The application of load and resistance factors design (LRFD) in soil-structure interaction modelling to evaluate the stability of retaining structures

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ABSTRACT
As part of the Toronto Eglinton Crosstown Light Rail Transit (ECLRT) Project, re-alignment of Eglinton Avenue E. is required to accommodate the proposed guideway and U-shaped portal near Black Creek. This re-alignment is to include the construction of a 2.4 m to 8.0 m high Cast-in-Place (CIP) retaining wall in what is currently a regional catchment area and recreational site. Throughout design of the retaining wall, several challenges were encountered including: weak foundation soils, complex lithology, heterogeneous backfill material, spatial constraints, the presence of a water main, and flood conditions. Due to these design challenges, the conventional analytical methods would result in an over simplification of the problem and, hence, did not provide the level of detail required to address the various design concerns. To overcome these design challenges, soil-structure interaction modelling was utilized to analyze the external stability of the retaining structure and evaluate the mitigation measures to handle the geotechnical constraints. In addition, the Project Agreement (PA) stipulates design be completed following the American Association State of Highway and Transportation Officials (AASHTO) methodology. In order to remain compliant with the PA, Finite Element modelling in PLAXIS 2D (PLAXIS) was completed applying the AASHTO Load and Resistance Factors Design (LRFD) method. This paper discusses the procedure of applying the AASHTO load and resistance factors into PLAXIS, as well as the solutions utilized to combat the aforementioned design challenges.

RÉSUMÉ
En relation avec le projet du Transport Léger sur Rail Eglinton Crosstown (ECLRT) de Toronto, le réajustement de l'Avenue Eglinton Est est de mise afin d'adapter la voie de circulation proposée et le portail en forme de U près de Black Creek. Ce réajustement devra inclure la construction d'un mur de soutènnment en béton moulé sur place de 2.4 m à 8.0 m en hauteur dans ce qui est présentment une zone de captage régionale et site récréatif. Durant la conception du mur de soutènement, plusieurs obstacles furent rencontrés, incluant: faible fondation des sols, lithologie complexe, matériel de remblai hétérogène, contraintes d'espace, présence de conduit d'eau et conditions d'innondation. En raison de ces obstacles de conception, les méthodes analytiques conventionelles résulteraient en une sursimplification du problème et en conséquence, fait défaut de fournir le niveau de détail nécessaire afin d'adresser les nombreuses préoccupations de conception. Pour surmonter ces difficultés, l'utlisation du modèle d'interaction de structure de sol pour analyser la stabilité internale de la structure de soutènment et évaluer les mesures d'atténuation pour gérer les contraintes géotechniques fut d'importance. De plus, l'accord de projet stipule que la conception doit être completée suivant la méthodologie de l'Association Américaine des Administrateurs des Autoroutes et du Transport des États Fédérés (AASHTO). Afin de demeurer conforme avec l'accord de projet, le modèle d'élements finis avec PLAXIS 2D (PLAXIS) a été complété en appliquant la méthode de Facteurs de Conception de Charge et Résistence de l'AASHTO. Cet article vise la procédure d'application de Facteurs de Charge et de Résistence d'AASHTO avec PLAXIS, en autant que les solutions utilisées pour combattre les obstacles susmentionnés.

1. INTRODUCTION

The Eglinton Crosstown Light Rail Transit (ECLRT) Project is currently undergoing construction in Toronto, Canada and is scheduled to start operation in 2021. The ECLRT Project is owned by Metrolinx, operated by the Toronto Transit Commission, and is to be delivered using the Public-Private Partnership delivery model. Crosslinx Transit Solutions, has been selected to deliever the ECLRT Project. SNC-Lavalin Inc. is engaged in the project as a member of the Design and Construction Teams. The ECLRT Project is comprised of 15 underground stations, 10 at-grade stops, a maintenance and storage facility, and 19 km of guideway between Mount Dennis (Weston Road) and Kennedy Station. Of the 19 km alignment, 8 km of guideway is to be constructed at grade, while the remaining 11 km will be constructed below grade within the existing twin bored tunnels constructed under separate contract, between Keele Street and Brentcliffe Road. Accommodation of the U-shaped retaining structure which will serve to transition the Light Rail Transit (LRT) system below grade, required the re-alignment of Eglinton Avenue E. near Black Creek. The existing alignment of Eglinton Avenue E. west of Black Creek is situated atop a fill embankment within the catchment area adjacent to

Black Creek. The elevation differential between the existing roadway and the natural topography within the catchment area ranges between 2.4 m and 8.0 m. It was identified early in the project that realignment of Eglinton Avenue E. would necessitate the construction of a soil retaining structure, due to the limited right-of-way (ROW) available south of Eglinton Avenue E. Construction of the retaining structure was completed in 2018.

1.1 Geotechnical and Flood Conditions

The Cast-in-Place (CIP) retaining wall at Keelesdale Park has been constructed within the catchment area adjacent to the existing Black Creek. Available geotechnical reports at the time of design, described the lithology at the site as being comprised of variable fill materials, overlying interbedded glacial and proglacial sediments atop bedrock of the Georgian Bay Formation.

The retained in-situ soil behind the CIP wall is comprised of heterogeneous fill material. Standard Penetration Tests (SPTs) previously completed within the region and soil descriptions within the available borehole logs indicated in-situ soils at the founding elevation were comprised primarily of weak surficial fill underlain by weak variable materials. These conditions were confirmed through the completion of Cone Penetration Tests (CPTs) within the footprint of the proposed structure.

In addition, existing geotechnical reports noted that the CIP retaining wall at Keelesdale Park is situated within a regional catchment area; therefore, a significant increase in porewater and surface water is known to be associated with regional flooding events.

1.2 Spatial Constraints

The design options available for the CIP retaining wall at Keelesdale Park were limited due to the presence of several spatial constraints including: the available ROW, the existing roadway, and existing construction works within the immediate vicinity of the structure.

1.3 Client Requirements

Supplemental to the acceptance criteria outlined within the Project Agreement (PA), Project Stakeholders requested that the soil retaining structure be designed as a conventional CIP retaining wall.

2. AASHTO LOAD AND RESISTANCE FACTOR DESIGN METHODOLOGY

2.1 Analytical Design Method and Considerations

The LRFD methodology outlined in AASHTO (2014), applies load factors and resistance factors in design, and is based on the principal that the factored resistance within the soil retaining system must be greater than or equal to the factored load within the system in order to achieve stability.

The design of a retaining structure following the LRFD methodology outlined in AASHTO (2014), requires that the external stability of the structure be evaluated in respect to 5 mechanisms of failure. The mechanisms of failure identified within AASHTO (2014) are: Sliding, Bearing Resistance, Limiting Eccentricity, Global Stability, and Excessive Settlement.

The term Capacity to Demand Ratio (CDR) is used to quantify the ratio of the factored resistance to the factored load. A CDR of 1.0 is indicative of a stable design. Resistance factors of 0.55 and 1.00 are used for bearing resistance and sliding, respectively for CIP retaining walls (AASHTO, 2014). The load factors applied are outlined in the AASHTO manual and are summarized in Table 1.

Table 1. Load Factors (AASHTO, 2014).

Type of Load	Load Factor	
	Maximum	Minimum
EH: Horizontal Earth Pressure	1.50	0.90
EV: Vertical Earth Pressure	1.35	1.00
LL: Live Load	1.75	-
DC: Dead Load of Structural Components	1.25	0.90

2.1.1 Sliding

Instability with regards to sliding occurs when the resisting force of the retaining structure, which is the shear resistance along the base of the wall or of a weak layer near the base of the structure, is less than the horizontal component of the thrust on the vertical plane at the back of the wall. Sliding stability for a horizontal back slope is evaluated in reference to Equation 1, (AASHTO, 2014).

$$\mathbf{\Phi_t R_t \geq P_d} \quad [1]$$

Where:
Φ_t = resistance factor
R_t = nominal sliding resistancel
P_d = factored horizontal driving force

2.1.2 Bearing Resistance

Failure by means of Bearing Resistance generally occurs as a result of two modes of failure: general shear failure, and local shear failure. General shear failure is the term used to describe failure resultant of the typical shear failure wedge; while local shear failure is characterized by a punching or squeezing of the foundation soil and occurs when soft or loose foundation soil is present. Bearing Resistance is evaluated in reference to Equation 2, (AASHTO, 2014).

$$\Phi q_n \geq q_{V-F}$$ [2]

Where:
q_n = nominal bearing resistance
Φ = resistance factor
q_{V-F} = factored bearing stress

2.1.3 Limiting Eccentricity

Limiting Eccentricity, formerly known as Overturning is a criteria based on evaluation of the distance between the resultant foundation load and the center of the base length. Eccentricity is calculated by the summation of the overturning and the resisting moments about the bottom, center of the base length, and dividing by the vertical load, as described in Equation 3, (AASHTO, 2014).

$$e = \frac{\sum M_D - \sum M_R}{\sum V}$$ [3]

Where:
M_D = driving moments about the bottom centre of base
M_R = resisting moments about the bottom centre of base
V = vertical load

2.1.4 Global Stability

Following the LRFD methodology, Global Stability is evaluated in reference to the Allowable Stress Design (ASD) method. The ASD method calculates the Factor of Safety (FS) against global stability failure as defined in Equation 4, (AASHTO, 2014).

$$FS = \frac{M_A}{M_R}$$ [4]

Where:
M_A = driving forces, primarily weight and thrust from the embankment
M_R = resisting forces, primarily mobilized shear strength along the length of the failure plane

2.1.5 Excessive Settlement

The evaluation of foundation settlement should be completed at the time of design and must consider both total and differential settlements that may occur as a result of the elastic, primary and secondary stages of settlement

2.1.6 Limitations and Assumptions

Application of the LRFD method is consistent with the current state of practice for evaluation of external stability for soil retaining structures. The LRFD method allows the designer to independently factor the horizontal and vertical forces generated by the various soil masses and surcharge loads. Although the analytical approach outlined within AASHTO (2014) has been widely adopted in North America, the Analytical LRFD Method (AASHTO, 2014) is not without its limitations. Key limitations of the Analytical LRFD Methodology outlined within AASHTO (2014) are listed below:

- The analytical method is only able to consider three unique soil zones (Retained, Foundation, and Reinforced [applicable only to evaluation of Mechanically Stabilized Earth structures]).
- The analytical method allows for calculation of the foundation soils with a maximum of two layers.
- The analytical method is not able to account for the presence of variable materials within the retained soil zone.

The limitations listed above are not inherent within the LRFD method, but are introduced as a result of the simplification required to make the analysis less cumbersome. For typical design cases, the analytical method outlined within AASHTO (2014) has been proven adequate for evaluation of external stability.

3. FINITE ELEMENT ASSISTED LRFD METHOD

3.1 The Finite Element Method

The Finite Element Method (FEM) is the predominant method within geotechnical mechanics by which various mathematical models may be divided into disjointed elements (finite elements), and subsequently solved. Due to the process of discretization, solution of the mathematical models representing each finite element using conventional methods would not be economical. In order to obtain a converging solution following the FEM, various Finite Element (FE) modelling programs have been developed. The available FE modelling programs may vary in regards to the requisite input parameters, user interface, 3D capability and mathematical models available to the user.

For the purpose of the FE Assisted LRFD Method outlined herein, the FE modelling program, PLAXIS 2D (PLAXIS), was utilized to evaluate the external stability of the CIP retaining wall at Keelesdale Park. The analysis was completed within the 2D version of PLAXIS; therefore, the plane strain assumption has been considered. For soil retaining structures of significant complexity and/ or discontinuous geometry, the use of a 3D FE modelling program may be advantages.

3.1.1 Application of Load Factors

Analysis in PLAXIS following the AASHTO LRFD methodology requires the application of the appropriate load factors. In order to simulate this condition in PLAXIS, the soil properties within the model were modified to result in factored forces. The unit weight (γ) of the materials was increased by the vertical earth pressure load factor (γ_{EV}). In order to apply the horizontal earth pressure load factor (γ_{EH}) while taking into account the modified unit weight, the active earth pressure coefficient was modified by a new load factor (γ_{ka}) determined by Equation 5. and Equation 6.

$$\gamma_{EH}F_1 \ = \tfrac{1}{2}\gamma_{ka}k_a\gamma_{EV}\gamma H^2$$

[5]

$$\gamma_{ka} \ = \frac{\gamma_{EH}}{\gamma_{EV}} = \frac{1.5}{1.35} = 1.11$$

[6]

Where:
γ_{ka} = new active earth pressure coefficient load factor
$\mathrm{F_1}$ = earth pressure driving force
γ_{EH} = horizontal earth pressure load factor
γ_{EV} = vertical earth pressure load factor
γ = unit weight
H = height

The load factor for the traffic load (γ_{LS}) was modified in order to account for the increase observed by factoring the active earth pressure by γ_{ka}. The modified load factor (γ_{mLS}) was determined by Equation 7 and Equation 8.

$$\gamma_{LS}F_2 \ = \gamma_{ka}k_a\gamma_{mLS}qH$$

[7]

$$\gamma_{mLS} \ = \frac{\gamma_{LS}}{\gamma_{ka}} = \frac{1.75}{1.11} = 1.58$$

[8]

Where:
γ_{mLS} = modified load factor for traffic loading
$\mathrm{F_2}$ = live load (traffic) driving force
γ_{ka} = new active earth pressure coefficient load factor
γ_{LS} = live load factor
q = live load (traffic) surcharge
H = total height

The driving forces are factored based on the method described above; therefore, the FS (as an output of PLAXIS) is described by Equation 9. The CDR is calculated by applying the corresponding resistance factor to the resisting forces as shown in Equation 10
.

$$FS = \frac{\sum F_r}{\sum \gamma F_d}$$

[9]

Where:
F_r = resisting forces
$\mathrm{F_d}$ = driving forces
γ = applied load factor

$$CDR = \frac{\theta \sum F_r}{\sum \gamma F_d} = \ \theta FS$$

[10]

Where:
F_r = resisting forces
$\mathrm{F_d}$ = driving forces
γ = applied load factor
θ = applicable resistance factor

In calculations where the minimum load factor is applied, such as in the case in evaluation of Sliding and Limiting Eccentricity, the unit weight is reduced by the applicable load factor.

3.1.2 Sliding

Analysis of the failure by means of sliding was conducted within PLAXIS by applying the appropriate load factors to the materials within the conventional zones as defined by AASHTO (2014) and presented in Figure 1.

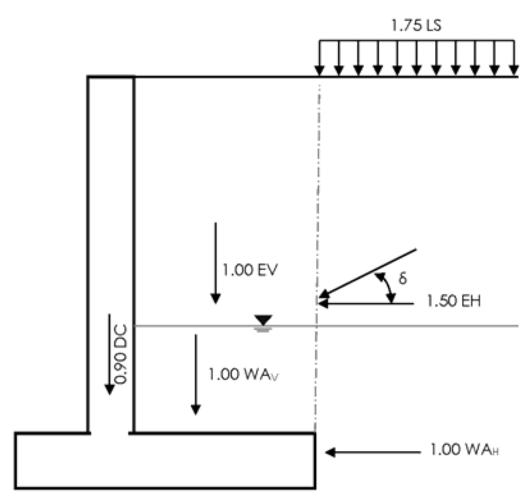

Figure 1. Typical application of load factors for sliding and eccentricity as defined by AASHTO (2014).

Through evaluation of a cross-section drawn at the base of the wall in PLAXIS, it is possible to obtain the shear stress (τ_1) and shear strength (τ_{max}) determined at the location of each node. The resistance to Sliding expressed in terms of a CDR was calculated by determining the weighted average of the CDR of each node, and is expressed in Equation 11.

$$CDR_{sliding} = \sum_{i=1}^{n} w_i{}^\circ \left(\frac{\theta\tau_{max}}{\tau_1}\right)_i$$

[11]

Where:
τ_1 = shear stress, factored as per Section 3.1.1
τ_{max} = interface shear strength
θ = applicable resistance factor
w_i = relative weight

3.1.3 Bearing Resistance

The bearing stress of the soil was determined in PLAXIS by first applying the appropriate load factors to the materials in the conventional zones as defined by AASHTO (2014) and presented in Figure 2.

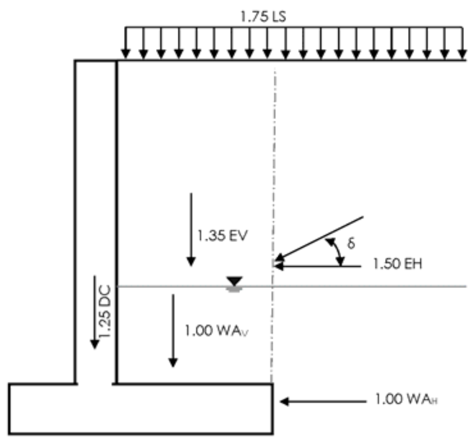

Figure 2. Typical application of load factors for bearing resistance as defined by AASHTO (2014).

The Bearing Resistance of the foundation is evaluated by applying a predefined displacement to the foundation and plotting the stress history calculated at two nodes close to the toe of the wall and the heel of the wall, respectively. The stress-strain curve depicts the stress (σ'_{yy}) at which the soil experiences failure (i.e. continuous displacement under a constant stress), and thus the unfactored ultimate bearing resistance, as depicted in Figure 3.

The ultimate bearing resistance as determined by the stress-strain curve is factored by the appropriate resistance factor, similar to the analytical method. The weighted average of the normal stress (σ'_N) obtained from each node is calculated and is utilized as the factored stress to determine the CDR as expressed in Equation 12.

$$CDR_{bearing} = \frac{\theta \sigma'_{yy}}{\sum_{i=1}^{n} w_i \, \sigma'_{N_i}} \quad [12]$$

Where:
σ'_{yy} = ultimate bearing resistance
σ'_N = maximum normal stress, factored as per Section
w_i = relative weight
θ = applicable resistance factor

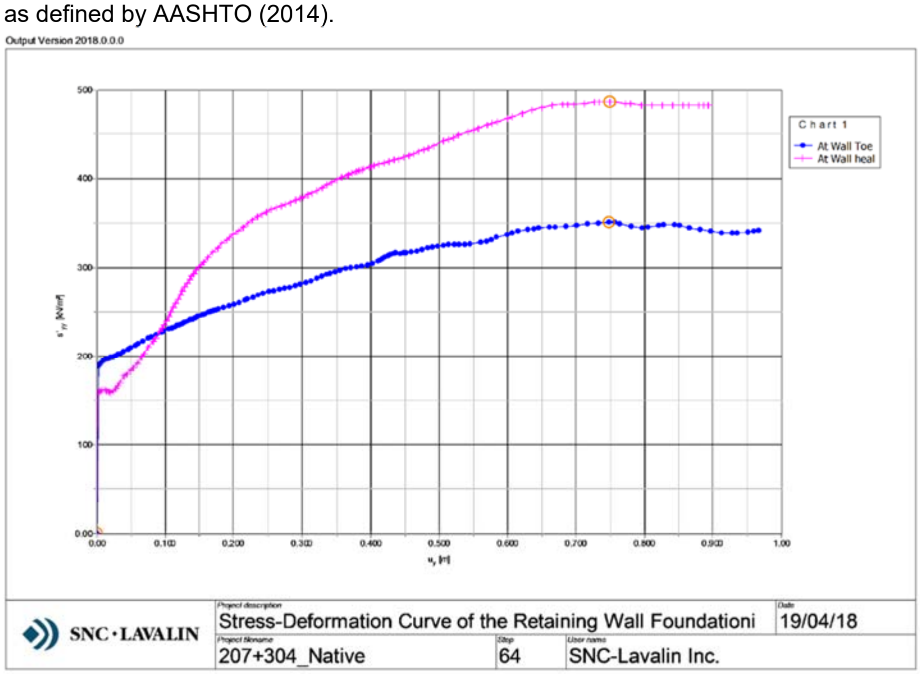

Figure 3. Stress-strain curve along the base of the wall as determined in PLAXIS.

3.1.4 Limiting Eccentricity

Limiting eccentricity, formerly known as Overturning is the distance between the midpoint of the base and the resultant of the effective normal stress (σ'_N) as depicted in Figure 4.

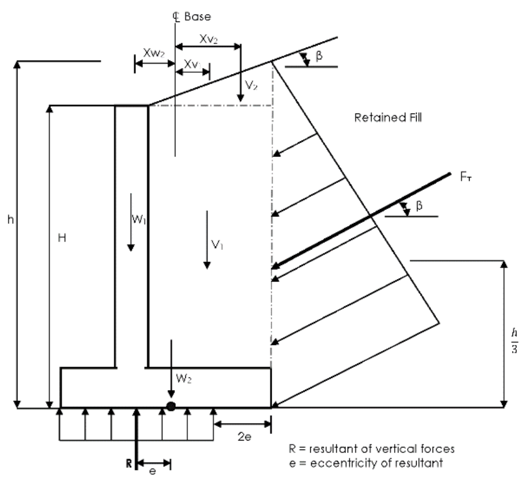

Figure 4. Depiction of the resultant of the stress distribution (AASHTO, 2014).

The location of the resultant may be arithmetically defined as the sum of the moments over the total area of the function as provided in Equation 13. The area is simply calculated by the integral of the stress distribution function as shown in Equation 14.

$$\bar{x} = \frac{1}{A}\int_a^b x\, f(x)\, dx \qquad [13]$$

Where:
A = area
$f(x)$ = normal stress distribution function
$\bar{x}$ = location of the resultant force
x = distance along base length

$$A = \int_a^b f(x)\, dx \qquad [14]$$

Where:
A = area
$f(x)$ = normal stress distribution function
x = distance along base length

The function of the normal stress distribution is determined by graphing the normal stress obtained from PLAXIS as shown in Figure 5 with the determined location of the resultant force.

The eccentricity is determined by subtracting the location of the midpoint of the base from the location of the resultant force. The eccentricity is then compared to the limits as defined by AASHTO (2014) for soil and rock foundations. The load factors utilized in the eccentricity calculation are defined in Figure 1.

3.1.5 Global Stability

Evaluation of failure by means of Global Stability using the FE Assisted LRFD Methodology is consistent with the Analytical LRFD Method (AASHTO, 2014) and is conducted using the limit equilibrium or finite element method of analysis.

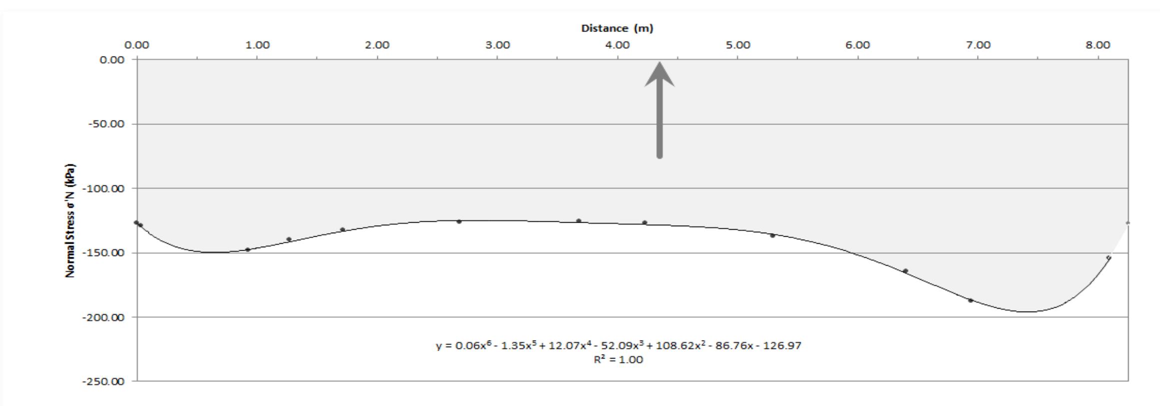

Figure 5. Normal stress distribution f(x) and corresponding resultant.

3.1.6 Excessive Settlement

Evaluation of failure by means of Excessive Settlement using the FE Assisted LRFD Methodology is consistent with the Analytical LRFD Method (AASHTO, 2014). The analysis is conducted using the FE models developed for the evaluation of Sliding, Limiting Eccentricity and Bearing Resistance.

3.2 Validation of Methodology

Evaluation of the stability soil retaining structures utilizing soil-structure FE programs such as PLAXIS has been researched and supported by Subramaniam (2011), Salma, Al-Shakarchi, Husain, and Sabre (2010), Simpson & Hocombe (2010), and Potgeiter (2016). However, conducting a soil-structure FE analysis following the AASHTO methodology requires that stability is not defined in terms of an overall FS against all means of failure; rather, it requires the method of analysis as well as the results comply with the requirements outlined in AASHTO (2014). In order to ensure that the AASHTO methodology is achieved, a simplified cross-section was analysed within PLAXIS as well as analysed following the typical AASHTO LRFD analytical method in order to confirm the method of analysis in PLAXIS. The PLAXIS modelling was conducted as outlined in Section 3.1. The results obtained by means of the FE Assisted LRFD Method and the comparison to the Analytical LRFD Method (AASHTO, 2014) are provided in Table 2. The results provided in Table 2 indicate that the FE Assisted LRFD Method presented within this paper is in agreement with the Analytical LRFD Method (AASHTO, 2014).

Table 2. Comparison of FE Assisted LRFD and Analytical LRFD Methods.

External Stability	FE Assisted LRFD	Analytical LRFD	Difference (%)
Bearing Resistance (CDR)	1.12	1.10	1.8
Sliding (CDR)	2.08	2.04	1.9
Limiting Eccentricity (e/L)	0.027	0.047	3.0[1]

[1]Difference divided by eccentricity criteria defined as the middle two thirds of the base for soil foundations (AASHTO, 2014).

4. RESULTS

Evaluation of the external stability and settlement of the CIP retaining wall at Keelesdale Park was completed for five distinct cross-sections. The five cross-sections were analyzed along the alignment of the wall as a result of differing geometry, changing soil conditions, the presence of existing construction works, and in an effort to minimize the required quantity of lightweight fill. The CDR and e/L values as determined following the FE Assisted LRFD Method for these cross-sections are summarized in Table 3.

Table 3. Results of the FE Assisted LRFD Method for the CIP retaining wall at Keelesdale Park.

Station	Sliding (CDR)	Bearing Resistance (CDR)	Limiting Eccentricity (e/L)
207+272	3.13	1.89	0.016
207+304	1.33	1.91	0.014
207+312	1.21	3.18	-0.002
207+328	3.15	1.46	0.001
207+352	2.86	2.02	-0.002

The Global Stability of the CIP retaining wall at Keelesdale Park was evaluated following conventional limit equilibrium methods. The Global Stability analysis was completed within SLOPE/W, available in the geotechnical software suite, Geostudio 2016. The results of the Global Stability analysis were compared to the acceptance criteria provided in Section 2.1.4.

The CIP retaining wall at Keelesdale Park was evaluated for failure by means of excessive settlement within PLAXIS. The settlement of the CIP retaining wall was reported for critical construction stages. The acceptance criterion was governed by the requirements of the ECLRT Project Agreement, and the maximum settlement that could be accommodated by the Structural Design Engineer.

5. DISCUSSION

The FE Assisted LRFD Methodology outlined in Section 3 was developed to utilize FE modelling programs, such as PLAXIS, to address the limitations and assumptions inherent within the Analytical LRFD Method (AASHTO, 2014). Although the FE models were developed to address and account for the presence of stratified soil deposits below the founding elevation and heterogeneous backfill, the method of analysis is consistent with the intent of the Analytical LRFD Method prescribed by AASHTO (2014). For the design of the CIP retaining wall at Keelesdale Park, reliance on the FE Assisted LRFD Methodology for completion of the external stability analysis enabled the designer to accurately identify the critical mechanism of failure for each of the cross-sections analysed, and to determine what measures were necessary to stabilize the structure at each cross-section.

5.1 Lightweight Fill

Due to the proposed geometry of the CIP retaining wall at Keelesdale Park, it was identified during preliminary design that excavation of the existing embankment material, and/ or the use of lightweight fill would be required at select locations along the alignment. Due to various factors, it was determined that the lightweight fill considered for the design of the CIP retaining wall at

Keelesdale Park would be comprised of cellular concrete.

Where the placement of lightweight fill was required to achieve external stability and settlement targets, a minimum 1.5 m layer of free draining Granular A (OPSS 1010, 2013) was incorporated into the design to provide an adequate subgrade for the realignment of Eglinton Avenue E.. The thickness of Granular A was increased where possible to decrease the amount of cellular concrete fill for economic reasons. In addition, the placement of Granular A (OPSS 1010, 2013) served to increase the total weight of the structure, such that concerns related to the buoyancy of the structure during extreme flooding events were addressed. The quantity of cellular concrete and Granular A (OPSS 1010, 2013) was optimized through an iterative FE analysis to ensure the final design would satisfy external stability requirements and achieve target settlements as well as its financial benefits. This procedure was conducted for each of the five cross-sections developed to evaluate the external stability of the retaining structure using PLAXIS as a result of the limitations inherent in the Analytical Method (AASHTO, 2014) related to design complexity.

Application of the FE Assisted Methodology allowed for multiple retained soil materials to be considered in addition to cellular concrete. The atypical pressure profile was able to be defined by the FE model, including pressures transferred through the cellular concrete that is normally not achievable utilizing the Analytical LRFD Method (AASHTO, 2014).

5.2 Ground Improvement

The preliminary design of the CIP retaining structure at Keelesdale Park was completed based on historical geotechnical data including borehole logs and laboratory test results, collected during previous geotechnical investigations within the region. Review of the historical data indicated the presence of weak surficial fill materials and weak subsurface materials.

Cone Penetrometer Tests (CPTs) were completed in June 2017 and confirmed the presence of weak subsurface materials underlying the proposed footprint of the CIP retaining wall. The Soil Behaviour Type (SBT) of the encountered weak materials consisted of clays, silt mixtures, and sensitive, fine grained. Based on the results of the CPT investigation, the subgrade preparation and requirements for improvement of the foundation soil were developed for three zones along the alignment of the wall. In order to ensure that the foundation soils would not undergo local shear failure, excessive settlements, and increase the FS associated with failure by means of Global Stability, the in-situ materials within the footprint of the CIP retaining structure at Keelesdale Park were sub-excavated and replaced with Granular B, Type II (OPSS 1010, 2013).

The FE Assisted LRFD Methodology considers the stress distribution through the prescribed ground improvement and the variable in-situ materials to determine the geotechnical resistance available at the base of the retaining structure. Moreover, the settlement along the base length of the retaining structure was determined and considers the presence of variable foundation materials.

5.3 Future Optimization of the LRFD Methodology

The FE Assisted LRFD Methodology has been relied upon for the design of the CIP retaining wall at Keelesdale Park. The application of the FE Assisted LRFD Methodology at this location allowed the Geotechnical Engineer to provide a design that is neither conservative nor aggressive in nature while fulfilling the requirements outlined within the Analytical LRFD Methodology (AASHTO, 2014).

The application of the FE Assisted LRFD Methodology for the design of the CIP retaining wall at Keelesdale Park serves to illustrate some of the benefits that may be achieved through further integration of soil-structure interaction modelling into the LRFD Methodology (AASHTO, 2014). Due to the accuracy and detail of the data obtained through completion of soil-structure interaction modelling, it may be possible to improve the LRFD Design Methodology (AASHTO, 2014) to incorporate standards related to: evaluation of the probabilistic means of analysis, complex lithology, heterogeneous backfill, horizontal movements, and complex movements. Should the existing LRFD Methodology be refined to incorporate the criteria listed above, further optimization during the design of future CIP retaining walls may be possible.

6. CONCLUSION

The design of the CIP retaining wall at Keelesdale Park has been completed following the FE Assisted LRFD Methodology outlined within this paper. Due to the presence of weak foundation soils, complex lithology, heterogeneous backfill material, spatial constraints, utility conflicts and flooding conditions, application of the FE Assisted LRFD Methodology was required to determine appropriate composite shear strength parameters for the retained and foundation soil zones. Through completion of FE modelling, it was possible to account for the presence of variable materials behind and below the retaining wall and conduct a comprehensive evaluation of the in-situ material strength.

Comparison of the results obtained using the Analytical LRFD Methodology (AASHTO, 2014) and the FE Assisted LRFD Design Methodology for a simplified typical design case was completed to evaluate the applicability of the FE Assisted LRFD Design Method. The results of this analysis are provided in Table 2, and serve to illustrate the minimal difference between the two methods of analysis.

The FE Assisted LRFD Methodology outlined within this paper is capable of providing an optimized design that is neither conservative nor aggressive in nature, but rather follows the intent of the Analytical LRFD Method (AASHTO, 2014). Application of the FE Assisted LRFD Method is time intensive, requires an intimate understanding of the surrounding soils and accurate material parameters to provide reliable results. Therefore, the FE Assisted LRFD Method should be applied only where deemed economically viable, as determined on a project by project basis.

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