Toronto York Spadina Subway Extension Tunnelling under Schulich Building.

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Abstract

Tunnelling under the York University Schulich Building was one of the milestones of the Toronto-York Spadina Subway Extension (TYSSE) Project. The challenge was to bore the tunnel under a flagship building only one diameter below the foundation without a compensation grouting process. Intensive finite element structural and geotechnical analyses of the building and the tunnel were performed, together with monitoring field measurements. The results showed that compensation, included in the initial design, was not necessary to ensure the building's structural integrity. The tunnel monitoring during crossing confirmed the results of the performed analyses. The paper highlights the difficulty of deciding whether a compensation grouting is necessary or not when tunnelling under buildings and explains the procedure that was followed in this particular case.

1. Introduction

Tunnelling under existing structures is a challenge that engineers often face in urban underground projects. Therefore, great efforts were devoted in the past to predict the impact of tunnelling on existing buildings, in order to anticipate any potential damage (Attewell et al. 1986; Lee et al. 1992a,b; Mair and Taylor 1997; Burland et al. 2001; Melis et al. 2002; Viggiani and Soccodato 2004; among others). The impact of tunnelling could be significant when sensitive buildings are affected (Boscardin and Cording 1989; Bilotta et al. 2017; Ledesma and Alonso 2017). Even in cases where there are no existing structures, predicting greenfield settlements is a difficult task due to geotechnical uncertainties and difficulties in simulating tunnelling operations. In fact, any ground excavation involves a stress release, and therefore, a proper analysis requires a good knowledge of the soil initial stress state and the constitutive soil behaviour for accurately studying the impact of unloading-reloading processes.

Furthermore, there is increasing evidence that tunnel-building interaction may be significant in some cases and thus, building stiffness should be taken into account when analyzing the impact of tunnelling on existing structures (Potts and Addenbrooke 1997; Dimmock and Mair 2008). However, the analysis of this type of “coupled interaction” is a difficult and time consuming task. For this reason, most analyses are performed in a conservative two-step process which consists of computing the tunnelling displacements first and then applying these to the structural model of the building.

This paper studies a real case involving tunnelling beneath an existing building for which compensation grouting (Mair 2008) was proposed in the project. Deciding whether compensation grouting is required is usually quite difficult. That decision should be based on the prediction of damage to the building due to tunnelling and, as stated above, should only be made after assessing the risk of the overall project.

The case refers to the Toronto-York Spadina Subway Extension consisting of two parallel tunnels passing beneath a building of York University Keele campus. The original project plan proposed compensation grouting for the building foundation. But, grouting was difficult to apply in practice, as drilling the compensating pipes was found to be very difficult in sandy soils below the water table. Based on experience with previous sections showing good performance of the TBM, the contractor proposed to proceed with tunnelling beneath the building without compensation grouting. The paper describes the main aspects of the original project and the analyses that were carried out to support the contractor's proposal. Tunnelling without compensation grouting was finally carried out without any significant impact on the University building.

2. Case Study Description

The Toronto-York Spadina Subway Extension (TYSSE) Project includes an underground station and tunnelling beneath the York University Keele campus. The construction was awarded to the OHL – FCC Limited Partnership in January 2011.

One of the most challenging aspects of the project was to tunnel under the Schulich Building, where the Schulich School of Business is located. It was feared that the short distance between the crown of the twin tunnels and the building's foundation, which is about one tunnel diameter, and the vicinity of the excavation of the station enclosure, could lead to dangerous building foundation settlements.

The YYSSE consists of two 5.4 m inner diameter tunnels bored by EPBs. The axes of the twin tunnels are separated by 13.6 m and the crowns are 6.9 m beneath Schulich Building's foundation which is equivalent to 1.14 tunnel diameters. In addition, a subway station was planned at York University Keele Campus. The enclosure for the station is formed by pile walls

and tiebacks struts substituted for the tiebacks at the ends of the enclosure. The enclosure was about 160 m long, 28 m wide, and 20 m deep below the surface. As shown in **¡Error! No se encuentra el origen de la referencia.**, its south end is 12 m away from the Schulich Building façade.

The Schulich Building is a three-storey building with a basement. It is a reinforced concrete structure supported on footings. The basement walls are reinforced concrete stiff walls and the columns of the structure rise from that basement wall. The reinforced concrete stair cores absorb possible lateral forces on the building. In some areas, such as where large classrooms are located, the slabs span up to 15.9 m. The tunnel crosses underneath the North and the East Wings of the building, as shown in **¡Error! No se encuentra el origen de la referencia..**

In order to assure that the building would not be damaged, three compensation grouting shafts were originally designed (**¡Error! No se encuentra el origen de la referencia.**). These shafts were designed to compensate for any unexpected settlement that could produce structural damage to the building. The building was to be fully occupied and operative during the tunnelling process.

From a geotechnical point of view, York University Station site is dominated by an alternation of cohesive and granular levels.

The geotechnical profile is shown in **¡Error! No se encuentra el origen de la referencia..** Two cohesive geotechnical units (Upper and Lower Till) are present. Soil tests showed that some samples taken from those two geotechnical units are more clayey in nature while others are siltier.

In addition, a continuous granular level (Upper Granular geotechnical unit) is present below the Upper Till level, while another granular formation (Lower Granular) is found at a depth of about 40 m. All the geotechnical units have an overconsolidated nature and a significant

stiffness due to the geological processes that took place during the last Quarternary glacial period.

From a hydrogeological point of view, piezometers show a vertical natural gradient between the Upper Granular levels (piezometric head values of about 195 masl) and the Lower Granular geotechnical unit (piezometric head value about 172 masl).

3. Application of the methodology to the case study

3.1 Geotechnical model

In order to obtain accurate predictions of the settlements under the Schulich Building footings, it was important to select an appropriate set of geotechnical parameters and constitutive models. Although the number of boreholes and the geotechnical testing performed during the design stage were satisfactory to obtain basic parameters of the different geotechnical units, other factors, such as the performance of the EPB (volume loss), or the earth pressure coefficient K_o could not be properly estimated before tunnelling started. Therefore,

it was decided to define the geotechnical parameters from initial in situ investigations, and perform a back-analysis of a monitored section to estimate the volume loss and K_o coefficient.

Several monitoring arrays were designated to collect data as tunneling took place. Monitoring arrays 32 & 33a (located at chainage 16+370) were selected for the back-analysis. As shown in Figure 3 for array 33.a the following monitoring devices were installed at each array:

- Surface monitoring points (SMP).
- A bidirectional inclinometer device, 3 m left from Northbound Tunnel.
- Two extensometers placed above the crown of each tunnel axis.

Settlements were so small that SMP measurements were within the range of error of the device. This fact prevented a primary calibration of the model with SMP values as usually

carried out in a typical back-analysis procedure. Therefore, only the data from the inclinometers were used to back-analyze the volume loss and the K_o coefficient.

Numerical modelling was carried out using PLAXIS 2D 2011 Geotechnical Finite Element Code (PLAXIS 2011). Ground displacements due to tunnelling should be far from failure and should remain within the elastic range. Therefore, elastic soil properties, rather than the classical cohesion and friction angle, become very important when estimating settlements in this condition which requires accurate evaluation of the stiffness at small strains (Clayton 2011). To this aim, the Small Strain Hardening Soil mechanical constitutive model (HSS) was adopted. It is an elastoplastic model implemented in PLAXIS that considers the high soil stiffness at very small strains and assigns different Young moduli for loading and unloading, using plasticity theory when approaching failure conditions. Details of the model formulation can be found in Schanz (1999) and in the Plaxis Manual (PLAXIS 2011).

An initial set of geotechnical constitutive parameters was estimated from available laboratory and in situ tests. Seismic survey data were analysed in order to estimate small strain elastic parameters; and results from pressuremeter tests were used for conventional elastic parameters. In granular geotechnical units (Upper Till granular, Upper Granular and Lower Till granular), SPT blow data correlations with elastic parameters were also considered. Table 1 shows the mechanical constitutive parameters adopted for input in the HSS model.

The back-analyses used the soil constitutive parameters from Table 1 and operational EPB parameters as fixed values. Different values of ground loss were imposed in PLAXIS using that specific option of the code. That value represents the performance of the EPB in a single number. Then, the coefficient of lateral earth pressure at rest (K_o) was estimated to match computed horizontal displacements to those obtained from inclinometer measurements.

Back-analysis of field measured responses can be performed to adjust input parameters by means of numerical optimization techniques (de Santos et al. 2011). But in this study, a trial

and error procedure was used to estimate the best values of volume loss and K_o . A reasonable match between measured and calculated responses was obtained with a volume loss of 0.25% and a K_o coefficient of 1.50. These values were assumed in the subsequent finite element simulations.

a. Schulich Building Geotechnical Model

Once the volume loss and K_o were calibrated by using measurements from monitoring arrays 32 and 33a, a geotechnical model for the Schulich Building foundation was performed.

Rather than using a 3D model for the foundation, it was decided to consider a simpler 2D plane strain geomechanical model to keep all analyses within a reasonable timeframe. However, a 3D model was developed for the structural analysis of the building due to its complex geometry.

The following aspects were taken into account in the geomechanical model:

- EPB excavation process: This includes the consideration of the distance of the excavation front from the analyzed cross section, the tunnel lining construction process, as well as mortar injection and hardening.
- Foundation model: Schulich Building foundations are mainly square or rectangular footings. Transforming the rectangular loads in a 3D geometry to an equivalent load in a 2D plane strain problem becomes crucial in settlement predictions.
- Geotechnical model: The two wings of the building affected by the excavation process have clearly different foundation geometries. As a result, a different 2D model was developed for each building wing.

Five cross sections labelled as 1, 1', 2, 2' and 2'' were modelled. As shown in **¡Error! No se encuentra el origen de la referencia.**, cross sections 1 and 1' are related to Schulich Building's East Wing and cross sections 2, 2' and 2'' are for the North Wing.

Cross sections 1 and 2 have different geotechnical profiles. Sections 1', 2' and 2'', only differ from sections 1 and 2 respectively in foundation geometry and loads. The geometries of cross sections 1 and 2 are shown in **¡Error! No se encuentra el origen de la referencia.** and **¡Error! No se encuentra el origen de la referencia..**

The geotechnical profile and the soil constitutive parameters were estimated from boreholes in the York University Station area using a similar procedure to that adopted for the analysis of arrays 32&33a. Table 2 shows the HSS mechanical constitutive parameters adopted in the analyses. The parameters are slightly different to the ones obtained from the analysis of arrays 32&33a because of the different properties measured during the soil investigation of this site. However, K_o parameter and the volume loss of the EPB machine were assumed to have the same values obtained from the back analysis of the measurements at arrays 32&33a.

The interface between each two segments of tunnel lining was modelled as a plastic hinge with a rotation stiffness based on Blom (2002) formulation and the results of the back-analysis of measurements performed for the Jubilee Line tunnels in London (de Santos et al. 2011).

Vertical loads acting on each foundation were obtained from the Seymour Schulich Building structural model, as explained in section 3.2.

The actual foundation footings are either square or rectangular. A proper 2D plane strain model has to take into account the different stress distribution produced by a strip load (the only type that can be modelled in 2D) to that of the actual rectangular load distribution in a geological medium. This is achieved using Jurgesson (1934) and Holl (1940) formulations.

b. Geotechnical Analysis Results

The geomechanical cases considered were:

- a. Basic cases: Settlement profiles and other geotechnical results were obtained for cross sections 1, 1', 2, 2' and 2'', taking into account building geometry and foundation loads.
- b. *Green field* cases. Settlement profiles in cross section 2 were analysed without the building. This can be regarded as a sensitivity analysis which shows the importance of the effects of foundations.

In all the basic cases, the modelled pre- and during tunnel construction stages were:

- Seymour Schulich Building construction.
- Foundation Consolidation process after building construction. This stage models the pore pressure dissipation after the excavation and application of foundation loads.
- Northbound tunnel boring by an EPB machine.
- Segment concrete lining construction, mortar injection and hardening.
- Time Interval between Northbound and Southbound tunnel boring: At this stage only foundation consolidation process takes place.
- Southbound tunnel boring by an EPB machine.
- Segment concrete tunnel lining construction, mortar injection and hardening.
- Final foundation consolidation process until pore pressure excess is dissipated.

The most remarkable results from the geomechanical simulation are summarized in this Section. **¡Error! No se encuentra el origen de la referencia.** shows the computed settlements for the cross section 2 in the basic case. The settlement profiles depicted in **¡Error! No se**

encuentra el origen de la referencia. were considered as “design profiles” for damage assessment of the Schulich Building, with a maximum estimated value of 6 mm.

For comparison purposes, a simulation without the building was also carried out. Two different geomechanical models without building were analyzed for cross section 2:

- Original profile case: Total overburden is considered (original natural profile).
- Minimum overburden case: An excavation up to lower foundation level is considered.

The results for these two hypothetical cases as compared to the results of the base case, are useful for assessing the relative importance of foundation loads and overburden on the settlements. Note that soil stiffness in the HSS model depends on confinement, and therefore, on the stress history of the soil. Figure 8 shows the computed settlements for these cases.

If settlement distributions are compared, two main conclusions can be outlined:

- Smoother settlement distributions are obtained in both *green field* cases compared with the base case. Footing foundations produce steeper profiles, as they concentrate settlements on loaded areas.
- The computed settlements in the minimum overburden case show the effect of the dominant horizontal *in situ* stress: a small heave is obtained in each tunnel axis. Maximum computed settlements are smaller than 3.2 mm.

3.2 Building structural model

A complete three-dimensional elastic finite element model was developed for the building. The model included foundations, supports, walls and slabs.

The geometry of the building and the cross sections of the structural elements were obtained from the building design drawings. The self-weight was estimated directly from the geometry. Other permanent loads were assumed to add up to 25% of the self-weight, which is a conservative value.

It is widely accepted that small settlements do not have any significant influence on the failure potential of concrete structures because extensive cracking eliminates bending moments produced by settlements. Thus, the ultimate limit state was not considered in this analysis.

Settlements may produce concrete cracking and displacements that could be detrimental to the proper functioning of windows and doors, or attachment of façade elements. Therefore, the aim of this calculation was to obtain the displacements and stresses due to settlements produced by tunnelling and compare them with those obtained from dead loads alone. In this analysis, the effects of the live loads were neglected, as this assumption would lead to conservative results.

¡Error! No se encuentra el origen de la referencia. shows the geometry of the model, which included the North and the East wings. Continuity boundary conditions were imposed on the slabs to model the connection to other parts of the building which were not included in the model.

a. Dead Load Results

When the dead load was applied, it was assumed that no settlements of the building were produced. Therefore, all the footings and the walls on the perimeter were supposed to be fixed. This is a conservative assumption because extensive cracking was observed on the foundation walls before tunnelling. Thus, as expected, settlements had already occurred due to the dead load alone.

259 **¡Error! No se encuentra el origen de la referencia.** shows the displacement contours on the
260 building due to the dead load alone. The maximum displacement in the structure is about 18
261 mm.

264 *b. Tunnel Settlements Results*

265 The settlements obtained from the 2D geotechnical model were imposed on the 3D structural
266 model using an approach that required several assumptions.

267 Basically, similar footings with the same distance to the tunnel axis were assumed to have the
268 same settlement, taking into account their location on the Northbound or the Southbound.
269 The maximum computed settlement was 7 mm under the East wing core, where the stairs are
270 located. The maximum computed settlement in the North wing was 5.5 mm in the footings
271 over the tunnels.

272 **¡Error! No se encuentra el origen de la referencia.** shows the displacements on the structure
273 due to the computed settlements produced by tunnel boring. Maximum displacements are
274 about 5 mm under the North wing and 6 mm under the East wing. Therefore, displacements
275 are less than 1/3 of those due to dead load, even though the last ones are underestimated by
276 assuming fixed walls and footings when the dead load is applied.

277 *c. North wall/North Wing Results* 278

279 Only the computed results corresponding to the North wall of the basement of the North wing
280 are discussed in this paper, because of the length limit. The North wall is at the basement level
281 and it is a 25 MPa concrete wall, 350 mm thick. It is the most sensitive element to settlements
282 because of its large stiffness on the wall plane.

Because of the no-settlement hypothesis for dead load, vertical foundation displacements are zero for that case. Therefore, the maximum normal stresses are very low, as shown in **¡Error! No se encuentra el origen de la referencia..** Obviously, most of the wall is in compression and some tension stresses appear only in the connection with the porch and the ground level slab. Tangential stresses are also very small for the same reason. Nevertheless, this wall was extensively cracked before tunnelling, as observed and mapped. That is, settlements were already produced after completion of the building.

¡Error! No se encuentra el origen de la referencia. shows the maximum normal stresses after tunnelling as computed by the structural model. On top of the tunnel axis, stresses of about 7 MPa were obtained. Green values are about 2.5 MPa, which are close to the expected tension strength of the concrete. The maximum tangential stress is about 3 MPa, far from the compression strength. If the material had originally been uncracked, then some cracks would have appeared after tunneling. But, as mentioned, the building had already undergone severe cracking before tunneling was initiated. Therefore, tunneling may produce to some small new cracks or, most probably, the existing cracks would be slightly wider and/or longer. Existing pre-tunneling cracks were concentrated close to the foundations, where maximum tensile stresses appeared due to settlements.

3.3. Trial Monitoring Area

Following TTC (Toronto Transit Commission) suggestions, a trial monitoring area was arranged prior to both EPBs passage under the Schulich Building. The purpose was to confirm geomechanical parameters and EPB performance parameters adopted in the previous analyses and confirm the feasibility of mining under the Schulich Building without the need to implement the compensation grouting planned for in the in the original project. This trial monitoring area was located in the York University Station enclosure (chainage interval 15+900

to 16+020), very close to the building. Geotechnical conditions of the trial monitoring area are very similar to the ones below the Schulich Building.

Monitoring devices included MPBX extensometers, inclinometers and vibrating wire piezometers. Most of the MPBX extensometers were placed on the tunnel crown, while inclinometers were installed at the tunnel axis or 1.0 m away from tunnel sidewalls.

The most relevant measurements were related to soil movements. In general, MPBX surface topographic levelling showed settlements smaller than 10 mm, as can be observed in **¡Error! No se encuentra el origen de la referencia..**

A numerical calibration process was carried out to assess the EPB performance parameters (face volume loss) that governed during the EPB tunnelling process along the York University Station trial monitoring area. Some relevant geomechanical parameters that strongly controlled settlement trough, such as lateral earth pressure coefficient, were also estimated. During this calibration process, a comparison of recorded data with geomechanical model results was carried out.

In a first stage, the analysis adopted the geotechnical constitutive models and parameters described in previous sections. In order to take into account the small variations of the geotechnical profile, two analysis sections (cross sections A and B) were considered in the trial monitoring area. Code PLAXIS 2D (PLAXIS 2011) was used for the analyses. Cross section A includes slightly more cohesive materials, whereas in cross section B, more granular geotechnical units (Upper Till granular, Upper Granular, Lower Till granular) are predominant.

MPBX results, that can be considered reliable, showed a small settlement gradient in depth. This fact is consistent with a lateral earth pressure coefficient, K_o , of about 1.0. Regarding the excavation volume loss, the comparison of numerical results with measurements from MPBX-2, MPBX-4 and MPBX-6, suggests a value of about 0.20% (**¡Error! No se encuentra el origen de**

la referencia.). An increase of the recorded settlements with time was noticed, due to consolidation, after the face excavation of the section. Therefore, it can be stated that measurements generally confirmed both the EPB performance and the geotechnical parameters considered in the initial analyses.

All analyses lead to the conclusion that it was possible to tunnel under the building without compensation grouting and no structural effects on the building were expected. Nevertheless, when this conclusion was stated, horizontal drilling activities from compensation shafts 2 and 3 had already started encountering major difficulties when preventing waterjets and sand erosion.

4. Schulich Building Monitoring Layout

Any urban underground project must be closely monitored. In this case, monitoring was essential and the results were very useful to confirm the methodology adopted for the analysis and to assess the integrity of the building itself.

The overall system included geotechnical instrumentation in the surroundings of the Schulich Building, geotechnical and structural monitoring installed inside the building and monitoring of the EPB working parameters.

The geotechnical instrumentation involved borehole extensometers, biaxial inclinometers and vibrating wire piezometers. In addition, Schulich Building monitoring was based on pressure transducer settlement monitor systems (PTSMS), precise levelling points (PLP), tape extensometers, uniaxial tilt meters (TM) and robotized theodolites for surveying. Both PLP and liquid level arrays measured the building vertical movements near the foundation level. As a result, they can be regarded as the most reliable measure of building settlements.

EPB performance was also monitored and the following parameters were controlled: thrust or torque time evolution, EPB face pressure with six load cells placed at different levels inside the earth chamber, mortar lines pressure and flow rate time evolution, weight of the excavated material for each ring, information regarding the additives (polymers or foam). Most of the data were recorded continuously.

a. Tunnelling Process

The positive results of the developed models, the small measured displacements, and the difficulties experienced when drilling the pipes led to the decision to tunnel under the building without compensation grouting. In fact, the Northbound drive started on September 26th 2012, 6:00 am and ended, after crossing under Schulich Building's East wing on September 28th 19:30. The Southbound drive started on October 1st 13:00 and ended on October 3rd 21:30.

Target EPB pressure (at tunnel axis level) was established to be 2.3 bars (230 kPa).

b. Monitoring Results

Building recorded settlements at foundation level during tunnelling were very small: heave or settlements values were below 1.5 mm. Both liquid levels arrays and precision levelling showed similar results. ¡Error! No se encuentra el origen de la referencia. presents settlement time evolution in liquid levels arrays at points near the tunnel axes. The values presented have been corrected in order to filter-nonexistent initial settlements, before EPB drives, in zones where no compensation grouting activities (borehole drilling) took place. This fact was especially relevant in the East Wing.

Another problem related to liquid levels arrays is that some of these showed a tendency to drift up for no apparent reason. That effect was also corrected.

Those problems did not arise with precise levelling data. **¡Error! No se encuentra el origen de la referencia.** shows the results.

As a result, it can be stated that tunnelling under the building produced negligible structural effects. In addition, no long-term settlements were observed.

The lack of consolidation settlements is especially evident from vibrating wire piezometers data analysis. Upper Granular readings showed a sharp increase of water pressure when each EPB started the boring process, as shown in **¡Error! No se encuentra el origen de la referencia..**

It has to be noted that cutter head was placed at Upper Granular level and that face pressure produced a water pressure increase of about 60 kPa (about 26% of prescribed face pressure).

As shown also in **¡Error! No se encuentra el origen de la referencia.**, pore pressure dissipation took place after each EPB start.

In addition, the lack of long-term settlements shows that geotechnical units present in Schulich Building area had a high stiffness when consolidation took place.

It was finally confirmed that compensation grouting was not required because the EPB operation was quite regular and recorded movements that were very small in agreement with the geomechanical model predictions. Finally, the correct operation of the EPB was verified and there were no unexpected incidents during tunnelling. No new cracks or crack width openings were reported in the Schulich building.

c. Comparison with Geotechnical Modelling

The calculated settlements obtained from the analysis were larger than the ones that actually took place during tunneling, even though those predictions did not produce relevant structural effects on the Schulich Building. Some of the reasons that may explain this fact are:

- Interaction between ground and Schulich Building is more complicated than two separate geomechanical and structural models are capable to reproduce. The adopted modelling approach is more conservative and leads to higher settlement values. Very likely, building stiffness plays an important role as reported in other cases (Potts and Addenbrooke 1997; Dimmock and Mair 2008).
- Adopted stiffness geomechanical parameters were realistic, but still slightly conservative.
- When trial monitoring area measurements (placed inside York University Station enclosure) were analysed, it was stated that EPB tunnelling could produce a face volume loss of about 0.20% instead of the 0.25% value adopted in the simulations. This also leads to more conservative results.

5. Final Remarks

This paper presents a comparison between the results of the analysis of settlements and related damage of a building under the effects of tunneling and actual measurements at the site during tunnel construction. The real case analysis and field measurement of the effect of tunneling on the settlement and damage to Schulich Building of York University in Toronto highlights the difficulties of anticipating the impact of tunnelling underneath existing buildings and particularly the dilemma on whether engineers should or should not design compensation grouting in advance. The observations made in this case may be useful for future similar works. In particular, the following conclusions can be stated:

- The proposed methodology allows to predict reasonably good estimates of settlements in the case of shallow tunnelling.

- 423 • During the design process, the estimation of geotechnical and EPB parameters is
424 conservative, which is logical , but may lead to overestimation of settlements and then
425 to designing mitigating solutions, such as compensation grouting, which may not be
426 necessary.
- 427 • Back analysis from tunnel monitoring shows to be a reliable technique, at least in this
428 case, to obtain geotechnical and EPB parameters. Tunneling projects should consider
429 exhaustive monitoring at the beginning of tunnelling in order to perform back analysis
430 at early stages of the work, and confirm the original design.
- 431 • Settlement predictions are linked to proper EPB machine performance, especially with
432 factors such as an adequate face pressure and mortar gap filling. Settlements are very
433 sensitive to the EPB operation. In this case, similar operation conditions to those
434 observed from arrays of field measurements were used in the finite element analyses
435 to improve the accuracy of the numerical results.
- 436 • Building footing foundation loads have a relevant effect on settlement trough
437 compared with *green field* cases. Settlements tend to increase below loaded areas.
438 This effect should be taken into account in the structural analysis.
- 439 • Predicted maximum settlements in Schulich Building analyzed in this paper ranged
440 from 6 to 7 mm at the foundation level. Those values are sensitive to many parameters
441 considered in the analyses. Even though, reasonable input parameters had been from
442 available in-situ field information, the final measured settlements were lower than
443 predicted.
- 444 • The settlements computed from the geotechnical models indicated that no significant
445 structural damage, nor serviceability problems, were expected for the Schulich

446 Building. Results from trial monitoring area, inside York University Station enclosure,
447 confirmed adopted geomechanical and EPB performance parameters.

448 • Recorded foundation settlements did not produce relevant structural effects on
449 Schulich Building. Those settlements were even smaller than the ones predicted. No
450 additional cracking, or increase of width or length of existing cracks was reported. No
451 malfunctioning of any element such as doors or windows was reported. Tunnelling was
452 imperceptible to the staff and students using the Schulich Building during construction.

453 • The monitoring devices (liquid levels or precise levelling) recorded no long-term
454 settlements.

455 • Recorded pore water pressure rise in Upper Granular geotechnical unit did not
456 produce noticeable consolidation settlements when pressure dissipation took place.

457 • EPB parameters during tunnelling process, mainly face pressure, were in agreement
458 with target values.

459 • Compensation grouting was not needed because the EPB operation was quite regular
460 and recorded movements were very small.

461 • Tunnelling with EPB machines can satisfactorily control settlements and structural
462 consequences on buildings, even if the depth of the crown of the tunnel is limited. An
463 integrated approach involving monitoring, numerical analyses and an elaborated
464 control of the EPB parameters in real time is key to success.

465

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Figure 1. Schulich Building foundation level and compensation grouting shafts plan view. The end of the York University Station enclosure is also shown. Geotechnical monitoring located near the Schulich Building. East Wing area: star symbols represent inclinometers, crosses are MPBX extensometers and circles are vibrating wire piezometers.

Figure 2. York University Station cross section geotechnical profile. It can be observed granular units (Upper Granular and Lower Granular), in yellow, and Upper and Lower till (cohesive) in green and brown (after Geotechnical Baseline Report – GBR- from the project).

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Figure 3. Cross section of monitoring array 33a.

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Figure 4. Schulich Building plan view with tunnels. Cross sections 1 and 1' (red lines) and 2, 2' and 2'' (blue lines) are shown.

Figure 5. Schulich Building cross section 1 geometric model. Upper and Lower Granular levels are represented in yellow (only Upper granular >50% fines content is present), Upper Till levels in brown and Lower Till level in green.

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545 *Figure 6. Schulich Building cross section 2 geometric model. Upper and Lower Granular levels are*
546 *represented in yellow and orange (Upper granular <50% fines content unit), Upper Till levels in brown*
547 *and Lower Till level in green.*

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551 *Figure 7. Schulich Building settlement profiles in cross section 2. Blue line corresponds to Northbound*
552 *tunnel boring and green one to Southbound tunnel excavation phase.*

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Figure 8. Cross section 2 “Green field” no overburden settlement profiles (circle symbols) compared with Base Case (rhombus symbols). Blue line represents Northbound tunnel boring and green one Southbound tunnel excavation phases.

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Figure 9. Model geometry. North wing on the left and East wing on the right.

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Figure 10. Vertical displacements due to dead load in mm

Figure 11. Vertical displacements due to tunnelling Settlements in mm. Same scale as **¡Error! No se encuentra el origen de la referencia.**

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573 *Figure 12.* Maximum normal stresses due to dead load on North wall of North wing in kN/m². Positive
574 means tension (dark blue, 2.5 MPa) negative compression (purple)

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578 *Figure 13.* Maximum normal stresses due to tunnel settlements on North wall of North wing in kN/m².

579 Positive means tension (dark blue, 5 MPa), negative compression (purple)

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Figure 14. Example of temporal evolution of surface settlements in MPBX devices placed on Northbound tunnel axis.

Figure 15. MPBX-4 results at different times after Northbound EPB boring (symbols) compared with numerical modelling results 8 days after boring (symbols and line in black) when a 0.2% face excavation volume loss is considered (cross section A). Red and green symbols (labelled as FE+3.0 m and FE+10.5 m respectively) indicate measurements obtained when EPB head is 2.0 m and 10.5 m distance from monitoring array.

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Figure 16. Schulich Building East wing liquid levels time evolution (arrays 1 and 3). EPB crossing under each array is represented by the vertical red lines.

Figure 17. Schulich Building settlements time evolution (precision levelling) in East Wing (arrays 1 and 3).

EPB tunnelling time under each array is represented by the vertical red lines.

Figure 18. Upper granular piezometric head recordings in Schulich Building North wing area. First pressure sharp increase corresponds to Northbound EPB and the second one to Southbound EPB.

617 *Table 1: HSS Constitutive model parameters monitoring array section 32&33a. Meaning: c' , cohesion; ϕ ,*
618 *friction angle; ψ , dilatancy angle; E_{50}^{ref} , Young modulus at 50% of the failure load for a reference*
619 *confinement of 100KPa; E_{ur}^{ref} , Young unloading-reloading modulus for a reference confinement of*
620 *100KPa; G_o^{ref} , Maximum shear modulus at very low strains, for a reference confinement of 100 KPa; $\gamma_{0.7}$,*
621 *shear deformation for which the shear modulus is reduced to 70% of the maximum value G_o .*

Geotechnical unit	c' (kPa)	ϕ (degrees)	ψ (degrees)	E_{50}^{ref} (kPa)	E_{ur}^{ref} (kPa)	G_o^{ref} (kPa)	$\gamma_{0.7}$
Fill	10.0	28.0	0.0	7000	21000	300000	$4.0 \cdot 10^{-5}$
Upper till cohesive	40.0	32.0	2.0	40000	120000	450000	$7.5 \cdot 10^{-5}$
Upper till granular	5.0	37.5	7.5	225000	675000	1170000	$3.75 \cdot 10^{-5}$
Upper granular	3.0	36.0	6.0	135000	405000	650000	$6.75 \cdot 10^{-5}$
Lower till cohesive	40.0	32.0	2.0	60000	180000	400000	$2.5 \cdot 10^{-4}$

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Table 2: HSS Constitutive model parameters. Seymour Schulich Building cross sections. See table 1 for

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symbols meaning.

Geotechnical unit	c'	ϕ	ψ	E ^{ref} ₅₀	E ^{ref} _{ur}	G ^{ref} _o	Υ _{0.7}
	(kPa)	(degrees)	(degrees)	(kPa)	(kPa	(kPa)	
Fill	10.0	28.0	0.0	7000	21000	300000	4.0 10 ⁻⁵
Upper till cohesive	20.0	30.0	0.0	40000	120000	450000	7.5 10 ⁻⁵
Upper till granular	5.0	37.5	7.5	100000	300000	281800	1.09 10 ⁻⁴
Upper granular (>50% fines)	3.0	36.0	6.0	75000	225000	284800	1.19 10 ⁻⁴
Upper granular (<50% fines)	0.0	36.0	6.0	75000	225000	284800	1.19 10 ⁻⁴
Lower till cohesive	40.0	30.0	0.0	64800	194400	386800	2.07 10 ⁻⁴