

# Calgary Cancer Centre – Design and Construction Challenges for a Deep Excavation and Building Foundations within Lacustrine Clay

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## ABSTRACT

The Calgary Cancer Centre, currently under construction within the Foothills Hospital Campus in Calgary, Alberta, presented challenges for design and construction of an effective shoring system and building foundations. The building has 13 stories above grade and extends 5 levels (18 m) below existing grades and occupies a tight footprint surrounded closely by major roadways and sensitive utilities. Weak and saturated lacustrine soils extend beneath the site to a depth of approximately 13 m, presenting concerns for maintaining adequate excavation support to allow for a deep excavation without damaging adjacent roadways or utilities servicing the hospital facility. This paper discusses the methods used to characterize the engineering properties of the site soils, the associated shoring and foundation design challenges and the selected designs.

This paper discusses site monitoring methodology and adjustments to shoring design that were necessary to maintain excavation stability and to facilitate foundation installation in response to localized variations in soil strength and groundwater. Given the aggressive design-build schedule, the success of the shoring installation and completion of foundations required effective communication and coordination between the geotechnical engineer, the contractor, the shoring designer, the shoring installation contractor and the structural engineer.

## RÉSUMÉ

La conception et la construction des fondations et d'un système d'échafaudage performant pour le *Calgary Cancer Centre*, présentement en construction sur le campus de l'hôpital Foothills à Calgary, ont posé plusieurs défis. Le bâtiment comporte 13 étages au-dessus du niveau du sol ainsi que 5 niveaux souterrains (18 m de profondeur) et occupe une empreinte restreinte à proximité d'artères principales et de services municipaux. Des sols lacustres mous et saturés vont jusqu'à environ 13 m de profondeur et posent problème au maintien de supports d'excavation adéquats permettant des excavations profondes sans endommager les routes adjacentes ou les services municipaux associés à l'hôpital. Le présent article discute des méthodes employées pour caractériser les propriétés du sol sur l'empreinte du site, les problèmes rencontrés lors de la conception des systèmes de fondation et d'échafaudage ainsi que les concepts retenus pour ces systèmes.

Une discussion sur la méthodologie de surveillance du site ainsi que les ajustements au concept d'échafaudage qui furent nécessaires pour maintenir la stabilité de l'excavation et faciliter l'installation des fondations en réponse à des variations locales de résistance du sol et de la nappe phréatique est présentée. En raison d'un horaire expéditif du modèle de conception-construction, le succès de l'installation du système d'échafaudage et la complétion des fondations reposaient sur une bonne communication et une coordination efficace entre l'ingénieur géotechnicien, l'entrepreneur, le concepteur du système d'échafaudage et son entrepreneur et l'ingénieur en structures.

## 1 INTRODUCTION

The Calgary Cancer Centre (CCC), currently under construction and scheduled for completion in 2023, consists of 13 stories of above grade hospital care, clinical, research and academic space (Figure 1). Below grade construction consists of 5 levels of parking adjacent to two levels of heavily loaded radiation vaults and treatment areas. The project is being constructed within the existing developed Foothills Medical Centre Campus northwest of downtown Calgary and occupies a site tightly bounded by the Trans Canada Highway (16<sup>th</sup> Ave), 29<sup>th</sup> Street NW and Hospital Drive, the main access to the Foothills campus.

## 2 SITE CHARACTERIZATION

The site's location is described in the Surficial Geology of the Calgary Urban Area (S. R. Moran 1986) as a region mantled with glacial lacustrine soils consisting of silt, clay, and very fine sand which can exceed 50 m in thickness (Figure 2.) These weak and typically saturated sediments contrast markedly with the more common glacial clay till, fluvial sand, and gravel soils of the Bow River valley and surrounding Calgary region. A review of historical aerial photos prior to development of the Foothills Medical Centre, indicated that a small pond or slough was present within the CCC building footprint, which likely contributed to the near surface weakness and high moisture contents noted within the site lacustrine soils (Figure 3).



Figure 1. Architectural rendering of the Calgary Cancer Centre building



Figure 2. Site location within lacustrine sediments (blue)

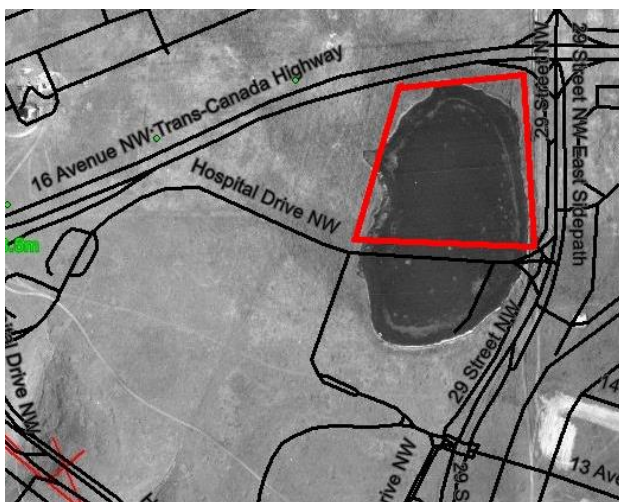


Figure 3. Pre-development aerial photo of site (1953)

Site exploration utilized both hollow-stem auger boreholes to observe and sample soil conditions with regular SPT testing and retrieval of Shelby tubes within cohesive sediments for laboratory strength and consolidation testing. The boreholes were supplemented with Cone

Penetration Testing (CPT) to evaluate the extent and properties of the lacustrine sediments and deeper clay till. Pressure meter testing was also conducted within the deeper till encountered beneath the lacustrine deposits to help assess the in-situ strength and suitability of the till for foundation bearing.

The site was underlain by varying surficial fill and organic clay extending to depths of 2 m to 4 m underlain by weak and saturated interbedded silt, clay and sand (corresponding to the expected glacial lacustrine sediments) which extended to depths of approximately 9 m. Between 9 m and 15 m was a transition zone where the lacustrine sediments appeared to have more clay and the observed sediments varied between thin bedding to having no apparent structure (till-like). Below 15 m depth, there was an observable increase in the overall strength and density of the soil, varying from very stiff to hard with less visible bedding structure. This soil unit was interpreted to be a glacial till deposit interlayered with over-consolidated lacustrine deposits, which extended to the maximum depth explored of approximately 30 m.

Based on boreholes completed for adjacent buildings and other projects in the vicinity, the glacial clay till extends to estimated depths of at least 45 m to 50 m below ground surface, where sedimentary bedrock of the Paskapoo Formation is expected. Groundwater was encountered between 3 m and 4 m in all boreholes. The soil conditions within the boreholes were generally consistent, with the notable exception of the northwest region of the site, where CPT testing met early refusal in what appeared to be a very dense fine sand and silt deposit. A subsequent borehole was completed in this area which revealed sand extending from approximately 14.5 m to 23 m in depth, with increasing interbeds of very stiff to hard silt and clay with depth.

### 3 FOUNDATION DESIGN

The boreholes performed within the CCC building footprint indicated that site conditions were similar to other previously developed sites within the Foothills Medical Centre, although there appeared to be more sand within the upper lacustrine sediments at the CCC site. The site exploration, CPT and laboratory testing, borehole pressure meter testing and results of previous full-scale load testing of piles at adjacent buildings, revealed that the clay till at depths of below 15 m could support the main building on conventional spread footings. The adjacent radiation vaults, which only extend 2 levels below grade, had to be supported on cast-in-place concrete piles extending to bear within the underlying till at similar depths as the adjacent spread footings of the main building as shown on Figure 4.

The complex building structure, both above and below grade, resulted in widely varying foundation column loads and spacing. The heavy loads of the radiation vaults required the use of belled piles up to 4.3 m in diameter. The juxtaposition of varying footing sizes beneath the building and the adjacent large diameter belled piles of the radiation vaults required consideration of the influence of proximity and loading in evaluating anticipated foundation settlements. Settle 3-D was used to calculate expected differential settlements between adjacent foundations.

This settlement analysis was performed in an iterative manner in close coordination with the structural engineer.

The individual footing and pile foundation layout, applied loading and structural details were adjusted as required to achieve a relatively uniform foundation response. A plan view of the various foundations is shown on Figure 5, illustrating the wide variety in footing spacing and size.

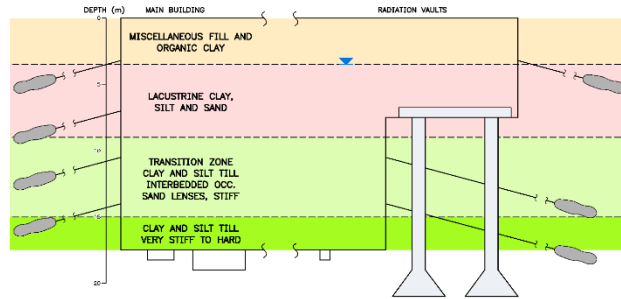


Figure 4. Generalized site subsurface profile

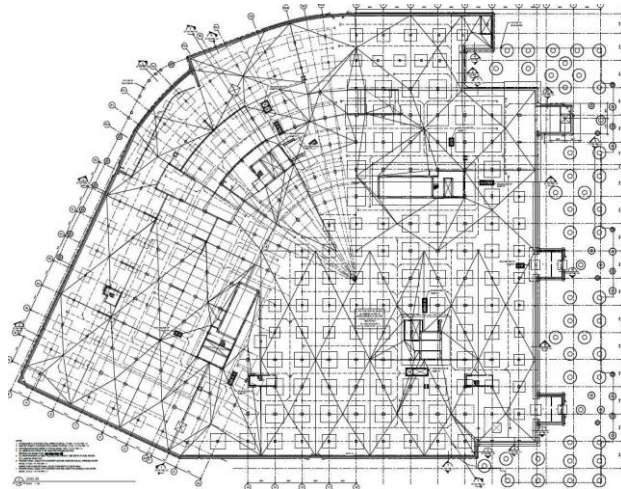


Figure 5. Plan view of varying foundation layout

#### 4 SHORING SYSTEM

A suitable shoring system was required to facilitate up to 18 m of below grade construction without disturbing adjacent access roads, buried utilities, and buildings. Both a concrete secant pile shoring system and a steel soldier pile and lagging system incorporating grouted tie-back anchors were evaluated. A concrete secant pile shoring option was considered to minimize deflections but was ultimately discarded due to concerns with below-grade waterproofing, access for perimeter utility tie-ins, schedule, and cost. A soldier pile and wooden lagging system was selected due to its flexibility to accommodate perimeter adjustments during construction, previous successful implementation on adjacent projects with similar depth of excavation, expedient construction and cost. Concerns for a soldier pile and lagging system consist of loss of ground during sequential excavation and installation of lagging along with achieving adequate installation and

performance of tie-back anchors bearing within the weak lacustrine soils.

#### 5 SHORING INSTALLATION

The first phase of site work involved the installation of steel soldier piles around the excavation perimeter to establish the below grade shoring system. Soldier piles were installed with a piling rig to extend to bear typically 2.5 m below the maximum depth of the excavation. In order to place the steel sections of the soldier piles, casing was required within the upper weak lacustrine soils to prevent caving and to reduce groundwater seepage into the holes.

Careful attention was taken to log the soil materials excavated during drilling of the piling holes to note the depths where the transition between weak lacustrine and underlying stiff till was encountered and to record the presence of cohesionless sand and silt layers within the till to assist in identifying changes in soil conditions that could affect subsequent excavation and installation of tie back anchors. Most of the shoring pile holes required casing to depths of 9 m to approximately 12 m where stiff clay till was encountered. Sand lenses within the till were noted in several boreholes along the north wall and near the northwest corner of the building, where the installation of shoring piles required casing to be utilized full-depth. Inclometers were installed within soldier piles at select locations around the shored perimeter to monitor the lateral movement of the temporary shoring system as excavation proceeded and after completion.

After completion of the perimeter piling, excavation and installation of timber lagging commenced. The typical shoring design included 4 rows of grouted tie-back anchors at depths of 3.2 m, 7.0 m, 10.5 m and 14 m to support a maximum excavation depth of 17.4 m. Excavation and installation of the upper soils to the depth of the first row of tie backs was executed smoothly. Upon encountering groundwater and lacustrine soils at depths below 4 m, excavation and installation of the lagging boards became problematic due to the saturated and frequently cohesionless sediments. Excavation depth was limited to 2 or 3 lagging boards at a time and was performed sequentially around the perimeter until the next deeper row of tie back anchors was reached. The time for leaving the sidewalls unsupported ahead of installing lagging boards was limited, as regions of cohesionless soil would be prone to sloughing. Where sloughing occurred behind the lagging boards, any voids between the lagging and in-situ soil were backfilled with cement grout or pea gravel. Excavation and installation of lagging proceeded slowly until the depth of excavation reached the transition zone where more competent till was encountered, typically at depths of 9 to 12 meters. Regions of cohesionless silt and sand within the deeper till required care to minimize sloughing. Given the saturated lacustrine soils at higher elevations and the porous nature of the lagging boards, groundwater seepage was constant along much of the face of the excavation between the 2<sup>nd</sup> and 3<sup>rd</sup> row of tiebacks (Figure 6). Groundwater seepage and below freezing temperatures during much of the excavation made for slow and challenging removal of sediments to allow for installation of lagging.

The installation of tie back anchors within the lacustrine soils was also problematic. Due to the weak subsurface conditions, tieback anchors extended up to 13.4 m to achieve the required capacity. Some of the grouted tieback anchors were installed quickly and achieved the required proof loads upon a single stage of pressure grouting. Other tieback anchors encountered very weak ground and required multiple stages of pressure grouting to provide adequate capacity. Concern over the long-term performance of the grouted anchors and their ability to maintain required loading lead to the installation of load cells at representative locations to monitor the anchor loads over time. Figure 6 shows the finished temporary shoring system. Of note is the pronounced seepage line across the excavation corresponding to the top elevation of weak and saturated lacustrine soils.



Figure 6. Completed temporary shoring system

## 6 FOUNDATION INSTALLATION

As excavation reached the base elevation of the radiation vaults (2 levels below ground), the installation of foundation piling commenced. The shafts for the radiation vaults extended into the deep till where a belled base was constructed. These pile shafts had 750 mm to 1500 mm shaft diameters, pile lengths of 7.6 m to 26.5 m, and belled base diameters ranging up to 4300 mm (For reference, a Honda civic is approx. 4500 mm long). Given the site investigation's findings of sporadic lenses of cohesionless silt and sand lenses within the deeper till soils, there was concern for the feasibility of constructing belled piles within potentially caving soils. Piling for the radiation vaults proceeded slowly with the larger piles taking a full day to complete. Out of 58 belled piles, only a few had to be relocated due to collapse of the bell. Once the use of the bellying tool was attempted, if the hole collapsed, that pile would be abandoned due to concerns with disruption of the concrete shaft integrity at that location upon casing withdrawal. Where belled piles could not be relocated due to proximity concerns, a belled pile was replaced with one or more straight shaft piles to provide equivalent capacity. The installation of straight shaft piles was less problematic, as they could be fully cased and easily extended to greater

depth to achieve adequate capacity where weak and caving soils were encountered. A down-hole camera was used to confirm proper cleanout of the belled piles prior to concrete placement.

The installation of spread footings within the main portion of the building commenced after the radiation vault piling was complete and the excavation reached bearing elevation. As the excavation progressed towards Level 5 elevation (17.4 m below grade), isolated lenses of a saturated fine-grained, low permeability, silty soil (referred in some publications as "Bull's Liver") were encountered within the till. These regions would quickly lose strength upon being exposed and would eventually flow when unsupported. Typically, these regions consisted of isolated pockets that could be over-excavated to reveal the surrounding more competent till and the individual foundation pads could be excavated as a rectangular prism. Most of the building foundation pads were cut into the till below floor slab elevation and cast neat against the surrounding soil (Figure 7). The low hydraulic conductivity of the surrounding clay till soils resulted in low rates of groundwater seepage within the cuts. Typically, the pads would be cut to dimension and the concrete poured immediately after completion to reduce degradation of exposed bearing surfaces and influx of groundwater.



Figure 7. Typical foundation pad cut within till



Figure 8. Foundation pad with sloped wall after removal of weak soil zone

In locations where the saturated cohesionless soils extended beyond a footing pad's perimeter and where space allowed, the weak side of the footing pad excavation was sloped up to the floor slab subgrade and wooden forms were utilized to cast the foundation pad (Fig 8). The use of over-excavation of cohesionless soil zones and partial footing pad forming allowed for foundation installation to proceed smoothly.

The majority of building footings were successfully installed as excavation continued towards the northwest portion of the site. The excavation in this portion of the site had not been completed to depth as it contained the remnants of the access ramp, previously used to allow truck access in and out of the site excavation. As well, this region was previously noted for having extensive sand and silt lenses based upon a review of existing borehole data and records from perimeter shoring pile installation. As the remainder of the ramp soil was excavated, it became apparent that the frequency of saturated cohesionless soils exposed at the base elevation of the excavation in this area was increasing.

The footing excavation for Tower Crane 1 required a large excavation to a depth of 2.6 m below the surrounding grade. A test pit was excavated ahead of performing the full footing excavation for Tower Crane 1 due to concerns with exposing too much weak soil at once. The location of the test pit and the extent of excavation at that time are shown on Fig 9. The wall of the test pit cut cleanly and initially stood vertically. Groundwater was observed seeping in through the walls and base of the test pit which quickly led to loss of strength and sloughing (Fig 10). Further excavation and site observations indicated that the zone of weak cohesionless soil observed in the test pit extended beneath the remaining portion of the Tower Crane 1 base excavation and appeared to be continuous in the direction of the adjacent shoring wall.

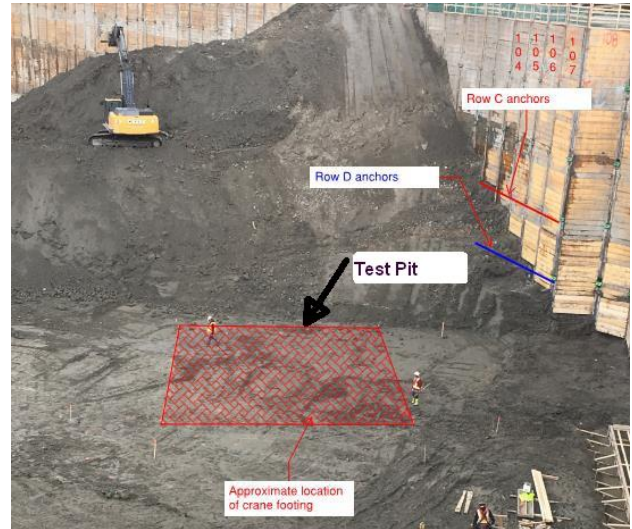


Figure 9. Tower Crane 1 footing test pit location

The probable extent of the cohesionless soils raised concerns for the installation of the remaining footings and stability of the temporary shoring due to the depth of soil to be removed near the shoring wall toe. The remaining soil ramp was flattened to facilitate drilling a series of 46 shallow boreholes within the excavation to help refine the approximate lateral and vertical extent of the cohesionless soil zone ahead of further excavation. It became apparent from the boreholes that the cohesionless soil varied in lateral and vertical extent. The approximate plan view regions of thick layers of saturated cohesionless soil are shown on Figure 11.



Figure 10. Sloughing soil in Tower Crane 1 test pit



Figure 11. Excavation boreholes and cohesionless soil

A review of the shoring stability revealed that the probable extent of cohesionless silt and sand would result in a loss of shoring stability upon further excavation. The shoring design would require modification to allow for safe completion of the basement level excavation and foundation installation. Several construction staging and design modification options were considered in order to complete foundation installation while maintaining excavation stability. Ultimately, an additional row of tie back anchors was installed along the base of the temporary shoring where the cohesionless soils were encountered. A secant pile wall was also installed running along the toe of the shoring system to provide additional support and to allow for deep excavation to occur immediately adjacent to the shoring wall toe.

Within the area of concern, the additional row of tie back anchors was installed along the base of the wall, followed by further excavation and installation of the secant pile wall. The bottom of the excavation was established without incident and the two footing pads immediately adjacent to the shoring wall were excavated and poured successfully. Excavation of Tower Crane 1 footing commenced in stages, requiring a mud slab to maintain the base next to the two adjacent footings. Within the cohesionless soil zone exposed along the west side of the crane footing, the excavation was sloped away from the base and concrete Jersey barriers were set at the toe of the slope. Filter fabric was placed on the slope and it was backfilled with drain rock to maintain the side of the excavation. A pump was placed within the drain rock to intercept groundwater. This allowed for the “flowing soil” along the side of the excavation to remain confined. The remainder of the footing was excavated, and concrete was placed successfully. The rest of the building footings were completed easily in comparison, requiring only shallow excavation without encountering significant cohesionless soil.

Figure 12 shows the footing excavation for Tower Crane 1 just as drain rock was to be placed behind the Jersey barriers. Evident in the photo is the secant pile wall along

the toe of the shoring wall and the mud slab installed within a portion of the base of the pad excavation. Only a portion of the pad side wall required stabilization using Jersey barriers and drain rock backfill. The weak cohesionless soil was in stark contrast to the surrounding competent till which can be seen cut vertically along the remainder of the pad.



Figure 12. Completion of Tower Crane 1 footing (facing west)

## 7 ACKNOWLEDGEMENTS

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## 8 REFERENCES

Moran, S.R. 1986. *Surficial Geology of the Calgary Urban Area*, Alberta Research Council, Bulletin No. 53.