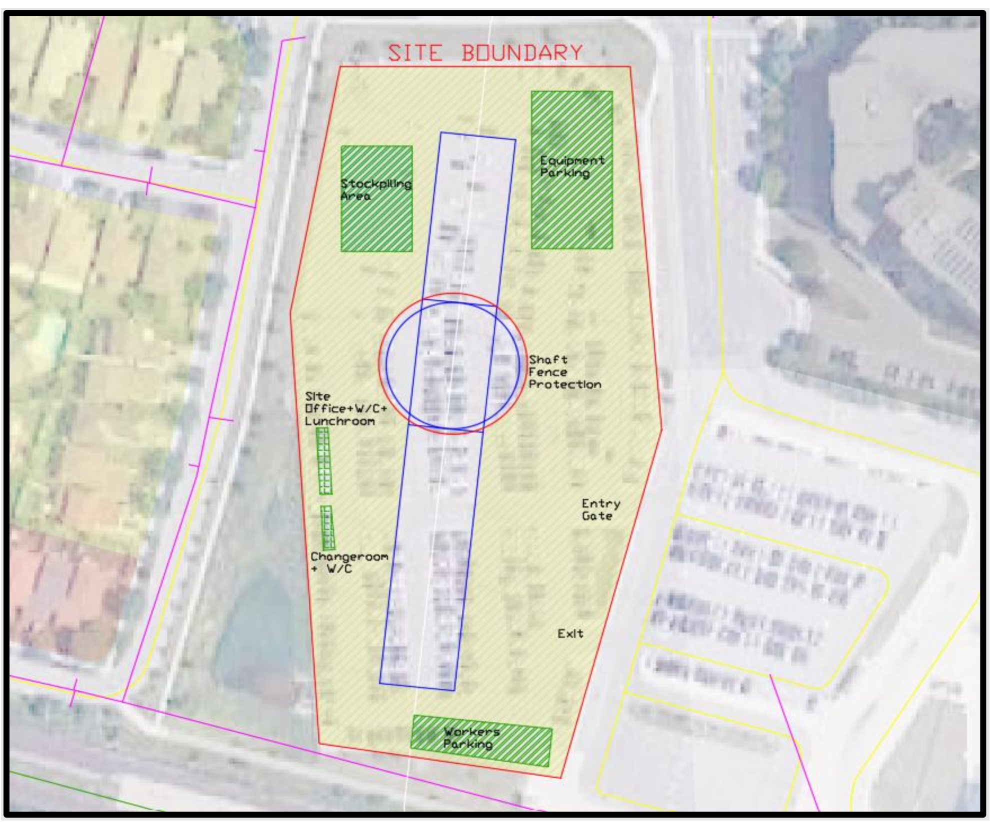


Figure 36 – Site mobilization layout alongside the site boundary established.

8.5. Fence and Site Boundary (4 Days)

The entire parking lot that the excavation sits on will be within the site boundary even though there will be a lot of access space that will not be utilized. This will be done to ensure there is sufficient space within the site to work safely and to ensure pedestrians and those utilizing the GO station will remain at an adequate distance away from the excavation zone. There will be no way for users to park in this area and put themselves in danger. Knowing that there is also a building for parking by Erindale GO station, it makes it easier to block off the entire parking lot as seen in Figure 36. The sidewalks bordering the site will be open for access as the excavation will be quite far from the perimeters of the site boundary.

By the law since the fences are being erected on a construction site, they will need to be a minimum height of 1.8 above the grade outside the enclosed area [35]. Fences will consist of galvanized steel chain links because they are durable, affordable, and can be put up in a fast timely manner [36].

8.6. Traffic Management Setup (3 Days)

Traffic management measures are important to reduce congestion and ensure safety to drivers of neighboring roads. Road flaggers will be employed for the mobilization, demobilization, and days when concrete trucks and haul trucks will be scheduled to come in and out of site, due to increased traffic. A police officer will be present during mobilization and demobilization on Rathburn Rd, to ensure safe normal operations of the GO transit during this busy time. Road flaggers are required during the stages mentioned above because during mobilization large equipment's will be coming into site and will need to be guided accordingly to ensure minimal to no interference to surroundings and can situate themselves in the correct parking area of the equipment. During concrete pourings there will be lots of concrete trucks coming into site which will need to be directed, and so to accommodate the greater traffic road flaggers would be critical for safety reasons. Due to works being near major roadways a police officer will be present when having the equipment's come into site during mobilization and when leaving the site during demobilization.

8.7. Installation of Secant Piles (20 Days)

Prior to excavation secant piles will be drilled around the circumference of our shaft face. It is desired to be done prior to excavation to ensure there is sufficient support. A rig will be used for this drilling and pouring due to its multifunctional works.

The sequence of installation starts with the drilling and pouring of every other shaft which would be the primary shafts [37]. During this process a wheel loader will be by the rig to take the displaced soil to the nearby stockpile as seen in Figure 36. Concrete trucks will be scheduled accordingly to ensure they are on site when pouring needs to be done. When the concrete is set in the primary shafts, the rig will then drill and pour the secondary shafts creating an interconnected system between the primary and secondary shafts [37]. Key consideration is the haul trucks and concrete trucks will not be entering and leaving site at the same time to ensure there is no traffic build up on the adjacent right of way roads and onsite. Hence haul trucks will be scheduled to come in when concrete trucks are not.

Roughly 90 piles will be needed (1.2 m diameter) which would mean a total of 1518m3 of soil displaced, hence for each pile 17m3 of soil is removed. Considering a wheel loader can carry a capacity of 15m3, for every pile dug up, the wheel loader will take all that material to the stockpile at once [38]. For every pile, 2 concrete trucks will be required to fill since these trucks carry a capacity of 8m3. Assuming 16 bored piles are done per day, it will take 6 days to complete the 90 piles. To do 16 bored piles per day, 32 concrete full loads will be required. Assume concrete curing will take place after the installation of all the primary shafts for 168 hours, then the install of the secondary shafts will be done followed by another 168 hours for their curing, resulting in a total of 20 days for the installation of secant piles.

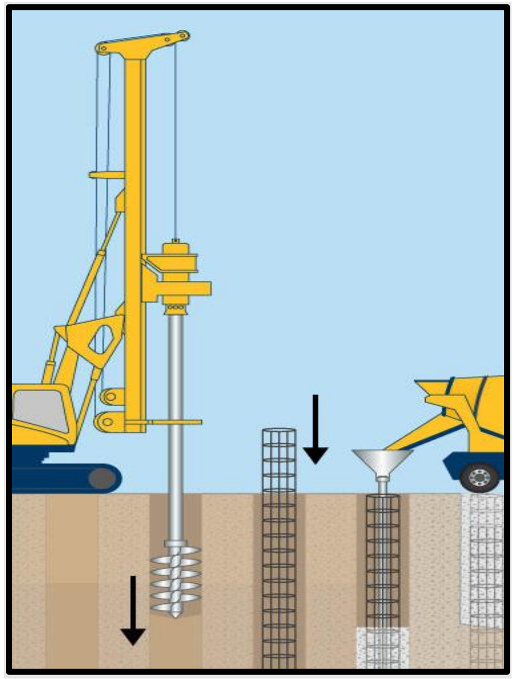


Figure 37 – Installation of the primary shafts [64].

8.8. Excavation of Access Shaft (93 Days)

One item to note is that a barrier will be placed 1.1m high at the top of our uncovered shaft and will consist of a top rail since the shaft is deeper than 2.4m (O. Reg. 213/91, s. 282); this will ensure we are compliant with the regulations [65].

The first excavation will begin with the shaft as it is the access point to the rest of the tunnel. Two hydraulic excavators carrying a 0.4m3 bucket capacity, with an average productivity of 20m3 for sand and 25m3 for hard ground will be utilized to complete the work [62]. Considering the crew will work 8hrs/day and their efficiency is 50min/60min, it will take 93 days to complete. Supporting calculations found in Appendix G.

For 34m3 of soil to be removed by the hour, it will require ~1.7 excavators (both excavators working at same time) and considering a dump truck can carry up to 12m3 [38], within this hour 3 haul trucks will come into site to take the material away. The excavators will dump the soil in a nearby stockpile where one wheel loader will be present to fill the haul trucks with the material to be sourced away. The wheel loader carrying a capacity of 15m3 [38] is capable of filling one haul truck at once and hence will be located only by the stockpile to serve this purpose. The stockpile will have a rectangular base to maximize the amount of soil that can be stocked.

8.9. Dewatering (93 Days)

During excavation of the shaft there will be dewatering taking place all throughout hence the timeframe is assumed to be the same number of days required for excavation of the shaft. Sump pumps will be used which will have a pipe extending to the nearest catch basin located at our site, which we have received a stormwater temporary discharge approval for as mentioned in the pre-construction. Prior to discharging the water, the overall process to commence this operation would be, having the water sampled and tested for cleanliness and accordance to standards, talk with the City of Mississauga to ensure there is sufficient capacity within the stormwater system and then we may proceed to discharge. Assume the water is clean enough.

An alternative that was explored is storing a tank on site but considering the cost for a tank, the additional fees for the contractors to discharge, coordination within the team, and space taken up by tank it was preferred to rather discharge into the catch basin.

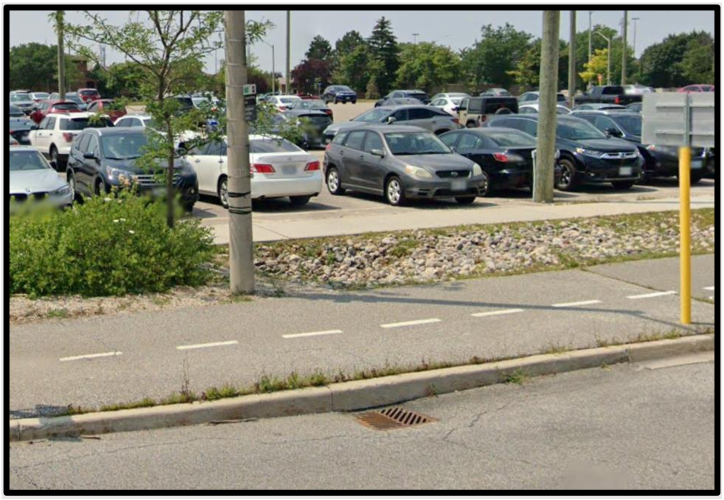


Figure 38 – The catch basin we will discharge water into is found within the parking lot [63].

8.10. Shotcrete, Compressible Material, Water Proofing (93 Days)

Secant piles are already installed so for the first 15m down the excavation of the shaft there will be no need for additional support. It is assumed the implementation of shotcrete, compressible material, waterproofing will take 93 days because it is dependent on the excavation of the shaft being completed, as it will happen simultaneously. The excavation of the shaft is the critical path here. 11403m3 of shotcrete will be required to fulfill a thickness of 200mm of the entire shaft. With a concrete truck capacity of 8m3, we will require 1425 full loads.

A Putzmeister mechanized concrete spraying machine will be utilized to complete the shotcrete. It consists of a spraying arm, concrete pump, and control software [39]. The max vertical reach of the spraying arm is 17m [39]. With 2 excavators in use for excavation, every 30 minutes there will be 2 areas dug up equaling to a total of 20m3. Hence every 2m depth and 2m width excavated, shotcrete will then be applied. The process will not be further discussed as the excavation would mainly be the critical path in this section.

8.11. Final Liner of Shaft (10 Days)

Assuming it takes one hour to complete 25m2 of formwork, it will require 55 hours to complete the 1366m2 required for the shaft. That will be a total of 7 days. Maintaining a thickness of 300mm for the final liner which will consist of a structural and architectural concrete with light gray and satin finish for the entire structure [40], a total of 406m3 of concrete will be required which will take 51 full concrete loads. With a concrete pouring rate of 11 minutes/meter per equipment, it will take 74 hours which translates to 9 workdays to do the pouring to complete the final liner of the shaft [41]. We will use multiple pipes to finish the pouring which will decrease the number of days to 3 days if we have 3 pipes performing at 11 minutes/meter [41].

8.12. Excavation of the Cavern (114 Days)

The critical path in this entire step would be the excavation of the cavern hence we will determine how long it will take to complete the excavation with the use of 5 excavators and during this process rock bolts will be installed, followed by shotcrete, compressible material, and waterproof membrane. The final liner will be accounted for after the completion of the excavation. This was done to account for the fact that installation of the final liner should be complete after the excavation is completed.

It will take 114 days (4 months) to complete the excavation of the cavern utilizing 5 excavators at productivity rates of 30m3/hr and efficiency of 50min/60min which can be supported with the calculation found in Appendix I.

8.13. Final Liner of the Cavern (14 Days)

Considering it takes one hour to complete 25m2 of formwork, it will require 142 hours to complete the 3562m2 required for the cavern. The final liner will be done on either end of the shaft at the same time which would bring it to 8 days to complete the formwork of the entire cavern.

Maintaining a thickness of 300mm for the final liner which will consist of a structural and architectural concrete with light gray and satin finish for the entire structure, a total of 1052m3 will be required which will take 131 full concrete loads [40]. With a concrete pouring rate of 11 minutes/meter per equipment, it will 24 workdays to do the pouring to complete the final liner of the shaft [41]. We will use multiple pipes to finish the pouring which will decrease the number of days to 6 days, if we have 4 pipes performing at 11 minutes/meter.

8.14. System to Move the TBM through the Tunnel (20 Days)

To move the TBM through the station we will be using a semi ring support system. In this method we'll be implementing semi ring concrete supports on the bottom of the station [42]. These rings will allow the TBM to push itself through the tunnel using its jacks located underneath the device. The image below illustrates the move direction of the TBM using the semi ring support system, and the jacks underneath the TBM used to push it through. The entire process will take 20 days to install, move TBM, and remove the system.

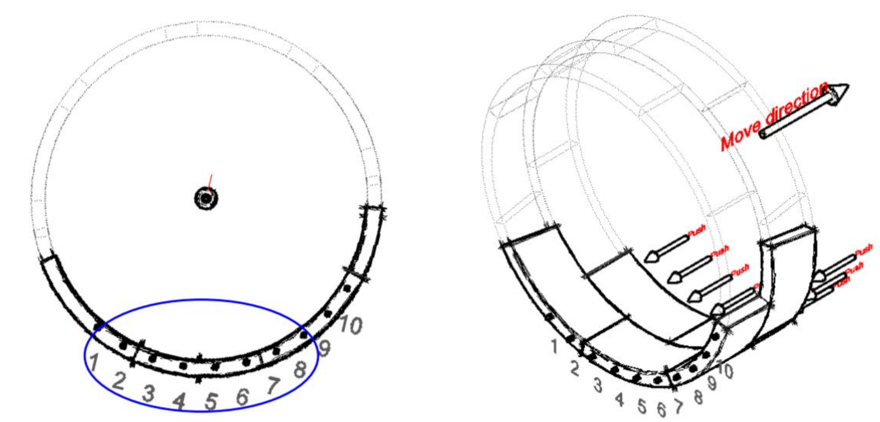


Figure 39 – Semi ring concrete support system for TBM [42].

8.15. Concrete Forming (36 Days)

With a rate of 25m2/hr for formwork it will take a total of 294 hours to complete 7357m2. This area consists of all the walls and platform levels of the station. 294 hours translates to 36 workdays to fulfill the job.

8.16. Concrete Pouring (34 Days)

With a pouring rate of 11 minutes/meter it will take 1348 hours to complete the entire pouring of concrete which would be a total of 168 workdays. Considering several pipes will be coming in to pour the concrete at this rate, it will take 34 workdays to complete with 5 lines. The concrete pouring will overlap with the concrete forming, as multiple levels will be worked on at the same time.

8.17. System Installation (30 Days)

It is assumed that to install the elevator, ticketing systems, HVAC ducts, HVAC mechanical equipment, electrical systems, lighting systems, AV Equipment, rail system installations and all the plumbing of the site it will take 30 days.

8.18. Construction of the Envelope (45 Days)

The building envelope will take 1.5 months to complete which will take place while system installation is undergoing [43]. The building envelope includes items like roofs, windows, doors, insulation, other control elements for outdoor weather along with the pathway to the GO station [44].

8.19. Finishing Work (10 Days)

Finishing work will be assumed to take 10 days, which may include items like painting, placing tiles where necessary etc. and will be accounted for after the completion of the envelope.

8.20. Commissioning and Testing of the Rails (10 Days)

It is assumed testing and commissioning of the rails will take a total of 10 days. It is assumed that no complications were indicated.

8.21. Landscaping (5 Days)

We will be adding trees to the entry ways of the station to make it appealing, which will at most be assumed to take 5 days.

8.22. Clean Up (5 Days)

Throughout the entirety of the construction of the subway station, it will be a priority to ensure clear workspace and clearing of materials when not needed. This will ensure that as we conclude the project there will be minimal cleaning remaining. We allotted 5 days to wrap up any remaining clean up required post finishing.

8.23. Demobilization (5 Days)

Demobilization will seek the removal of all the temporary facilities that were installed on site (temporary trailers for office site and change room, portable toilets) and will have the site boundary removed from the perimeter of the area to make the area accessible to the public now.

8.24. Contingency

There will be a total of 14 statutory holidays, and we assumed roughly 6 snow days to take place during the entire construction timeline. A total of 20 days was added at the end of the timeline to account for the statutory holidays and snow days.

8.25. Total Construction Timeline

To conclude construction will commence on January 2nd, 2024, and will end on August 6th, 2025, totaling 582 days or roughly 1.6 years. Refer to figure 40 below for the Gantt chart demonstration.

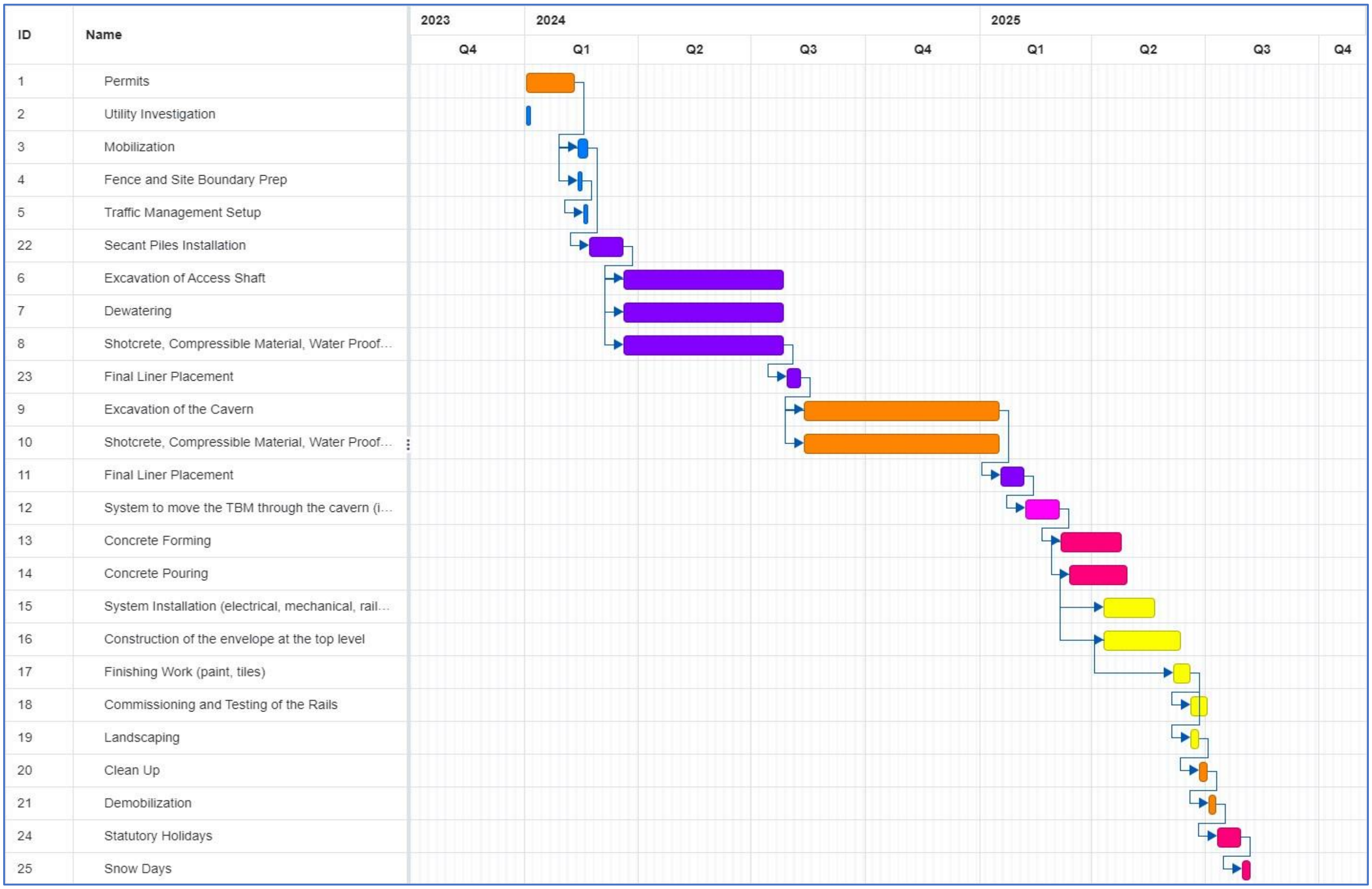


Figure 40 – Gantt chart to show the entire construction timeline.

9. Construction Costs

9.1. Permits

Assumption of permit fee is calculated based on the following formula which is mentioned in the City of Toronto's website [45].

$$Permit\ fee = Service\ Index * Area$$

In this case our service index is based on group A (Assembly Occupancies) classified as transit stations which is equal to $22.14 [x]. Our estimated floor area in m2 of work involved is about 6902.37 m2. To see detailed calculations for this number please look at appendix J.

This means the Permit fee for the site will be around $152,818.47. Subsequently, a payment of $3,177.50 payable to the "Region of Peel" for processing a full site plan application.

This brings us to **$155,995.9718** for the permit fees.

9.2. Utility Investigation

The utility Investigation price is assumed to be $4 over the Construction area and we will be using companies such as T2ue who have experience in the GTA for the task [46]. This will bring the costs to 8 * 1,236.87 = **$9,894.96**

9.3. Mobilization

In terms of Mobilization the major costs consist of site office implementation which would cost around $30,000. In addition, initial site cleaning will cost us 0.15 per sqft[47]. The mobilization area is 2,000 m2 which brings the cleaning price to $3,230. So, for mobilization we have an estimated cost of **$33,230**.

9.4. Fence and site boundary preparation

The fencing will be placed on the boundary of the site. The price to rent the fence is $0.7 per linear foot per month [48]. The boundary of the site is 528.7m and we assume an 18months construction period. This brings us to **$21,855.708** for fence rental.

9.5. Traffic Management

For the traffic management aspect, we will hire 2 flaggers for the entire project to oversee the deliveries throughout. Each flagger will be paid 20$/hr [49]. The flaggers will work for a total of 9 months throughout. This results in a total cost of $57,600 for the flaggers. A police officer will also be required during specific times of the project such as mobilization and demobilization. The officer will cost us about $150/hr and they will be involved during 2 months of the project [50]. The total office cost will be $72,000. We are estimating a $5,000 fee as well for printing and placing of the traffic management signage throughout the site. The total traffic management cost comes down to **$134,600**.

9.6. Excavation

9.6.1.Shaft excavation

The average excavation machinery cost per m3 is about $230 and labor cost will be 250[51]. Our shaft excavation volume will be 24,895m3. We will also be using two different excavators for the shaft. This leads to a total shaft excavation price of **$23,899,200**.

$$Shaft\ excavation\ volume = 17.1 * 17.1 * pi * 27.1 = 24{,}895$$

9.6.2. Cavern excavation

Using the same per m3 costs as before we will add premiums as the construction of the Cavern is a more complex task therefore the machinery and labor need to have higher skill. This brings us to an estimate of $1600 per m3 of excavation. The cavern excavation volume is 30,324m3. The total cost for cavern excavation will be **$48,518,400**.

$$Cavern\ excavation\ volume = 202.16 * 150 = 30{,}324$$

9.7. Shoring

The shoring cost for the shaft excavation will be comprised from secant piles and shotcrete. The volume of concrete is 1,518m3 as mentioned previously. The price of concrete is $241 assuming 25MPa [52]. We will also assume $170 per m3 for placing the concrete [53]. In addition, the drilling and excavation of the wholes will cost us $1000 per m3 for both renting the machinery and labor. This brings the secant pile implementation cost to **$2,141,898**. The shotcrete portion of the access shaft will be calculated in the next section.

9.8. Shotcrete, compressible material, water proofing and Liner

The total surface area where we need to apply compressible material and waterproofing is 9,672 m2. The price per m2 for the waterproofing and compressible material is estimated to be $900 [54]. We will also assume a $600 placement fee for each. This gets translated into a total cost of **$14,507,331.**

The total shotcrete volume of the access shaft and cavern is 2,648m3. For the price of concrete, we will assume a $150 dollar premium. This translates into $391 for the shotcrete and the placing fee of $700 as we will require more skilled workers. In addition, we assume another $2,000 for the supply and installation of the cavern liner. This translates into **$ 8,184,968** for the shotcrete and cavern liner aspect of the project.

9.9. System to move the TBM through the tunnel

As mentioned previously a semi ring support system will be implemented to move the TBM through the station. Implementation of such a system could be costly as semi ring support components need to be specifically built for the device and be pre-installed in the station. To have a conservative estimate for this section and as it requires specific design, we are allocating **$40,000,000** to this task.

9.10. Concrete forming and placing

For the forming and placing of the concrete we first need to find the total volume of the work in the project. The total concrete volume for the station which was calculated based on the designed dimensions of the station is 7357m3. The price of the concrete is $241 per m3, if we assume a combination of 20,25 and 30 MPa has been used. For the forming and pouring of the concrete we assume $600 respectively. This brings us to a total concrete forming and placing cost of **$12,374,474**.

9.11. System Installation

In terms of system installation, we need to implement everything related to elevator installation, ticketing system, HVAC ducts, HVAC mechanical equipment, electrical systems, lighting systems, AV Equipment, rail system installation and all the plumbing of the site. All these components combined will cost us about **$350,000,000** [55]. This cost incorporates everything from supply of equipment and material to labor for implementation.

9.12. Envelope

For the envelope of the above ground structure along with the pathway to go station we are assuming $340 per m2 [56] for supply of the materials and $900 for delivery, labor and installation. The area for the envelope of above ground structure assuming the pathway height is 3 meters with a length of 102.73m is 1062 m2. This results in **$1,316,880** for the above ground structure cost.

9.13. Finishing work

The finishing component of the excavation consists of all the tile installation, final paint, wall finishings, artwork installation and furnishing of the subway station. All the finishing work along with material supply, labor and installation will cost about $**10,000,000**.

9.14. Commissioning and testing of the rails

The commissioning and testing of the rail system along with the systems within the building will cost **$5,000,000.** It is good to note that any permits and final signatures are included in this cost estimate.

9.15. Landscaping

The landscaping cost for the project, which is focused on the area surrounding the main entrance, will cost around $2,000,000. This will include any pavement work along with trees and benches surrounding the construction area.

9.16. Clean up

The total final cleaning area assuming the exterior of the station will also be cleaned is about 8000m2. Assuming a $3 for full cleaning per m2, we are looking at a total final cleaning cost of **$24,000**.

9.17. Demobilization

The demobilization cost is assumed to incorporate items such as removing the fence, removing the equipment, and demobilizing the site office. The estimated cost for this section is **$30,000**.

9.18. Other considerations

9.18.1. Easements

As mentioned in previous sections the initial TBM route is being rerouted to accommodate for the construction of the proposed station. It is good to note, due to this change the TBM will be crossing underneath a residential area which means there needs to be conversations with the owners to get permission for this change. **$100,000,000** has been allocated to address this process.

9.18.2. Overhead

The total overhead assumed for the project is around 10%. This rate will be applied to final cost to account for the ongoing business expenses.

9.18.3. Construction Bond

A 1% construction bond has been assumed to protect the project against any adverse event throughout the construction.

9.18.4. Contingency

A 10% contingency is applied to account for any unexpected issues within the construction duration.

9.19. Final cost summary

The table below summarizes the total cost for the project and gives a general overview of the costs.

Table 14 – Cost Summary

Task	Cost
Permit fee	$152,818
Processing fee	$3,178
Permits	**$155,996**
Utility Investigation	**$9,895**
Site Office Mobilization	$30,000
Initial Site Cleaning	$3,230
Mobilization	**$33,230**
Fence and site boundary	**$21,856**
Flaggers	$57,600
Police Officer	$72,000
Traffic Management Signage	$5,000
Traffic Management	**$134,600**
Access Shaft Excavation	$23,899,200
Cavern Excavation	$48,518,400
Excavation	**$72,417,600**
Shoring	**$2,141,898.00**
Compressible material and waterproofing	**$14,507,331.00**
Shotcrete and Cavern liner	**$8,184,968.00**
System to move TBM through tunnel	**$40,000,000.00**
Concrete Forming and Placing	**$12,374,474.00**
System Installation	**$350,000,000.00**
Envelope	**$1,316,880.00**
Finishing work	**$10,000,000.00**
Commissioning and testing	**$5,000,000.00**
Landscaping	**$2,000,000.00**
Clean up	**$24,000.00**
Demobilization	**$30,000.00**
Easements	**$100,000,000.00**
Project Cost Before over head and contingency	$618,352,728
Overhead	$61,835,272.76
Construction Bond	$6,183,527.28
Contingency	$61,835,272.76
Total Project Cost	**$748,206,800.44**

10.References

[1] J. Do Rego, "Mississauga - Transportation," Townfolio, https://townfolio.co/on/mississauga/transportation (accessed Dec. 6, 2023).

[2] "Expand public transportation systems and offer incentives," U.S. Department of Transportation, https://www.transportation.gov/mission/health/Expand-Public-Transportation-Systems-and-Offer-Incentives (accessed Dec. 6, 2023).

[3] A Study on the Estimation Method of Rock Load Applied to Concrete Lining Using Back Analysis - Scientific Figure on ResearchGate. Available from: https://www.researchgate.net/figure/Modified-Rock-Load-Classification-Table-of-TerzaghiDeere-et-al-1970-Rose-1982_tbl1_264017372 [accessed 6 Dec, 2023]

[4] Canadian Commission on Building and Fire Codes, "Canadian Commission on building and fire codes," Canada.ca, https://nrc-publications.canada.ca/eng/view/object/?id=515340b5-f4e0-4798-be69-692e4ec423e8 (accessed Dec. 6, 2023)

[5] "Ontario Building Code 2012." Government of Ontario, 2015

[6] "Storm sewer network - storm pipes," Open Data Catalogue, https://data.mississauga.ca/datasets/mississauga::storm-sewer-network-storm-pipes/explore?location=43.569058%2C-79.668287%2C16.85%C2%A0 (accessed Dec. 6, 2023).

[7] "Development Requirements Manual SECTION 2 - DESIGN REQUIREMENTS ." City of Mississauga Transportation and Works Department , Sep. 2016

[8] "Sanitary Sewer Design Criteria," Region of Peel, https://www.peelregion.ca/public-works/design-standards/pdf/sanitary-sewer-design-criteria.pdf%C2%A0 (accessed Dec. 6, 2023).

[9] "Settlements due to tunnel construction [GSE] - Plaxis Tutorial Manual 2D." Plaxis 2D - Bentley Communities

[10] P. J. Hannigan, F. Rausche, G. E. Likins, B. R. Robinson, and M. L. Becker, "Design and Construction of Driven Pile Foundations – Volume I." U.S. Department of Transportation Publication Federal Highway Administration , Jul. 2016

[11] B. C. Crawford, "NRC Publications Archive - Foundation movements." National Research Council Canada, Division of Building Research, Apr. 1972

[12] "Eurocode 7: Geotechnical design- Part 1: General rules." Europeans Committee for Standardization, Nov. 2004

[13] Silvana Micic and K. Y. Lo, "Laboratory Swell Testing of DGR-2 Core." Intera Engineering , Jun. 2010

[14] "SUBWAY STATION ARCHITECTURE DESIGN STANDARD." Metrolinx, Nov. 2020

[15] "DS-04 GO STATION ARCHITECTURE DESIGN STANDARD." Metrolinx, Jul. 2019

[16] "Transit toronto," The Toronto Rocket Trains (T35A08) - Transit Toronto - Content, https://transittoronto.ca/subway/5511.shtml (accessed Dec. 6, 2023).

[17] “T series (Toronto subway),” Wikipedia, https://en.wikipedia.org/wiki/T_series_(Toronto_subway) (accessed Dec. 6, 2023).

[18] G. Grasselli and T. de Boer, “CME 525 Tunnel Design Problem Assignment 2: Twin-Bore Tunnel Design Report.” Toronto, Oct. 2023

[19] H. Al-maamori, M. El Naggar, and S. Micic, “A Compilation of the Geo-Mechanical Properties of Rocks in Southern Ontario and the Neighbouring Regions.” Open Journal of Geology, 2014

[20] “Technical Manual for Design and Construction of Road Tunnels — Civil Elements .” U.S. Department of Transportation Federal Highway Administration , Dec. 2009

[21] “ DYWIDAG THREADBARS - TECHNICAL DATA.” DSI Canada, Surrey, BC, Aug. 2014

[22] C. Li, “Principles of rockbolting design.” Journal of Rock Mechanics and Geotechnical Engineering., 2017

[23] T. Lam and S. Usher , “Regional Geomechanics – Southern Ontario .” Nuclear Waste Management Organization and AECOM Canada Ltd. , Mar. 2011

[24] G. E. Neu, “Tunnel Linings,” in Interaction modeling in Mechanized Tunneling, SPRINGER INTERNATIONAL PU, 2023, pp. 253–327

[25] K. Wu, X. Zheng, N. Zhao, and Z. Shao, “Effect of compressible layer on time-dependent behavior of soft-rock large deformation tunnels revealed by mathematical analytical method,” Applied Mathematical Modelling, https://www.sciencedirect.com/science/article/abs/pii/S0307904X23004614 (accessed Dec. 6, 2023).

[26] “File a work notice,” Ontario.ca, https://www.ontario.ca/page/file-work-notice (accessed Dec. 6, 2023).

[27] “SHAFT CONSTRUCTION,” Bradshaw Construction Corporation, http://www.bradshawcc.com/shaft_info.php (accessed Dec. 6, 2023).

[28] “Tunnel,” Wikipedia, https://en.wikipedia.org/wiki/Tunnel (accessed Dec. 6, 2023).

[29] “SECANT PILE WALLS.” Avopiling Foundation Specialists

[30] A. McGlenn, “Shaft/Portal Design & Construction.” McMillen Jacobs Associates, Sep. 2014

[31] “Feasibility Study and its importance in project management,” Simplilearn.com, https://www.simplilearn.com/feasibility-study-article#:~:text=A%20feasibility%20study%20is%20a%20preliminary%20exploration%20of%20a%20proposed,%2C%20legal%2C%20and%20environmental%20considerations. (accessed Dec. 6, 2023).

[32] City of Mississauga, “Apply for a noise exemption,” City of Mississauga, https://www.mississauga.ca/services-and-programs/home-and-yard/noise-and-lighting/noise-exemptions/ (accessed Dec. 6, 2023).

[33] “Storm sewer use by-law,” City of Mississauga, https://www.mississauga.ca/publication/storm-sewer-by-law/ (accessed Dec. 6, 2023).

[34] “Building Transit Faster Act, 2020, S.O. 2020, c. 12,” Ontario.ca, https://www.ontario.ca/laws/statute/20b12#BK1 (accessed Dec. 6, 2023).

[35] “Proposed Construction Fence By-law.” City of Toronto, 2000

[36] “Construction site fences advantages,” MSK Construction, https://www.mskconstruct.co.za/construction-site/why-do-construction-workers-put-up-fences-around-the-construction-site/ (accessed Dec. 6, 2023).

[37] “Secant Pile Walls & Cofferdams,” CDI - Construction Drilling, https://condrill.ca/services/shoring-walls-cofferdams-cut-off-walls/secant-pile-walls-cofferdams/#:~:text=Secant%20pile%20walls%20and%20coffer,underground%20parking%20of%20large%20buildings (accessed Dec. 6, 2023).

[38] “Wheel Loaders Buyer’s Guide,” Construction Equipment Guide, https://www.constructionequipmentguide.com/buyers-guide/wheel-loaders#:~:text=Compact%20machines%20typically%20can%20lift,flow%20capacity%20and%20machine%20weight. (accessed Dec. 6, 2023).

[39] “Shotcrete equipment,” Putzmeister, https://www.putzmeister.com/web/india/shotcrete-equipment (accessed Dec. 6, 2023).

[40] V. Nasri, “Waterproofing and Final Tunnel Liner - Tunneling Short Course .” University of Colorado , Sep. 2016

[41] “How long does it take to pour concrete?,” Quora, https://www.quora.com/How-long-does-it-take-to-pour-concrete (accessed Dec. 6, 2023).

[42] M. Bastami, M. Eftekhari, K. Ahangari, and GH. Sahami, “TBM Crossing of Metro Station Using Semi-Ring Support.” Ahab Construction Company , Tehran, Iran, Feb. 2019

[43] R. Meagher, “How to manage your own home building project,” BVM Contracting, https://www.bvmcontracting.com/blog/be-your-own-builder-gc-bvm-contracting (accessed Dec. 6, 2023).

[44] “What is your building envelope and how can you improve it?,” Clean & Polish Building Solutions, Inc. | Washington, D.C. | Building Maintenance, https://www.cleanandpolish.com/blog/what-is-your-building-envelope-and-how-can-you-improve-it/#:~:text=Building%20envelopes%20are%20typically%20made,included%20in%20the%20building%20envelope. (accessed Dec. 6, 2023).

[45] City of Toronto, “Building permit fees,” City of Toronto, https://www.toronto.ca/services-payments/building-construction/apply-for-a-building-permit/building-permit-fees/ (accessed Dec. 6, 2023).

[46] “Subsurface Utility Engineering (sue),” T2 Utility Engineers, https://t2ue.com/services/subsurface-utility-engineering/ (accessed Dec. 6, 2023).

[47] A. Rubin, “Post-construction cleaning services in Toronto: A pricing guide,” Hellamaid, https://hellamaid.ca/construction-cleaning-guide/how-much-is-post-construction-cleaning-services-rates-in-toronto-a-pricing-guide/#:~:text=Average%20Post%2DConstruction%20Cleaning%20Rates%20in%20Toronto&text=Small%20Residential%20Projects%3A%20%240.20%20 (accessed Dec. 6, 2023).

[48] J. Bieda, “How much does it cost to rent construction fence?,” Modu-Loc, https://moduloc.ca/blog/how-much-does-it-cost-to-rent-construction-fence/ (accessed Dec. 6, 2023).

[49] “USA salary database: Compare & estimate salary for any job - ziprecruiter,” Zip Recruiter, https://www.ziprecruiter.com/Salaries (accessed Dec. 6, 2023).

[50] “Request a paid duty officer,” Toronto Police Service, https://www.tps.ca/services/request-paid-duty-officer/ (accessed Dec. 6, 2023).

[51] “A guide to excavation costs,” ProEst, https://proest.com/construction/cost-estimates/excavation-cost/#:~:text=considerations%20during%20excavation.-,How%20to%20calculate%20excavation%20costs,of%20the%20area%20per%20unit. (accessed Dec. 6, 2023).

[52] “2022 Ontario Price List.” Dufferin Concrete, Jan. 2022

[53] A. Azzan, “How much does concrete cost in 2024?,” Lawn Care Blog | Lawn Love, https://lawnlove.com/blog/concrete-cost/#:~:text=The%20typical%20cost%20of%20concrete,pour%20and%20install%20the%20concrete. (accessed Dec. 6, 2023).

[54] “Basement waterproofing cost Toronto,” RCC Waterproofing Toronto Wet Basement, https://rccwaterproofing.com/services-overview/service-pricing-guide/ (accessed Dec. 6, 2023).

[55] A. Levy, “Construction costs: Metro stations,” Pedestrian Observations, https://pedestrianobservations.com/2018/01/27/construction-costs-metro-stations/ (accessed Dec. 6, 2023).

[56] S. Mariani, F. Rosso, and M. Ferrero, “Building in Historical Areas: Identity Values and Energy Performance of Innovative Massive Stone Envelopes with Reference to Traditional Building Solutions. ,” Research Gate, https://www.researchgate.net/figure/Costs-per-square-meter-and-total-cost-of-the-envelope-solutions-of-the-different-case_tbl3_323111469 (accessed Dec. 6, 2023).

[57] “Air content (gravimetric) of concrete,” Iowa Government , https://www.iowadot.gov/erl/current/IM/content/340.htm#:~:text=Normal%20weight%20concrete%20is%20in,of%201.5%20lbs.%2Fcu. (accessed Dec. 6, 2023).

[58] “Building code requirements: Build or buy a tiny home,” ontario.ca, https://www.ontario.ca/document/build-or-buy-tiny-home/building-code-requirements (accessed Dec. 6, 2023).

[59] “What does a house weigh? Some mental heavy lifting,” The Seattle Times, https://archive.seattletimes.com/archive/?date=20041219&slug=homehay19 (accessed Dec. 6, 2023).

[60] “Video: Going loco for go’s new Tier 4 locomotives,” Metrolinx, https://www.metrolinx.com/en/news/video:-going-loco-for-go%E2%80%99s-new-tier-4-locomotives (accessed Dec. 6, 2023).

[61] “Go Train Quick Facts.” Metrolinx, Jun. 2013

[62] Collaborative online estimating, gantt &, takeoff applications., https://www.methvin.org/estimating-production-rates/excavation/trench-excavation (accessed Dec. 6, 2023).

[63] Google maps, https://www.google.com/maps/@43.5678115,-79.6666236,3a,75y,184.67h,90t/data=!3m7!1e1!3m5!1sP_BRCOCJLahuFtb48oyzTw!2e0!6shttps:%2F%2Fstreetviewpixels-pa.googleapis.com%2Fv1%2Fthumbnail%3Fpanoid%3DP_BRCOCJLahuFtb48oyzTw%26cb_client%3Dmaps_sv.tactile.gps%26w%3D203%26h%3D100%26yaw%3D184.67014%26pitch%3D0%26thumbfov%3D100!7i16384!8i8192?entry=ttu (accessed Dec. 6, 2023).

[64] “Secant or tangent (contiguous) piles,” Keller North America, https://www.keller-na.com/expertise/techniques/secant-tangent-piles (accessed Dec. 6, 2023).

[65] “Law document english view,” Ontario.ca, https://www.ontario.ca/laws/regulation/910213 (accessed Dec. 7, 2023).

11.Appendices

Appendix A – Borehole Data

Table 15: Borehole data surrounding Erindale Go.

BH Data Code	Soil Type	Depth	BH Data Code	Soil Type	Depth
BH2-16	Clayey silt, sand, gravel	0-0.8	BA-MW26-15	Sand and gravel	0-0.8
	Silty clay, sand, gravel	0.8-1.7		Silty clay till	0.8-2.4
	Silty Clay Till	1.7-2.3		Silty clay till	2.4-4.6
	Silty Clay Till / Shale	2.3-4.6		Georgian bay formation	4.6-23.1
	Georgian Bay Shale	4.6-24.5			
BH2-51	Gravelly sand, some silt	0-0.76			
	Silty clay, some sand	0.76-1.98			
	Silty clay till	1.98-4.01			
	Silty sand till	4.01-4.57			
	Clayey silt till/shale complex	4.57-6.1			
	Georgian bay shale	6.1-25.53			

Table 16: Rock quality designation values.

BH Data Code	RQD	BH Data Code	RQD	BH Data Code	RQD
BH2-16	54	BH2-51	38	BA-MW26-15	54
	60		23		88
	82		47		88
	92		51		97
	100		30		98
	100		66		85
	87		69		94
	98		58		98
	97		65		100
	100		76		100
	97		59		100
	97		78		90
	100		78		
Average	89.53		56.77		91

Appendix B: Load Calculations

Apartment

A distributive load of 2300 kg/m^3 [57] will be taken across the entire top of the building, representing the weight of concrete. The apartment is 9 storeys at 2.3 m each [58], bring the final load up to 47'610 kg/m^2 (467 kN/m^2). This load will then be multiplied by 1/3 representing the total space filled concrete in the structure, bringing the load down to 156 kN/m^2. This load will then be factored with a 1.25 factor [5], bring the final load up to 203 kN/m^2.

A live load of 1.9 kN/m^2 will be applied [4] in addition to the dead load. The factored load of 1.5 time the live load will bring this value up to 2.85. The final load will be dead plus live, equal to 206 kN/m. This load will be applied as a line load of 1m width since the foundation will be assumed to apply this load across the entire bottom.

Residential Homes

A distributed load of 13.2 kN/m^2 [59] will be taken for the two-storey residential homes that are around the excavation. Using google maps, a sliver will be taken perpendicular to the excavation, to represent a line load applied at the surface. This sliver will be 1 m wide allowing the distributed load to be created as a line load. Since the houses are similar in width, the width will be taken as constant for each of the residential house line loads.

Go Train

There is a go line running directly behind the excavation, that will place a 129,000kg load [60] over 20.73m [61]. This ends up with a line load of 6223 kg/m (61 kN/m).

Carloads

Erindale go has 714 spots available for use. These spots are over an area of 19.905 m^2. With the average car weight being taken as 18.21 kN, a distributed load of 363 kN/m^2 can be found.

Appendix C – Tunnel Shape Forces

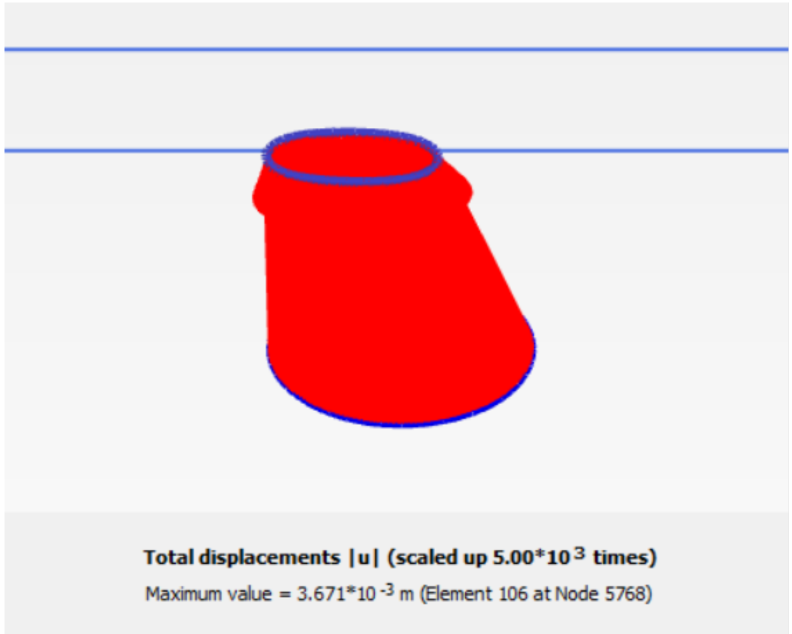


Figure 41 – Displacement values for more ellipse shape, Ko = 2.

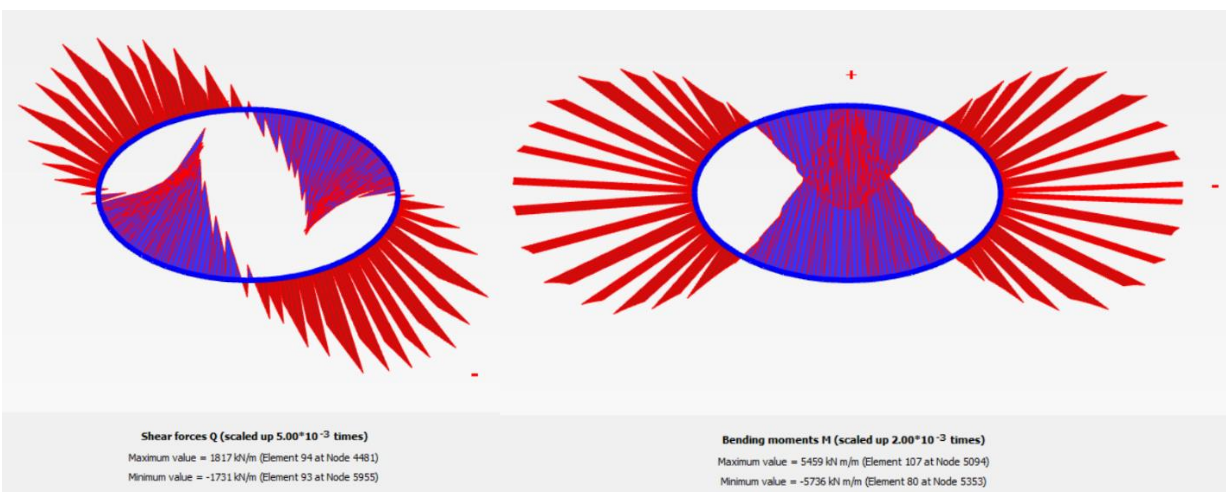


Figure 42 – Shear and moment values for more ellipse shape, Ko = 2.

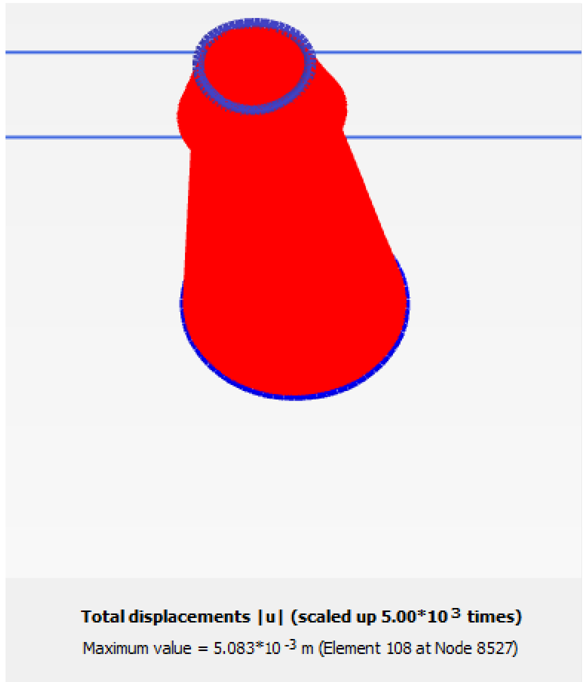


Figure 43 – Displacement values for more circular shape, Ko = 2.

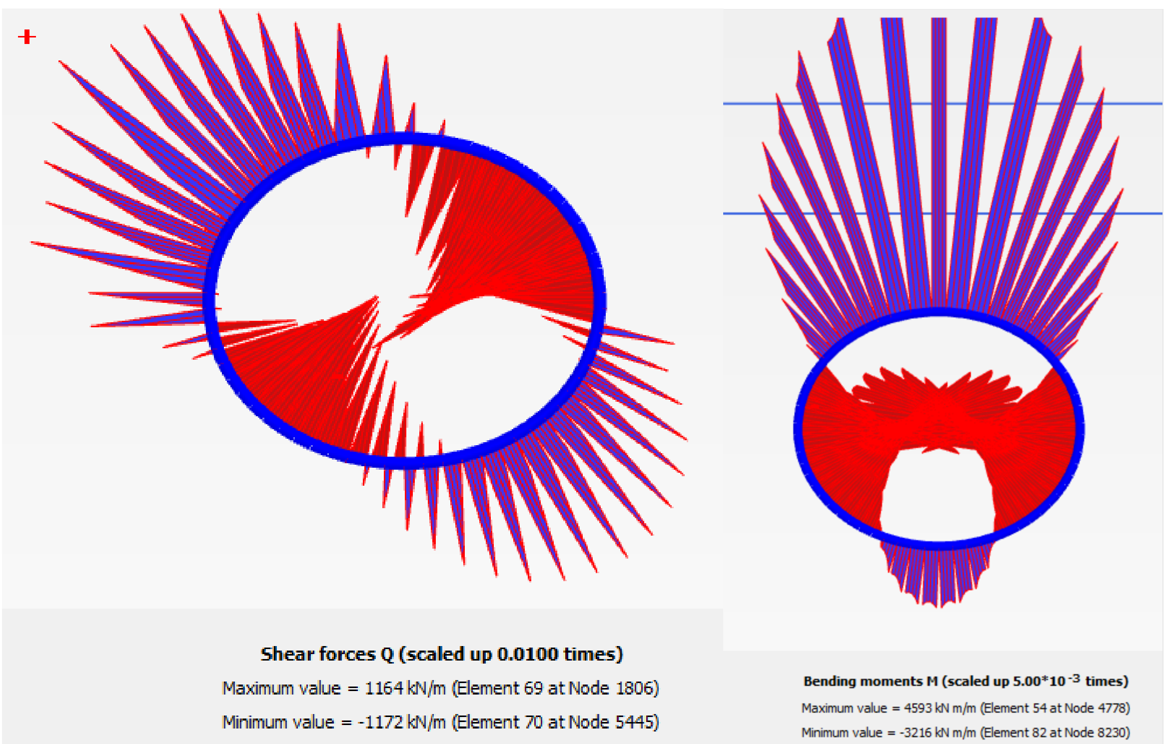


Figure 44 – Shear and moment values for more circular shape, Ko = 2.

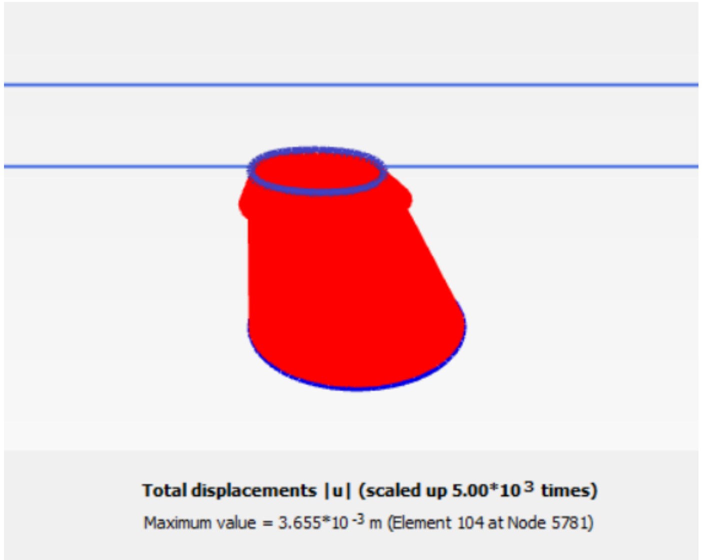


Figure 45 – Displacement values for ellipse shape, Ko = 3.

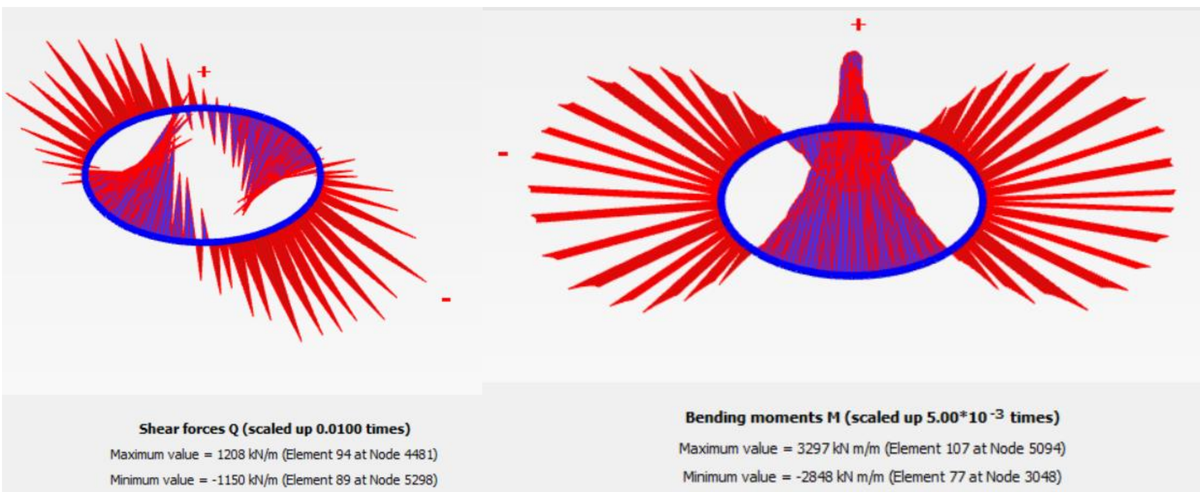


Figure 46 – Shear and moment values for more ellipse shape, Ko = 3.

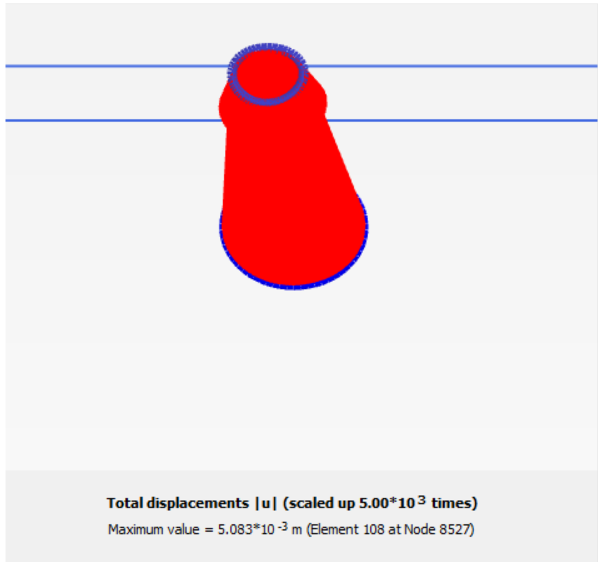


Figure 47 – Displacement and moment values for more circular shape, Ko = 3.

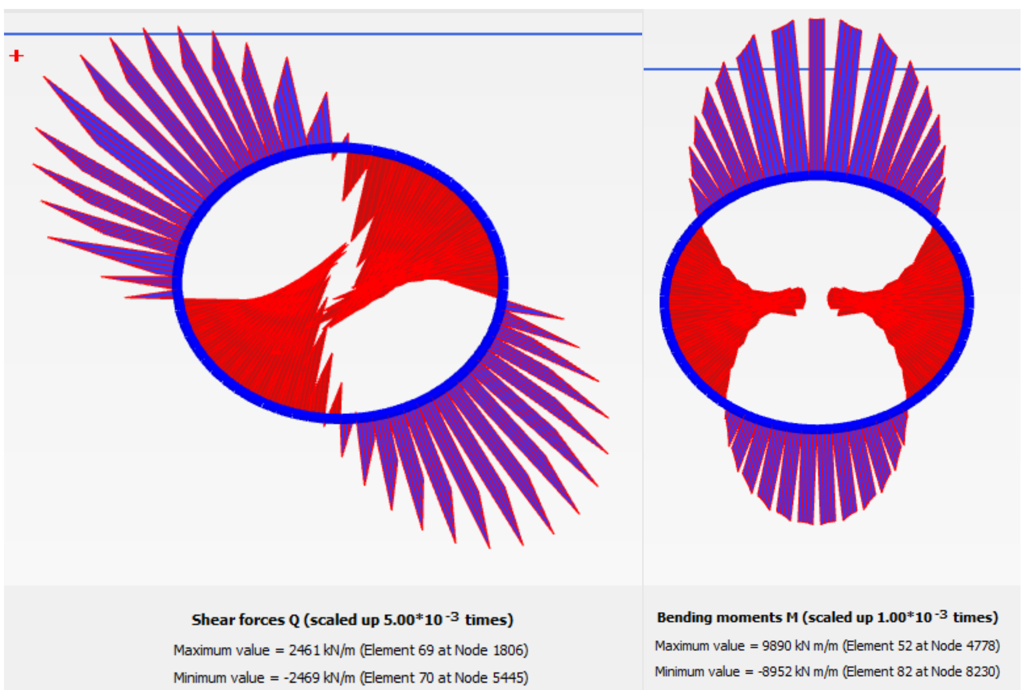
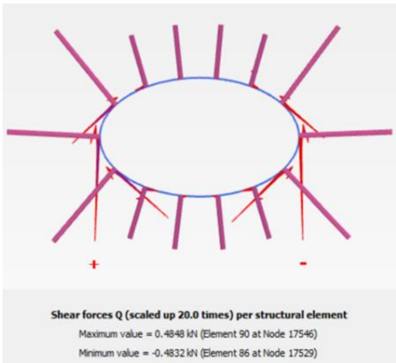
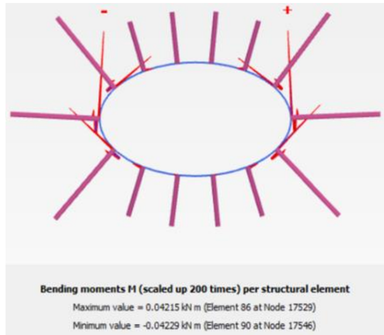


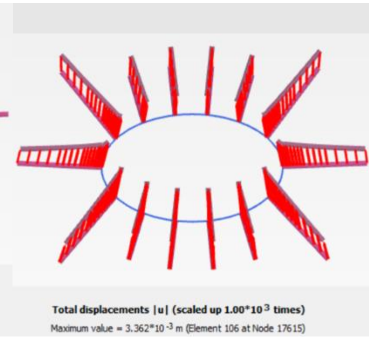
Figure 48 – Shear and moment values for more circular shape, Ko = 3.

Appendix D – Tunnel Reinforcement

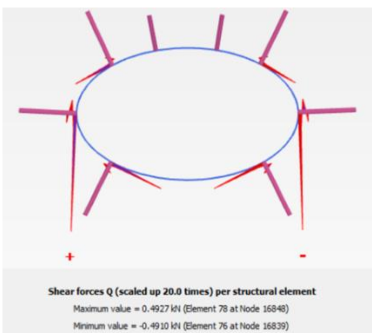
Case 1 **Figures 49-51**

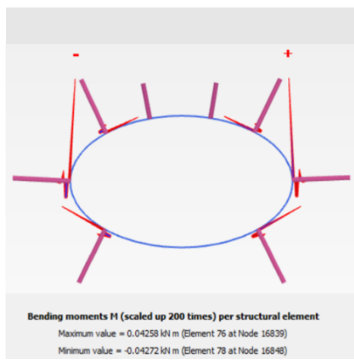


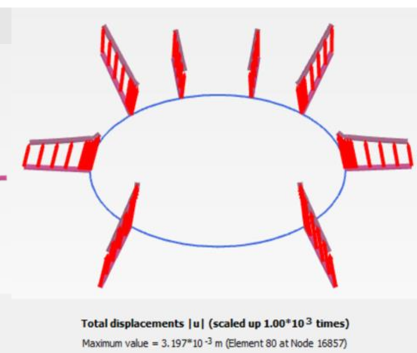




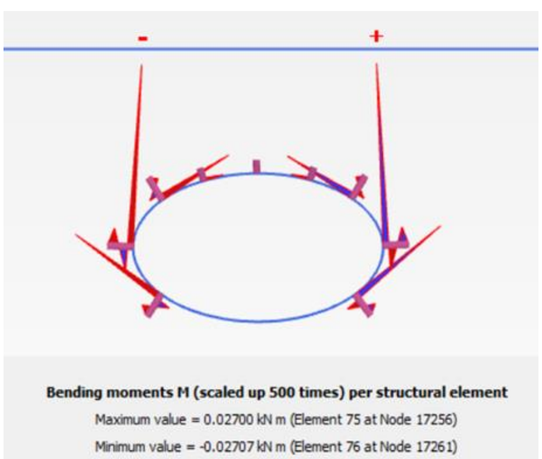
Case 2 **Figures 52-54**



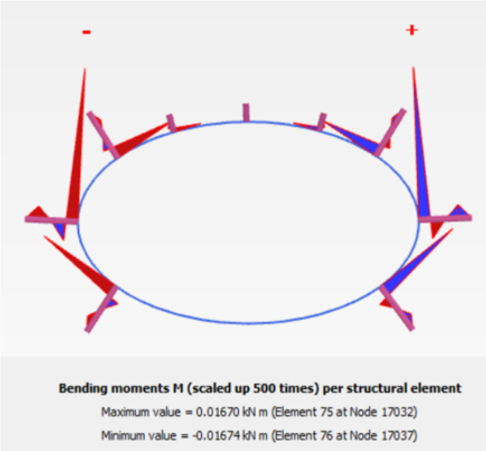




Case 3 **Figure 55**



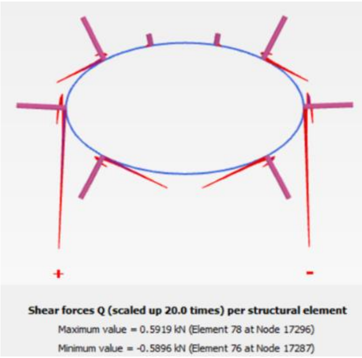
Case 4 **Figure 56**

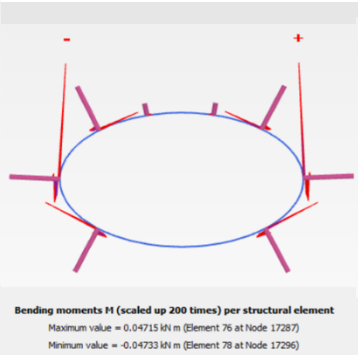


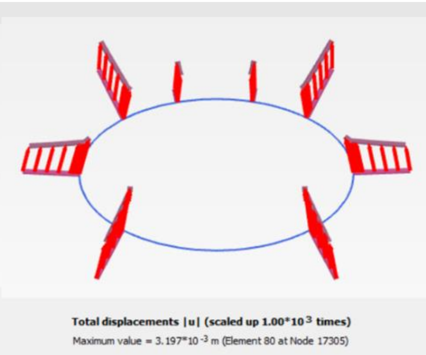
Case 5 – No improvements, screenshot not taken

Case 6 – Values same as Case 2

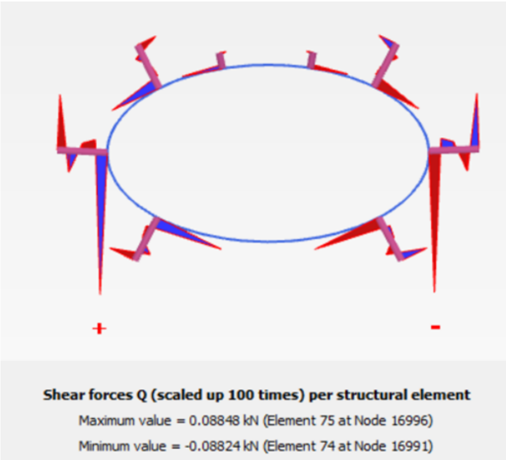
Case 7 – BEST **Figures 57-59**

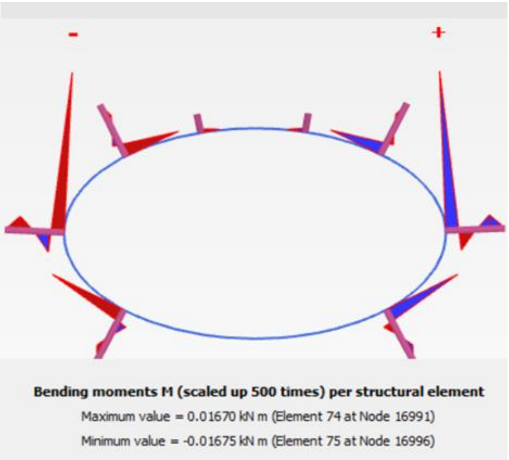




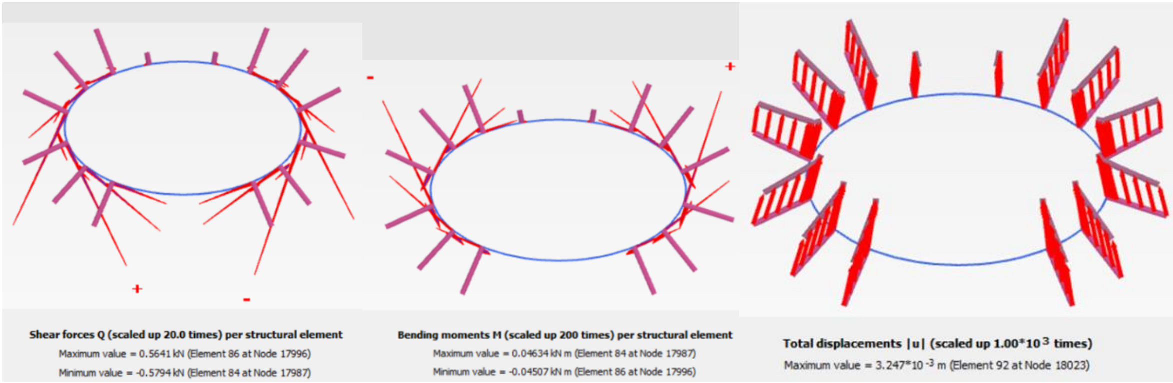


Case 8 **Figures 60-61**





Case 9 **Figures 62-64**



Appendix E – SEM Excavation

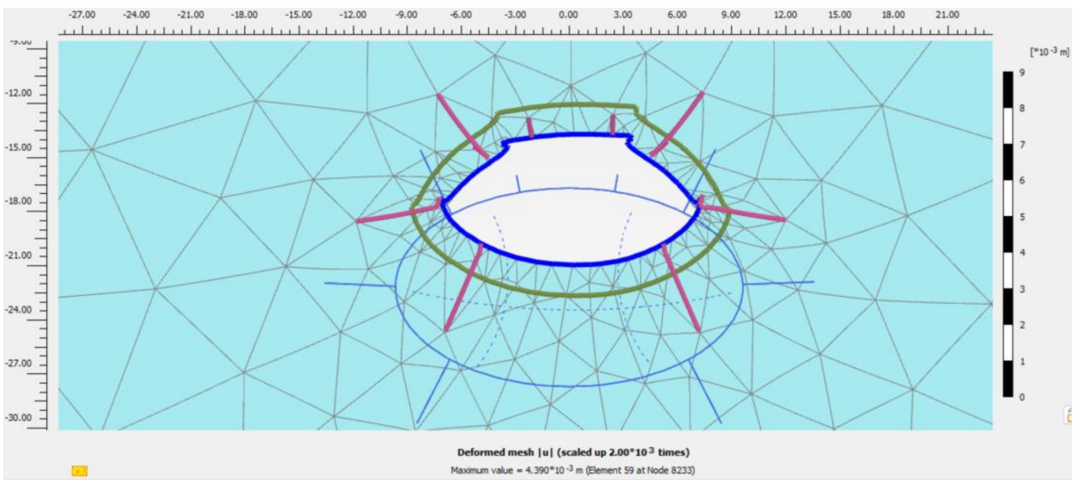


Figure 65 – Deformed mesh of the face of the excavation.

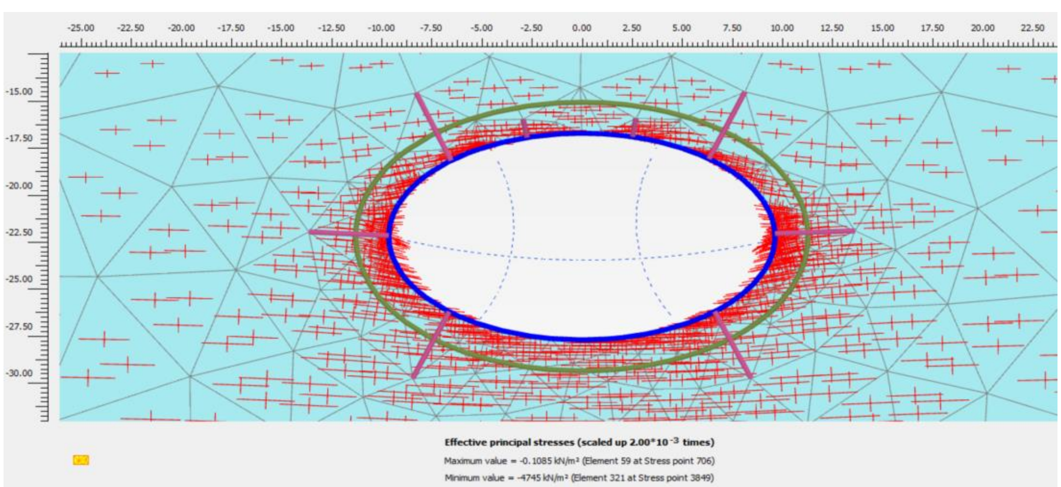


Figure 66 – The effective principal stresses of the excavation.

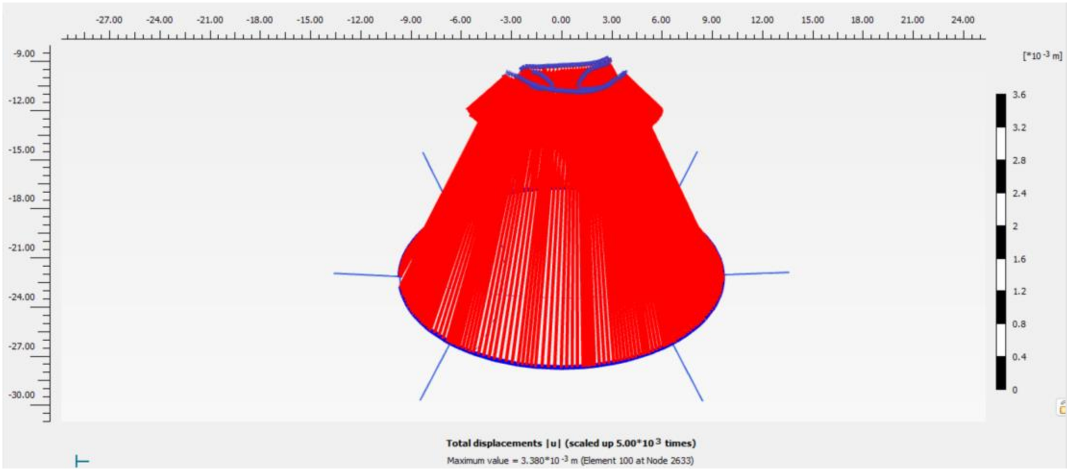


Figure 67 – Total displacements around the excavation.

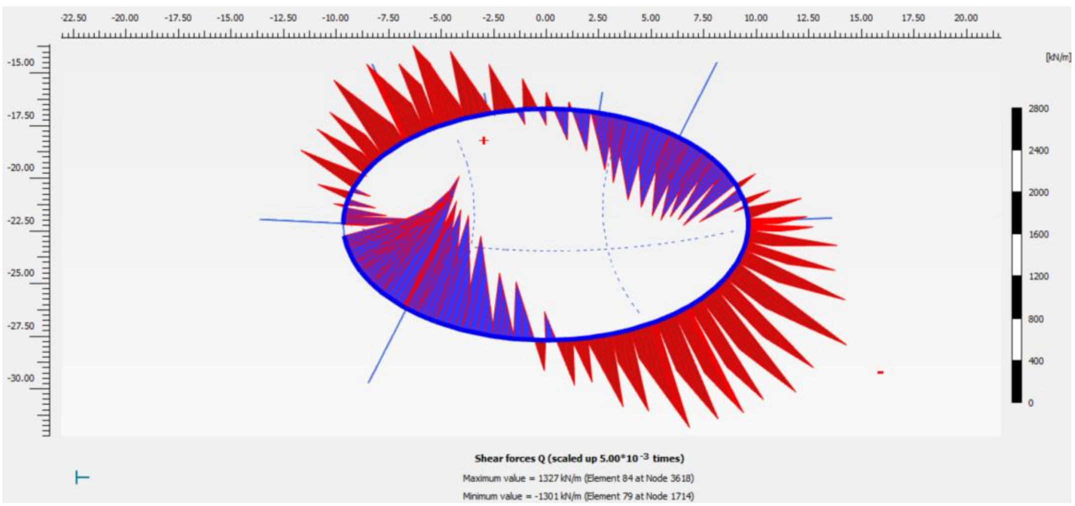


Figure 68 – Shear forces around the excavation.

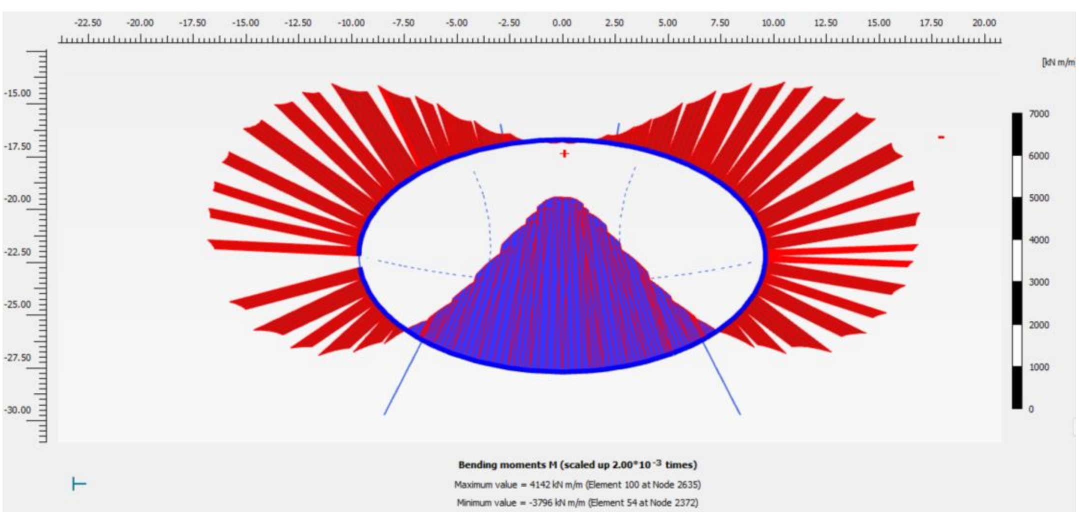


Figure 69 – Bending moments around the excavation.

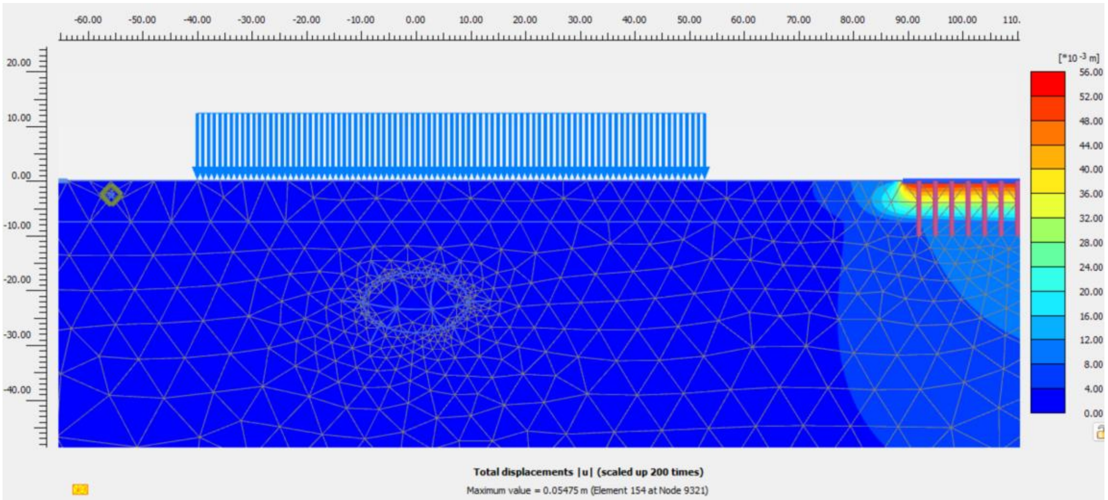


Figure 70 – Total displacement before the SEM excavation.

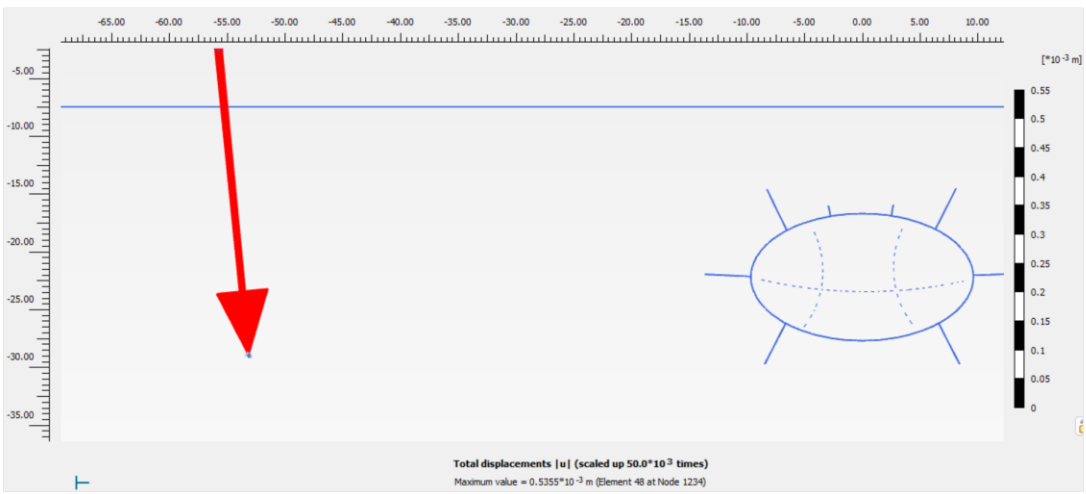


Figure 71 – Deformation on nearby sewer tunnel.

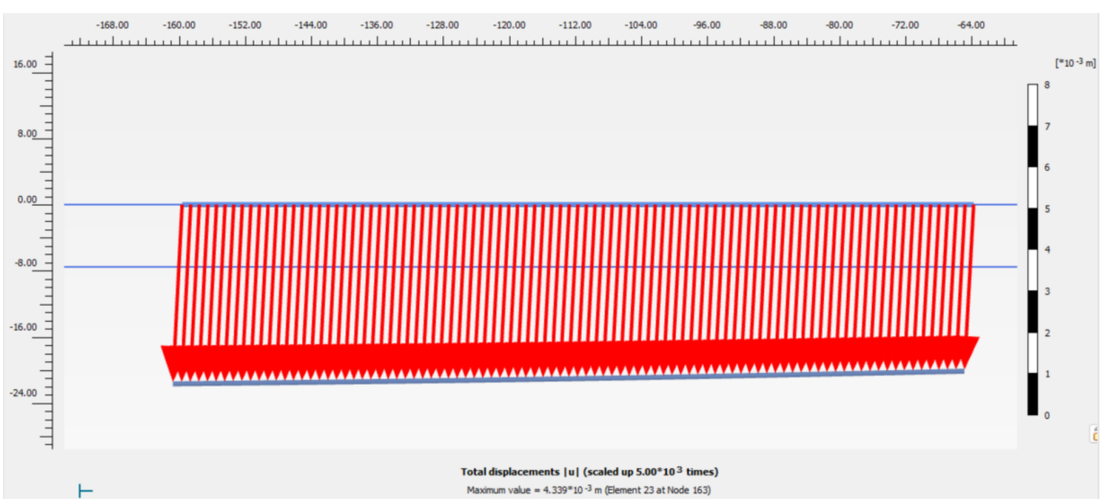


Figure 72 – Deformation on residential houses nearby.

Appendix F: Shaft design

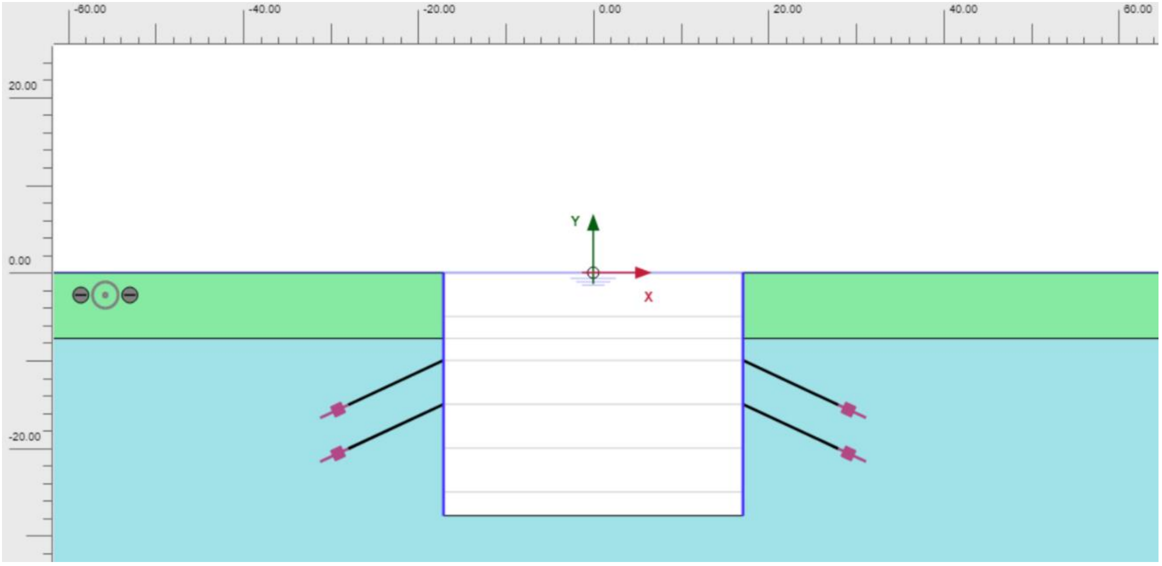


Figure 73 – Model of the circular shaft with tiebacks and shotcrete.

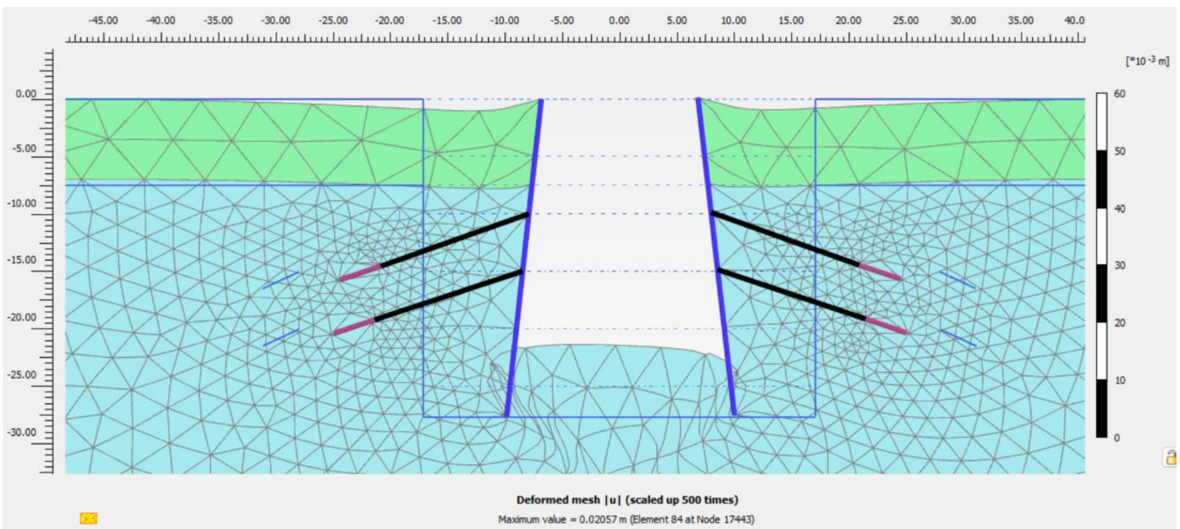


Figure 74 – Deformed mesh of the shaft with tiebacks and shotcrete.

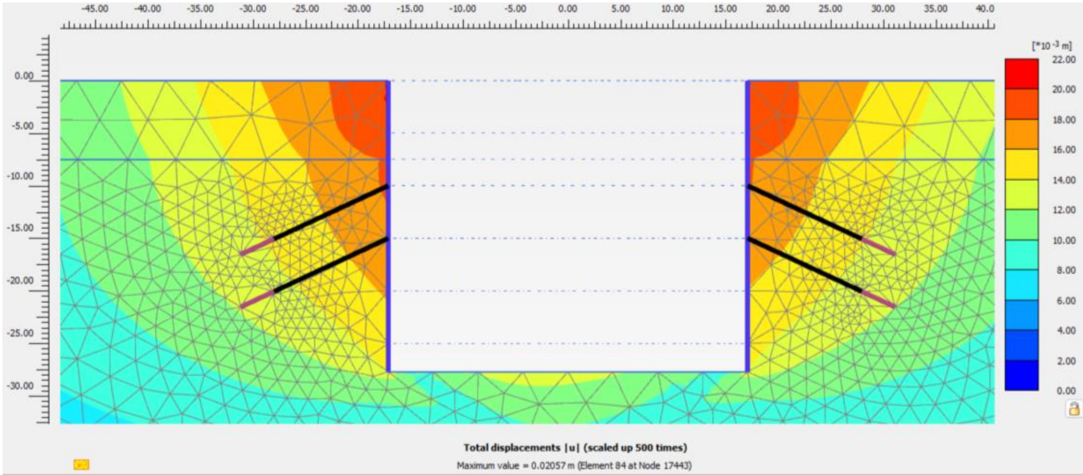


Figure 75 – Total displacement of the shaft with tiebacks and shotcrete.

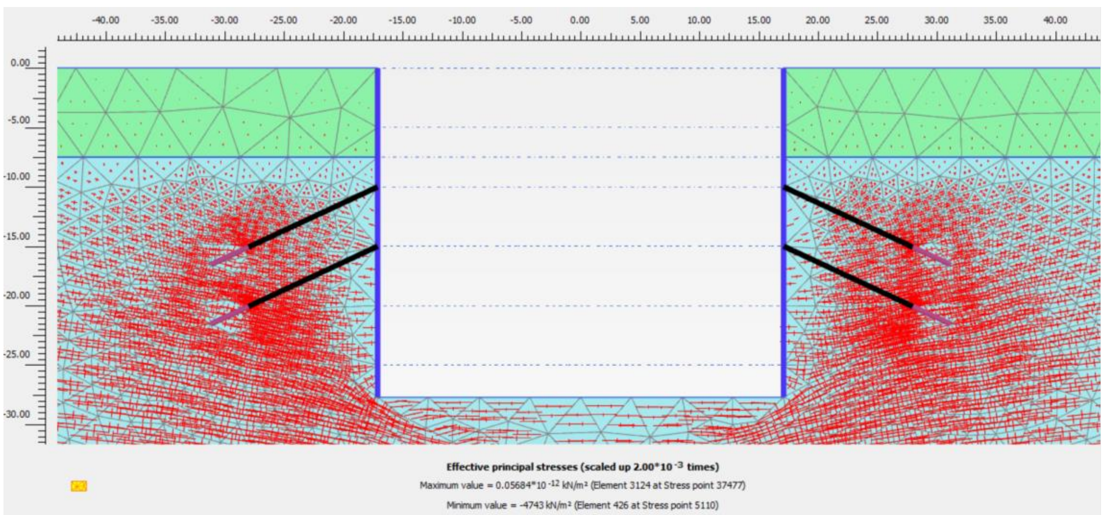


Figure 76 – Effective principal stresses of the shaft with tiebacks and shotcrete.

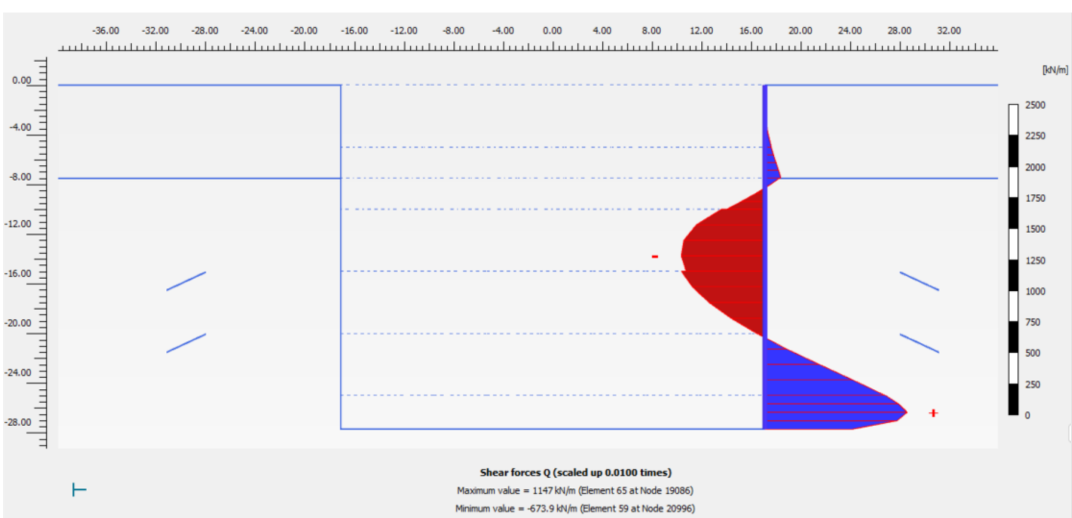


Figure 77 – Shear forces of the shaft with tiebacks and shotcrete.

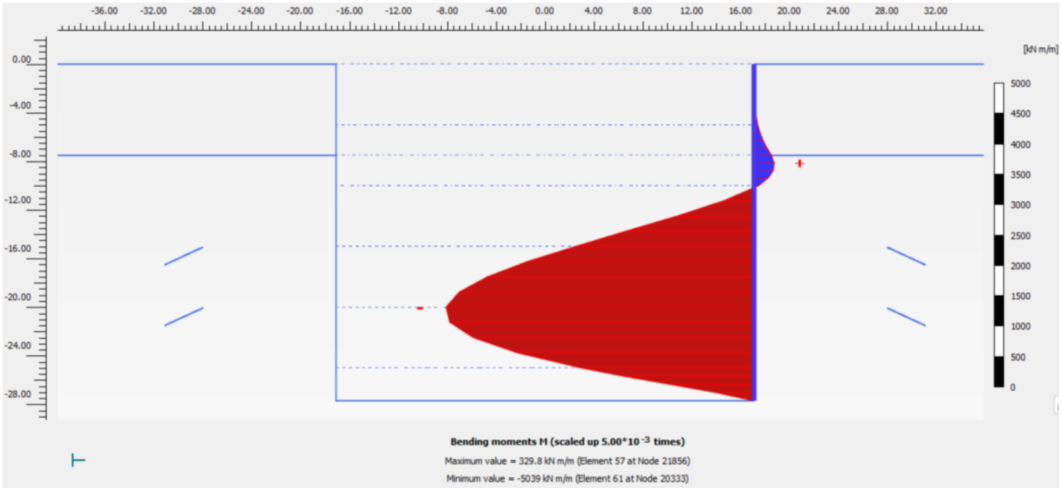


Figure 78 – Bending moment of the shaft with tiebacks and shotcrete.

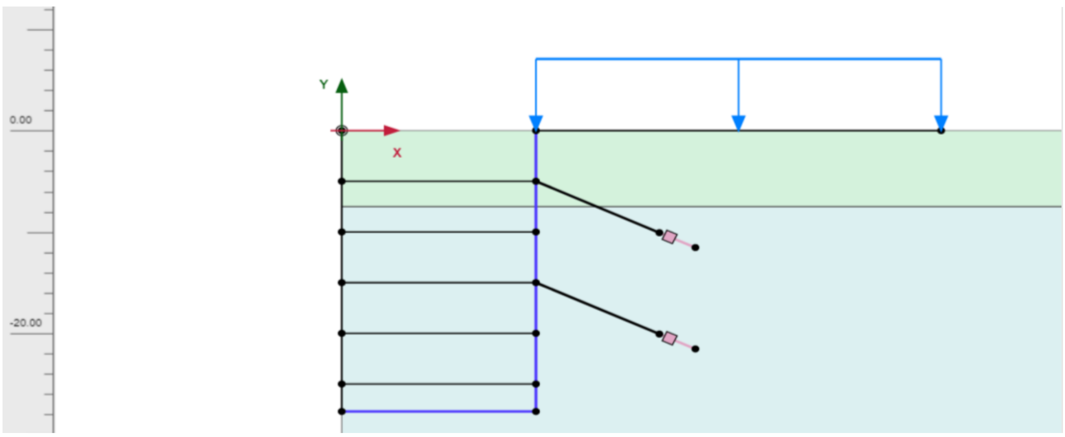


Figure 79 – Asymmetric model of the shaft with tiebacks and shotcrete.

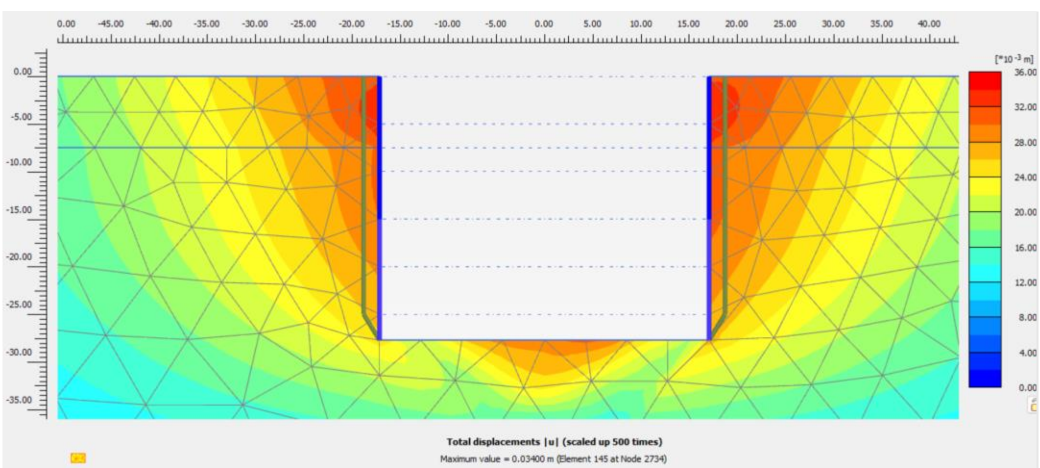


Figure 80 – Total displacement of the shaft with secant piles and shotcrete.

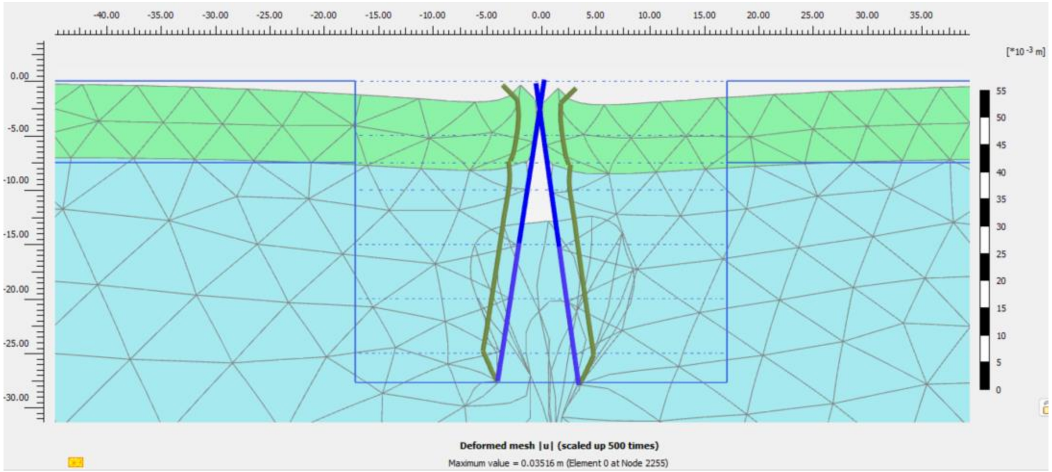


Figure 81 – Deformed mesh of the shaft with secant piles and shotcrete.

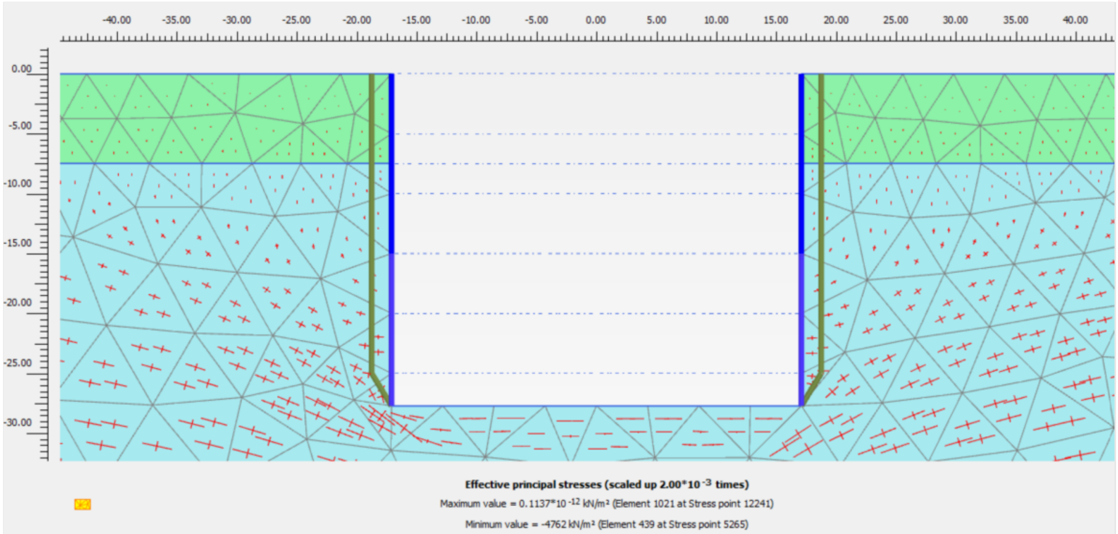


Figure 82 – Effective principal stresses around the shaft with secant piles and shotcrete.

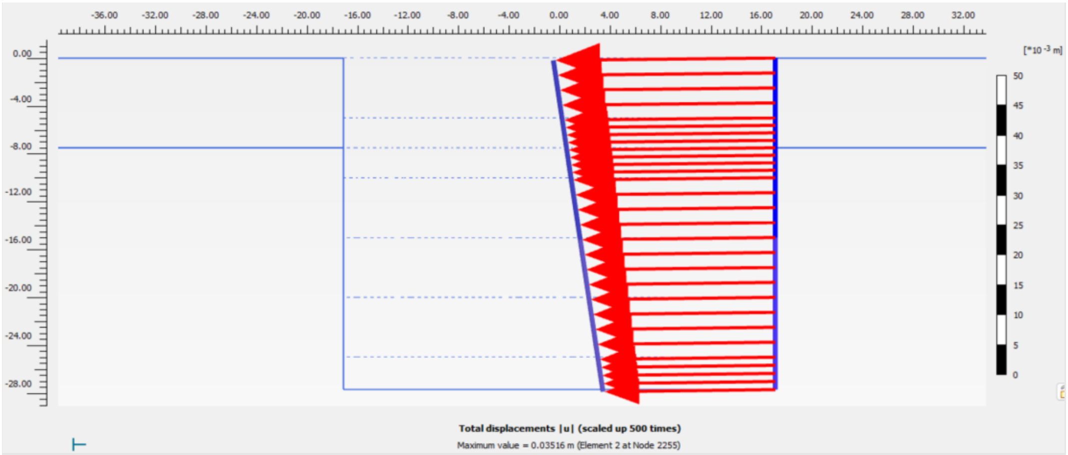


Figure 83 – Total displacement around the shaft with secant piles and shotcrete.

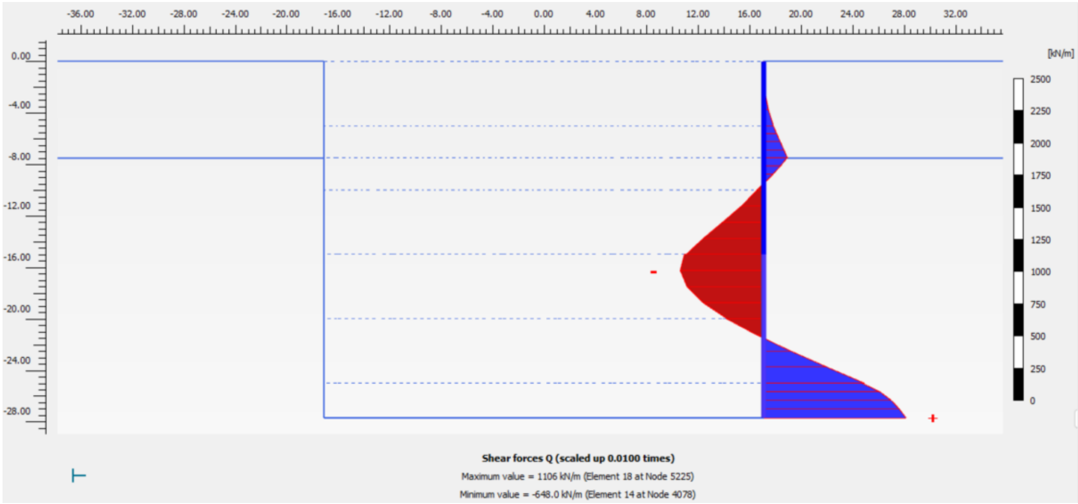


Figure 84 – Shear forces around the shaft with secant piles and shotcrete.

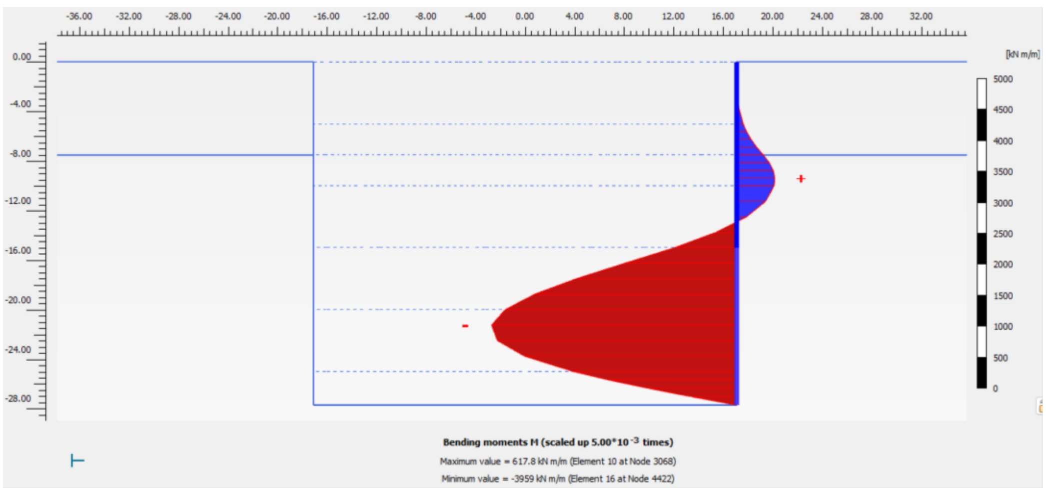


Figure 85 – Bending moments around the shaft with secant piles and shotcrete.

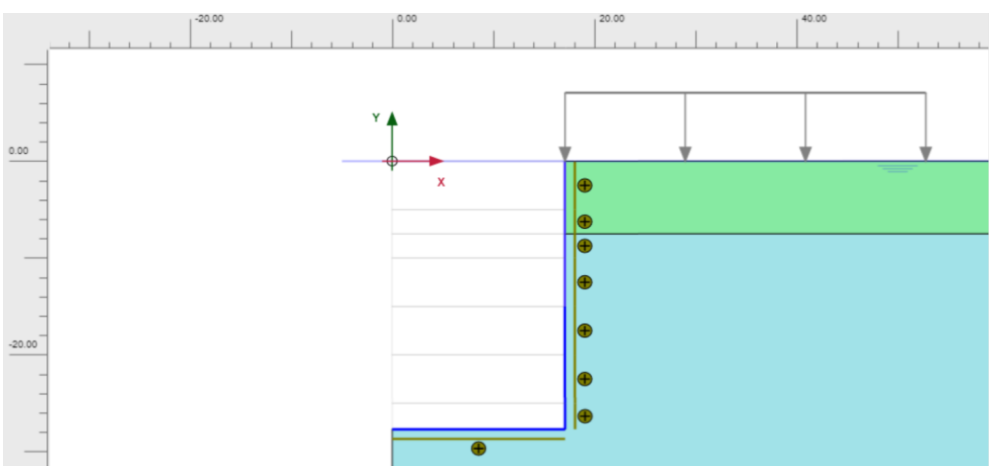


Figure 86 – Plaxis asymmetric model of the secant piles with shotcrete.

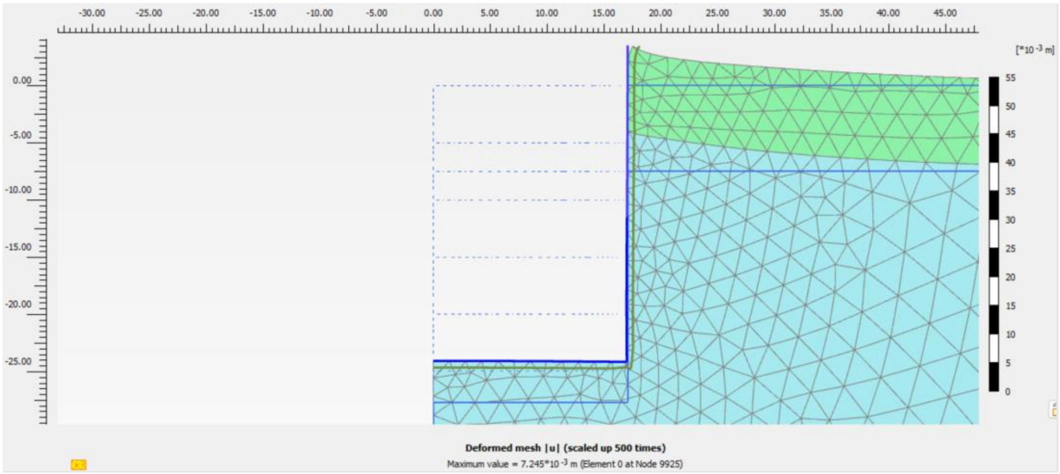


Figure 87 – Deformed mesh of the shaft with secant piles and shotcrete.

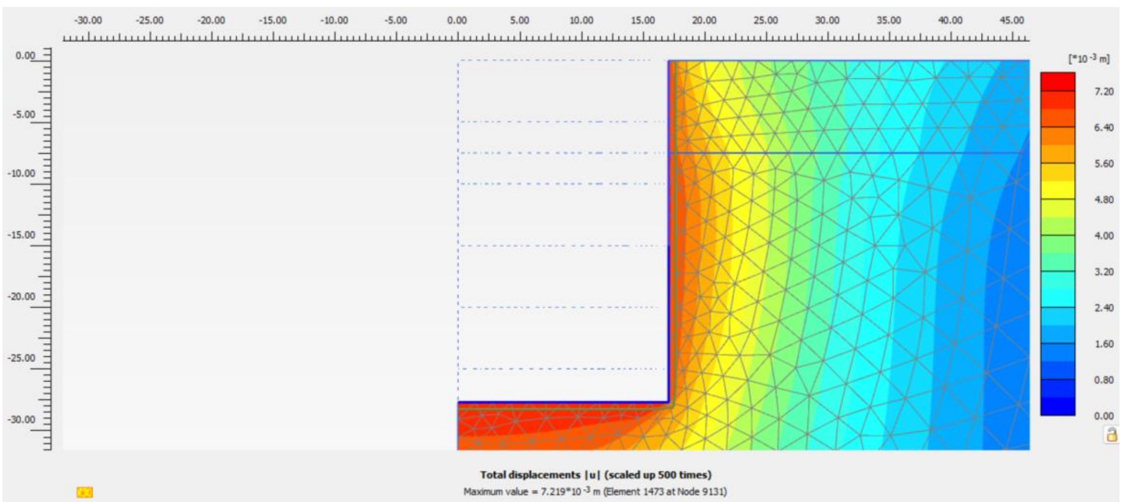


Figure 88 – Total displacement of the shaft with secant piles and shotcrete.

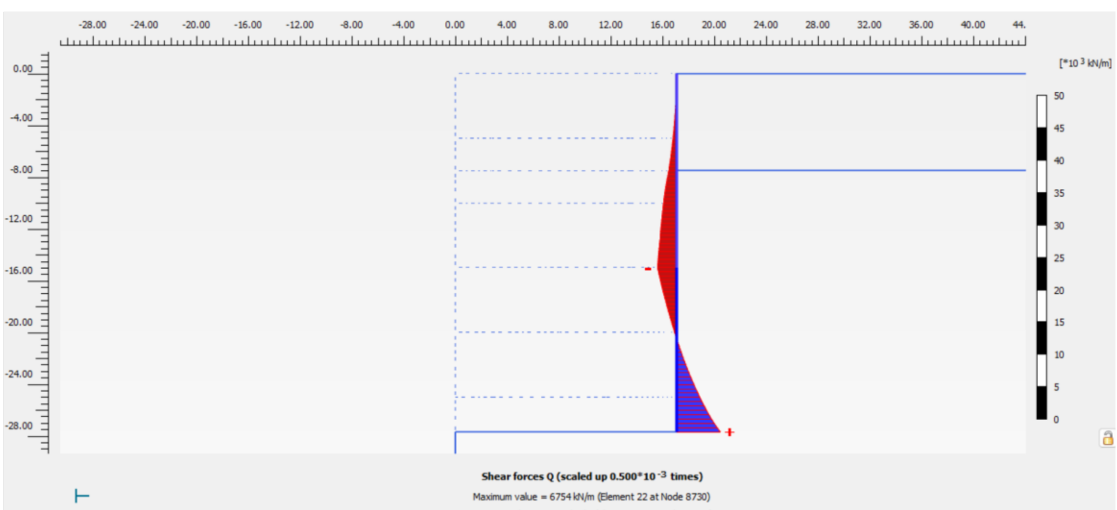


Figure 89 – Shear forces of the shaft with secant piles and shotcrete.

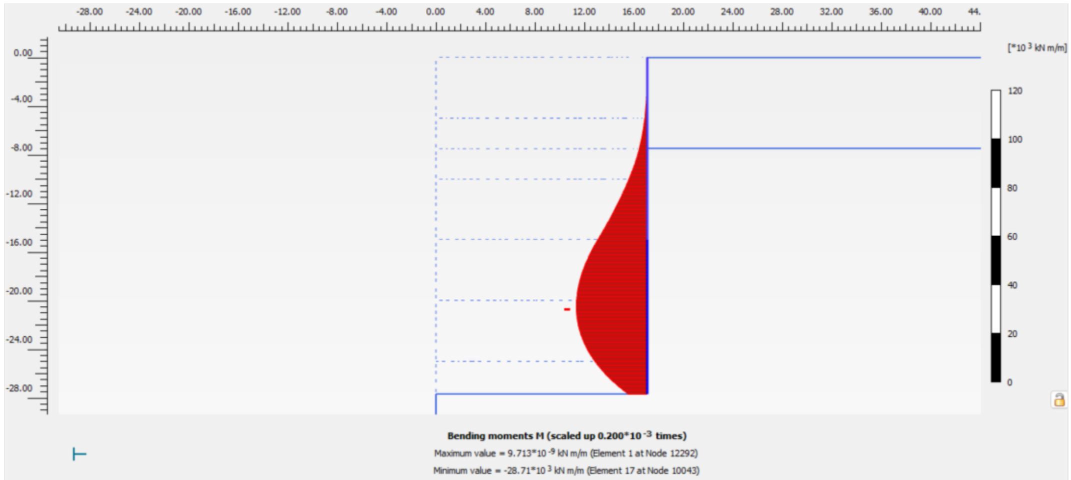


Figure 90 – Bending moments of the shaft with secant piles and shotcrete.

Appendix G – Days required to excavate the access shaft [62]

Ground makeup: Sand (0-2.5m), Glacial Till (2.5m-6.5m), Georgian Bay Shale (6.5m-100m)

Shaft Dimensions (length x width x depth) = 33m x 23.7m x 27.71m

Avg productivity of excavator (0.4m3 bucket) for sand and soil = 25m3/hr.

Avg productivity of excavator (0.4m3 bucket) for hard ground = 20m3/hr.

Assume glacial till and Georgian bay shale to be "hard ground" to be conservative when determining the type of excavator and productivity.

Assume working 8 hours/workday with efficiency of 50min/60min.

Assume 2 excavators will be used.

$$Time\ to\ excavate\ the\ sand = \frac{(volume\ of\ sand\ to\ be\ excavated)}{(average\ productitivty\ of\ hydraulic\ excavator * hours\ used\ per\ work\ day * efficiency)}$$

$$= \frac{(pi * 17.1^2 * 2.5)}{\left(25 * 8 * \frac{50}{60}\right)} = 14\ days$$

$$= \frac{14\ days}{2\ excavators} = 7\ days/excavator$$

$$Time\ to\ excavate\ the\ glacial\ till\ and\ the\ Georgian\ bay\ shale = \frac{(volume\ of\ glacial\ till\ and\ the\ Georgian\ bay\ shale\ to\ be\ excavated)}{(average\ productitivty\ of\ hydraulic\ excavator * hours\ used\ per\ work\ day * efficiency)}$$

$$= \frac{(pi * 17.1^2 * (27 - 2.5))}{\left(20 * 8 * \frac{50}{60}\right)} = 173\ days$$

$$= \frac{173\ days}{2\ excavators} = 86\ days/excavator$$

$$Soil\ removed\ each\ day = \frac{(pi * 17.1^2 * (27 - 2.5))}{86} = 269\frac{m3}{day} = 33.6m3/hour$$

Appendix H – Sample calculation of number of full loads of concrete trucks required.

Volume of shotcrete needed = (pi)(17.1^2-16.9^2)(12.71) = 272m3

Concrete trucks capacity is 8m3

Number of full loads of concrete trucks needed = 272/8 = 34 loads

Appendix I – Days required to excavate the access cavern.

$$Time\ to\ excavate\ the\ Georgian\ bay\ shale = \frac{(volume\ of\ the\ Georgian\ bay\ shale\ to\ be\ excavated)}{(average\ productitivty\ of\ hydraulic\ excavator * hours\ used\ per\ work\ day * efficiency)}$$

$$= \frac{(pi * 19.27^2 * 115)}{\left(30 * 8 * \frac{50}{60}\right)} = 574\ days$$

$$= \frac{574\ days}{5\ excavators} = 114\ days/excavator$$

Appendix J – Cost Calculations

$Cavern\ Total\ floor\ Area = 23.4 * 150 = 3{,}510\ m2$

$Acess\ Shaft\ Floor\ Area = 17.1 * 17.1 * pi\ =\ 918.63m2$

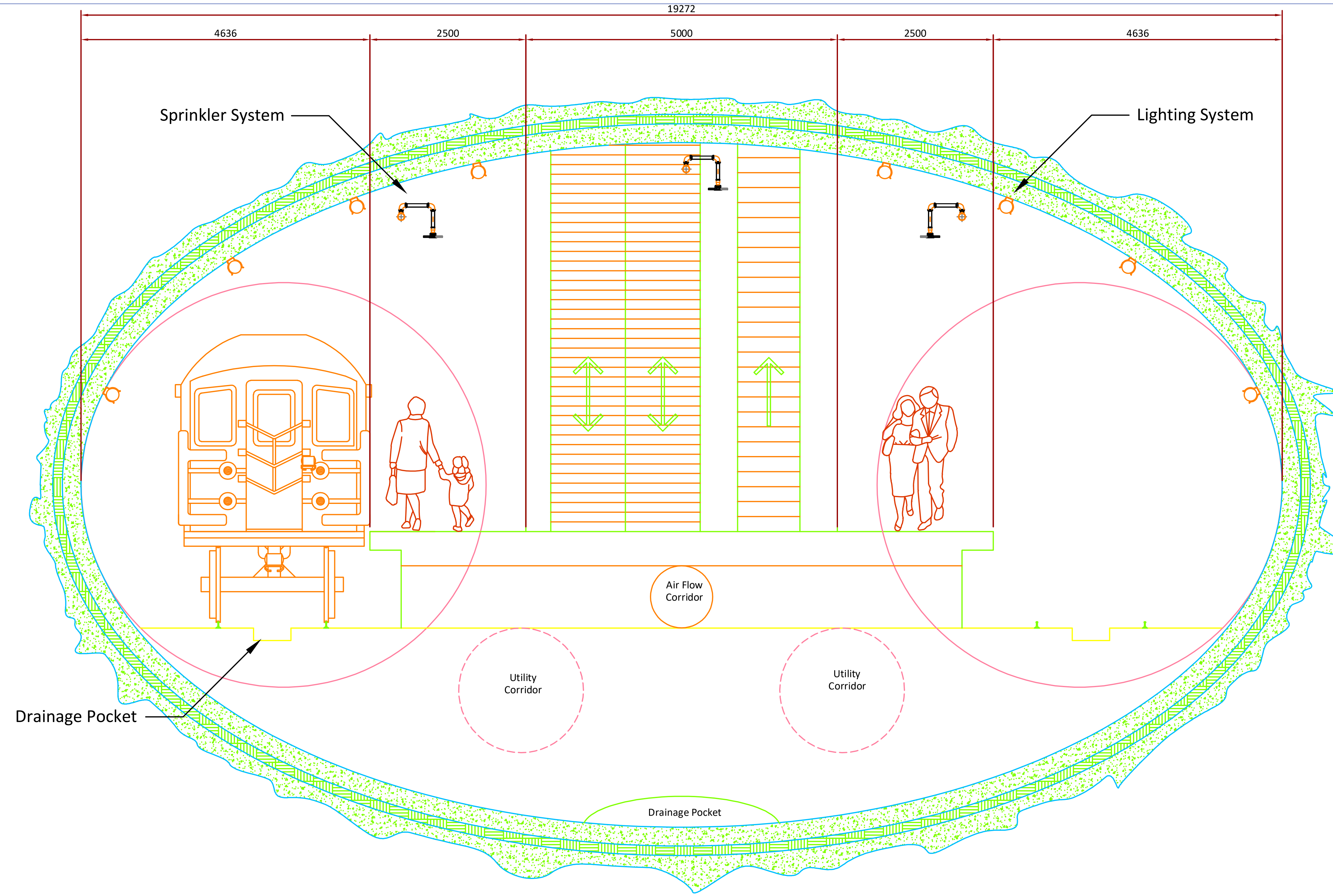
$Ground\ and\ second\ floor\ Area\ from\ AutoCad\ individually = 1236.87$

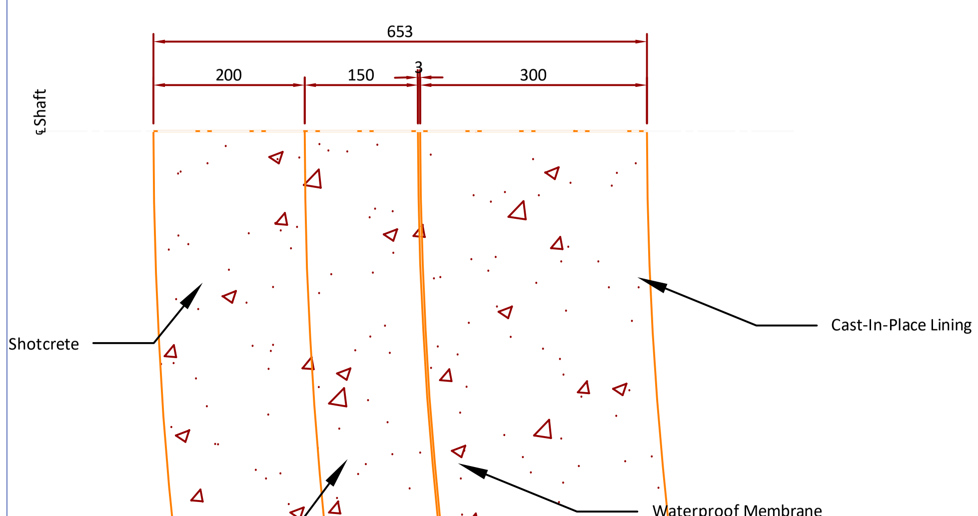
$Total\ Area =$ 6902.37

Appendix K – Temporary Liner Properties

thickness	0.2	m
a1	33.8	
a	34.2	
b1	33.8	
b	34.2	
Ix	3087.035756	m^4
Iy	3087.035756	m^4
A	21.36283004	
f'c	50	Mpa
Econc	30234523779	N/m^2
Unit Weight	2400	kg/m^3
Unit Weight	23.544	kN/m^2
EA	645894993	kN
EI	93335055959	kN

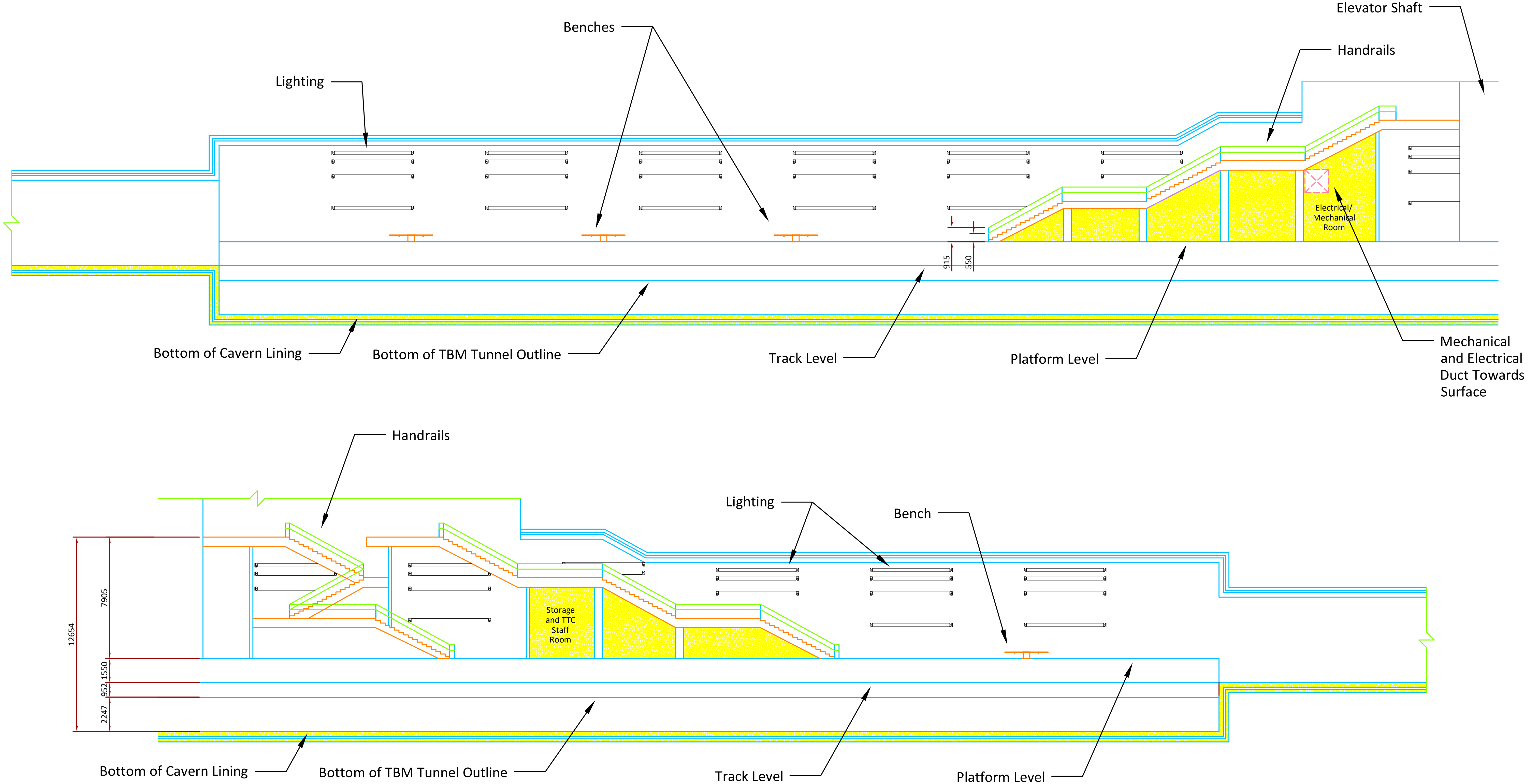
Figure 91 – Automated calculations done to calculate EA and EI.



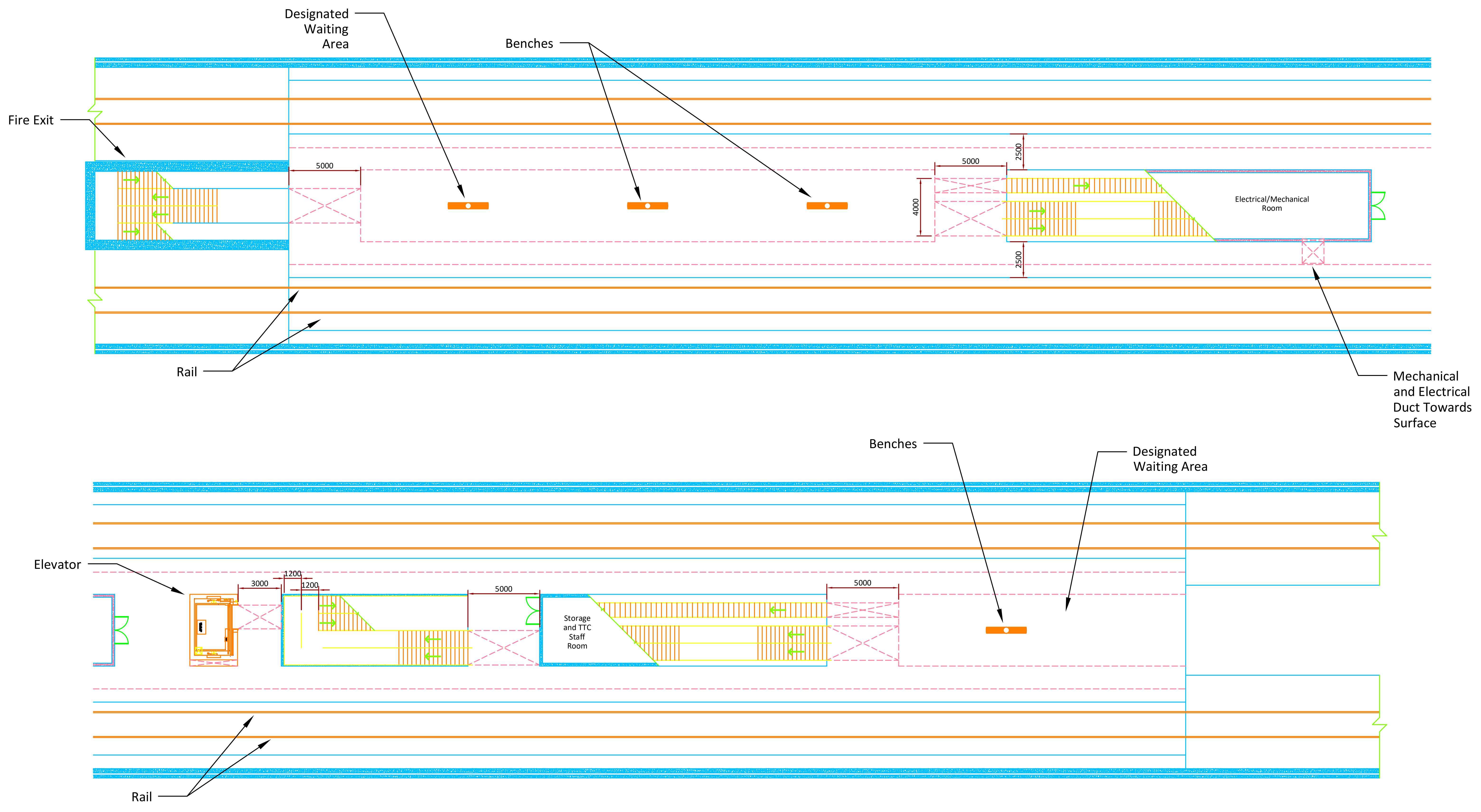


Lining Cross Section

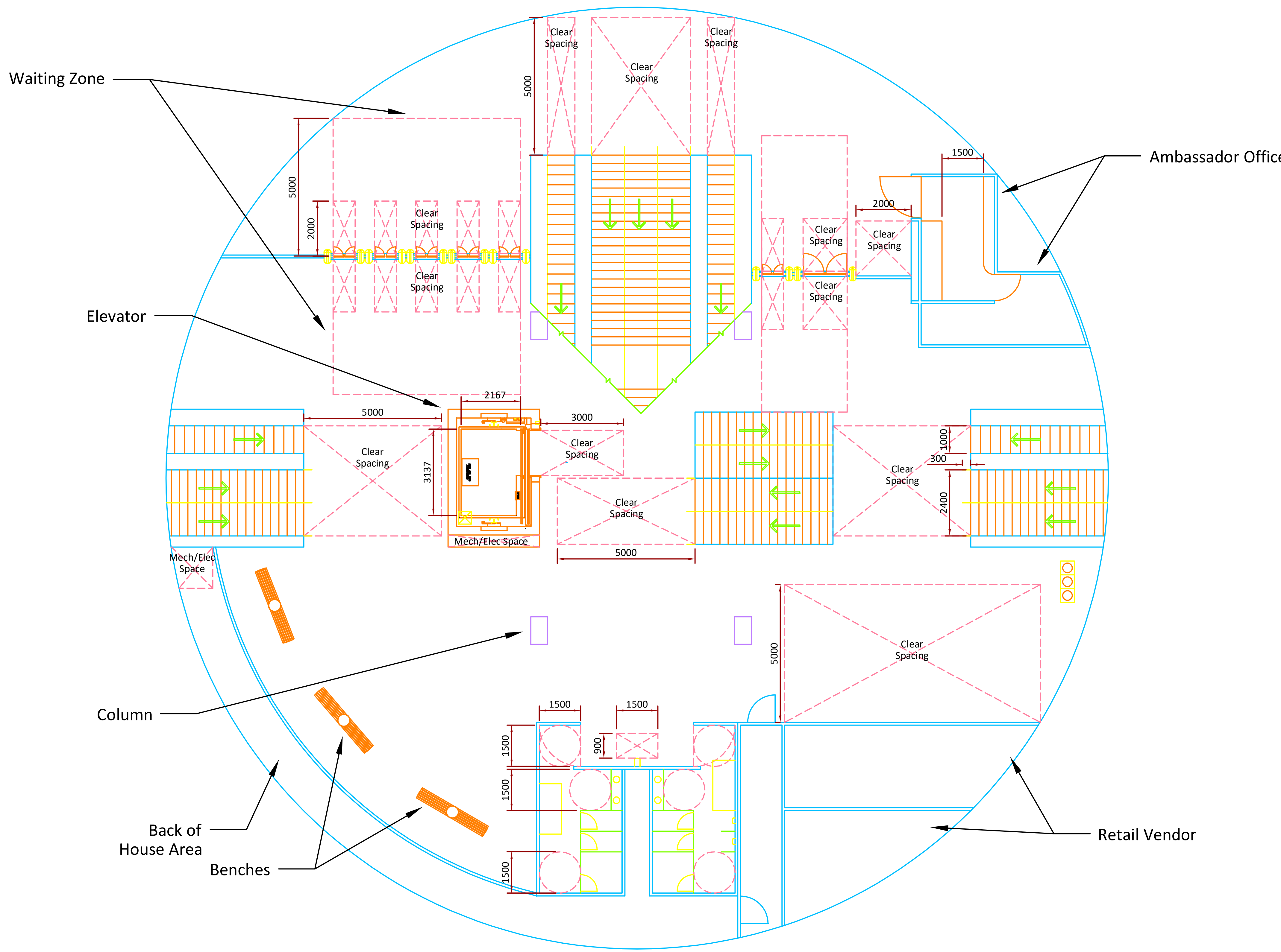
U of T CME 525
Subway Station Design
December 6, 2023



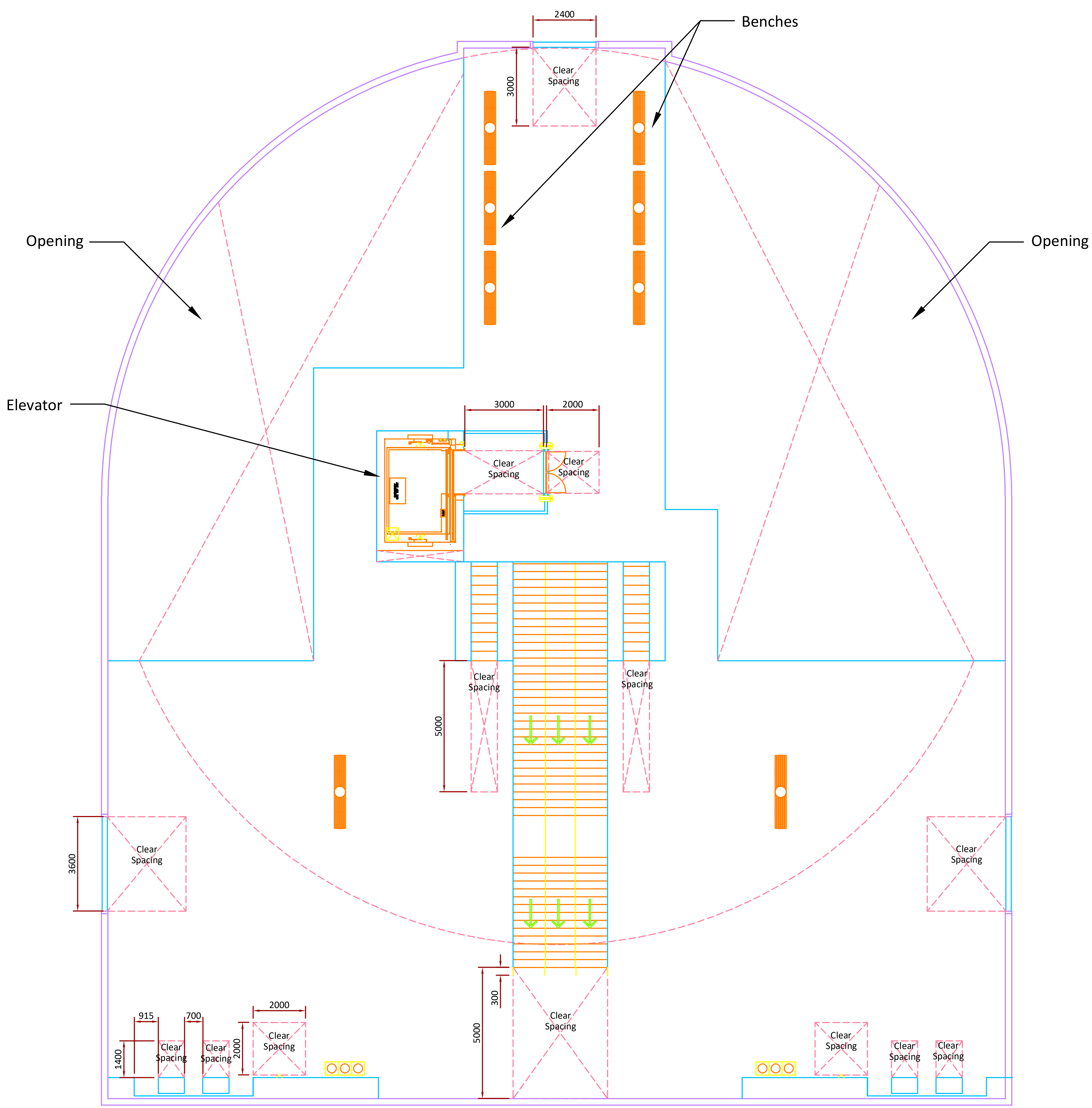
U of T CME 525
Subway Station Design
December 6, 2023



U of T CME 525
Subway Station Design
December 6, 2023



U of T CME 525
Subway Station Design
December 6, 2023



U of T CME 525
Subway Station Design
December 6, 2023