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ENGINEERING MEMO

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To: Mark Elliott – Chief Nuclear Engineer, OPG

Date: 16SEP2014

Re: HWMB West Annex – Technical Viability of Raising the Foundation Elevation

Purpose

The purpose of this memo is to address the technical viability of elevating the foundation of the Heavy Water Management Building West Annex (HWMBWA) from the current design elevation of 87.0m to a higher elevation of 94.8m.

The current analysis was not to determine the strength of the structure (ie. concrete design). This would be completed during the redesign effort once a stable arrangement was determined, and a decision to proceed with the alternate design is made.

This memo addresses only design considerations with respect to building stability and does not evaluate other design impacts of this elevation change (i.e. building systems, etc).

This analysis only reviewed the feasibility of elevating the currently designed building with limited modifications within the current building footprint, without soil improvement and deep foundations. Designing the building founding on soil could have been a viable option if it had been considered in the conceptual design phase.

Background

The HWMBWA has been designed, in accordance with the Conceptual Design Report, with the basement level at EL87.0, which is the same as the existing facility. The depth of excavation is approximately 15m, and a caisson wall has been designed, and is currently being installed in order achieve the depth of excavation to construct the building in the 'current design' configuration.

To date the construction has experienced water ingress during tie-back installation (N-2014-23031) on the east wall and pull test issues. It is unknown what other issues may result as the dig continues. In addition, there are schedule concerns surrounding the installation of rock anchors and construction of the buried pipe chase linking the West Annex and the existing TRF/HWMB.

As a result of increasing construction risk and schedule concerns, RCMT was asked by OPG to look at changes to the current design with potential to mitigate cost/schedule risk including relocating the base of the foundation at or near the current depth of excavation.

This memorandum focuses on the viability of raising the building elevation as close as possible to the current depth of excavation.

Raising the elevation of the building would preclude the need to install the lower level of caisson wall tiebacks and would eliminate the need for rock anchors in the final design, thereby improving the overall construction schedule to support the Refurbishment Project.

Calculations to support this proposal will verify:

- The stability of the structure against overturning and sliding during a seismic event.
- That adequate soil bearing capacity is achieved without the use of deep foundations to bedrock.

Overall Methodology

The approach was based on evaluating the building with simple conservative assumptions and simple calculations. Initially, a simple overturning analysis using static seismic accelerations which were considered to be conservative was used.

When this simplistic approach did not provide positive results, a dynamic analysis was undertaken. Due to limited time this analysis used certain methodology and assumptions in order to complete the analysis within a reasonable timeframe to support the Engineering Decision Making process.

The following loading conditions were considered in the analysis:

- Building Dead Weight
- 20% of live load and fixed equipment (mass for seismic analysis)
- Upward water pressure
    - The upward water pressure assumed the top of the ground water to be at Elevation 97.3.
    - The base of the mat was assumed to be at Elevation 93 with the thickened perimeter down at Elevation 92.1
    - The total upward water load from the water pressure was calculated as 9,600 kips.
- Soil lateral loads on the embedded foundation due to seismic events

  As the resultant moment will be added to the calculated seismic overturning moment using square root of the sum of the squares (SRSS) method, and is much less than the overturning moments from the analyses, the load would have no impact on the evaluation and is not considered.

  The force is included in the evaluation for sliding.)

- Seismic loads per NK38-DG-03650.2B Rev 06, "Common Mode Incidents Seismic Design" (0.12g).
- Passive soil pressure is provided by lateral soil springs in analysis models. The calculated passive force on the long wall was calculated as 12,200 Kips

For the evaluation of overturning, two conditions were evaluated. The minimum acceptable factor of safety for overturning is 1.25 per clause 5.9.3 of CSA N289.3-10. The conditions evaluated are:

- Seismic overturning moment resisted by the building loads
- Seismic overturning moment resisted by the net building load after reduction for the vertical seismic forces and the upward water pressure.

For bearing pressure, an additional case was considered which added the vertical seismic to the building loads.

For each case the eccentricity caused by the overturning moment was calculated to check the stability of the structure and the bearing pressure.

Initial Evaluation – Static Analysis

Methodology

An initial evaluation was performed to evaluate the need for more advanced analysis. For the lateral seismic loads, the building floor weights were factored by a constant seismic "g" factor.

Horizontal acceleration was determined by calculating the peak response spectra acceleration (g) at 5% damping x static analysis factor x soil factor

$$= 0.35 \times 1.5 \times 1.25 = 0.656g$$

Where:

- Peak acceleration = 0.35g
- Static factor = 1.5
- Soil factor = 1.25 (based on difference in Building Code Soil factors for Class B (rock) and Class C (stiff soils) soil classes

In accordance with Clause 4.3.3.3, CSA N289.3-10, the vertical acceleration is estimated to be 2/3 of the horizontal = 2.3 x 0.656 = 0.437g.

Results

The analysis based on the above criteria resulted in overloading of the bearing capacity of the soil and an unstable structure. The calculated eccentricity was outside the width of building. This is a result of large reduction of the resisting dead weight load due to the upward vertical seismic acceleration.

Dynamic Response Spectra Analysis

In order to perform a more realistic analysis, a dynamic response spectra analysis was performed.

The purpose of the analysis was to determine the design acceleration, lateral force and overturning moment values, particularly for the east-west, or short, direction of the building which would have the greatest effect on the stability of the structure. Additionally, the overturning moments would increase the bearing forces on the soil as the building weight would be redistributed.

Methodology

1. Modelling

The analysis utilized existing STAAD models from the original design effort. In order to provide a representation of the dynamic response from the soil the following simplifications were made within the model:

- The analysis assumed the vertical and rocking soil springs could be represented by vertical springs at the corners. This simplified the calculation of the overturning moments.
- The lateral springs were applied at the side walls to a height of ½ of the embedment as recommended by ASCE 4.
- All floors were modeled as rigid diaphragms.

A second simplified stick figure model was used as a simplified approach to evaluate alternatives. Although the model had few masses and elements, the results for base shear and overturning were consistent with the results from the more detailed model.

During the development of the methodology discussed above, OPG was involved and raised a number of concerns with respect to the use of STAAD as the analysis tool. RCM agreed that STAAD would not perform the dynamic analysis that would be required to finalize this engineering product, however given the timeframe to complete this preliminary analysis, and the tools at hand RCM is confident that this methodology can provide a reasonable representative basis of the amplification through the soil.

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2. Building Stability and Soil Bearing

The probable amplification of the ground motion due to the soil layer between the foundation and the rock is not defined. As such, the building stability was evaluated at various amplification factors and damping values to determine the design margins within these variables. The evaluations were performed on the stick model, and the STAAD model at 5% and 7% damping. The acceleration was incrementally increased from 1.0, (representing the acceleration at bedrock) to 2.0 in order to determine at which point the building would become unstable, or when soil bearing would exceed the allowable capacity.

The evaluation was limited to the east-west overturning, or the narrow width of the building. The overturning moment for the detailed model was calculated from the vertical spring reactions at the corners of the building. The weight is the seismic weight used in the response spectra analysis. Shear vertical seismic forces are directly calculated by the analyses.

Values for water uplift, lateral force due to the soil seismic load and soil resistance are fixed and independently calculated.

3. Soil Structure Interaction

The available properties of the soil and rock where used to determine the probable range and most likely value of the amplification of the ground motion from the rock base to the underside of mat foundation.

a. Impedance Factor

An upper bound of the amplification can be calculated from the rock and soil properties, particularly the shear velocity and the density of the underlying rock and the soil cover. Some properties of the rock are available in the original site calculation files. Material properties of the overburden were provided in the geotechnical reports. Using these values, the necessary properties of the materials were estimated. This value calculated does not consider the damping of the soil which would slightly lower the value.

The value calculated based on the estimated material properties is 3.

b. Building Code

The National Building Code of Canada and Ontario Building Code provide ratios of based on soil types and ground acceleration to simulate the effect of soil structure interaction. For building design, the underlying soil is classified and the applicable factors are provided in the codes for each soil type.

The soil types are based on the shear wave velocity and the blow counts. The geotechnical reports classified our soil type as Class C, a stiff soil. Two factors are provided in the codes, one at a period of 0.2 seconds and a second at a period of 1 second. The 1 second period

would fall within the constant velocity range of the site response spectra and the 0.2 second factor would represent the constant acceleration range of the response spectra.

From the modal analysis, the dike natural periods were approximately 0.2 seconds and lower. Thus the factors for 0.2 would be the more representative of the amplification factor.

A review of the calculated shear velocity identified that the soil would fall in the upper end of the Type C classification. For the estimate of amplification, the average of the Class C and Class D factors was used. This value was compared to the factors for Class A for solid rock.

The calculated amplification for 0.2 second period is 1.61.

The soil factors were derived assuming 30 meters of soil above the rock. The codes would recommend that the soil properties could be averaged over the depth. This would lead to a lower factor when averaging the rock and the soil. The 1.61 is likely an upper bound on the soil factor.

c. Dynamic Load Factor

The natural period of the soil over the rock is a function of the travel time of the wave through the depth of the soil. Based on the soil depth and the shear wave velocity calculated from the soil properties provided within the geotechnical reports, the first natural frequency of the soil layer was calculated as 15.78 cycles per second.

The dynamic load factor for a structure subject to harmonic loading is a function of the forcing function frequency, the natural frequency of the structure and the damping value. The forcing function frequency can be based on the site MDE curve which provides acceleration values vs. frequency. The soil damping ratio was estimated to be approximately 5% using the formulas presented in NIST GCR 12-917, "Soil Structure Interaction for Buildings and Structures". Using the estimated first mode of the soil, amplification factors at various frequencies can be calculated which could be used to calculate a pseudo MDE response spectra.

This approach would limit the amplification for frequencies which are not close to the soil value and provide greater amplification to the resonant frequencies of the structure. The estimated frequency of the soil is based on the shear wave velocity value. To try to envelope any soil material property variability, the soil natural frequency was modified by 10%, 25%, and 50%, and the dynamic load factors for the various frequencies were calculated.

The pseudo response spectra based on the soil natural frequency were utilized to determine the expected overturning moments and forces. It is assumed this approach would give the best estimate of the amplification of the rock accelerations by the soil.

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Inputs

1. Soil Properties

Additional testing of the soils was requested of EXP in order to determine the required soil properties. Values of the bearing capacity, shear modulus (G), Young's modulus and Poisson's ratio were determined by EXP based on additional testing at the site.

The estimated soil shear wave velocity used to calculate the soil damping value was based on the shear modulus provided by EXP.

- E (Young's modulus) – 650 MPa (94.27 ksi)
- G (Shear modulus) – 250 MPa (36.259 ksi)
- Poisson's ratio – 0.3
- Ultimate bearing capacity – 700 kPa (14.6 ksf)

Assumptions

Assumptions were made as to the soil properties and seismic loadings in order to proceed. These assumptions will be validated during detailed design.

1. Soil Springs

The stiffness of the soil would be based on spring constants calculated using the provided soil properties in accordance with the NEHRP "Soil-Structure Interaction for Building Structures" (GCR12-917-21) , including adjustments for embedment and dynamic loading.

2. Response Spectra

The evaluation was based on increases to the ground motion due to the earthquake on rock for which the existing site spectrum was generated. A soil-structure interaction analysis (ie. FEA modelling) will be performed during detailed design to validate the response spectra amplification value.

The responses spectra utilized the maximum ground acceleration of 0.12g (Design Guide) modified for 5% and 7% structural damping.

The design response spectra for the soil level will need to be calculated and verified based on definitive soil perimeters by a geotechnical engineer.

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Limitations of the Analysis Performed

1. Response Spectra

As discussed in the methodology section above, an equivalent method to determine the amplification due to dynamic analysis was completed. This entailed some empirical design, engineering judgment, and engineering research. A detailed analysis will be completed as a part of the final design to validate the amplification factors that were calculated in this preliminary analysis.

2. Damping

Clause 6.6 of CSA N289.3 permits the use of 7% damping in concrete structures when designing for the DBE earthquake, or if stress levels are greater than 50% yield. The concrete foundation (slab and walls) has relatively low levels of stress, with most of the structure well below the 50% level. Although permitted by code, RCM is concerned that the use of 7% damping may not be an adequate representation of this structure. Additional research is required to validate the use of 7% damping.

Results

1. Modelling

The response spectra analysis results showed:

- The building natural frequencies were in the area of the highest accelerations on the site response spectra;
- For each direction of earthquake, only one or two modes contributed to the loading.
- Each direction of earthquake had minimal effect on the forces in the tangential directions.

As the soil springs values can have considerable variability, the STAAD calculations are required to consider a +/- 50% range, as per clause 5.2.3 of N289.3 Adding mass or stiffening of the soil to modifying the natural frequencies would not likely change the building accelerations.

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2. Acceleration Threshold

The results of the analyses determined that soil bearing is not a concern until the seismic factors are near the upper bound for the 5% damping scenario. The governing limitation for the building is stability (overturning), and the upper thresholds for which the accelerations can increase are;

- 1.25 times for the 5% damping response spectra, and
- 1.4 times for the 7% damping response spectra.

The range for allowable amplification is 1.25 to 1.4.

3. Acceleration Demand (Dynamic Load Factor)

The dynamic load factor was capped at an amplification of 3.0 based on the impedance factor. The results from the dynamic loading for the expected range of the soil frequency is an 8% to 15% increase (ie. 1.08 to 1.15) of the building overturning moment.

Further, if the soil frequency variability is adjusted by -50%, as per 5.2.3 of N289.3, it would equal the building frequency, and the increase in loading is 33%, therefore the likely upper bound of the amplification is 1.33.

The range for expected amplification is 1.08 to 1.33.

Conclusion

It was anticipated upon the completion of the preliminary analysis that significant margin would exist between the resistance threshold values for the amplification of response spectra acceleration and the conservative estimate of the demand amplification through sound engineering principles.

At the current stage of the analysis, the lower range of threshold amplification values overlap the upper range of the expected demand.

Recommendation

The results of the preliminary analysis do not provide RCMT with adequate certainty that a detailed analysis would produce a stable structure, and as such, RCMT cannot endorse a decision to elevate the foundation of the HWMB West Annex at this time.

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References:

NK38-DG-03650.2B Rev 06
Geotechnical Reports (EXP)
NK38-REP-38000-10001
NK38-REP-38000-10123
Soil Parameters for Base Slab Re-Design – 08SEP2014
CSA N289.3-10

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