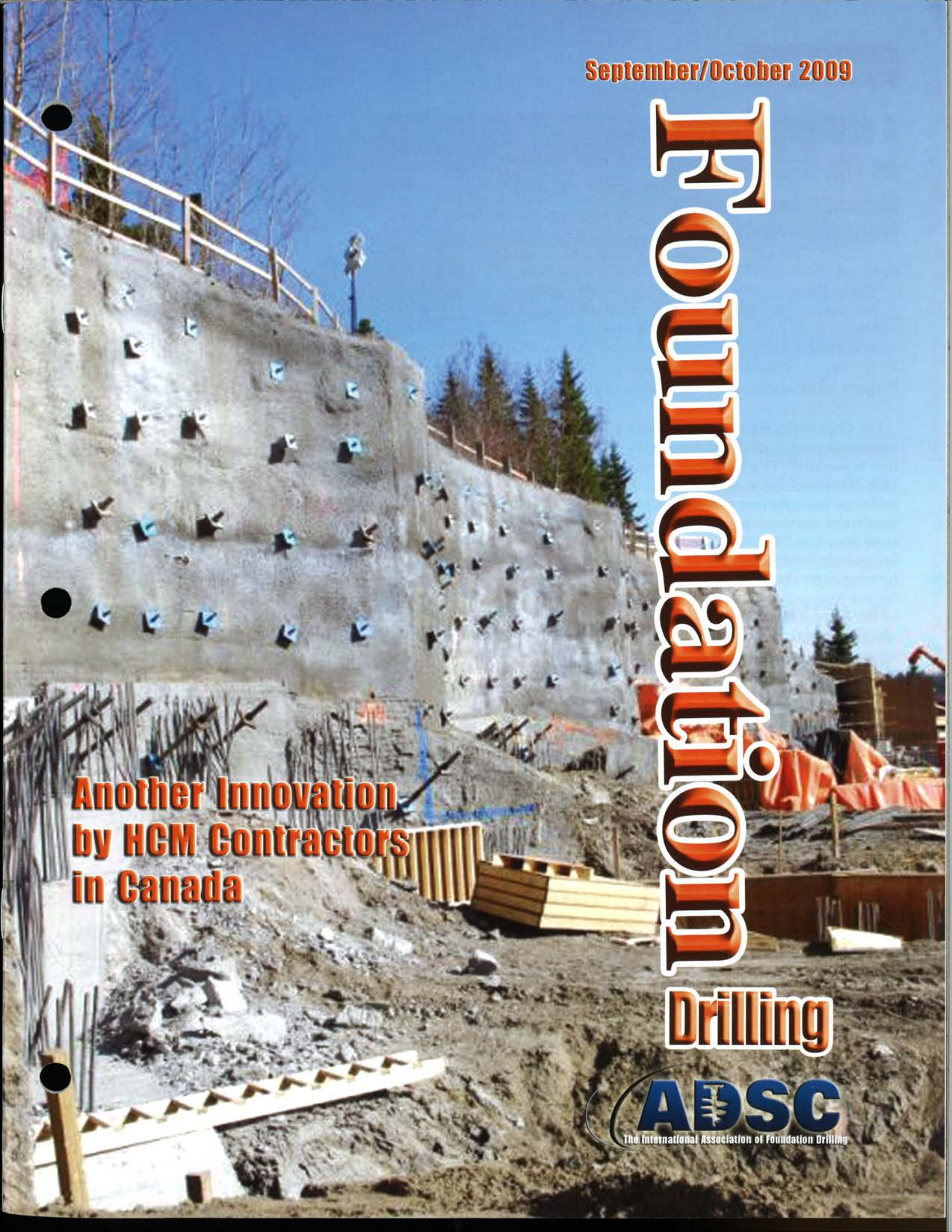

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Foundation Drilling

Another Innovation by HCM Contractors in Canada

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COVER FEATURE

A Re-Design Proves to be the Best Solution

Whitemud Drive Retaining Wall, Edmonton, Alberta, Canada

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Project Summary

The City of Edmonton is undertaking a multi-stage project, scheduled for completion in late 2010. This includes widening both Whitemud Drive and the Quesnell Bridge to ensure the transportation system is safe and continues to keep pace with rapid growth in Edmonton. The Quesnell Bridge is the busiest bridge in Edmonton, carrying approximately 120,000 vehicles

The HCM/Isherwood design-build team's innovative design for the retaining wall was proposed to the City of Edmonton as an alternative to the original tender.

on an average day. The additional lanes and widened shoulders will enhance traffic flow and improve the overall safety of the road. This project is the first major rehabilitation of the Quesnell Bridge since its construction in 1967.

The widening of the road north of the Quesnell Bridge is made possible by a cast-in-place retaining wall constructed in front of a temporary tieback/soil-nailed shotcrete shoring system. The new retaining wall replaces a two-tier bin wall that was installed in 1972 to address a previous hillside failure. Analysis of the slope failure in 1971 generated soil strength parameters issued in the tender soil report documents. In the field, however, this was revealed to be incorrect; the actual soil strength was found to be much lower.

The completed shoring wall adjacent to Whitemud Drive.

The HCM/Isherwood design-build team's innovative design for the retaining wall was proposed to the City of Edmonton as an alternative to the original tender. Figure 1 shows the original design concept: a series of high-strength, interlocking secant walls to depths of up to 25 metres (80 ft.), which raised constructability concerns with bidding contractors. Maintaining the original concept's geometry, the alternative wall consists of 14 abutting arches with a total length of approximately 250m (800 ft.), as shown in Figure 2. The wall is supported on

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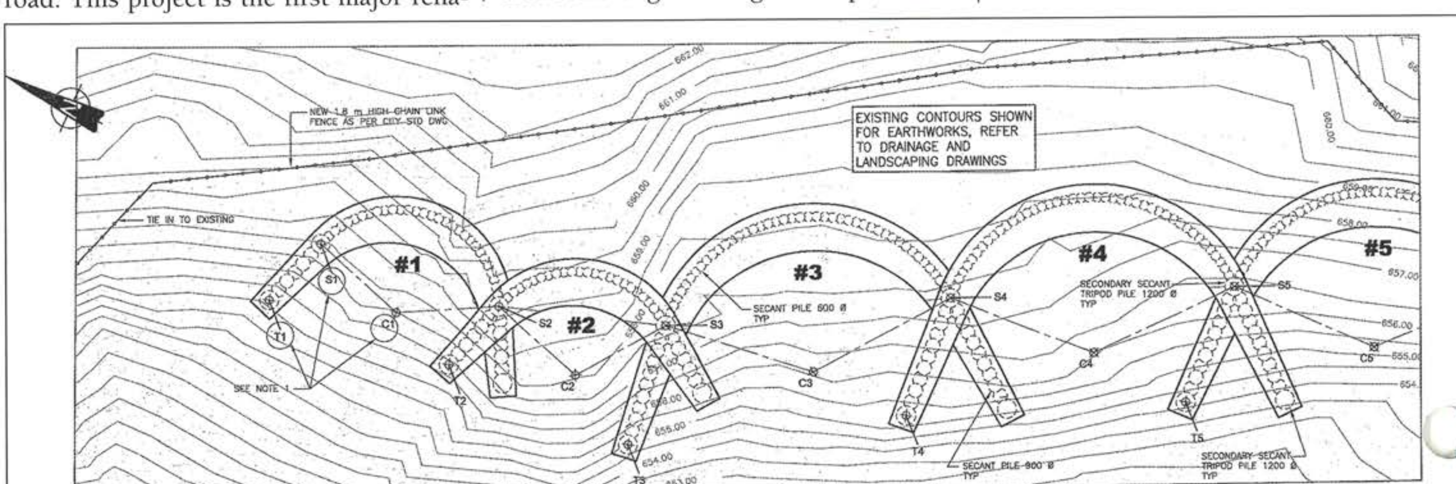

Figure 1: Tendered Design of the Whitemud Drive Retaining Wall.

1200mm (48 inch) drilled caissons, capped by robust ring footings that step downward into the low slope as it approaches Whitemud Drive.

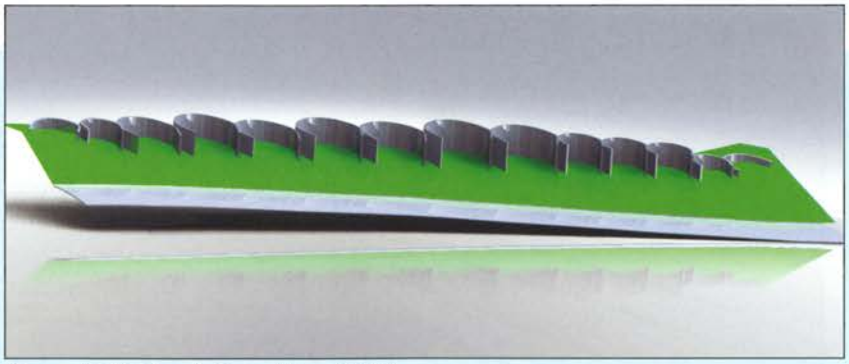

Figure 2: 3D Rendering of Final Wall.

Existing Site Conditions, Geology and History

Prior to construction, the existing slope ranged from 19 to 24m high above the Whitemud Drive's elevation and ascended to the roadway at an approximate overall 2H:1V slope. Existing grade was at elevation 660.0m +/- at the highest points along the wall. The subsurface geology is composed of fill material overlaying native silty clay and glacial clay till overlaying Empress sand to an elevation 630.0m +/- with bedrock below.

The native silty clay extends to elevation 654.0m +/- and is typically stiff. The Standard Penetration Test (SPT) results ranged from nine to 18 blows per foot with a natural moisture content ranging from 20% to 45%. Slickensides (polished shear surfaces caused by previous shearing) were noted to exist between the elevations of 655 and 657m during preliminary geotechnical investigations. The glacial clay till extends to elevation 647m +/, is typically very stiff, with SPT values generally between 30 to 50 blows per foot and a natural moisture content between 13% to 21%.

The Empress sand extends to elevation 630.0m +/- and is comprised of two main sub-strata. The first is an upper layer located between 645 and 647m of elevation and has a natural moisture content ranging between 6% and 20%. The second layer, located below 645m elevation, is composed of dense to very dense sand. This second layer has SPT values that are generally higher than 40 blows per foot with natural moisture content between 3% and 7%.

The upper sands layer, located between 645 and 647m of elevation, was of particular interest to the design-build team. A slope failure occurred at the site in 1971 following the 1968 slope cutting for the original construction of the Whitemud freeway. A previous geotechnical investigation determined the slope failure was caused by the stress reduction in the backslope soils resulting from a 24m high 2H:1V cut into the original ravine slope. Following the landslide, the slope was stabilized by the installation of a two-tiered bin wall at the toe of the slope seven to eight metres high with a 3H:1V regrading of the hillside. Following its reconstruction, on-going inclinometer monitoring by the City of Edmonton indicated the presence of shear movements at the elevation of this upper sand layer at a rate of about 2mm/year. The inclinometer readings were taken from 1984 to 2004, the year the inclinometers were damaged.

Concerns with Original Retaining Wall Design

The expansion of the road required the removal of the two-tiered bin walls that were installed following the 1971 slide. The new permanent retaining wall had to be installed further up slope from the original bin wall location. To improve slope stability, the new retaining wall was required to penetrate the historical failure plane in the upper sand layer and anchor into the dense sands below. The tendered retaining wall design used a secant pile arch wall, consisting of 14 overlapping arches, varying in their exposed height above the final grade from 3.0 to 7.9m. This initial retaining wall design consisted of 600mm (24 inch) and 900mm (36 inch) interlocking secant piles, with a "tripod" arrangement of 1200mm (48 inch) secant

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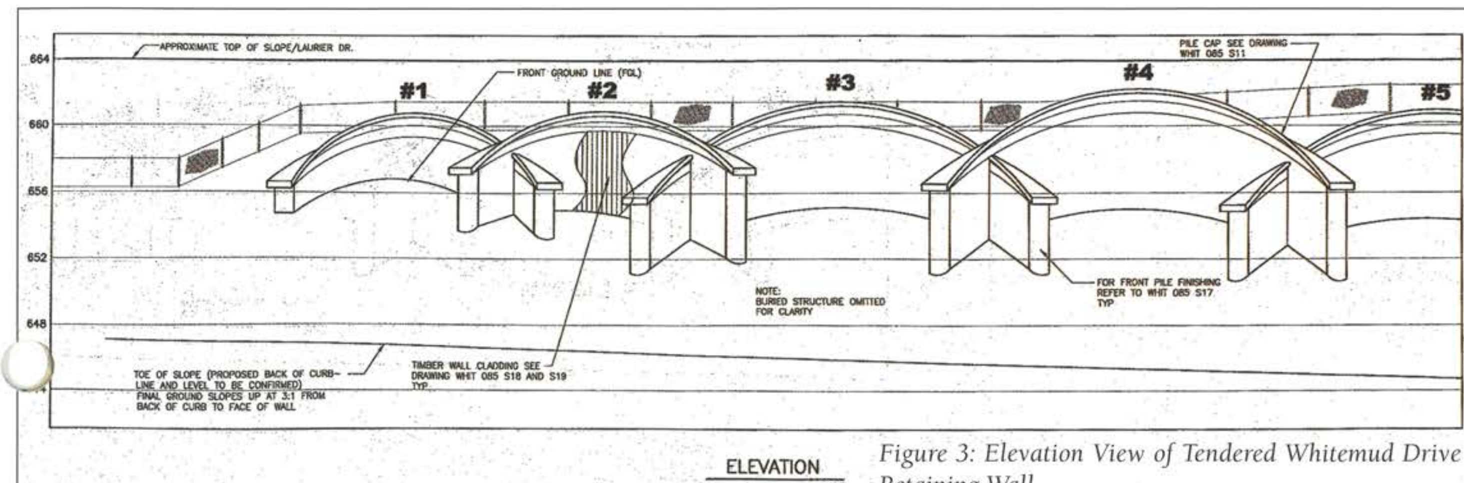

Figure 3: Elevation View of Tendered Whitemud Drive Retaining Wall.

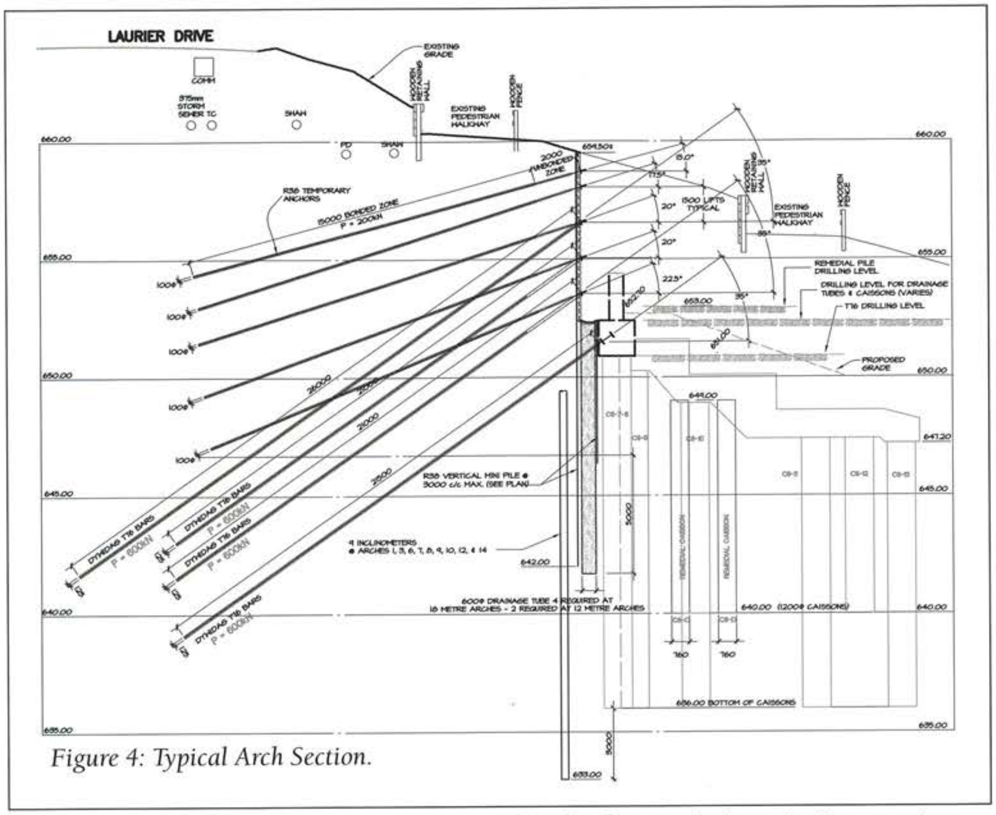

Figure 4: Typical Arch Section.

piles extending in front of the arch overlaps, as shown in Figure 3. The secant pile concrete strength was specified at 30 MPa (4400 psi) with the deepest piles extending nearly 25m (80 ft.) below the existing grade. A special curved concrete cap was required to improve both the appearance and the structural integrity of the wall, as the secants alone had no lateral reinforcement.

The tendered design presented several constructability challenges to HCM. First, maintaining the stability of the hillside and equipment and worker safety with the as-tendered design would have been difficult due to the imported-fill platforms and heavy equipment required to construct the interlocking secants, especially given the sliding history of the slope. Temporary shoring was added to the original contract scope as an addendum to accommodate the fill platforms and would have been substantial in order to support the equipment necessary to install the deep secants. Secondly, the required schedule for completion of the secant pile installation was nine months, a figure that HCM estimated would be exceeded by at least three months due to the temporary shoring and confined work areas present on the site. Thirdly, the interlocking, high-strength secant piles required to 25m design depths were problematic due to the concrete strength and severe tender penalties for any non-interlocked final conditions. Finally, even with the highest quality standards met, a lack of control joints in the design raised concerns about leakage at the secant joints.

Innovative Solutions in Design and Construction

The HCM/Isherwood design-build team proposed an alternative that provided a similar final geometry to that of the original design while providing higher structural integrity, an improved construction schedule, and significant cost-savings for the Prime Contractor. The HCM/Isherwood team delivered a solution when no other party would tender the base design with confidence.

Temporary Shotcrete Shoring

The tight schedule requirements were addressed by employing temporary shotcrete shoring. The shotcrete shoring was advantageous for two reasons: First, it could commence without any low slope shoring, greatly improving hillside stability. Second, excavation could commence simultaneously with the temporary shoring installation, as opposed to waiting for piling to be completed, greatly improving the construction schedule.

The alignment of the temporary shotcrete wall was established to allow two-sided forming with 1200mm (4 ft.) of free-draining backfill material. This eliminated the need for the concrete swale originally designed behind the top of the wall. The base of the shotcrete wall would be stepped towards the low slope direction to act as a backside form and create a one-sided forming solution for the foundation of the permanent structure, as shown in Figure 4. The temporary shoring system employed R38 hollow bar micropiles supplied by Dywidag Systems International (DSI)* to reinforce the earth mass behind the 150mm (6 inch) thick shotcrete face.

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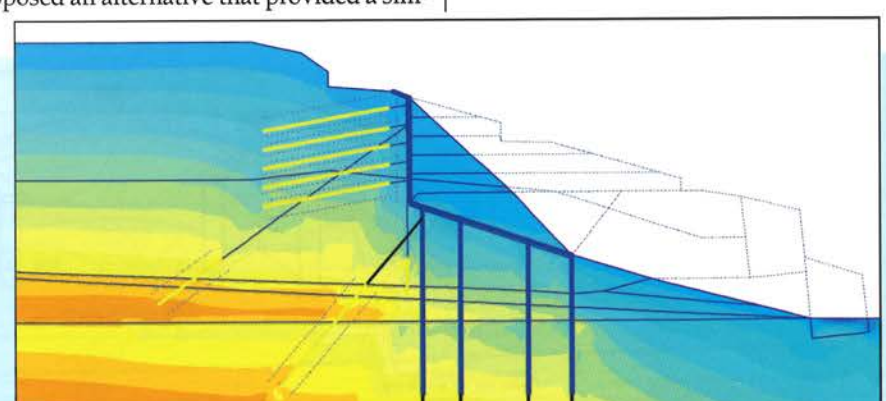

Figure 5: PLAXIS Modeling of Stress Distribution.

...ical R38 micropiles were used to add vertical capacity to the shotcrete face and to act as layout points for the irregular geometry of the arches. Longer, 1500kN (340 kip) capacity, T76S (DSI) tension micropiles were installed at a 35- degree inclination to actively anchor the reinforced soil mass to the underlying dense sands. The tension micropiles are permanent elements used in both the upper temporary shotcrete wall and the permanent ring footing.

The stressing and pre-loading program designed by HCM/Isherwood used both the active resistance of a pre-loaded micropile system and a passively reinforced soil-nail wall with 50% of the calculated design load locked in to avoid any tendency for vertical settlement of the temporary wall.

Figure 6: Excavator-mounted TEI Drill.

Permanent Retaining Wall

HCM/Isherwood designed the permanent retaining wall alternative in partnership with Peter Sheffield and Associates, ... of Canada's leading structural engineers. The final wall would consist of a 600mm (24 inch) thick cast-in-place arch that would maintain the tripod concept with one distinct difference: the new design would implement construction joints in between arch pairs to allow the loading of the tripods to occur inside of each arch as a "horseshoe" instead of the outside of the overlapping arches as originally designed. This approach would improve crack control in the extreme freeze-thaw environment of Edmonton. Maintenance on the wall would be simplified as arches acting individually could be repaired with fewer complications should problems arise in the future. The lateral reinforcement of a cast-in-place wall improved the structural integrity of the wall, eliminating the structural requirement of the wall cap. Additionally, 600mm (30 inch) drainage tubes positioned below the shotcrete wall and behind the permanent structure would prevent the build-up of hydrostatic pressure from ground water.

The exposed portion of the wall would be built upon a curved footing that would step down to match the final 3H:1V slope profile in front of the wall. Tension micropiles, to be installed at 35 degrees, were designed to mitigate total deflection and potential creep of the final structure, as well as tie the ring footing into the dense sands 9m below. 1200mm diameter, heavily reinforced concrete caissons developed into the dense sands would support the ring footing. The sands were deemed by the HCM/Isherwood design-build team to be more consistent than the bedrock and capable of high capacity at low strain. The

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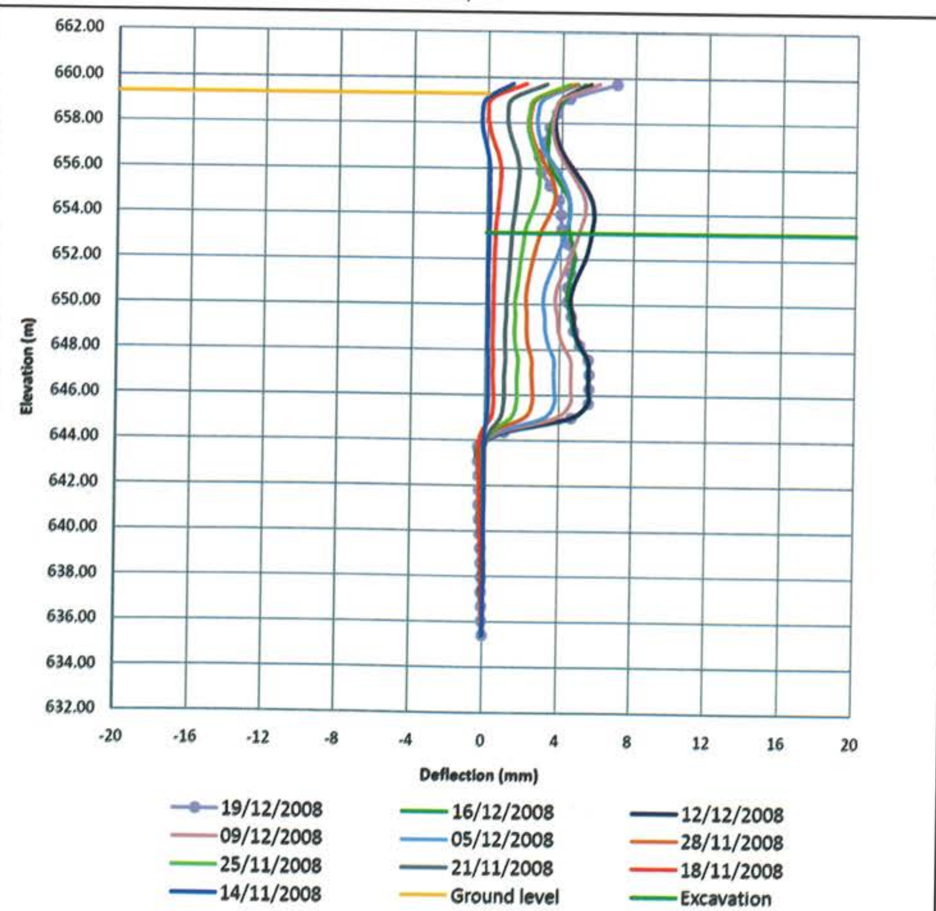

Figure 7: Mid-December Inclinometer Plot.

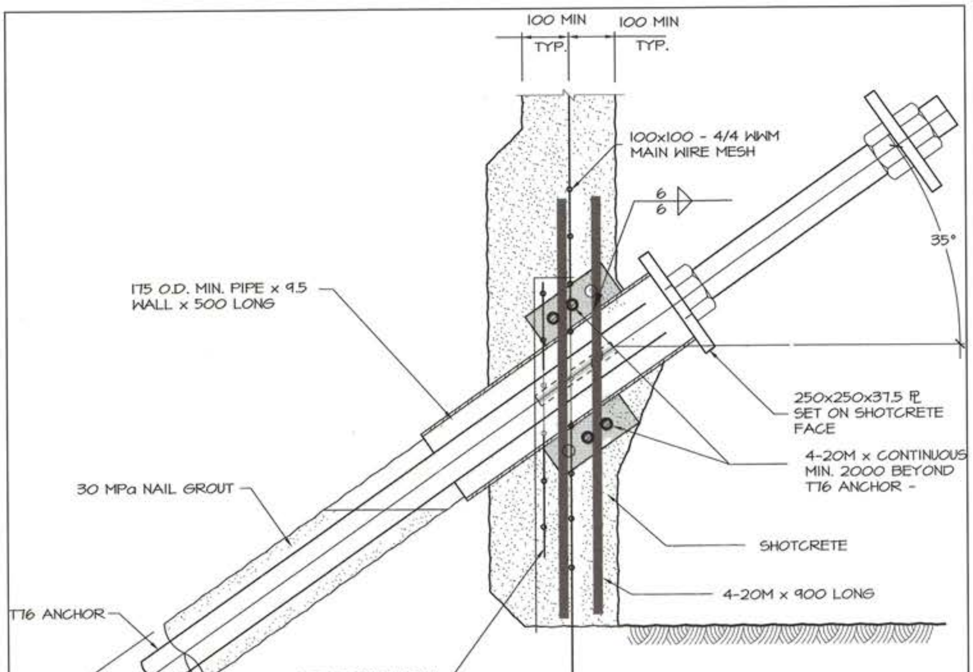

Figure 8: Pipe Connection Detail.

arches, anchored in the dense sands, would transfer active soils loads to the concrete piles pivoting between the upper and lower extremes of the arch and tripod support. The arches were designed with seven to nine caissons each and assumed a non-soil-flow rheology between the piles. Piles were spaced on average at two pile diameters. Assuming a conservative N value of two, the design incorporated the mobilization of the entire passive resistance of the soil.

Computer Modeling and Monitoring

The HCM/Isherwood design-build team relied on deformation modeling and subsequent construction slope monitoring to ensure stable performance of the shoring system and confirm the models and design parameters. Loading and resultant deflections for the design were modeled using standard slope stability analysis (limit-equilibrium) and the finite element method to provide guidance for construction progress and staging. Modeling with PLAXIS, a finite-element package used to analyze deformation and stability in geotechnical applications, enabled Isherwood/-Sheffield to model the interaction of the structures and the soil.

The historical behavior of the hillside slope presented significant risks to the project team. As such, a monitoring program was implemented to provide reliable information pertaining to shoring performance and the behavior of the suspect upper sands as excavation proceeded. Movement data was collected via nine deep inclinometers installed along the back of the shotcrete wall to depths of 21m (70 ft.), extending past the toe elevation of the caissons. The inclinometers were read weekly to supply valuable movement data on the excavated slope. Total Station Targets were also installed on the shotcrete face to monitor real displacements. Employing Peck's Observational Method, Isherwood analyzed the data on an ongoing basis during construction to verify design assumptions, compare with deformation modeling results, monitor shoring performance, and ensure the safety of workers and the public on the Whitemud freeway.

The HCM/Isherwood alternative design was accepted by the City of Edmonton's consulting team and construction began.

Initial Construction

HCM began construction in October 2008 with the installation of vertical R38 micropiles. These elements were installed between 12m (40 ft.) and 21m (70 ft.) deep into the dense sands by three TEI 550* drills mounted on Caterpillar 320 excavators, as shown in Figure 6. The pairing of the drill attachments to the highly maneuverable excavators facilitated installation of the verticals on the sloped hillside and required minimal excavation prior to drilling. Once verticals were completed, horizontal R38 anchors were installed at

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Figure 9: Hillside Congestion.

m (5 ft.) spacing. Initially, a panel sequence of shotcrete application was employed to understand how the upper clays would respond to the excavation. Once it was apparent the soils were capable of handling ribbon-style excavation, production was ramped up and subsequent lifts of the arches were completed one row at a time. All anchors were spaced in a regular grid pattern (1.5m by 1.5m), which simplified layout and helped protect the inclinometer casings.

Changed Soil Conditions

As excavation advanced to 6m below the original grade, the inclinometers along the south half of the wall indicated surprisingly large movements over a short vertical distance near the elevation 645.0m, as shown in Figure 7. This coincided with the elevation of the shear movements measured during the 1971 slide, raising concerns as the partially excavated geometry should have provided a high factor of safety. A stop excavation order was put in place for Arches C6 to C14 to allow for back-calculation of soil-strength parameters as a function of the observed hillside deformations. Isherwood immediately began a limit equilibrium analysis based on the as-built condition. The analysis indicated the soil strength parameters of the upper sands to be significantly lower than originally reported in the tender geotechnical report. The actual soil strength parameters of the slope were established assuming that plasticity had occurred within the observed movement layer and, using the limit equilibrium model, determining what soil strength parameters would yield a factor of safety of 1.0. The results of the analysis indicated a significant reduction in the internal angle of friction of the upper clay-seamed sands between the elevations 645 and 646m. The available passive resistance of the soil above this layer was significantly reduced by the weak layer. As an analogy, the resistance of the clay soils above the weak layer could be imagined as a block being pushed on asphalt. Between the elevations 645 and 646 m, it was as if the block was suddenly being pushed on a surface of ice.

In order to return the factor of safety for both the temporary shoring and permanent retaining wall to acceptable levels, four main measures were added to supplement the original design:

- Temporary 750mm (30 inch) caissons with HP360x152 steel reinforcement were installed from Arches C6 to C14 before excavation was allowed to continue.
- Four additional 1200mm structural caissons were added to the highest arches in the middle of the wall.
- Ring footings, originally designed with a combination of R38 and T76 micropiles to resist lateral movements, were upgraded to use T76 micropiles throughout.
- In an effort to maintain the hillside stability during the temporary shoring installation, structural caissons were installed from a grade 1.5m higher than originally designed.

In short, the strategy was to create supplementary shear-resistance across the developing slip-surface, which would not be fully loaded until excavation of the slope below the wall was completed and the existing bin walls were removed.

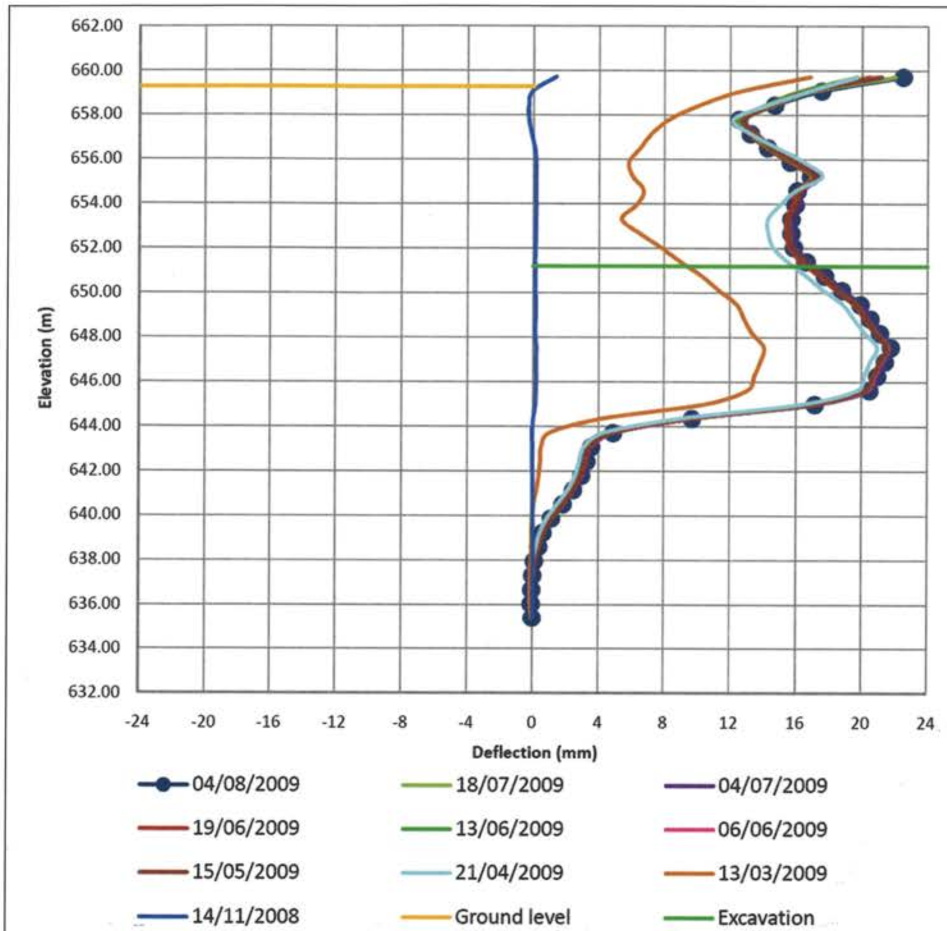

Figure 10: Arch C9 Inclinometer Plot.

Construction Completion

HCM had to respond to the changed hillside stability conditions proactively. Despite the partnering approach implemented by the City of Edmonton, the HCM/Isherwood Team was pressured to find solutions quickly. There were numerous project delays; for example, the construction of the temporary wall was delayed due to extended approvals, and the hillside movement in December 2008 stopped the shotcrete work. These delays, coupled with hillside stability concerns, resulted in the caissons being drilled from a higher elevation and more than half of the shotcrete work being done concurrently during the winter months.

HCM innovated a new micropile con-

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nection at the onset of winter conditions to enable soil cutting and shotcrete application prior to drilling micropiles. Figure 8 shows a 600kN connection that was developed and tested to 900kN on the shotcrete face for the tension piles that were part of the temporary system.

Caisson construction occurred concurrently with rebar detailing, necessitating drawings to be released on a daily basis. Designers in Toronto were working overtime to produce rebar details to maintain production. Some of the revised reinforcement for the caissons in the highest arches were 60-35M (1.5 inch) bars with horizontal tie spacing reduced to as low as 75mm (3 inch) in the areas of greatest loading.

Caissons were installed through the old failure plane and into the dense sands by pre-drilling the very stiff upper clay and then driving 1200mm (48 inch) liners with an APE 200* on an 80-ton crane. The dense sands required top head drive rigs and careful drilling below the liner to achieve penetration.

HCM added further innovation on the project by stepping the base of the shotcrete wall in to act as a back-side form for the ring footing and to be used as a swale to carry surface water through the granular backfill and out from behind the permanent retaining wall via weeping tile. HCM employed two Casagrande M9* hydraulic tieback drills to install the high capacity T76 self-drilling permanent micropiles prior to footing installation. To minimize schedule impacts, multiple operations were run concurrently, which resulted in high hillside congestion, as shown in Figure 8.

By working overtime and on weekends, HCM was able to restore most of the lost time that resulted from the late start and the hillside movement delay. The completion of the project at the end of April 2009 was one month late from what HCM originally forecasted. The formed wall was judged sufficient for the final appearance, which allowed the elimination of the concrete cap from the design. This allowed for the complete restoration of the original project schedule.

Shoring Performance

At the time of completion, the maximum movement detected along the shotcrete wall was 27 mm (1.1 inches). Figure 9 shows the deep-seated movement near elevation 645m at Arch C9. The upper tier of the double bin wall was removed first, the monitoring observed and then, with all ring footings in place, the complete low slope was removed. Performance has proven to be stable and no creep has been observed, demonstrating a successful conclusion to a logistically difficult and technically challenging project.

Concluding Remarks

The final project product is visible to many commuters and the City of Edmonton received a long-term, maintenance-free product. Figure 10 shows the unique geometry and aesthetically pleasing design of the finished structure. The calculation of the reduced phi angle and soil strength became a fundamental argument in the later stages of the project. This presented disagreements between the design-build contractor and the owner with respect to responsibilities for changes in the soil conditions from what was presented in the tender documents. The matter was deliberated as work continued in order to mitigate any potential impact on the project schedule. Test pits dug upon completion of the slope regrading revealed strain-weakened thin layers of slickensided clay that were not identified at the time of tender, improving the case by the contractors for changed conditions.

The Whitemud project is a unique example of a hill stability challenge and significant knowledge was gained for future projects. It is expected that more advanced technical papers will be tabled in the future and it is likely that several graduate students will proceed with thesis work in the years following completion.

**Indicates ADSC Members.*■

PROJECT TEAM

Shoring Design-Build Contractor	HCM Contractors Inc.* Martin Halliwell, P. Eng., Project Manager Geoff Jilg, Operations Manager Jeff Gibbs, Superintendent
Shoring/Retaining Wall Designer	Isherwood Associates* Matthew Janes, P. Eng., Senior Engineer Mark Spence, P.Eng., Project Manager Peter Sheffield, P. Eng., Structural Engineer
Monitoring	Micro EnGEO Inc. Mohamed El Habiby, Project Manager HCM Contractors Inc.* Brendan Lemieux, Site Engineer
Construction Manager	ConCreate USL Ltd. Stewart Nelson, P. Eng., Project Manager Borje McKinnon, Operations Manager
Geotechnical Design	Thurber Engineering Ltd.* Robin Tweedie, P.Eng., Project Manager Linda Bobey, P.Eng., Project Engineer

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