Case Study: A simplified evaluation of load transfer from a monopole foundation

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ABSTRACT
This case study outlines the evaluation of load transfer from a monopole foundation to an earth retention structure (three pile diameters away) and a deep utility (one pile diameter away). In the absence of as-built detail for the existing infrastructure, two design approaches were considered. Initially, a cautious estimate of the anticipated load transfer on each structure was evaluated using a combination of computer modelling and simplified analysis. In the second approach, load transfer within the influence of the existing infrastructure was eliminated with a double casing design approach. The end result was the successful completion of a pile foundation that mitigated risk stemming from unknown design and construction details of surrounding infrastructure. The case study also provides insight on the lessons learned that could be incorporated into future projects.

RÉSUMÉ
Cette étude de cas porte sur l'évaluation des transferts de charge entre une fondation à pieu unique et deux infrastructures à proximité: une structure de soutènement des terres située à une distance de trois diamètres de pieu et une installation profonde de service public située à un diamètre de pieu. Comme les informations sur la nature exacte des structures ne sont pas disponibles, deux approches sont considérées pour la conception. Dans la première, une estimation prudente des transferts de charge sur chaque structure a été faite en combinant la modélisation numérique et l'analyse simplifiée. Dans la seconde, les transferts de charge dans la zone d'influence des structures existantes ont été supprimés en utilisant une conception en paroi double. Il a ainsi été possible de réaliser avec succès une fondation sur pieux en atténuant les risques résultant de l'absence d'information détaillée sur la conception et le type de construction des structures situées à proximité. Cette étude permet aussi de tirer des enseignements dont pourraient bénéficier de futurs projets.

1 INTRODUCTION

Design of new foundations requires assessment of subgrade conditions to confirm that design loading can be transferred from the structure to the soil without resulting in an ultimate failure or excessive deflection. When completing new designs within previously developed areas, it is necessary to confirm that the stress transfer into the soil will not adversely affect the surrounding infrastructure. Proper assessment requires knowledge of the design tolerances and as-built construction of the surrounding infrastructure; however, it is often that this information is not available in an adequate level of detail. If not economically feasible to confirm details of the existing infrastructure through exploration, the engineers must make cautious assumptions in their assessment. The foundation design must then confirm that stress transfer is within acceptable limits defined by the owner of the surrounding infrastructure or otherwise modify the design prevent the excessive stress transfer.

This paper outlines a case where a monopole foundation was proposed within close proximity to an existing earth retention structure and a deep utility line where little to no as-built information could be obtained, in part, due to the age of the infrastructure. Other spatial constraints that prevented relocation of the foundation included a public pathway, roadway with parking lane, as well as residential and commercial buildings. The project site is within a city in Alberta; however, due the on-going nature of this project at the time of writing, the specific location, proposed development and client will remain confidential.

There were multiple loading cases for the proposed foundation, which resulted in lateral loading in several directions radially outwards from the centre of the foundation. The maximum loading condition was evaluated for assessment of the proposed foundation while the earth retention structure and deep utility were evaluated based on critical case loading in their respective directions.

In the absence of as-built detail for the structures, two design approaches were considered. Initially, the anticipated load transfer on each structure was evaluated for the critical case loading conditions using a simplified analysis. The second design approach incorporated a means of eliminating load transfer from the monopole foundation to the surrounding infrastructure.

1.1 Project Scope

Stantec Consulting Ltd. was approached by the monopole foundation designer to complete a geotechnical investigation at the proposed structure location. A previous site investigation had been completed by others for this proposed foundation, where one borehole had been advanced until auger refusal at a depth of 7.6 m. Based on the original investigation, the foundation designer indicated that significant embedment beyond 7.6 m was required to satisfy loading conditions.

The foundation designer defined the project scope to include two boreholes within the footprint of the foundation, each to a depth of 15 m. The designer confirmed the depth and close proximity of the boreholes was to ensure any expected variability was well defined as the fast approaching construction schedule would not permit any additional field programs. The purpose of the new investigation was to define appropriate unit resistances to use in design, confirm construction methodology for the concrete caisson, and identify moment, shear and deflection for the detailed design. At the onset of the investigation, the designer provided one set of unfactored loads at the pile head. Maximum tolerable pile head deflection was to be kept below 38 mm. As part of the assessment, potential impact of the proposed foundation loading on the surrounding infrastructure also needed to be evaluated. Due to spatial and access constraints onsite, a borehole closer to the existing earth retention structure could not be advanced.

1.2 Project Site

The designer proposed a drilled cast-in-place concrete pile foundation, with a diameter ranging from 2.1 m to 2.3 m and a pile length of 9 m to 12 m. The final dimensions of the pile were to be confirmed after completion of the geotechnical investigation.

Identified surrounding infrastructure of concern included a water main and an earth retention structure. Figure 1 details the foundation location in relation to the subsurface surrounding infrastructure, and also the surface developments including the roadway and pedestrian walkway.

Precise direction of the resultant critical loading was not identified at the onset of the project; however, based on identified connections to the foundation, it was understood that the critical load case was in the general direction of the earth retention structure. Radial distances from centre of the foundation to the exposed sections of the earth retention structure ranged from approximately 6 m to 13 m.

The water main was within 3.5 m centre to centre of the proposed monopole foundation. The water main was identified by facility owners as a 250 mm diameter cast iron pipe with mechanical joint connections. The water main was installed in 1965 and as-built records were not available; however, the pipe was expected to be located approximately 3 m below existing grade. While not in the direction of the identified critical load case, with a minimum clear distance less than one pile diameter, design needed to consider influence of installation as well as potential loss of support to the structure during any potential future maintenance or replacement of the water main.

Early into the review, it was confirmed that the existing earth retention structure was a former abutment for a traffic bridge that had been in service from 1909 to 1947. No as-built information could be obtained for the existing earth retention structure, other than the measurements that could be taken onsite. The measured exposed height of the structure was approximately 7 m. Depth and nature of the supporting foundation elements were unknown. The concrete thickness was variable along the wall height, which due to inaccessibility, could not be confirmed. The exposed surface of the retaining wall appeared to be in a sound condition and no signs of distress were observed. Grade behind the wall was relatively flat and gravel surfaced. Outside of the wall area the ground sloped from the asphalt walkway at an approximate 33 degree grade down to an adjacent river.

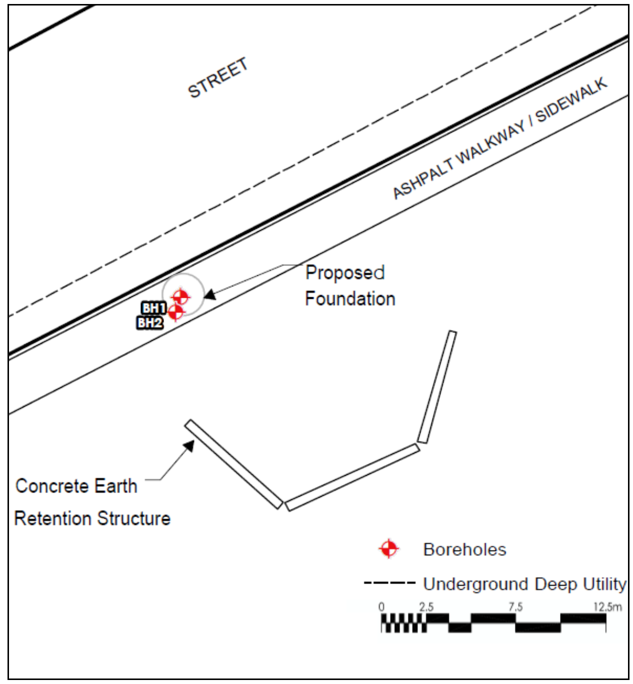

Figure 1. Proposed foundation location showing proximity to surrounding infrastructure

2 GEOTECHNICAL INVESTIGATION

The geotechnical investigation was conducted as per the scope specified by the designer. Published surficial geology identified expected native ground conditions as gravel, sand and silt alluvium overlaying sedimentary bedrock. Boreholes confirmed an approximate 5 m thick overburden underlain by sedimentary bedrock. Groundwater was observed at approximately 7 m below existing grade, closely matching the elevation of the adjacent river.

Overburden soil was sampled off of the auger flights and from split spoon samplers during Standard Penetration Tests, and the bedrock was sampled with HQ rock cores. Based on field classification following the Unified Soils Classification System (USCS) the upper 5 m of overburden may generally be described as silty sand with gravel (SM). The compactness of the sand was variable throughout its depth in both boreholes. Based on N-Values ranging from 4 to 47, the sand may be described as loose to dense.

The sedimentary bedrock varied between siltstone and claystone. From approximately 5 m to 7.5 m below grade, the bedrock was completely to highly weathered, and recovered core samples had Rock Quality Designation

(RQD) values ranging from 15% to 70%. Below 7.5 m the recovered core samples had RQD values ranging from 27% to 72% and were observed to have higher strength. Unconfined compressive tests were completed in the lower slightly weathered to fresh bedrock core samples. The tested unconfined compressive strengths varied irrespective of depth, ranging from 1.1 MPa to 35.2 MPa.

Based on variable compactness of the sand and expected influence from past construction of the surrounding infrastructure, the sand was considered to be uncontrolled fill consisting of local alluvial soils. Due to its completely to highly weathered nature, the bedrock from 5 m to 7.5 m was considered to have properties closer to a soil matrix than sound bedrock. For design and construction purposes, the bedrock below a depth of 7.5 m was defined as competent.

3 ANALYSIS

The analysis proceeded by first assigning parameters to represent the existing ground conditions. Based on assigned unit resistances, adequacy of proposed cast-in-place pile dimensions to satisfy design loading was then confirmed. Lastly, the influence foundation loading on surrounding infrastructure was evaluated.

Resistance to axial loading and constructability of the pile for the conditions encountered were evaluated and reported on following standard practices outlined in CFEM (2006) and FHWA (2010). This aspect of the design was relatively straightforward and is not further detailed in this paper. The design basis for the lateral pile response, and the methodology and results for the modeled load transfer are outlined in the following sections.

3.1 Parameter Selection

Based on the variability observed in the compactness of the sand and strength of the bedrock, as well as the level of uncertainty on potential variation of the ground conditions radially outwards from the pile location, further advanced laboratory testing on collected samples was not deemed necessary or beneficial. Expected cautious estimates of the parameters that may control the lateral load response were selected based on observed field conditions, unconfined compressive strength testing on rock cores, published correlations (Bowles, 1996), and local experience. Additionally, a parametric study to consider sensitivity of the analysis was completed during the design process. The selected parameters used for each layer in the lateral pile response are summarized in Table 1.

Table 1. Unfactored Soil Parameters

Layer Parameters (depth range)	Sand (0-1m)	Sand (1-5m)	Bedrock[1] (5-7.5m)	Bedrock[2] (>7.5m)
Total Unit Weight (kN/m^3)	18	20	21	21.5
Undrained Shear Strength (kPa)	N/A	N/A	200	550
Angle of Internal Friction (degrees)	28°	32°	N/A	N/A
Unconfined Compressive Strength (kPa)	N/A	N/A	400	1100
RQD (%)	N/A	N/A	N/A	50
Strain Factor, k_{rm}	N/A	N/A	N/A	0.0005
Young's Modulus, E_s (MPa)	N/A	N/A	N/A	500

[1]Parameters for weathered bedrock
[2]Parameters for competent bedrock

3.2 Modelled Response

Initially, the modelled response was evaluated for the critical load case provided in the general direction of the earth retention structure. Influence with the water main was considered from a loss of support condition, where potential future excavation for repair or replacement of the pipe may disturb or otherwise remove soil in contact with the pile.

To allow for an optimized design, further detail was requested from the designer. As shown in Figure 1, proximity of the earth retention structure to the proposed foundation varied depending of radial direction outwards from the pile; therefore, the vector for the resultant load was requested. The conditions for the critical load were also requested, with the understanding that if the critical load may be tied to an extreme cold weather condition, load response would be affected by frozen subgrade conditions.

The foundation designer provided another thirty-six load cases for review, in addition to the original one provided during the proposal stage. From these additional load cases, six critical load cases were identified. The approximate resultant loading directions for each of the critical load cases is shown in Figure 2.

The lateral response for each of the six critical load cases where analyzed with the software program LPile (version 2012) developed by Ensoft Inc. Soil parameters from Table 1 were applied in the design using the software's standard Reese p-y curve models for sand, stiff clay without free water, and weak rock. The stiff clay model was selected for the weathered bedrock as the siltstone and claystone was almost completely disintegrated into a soil matrix throughout the weathered zone. Within LPile, the unfactored loads for each loading case were applied to the head of the proposed pile design to confirm lateral deflection remained within design tolerance.

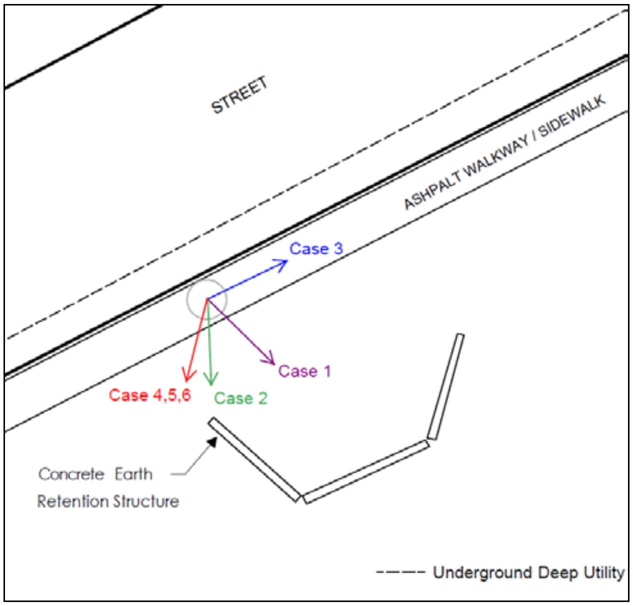

Figure 2. Approximate Critical Case Loading Vectors

Once the proposed pile dimensions were confirmed adequate to resist design loading at ultimate and serviceability limit states, influence of the critical load cases on the surrounding infrastructure were evaluated. Due to level of uncertainty of the as-built construction of the surrounding infrastructure and variable compactness of the sand, a simplified evaluation of the load transfer was considered appropriate. Provided cautious estimates of layer parameters were applied using a conservative design approach. The expectation was that the design could be validated provided resulting predicted load transfer to the surrounding infrastructure was tolerable.

The simplified method employed used LPile to model mobilized soil reaction. The soil reaction across the width of the pile was in turn treated as a uniformly loaded footing with stress distributed outwards from the pile in an approximate Boussinesq distribution.

An example of the mobilized soil reaction versus depth generated by LPile for one of the critical load cases is shown in Figure 3. For influence on surrounding infrastructure, the maximum mobilized soil reaction within the sand layer was recorded for each critical case loading condition. The maximum mobilized soil reaction generally occurred at or near 3 m below existing grade for all critical load cases.

By treating the maximum mobilized soil reaction as the loading (in kN/m) over the proposed pile diameter a calculated pressure (in kPa) was determined. A simplified Boussinesq distribution applied for footings in the vertical plane was applied across the horizontal plane. Stresses at the point of intersection with the nearby infrastructure were approximated by distributing the pressure on the loaded area outward at two vertical to one horizontal, 2V:1H, as shown in Figure 4.

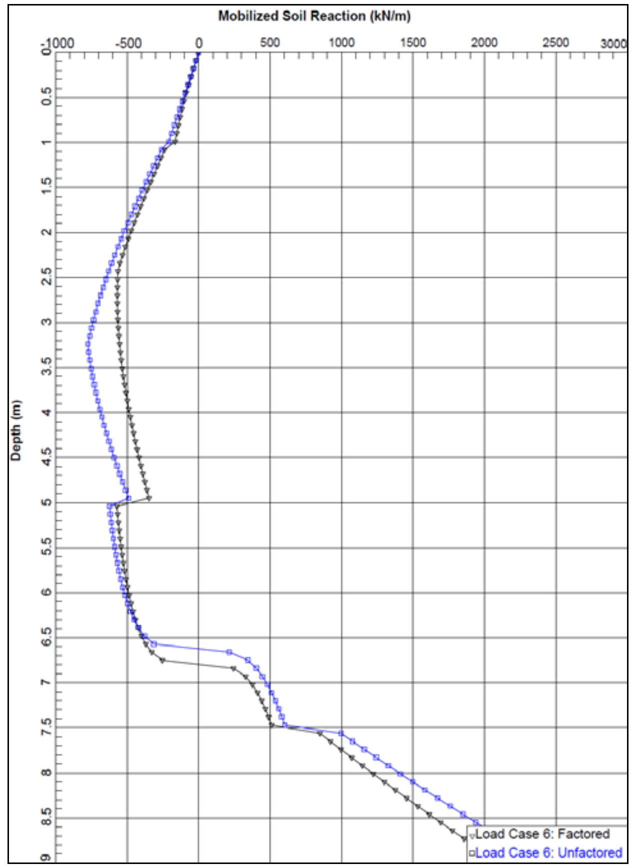

Figure 3. LPile mobilized soil reaction output for one of the critical case loads

This approach approximated the maximum pressures resulting from the proposed pile foundation on the water main and earth retention structure. By considering the corresponding deflection at the depth of maximum mobilized soil reaction, an equivalent spring was calculated for the soil. By assuming this spring value would remain constant throughout the distance from the pile to the surrounding infrastructure, a conservative estimate of the amount of movement required for the stresses to fully dissipate could be calculated. As this was a simplified analysis, considerations such as heave at the surface were not considered when calculating resulting pressures and required soil movement to dissipate those pressures.

It is recognized that the 2V:1H approximation can introduce measureable errors for distances less than four times the footing width (Hunt, 1986); therefore, a more rigorous numerical approach as outlined in Craig (1993) was also checked for comparison. It was confirmed that for the predicted loading and deflection the differences between the numerical approach and the 2V:1H approximation was negligible.

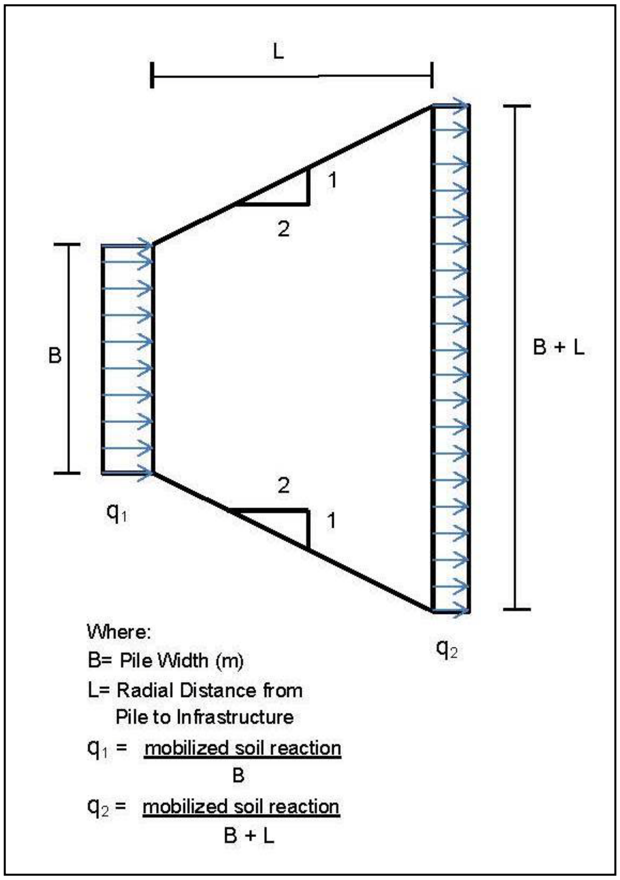

Figure 4. Calculation of stress transfer to a distance radially away from proposed pile

4 RESULTS

The load transfer calculations outlined above were summarized and provided to the foundation designer for review.

4.1 Calculated Stresses

Based on the provided load cases, using the above described simplified approach, the maximum stress transfer to the existing infrastructure was calculated to be in the order of 40 kPa on the existing earth retention structure and in the order of 60 kPa on the existing water main. Based on the modelled soil conditions, these transferred stresses were expected to fully dissipate provided the infrastructure and surrounding soils were permitted to deflect in the order of 3 mm.

At the request of the designer, the impact to the surrounding infrastructure was further reduced by computing the stress transfer while considering a more robust pile design, of longer length and greater diameter. For the updated pile design, again using this simplified approach, the stress transfer to the existing infrastructure was calculated to be in the order of 25 kPa on the earth retention structure and in the order of 35 kPa on the water main. Based on the modelled soil conditions, the transferred stresses were expected to fully dissipate provided the infrastructure and surrounding soils were permitted to deflect in the order of 2 mm.

These stresses and deflections were associated with extreme loading events and their magnitude was considered to be small by the authors. However, given the uncertainty with as-built construction details for the adjacent earth retention structure and water main, foundation designers were unable to confirm that the infrastructure could tolerate the calculated movements and stresses. Furthermore, any potential risk to surrounding infrastructure would not be acceptable unless deemed appropriate by the facility owners.

4.2 Stress Transfer Elimination

In the absence of as-built details for the structures, the facility owners were unable to reconcile with the potential risk; therefore, a second design approach was applied. This approach considered eliminating load transfer from the monopole foundation to the surrounding infrastructure.

In order to eliminate the load transfer to the adjacent water main and earth retention structure, an outer casing, a minimum of 60 mm radially larger than the pile diameter and embedded into the weathered bedrock was proposed. The minimum 60 mm radial difference was calculated based on calculated maximum pile head deflections for each of the critical cases, using LPile as well as considered nominal casing sizes.

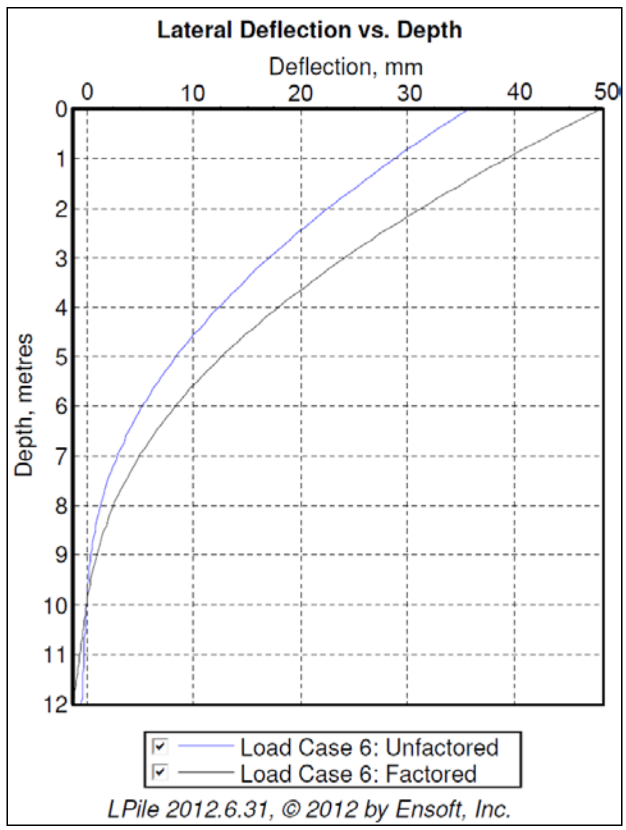

Figure 5. LPile deflection output for critical case loading with maximum pile head deflection for 5 m cantilevered pile

In this proposed design, two sets of casing would be required, an inner and outer casing. All soils within the outer casing would be required to be removed for its full embedded length to prevent any load transfer from the pile. The inner casing for the pile was recommended to extend into the bedrock layer sufficiently to permit casing to free stand during excavation and construction for the pile.

This resulted in a cantilevered pile design, with no lateral support in the upper 5 m of the pile. Reinforcing steel was increased accordingly to accommodate the column design. The minimum 60 mm space between the pile and the outer casing must remain void for the service life of the proposed foundation; therefore, a cap to prevent accumulation of water infiltration, debris or other materials was required. This was addressed by allowing for a metal cap bolted to the inner casing that extended over and outwards of the outer casing. The cap was designed such that a vertical and horizontal spacing of 75 mm exists from the cap to the outer casing, again to accommodate the calculated deflection at the pile head for the various critical load cases.

During construction, the void space would be confirmed to target depth prior to placement of the cap. As the project site is adjacent to a pathway, as well as residential and commercial developments, the construction details of the cap were required to be reviewed and approved by the municipality to ensure aesthetics and public safety were also considered.

5 DISCUSSION

Due to the uncertainty with the design and construction details of the surrounding infrastructure, taking a more simplified approach to start was an efficient means of analysis for this project. By using a simplified approach we were able to get reasonable results to estimate the effects the proposed pile would have on the nearby water main and earth retention structure. As discussed above, it was determined the facility owners were unable to accept the risk of the calculated stresses, therefore a design approach to eliminate stress transfer to the surrounding infrastructure was the chosen project solution.

5.1 Lessons Learned

Lessons learned on this project are considered applicable on future projects of a similar nature.

Communication is a key component on any project, but for this project, earlier communication with facility owners should have been initiated. A meeting with the facility owners to discuss acceptable risk should have taken place before any design or analysis was undertaken. This would have made it clear to all involved parties that no stresses on the existing infrastructure were acceptable and would have ensured that time would not have been spent on stress reduction design mitigation efforts.

At the time of the proposal submission, only one critical load case was identified. During the analysis stage, another thirty-six (36) load cases were provided for review. From this, a total of six (6) critical load cases were identified. Had this information been readily available at the beginning of the project scope, the analysis could have been streamlined.

Another design approach that could have been considered in the beginning of this project would be a reliability based design. Given the number of unknowns that existed with this project, probability of critical case loadings could have been factored into the design. This approach may have proven more acceptable to the facility owners. If there had of been clear expectations outlined by the facility owners, including potential acceptable levels of risk, an alternate solution could have been reached.

Finally, the geotechnical team was brought in on this project much later in the design phase of this work. Construction was already scheduled and was to follow shortly after the additional investigation. Had there been more time to assess the problem, and discuss solutions with facility owners and the client, other solutions could have been potentially considered. There may have been potential to engage a pipeline stress engineer that may have been able to confirm if anticipated stress and deflection were within the water main tolerances.

Overall the project was considered a success. Even though it occurred later in the geotechnical design phase, through meetings with the owner’s design team, an acceptable solution was identified that allowed construction to proceed.

6 CONCLUSION

This case study has outlined design recommendations developed for a monopole foundation proposed within close proximity to an existing earth retention structure and a deep utility line. Multiple critical loading cases were considered in design on the earth retention structure at a distance of approximately three pile diameters away, and on the deep utility at a distance of approximately one pile diameter away. Minimal as-built detail for the surrounding infrastructure was available due to the age of the deep utility and earth retention structure. A simplified analysis approach making use of LPile software to identify maximum mobilized soil reaction, combined with a simplified 2V:1H Boussinesq distribution to predict stress dissipation allowed for cautious estimates of stress transfer to be predicted in a short time frame.

Ultimately, the design approach adopted for the project was to eliminate stress transfer to surrounding infrastructure using a double cased pile where an inner void greater than the predicted maximum deflection was maintained. The cantilevered design proved to be a cost effective solution given that it closely matched original construction methodology and adequate resistance immediately below the cantilevered zone was provided by relatively shallow bedrock conditions to prevent significant increase to pile length. The end result was the successful completion of a pile foundation that mitigated risk stemming from unknown design and construction details of surrounding infrastructure. The case study also provides insight on the lessons learned that could be incorporated into future projects.

ACKNOWLEDGEMENTS

Although owner and foundation designer names and affiliations cannot be released due to confidentiality, the authors would like to acknowledge that the successful completion of this project could not have been done without the cooperation and good communication that developed as the project proceeded over a short time frame.

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