CALIBRATION OF *P-Y* CURVES USING LATERAL LOAD TEST RESULTS IN COHESIVE AND COHESIONLESS SOILS FOR LARGE DIAMETER DRILLED SHAFT

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ABSTRACT
In 2018, the Caisse de Dépôt et Placement du Québec (CDPQ) awarded a $6.3 Billion Design-Build contract to the Joint Venture (JV) team NouvLR (led by SNC-Lavalin) for the design and construction of the Réseau Express Metropolitain (REM) in Montreal. Out of the 67 km network, over 25 km will be constructed on elevated structure founded on single drilled shafts socketed into rock. In order to optimize the response of the shaft to lateral loading condition, the JV team performed two fully-instrumented lateral load tests in critical areas along the alignment.

This paper presents a comparison of the lateral load test results with the lateral shaft response as simulated by LPile software, using several *p-y* curve models traditionally used for cohesionless and cohesive soils as well as for rock. The *p-y* curve as obtained from the pressuremeter tests are also compared to the lateral load tests.

RÉSUMÉ
En 2018, la Caisse de dépôt et placement du Québec (CDPQ) a confié à NouvLR le contrat d’ingénierie, d’approvisionnement et de construction des infrastructures du Réseau express métropolitain (REM). Un contrat de 6,3 milliards dollars, pour un réseau de 67 km avec plus de 25 km qui seront construits sur des structures surélevées reposant sur des caissons avec emboîture au roc. Afin d’optimiser la conception des fondations des différents ouvrages soumises aux charges latérales, l’équipe conception a réalisé deux essais de chargement statique latéral aux zones critiques le long du tracé.

Cet article présente une comparaison entre les résultats des essais de chargement latéral et la réponse du caisson obtenue par le logiciel LPile, en utilisant plusieurs modèles traditionnels de courbes *p-y*, pour les sols cohésifs et non cohésifs ainsi que pour le roc. La courbe *p-y* obtenue à partir des essais au pressiomètre a été également comparée à l’essai de chargement latéral.

1 INTRODUCTION

Réseau Électrique Métropolitain (REM) is being constructed by a Joint Venture (NouvlR) comprised of SNC Lavalin, Dragados Canada, Group Aecon Quebec, Pomerleau, and EBC, partnering with Aecom and SNC Lavalin as the lead design firms. The project, referred to as REM, is a fully automated light rail transit (LRT) proposed by the Caisse de dépôt et placement du Québec (CDPQ) Infra, to serve the major metropolitan areas in Montreal, Canada. Once completed, the 67 km REM will be one of the largest automated transportation systems in the world and will represent the largest public transportation infrastructure for the metropolitan area since the Montreal metro, inaugurated in 1966.

As shown on Figure 1 below, the new facility will link downtown Montreal, South Shore, West Island (Sainte-Anne-De-Bellevue), North Shore (Deux-Montagnes), and the Montreal-Pierre Elliott Trudeau International Airport.

The project comprises four segments: South Shore (SS) segment with a total length of 15 km will extend from downtown (Central Station) to the DIX30 commercial district passing across to Nuns' Island and then will use a rail deck constructed on the new Champlain Bridge (still under construction) to cross the St. Lawrence. Over 3.5 km will be supported by an elevated structure.

The Deux Montagnes (DM) segment will be mostly at grade and will consist of a direct conversion of the existing Deux-Montagnes line.

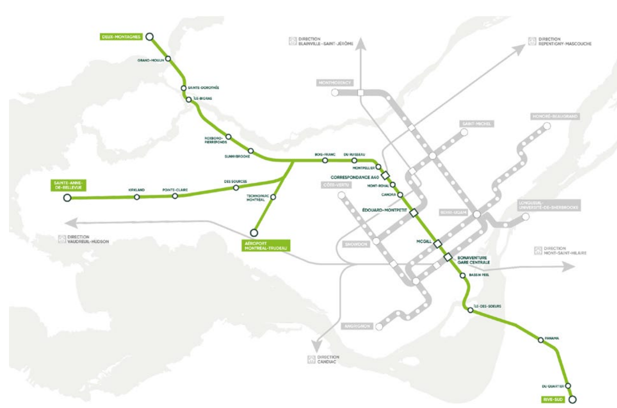

Figure 1. REM project alignment

The Sainte-Anne-De-Bellevue (SADB) segment will begin near highway A-13 and end at SADB with 17 km of elevated structure supported by two abutments and 340 piers.

The airport segment will divert from SADB line to make a stop in Technoparc St-Laurent before terminating at Montreal–Pierre Elliott Trudeau International Airport. The airport segment will include approximately 1 km of elevated guideway consisting of 2 abutments and 21 piers.

The foundation system for the structures will be single drilled shafts, socketed into sound rock. Depending on the structural loads and site conditions, the drilled shafts range from 2.0 m to 3.2 m in diameter with embedment of up to 9.0 m within the sound rock.

Major factors affecting the design of the foundation at the REM bridges included lateral and overturning forces from wind, collision forces, and seismic demands. In fact, drilled shaft design is often controlled the horizontal loads acting on the substructures rather than by vertical load. Therefore, accurate prediction of the shaft behavior subject to lateral loads is an important step in the optimization process.

2 SITE GEOLOGIC CONDITIONS

The general geology of the layout consists of a till deposit overlying the bedrock at varying depths (2 to 17 m). The overburden soil overlying the till is mostly granular fill although clay deposits from the Champlain Sea up to 9 m in thickness is encountered in some areas, particularly along the SADB segment. The surface of the bedrock is altered and fractured for depths varying from 1 to 3 m. Three rock types are encountered along the REM alignment. Limestone and dolomite are the main rock formations along DM and SADB. Shale is the only rock type along the SS segment.

3 OBJECTIVES AND METHODOLOGY

The design of the drilled shafts subjected to horizontal loads is addressed by lateral support from surrounding soils and, depending on the soil condition and overburden thickness, by the rock socket. In order to optimize the lateral resistance design, the JV team decided to perform two full-scale lateral load tests, in addition to two bidirectional (Osterberg Cell) static axial load tests, which are the subject of separate paper. The O-cell axial tests were to be performed in each of the rock formations encountered in the project, namely limestone, dolomite and shale.

The selection of load test sites was the subject of an intensive investigation and examination of several candidate sites. This selection was determined in conjunction with the structural engineers based on structural loading conditions, depth to the rock, soil type and profile. Furthermore, the drilled shafts tested for axial load were to be used as reaction piles and therefore, the selection of the site was also contingent on the rock formation.

One lateral load test was selected at Deux-Montagnes (DM) segment in mostly cohesionless soil and the other test was performed at Sainte-Anne-De-Bellevue (SADB) segment where clayey soil from the Champlain Sea is encountered. The purpose of the test was to validate and calibrate the *p-y* curves anticipated to be used to simulate the soil behavior at the site, as well as the lateral soil parameters. The results and analysis of the two load tests are presented in this paper.

LPile version 2018 was used to simulate the load test. Several soil models were used to examine the best suited models for the soils on site. At the SADB load test where cohesive soil is encountered, the results of the load test were compared to LPile results, but very large difference was immediately noticed regardless of the soil model used in the analysis. In fact, the lateral load test showed a much lower capacity than expected which led the design team to believe that some soil disturbance had occurred during construction. As a result, a second soil investigation was performed (2 weeks after the load test was completed) in order to reassess the soil parameters. The results of the second investigation showed the soil parameters correspond largely to the remolded soil parameters as explained below.

4 SUBSURFACE AND LABORATORY INVESTIGATION

A multi-phased site exploration was conducted at the site and included investigative borings both before and after the award of the construction contract. The design-phase investigation included one boring per substructure. All of the borings extended to a minimum of 10 m into rock but in no case less than 5 m below the estimated shaft tip elevation.

Intensive soil and rock investigation was performed at the load test locations, which includes SPT boring, pressuremeter (in soil) and dilatometer (in rock), Cone Penetration Test (CPT), and Nilcon vane shear test in soil. The pressuremeter and dilatometer models used for the testing were the Texam model and the Probex model respectively. The laboratory program consisted of sieve and hydrometer analysis of soil and unconfined compressive test and elastic modulus in rock.

At the DM load test location, the boring indicated that the upper 3 m of the overburden consisted of fill material containing predominantly silty sand with an average SPT 'N' value of 6 blows/0.3 m underlain by about 3.1 m thick layer of silty sand (glacial till) with an average SPT 'N' value of 18 blows/0.3 m. Bedrock consisted of limestone extending to the bottom of borehole at a depth of 12.1 m. The upper 3.2 m limestone was of very poor to poor quality with an RQD of 28 percent. The bottom rock was of fair quality rock with an average RQD of 54 percent.

A pressuremeter test was performed at a depth of 4.35 m in the glacial till. The pressuremeter modulus was 5.4 MPa and the creep and limit pressures were 431 and 754 kPa, respectively. The dilatometer test in rock showed a dilatometer modulus along the shaft of 12.2 GPa in the bottom good to excellent rock.

At the SADB load test location, the upper 4.35 m of the overburden consisted of clay deposits overlying 5.27 m layer of sandy and silty gravel (glacial till) with SPT ‘N’ values varying from 15 blows/0.3 m to 66 blows/0.3 m. The initial investigation (pre-construction of the test pile) showed the upper 1.4 m clay layer with undrained shear strength S_u of about 70 kPa, underlain by 3 m thick clay with an average S_u of 50 kPa. Measurements were taken with Niclon vane shear test at intervals of 0.5 m. The rock was encountered at a depth of 9.62 m and consisted of dolomite extending to the bottom of the borehole at a depth of 16.9 m. The dolomite was of fair to good quality with a RQD varying from 60 to 79 percent.

A pressuremeter test was performed in the clayey soil as well as in the glacial till. The clay soil, which was tested at 1.50 m, 3.00 m and 4.00 m, showed an average pressuremeter modulus of 4.27 MPa and an average creep and limit pressures of 220 kPa and 390 kPa respectively. The pressuremeter modulus was 5.20 MPa and the creep and limit pressures were 632 kPa and 1,740 kPa, respectively for the test performed at a depth of 6.25 m in the glacial till.

As explained below, the construction method used by the contractor caused remolding of the soil around the shaft. This was noticed during the analysis when the results were compared to LPile models. As a result, additional soil investigation was carried out 2 weeks after the shaft construction to verify the compactness of the overburden soils and the shear strength of the cohesive soils. The investigation consisted of SPT boring, CPT and vane shear test, all performed about 500 to 600 mm from the shaft.

The undrained shear strength (S_u) of the clay, as measured by the vane apparatus and confirmed by the CPT results, indicated average values of 24 and 9 kPa in the upper and lower clay layers, respectively. For the purpose of comparison, a vane shear test was performed at about 4 m from the shaft, in an area not affected by the shaft construction. Both undisturbed and disturbed soil parameters were measured. These parameters were almost identical to pre-construction and post-construction soil parameters respectively. The SPT N values in the glacial till were also slightly lower than the N value in the pre-construction phase. The results confirmed that the soils around the steel casing were disturbed from their initial state and no longer offer the required lateral resistance.

5 CONSTRUCTION PROCEDURE

Activities for the test pile construction procedure are separated in the following five sub-sections:

5.1 Casing Placement and Overburden Excavation

A 1,300 mm diameter permanent steel casing was placed at the proposed location. A hydraulic rig (LB 36-410) was used to insert the casing through the overburden and the fractured rock until refusal on the rock surface was reached. The overburden was excavated with an auger mounted on the drilling equipment. However, at the SADB site, before the installation of the casing the contractor carried out a pre-hole in the clayey soil with a 1,180 mm diameter auger in order to facilitate the casing placement. Figure 2 shows the placement of the casing and the reinforcement cage in the excavated hole.

Figure 2. casing placement and soil excavation

5.2 Rock Drilling and Cleaning

The 1,180 mm diameter auger drilled the rock surface at the bottom of the steel casing with a minimum of 300 mm of recess. Once the shaft had been sealed, rock drilling was done with a rock drilling bucket. To obtain a relatively flat base on the rock surface, the sub-contractor used a cleaning bucket (KBF-K) to flatten the surface. The bottom and the rock socket walls of the shaft were cleaned with an airlift multiple times until the tolerance requirements were met. During the excavation and the cleaning of the soil inside the casing at SADB, the contractor did not adjust the water level in the casing with respect to the natural water table level outside the casing. In other words, the casing was not filled with water to maintain a positive pressure head. At that time, no soil subsidence was observed on the surface.

5.3 Rebar Cage & Load tests Instrumentation

The frame containing two inclinometer casings and four Crosshole Sonic Logging (CSL) tubes is shown on Figure 3 below. The rebar cage was inserted into the excavated hole in a vertical position until the pre-determined elevation was reached. Then the rebar cage was suspended to avoid the rebar cage siting directly at the bottom of the shaft.

Figure 3. Overall view of the cage and O-cell

5.4 Concrete Pouring

The concrete was pumped through a 5-inch diameter O.D. tremie line into the base of the shaft. During the concrete pouring, the volume of concrete in the pile was monitored as well as the volume that was poured from the delivery trucks combined with the concrete elevation which was measured on site directly into the shaft. The tremie pipe was held at least 3 meters in concrete at all times.

5.5 Quality Control

Drilled shafts were subjected to multiple levels of quality control measures. Open shaft verticality through overburden soils was measured by the drilling rig alignment control and with the help of a 1.2 meters long spirit level for each shaft. This confirmed shaft verticality of the order of 0 - 2% was achieved. Once the casing and rock socket drilling was completed a Sub-Camera inspection device was used to confirm the rock socket base cleanliness using five spot sediment checking criteria (50 % less than 40 mm, no location exceeding 15 mm). Then, construction quality control involved concrete integrity testing using Ultrasonic Crosshole Testing (CSL) and pile integrity testing. Figure 4 shows the lateral load test setup.

Figure 4. Lateral Load Test Setup

LOAD TEST RESULTS AND PROCEDURES

Lateral load tests were performed on sacrificial drilled shafts by LoadTest, Inc on July 11 and 12, 2018 for the SADB and DM segments, respectively. The drilled shafts tested previously for axial load (O-cell) were used as reaction piles and were located at about 6 m from the laterally tested shafts. In each tested shaft, two inclinometer casings were attached to the reinforcing cage and positioned at 90° to the direction of loading. In each of the two casings, seven inclinometers (Geokon Model 6300 Series) were installed to monitor the variation of deflection and tilt response with depth. The depths at which the inclinometers were placed are indicated below. The hydraulic jack used to apply the load consisted of 330 mm diameter Osterberg Cell (O-cell) calibrated to 4996 kN. A W12x79 beam section connected the loading assembly to the reaction pile. Fifty millimeter thick steel plates were positioned between the loading apparatus and the pile to provide a flat, uniform loading surface. The tested drilled shaft properties are summarized in Table 1 as follows:

Table 1. Drilled Shaft Properties

Characteristics	SADB	DM
Nominal pile diameter in soil and fractured rock	1,300 mm	1,300 mm
Nominal pile diameter in sound rock	1,180 mm	1,180 mm
Assumed concrete unit weight	2,322 kg/m^3	2,322 kg/m^3
Concrete compressive strength the day of the test	47.4 MPa	41.4 MPa
Average ground surface elevation	26.85 m	32.49 m
Pile tip elevation	13.75 m	19.49 m
Bottom of permanent casing elevation	15.15 m	22.59 m
Level 0 to 7 inclinometers depths (m)	0, 1.7, 3.2, 4.8, 6.1, 9.3, 12.1	0, 1.5, 3.1, 4.6, 6.1, 7.7, 11.25

The loading was applied in 10 increments and each successive load increment was held constant in accordance with ASTM D3966, Standard Test Methods for Deep Foundations Under Lateral Load.

At DM (cohesionless soil) the maximum lateral load applied to the pile was 2.0 MN with a maximum head deflection of 71.06 mm. The pile head rotation angle was 0.61° from vertical. At SADB (cohesive soil) the maximum lateral load applied to the pile was 1.01 MN with a maximum head deflection of 66.60 mm. The pile head rotation angle was 0.54° from vertical.

The load-displacement curves for DM and SADB shafts are shown in Figures 5a and 5b, respectively. On the left side of the figures is the load-displacement curve for the reaction piles.

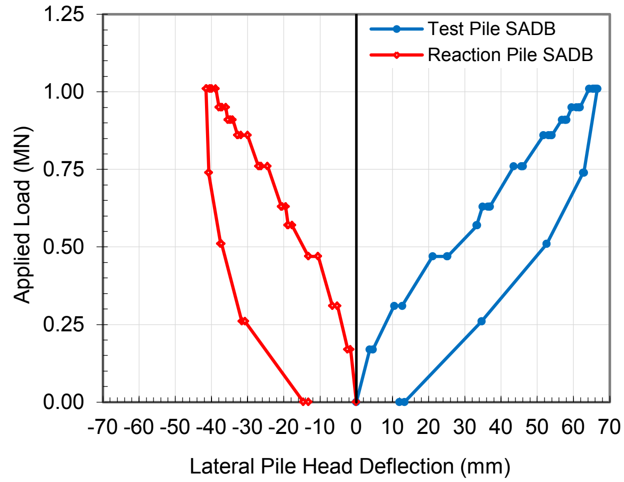

Figure 5a. Load-Displacement Curve - Cohesive soil

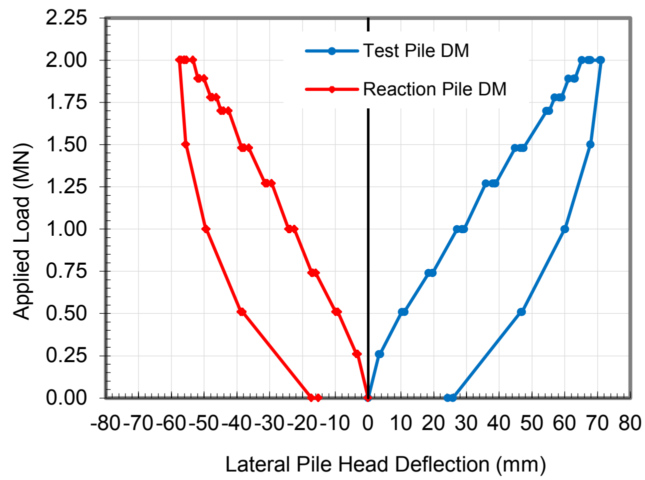

Figure 5b. Load-Displacement Curve – Cohesionless soil

6 CALIBRATION OF LATERAL RESISTANCE AGAINST TEST RESULTS

6.1 P-Y criterion used for Soil and Rock

The most common method of analysis used in the design of deep foundations for lateral loading is the *p-y* method, represented by the finite difference model implemented in the computer code COMP624 and more recently in LPile software. The analysis is based on replacing the soil around the pile by a set of nonlinear springs which provide the soil resistance *p* as a nonlinear function of the pile deflection *y*. The approach has been widely accepted because of its simplicity and ability to capture the essential aspects of pile behavior.

The lateral response of the soil was analyzed and calibrated against the lateral load test results. LPile version 2018 was used to compute the lateral deflection, moment and shear along the pile shaft using the following *p-y* soil models for the granular fill material and the glacial till:

- API Sand model: the *p-y* curve for this model is characterized by a hyperbolic curve with an ultimate resistance (P_u). The API Sand *p-y* spring is dependent on internal friction angle of the soil, φ, the soil unit weight, γ, the coefficient of lateral subgrade reaction, k_h and the pile diameter.
- Reese model for sand: the shape of the *p-y* curve for the Reese model consists of an initial linear stiffness followed by a parabolic curve up to a displacement of 1/60 of the pile diameter and then a linear portion up to the ultimate capacity. The parameters required to define the *p-y* curve are the same as the API model with slightly different k_h values.

The coefficient of lateral subgrade reaction k_h for the API sand and Reese models were determined as a function of the friction angle as recommended by the authors of the two methods and clarified in the LPile technical documentation.

The lateral response of the cohesive soil has been analyzed and calibrated against the lateral load test results using the following soil model:

- API Soft Clay model: The *p-y* curve for this model is characterized by a hyperbolic curve with an ultimate resistance (P_u). The API Soft Clay *p-y* spring is dependent on undrained cohesion (S_{ur} for disturbed and S_{ui} for intact properties), the soil unit weight, γ, the strain factor, ϵ_{50}, the coefficient of lateral subgrade reaction, k_h, the J-factor and the pile diameter.

Also, for both the cohesive at SADB and cohesionless at DM soils, the *p-y* curve derived from the pressuremeter test results was also used for the calibration with the load test results. The pressuremeter data was converted to a *p-y* curve according to Baguelin et al (1978) method. The *p-y* curve as developed from the pressuremeter test is characterized by three linear segments with slopes of k_h, ½ k_h and zero. Where k_h, is estimated based on the following equation proposed by Baguelin et al. based on Menard theory for pile diameter larger than 0.6 m:

$$\frac{1}{k_h} = \frac{2.B_0}{9.E_m}\left[\frac{2.65.B}{B_0}\right]^{\alpha} + \frac{\alpha.B}{6.E_m}$$

Where E_m is the pressuremeter modulus and B_0 is a reference diameter equal to 0.6 m. The parameter α is an empirical coefficient that depends on the soil type and the ratio E_m/P_l (where P_l is the limit pressure) and varies from 0.25 for gravelly soil to 1.0 for over-consolidated clayey soil. Values of E_m were taken as the average of E_0, the pressuremeter first load modulus, and E_r, the pressuremeter reload modulus.

Both the fractured and sound rock was modeled using the Rock Mass model developed by Liang et al. (2009). According to the model the *p-y* curve is expressed by a hyperbolic curve defined in term of the spring modulus, K_i, and the ultimate resistance per unit length of shaft P_u. K_i and P_u are dependent on the rock unconfined uniaxial compressive strength, q_u, Geological Strength Index GSI, the Hoek-Brown material index, m_i, Poisson ratio, ν, the rock mass modulus, E_r and the drilled shaft bending stiffness EI.

The bending stiffness of the tested shaft was calculated by the program using the as-built steel reinforcement of 1 percent of the gross cross-sectional area for drilled shaft, with a 19 mm thick steel casing. The calculated bending stiffness EI was 8x10^5 kN-m^2.

6.2 Summary of Soil and Rock Parameters

Table 2 below shows a summary of the soil parameters, used in LPile software, at the load test performed in the cohesionless soil at DM segment.

Table 2. Summary of Soil Parameters for DM Segment

		Soil Parameters	
		Fill	Glacial Till
Thickness (m)		3.0	3.1
Friction angle, φ (^{o})		30	32
Unit Weight γ (kN/m^3)		19	21
Coefficient of Lateral Subgrade Reaction (MN/m^3)	API Sand	13.0	15.0
	Reese	10.5	16.4
	Pressuremeter	13.1	15.2

The rock parameters for the same segment were obtained from laboratory testing and rock structural description and are summarized in Table 3.

Table 3. Summary of Rock Parameters for DM Segment

	Rock Parameters (DM)	
	Fractured Rock	Sound Rock
Rock Type	Limestone	
Compressive strength, q_u (MPa)	48.5	48.5
Geologic Strength Index (GSI)	22	45
Hoek-Brown index, m_i,	8.4	8.4
Poisson Ratio, ν	0.29	0.29
Intact Rock Modulus, E_i (GPa)	56.7	56.7

As for the SADB load test where the upper soil is cohesive, the analysis was performed for the pre-construction phase, using undisturbed soil parameters, and post-construction phase, using remolded soil parameters. A summary of the design parameters used in LPile is shown below.

Table 4. Summary of Soil Parameters for SADB Segment

Soil Type	Thickn-ess	Pre-Construction		Post-Construction	
	(m)	S_{ui} (kPa)	ϵ_{50}	S_{ur} (kPa)	ϵ_{50}
Upper clay	1.4	70	0.0079	24	0.0144
Lower clay	3.0	50	0.0095	9	0.0259
Glacial Till	Thickn-ess	Φ (^{o})	γ (kN/m^3)	Φ (^{o})	γ (kN/m^3)
	2.0	33	9	28	8
	3.0	38	12	38	12

The rock parameters, also obtained from the laboratory testing and rock structural description, are summarized in Table 5 below.

Table 5. Summary of rock parameters for SADB Segment

	Rock Parameters (SADB)
Rock Type	Dolomite
Compressive strength, q_u (MPa)	160
Geologic Strength Index (GSI)	61
Hoek-Brown index, m_i,	9.2
Poisson Ratio, ν	0.33
Rock Mass Modulus, E_r (GPa)	12.2

ϵ_{50} in the table above is the strain at 50 percent of the failure stress in uniaxial compression. The J factor was taken equal to 0.5.

7 TEST RESULTS ANALYSIS

In order to simulate the load test with LPile software, the lateral displacements obtained in four of the load test increments were imposed at the pile head in LPile. The boundary condition at the top of the shaft was assumed as free head, and the lateral force was applied 0.6 m above the ground surface to simulate the load test. The resulting shear forces at the pile head were then compared to those obtained by the load test.

7.1 Cohesionless Soils (DM)

For the cohesionless soil at DM, the deflection profile as obtained by the LPile for the two soil models and the pressuremeter data were plotted against the deflection profile of the test pile generated from the inclinometer data. Figure 6a shows the deflection profile and Figure 6b shows the shear forces versus displacement at the pile head obtained by simulating the soil with the API sand and Reese models as well as by the *p-y* curve obtained from the pressuremeter data as explained above.

It can be seen that the deflections along the pile shaft as obtained with the three methods are similar and match well with the load test deflection curves

Moreover, the point of fixity, defined as the first point of zero deflection, for this particular case, is located about 1.7 m (Elevation 24.8 m) below the top of the fractured rock and therefore, the rock quality does not have significant effect on the lateral behavior of the shaft. It can also be concluded that, for deflections less than 25 mm (See Figure 6b), the API sand and Reese models seem to slightly over estimate the lateral soil response whereas the pressuremeter test matches well with load test results. As for large displacements (larger than 25 mm), the API sand is more in agreement with the lateral load test results to represent the behavior of the granular soil (fill and till).

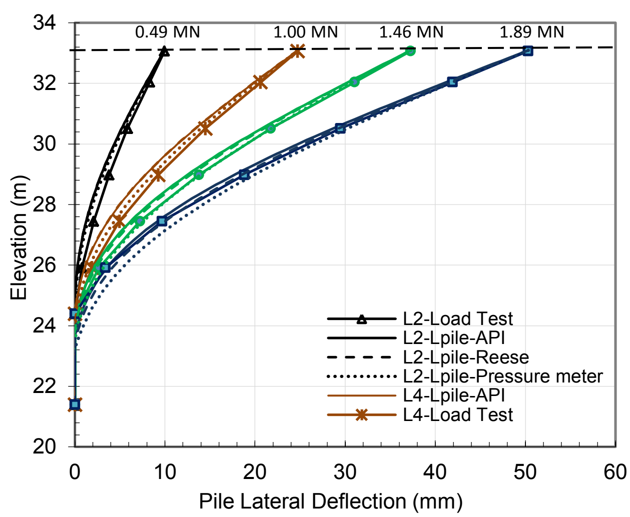

Figure 6a. Deflection profile at DM.

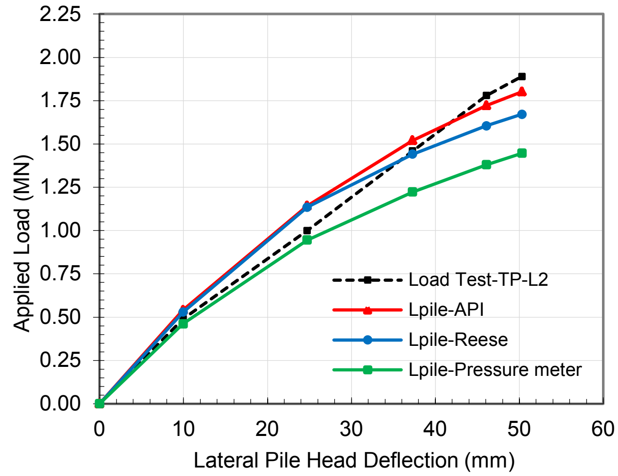

Figure 6b. Shear force vs deflection curves at DM

7.2 Cohesive Soils (SADB)

As for the cohesive soil at SADB, the predicted lateral movement at the top of the shaft using the API Soft clay model described above and utilizing the remolded soil parameters was compared to the measured lateral movement from the test shaft as shown on Figure 7. For comparison purposes, soil parameters from the pre-construction phase (undisturbed), including the *p-y* curve obtained from the pressuremeter data, were plotted on the same figure to illustrate the disturbance effect on the shaft behavior.

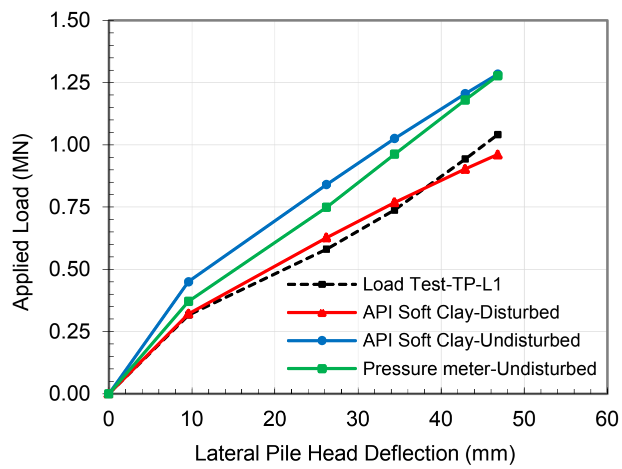

Figure 7. Shear forces vs deflection curves at SADB

8 CALIBRATION AND BACK CALCULATION OF DESIGN PARAMETERS

The *p-y* criterion for several soil models were used to back calculate the soil parameters to match the deflection from the lateral load test. The back calculated parameters were used to predict the behavior of the production drilled shafts along the alignment when similar soil types are encountered.

For the cohesionless soil, the *p-y* curve from the pressuremeter test as well as both the API sand and

Reese models could be used to predict quite accurately the lateral soil response of the granular material (fill and glacial till). However, for deflections less than about 25 mm, the API Sand and Reese models seem to slightly over estimate the lateral soil response whereas the pressuremeter test matches well with load test results. As for large displacement (larger than 25 mm), the API sand is more in agreement with the lateral load test results to represent the behavior of the granular soil (fill and till).

For the production pile, since the allowable lateral deflection at the ultimate limit state was limited to 25 mm, it was recommended that the k_h values for the API sand and Reese models be reduced by about 5 percent to better represent the soil behavior at small displacement. If pressuremeter data is available, the back-calculated *p-y* curves can capture the soil response for small displacement pretty accurately with no adjustment required.

The API Soft Clay model predicts very well the lateral behavior of the shaft as illustrated by the simulation using the disturbed (post-construction) soil parameters. No adjustment to the soil parameters is required. Due to the disturbance of the soil at the SADB site, it was difficult to draw a reliable conclusion with regard to the behavior as predicted by the pressuremeter test. However, it seems that both the pressuremeter and the API Soft clay predict similar load - deflection curve at small as well as large displacement with the pressuremeter predicting slightly larger displacement.

9 SUMMARY AND CONCLUSIONS

The results from the lateral load test were used as a basis to back-calculate the soil parameters that will result in the same movement at the top of the shaft utilizing LPile program. The load deflection curve at the top of the shaft was used as the reference in the back analysis. For both test shafts, the movement was resisted mostly by the overburden and slightly by the rock socket. The overburden consisted of granular material (granular fill over glacial till) at DM segment and cohesive soil (soft clay over glacial till) at SADB segment. The parameters for the soil layers were calibrated to back calculate the deflection to match the deflection from test shafts. API Sand and the Reese criterion were used to model the granular material and the API Soft Clay was used to simulate the cohesive soil behavior. Moreover the *p-y* curve as obtained from pressuremeter data was also analyzed. The Mass Rock model proposed by Liang et al. was used to model the rock layers. Slight adjustments to the soil parameters were suggested to be used to match the shaft top deflection at each load increment. This adjustment consisted basically of reducing the coefficient of lateral subgrade reaction of the API sand and Reese models by about 5 percent for the granular fill and glacial till encountered along the alignment to better match the results of the lateral load test. No adjustment was required to the cohesive soil parameters when using the API Soft clay model.

The load test performed in the cohesive soil showed the behavior of drilled shaft was very sensitive to remolding of surrounding soils. Great care should be taken in the construction methods to ensure no remolding of surrounding soils.

Instructions were given to the contractor to avoid subsidence or any soil disturbance, immediately around the steel casing by not performing any pre-holes during shaft construction and maintaining positive water head by keeping the water level inside the casing a minimum of 0.5 to 1 m above the ground water table.

10 ACKNOWLEDGMENT

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