Geotechnical Bedrock Characterization, Centre Block Rehabilitation Project, Ottawa Ontario

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GeoCalgary 2022 October 2-5
Reflection on Resources

ABSTRACT
The Centre Block Rehabilitation (CBR) Project is currently the largest heritage building rehabilitation project in Canada. Over the next decade, major upgrades to Centre Block and the Peace Tower will be undertaken. Extensive bedrock excavations up to 24 m deep are required to construct a new 3-storey subterranean Parliament Welcome Centre (PWC), and new office spaces, meeting rooms and service linkages extending beneath Centre Block. A geotechnical site investigation was conducted to characterize subsurface conditions beneath Parliament Hill and inform geotechnical designs. This paper discusses select field procedures and laboratory tests to assess critical engineering properties of the bedrock formations, and their applications in numerical modeling of excavation sequencing and rock support designs. A retrospect on the investigation program is presented to inform geotechnical practice in similar settings.

RÉSUMÉ
Le projet de Réhabilitation du Bloc Centre (RBC) est présentement le plus large projet de réhabilitation d'un bâtiment patrimonial au Canada. Pour la prochaine décennie, des améliorations majeures seront réalisées sur le Bloc Centre et la tour de la Paix. Des excavations majeures dans le socle rocheux, jusqu'à 24 m de profond, sont requises pour la construction d'un nouveau Centre des Visiteurs de trois étages souterrains, en plus d'espaces pour de nouveaux bureaux, des salles de réunion et d'entretien, s'étendant sous le Bloc Centre. Une étude géotechnique a été réalisée pour caractériser les conditions souterraines de la colline parlementaire, et pour permettre d'avancer la conception géotechnique. Ce papier discute de certaines interventions sur le terrain et d'essais de laboratoire permettant d'évaluer des propriétés critiques de la formation rocheuse, ainsi que leur application dans la modélisation numérique de l'excavation, du séquencement et du support rocheux. Une rétrospection sur le programme d'investigation est présenté dans le but d'informer la pratique géotechnique dans des conditions similaires.

1 INTRODUCTION

Geotechnical engineers endeavor to understand critical material properties and ground behavior for the design of complex construction projects, and to properly scope geotechnical site investigations and testing informing their analyses. Owing to the unique and varied characteristics of construction sites and projects in general, it is important to review available background and published information, develop a good understanding of the physical site setting, geology, project challenges and constraints, and carefully plan geotechnical investigations to obtain critical design parameters (Culshaw and Price 2011; Das and Sobhan 2013). This paper describes the investigation considerations and approaches taken to characterize the CBR Project for the purposes of analyzing behavior of bedrock excavations and adjacent structures, using Finite Element Modeling (FEM) and kinematic methods as the primary design tools.

1.1 CBR Project

The CBR Project, located at Parliament Hill in Ottawa, ON, is a complex, multi-year rehabilitation project being undertaken in a lean-integrated project delivery model. Centre Block will be fully renovated and upgraded to meet National Building Code requirements, and a new subterranean Parliament Welcome Centre (PWC) will be constructed within a 24 m deep excavation to the south of and below Centre Block (Public Services and Procurement Canada (PSPC) 2022). An architectural-engineering Joint Venture (CENTRUS) is under contract to PSPC to design the project, and PCL-ED is the Construction Manager engaged to complete the project over the next decade. Figure 1 shows the site location.

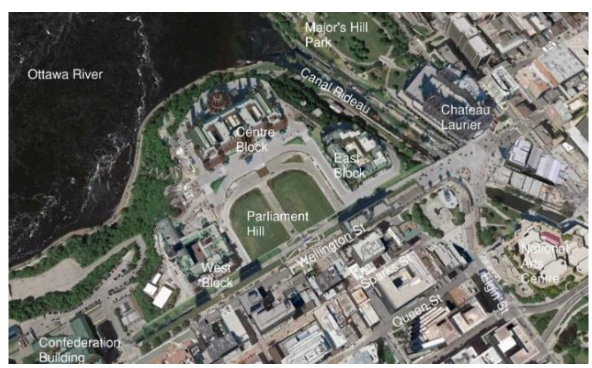

Figure 1: Site location - Downtown Ottawa

Geotechnical site investigations for the CBR Project were undertaken between 2018 and 2020 when the project was going through its Schematic Design stage. Based on the available background and design information from neighboring Parliament Hill projects (Public Services and Procurement Canada (PSPC) 2022; Senate of Canada 2021), geotechnical engineers considered that mass excavations for the project would be approached using mainly mechanical rock breaking, drilling, and blasting techniques, with rock support installed progressively to address the jointed and faulted nature of the bedrock subsurface. It was known from project inception that advanced analytical methods would be necessary to evaluate the overall stability of bulk excavations and to design rock support requirements. This was mainly driven by low tolerances to ground movement imposed by the adjacent heritage buildings; for example, structural modeling indicates that excessive building distress typically occurs at approximately 5 mm horizontal displacement and 3 mm differential settlement (CENTRUS 2020a, 2020b). In some places the excavations will be offset less than 1 m from existing foundations loaded in excess of 1MPa, and in other areas existing foundations will be underpinned or replaced. For such purposes it was necessary to evaluate a relative wide range of bedrock parameters, including intact rock and rock mass properties, and particularly the influence of discontinuities and in-situ field stresses on rock deformation.

1.2 SITE LOCATION AND GEOLOGY

Parliament Hill is situated in downtown Ottawa on an approximate 45 m high bedrock promontory along the south side of the Ottawa River. The natural escarpment along the northern boundary of the site is steep (approaching near vertical in places) and comprises vegetated overburden slopes and locally exposed bedrock. The bedrock is middle Ordovician age limestone with interbedded shale. The Lindsay Formation forms the upper unit at site and is approximately 10 m thick. The Lindsay is underlain by the Verulam Formation which generally contains higher shale content and lower weathering resistance. Geotechnical rock mechanics properties of the two formations are generally similar and the geological contact between the formations is gradational. Both rock formations exhibit sub-horizontal bedding and two prominent steeply dipping joint sets are present with variable spacing. Figure 2 below shows joint conditions and rock mass quality in the excavation face south of Centre Block, whereas Figure 3 presents a recovered core sample showing all the bedding joints (CENTRUS 2021; Lawrence 2001).

Groundwater (mostly from infiltration of precipitation and surface water) is locally perched within bedrock fractured zones above the elevation of the Ottawa River (CENTRUS 2020a).

2 KEY GEOTECHNICAL INPUT FOR MODELING

Based on the design and analytical objectives of the project and to determine the investigation methodology, a list of the critical geotechnical parameters and bedrock characteristics was developed.

Rocscience FEM software (RS2 and RS3) were mainly utilized for geotechnical design and require input for rock mass and joint slip/failure criteria. The Generalized Hoek and Brown criterion was chosen as most appropriate for rock mass strength, while Mohr-Coulomb slip criterion was considered for assessing joint properties. The following inputs are necessary in FEM (Rocscience 2021):

Figure 2: PWC partial north excavation face (Peace Tower foundation middle left).

- Intact rock properties: Uniaxial Compressive Strength (UCS), Unit Weight, Young's Modulus (E), and Poisson's Ratio (v);
- Joint mechanical properties, specifically both Peak and Residual values for Cohesion, Friction Angle and Tensile Strength, and shear and normal stiffness; and
- Rock Mass Classification in the form of a Geological Strength Index (GSI).

Other input parameters are also needed such as:

- Joint orientations (dip, dip direction) and the number of identified sets and typical spacings, which would allow for an explicit jointed model in RS2, whereas in RS3 the values could be used as anisotropies within the rock mass;
- Groundwater levels (static and transient) and projected groundwater drainage in the final stage; and
- Field stress parameters, including orientation of locked-in horizontal stresses at surface, and stress ratios with depth.

In consideration of Barton–Choubey and similar kinematic analyses, the joint surface roughness coefficient (JRC) is also a critical input parameter that needs to be considered. JRC may be estimated from inspections of rock core samples and available rock exposures, or it can be determined from laboratory tilt tests, where tilt angle is determined at the point of sliding along the joint, and direct shear tests. The significance of shear testing methods that consider effects of asperity damage (i.e., single stage vs. multi-stage direct shear testing) is discussed later in this paper.

Finally, even though they are not geotechnical or geomechanical parameters, and are not discussed further in this paper, the following inputs are also critical for modeling:

- Seismic loading Obtained from Goverment of Canada (2021);

Figure 3: Bedded limestone and shale rock core recovered from one the boreholes.

- Building loads and geometry as obtained from architectural and structural reviews;
- Excavation geometry as established by project designers in collaboration; and
- Rock support parameters as obtained from various suppliers.

3 INVESTIGATION APPROACH

Over several decades, many geotechnical investigations have been conducted in support of various projects within the Parliamentary Precinct. A thorough desktop study was a crucial first step to review existing information and to develop the initial understanding of site conditions and bedrock geology (CENTRUS 2021). With historical investigations, hundreds of boreholes covering Parliament Hill were documented dating back to the 1960s.

New investigations were necessary to address information gaps pertaining to project requirements, rock properties and the modern analytical approach.

3.1 Field investigations

Field investigations for the CBR Project included the execution of over thirty (30) test pits and sixty-nine (69) boreholes (depths varying up to ~37m) beneath and surrounding Centre Block and surrounding the future Parliament Welcome Centre, to classify and obtain samples of the overburden soil materials and the bedrock. The work was conducted in five phases over a two-year period, with investigation targets in later phases informed by the earlier findings. Selected in-situ and laboratory testing methodologies are summarized below.

3.1.1 Dilatometer testing

Dilatometer (i.e., pressuremeter) testing was completed in two NQ size boreholes in January 2019. The testing followed the method outlined in USBR 6575-09 Determining In-Situ Modulus using a Flexible Volumetric Dilatometer.

3.1.2 In-situ field stress determination

Three (3) in-situ stress measurements were completed in November/December 2018 and January 2019. These tests were completed using USBM type deformation gauges, shown in Figure 4 below, using the over-coring stress relief technique in general accordance with (ASTM D4623-16 2017). Results were compared to historic data reported by others (e.g. EXP 2013).

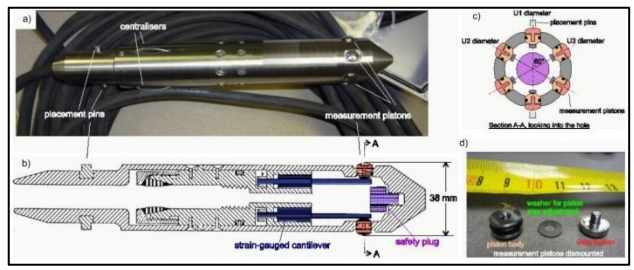

Figure 4: USBM Borehole Deformation Gauge or BDG (Gonzalez and Lanteigne 2018)

3.1.3 Hydraulic conductivity testing

In-situ hydraulic conductivity testing was completed in several boreholes using both double-packer testing and single well response tests. Packer testing consists of measuring hydraulic conductivity across specific depth intervals within the borehole by isolating the test depth using nitrogen-filled packers and pumping water into the borehole, as per the ASTM D4630 standard.

3.2 Geophysical investigations

Geophysical surveys were carried out in twenty-four (24) of the boreholes, and included determination of shear wave velocity, self-potential, resistivity, natural gamma and spectral gamma logging, and acoustic televiewer surveys. Optical televiewer surveys were also carried out in selected boreholes. K-Bentonite stratigraphic marker correlations using gamma logging was carried out to help identify discontinuities in the rock mass (El Madani et al. 2022).

Televiewer data was used to assess joint sets, along with their dip direction and dip, as well as apparent spacing.

In addition to borehole surveys, surficial surveys were conducted in some areas, with Ground Penetrating Radar (GPR) scans completed to help interpret bedrock surface contours beneath the overburden material.

3.3 Laboratory testing

An extensive program of laboratory testing program on rock core samples was conducted to interpret a range of geomechanical parameters. Tests included:

- Rock density;
- Unconfined Compressive Strength (UCS) with Elastic Modulus and Poisson's ratio, which provided intact rock strength data;
- Triaxial Compressive Strength, to evaluate intact rock friction angle and cohesion;
- Direct and Indirect tensile strength (Brazilian tests) to interpret tensile strength of intact rock;
- Direct shear peak and residual values (both single stage and multi-stage tests) to interpret cohesion and friction angle for joints;

Additional non-geomechanical tests (abrasiveness, swelling potential, whole rock chemistry, durability in aggregate) were also completed.

3.4 Joint mapping

The early works and initial stages of site preparation construction permitted detailed inspections of exposed bedrock surfaces in front of the Centre Block building and around the Peace Tower foundation. Joint mapping of bedrock surfaces exposed by demolition activity in the Centre Block basement and in exposed faces of the PWC excavation provided valuable geo-structural information, including identification and orientation of faults and prominent open joints. Also, the mapping presented opportunity to inspect the surface condition of joints and some of the infilling materials. Mapping was supported by LiDAR scanning and surveying to develop rock mass ratings (RMRs). An example of the LiDAR scanning and rock face inspection log is shown below in Figures 5 and 6, respectively.

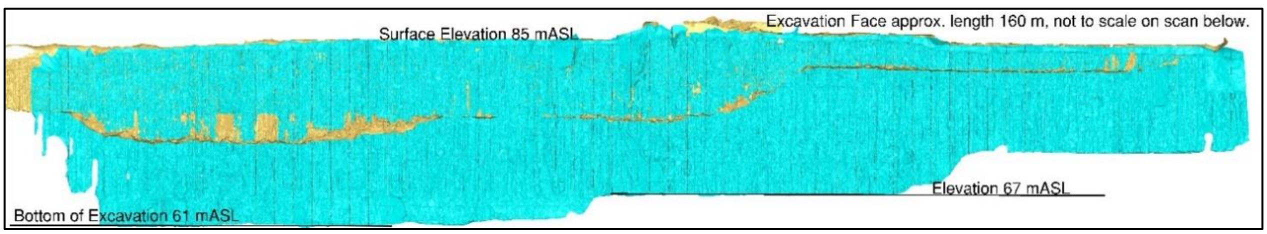

Figure 5: LiDAR scan of north excavation face PWC. Joint locations and facial distortion may be analyzed by inspection.

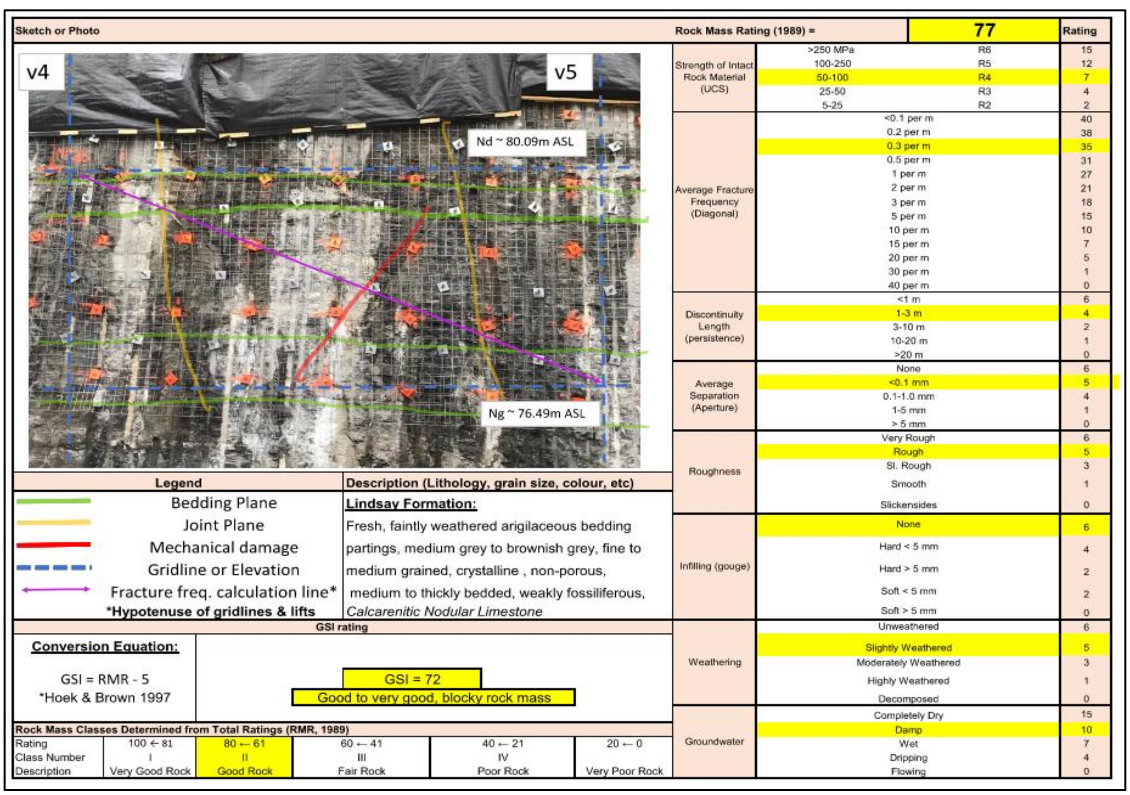

Figure 6: Example of excavation face joint mapping data (annotated photo log with RMR estimate).

4 CRITICAL RESULTS

Critical modeling parameters required for FEM were obtained from the investigation and testing. Results are summarized in the following sub-sections.

4.1 Field stress

Field stress measurements obtained from over-coring tests are summarized in Table 1. Of the three (3) in-situ stress measurements completed, one of the tests appeared to overestimate stresses by approximately 40%; therefore, it was discarded (CENTRUS 2021).

From the above readings, a linear interpolation shown in Figure 7 was used to represent locked-in major and minor principal stresses (i.e., 0.9 MPa and 0.5 MPa at surface, respectively, after rounding up).

Table 1. In situ stress measurements

Depth of reading (m)	Major Principal Stress P (MPa)	Minor Principal Stress Q (MPa)	Azimuth (Major Principal Stress) (degrees)
8.2	1.30	0.68	81.4
15.4	1.74	0.85	82.9

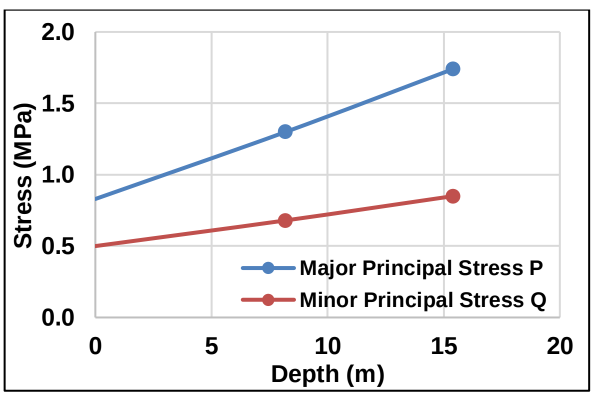

Figure 7: Locked-in Field Stress Interpretation

From the trend lines, the effective stress ratio with regards to vertical stress (weight of rock) was established as follows:

P = 2.σ'$_v$ + 0.9; and
Q = 1.σ'$_v$ + 0.5

For FEM modeling purposes, both locked-in stresses and effective stress ratios were needed as input.

4.2 Rock Mass Properties

Considering the jointed rock conditions, with little variability noted in geo-mechanical properties between the two geological formations in the subsurface profile (i.e., Lindsay and Verulam Formations), the rock mass was considered to be relatively homogeneous with the exception of the more heavily fractured upper 2 m layer at surface. For defining the rock mass properties, FEM modeling allows the Generalized Hoek-Brown failure criteria to be defined as (Rocscience 2021):

$$F_s = \sigma_1' - \sigma_3' - \sigma_{ci}\left(m_b\frac{\sigma_1'}{\sigma_{ci}} + s\right)^{\alpha} = 0 \qquad (1)$$

Where,
σ_1' *and* σ_3'= confining effective major and minor principal stresses, respectively.
σ_{ci}= UCS of intact rock material,
m_b= reduced value (for the rock mass) of the material constant m_i (for the intact rock), and
s *and* a = constants which depend upon the characteristics of the rock mass
To apply the defined failure criterion, critical characteristics of the rock as summarized in Table 2 were used.

Table 2. Input properties of rock mass characteristics*

Characteristics	Fractured Rock	Sound Rock	Source
Unit Weight (kN/m^3)	26.5	26.5	Lab testing
Intact UCS (MPa)	100	100	Lab testing
Peak GSI	60	40	Joint mapping, rock core visual description
Residual GSI	40	40	
Intact Rock Constant (m_i)	10	10	Practical Rock Engineering estimate (Hoek 2000)
Intact Rock Modulus (E_i) (GPa)	55	55	Lab testing
Poisson's Ratio	0.22	0.25	Lab testing

* Based on the results above, little difference was noticed between intact rock properties for the fractured and sound rock material.

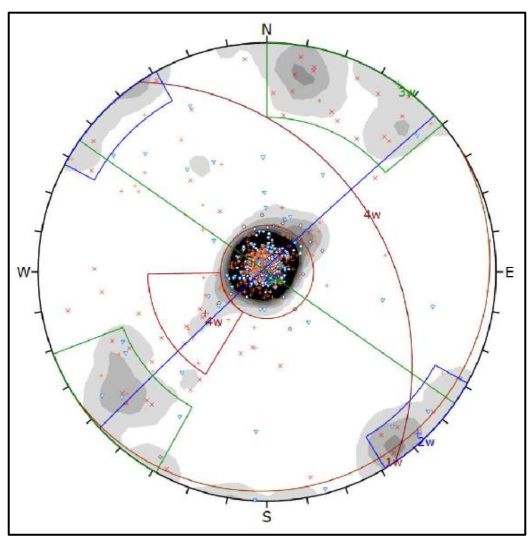

Figure 8: Stereographic plot for all encountered joints in televiewer survey.

4.3 Joint Sets

Televiewer data facilitated interpretation of three persistent joint sets across site, with an additional set only present on the southern extent of projected excavations for the PWC. The four sets were later identified and mapped in rock exposures and excavations. s started as is still ongoing.

Figure 8 below presents the stereographic plot for joint sets identified at the site.

All three sets were added to the model to create a discontinuous rock mass. The fourth set was added on the southern side of the excavation only. A significant fault was identified from coring and K-bentonite analysis, and was applied as a discrete discontinuity in the FEM. The fault notably belongs to Joint Set #3 based on orientation. Table 3 summarises identified joint sets and their inferred spacing for modeling.

Table 3. Joint set orientation and inferred spacing

Characteristics	Set # 1*	Set # 2	Set # 3	Set # 4**
Dip (deg)	3	87	87	36
Dip direction (deg)	141	318	211	056
Spacing (m)	0.4	3	3	3

* Bedding joints
** South side of PWC excavation only

4.4 Joint Geo-mechanical Properties

To evaluate the bedrock joint properties, direct shear testing along natural discontinuity planes was conducted mainly on rock core samples, providing both peak and residual values. Most of the core samples that were tested contained ubiquitous bedding joints, and most of the sub-vertical joints that were encountered were open and could not be recovered in sufficient quantities or quality for testing. Conservative estimates were therefore made for modeling. Direct shear laboratory testing allowed for the interpretation of joint cohesion and friction angle, which are critical input parameters in the Mohr-Coulomb Failure criterion. Single stage testing, which requires multiple specimens with similar joints to evaluate strength at different normal loadings, provided peak values for cohesion and friction. Multi-stage testing, completed by shearing a single sample at multiple loadings (thereby damaging the joint asperities) was only considered to assess residual joint strength properties. Table 4 below summarises the average, range and standard deviation of the direct shear testing completed for the CBR Project.

Direct Tensile Strength tests were also carried out on joints. Results are summarized in Table 5 below.

It should be noted that the tensile strength values are only pertinent to non-open bedding joints. However, due to the high variability of results, and for modeling purposes, it has been assumed that all joints have zero tensile strength.

As for cohesion and friction angle, high variability also allowed for using values on the conservative side, rather than the average, for FEM.

Finally, normal and shear stiffness parameters for the joint were estimated from rock mass properties based on Barton (1972), using the following equations:

Table 4. Direct Shear testing results – bedrock core samples

Characteristics	Peak Value	Residual Value
Cohesion Average (kPa)	564	127
Cohesion Range (kPa)	100 – 1950	0 – 1000
Cohesion Standard Deviation (kPa)	644	221
Friction Angle Average (°)	51	30
Friction Angle Range (°)	18 – 76	17 – 44
Friction Angle Standard Deviation (°)	19	8
Total of tests	14	19

$$k_n = \frac{E_i E_m}{L(E_i - E_m)} \quad (2)$$
$$k_s = \frac{G_i G_m}{L(G_i - G_m)} \quad (3)$$

Where:
k_n and k_s are the normal and shear stiffness respectively;
E_m and E_i are the rock mass and intact rock moduli;
G_m and G_i are the rock mass and intact rock shear moduli; and
L is mean joint spacing.

Input values for joint parameters are summarized in Table 5.

Table 5. Typical mechanical properties of joints used in FEM.

Characteristics	Set # 1(1)	Set # 2	Set # 3(2)	Set # 4(3)
Tensile strength (MPa)	0	0	0	0
Peak cohesion (kPa)	200	5	5	5
Peak friction angle (°)	38	38	38	38
Residual cohesion (kPa)	50	1	1	1
Residual friction angle (°)	25	25	25	25
Normal stiffness (MPa/m)	200,000	12,000	12,000	12,000
Shear stiffness (MPa/m)	80,000	5,000	5,000	5,000

(1) Bedding joints
(2) Except for fault (properties yet to be defined).
(3) South side of PWC excavation only

4.5 Groundwater

Groundwater levels are critical to stability analysis and were measured across the site in twenty-one (21) boreholes. Groundwater levels varied from ~65 masl to

~75 masl, and is believed to be locally perched within bedrock fracture zones above the Ottawa River elevation (~42 masl).

Long-term subdrainage is expected for the future deep structures, thus removing hydrostatic pressure concerns. Considering the continuous drainage state, it is believed that groundwater will have little to no effect on final global stability of the excavations.

For modeling purposes, an initial assumption has been made that the groundwater level is at ~75 masl, but continuously drained at the final stage, below ~61 masl. It has been noted that groundwater has little to no effect once removed from the models, so later models assumed the absence of groundwater around the excavations.

5 RETROSPECT ON INVESTIGATION APPROACH

Following months of FEM, review of instrumentation data, sensitivity analyses, and construction inspections, several of the rock tests and parameters were considered more critical to our modeling, with others being somewhat less important.

Considering the strength of the limestone bedrock, most of the predicted and observed rock displacement is governed by the following:

- Field Stress
- Joint Properties
- Seismic Loading

Below is a discussion on field stress and joints, as well as related investigation items. Seismic loading was a provided by Government of Canada data and was not subject to further review.

5.1 Field Stress

Field Stress was established by the over-coring method as noted previously. This testing method is very specialized and was found to be sensitive to site conditions; only two of the three tests were viable to establish a relatively simple linear interpretation of both major and minor Field Stress elements.

Since the site is at the edge of a rock escarpment, field stresses will vary depending on proximity and historic rock relaxation.

However, considering the importance of field stress to modeling, and in hindsight, more measurements across boreholes on site and providing increased depth coverage would have allowed for improved understanding. Localized higher than expected rock relaxation effects in some areas of the PWC excavation have resulted in the need for additional rock support during construction, and it is suspected that Field Stress effects are a significant contributor to the observed behavior. Therefore, the authors recommend a thorough consideration of this issue in similar important projects.

5.2 Joint Properties

Considering the discontinuous rock mass model that was created by the presence of joint boundaries, joint elements have an expected and notable effect on the displacement mechanism, whether from the effect of applied loadings or from Field Stress relief. Displacements occur predominantly along joint planes, either as slip failures or dilation, depending on relative orientation to excavation faces.

From the investigation, the televiewer data that was collected has proved invaluable in defining orientation and spacing of joints and overall kinematic behavior. The importance of inclined and oriented boreholes cannot be over-emphasized in this regard, as important sub-vertical joints can only be intercepted and assessed in this manner.

Joint strength properties were mostly defined by using laboratory direct shear testing. However direct shear peak values could only be used on non-open joints, usually presenting as joint bedding with undulating relatively smooth shale seams in the subject site formations. Some residual values were obtained from testing on open joints. Overall, the direct shear test values were relatively useful, even though they presented high variability, especially when results were obtained from single stage testing where selection of similar jointed specimens proved difficult. Additional single stage tests might have been useful in that sense, and might have defined a better trend in peak values. As previously presented by (MacDonald et al. 2021) multi-stage tests are useful and cost-efficient to interpret residual shear strength values, but obtaining peak values from these tests must be approached with due caution, owing to asperity damage that occurs due to repeated cyclic shearing.

Direct Tensile strength tests that were conducted on some non-open bedding joints gave relatively variable results. Considering the relative complexity of these tests and their higher cost, and also considering they are only useful on non-open bedding joints, they were deemed less important to the project overall. Further, design with zero tensile strength open joints seemed appropriately conservative in our case.

5.3 Rock Mass Properties

Considerable data was collected on intact rock properties, such as uniaxial compressive strength, Young's Modulus, Poisson's ratio, as well as rock quality data, which allowed interpretation of RMR and GSI values for the rock mass.

Rock mass properties were deducted from the Generalized Hoek-Brown theory using intact rock properties along with the GSI values provided in published charts.

Rock Mass Modulus obtained from dilatometer testing was comparatively less informative on this project and therefore given less weight in the modeling input. However, it might have been useful to have more testing to provide improved correlations with theoretical values.

6 CONCLUSIONS

Even though hundreds of historical geotechnical boreholes were completed from the 1960s to 2015 on Parliament Hill, information gaps were identified to address the magnitude of the project. The additional investigations were needed to improve understanding, specifically with regards to field stress, joint strength parameters and the identification of a previously unmapped fault passing beneath Centre Block.

Geotechnical investigation was a critical component of the Centre Block Rehabilitation Project design. The multi-phased investigation approach proved to be very useful as it allowed for adjustments and additions with regards to in-situ and laboratory testing approaches.

From all information collected, certain properties were found to have greater importance to FEM and kinematic analysis results compared to others. FEM models were most sensitive to joint properties and orientation, as well as in-situ field stress and rock damage from construction (particularly blasting). Additional tests for field stress might have beneficial to project design, but these tests are relatively complex, sensitive to site conditions, and expensive.

Considering that construction is ongoing, back-analyses is being done and is seen as crucial to corroborate theoretical FEM results with real-time observations obtained through instrumentation. Where necessary, models and input parameters can be refined to improve predictions of rock behavior. From early observations, it has been noted that blasting has a significant effect on rock damage and related displacements, particularly in areas with shallower support systems and minimal attenuation features (e.g., trenching). Site-specific blasting forces are being introduced into the models, and ongoing observations are being applied to calibrate and optimize FEMs for future phases of the project.

Test pit investigation, although very briefly discussed, was very useful in permitting direct inspections of conditions below existing foundations and slabs in the Centre Block Building. Coring programs into various building foundation and slab elements, as well as geophysical surveys, were completed but left for subsequent discussions.

Finally, some of the investigative tools proved to less conclusive, such as broad scale ground penetrating radar testing, which was carried out to map bedrock surfaces. This test was challenging at the site due to the presence of many historical structures and associated debris (e.g., old concrete slabs, steel, archaeological elements, etc.). At least this testing did help coordinate some of the borehole locations to avoid significant structural debris that is better investigated by test pitting.

7 ACKNOWLEDGEMENTS

The authors would like to thank Public Services and Procurement Canada (PSPC) for the opportunity to present this study. Also, special thanks all CENTRUS staff who worked on producing the data. Also, the authors would like to thank Mr. Benoît Latapie for peer-reviewing this paper.

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