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MEMORANDUM

DATE: December 13, 2018

TO: Mr. Steve Faulkner, P.E., Pacific Forest Resources, Inc.
Mr. Barry Gerbracht, M.Eng., P.Eng., All-Span Engineering & Construction, Ltd.

FROM: Khaled M. Shawish, PE
Carston T. Curd, GIT

KHALED M. SHAWISH
STATE OF WASHINGTON
35215
REGISTERED PROFESSIONAL ENGINEER

RE: Geotechnical Pile Plan Review - REVISED
Swan Creek Pipeline Bridge
Tacoma, Washington
NGA File No. 1012417

This memorandum documents our review of preliminary design for pile elements supporting the abutments for the Swan Creek Pipeline Crossing, and verification that they are in concurrence with our most recent geotechnical recommendations and reports.

INTRODUCTION

We previously completed a geotechnical evaluation for this project, the results of which were summarized in a report titled "**Geotechnical Engineering Evaluation - REVISED – Swan Creek Pipeline Crossing – East 68th Street & East Pipeline Road – Tacoma, Washington**," dated April 30, 2018. At the time, the project consisted of replacement of an existing roadway embankment across Swan Creek with a pipeline bridge, removing the existing culverts for the creek. The report based recommendations for drilled and driven support piles for the pipeline and pedestrian bridge from two geotechnical boreholes completed to depths of approximately 51 feet below the existing ground surface. The report also summarized a seismic study completed for the site, as well as detailed recommendations for earthwork, abutment support, and erosion control.

PLAN DESCRIPTION

We were provided with a 60% submission project plan set dated July 5, 2018, and pile design calculations dated September 18, 2018, produced by All-Span Engineering & Construction, Ltd. We were also provided with undated schematic cross sectional and topographic sections for the project.

The cross sectional sketches and 60% submission plan set show a proposed 148-foot long pedestrian bridge in the alignment of the existing pipeline, which is currently supported by an embankment. The existing road fill and 54" culverts are called to be removed and regrading completed to create a channel for the creek. A new pipeline bridge with a pedestrian upper deck is shown to cross the ravine along the same alignment as the road. In the drawings, it appears to be supported by steel piles with concrete caps in two locations along the bridge. The western abutment will require a partial cut up to ten feet, while the east abutment will require a partial cut up to six feet in height. The cuts are shown to be stabilized with lock blocks which are proposed to retain the fill slope in front of the respective pilasters for the decking. Abutments will be reinforced. On "Abutment Details, Sheet 2," the typical pile section is shown to be a 30-inch diameter, 0.75-inch thick pipe with reinforcement. We initially provided recommendations for drilled augercast piles prior to updating our geotechnical report. We understand that the current design calls for driven piles.

Provided structural calculations walk through three design load cases. We understand that the seismic loading conditions govern the design of the piles. The lateral subgrade modulus utilized in the calculations is 1274 kips/ft^3 [200 MN/m^3] which is approximately 700 PCI. According to the structural calculations, embedment of 30 feet [9 m] is sufficient to provide the intended lateral capacity.

DYNAMIC PILE CAPACITY ANALYSIS

We analyzed pile capacity based on the Hiley pile driving formula. Assumptions of the calculations rely on a direct relationship between the resistance of pile to the impact of the hammer at the time of driving, and the resistance of the pile to settlement under the ultimate loads. The formula utilizes the physical parameters of the driving hammer and the efficiency of each hammer strike to calculate the ultimate driving resistance in granular soils. We utilized the Vulcan 530 onshore hammer, which weighs 30 kips [25,021 Kg], and has a rated strike energy of 150,000 ft-lbs [204 kJ], with a nominal stroke of 60 inches [1.524 m]. We assumed a 50% hammer efficiency and a coefficient of restitution of 0.5. Based on these assumptions, this hammer can produce an ultimate pile capacity of roughly 1000 tons [9,964 kN]. However, in order to limit damage to the pile and for the purposes of reducing drive time, we recommend limiting the refusal criterion to 20 blows per foot. If this refusal criterion is reached, we calculated 600 tons [5,978 kN] of ultimate axial compression capacity per pile. Review of structural calculations show a maximum working compression capacity is approximately 180 tons [1,635 kN]. Therefore, a refusal criterion of 20 blows per foot results in an approximate safety factor of 3.0. If piles reach an absolute minimum embedment of 24 feet [7.3 m] before the refusal criterion is reached, blow counts as low as 11 blows per foot would still provide a factor of safety of at least 2.0.

CONCLUSIONS

Upon review of the provided plans, calculations and specifications, we found that the seismic loading condition governs the design of the piles. The lateral subgrade modulus of 1274 kips/ft^3 [200 MN/m^3] used for the lateral pile analysis is adequate. Based on a 24-foot [7.3 m] embedded length of the pile and the previously provided design values of 1800 pounds per square foot (psf) [86 kN] skin friction and 6000 psf [287 kN] end bearing capacities, a 180 ton axial compression design capacity should be considered during a seismic event with a peak horizontal ground acceleration of 0.3g. Values for skin friction and end bearing which were provided in the geotechnical report include embedded factors of safety. Due to anticipated difficult driving conditions, we do not recommend using a smaller hammer or thinner pile wall.

We trust that this memorandum should satisfy your needs at this time. Please do not hesitate to contact us should you require anything further.

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