Geotechnical Issues during Construction of Athletic Centre at UNB in Fredericton, NB

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ABSTRACT
A multi-purpose building is being constructed on the University of New Brunswick campus in Fredericton. The building is located within an existing slope with four underground stories in the back and the full six storey structure above existing grade in the front of the building. The location of the building within a slope and the rock conditions encountered during construction resulted in a number of changes to the initial design. Up to 13 m of sandstone bedrock had to be removed by blasting. Pre-shearing of the rock mass, rock anchors, and rockfall protection mesh were utilized due to concerns regarding the stability of the near vertical excavation in the rock mass. The original foundation design was based on the assumption that the lateral earth pressures on the south foundation wall will be resisted by a series of individual shear keys along the east and west foundation walls. However, the highly fractured nature of the bedrock at the foundation elevation resulted in eliminating the initially proposed individual shear key option. Instead, a system of high capacity foundation anchors along the east and west foundation walls and a continuous shear key along the south foundation wall was installed to resist sliding. The use of high capacity foundation anchors to provide resistance against building sliding and the load transfer mechanism through the structural floor slabs to transfer loads between the exterior foundation walls are unique aspects of this case study.

RÉSUMÉ
Un édifice sportif à but d'utilisations multiples est couramment en construction au campus de l'Université du Nouveau Brunswick, situé à Fredericton, N-B. L'édifice est incorporé dans une pente, avec quatre étages sous-terraines du côté sud et aucune étages sous-terraines du côté nord. Cette localisation, couplé avec la condition très fracturé du roc qui fut observé lors de la construction a nécessité plusieurs changements aux fondations. Le plus grand de ces changements fut de remplacer le système proposé pour résister les pressions latérales qui veulent faire glisser l'édifice. Le nouveau système de fondation incorpore l'utilisation d'ancrage à haute capacité dans le roc le long des côtés est et ouest de l'édifice et l'enfonçage de la semelle située le long du côté sud de l'édifice afin de fournir une résistance au cisaillement adéquate.

1 INTRODUCTION

The Richard J. Currie Centre is a multi-purpose building being constructed at the University of New Brunswick in Fredericton. Located in the northeast corner of the campus, the building will overlook the Saint John River. The Currie Centre is one of the largest construction projects ever undertaken in Fredericton. Researchers focused on fitness and lifestyle assessment will use the \$50-million building which will also contain a Human Performance Laboratory. Two gymnasium areas will be located on different levels of the structure along with state-of-the-art resources for teaching. The facility is being constructed in a two year period, with a completion date in 2010.

The building is located within an existing slope with four underground stories in the back and the full six storey structure above grade in the front of the building. The location of the building within a slope and the rock mass conditions encountered during construction resulted in the need to stabilize a 13 m deep near vertical cut in the rock mass and to stabilize the building foundations against sliding forces due to unbalanced earth pressures.

The original foundation design utilized a system of individual shear keys to resist the lateral forces on the structure. However, the poor bedrock conditions encountered at foundation grade resulted in changing the design to a single deep continuous shear key along with high capacity rock anchors, an approach not commonly used for buildings. Another unique aspect of this project relates to how the lateral loads are transferred through the structural floor slabs (acting as diaphragms) to the side foundation walls, where the high capacity anchors were installed.

2 BUILDING LAYOUT AND SITE DETAILS

The Currie Centre will be a six storey structure approximately 28.1 m in total height. The building foundations are being constructed within an existing natural slope that was originally covered in trees and grass. The overall slope is approximately 18 m in height and sloped at approximately 3H:1V towards the northeast (Figure 1). The foundation for the south wall of the building is being constructed approximately 17 m below final grade. In contrast, the ground floor on the north side of the building will be at final grade (Figure 2). The foundation consists of a system of strip and spread footings bearing on bedrock (Figure 3). The overall size of the building area is approximately 52 m x 62 m. An existing building located at the top of the slope (near the southeast corner of the building) will be approximately 25 m away from the new structure. In addition, a major street runs along the southwest corner of the new building approximately 15 m away.

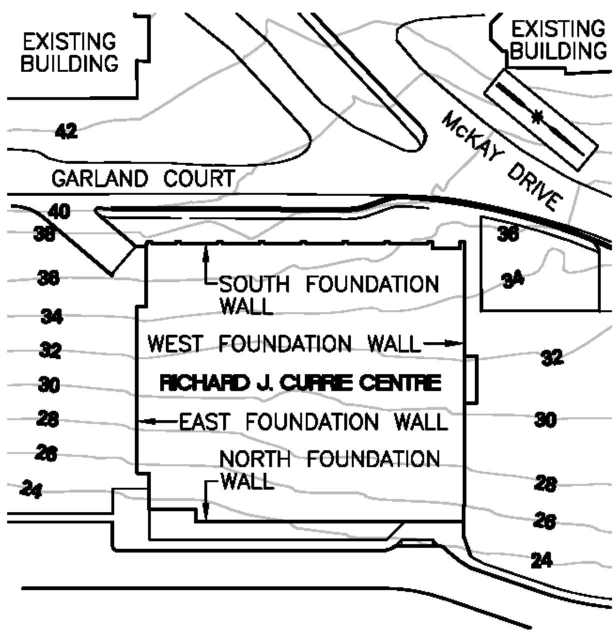

Figure 1: Plan view location of Currie Centre within existing slope.

3 BEDROCK CONDITIONS

In general, the subsurface conditions at the site consisted of a layer of silty sand with gravel (SM) overlying sandstone bedrock. The silty sand with gravel (SM) is locally referred to as a glacial till and ranged in thickness between 2 m and 6 m. The upper zone of very poor quality highly weathered bedrock ranged in thickness from 1 m to 3 m and was easily removed using an excavator. Immediately below the weathered zone, the rock mass varied in quality from excellent to fair.

A continuous clay seam was encountered at depth within the rock mass over the entire building area. The clay seam was found near the elevation of the building foundations along the western portion of the site and several metres above the foundation elevation along the eastern portion of the site. The clay seam ranged from approximately 300 mm to 500 mm in thickness (Photograph 1). The clay seam had a consistency described as firm and with water discharging from the seam.

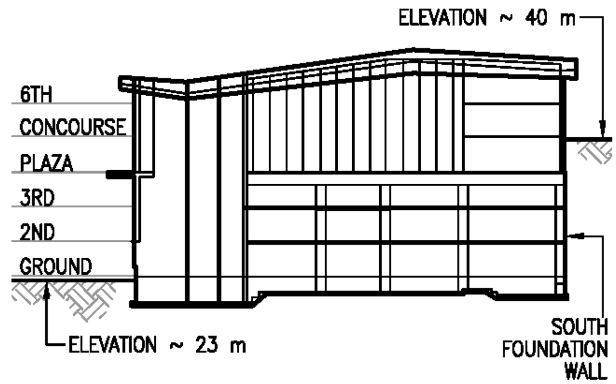

Figure 2: North – south cross-section through building illustrating unbalanced earth pressures.

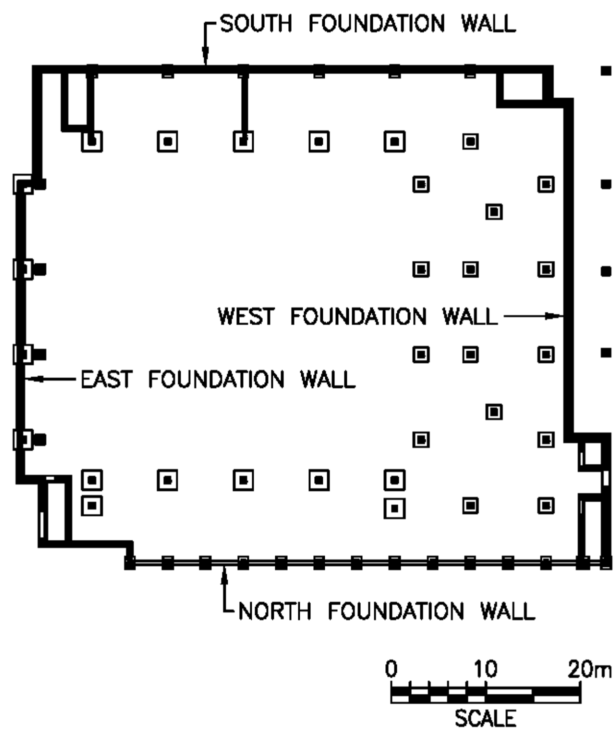

Figure 3: Foundation layout plan.

Over most of the site, the rock mass quality below the clay seam was found to be of a fair to poor quality. Within the southeastern portion of the site, the overall rock mass went from good or very good quality above the clay seam to fair quality below the clay seam (Photograph 2). However, within the western portion of the building area, the overall rock mass below the clay seam was described as of poor quality.

After blasting the bedrock to a depth corresponding to the proposed foundation depths, the bedrock within the western portion of the site at the proposed foundation elevation was shattered and very severely fractured. Therefore, additional material was over-excavated (more than 1 m below the proposed elevation of the foundations) until marginally better quality bedrock was encountered. In addition, because the clay seam along the western side of the building was located at approximately the proposed foundation level, additional material was removed to reach better quality bedrock at a greater depth.

Photograph 1: Continuous brown clay seam approximately 400 mm thick.

Photograph 2: Fair quality rock mass below the brown clay seam in southeast corner.

The bedrock at foundation level is described as a severely fractured to fractured grey sandstone. The sandstone is a moderately strong rock with close to moderately spaced discontinuities. The orientation of the discontinuities are moderately inclined to the horizontal and partly open with rough surfaces. The rock mass rating (RMR) for the bedrock at foundation level ranged between 30 and 35. The unconfined compressive strength was approximately 40 MPa for intact specimens. Mechanical rock breakers were used to prepare the final surface for the foundation components. If used aggressively, the mechanical rock breakers would over-break the bedrock up to 1 m below the elevation being prepared. Furthermore, a large excavator could rip the bedrock several metres below the required foundation grade with relative ease. Therefore, care was taken with the final stages of preparation for the bedrock surface before constructing the foundation components.

Based on the bedrock conditions encountered during the original geotechnical investigation, the initial Factored Ultimate Geotechnical Resistance (ULS) was 3000 kPa (using a geotechnical resistance factor of 0.5). Also based on the original information, the initial recommended Serviceability Resistance bearing pressure (SLS) was 1200 kPa, based on total settlements not exceeding 20 mm for the footing sizes proposed. However, the poor quality bedrock conditions exposed during construction required that the original ULS and SLS recommendations be reduced. Therefore, the final recommended Factored Ultimate Geotechnical Resistance (ULS) for the building foundation system on the fractured rock mass was 1500 kPa (using a geotechnical resistance factor of 0.5). Furthermore, the final recommended Serviceability Resistance bearing pressure (SLS) was 900 kPa, based on total settlements not exceeding 10 mm for the final footing sizes.

4 STABILIZATION OF ROCK MASS ON EAST END

Excavation in the southeast corner of the building area required a 13 m deep cut in the bedrock. Overall, the rock mass along the east end of the excavation for the south foundation wall was relatively competent and of good quality. However, the major discontinuities dip approximately 30 to 40 degrees towards the northeast creating the potential for a planar or block failure to occur (Photograph 2). Therefore, this area required stabilization with rock anchors and protective wire mesh to provide safe access for construction of the building foundations.

The rock slope stability was evaluated as a simple two-dimensional wedge failure. The potential failure plane was nearly parallel to the slope face. The failure plane day-lighted into the excavation and the dip of the failure plane was approximately equal to the angle of friction along discontinuities for sandstone. Furthermore, there were defined near vertical discontinuities that would act as release surfaces for a wedge failure. It was assumed that a tension crack full of water and full hydrostatic pressure along the failure plane could exist. Continuous seepage from the excavation face, in particular after significant rainfall events, supported this assumption. A well defined seam of clay approximately 300 to 500 mm in thickness, slowly discharging water at select locations, was also encountered lower on the rock face. In addition, the overburden soils at the top of the rock cut acted as a surcharge. Based on all these assumptions and assuming a fully defined failure plane, the calculated factor of safety was approximately 1.0. Therefore, rock anchors were evaluated to increase the factor of safety to 1.5.

To stabilize the overall rock mass, the rock anchors were spaced approximately 1 m apart. The anchors were installed to a depth of approximately 12 m into the rock mass inclined approximately 30 degrees down from the horizontal. The required geotechnical capacity of the rock anchors was 280 kN. Four rows of rock anchors were required to intercept all major discontinuities observed. Figure 4 presents the final design for rock anchors and protective mesh to stabilize the rock mass in the east end of the excavation. Based on discussions with the rock anchor supplier, 25.4 mm diameter uncoated threaded bar solid anchors were chosen because they were in-stock and could be delivered to site quickly. Uncoated bars with no corrosion protection were selected as the anchors would only be temporary. The bond length was specified at 5.5 m with an overall embedment length of 12 m for each anchor. To stabilize the rock mass against a potential wedge failure, the anchors were pre-tensioned.

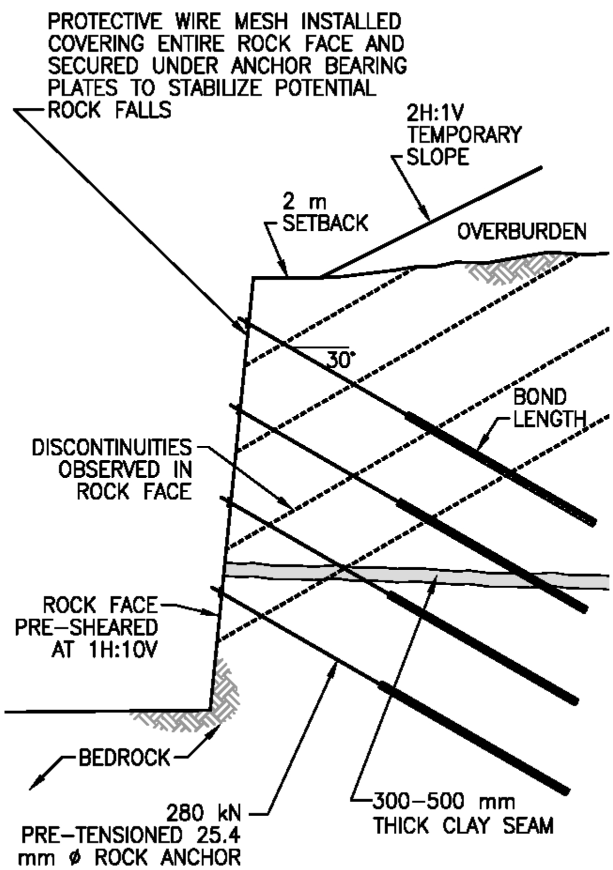

Figure 4: Location of discontinuities in rock mass and layout of rock anchor stabilization plan.

To provide temporary support, coarse rock fill was placed against the exposed rock face. The fill material was also required as a working platform to drill the holes and install the anchors. Approximately 110 rock anchors were installed to stabilize the rock mass along the east end of the excavation for the south foundation wall (Photograph 3).

Stressing and testing was required for every anchor, to demonstrate that the anchor met the acceptance criteria and to stress and lock-off the anchor at its specified load. Stressing did not begin before the grout had reached a compressive strength of 30 MPa. Samples of the grout were obtained from each batch and tested. Typically, the required strength was realized in 5 to 7 days using high-early cement. The specimens were moist cured at 4 ^{o}C to simulate field curing conditions.

The first three anchors were performance tested and thereafter approximately 10% of the remaining anchors were tested (Photograph 4). All other anchors were proof-tested. The performance tests evaluated load carrying capacity, apparent free-stressing length, magnitude of residual movement, and the rate of creep. The performance test was conducted by cyclically and incrementally loading the anchor. The anchors were tested up to 1.33 times the design load. Figure 5 shows a typical plot of the performance test data. The proof test was used to quickly determine the load carrying capacity, apparent free-stressing length, and the rate of creep. Each anchor was also subjected to a lift-off test before being locked-off at 1.1 times the design load, to account for seating losses.

Photograph 3: Rock anchors installed to stabilize rock anchors on east end of the excavation.

Rock anchors were approved if the creep, movements, and lock-off loads all meet the acceptance criteria. The creep amount was not to exceed 1 mm at Test Load during the period of 1 to 10 minutes. If this value was exceeded, then the total creep movement within the period of 6 to 60 minutes was not to exceed 2 mm. The minimum apparent free tendon length at Test Load, as calculated on the basis of elastic movement, was not to be less than 80% of the designed free tendon length plus the jack length. The maximum apparent free tendon length at the Test Load, as calculated on the basis of elastic movement, was to be less than 100% of the free length plus 50% bond length plus the jack length. Less than 3% of rock anchors failed the proof testing and had to be de-rated. Anchors were de-rated to 50% of the Test Load that resulted in no measureable creep (PTI 2004). On average, the affected anchors were de-rated to approximately 50% of the design load. Therefore, no anchors had to be replaced as the impact of lower capacity anchors on the overall factor of safety calculation was negligible.

Photograph 4: Rock anchor stressing and testing in southeast area of exposed rock face.

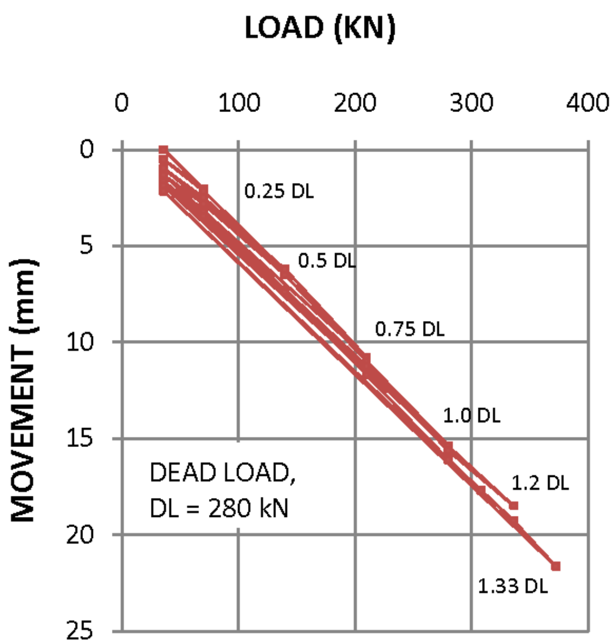

Figure 5: Typical plot of performance test results on rock anchors to stabilize excavated rock face.

5 FOUNDATION SLIDING PROBLEM

The south foundation wall is located within an existing slope with approximately 17 m of the structure buried below final grade. In contrast, the north face of the building will be entirely above exterior grade. Therefore, the earth pressures on the south foundation wall need to be resisted by the building foundations in order to prevent sliding of the entire structure.

Photograph 5: Continuous shear key along south foundation wall created by pre-shearing and mechanical removal of the rock.

The bedrock excavated for the building foundation during construction and subsequently processed through a crusher on site was proposed as backfill behind the south foundation wall of the building. Based on the grading limits and the coarse nature of the material, the unit weight and internal angle of friction was estimated to be 21.5 kN/m^3 and 40 degrees, respectively. Therefore, the coefficient of earth pressure at rest (K_o) was estimated as 0.36 for the proposed backfill material. The structural engineers had proposed to utilize a shear key along the base of the south foundation wall. The heavily reinforced wall section for the first floor allowed for the transfer of the lateral thrust from the lower half of the first floor to the continuous shear key along the south foundation wall. Pre-shearing and careful removal of the rock by mechanical breakers produced a well-defined shear key in the fractured bedrock (Photograph 5). Based on the condition of the bedrock, an allowable horizontal pressure on the rock mass from a shear key of 400 kPa was recommended. The allowable horizontal pressure was based on an analysis of a 1.2 m deep passive wedge with a factor of safety of 2 against movement. The remaining lateral forces on the south foundation wall were assumed to be transferred through the structural diaphragms for each floor of the building to the east and west foundation walls. Therefore, the total lateral sliding force remaining to be resisted was 860 kN/m. Since the south foundation wall is approximately 54.5 m wide, the total lateral thrust being transferred to the east and west foundation walls is 46 950 kN. Hydrostatic pressures on the south foundation wall were not considered in calculating the total lateral thrust as an elaborate perimeter drainage system is being installed. Any buildup of hydrostatic pressures behind the south foundation wall will be continuously monitored with a series of piezometers as part of the operation of the building. In addition, de-watering points are installed as a contingency in the event hydrostatic pressures begin to develop. The dead load of the building structure along the east and west foundation walls was insufficient to resist the total lateral sliding force of 46 950 kN.

The use of shear keys incorporated into the bedrock under the east and west wall foundations was the original design concept to resist the sliding force of 46 950 kN. The number of shear keys along each wall had to be increased significantly compared to the original design concept after the lateral loads on the south foundation wall were revised and the allowable horizontal pressure on the poor quality rock mass was re-evaluated. Therefore, the number of shear keys required along both the east and west foundation walls resulted in a very close spacing interval that was believed to be too difficult to construct. Figure 6 illustrates the layout and size of the individual shear keys proposed in the original design along the east and west foundation walls. Even by using pre-shearing, over-break of the rock mass during construction of the shear keys would have compromised the rock in front of the adjacent shear key thereby rendering it ineffective. This was supported by observations made during the previous rock removal activities in these areas. Therefore, alternative options to resist the sliding forces were examined.

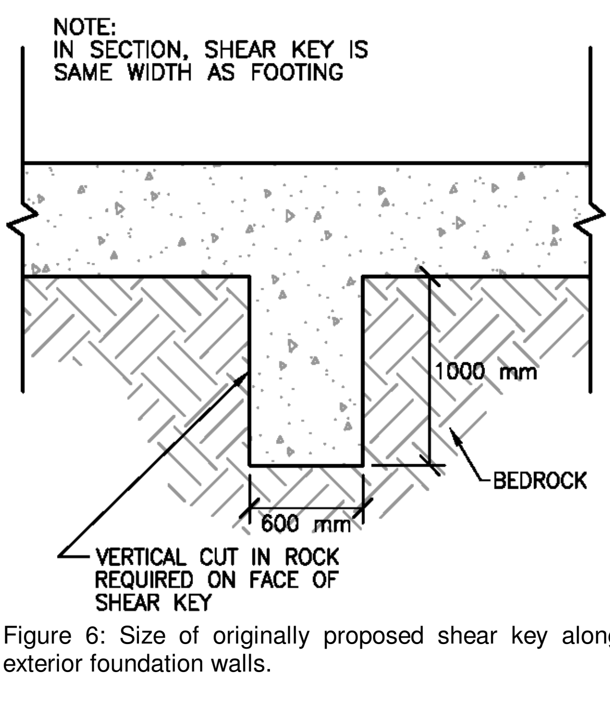

Figure 6: Size of originally proposed shear key along exterior foundation walls.

To increase the sliding resistance, pre-stressed rock anchors are routinely used in gravity dam structures. This approach is not common for buildings but had to be adopted for this structure. Both vertical and inclined foundation anchors installed through the footings for both the east and west wall foundations were then considered as options to address the sliding design condition. If vertical anchors were installed, the dead load of the structure could be included when calculating the frictional resistance. However, only the horizontal component of an inclined anchor option could be used when calculating the resistance force. To include both the horizontal component and the vertical force as frictional resistance of an inclined anchor (along with the dead load of the structure) would be considered inappropriate as the two mechanisms engage different modes of failure or movement and are therefore incompatible. Additional problems with the inclined foundation anchor option were also discovered from a constructability perspective. Ideally, the anchors would be installed along the centerline of the footings to simplify the structural steel reinforcing detail required for the additional vertical loads. However, the inclined anchors could not be tensioned immediately as they would likely cause the footings to displace laterally. Therefore, the anchors would have to be installed offset from the foundation walls and tensioned incrementally as backfill is being placed behind the south foundation wall so that the loads remain balanced to prevent movement of the footings in either direction. The additional structural issues presented by offsetting the foundation anchors and the issues with staging the tensioning and backfilling behind the south foundation wall were deemed unacceptable and therefore, the inclined anchor option was discarded.

The use of vertical foundation anchors installed in both the east and west wall foundations allow for the dead load of the structure to be included when calculating the frictional resistance against sliding. A frictional coefficient of 0.6 was used for concrete cast on bedrock. The dead load of the structure along the east foundation wall was 265 kN/m. The west foundation wall was more lightly loaded with only a dead load of 133 kN/m. The structural engineers evaluated options to engage the interior footings and elevator / stairwell foundations as well using grade beams connected to the perimeter foundation walls. By structurally connecting the interior foundation components, large dead loads from the overall structure could be included in the calculation and thereby reduce the vertical loads required from foundation anchors. The size of the grade beams required and the associated reinforcing steel requirements resulted in this option being deemed cost prohibitive. Therefore, the additional vertical load required to produce sufficient frictional resistance had to be obtained solely from vertical anchors.

Using the dead load of the structure, a frictional coefficient of 0.6, and a factor of safety of 1.5 against sliding (unfactored resistance > 1.5 x service loads), the additional vertical load for each footing was calculated. A factor of safety of 1.5 against sliding using unfactored loads was used as recommended by the Canadian Foundation Engineering Manual (CFEM 2006). Rock anchors of different sizes and capacities were evaluated. If the anchor capacity was too low, then too many anchors would be required and the spacing between anchors would be too close creating installation problems and potential group effects. In contrast, if the anchor capacity was too high, fewer anchors would be required but then the anchors would become cost prohibitive and the structural reinforcement required to strengthen the footing against the large concentrated vertical loads would also become problematic. Based on discussions with the anchor supplier and the structural engineer, 65 mm grade 1030 MPa rock anchors (threaded bar) were chosen. As these would be permanent anchors, double corrosion protection was specified. Based on the rock mass parameters outlined previously, the working bond strength was assumed to be 450 kPa (Wyllie 1992). The foundation anchor design details are summarized in Table 1. The bond and free stressing lengths maintained the 13.7 m fabrication length of the anchors and eliminated the use of splices.

Table 1. Foundation Anchor Design Details.

Design Parameter	Value
Geotechnical Design Capacity (Design Load, DL)	1800 kN
Anchor Ultimate Tensile Strength, $P_{ultimate}$	3471 kN
Anchor Yield Strength, P_{yield}	2777 kN
Anchor Hole Diameter	150 mm
Anchor Bond Length	8.5 m
Anchor Free Stressing Length	4.6 m
Anchors Required along East Foundation Wall	32
Anchors Required along West Foundation Wall	35

Based on the additional vertical loads from the anchors, the size and strength of the footings was increased to satisfy the recommended SLS and ULS loads and the reinforcing steel requirements augmented to account for the additional stress being transferred through the footings (Figure 7). Therefore, the spacing between the anchors required was 1.7 m along the east wall foundation and 1.6 m along the west wall foundation. Numerical modelling was used to verify that there was no interaction from the anchors under the footings where the frictional resistance was mobilized. However, group effects between the anchors were still considered because of the anchor spacing. Since each anchor would be tested, it was decided that any reduction in anchor capacity would be evaluated after the testing if required. Based on the results of the testing, group effects did not reduce the capacity of the anchors.

The footings for the east and west wall foundations were poured prior to drilling and installing the rock anchors. Photograph 6 shows the plastic sleeves cast within the footings at the anchor locations so that the concrete did not have to be cored (possibly damaging the reinforcing steel). However, accessibility problems with the vertical reinforcing steel protruding from the footings created problems for the rock anchor installation. After evaluating a number of options, the structural engineer agreed to have the reinforcing steel cut off flush to the footing at each anchor location (Photograph 7). After the anchors were installed and tensioned, the reinforcing steel was then replaced by drilling into the footings and setting new bars with epoxy.

The rock anchor stressing and testing procedures provided by the Post-Tensioning Institute (PTI 2004), and discussed previously, were followed. A large centre-pull jack had to be used to tension the high capacity foundation anchors. Due to the concentrated loads, stressing of the anchors did not begin until the compressive strength of the concrete for the footings had reached 45 MPa. All foundation anchors passed the performance and proof testing and no anchors had to be de-rated or replaced. However, approximately 20% of the anchors had to be reset and re-tested after the Lift-off test prior to final approval.

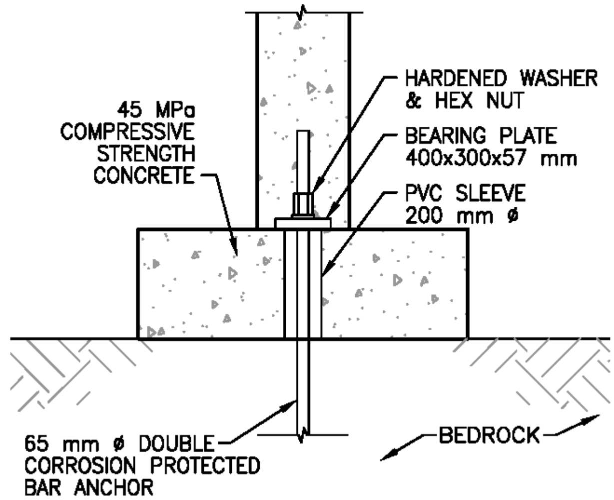

Figure 7: Cross-section through footing where foundation rock anchors were installed.

6 SUMMARY AND CONCLUSIONS

The location of the building within an 18 m high slope and the rock mass conditions encountered during construction of the Currie Centre resulted in a number of geotechnical issues. The highly variable rock mass conditions and the extremely tight construction schedule forced the design team to continuously re-evaluate and change many of the foundation components.

Excavation in the southeast corner of the building area required a 13 m near vertical deep cut in the bedrock. Major discontinuities within the rock mass created the potential for wedge failures. The potential wedge failures, coupled with the continuous clay seam at depth within the excavation face required an extensive rock anchoring program to stabilize the overall rock mass. Approximately 110 temporary pre-stressed anchors were installed 12 m into the rock mass to intercept the major discontinuities and increase the factor of safety against failure. Protective wire mesh was also installed over the entire exposed rock mass to prevent rock falls.

The unbalanced earth pressures on the south foundation wall need to be resisted by the building foundations in order to prevent sliding of the entire structure. The original foundation design concept for the building utilized a system of individual shear keys along the east and west foundation walls. However, the poor bedrock conditions encountered at foundation grade and revisions to the lateral loads on the south foundation wall resulted in changing the design to a single deep continuous shear key with high capacity rock anchors. To provide sliding resistance from earth pressures acting on the south foundation wall, vertical rock anchors were installed along the east and west foundations and a continuous shear key was installed under the south foundation wall. The vertical rock anchors increased the dead load on the building foundations to provide sufficient frictional resistance against sliding. Due to the fractured nature of the bedrock, pre-shearing and the careful use of mechanical breakers was required to create a well-defined shear key.

Photograph 6: Plastic sleeves installed within the footings where the foundation anchors will be installed.

A unique aspect of this project is that the lateral loads from the south foundation wall are transferred through the structural floor slabs within the building (acting as diaphragms) to the east and west walls foundation, where the high capacity rock anchors were installed. Furthermore, the use of rock anchors is not commonly used to provide sliding resistance in buildings.

The original foundation design for this structure was based on the premise that this site had simple bedrock conditions. However, the poor quality rock mass conditions encountered during construction required a major change in foundation design to be developed. Therefore, this case study demonstrates that the geotechnical investigation program must be designed to fully evaluate the rock mass conditions.

Photograph 7: High capacity foundation anchors installed in the footings between the reinforcing steel.

7 REFERENCES

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