

Case Study - Challenges faced in Port Lands Flood Protection and Enabling Infrastructure (PLFPEI) Toronto Project

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ABSTRACT

This paper focuses on the installation of a full depth secant pile wall around the perimeter of the area of Work in Port Lands Flood Protection and Enabling Infrastructure (PLFPEI) project immediately east of downtown Toronto. The wall consisting of both structural and non-structural components is designed to serve as groundwater cut-off and facilitate excavation and construction of a naturalized river channel to protect against flooding.

Several challenges were faced during construction such as the high water infiltration rate from Lake Ontario leading to concrete placement through the tremie method, the strict verticality requirement for the deep piles resulting from the highly varying bedrock levels, drilling obstructions and void propagation.

The complexity of this project from conception to implementation necessitated the employment of numerous measures and techniques required for adequate construction as highlighted in this paper.

RÉSUMÉ

Cet article se concentre sur l'installation d'un mur en pieux sécants, insérés à pleine profondeur, autour du périmètre de la zone concernée par le projet de renforcement des infrastructures et de protection contre les inondations à Port Lands (PLFPEI) immédiatement à l'est du centre-ville de Toronto. Le mur composé à la fois d'éléments structuraux et non structuraux, est conçu pour servir d'obstacle aux eaux souterraines. Il doit aussi faciliter l'excavation et la construction d'un canal fluvial naturalisé protégeant ainsi contre les inondations.

Plusieurs défis ont été rencontrés pendant la construction de ce projet comme l'infiltration importante par l'eau du lac d'Ontario amenant à la mise en place du béton par la méthode tremie, l'exigence de verticalité stricte pour les pieux profonds suite aux niveaux de substrat rocheux très variables, les obstructions de forage et la propagation des vides.

La complexité de ce projet, allant de sa conception jusqu'à sa réalisation, a nécessité l'emploi de nombreuses mesures et techniques nécessaires à une construction adéquate, comme souligné dans cet article.

1 INTRODUCTION

1.1 Project overview and background

The City of Toronto, the Province of Ontario, and the Government of Canada are undertaking a 1.25 B\$ design and redevelopment project of the Port Lands which is the largest urban redevelopment project currently underway in North America. Waterfront Toronto, the project owner, was established in 2001 by the local, provincial, and federal governments in order to manage public waterfront development in Toronto. In 2005, Waterfront Toronto developed a Sustainability Framework for their projects signaling the intention to build vibrant downtown communities that embraced 21st Century ecological stewardship and resiliency (Solano 2017). The Port Lands are a 1,000-acre area on the shore of Lake Ontario,

immediately east of downtown Toronto. Figure 1 shows an aerial view of the Port Lands.

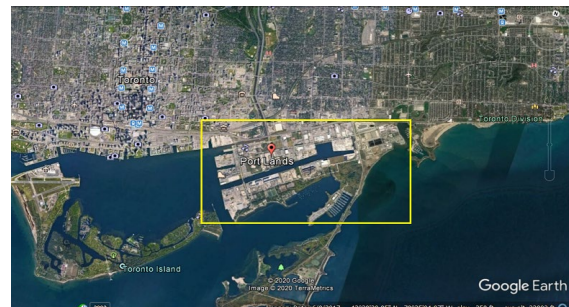


Figure 1. Aerial view of the Port Lands (Google Earth)

The area was man-made through decades of infilling of historic wetlands and has historically been used for heavy industry. Contaminants of concern primarily include petroleum hydrocarbons and metals.

In 1892, an emergency relief plan prepared by City Engineer E.H. Keating was approved. It consisted of the construction of a channel running east-west along the northern perimeter of the marsh, with openings into Toronto Harbour in the west and to Lake Ontario in the east to facilitate water circulation through the marsh (Bonnel and Fortin 2009). However, the flooding has increased in severity and there has been recent push to naturalize the mouth of the river. Much of the area is within the flood plain of the Don River and therefore flood protection must be created before the area can be fully developed. This includes the design and construction of a new River Valley that will connect the existing Don River to Lake Ontario. Figure 2 shows the affected flood-vulnerable area.



Figure 2. Flood-vulnerable area (Toronto and Region Conservation Authority (TRCA) 2020)

As part of the flood protection, a groundwater cut-off wall was designed to facilitate excavation and construction of a naturalized river channel.

Three types of cut-off walls were considered in the preliminary stages: diaphragm walls, secant pile walls, and soil mixing walls. The secant pile wall was eventually selected to meet construction deadlines. This paper is focused on the full depth secant pile wall construction with emphasis on challenges faced and corrective measures implemented.

Construction on this phase of the project began in May 2019 and spanned over 1 year. Overall, 1500 piles were completed by 6 drilling rigs operating over 2 shifts and 6 days per week. The perimeter of the work (the "Site") is shown in Figure 3.



Figure 3. Perimeter of work

The north and south walls, as shown in Figure 4 are structural providing support during excavation and backfill operations whereas the east and west walls are non-structural.

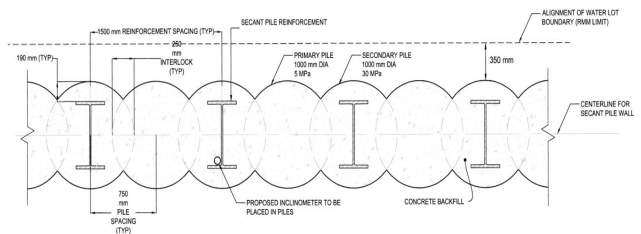


Figure 4. North and south structural walls

1.2 Geotechnical Settings

The field investigation activities consisted of the advancement of 37 soil borings using the mud rotary method, the collection of disturbed soil samples using the SPT sampler and relatively undisturbed Shelby tube samples, the collection of bedrock samples using HQ-sized rock coring. 10 shear vane tests were also performed within the borings and 15 monitoring wells were installed. In addition, 24 Seismic Cone Penetration Tests (SCPTs) with pore pressure dissipation were advanced.

The Site lies within the Iroquois physiographic region, which is characterized by a sand plain. The Site is situated in an area of lacustrine deposits consisting of sand, gravel, silt and clay (Barnett et al. 1991). Bedrock consists of shale (Georgian Bay Formation).

The subsurface stratigraphy at the Site was developed based on boring logs obtained from the field investigation and consists of the following layers:

- Fill:** The Fill material at the Site is variable and consists of sand, gravel, silty sand/sandy silt, and silty clay with inclusions of organic material and construction rubble. The density is highly variable, as indicated by SPT N60 values ranging from 1 to over 100 (the high SPT N60 values may have been influenced by cobbles or construction debris.)
- Sand:** The Sand material is interbedded with lenses of organic and cohesive soils. SPT N60 values from 0 to greater than 100 (very loose to very dense) but are typically between 4 and 30 (loose to medium dense). The Sand material is predominantly a fine to medium sand with varying amounts of coarse sand, silt, and gravel.
- Organic Soil:** An organic layer consisting of organic silt and peat was encountered in most of the explorations near the top of the Sand unit. SPT N60 values typically varied from 0 to 4, indicating a very soft to soft consistency. CPT-correlated undrained shear strengths also showed a very soft to soft consistency.
- Silt:** Inorganic cohesive soils generally classified as silts and clays were encountered in many of the explorations. The Silt material was most often encountered above the bedrock. The consistency of the Silt varied from very soft to hard but was typically stiff to very stiff.
- Bedrock:** The Bedrock consists of shale from the Georgian Bay Formation. The shale was generally observed to be grey, laminated, completely weathered at

its surface and gradually became fresh/sound with depth. Rock Quality Designation (RQD) values range from 0 to 100 percent and are generally lowest at the top of rock and increase with depth.

Groundwater levels are heavily influenced by Lake Ontario and typically range between elevation 74 and 76 meters (m); this is similar to Lake Ontario's typical water elevation of 75 m.

2 CHALLENGES FACED

2.1 High water infiltration rate

As highlighted in section 1.2, the Port Lands are adjacent to Lake Ontario which leads to a shallow groundwater level and high water infiltration rates. The secant piles, installed through caisson drilling techniques, had to be embedded at least 3 m into the shale bedrock. A series of pilot tests were initially performed to establish the optimal rock embedment for hydraulic cut off from the weathered shale bedrock. The test consisted of excavating 3 m below the top of the weathered shale followed by a water infiltration test. If the test failed, an additional 0.5 m rock embedment was excavated and the test was repeated. Excavation continued up to 4 m below the top of the weathered shale. Following these tests alongside the completion of a detailed cost-benefit analysis comparing additional embedment costs to water treatment costs, it was concluded that a 3 m rock embedment on the North and South walls and a 3.5 m embedment on the East and West walls were sufficient to provide adequate water cutoff.

Prior to concrete placement, the outer liner had to extend at least 1 m into the rock in order to combat the high infiltration rate and sediment built up. For several piles especially on the North West corner of the site, sediments were accumulating at rates exceeding 1 m/hour. In such cases, liner was advanced to full depth and additional clean-out passes were done either with the drilling or bailing bucket just prior to concrete placement.

The high-water infiltration rate necessitated the placement of concrete through the tremie method instead of free fall. Before the start of the pour, concrete pumps were primed with grout which served as a coat for the interior of the pumping lines and prevented blockages from forming. Concrete was then pushed to the base of the drilled shaft through the tremie pipe which was connected to the concrete pump. On average, concrete placement required 4 hours and up to 6 hours for the deeper piles.

2.2 Deep piles and drilling obstructions

The design requirements within the site-specific geologic conditions as discussed in section 1.2 and shown in Figure 5 contributed to construction challenges.

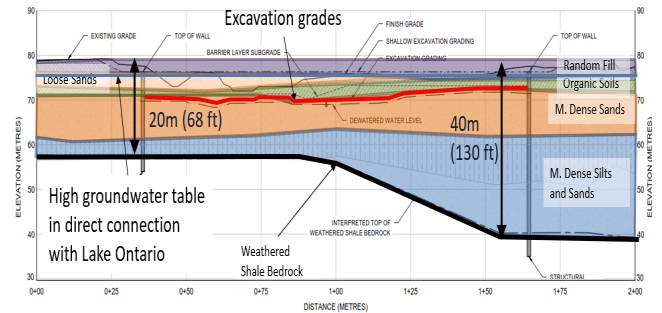


Figure 5. Site Subsurface Stratigraphy

The anticipated rock embedment depth was determined through packer testing and borehole geophysics analysis. The bedrock surface is highly variable spatially. It averages at around 17.6 m on the East wall and 38.8 m on the West wall.

The varying bedrock levels lead to piles exceeding 40 m in total depth in order to reach the minimum required embedment into weathered shale. The top of bedrock was determined on-Site through visual inspection of material removed from primary filler piles during excavation. Top of bedrock for secondary piles was determined as the maximum bedrock depth of the two adjacent primary piles due to the difficulty of visual shale identification amidst the concrete from the fillers.

The deep secant piles have caused difficulties for the contractor in maintaining the strict verticality requirement of 0.35% as required in the design. The various methods of verticality measurement and monitoring are provided in section 2.3. The pile distribution by total drill depth in Table 1 shows that the majority of the installed piles had a total depth between 20 and 25 m.

Table 1. Pile Distribution by Depth

Drill Depth (m)	Pile Distribution (%)
< 20	3
20 -25	47
25-35	3.7
35-40	5.3
>40	41

Additionally, the subsurface conditions from previous industrial site use lead to several liners getting stuck. Wood obstructions were the primary obstruction type that hindered liner advancement as seen in Figure 6.



Figure 6. Wood obstruction

Figure 7 shows the location of those obstructions throughout the work perimeter.

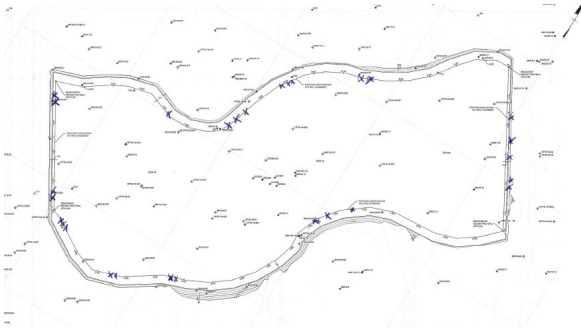


Figure 7. Wood obstruction locations

When a liner got stuck due to an obstruction, the contractor would spend few hours trying to push the liner further down. However, if no advancement is made, the hole would be poured with 8 Megapascal (MPa) concrete and then re-drilled at a later stage.

2.3 Verticality Requirement

In order to maintain the continuity of the secant pile wall, the verticality tolerance as required by design was 0.35% (more stringent than typically seen in construction). This was considered challenging for the contractors especially for deeper piles in order to ensure appropriate interlock between primary and secondary secant piles. Verticality is critical for hydraulic cutoff given that any deviation would significantly increase the dewatering volume and subsequently cause excessive project risk.

In terms of verticality measurement, several methods were considered such as: LiDAR, sonic caliper system and the Jean Lutz sensor system.

LiDAR measurements were performed on a limited number of dry piles as the equipment does not work underwater. The sonic caliper system provides x-y-z point cloud for the entire shaft; however, it would have slowed

down production since it needed more than 30 minutes for completion.

The Jean Lutz system was agreed upon for verticality measurement since it required easy modifications to the contractor's equipment and met the test time constraints. The system was attached to the drill rig as shown in Figure 8. The process involves a wireless sensor PRAD attached to the Kelly block as it descends inside the hole during excavation. The PRAD sensor measures the deviation and bottom pressure during excavation and transmits to an interface accessible to the operator. (JEAN LUTZ North America 2018)



Figure 8. Jean Lutz Equipment

Jean Lutz measurements took on average 10-20 minutes to complete. Figure 9 shows an example of the Jean Lutz report where x and y deviations are highlighted.

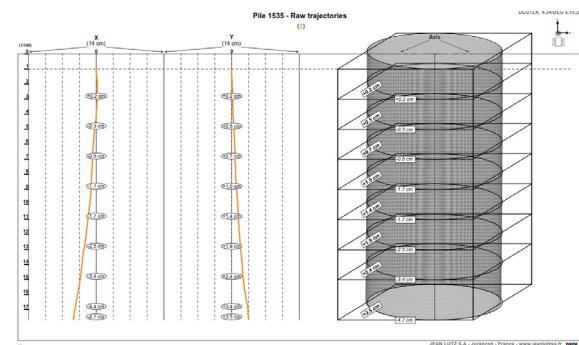


Figure 9. Jean Lutz Report

The Jean Lutz reports and all construction records for each individual pile were updated to the Site-specific Information Management system (IMS). The IMS allows the tracking of the project progress and all the QC records. It is a tool accessible by all project parties and is updated on a regular basis which allows the visualization of the wall quality as construction is progressing.

Through the IMS, it was determined that nearly 90% of the piles met the 0.35% verticality target as shown in Figure 10.

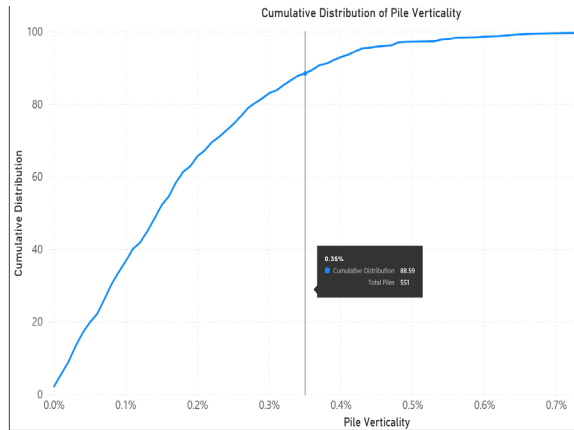


Figure 10. Cumulative Distribution of Pile Verticality

For piles with greater than 0.35% tilt, the pile toe overlap was reviewed. 99.87 % of the piles exceeded the 5-cm target overlap requirement for pile toe caps as shown in Figure 11.

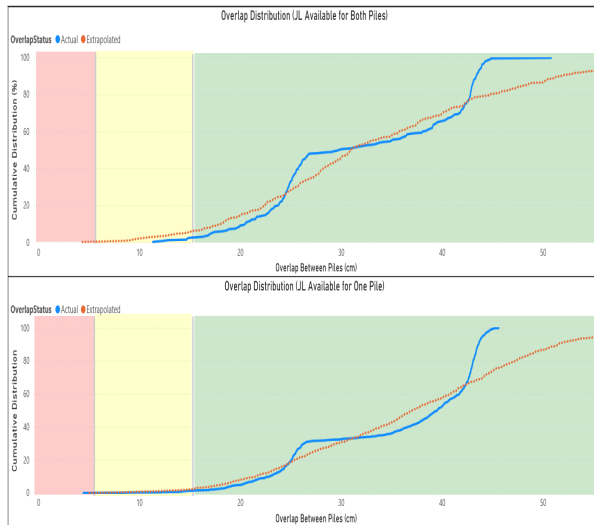


Figure 11. Overlap Distribution

The verticality of the steel reinforcement was also critical. The deep W-shape steel rebars limited the available concrete cover and did not extend to the full pile depth which necessitated the installment of the steel beams after liners were extracted and concrete placement was completed. Sacrificial test piles were employed and inclinometers were added to select steel beams in order to study the effectiveness of the installation methods.

The study of the use of centralizers on the beams was conducted to check their effect on beam deviation from the center. The centralizers consisted of welded rebar

reinforcement at multiple levels (usually placed at 6 m increments) as shown in Figure 12.



Figure 12. Centralizers on the steel beam

Figures 13-a and 13-b show the increased efficiency of the use of centralizers on the beam's verticality.

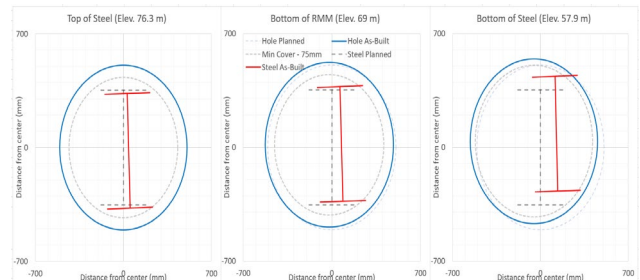


Figure 13.a. Reinforcement Verticality without centralizers

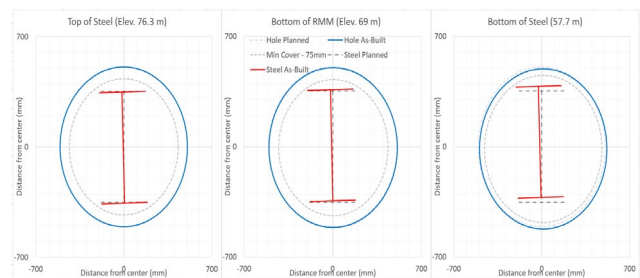


Figure 13.b. Reinforcement Verticality with centralizers

At 18 m depth, the piles are projected to protrude through the pile footprint without the use of centralizers, whereas when the centralizers were added on the steel beam, a better control was observed.

2.4 Void Propagation

Another major challenge faced during the construction of the secant pile wall was the propagation of voids on different sections of the work area.

The voiding could be attributed to the loose fill and sand material shown by the low SPT N60 values and the shallow groundwater level as discussed in section 1.2. It may also

be caused by poor ground preparations prior to mobilization.

The first void occurred in July 2019 on the west wall of the site. Multiple voids appeared on surface through the engineered work platform of that same wall throughout the summer of 2019 which lead to a complete production shut down on the west wall. Figure 14 shows the void at the South-West corner which occurred in September 2019 and caused the West wall work to shut down while an engineering solution was implemented.



Figure 14. South-West corner void

The excessive voiding on the west wall was remediated by excavating a concrete pad through the drilling platform in January 2020 as shown in Figure 15. The pad had an approximate depth of 2 m and width of 2.5 m.



Figure 15. Concrete pad

Further voiding was found in October and December on the North and South walls. Table 2 shows the distribution of the voids on the four walls of the work area.

Table 2. Void Distribution by location

Wall Location	Wall Length (m)	Void Distribution (%)
North	422	14
East	141	0
South	441	38
West	110	48

The voids are highlighted in their respective locations on figure 16.



Figure 16. Void Propagation throughout work area

The void remediation measures taken on-Site consisted of immediately vacating the area following the discovery of a surface void due to the significant safety hazard posed. An investigation was then conducted by excavating around the void (using a mini-excavator) to expose its extent as shown in Figure 17.



Figure 17. Void Investigation

A remedial solution was then proposed according to the extent of the void. Often, the void and the excavated area were filled with 5 MPa concrete. Work was typically resumed 48 hours following the remedial backfill activities.

In order to limit the propagation of surface voids, the drill rig working platform was frequently inspected by the Contractor. Drilled shafts with anticipated overburden exceeding 25 m were flooded with water following the first void occurrence in July 2019. The 25 m depth limit was selected based on observations of voiding and location of high water infiltrations. Flooding commenced after the second piece of liner was installed at an approximate drill depth of 6 m. Thereafter, the shaft was topped off with water with each subsequent liner piece.

In December 2019, following the void occurrences on the North and South Walls near piles less than 25 m in depth, the flooding process was implemented by the Contractor on all piles regardless of depth.

Additionally, in order to detect any void before propagating to the surface, a vibratory pad foot drum roller was used on the working pad in the expected location of the drilling rig. Geophysics ground penetration radars were also used in order to detect any near-surface voiding.

3 CONCLUSION

Several challenges were faced during the construction of the full-depth secant pile wall in the PLFPEI project. Due to the design requirements and the site-specific geotechnical settings, the piles had to extend at least 3 m into the bedrock to combat the high water infiltration rate. Concrete was also placed through the tremie method instead of free-falling.

The verticality requirement for the drilled shafts was set at a strict 0.35% which was measured by the Jean Lutz system and monitored through the IMS which is an impartial tool accessible to all parties used to record the progress achieved and store all quality archives for future referral. The reinforcement verticality was controlled using centralizers.

Additionally, given that voiding has not subsided since flooding all the drilled shafts, frequent investigation of the working pad was instrumental for early void detection and sequential remedial measures.

Hence, it can be concluded that the challenges faced while constructing the deep secant piles per the strict design requirements and in the site-specific conditions can be overcome. The construction is feasible with the suitable measures discussed in this paper along with timely monitoring.

4 ACKNOWLEDGMENT

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