Coquitlam Ultraviolet Disinfection Facility Geotechnical Investigation, Design and Construction Challenges

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Des défis du Nord au Sud

ABSTRACT
Metro Vancouver (MV) manages and supplies drinking water for the 2.4 million residents in 21 municipalities of the Lower Mainland. In order to comply with new requirements under Health Canada's Guidelines for Canadian Drinking Water Quality, in 2014, MV upgraded the treatment facilities in The City of Coquitlam with ultraviolet (UV) disinfection. The constraints at site for the new facility included adjacent slopes with a history of instability; the nearby Coquitlam River and an operating chlorination plant, as well as other infrastructure, in close proximity to the proposed facility. The geotechnical design and construction challenges presented by these constraints were significant. The final design had a smaller overall footprint compared to similar facilities, but deeper underground structure. Excavations up to 17 m in depth within a varved, non-plastic silt required careful design, extensive dewatering, and shoring. The paper discusses the geological and physical site conditions, the surface and subsurface investigation completed to characterize the soil and groundwater conditions and the geotechnical design and construction of the works.

RÉSUMÉ
Le Metro Vancouver, MV gère et fournit de l'eau potable pour les 2,4 millions de résidents répartis dans 21 municipalités du Lower Mainland. Afin d'atteindre les nouvelles normes de Santé Canada pour la qualité de l'eau potable au Canada en 2014, MV a renouvelé les centrales de traitement des eaux dans la ville de Coquitlam avec un système de purification à base de rayons ultraviolets (UV). Des contraintes sont associées à la localisation du site: des pentes adjacentes ayant des antécédents d'instabilité, la Rivière Coquitlam à proximité, ainsi que la présence de plusieurs établissements et infrastructures, tels qu'une usine de chloration en opération. Ces contraintes ont représenté d'importants défis pour la conception et la construction. La conception finale a une empreinte globale plus petite comparée à des installations similaires, mais possède une structure souterraine plus profonde. Des excavations jusqu'à 17 m de profondeur dans un silt varvé et non-plastique ont requis une conception minutieuse, de l'assèchement des sols et de l'étayage. Cet article traite des conditions géologiques et physiques du terrain, de la caractérisation des conditions de sol et des eaux souterraines, ainsi que de la conception géotechnique et la construction des ouvrages.

1 INTRODUCTION

The Coquitlam UV disinfection facility (UVD) is located within the upper Coquitlam River Valley, north of the City of Coquitlam, BC (Figure 1). The facility treats and supplies approximately 370 million litres of potable water on an average day, or approximately one third of the total water supply delivered in the region

Figure 1. Coquitlam UVD site location.

(Metro Vancouver, 2015). The need for the facility arose from Health Canada Drinking Water requirements which necessitated additional disinfection. Design for the project commenced in 2008; construction commenced in spring 2011. The plant has been operational since early 2014.

2 SITE AND PHYSICAL FEATURES

The Coquitlam Lake Reservoir, located at the northern end of the valley, is fed by runoff from the surrounding mountain streams which drain into Disappointment Lake. The Coquitlam River, regulated by the reservoir dam, flows from the reservoir to the lower flood plain and ultimately into the Fraser River, some 13 km south.

The valley walls comprise glacial deposits with terraces on the lower valley formed by river erosion. Post-glacial alluvial sediments laid down by the Coquitlam River cover most of the valley bottom. The terraces and slopes are densely vegetated with mature evergreen trees.

The site is located at the toe of the western valley wall, within a local widening of the valley bottom north of the existing chlorination plant and west of Pipeline Road, the access road to the reservoir (see Figure 2). Poorly drained upland swamp and bog has accumulated at the toe of the slopes. The pre-construction site was covered with fill associated with previous development forming a prominent mound. Ground surface elevation within the site varied between about +116 m and +120 m. Terrace slopes to the north and west rise steeply to approximately +132 m before levelling off and joining the valley wall.

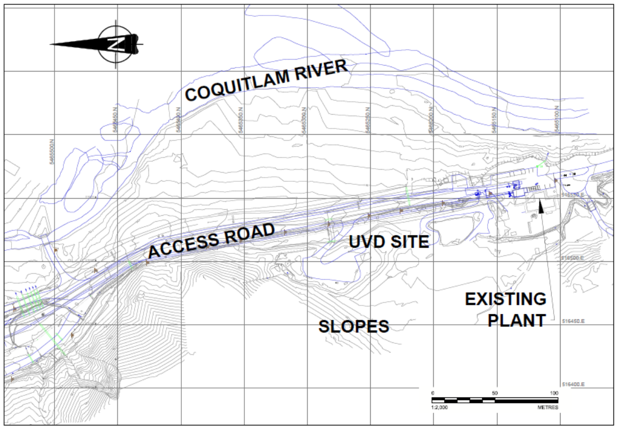

Figure 2. Site plan.

3 GEOLOGY

The Coquitlam River Valley is a glacier-carved bedrock valley (Armstrong and Brown, 1954). Quaternary deposits which have been deeply incised by the Coquitlam River have filled the valley through a series of at least two major glacial advances and retreats and at least one non-glacial interval (Hicock, 1973). Deposition and erosion, in conjunction with glaciation, sea level changes and isostatic rebound have resulted in highly complex surficial geology.

The major surficial geological units in the area include glacial drift and till-like deposits of the Vashon Drift, and undivided pre-Vashon Deposits. The Vashon deposits consist generally of till, glaciofluvial, glaciolacustrine and ice-contact soils. Within the site, these soils comprise sand and silt tills and finer-grained, glacio-marine and glacio-lacustrine silts. Sand and gravel outwash deposits are also indicated in the area (Armstrong 1957). In addition to the glacial deposits, Pre-Vashon deposits consisting of glacial, non-glacial and glaciomarine deposits are indicated on geological maps of the area (GSC 1484A, 1980). These deposits, which generally underlie the glacial deposits, consist of Quadra inter-glacial fluvial channel fill and deltaic sands and gravels. Extensive sand and gravel deposits are exposed to the south of the site and active quarry operations continue today. Sea level rise during non-glacial events resulted in the deposition of glaciomarine sediments within the valley which during this time was a fjord.

There appears to be evidence that ice dammed the Coquitlam River valley during historical interglacial events (Hicock, 1973). This would have resulted in extensive deposition of outwash sands and gravels, which are evident in the lower valley in quarry exposures. The appearance of varved silts on the valley walls and lower terraces near the site, could be explained by the formation of a glacial lake. Subsequent melting of the stagnant ice would have resulted in extensive erosion and downcutting of the deposits and the formation of terraces. Post-glacial alluvial sediments of sand, gravel, cobbles and boulders have been deposited over the valley floor by the Coquitlam River.

4 DESIGN CONCERNS

Prior to the construction of the UVD the existing Coquitlam water treatment facilities included chlorination and ozonation disinfection, as well as corrosion control. The new UVD is a 1,600 m^2 facility comprising a UV Reactor Building, housing eight 1000 Watt reactors; an operations and maintenance centre, which includes a control room, offices, meeting rooms and a water testing laboratory; and associated valve chambers and piping (Figure 3).

Figure 3. Site plan showing borehole and monitoring well locations.

Two water mains, the Contactor Main, a 3.1 m diameter steel pipe; and the Coquitlam Main No. 3, a 2 m diameter steel pipe transport raw water from the reservoir through the ozonation treatment plant, via a Quenching Chamber, to the UVD. At the north end of the plant , the Contactor and Coquitlam mains divert into Valve Chamber 7 (Combined Valve Chamber), located north of the UV Building. The foundation slab of the valve chamber is at elevation +107.8 m and it is founded at about +106.9 m. From the valve chamber a series of four pipes convey water into the UV Reactor Building through a secondary valve chamber adjoining the building.

The Reactor Building houses a valve chamber at its north end. The UV reactor galleries, mechanical and electrical rooms, and the operations and control centre are at its south end. The foundation elevation of the UV Building varies from about +108.5 m in the valve chamber, +109 m in the galleries to +112 m in the mechanical, electrical and operations and control centre. The depth to foundation below the existing ground surface varies between about 6 m at the south end to almost 17 m adjacent to the slopes.

The presence of existing infrastructure, including the raw water conveyance mains, the existing chlorination facility, and other associated above- and below-ground features impose significant design and construction constraints. At the north end of the site, excavation was required to cut into the slopes, resulting excavations of substantial depth. The varying elevation and depth below existing ground level of the foundation of different parts of the building

required the design of excavation to be a combination of vertical shoring and open excavation.
Considering the depth of the excavation and the proximity of Coquitlam River, a clear understanding of the hydrological regime influencing the groundwater at the site was critical for the design of excavation.
The depth of the structures and their configuration meant that, for the most part the foundation loads are generally less than the previously existing overburden. Bearing failure of the foundation was therefore not a prime design consideration.

5 GEOTECHNICAL INVESTIGATION

Geotechnical investigation was carried out in six phases: field reconnaissance; terrain hazard assessment; preliminary geotechnical borehole investigation; supplemental borehole investigation; hydrogeological investigation and pumping test; and test excavations. During the conceptual stage of the project, plant siting and overall layout were established. This was followed by preliminary design and geotechnical investigation. Preliminary design saw a significant change in plant configuration from a horizontal arrangement of reactors to vertical which resulted in a deeper foundation. During detailed design, supplemental investigation and *in-situ* hydrogeological testing was carried out to obtain deeper and more detailed information in part as a result of design changes.

5.1 Field Reconnaissance

Field reconnaissance included visual inspection of the site, the terrain and exposures of soils within the valley walls. Till-like and glaciolacustrine varved silts were observed at the foot of the valley wall and on the terraces (Photograph 1). Prominent banding of the soils with alternating lighter and darker silt, clayey silt and sandy silt layers was observed. Occasional gravel and cobble inclusions embedded in the varved silts visible within the exposures indicated drop stones were likely to be present within the body of the silt. Poorly drained swampy conditions were evident at the foot of the slopes. The site for the proposed UVD was a sparsely vegetated fill platform, likely formed by stock piling spoils from excavation for the construction of the adjacent structures. (Photograph 2).

5.2 Terrain Hazard Assessment

Terrain assessment which included on-site inspection of the slopes to the north and west of the site indicated few active geomorphic processes and associated localized surficial slope instability. These included surficial soil creep and gulleys.
Review of the available historic aerial photographs reveal a mass-wasting event, which was confined to a steep northeast-facing slope about 125 metres south of the existing chlorination plant and to the west of Pipeline Road. A concave feature, possibly a ravine, was observed upslope from and adjacent to the site.

Photograph 1. Surface exposure of varved silt at foot of north slope. Note alternating dark and light layers and gravel.

Photograph 2. View of pre-development site looking north. Chlorination building in foreground.

The sidewalls of this feature did not indicate instability, but a fan-like feature existed at its bottom. Recent aerial photographs reveal a noticeable thinning and deterioration of the forest vegetation.

5.3 Preliminary Borehole Investigation

A preliminary borehole investigation was carried out to obtain subsurface information over the site. It consisted of four mud rotary boreholes (Figure 3). Drilling was carried out using a B-53 mud rotary drill rig equipped with H-rods, open hole advance and a 100 mm diameter tricone drill bit. The Standard Penetration Test (SPT) was conducted in accordance with ASTM D1586-08A at 0.75 m to 3 m intervals within all boreholes in order to collect samples and obtain penetration resistance data on the underlying soils. The boreholes were drilled to depths, which varied from 5.5 m to 13.9 m below the existing ground surface. Two 19 mm diameter PVC standpipe piezometers each with 1.5 m screen were installed in order to allow monitoring of the groundwater level at these locations. Water levels were measured in piezometers manually using an electric water-level meter. Geotechnical index testing, including Atterberg Limits, water content determination and grain size analyses were carried out on selected samples obtained from the investigation.

5.4 Supplementary Borehole Investigation

Following preliminary investigation and during the development of the design, a significant design decision was made by the project team to change the UV Reactor Building Bay configuration from horizontal to vertical. This was done in large part because of the limited space available for construction, but also because of operational reasons. In addition, the design location of the combined valve chamber regulating the water flow to the UV plant was positioned further to the north toward the slopes. These design changes resulted in both a deeper foundation of the facility and the need to excavate more significantly into the adjacent slopes. The preliminary investigation data were not considered to be sufficient and a supplemental investigation was initiated at the start of detailed design.

The supplemental investigation conducted in January 2009 comprised two boreholes (BH09-1 and BH09-2), drilled to 26 m in depth using a truck-mounted Sonic ATV drill rig (Figure 3). Sonic drilling utilizes a dual-cased single tube core barrel system that employs high frequency mechanical vibration to obtain continuous core samples of the soils. Standpipe piezometers were installed in both boreholes to depths of 24.4 m and 15.2 m, both with 3.1 m long screens. Single-well-response tests were conducted in both boreholes to measure hydraulic conductivity of the material within the recharge zone of the piezometer screen.

5.5 Hydrogeological Investigation and Pumping Test

The data from the Preliminary investigation indicated that the groundwater level was at about elevation +111 m to +113 m. This meant that during construction, the subgrade would be up to 7 m below the groundwater level. At its nearest point the excavation was between about 80 m and 100 m from Coquitlam River. A key question with implications for dewatering design was whether and to what extent the river was hydraulically connected to the subsoils underlying the site. Another question was the amount of groundwater inflow expected through runoff from the adjacent slopes. A hydrogeological investigation and a pumping test was undertaken to understand the hydrogeological conditions likely to impact the excavation and foundation design.

A total of four monitoring wells were constructed at the locations shown on Figure 3. A track-mounted sonic drilling rig was utilized to advance 150 mm diameter casing to depths ranging from 7.6 m to 18.3 m below ground surface. These were drilled in order to monitor groundwater drawdown at specific distances from the test well, and to obtain additional subsurface information near the Coquitlam River.

Each borehole was converted to a monitoring well by installing a single 50 mm diameter PVC pipe to the base of the boreholes. The bottom of each pipe included a 1.5 m long section of machine-slotted screen.

Pressure transducers with automated dataloggers were installed in the monitoring wells. All devices were programmed to record water pressures in a synchronized manner at a 5-minute interval. These data were converted to equivalent elevations using monitoring well collar elevations.

A steel-cased Test Well was constructed on the site within the proposed excavation area (Figure 3). A truck-mounted air-rotary drill rig was used to construct a 150 mm diameter steel well to a depth of 25.0 m. The well was completed with a 2.4 m long section of 0.25 mm slotted stainless steel screen installed between 22.2 m and 25.0 m. The Test Well was designated TW10-01.

A 24-hour pumping test in TW10-01 was carried out to assess the groundwater response to the withdrawal of water from the proposed excavation area. Pumping from TW10-01 commenced August 18, 2010. A pumping rate of 30 US gpm was selected as the maximum pumping rate to be applied to TW10-01 during the anticipated 24-hour pumping period. Throughout the pumping test, water levels in TW10-01, BH08-01 and BH08-4 were measured. Water levels in TW10-01 were also recorded continuously by an automated transducer installed prior to commencing the pumping test.

After 24 hours of continuous pumping from TW10-01, water levels in all monitoring wells were manually measured to determine the extent and magnitude of the hydraulic effects imposed by the pumping test.

Based on review of the drawdown data, it was determined the pumping test duration should be extended to 48 hours to obtain data suitable to support the dewatering analysis and associated numerical modeling. After 48 hours of pumping from TW10-01, water levels in all monitored wells were again measured and it was confirmed that water levels within the monitoring wells had stabilized.

6 SUBSURFACE CONDITIONS

The results of preliminary mud-rotary drilling indicated the presence of about 2.1 m to 4.9 m of surficial, mainly granular fill overlying about 1.7 m to 3.8 m of native sediments consisting of coarse-grained sediments. The recovery of SPT split spoon samples through this layer was poor. The tricone drill bit experienced difficulty in advancing through this layer and the drilling suffered loss of circulation confirming its granular texture. This layer is considered to represent alluvium. Low-plasticity silt underlies the alluvium. The silt deposit extends to depths of about 18.3 m to 20.1 m below ground surface (elevation 97.7 m to 100.2 m). SPT blow counts recorded within this layer ranged from 24 to 59 blows per 300 mm penetration with an average value of about 42. Groundwater levels within the alluvial layer ranged from elevation 111 m to 114 m.

Supplemental drilling with the sonic drill confirmed the overall stratigraphic sequence encountered in the Preliminary Investigation, but revealed greater detail on the fabric of the subsoils. Sand, gravel and cobbles were encountered in the core samples obtained from the alluvium. The varved nature of the silt was confirmed. The silt is predominantly low- to non-plastic with alternating layers of low-plasticity clayey silt and granular sandy silt to silty sand. The supplemental drilling was extended through the lower horizon of the silt which was underlain by sand with layers of gravelly sand and silty sand. The supplemental boreholes were terminated within the sand. The pumping test described above was carried out within the underlying sand deposits. A stratigraphic section through the site is shown on Figure 4.

Atterberg Limits tests on samples of the silt indicate it to be low- to non-plastic. Most tests returned non-plastic results with two tests indicating a plasticity index of about 2 per cent. Grain size analyses were conducted on samples of the silt. The results show the consistency of the grain size distribution and that the material comprises mostly silt (Figure 5). Water content of samples of the silt ranged from 17 to 22 per cent. Grain size analysis of selected samples of the deep sand layer indicate fine to medium sand to silt and sand mixtures (Figure 6).

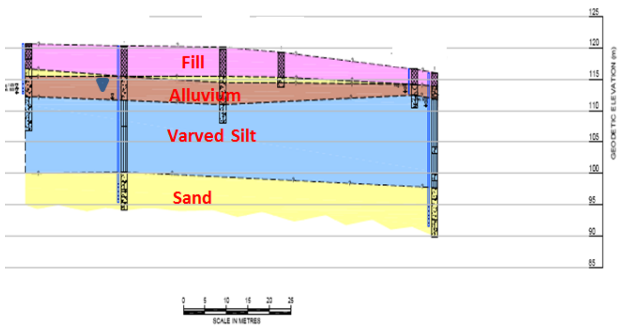

Figure 4. Stratigraphic section through site.

The four monitoring wells installed during the pumping test, were drilled some 80 m to 100 m east of the site, closer to Coquitlam River. The drilling for these wells either did not encounter the silt layer, or found it only in thin layers. Erosion by the river appears to extend completely through the silt and into the underlying sands. This is supported additionally by the fact that the piezometric levels within the sand are similar to the river level.

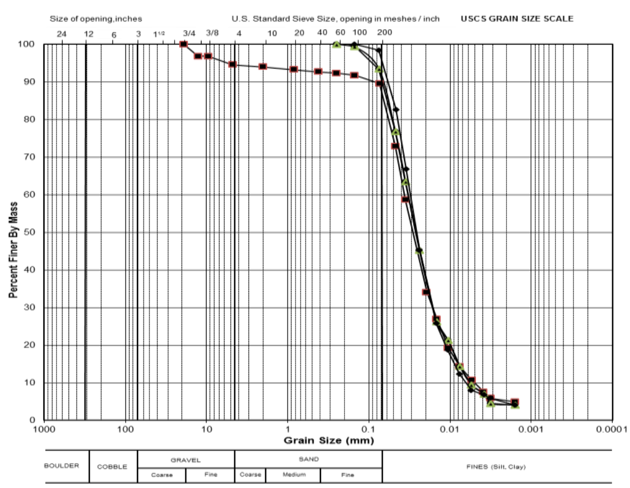

Figure 5. Grain size distribution test results in silt.

The hydraulic conductivity (K) of the subsurface materials was computed using the Bower-Rice (1976) solution in AQTESOLV©, a commercially available software package for aquifer test analysis. The Theis, Theis Recovery and

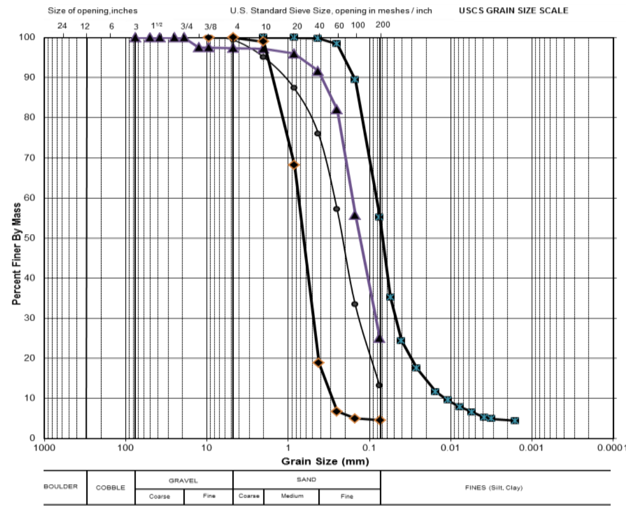

Figure 6. Grain size distribution test results in deep sand.

Butler solutions in AQTESOLV© were also utilized to analyze the long-duration pumping tests data. Based on the results, together with the hydrogeological data collected during the previous site investigations (Golder, 2009), the bulk hydraulic conductivity of the sand/gravel aquifer underlying the silt sequence was estimated to range between 6×10^{-5} m/s and 1×10^{-4} m/s.

7 EXCAVATION AND DEWATERING DESIGN AND CONSTRUCTION

Construction of the UVD required excavations up to 17 m in depth through coarse-grained alluvium, and into non-plastic silt. The low plasticity and fine-grained nature of the silt was highly susceptible to disturbance and erosion. Since the foundations were all within the silt layer, key to preserving the foundation integrity was adequate subgrade preparation and control of surface and groundwater inflows. This required suitable excavation and surface and groundwater control.

Figure 7 shows the conceptual excavation design in plan. The presence of the contactor pipe immediately to the east of the facility and the existing chlorination facility to the south, required vertical shoring along these edges. The importance of the main and the operating chlorination plant and the need to limit deformations and loss of support required careful design to produce a shored excavation that avoided these problems. At the north end of the site where the excavation encroached into the slopes, vertical shoring was necessary to limit excavation depth. Along the west side of the site, room was available to develop open cut excavation slopes at nominal 1 horizontal to 1 vertical gradients.

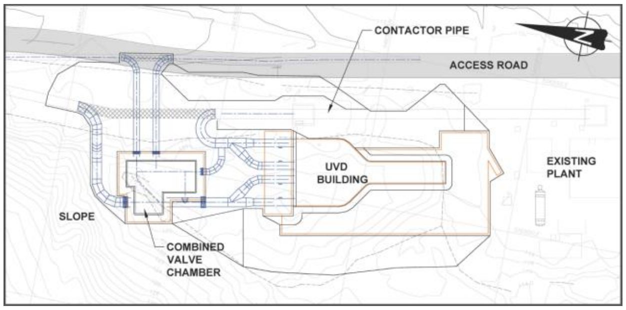

Figure 7. Excavation design plan.

Groundwater control was accomplished with perimeter dewatering wells seated within the deep underlying sand. These were equipped with eductor pumps within wells which pumped to a manifold on surface (Photograph 3). Dewatering wells were installed about a month in advance of full excavation in order to allow piezometric levels to stabilize and to develop some depressurization within the low-permeability silt. Estimated pumping rates based on

Photograph 3. Perimeter dewatering wells.

the pumping test were on the order of 2,200 m^3/day 3,200 m^3/day. Groundwater within alluvium was managed with the use of cut off trenches which served a secondary purpose of relieving overburden and overall vertical cut depths.

A number of options were available for vertical shoring, including shotcrete and anchors, soldier pile and lagging with tie-backs, and secant piles with anchors. Soldier piles and lagging were not selected due to the highly erodible nature of the low-plastic silt which was considered likely to migrate through the lagging and result in severe loss of ground. For modest cut depths of up to 6 m, shotcrete and anchor shoring was selected by the contractor as the preferred methodology. Along the west side where the Contactor Main was within a few metres of excavation, design incorporated a relief cut to initially expose the main in order to unload the pipe, limit cut heights and to allow monitoring and protection of the main.

The coarse-grained alluvial layer presented a challenge for excavation design given the presence of cobbles and boulders. Where possible, cut slopes were developed through this layer to avoid having to develop vertical cuts which were challenging due to the potential for face instability. Where this was not possible, along the south edge, cuts were developed in narrow panels with berms in between. Boulders were cast directly into the shotcrete facing. Loss of ground was limited.

Shoring along the north side of the site required special attention given the significant cut heights and the uncertainty in the subsurface soil and groundwater conditions underlying the slopes which were not accessible for investigation during design. Initial attempts by the contractor to use conventional shotcrete and shoring were unsuccessful, due to severe face instability, loss of ground behind the shotcrete and challenges with managing run off (Photograph 4).

Secant piles (600 mm diameter) were selected during construction to complete the shored excavation in this area. Soil anchors were drilled on 1.8 m centres through the secant piles. Secant piles were drilled to approximately 0.5 m below subgrade elevation.

Photograph 4. Collapse of shotcrete facing due to erosion from surface/subsurface flow.

Excavation proceeded easily to subgrade elevation in conjunction with installation of soil anchors. There was concern about significant groundwater seepage at the base of the piles and the potential for softening of the subgrade soils due to the limited embedment. To control against this, sand backfill around the dewatering wells installed behind the wall provided some drainage to the silt and reduced seepage. Conventional weep drains were installed through the wall for additional drainage.

Open cut excavation along the west side and through the alluvium on the east side were relatively stable throughout construction. Interceptor drains were installed at the base of open cuts through the alluvium which were connected to conventional sumps with pumps. Protection of the cut face was provided by simple plastic sheeting which worked reasonably well; however, erosion and transport of fines was a constant problem on the site, taxing the sedimentation pond.

8 REFLECTIONS AND LESSONS LEARNED

The UVD was an ambitious undertaking that faced a number of challenges during excavation and throughout construction. A number of lessons were drawn from the project.

Subsurface investigation for large projects within complex geological conditions should be carried out in stages and include a combination of established standard methods such as the Standard Penetration Test within mud rotary boreholes, but importantly should also include some form of continuous core sampling, such as Sonic drilling. The value of Sonic drilling this is that near-continuous core samples of the subsoils can be obtained. This proved invaluable in obtaining samples of the alluvial material, where the SPT split spoon sampler was incapable of acquiring samples, and in silt. This information allowed the designers to understand the detailed fabric of the silt and to confirm its engineering characteristics and provided ample amounts of soil for testing. This information was provided in the RFP documents given to the contractors. It is believed the detail and quality of the information provided to the contractors helped reduce the number of contractor's claims based on "changed ground conditions" and assisted in fairly adjudicating the claims when they were received.

Control of groundwater was critically important to the success of the project. This became evident during construction of shoring along the eastern edge of the site next to the Contactor Main. Inadequately installed weeper drains, together with an initial attempt to cut this side full-height without interceptor drains within the alluvium resulted in loss of ground behind the shoring and compromised support to the main.

The value of robust technical specifications bears emphasis. Technical specifications for the shoring, excavation and dewatering included clauses specifying that these designs and their execution in the field be coordinated by the individual contractors. This approach acknowledged the interdependence of these designs and their construction. This provided the design team and the owner with an important tool to ensure coordination and avoid incompatible designs.

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