# APPENDIX 'C'

# GEOTECHNICAL REPORTS

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**RE RFP No. 180-017 - Promenade Taché**
**Geotechnical Recommendations for Sheet Pile Wall and Lookout Structure Foundations**

This letter provides geotechnical recommendations for the structural design of foundations and sheet pile walls for the Promenade Taché sidewalk widening and treetop lookout structure. Morrison Hershfield Ltd. (MHL) was retained by TREK Geotechnical Inc. (TREK) as the structural and civil sub-consultant for the project.

## Background and Existing Information

TREK's understanding of site conditions is based upon site reconnaissance carried out by TREK staff on multiple dates (May to August, 2017), updated instrumentation monitoring by TREK in the summer of 2017, and the results of previous geotechnical investigations and monitoring recently carried out by KGS Group, as well as numerous studies at the site. The following reports form the basis of our understanding:

- St. Boniface Rivertrail Tree Top Lookout and Sidewalk Expansion: Riverbank Condition Assessment and Functional Design Report (KGS Group, Jan. 2016)
- Saint-Boniface Rivertrail: Preliminary Field Investigations, Instrumentation and Monitoring – Summary Geotechnical Report of Findings (KGS Group, Sep 1, 2016)
- Riverbank Stability Assessment Report: De La Cathedrale Outfall (KGS Group, Dec. 2008)
- Riverbank Stability Assessment Report: Despins Outfall (KGS Group, Jan. 2008)
- Geotechnical Report on Riverbank Stability – Taché Avenue (A. Dean Gould, Nov. 1998)
- Geotechnical Investigation: Gate Chamber Replacement: Despins Flood Station (Geokwan Engineering, Sep. 1989)

Test hole logs from the recent drilling investigation by KGS Group have been provided previously to Morrison Hershfield Ltd.

## Limit States Design

Limit States Design recommendations for deep foundations in accordance with the Canadian Highway Bridge Design Code (CHBDC, CAN/CSA-S6S1-10, 2010) are provided below, using resistance factors as specified in Table 1. Limit States Design requires consideration of distinct loading scenarios comparing the structural loads to the foundation bearing capacity using resistance and load factors that are based on reliability criteria. Two general design scenarios are evaluated corresponding to the serviceability and ultimate capacity requirements.

The **Ultimate Limit State (ULS)** is concerned with ensuring that the maximum structural loads do not exceed the nominal (ultimate) capacity of the foundation units. The ULS foundation bearing capacity is obtained by multiplying the nominal (ultimate) bearing capacity by a resistance factor (reduction factor), which is then compared to the factored (increased) structural loads. The ULS bearing capacity must be greater or equal to the maximum factored load. Table 1 summarizes the ULS resistance factors that can be used for the design of foundations as per the CHBDC (2010) depending upon the method of analysis and verification testing completed during construction.

The **Service Limit State (SLS)** is concerned with limiting deformation or settlement of the foundation under service loading conditions such that the integrity of the structure will not be impacted. The Service Limit State should generally be analysed by calculating the settlement resulting from applied service loads and comparing this to the settlement tolerance of the structure. However, the settlement tolerance of the structure is typically not yet defined at the preliminary design stage. As such, SLS bearing capacities (or unit resistances) are provided that are developed on the basis of limiting settlement to approximately 25 mm or less, unless otherwise specified. A more detailed settlement analysis should be conducted to refine the estimated settlement and/or adjust the SLS capacity if a more stringent settlement tolerance is required.

**Table 1. ULS Resistance Factors for Foundations (CHBDC, 2010)**

| Foundation / Method / Soil | Resistance Factor |
|---|---|
| Deep Foundations in Compression | |
| Static analysis | $\phi = 0.4$ |
| Dynamic Testing (PDA with CAPWAP on production piles) | $\phi = 0.5$ |
| Deep Foundations in Uplift | |
| Static analysis | $\phi = 0.3$ |

## Lookout Structure Foundations

Cast-in-place end-bearing piles, driven precast concrete piles or driven steel piles are considered to be feasible foundation alternatives for the site, however cast-in-place end-bearing piles are considered most suitable for the single-column pier system proposed for the lookout structure.

### End Bearing, Cast-in-Place Concrete Piles

Cast-in-place concrete (CIPC) caissons installed in dense silt till will derive a majority of their resistance in end-bearing with a relatively small contribution from shaft friction. Caissons subjected to frost jacking (exterior piles) and tension loads will derive a majority of their axial-uplift resistance in shaft friction. Table 2 provides the recommended ULS and SLS end bearing and shaft friction (adhesion) resistance values for axial-compressive and axial-tensile (uplift) loading conditions for mechanically-cleaned caissons bearing on dense silt till.  The SLS capacity of the caissons is settlement-dependent and is based on a maximum settlement of 25 mm. The elastic shortening of the pile should be added to the tip displacement to calculate the pile head settlement.

Table 2: Unit Resistances for CIPC caissons on Dense Till or Bedrock

| Pile Type | ULS End Bearing Resistance (kPa) | ULS Uplift Resistance (kPa) | | SLS End Bearing Resistance (kPa) |
|---|---|---|---|---|
| | | $\phi = 0.3$ | | |
| | $\phi = 0.4$ | Clay | Till | |
| CIPC Caissons Note 1 | 900 | 10 | 10 | 720 |
| *Note 1: presence and thickness of competent till is variable, caissons may need to be advanced to sound bedrock* | | | | |

It should be noted that the silt till encountered at the site may soften when exposed to water, which could lead to disturbance of the caisson base and a reduction in capacity. As such, it is critical that water not be permitted to enter the caisson during drilling.  Full length sleeves will likely be required to maintain a dry shaft.

Additional Caisson Design Recommendations:

1. The weight of the embedded portion of the pile may be neglected.
2. Shaft adhesion should be neglected within the upper 2.4 m below ground surface for the calculation of uplift resistance.
3. Caisson bases must be founded on dense silt till.
4. Caissons should have a minimum shaft diameter of 406 mm.
5. Caissons should have a minimum spacing of 2.5 diameters measured centre to centre. If a closer spacing is required, TREK should be contacted to provide an efficiency (reduction) factor to account for potential group effects.
6. Caissons should be designed by a qualified structural engineer to resist all applied loads induced from the structure as well as tensile forces induced from seasonal movements of the bearing soils.

Additional Caisson Installation Recommendations:

1. Caisson bases should be free of debris and any deleterious material.
2. Temporary steel casings (sleeves) should be available and used if sloughing or caving of the caisson hole occurs and/or to control groundwater seepage if encountered. Care should be taken in removing sleeves to prevent sloughing (necking) of the shaft walls and a reduction in the cross-sectional area of the caisson.
3. Concrete should be placed in one continuous operation immediately after the completion of drilling the shaft to avoid construction problems associated with sloughing or caving of the shaft and groundwater seepage. Concrete should be poured under dry conditions. If groundwater is encountered, it should be controlled and removed. If water cannot be controlled and removed, the concrete should be placed using tremie methods.
4. Concrete placed by free-fall methods should be directed through the middle of the pile shaft and steel reinforcing cage to prevent striking of the drilled shaft walls to protect against soil contamination of the concrete.
5. All piles should be inspected by TREK personnel to verify suitable end-bearing materials, proper base preparation, seepage control and concrete placement.

Lookout Structure Abutment Foundations – Sheet Piles

We understand sheet piles will be used to accommodate sidewalk widening and that short, cast-in-place jump slabs will extend from the outer piers of the lookout structure to abutment pile caps bearing on sheet piles driven or vibrated to till.

Sheet piles bearing on dense silt till will derive their resistance in both shaft friction and end bearing. Table 3 provides the recommended ULS and SLS end bearing and shaft friction resistance values for axial-compressive and axial-tensile (uplift) loading conditions for sheet piles driven to dense silt till. The recommended shaft friction should be applied along the projected area of the pile (the Z-shape of the pile should be neglected in calculation of shaft adhesion) and the end-bearing resistance should be applied on the cross-sectional steel area. The SLS capacity is based on a maximum settlement of 25 mm.

Table 3: Unit Resistances for CIPC caissons on Dense Till or Bedrock

| Pile Type | ULS Compressive Resistance $\phi = 0.4$ | | ULS Uplift Resistance $\phi = 0.3$ | SLS Unit Shaft Resistance |
|---|---|---|---|---|
| | Shaft | End-Bearing | Clay or Till | |
| Sheet piles driven to dense till | 12 kPa | 3,000 kPa | 10 kPa | 11 kPa |

Additional Sheet Pile Design and Installation Recommendations:

1. The weight of the embedded portion of the pile may be neglected.
2. Shaft adhesion should be neglected within the upper 2.4 m below ground surface for the calculation of uplift resistance.
3. Sheet piles should be driven to practical refusal on dense silt till, otherwise end-bearing resistance should be neglected in calculation of axial compressive pile capacity.
4. Sheet pile installation should be inspected by TREK staff to verify proper pile installation.

## Lateral Pile Analysis

The soil response (subgrade reaction) to lateral loads can be modeled in a simplified manner that assumes the soil around a pile can be simulated by a series of horizontal springs for the preliminary design of pile foundations. The soil behaviour can be estimated using an equivalent spring constant referred to as the lateral subgrade reaction modulus ($k_s$). Table 4 provides the recommended subgrade reaction modulus for the lateral load analysis. The majority of lateral resistance will typically be offered by the upper 5 to 10 m of soil, depending on the relative stiffness of the pile and soil materials. Any voids surrounding the pile due to temporary or permanent casings should be infilled using lean-mix concrete or grout to provide proper contact with the surrounding soil.

Table 4. Recommended Values for Lateral Sub-grade Reaction Modulus ($K_s$)

| Soil | Approximate Elevation (m) | $K_s$ (kN/m$^3$) |
|---|---|---|
| Clayey Sand with Gravel Note 2 | 224.6 to 221.7 | 1,300*Z/d Note 3 |
| Silty Clay and Clay Fill (Stiff) | Grade to 219.0 | 3,400/d |
| Silty Clay (Soft to Firm) | 219.0 to 216.0 | 1,700/d |
| Till | < 216.0 | 20,000/d to 50,000/d Note 1 |

*Note 1: ks variable due to natural heterogeneity of till, laboratory testing, and soil description.*
*Note 2: Clayey sand and gravel layer only present in TH14-02.*
*Note 3: ks depends on the layer depth below grade.*

As part of detailed design, a more rigorous lateral pile analysis that incorporates the material and section properties of the pile, applied loads, final lateral deflection criteria and a more realistic elastic-plastic model of the soil response to loading should be carried out by to confirm the lateral load capacity of the piles. Elastic-plastic spring models (p-y curves) for pile increments were provide to MHL in both graphical and digitized format; the graphical p-y curves are attached. The piles can be represented in the structural analysis model using linear springs and varying spring stiffness values along the pile shaft. The stiffness values should be varied in an iterative procedure such that the calculated spring forces and deflections match the p-y curves provided for each pile segment. Once a reasonable match has been obtained, the pile head conditions (force and moments) should be provided to TREK to confirm pile deflection, shear and bending moment distributions using an L-pile model and p-y curves.

## Cantilevered Walls

### Rankine Earth Pressure Parameters

A permanent cantilevered sheet pile wall is proposed to accommodate the sidewalk widening from Rue Despins to the Taché Dock. Table 5 provides the recommended earth pressure coefficients and bulk unit weights of each soil layer for calculation of lateral earth pressures. Surcharge loads and hydrostatic water pressure below the groundwater table should be incorporated into the design of cantilevered walls, as well as an adequate factor of safety against instability. Figure 1 shows the recommended earth pressure diagram for preliminary design of the sheet pile wall. The surcharge pressure should be selected by the structural engineer for any sustained loads.

An active earth pressure coefficient ($K_a$) should be used to calculate lateral loads against sheet pile walls which are free to translate horizontally away from the retained soil by more than 0.2% of the wall height. A passive earth pressure coefficient ($K_p$) should be used if the wall is free to translate horizontally towards the retained soil by more than 2% of the wall height. An at-rest earth pressure coefficient ($K_o$) should be used if the walls undergo less than 2% movement of the wall height towards the retained soil and less than 0.2% of the wall height away from the retained soil. The table below provides $K_a$, $K_p$, and $K_o$ values for calculation of lateral earth pressures acting on below grade walls.

Table 5. Recommended Design Parameters for Cantilevered Walls

| Design Parameter | Granular Fill (Note 1) | Silty Clay | Clayey Sand with Gravel (Note 2) | Till |
|---|---|---|---|---|
| Active Earth Pressure Coefficient ($K_a$) | 0.3 | 0.5 | 0.3 | 0.3 |
| Passive Earth Pressure Coefficient ($K_p$) | 3.7 | 2.0 | 3.0 | 3.7 |
| At-Rest Earth Pressure Coefficient ($K_o$) | 0.4 | 0.7 | 0.5 | 0.4 |
| Bulk Unit Weight, Y (kN/m$^3$) | 21.0 | 17.5 | 18.0 | 21.0 |
| Effective Unit Weight, Y' (kN/m$^3$) | 11.2 | 7.7 | 8.2 | 11.2 |

### Finite Element Analysis of Sheet Pile Walls

Following preliminary analysis by MHL based on the earth pressure parameters provided in Table 5, it was determined that excessive sheet pile lengths were required to resolve the force and moment equilibrium calculations (global stability) based on the analytical method selected. The Rankine analysis provides an overly conservative pressure distribution on both the active and passive sides of the sheet piles due to simplifying assumptions of wall deflections that may not accurately represent the deflected wall shape.

A finite element model was developed by TREK to provide a more realistic representation of the constitutive behaviour of the soil, in order to better evaluate the mechanism of global stability. The wall was represented as structural beam elements within a continuum of elastic-plastic soil elements and the deflected shape, shear and moment were calculated by applying the soil unit weight and surface surcharge load (5 kPa) in one time step, representing the upper bound condition whereby the entire slope undergoes movement. The shear, moment and deflection plots are shown in Figures 2 to 4, respectively. A 10.4 m sheet pile was analysed with the maximum cantilever height of 2.4 m, and a 9.1 m long sheet pile was analysed with a cantilever height of 1.3 m. As shown in the figures, the shear force distribution shows a reversal towards zero near the pile tip and the deflected wall shape also shows restraint at the toe, which is considered an indication that global stability of the wall is satisfied. A 1.5 m thick layer of cellular concrete was also incorporated into the model and resulted in reduced top of pile deflections.

The results of the analysis are generally considered qualitative, and the maximum values of shear, moment and deflection should not be used for structural design purposes.

On the basis of the finite element analysis results, the sheet pile embedment should be calculated as follows:

1. Sheet pile lengths should allow for up to 0.5 m of cut-off that may be required due to the irregularity of sheet pile embedment that will occur during construction. We have assumed based on existing corbel details that the sheet pile cutoff is 0.5 m below the upslope grade. Therefore, the depth of embedment can be taken as the sheet pile supply length, minus the cantilever height.
2. A minimum pile embedment of 8.5 m is required for global stability of the maximum cantilever of 2.4 m. Therefore, a minimum sheet pile length of 10.9 m (36 ft) is required in this zone. We anticipate 12.2 m (40 ft) long sheet piles will be used in this area.
3. A minimum pile embedment of 4.0 times the cantilever height should be used elsewhere. The following maximum cantilever heights can be used for standard 10-ft increments of supplied pile length:

   - 30 ft (9.1 m) length – 1.8 m maximum cantilever height
   - 20 ft (6.1 m) length – 1.2 m maximum cantilever height
   - 10 ft (3.0 m) length – 0.6 m maximum cantilever height

## Closure

The geotechnical information provided in this report is in accordance with current engineering principles and practices (Standard of Practice). The findings of this report were based on information provided (field investigation, laboratory testing, geometries). Soil conditions are natural deposits that can be highly variable across a site. If subsurface conditions are different than the conditions previously encountered on-site or those presented here, we should be notified to adjust our findings if necessary.

All information provided in this report is subject to our standard terms and conditions for engineering services, a copy of which is provided to each of our clients with the original scope of work or standard engineering services agreement. If these conditions are not attached, and you are not already in possession of such terms and conditions, contact our office and you will be promptly provided with a copy.

If you have any questions regarding the findings or recommendations presented, please contact the undersigned at your earliest convenience.

Kind Regards,

**TREK Geotechnical**
**Per:**

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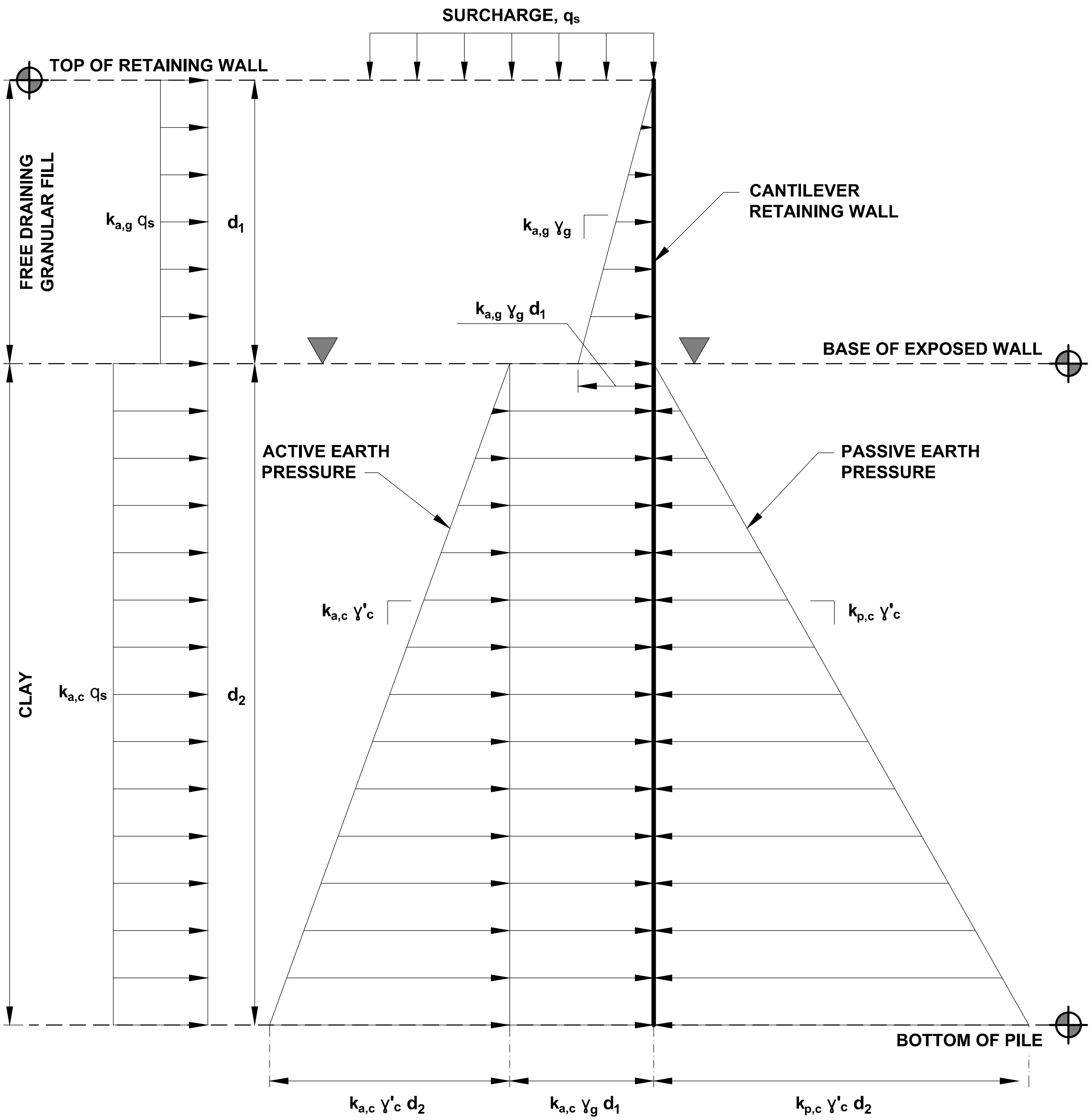



## NOTES:

1. **WALL DRAINAGE TO BE PROVIDED TO BASE OF EXPOSED WALL / FREE DRAINING BACKFILL.**
2. $\gamma'_c$ **= 7.7 kN/m$^3$**
3. $\gamma_g$ **= 21 kN/m$^3$**
4. **$k_{a,c}$ = 0.5**
5. **$k_{a,g}$ = 0.3**
6. **$k_{p,c}$ = 2.0**
7. **$q_s$ = SURCHARGE TO BE SELECTED BY WALL DESIGNER.**



# Figure 01
# Lateral Earth Pressure Distribution
Lateral Retaining Wall

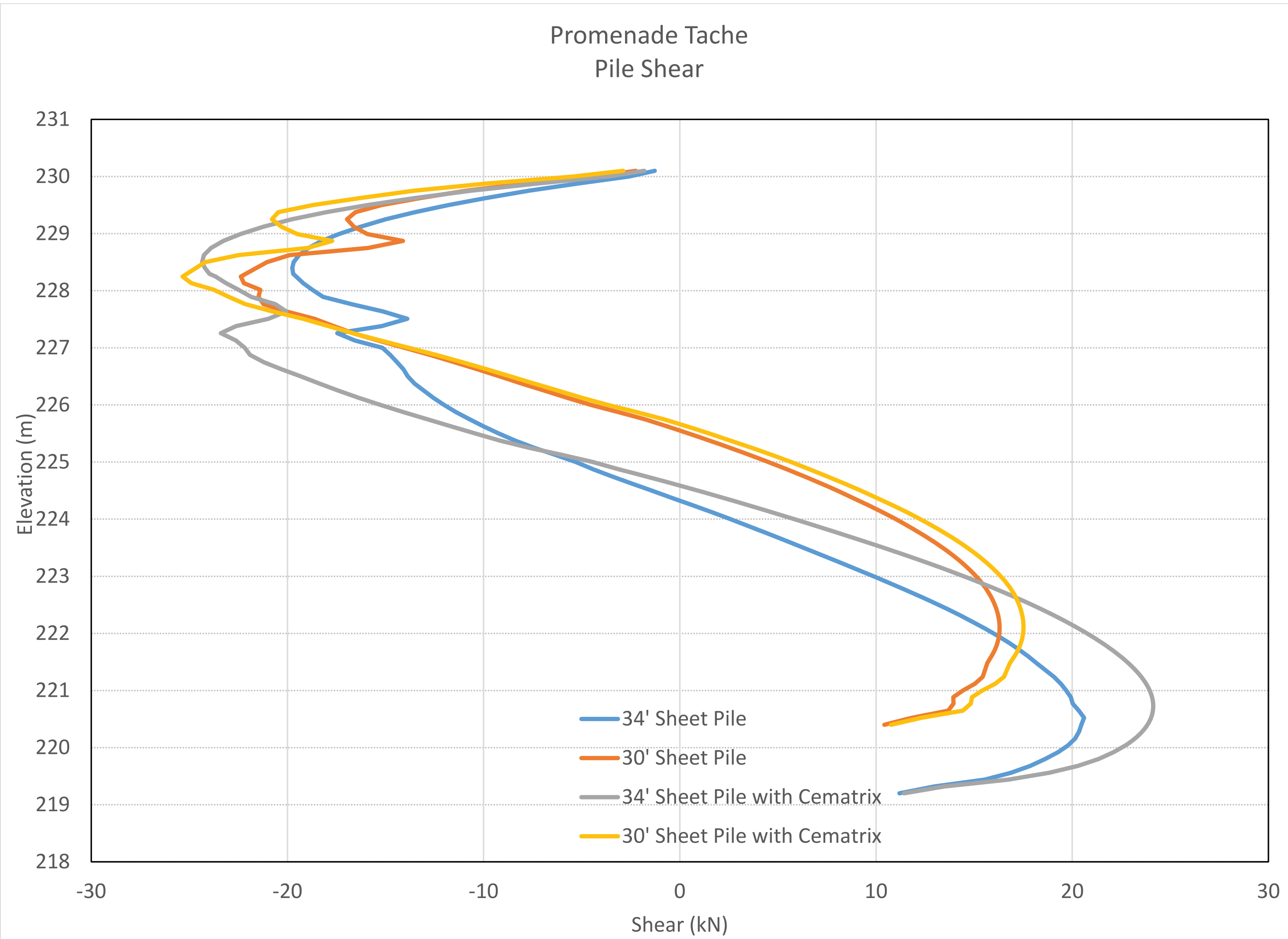
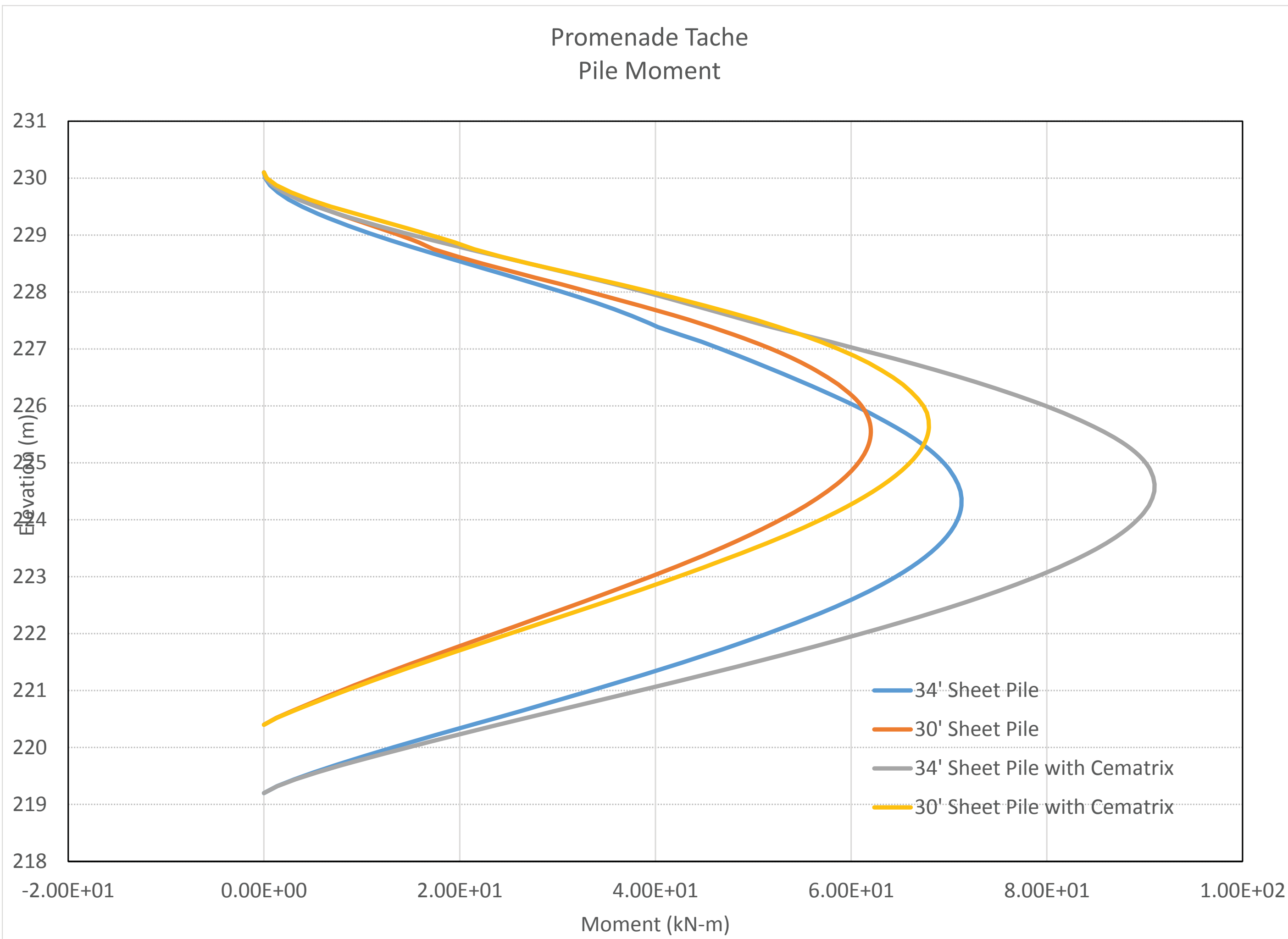



Figure 2

Finite Element Model of Sheet Pile in Clay

Calculated Shear Diagram




# Figure 3

Finite Element Model of Sheet Pile in Clay
Calculated Moment Diagram

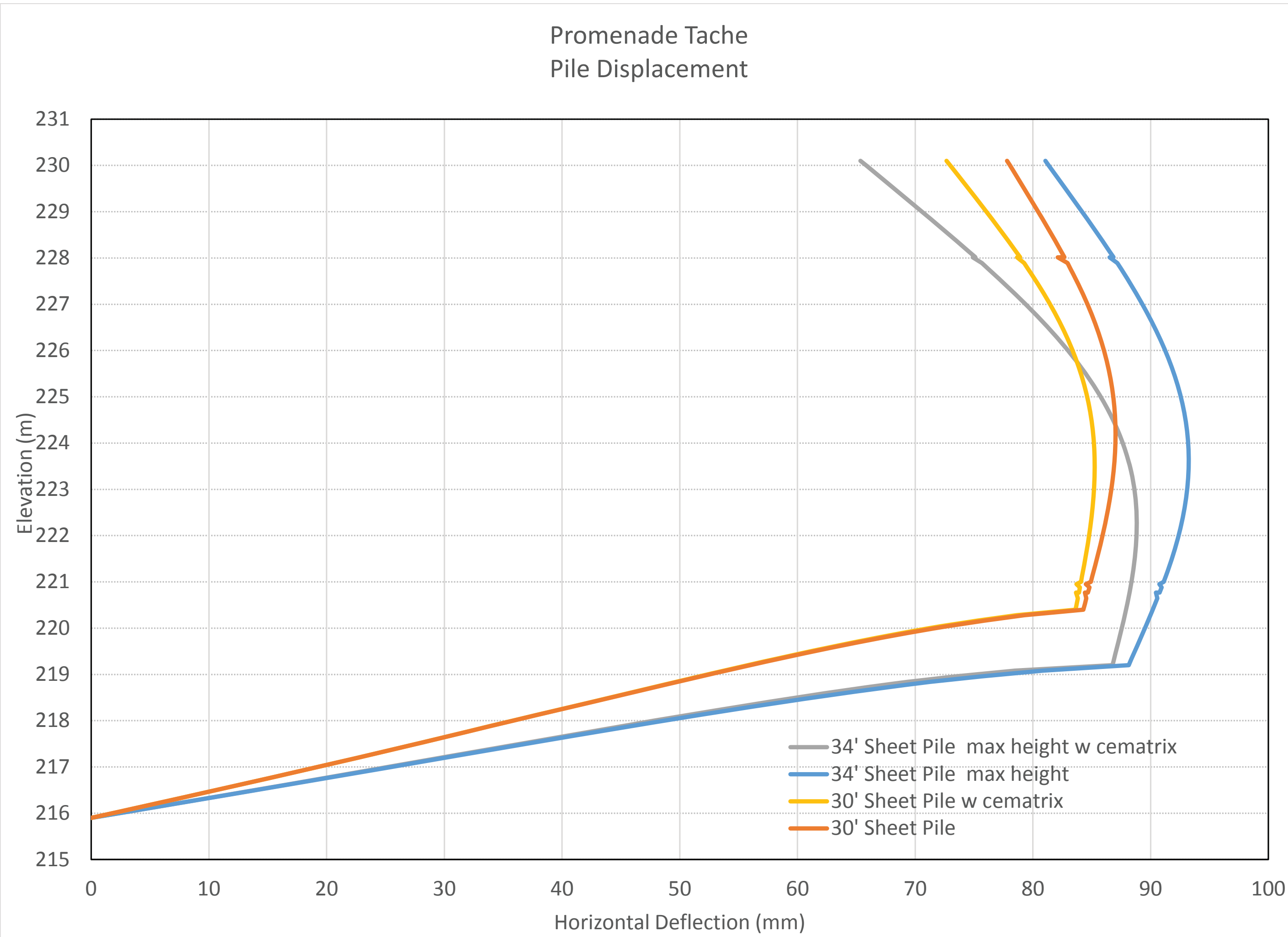
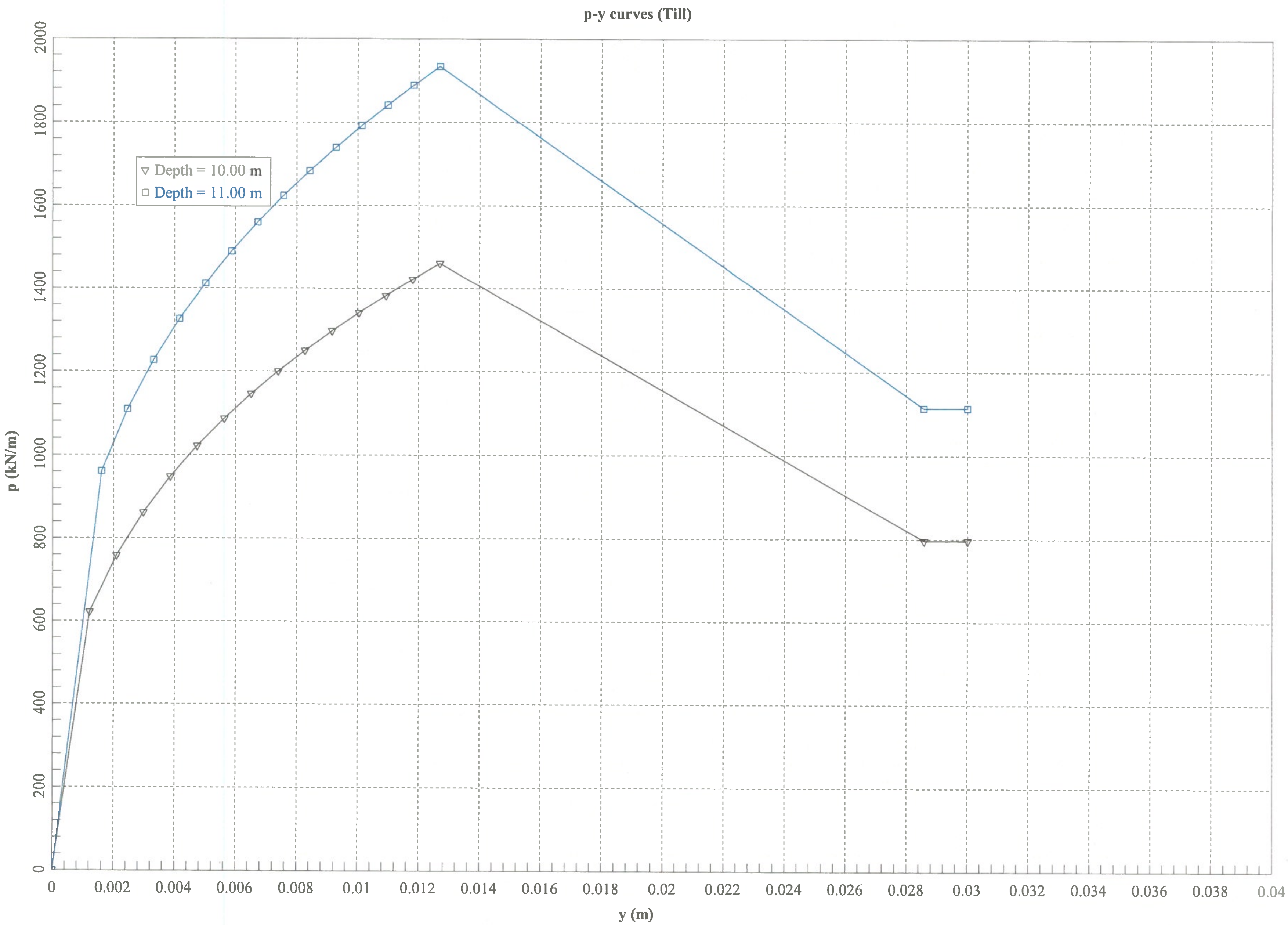



Figure 4
Finite Element Model of Sheet Pile in Clay
Calculated Deflection Diagram


p-y curves (Till)
Depth = 10.00 m
Depth = 11.00 m
p (kN/m)
0
200
400
600
800
1000
1200
1400
1600
1800
2000
y (m)
0
0.002
0.004
0.006
0.008
0.01
0.012
0.014
0.016
0.018
0.02
0.022
0.024
0.026
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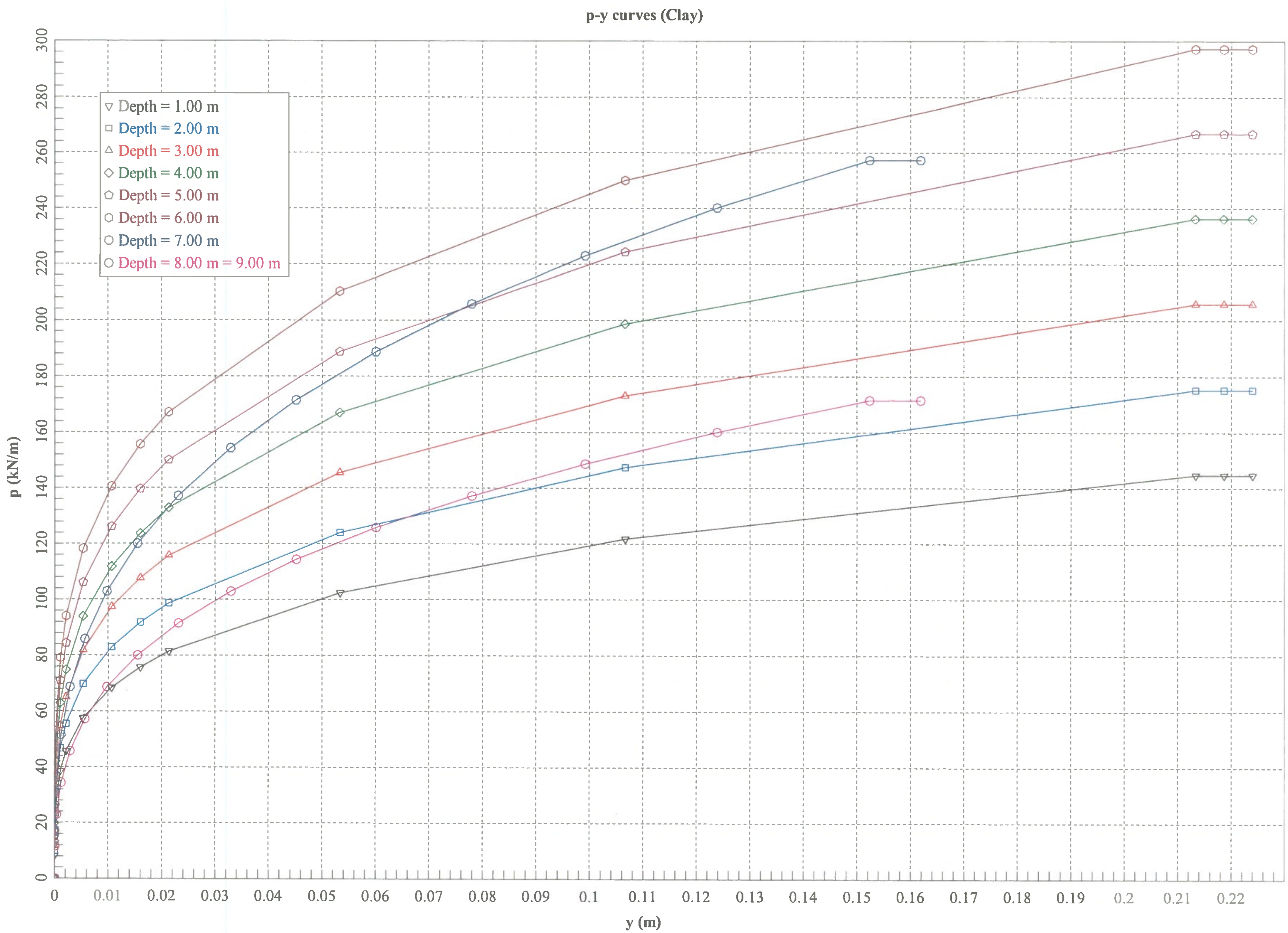
0.03
0.032
0.034
0.036
0.038
0.04


p-y curves (Clay)
p (kN/m)
y (m)
Depth = 1.00 m
Depth = 2.00 m
Depth = 3.00 m
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Depth = 5.00 m
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Depth = 8.00 m = 9.00 m