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MEMORANDUM

To: Brian Nigg, PE; and Craig Totten, PE, SE / KPFF Consulting Engineers

Date: September 1, 2022 **(REVISED)**

GRI Project No.: 6637-A

From: Scott M. Schlechter, PE, GE and Declan P. Schade, PE

Re: Conceptual Geotechnical Considerations Memorandum
City of Bend Midtown Crossings
Bend, Oregon

DRAFT

At your request, GRI has prepared this conceptual geotechnical considerations memorandum to evaluate the geotechnical hazards at the site and a feasibility level assessment of foundation types. This phase of the project was limited to a review of available subsurface information and a site reconnaissance conducted by a licensed professional engineer (PE) specializing in geotechnical engineering.

PROJECT DESCRIPTION

In 2020, City of Bend voters passed a bond funding measure that funded a key City of Bend Transportation System Plan (TSP) project known as the Midtown Pedestrian and Bicycle Crossings. This project includes a study to determine the timing, feasibility, and needs for new or improved crossings of the US 97 Bend Parkway and BNSF railroad for walking and bicycling in Bend's Midtown at three specific locations: NW Greenwood Avenue, NW Hawthorne Avenue, and NW Franklin Avenue. We understand that the improvements at the Greenwood Avenue and Franklin Avenue crossings are currently planned to be modifications to the existing under-crossings. The improvements to the crossing at the Hawthorne Avenue location is currently planned to be a pedestrian and bicycle bridge that spans above US 97 and the BNSF railroad.

Two alternatives are being considered for both the Greenwood Avenue and Franklin Avenue under-crossings. Alternative 1 at Greenwood Avenue includes relatively minor grade changes and development of a shared use path, within the travel lanes while Alternative 2 includes construction of additional retaining walls through much of the alignment to better match current elevated sidewalk grades on either side of the street. Alternative 1 at Franklin Avenue includes a new, narrower path and wall construction on the east side, while Alternative 2 includes new shared use paths up to 18 feet wide and replacement of both the US 97 and railroad bridges with longer spans.

SITE DESCRIPTION

Topography

The project site is broken up into three locations: the intersections of NW Greenwood Avenue, NW Hawthorne Avenue, and NW Franklin Avenue with US 97 Bend Parkway and the BNSF railroad. The three crossing with US 97 generally trends east-west. The Greenwood Avenue and Franklin Avenue crossing locations are established under-crossings that fall below the highway and the BNSF railroad. Current pedestrian and bike access for the established under-crossings are limited to relatively narrow elevated sidewalks running along both directions of traffic. The current interchange at Hawthorne Avenue includes a merge lane onto southbound US 97, but there is not an existing under-crossing or over-crossing at this location.

The general project area includes moderate to dense residential and commercial development, primarily to the west of US 97, and commercial and industrial development, to the east of US 97. The ground surface is covered in existing structures, landscaping areas, asphalt, and/or gravel surfacing.

Geology

The general stratigraphy of the area consists of variable thicknesses of granular uncontrolled fill and/or unconsolidated native silty sand soil derived from volcanic ash and weathered pyroclastic material overlying basalt bedrock. Published geologic maps of the area indicate that the bedrock unit is basaltic lava flows deposited from the Newberry volcano (Qbn) (Sherrod et al., 2004). In the Quaternary age, the basaltic lava flows erupted from the vents on the northern and northwestern flanks of the Newberry volcano, covering a broad geographic region east of the Deschutes River. The deposition of the basalt flow is characterized by broad hummocky plains and columnar jointing in bedrock exposures.

No mapped or historic landslides were identified within the limits of the project site on the Oregon Department of Geology and Mineral Industries (DOGAMI) statewide landslide hazard database (SLIDO Version 4.0). DOGAMI is the state agency responsible for geologic hazard mapping in Oregon.

No faults are mapped within the proposed alignment of the three crossings or in the near vicinity of the project site. The nearest mapped fault listed in the U.S. Geological Survey (USGS) Quaternary Fault and Fold Database is the Sister's Fault Zone, located approximately 0.5 miles east of the site. The Sister's Fault Zone is comprised of numerous northeast-and southwest dipping normal faults. The faults that make up the Sister's Fault Zone have a strike that generally trends to the northwest and is considered seismogenically active with the interpreted age of the most recent prehistoric significant deformation (Mw 4.0 +) listed as middle- and late-Quaternary (Personius and Haller, 2016).

SUBSURFACE CONDITIONS

Based on available geotechnical information from ODOT bridge foundation sheets for the current Greenwood Avenue and Franklin Avenue crossings, geotechnical reports from others provided by the City of Bend, and our experience near the project area, we anticipate that the following general subsurface conditions will be encountered in the project area.

FILL

Uncontrolled fill containing coarse-grained soil ranging from sand to gravel may be encountered, given the urban development in the area. Fill was noted in a recent geotechnical report for the Sunlight Solar building on the east side of the US 97 along Hawthorne Avenue. Cobble to boulder-sized clasts and silt can be found in the matrix of the fill. Relative density of the fill can be highly variable, ranging from very loose to very dense. Deleterious fill materials have also been documented in the fill present in the region; the deleterious material ranges from concrete, wood, metal, plastic and glass debris. Horizontal and vertical variability should be anticipated wherever uncontrolled fill is encountered.

Silty SAND

Native brown silty sand overlying the basalt bedrock is anticipated within the project area. This unit has been documented to contain gravel, cobbles, and boulders within the silty sand matrix. Relative density of the sand ranges from loose to very dense. Pumice and ash lenses have also been documented within this unit.

BASALT (Newberry Basalt)

The Newberry Basalt underlies the soil overburden; the unit is predominantly gray, moderately weathered to fresh, medium-hard to very hard (R3 to R5) basalt. The vesicularity of the basalt typically ranges from some vesicles to highly vesicular, with localized zones being scoriaceous. Secondary mineralization infilling and staining have been documented in the vesicles and discontinuities of the basalt. The unit generally exhibits very close to moderately close discontinuity spacing, although zones of wide joint spacing have been documented. Localized zones of extremely soft to soft, decomposed to predominantly decomposed basalt can be expected throughout this unit.

Groundwater

Our review of available subsurface information and nearby published Oregon Water Resources Department (OWRD) groundwater study well logs (OWRD, 2022) suggest the groundwater level at the site vicinity occurs at a depth of about 500 feet to 700 feet below the ground surface. However, localized perched groundwater conditions may develop at shallower depths within the silt and sand layers that overlay the basalt bedrock and may approach the ground surface during the wet winter and spring months or following periods of prolonged or intense precipitation.

CONCEPTUAL DESIGN RECOMMENDATIONS

Based on our discussions with you, we understand that the proposed crossing improvements are still in the preliminary phase of design. Preliminary drawings indicate that the planned improvements at the Hawthorne Avenue overcrossing structure could be supported by a combination of shallow foundations, pretensioned rock anchors, micropiles, and drilled piers. Bridge replacements with a longer span are also planned as part of Franklin Avenue Alternative 2. Based on the review of available subsurface information, we anticipate that the planned improvements will be most cost-effective, founded in the shallow basalt that is present in the site vicinity. Recommendations regarding conceptual foundation support and seismic criteria are provided in the following sections. Additional explorations should be completed as part of the next phase of design to better evaluate the subgrade conditions and the preferred foundation system for the selected alternatives.

Conceptual Engineering Considerations Support

Shallow Foundations

We understand that spread footings and deep foundations are both being considered for the design of the Hawthorne Avenue crossing improvements, depending on the depth of rock encountered during subsequent design. Conceptual drawings for the crossing improvements have bridge abutments, bents, and staircases possibly supported by shallow foundations. Based on the rock observed in the existing cuts for the Franklin Avenue crossing, we understand that spread foundations will likely be the preferred alternative for those structures unless excavation next to existing structures precludes their use.

Spread footings embedded at least 1 foot into the soft (R2) or harder basalt can be designed using a nominal bearing resistance of up to 20,000 pounds per square foot (psf) for the Strength and Extreme Limit States. For the Service Limit State, we estimate a nominal bearing resistance of 8,000 psf will induce static settlements of less than 1 inch for foundations on R2 or harder rock.

Spread footings embedded at least 2 feet into the silt and sand with a width of at least 3 feet can be designed using a nominal bearing resistance of up to 5,000 psf for the Strength and Extreme Limit States. For the Service Limit State, we estimate a nominal bearing resistance of 2,000 psf will induce static settlements of less than 1 inch for foundations on silt and sand. These assumed values should be reevaluated once explorations are completed in the area.

Nominal coefficients of base friction values of 0.55 and 0.40 may be used for determining sliding resistance of foundations established on basalt and compacted sand and gravel, respectively. If additional lateral resistance is required, passive earth pressure against abutments in soil can be evaluated on the basis of a nominal equivalent fluid unit weight of about 5400 pounds per cubic foot (pcf) for the Strength and Extreme Limit States, assuming the footing is cast neat against backfill consisting of undisturbed soil or granular structural fill. For the Service Limit State, we recommend using a nominal equivalent fluid unit weight of 200 pcf to limit horizontal displacements. These values are only applicable if the footings are cast neat against undisturbed soil or backfilled using granular structural fill, following the recommendations provided in this report. For the extreme limit state, lateral resistances due to passive earth pressures for footings embedded in soft (R2) or harder basalt can be computed on the basis of a nominal equivalent fluid having a unit weight of 1,000 pcf for all limit states. These values are only applicable if the footings are cast neat against basalt rock. These values also assume the ground surface surrounding the footings is horizontal and does not slope downward away from the footings. For this reason, we do not recommend using passive earth pressure on the downslope side of foundations for resistance to lateral loading. In accordance with the AASHTO LRFD BDS, the following resistance factors should be used when evaluating the geotechnical capacity of spread footings.

Table 1: RESISTANCE FACTORS FOR SHALLOW FOUNDATION DESIGN

		Spread Footing Resistance Factors	
	Strength Limit State Resistance Factor	Service Limit State Resistance Factor	Extreme Limit State Resistance Factor
Bearing Resistance	0.45	1.00	1.00
Sliding Resistance	0.80	1.00	1.00
Passive Component of Sliding Resistance	0.50	1.00	1.00

Foundation subgrade excavation should provide a horizontal surface. All rock excavations required to establish foundation subgrade should be accomplished using blasting, line drilling, percussion, and/or chipping methods if necessary. Footing subgrade should be observed and evaluated by geotechnical staff from GRI. If soft or highly weathered rock or loose rock associated with major joints is encountered at foundation grades, the subgrade may need to be overexcavated to relatively hard rock to achieve the assumed design values.

Deep Foundations

Based on preliminary drawings provided by you, we understand that micropile, pretensioned rock anchors, and drilled shafts are being considered to provide an increase in axial capacity and uplift resistance in foundation elements for the crossing improvements. We understand the maximum axial load on each micropile and pretensioned rock-anchors is estimated to be on the order of 150 kips to 300 kips and the maximum axial load on each drilled shaft is estimated to be on the order of 90 kips.

Micropiles

Micropiles are small-diameter, cast-in-place, cased piles that provide approximately the same capacity in compression and tension. Micropiles are typically constructed by drilling a cased hole into a bearing layer, placing a reinforcing bar to the bottom of the hole, and pumping grout to form a bond zone as the casing is withdrawn. The steel casing typically extends from the pile cap to slightly below the top of the bond zone to provide load transfer into the bond zone and structural rigidity at the top of the pile. The piles can also be installed on a batter to help resist lateral loading. Typical micropile diameters range from about 7⅝ inches to 9⅝ inches. Bond-zone lengths typically depend on the type of soils and the contractor's construction methods. Micropiles are typically designed by a specialty contractor. However, for the conditions existing at this site, we recommend a minimum bond-zone length of 10 feet established in the basalt that underlies the site. We assumed the permanent casing would extend into the basalt layer. The required depth into the basalt should be evaluated by the specialty contractor and reviewed by the design team. Based on our experience with similar subsurface conditions, we anticipate micropiles with a minimum diameter of 7⅝ inches can achieve a nominal geotechnical axial resistance of 20 kips/foot of bond zone in basalt. Based on our review of Section 10.5.5.2.5 of the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications (BDS) (AASHTO LRFD BDS), a resistance factor of 0.70 is appropriate for evaluating the factored axial resistance of single micropiles in compression and uplift, respectively, for the Strength Limit State. For the Extreme Limit State, resistance factors of 1.0 and 0.8 can be used to evaluate the factored axial resistance in compression and uplift, respectively.

Due to the relatively small diameter of micropiles, their lateral capacity is often limited, and lateral loads can be resisted by battered micropiles in axial compression or tension and passive earth pressure against embedded portions of pile caps. In our opinion, group effects for axial pile loading will be negligible if the micropiles are installed using a minimum center-to-center spacing of about 5D, where D is the pile diameter. We anticipate settlement of the micropiles in rock would be limited to the elastic shortening of the pile plus ¼ inch.

Pretensioned Rock Anchors

Rock-anchor installation and design is typically performed by a specialty foundation contractor, and installation procedures are very similar to those described above for micropiles. For the vertical rock anchors in these conditions, the rock-anchor loading is typically controlled by the structural capacity of the rock anchor bars. Rock anchors should have a minimum grouted length of 10 feet into the basalt. The grouted length may need to be modified based on the actual rock conditions encountered during drilling and the contractor's equipment and procedures. The anchors should be installed in holes with a minimum diameter of 6 inches. We anticipate the actual horizontal spacing of rock anchors will be based on structural requirements; however, we recommend a minimum center-to-center spacing of 3 feet. For conceptual level analyses, anchors installed as described above can be assumed to develop a nominal bond capacity of 20 kips/foot of bond zone.

To limit deflections and provide additional lateral resistance, we recommend rock anchors consist of steel bars rather than strands. All bars should be provided with double corrosion protection. We recommend a minimum grout compressive strength of 4,000 pounds per square inch. Verification and proof testing of the anchors should be completed in accordance with the most recent requirements of the Post-Tensioning Institute.

Drilled Piers

Based on the preliminary drawings, we understand drilled piers are being considered for foundation support of the approach bents for the arch bridge design at the Hawthorne Avenue crossing location. We understand that the drilled piers are planned to have a diameter of 3 feet and the maximum axial load on each drilled pier is estimated to be on the order of 90 kips.

Drilled cast-in-place concrete piers can be constructed by drilling a shaft or socket into the underlying soft to very hard (R2-R5) basalt, placing the steel reinforcement, and filling the shaft with concrete. Drilled piers develop their axial load capacity in skin-friction and end-bearing resistance. Skin friction is mobilized with relatively small amounts of vertical shaft movement compared to end-bearing resistance. Therefore, the allowable axial compressive capacity of drilled piers is somewhat dependent on the contractor's means and methods, as installation procedures can have a significant influence on the load-displacement behavior of the pier tip. GRI should review the proposed pier installation methods with respect to axial compressive capacities during design. We anticipate that the drilled piers socketed a minimum of 5 feet into soft or harder (≥R2) basalt will achieve axial capacity of at least 90 kips; however, the shafts will likely be closer to a minimum of 10 feet for lateral capacity considerations. The allowable capacities assume the piers will be installed with a center-to-center spacing of at least three pier diameters, 3D, where D is the pier diameter. The concrete and steel design may control the allowable capacities for the cast-in-place piers.

It should be anticipated that the drilled pile shafts may encounter loose fill or some perched water seeping through the more permeable zones within the overburdened soils and basalt. As a result, it would be prudent for the piling contractor to assume that casing may be necessary to retain the overburdened soils in at least some of the drilled shafts.

Retaining Walls

As discussed previously, the Greenwood Avenue retaining walls for Alternative 2 will likely include fills up to about 6 feet to more closely match the existing sidewalk elevations. Based on the available subsurface information, we anticipate rock will be relatively shallow and that both mechanically stabilized earth and cast-in-place walls will be reasonable alternatives for constructing Alternative 2 without significant settlement considerations of the adjacent structures.

Temporary Excavations

Based on our discussions with you regarding the grading for the alternatives, one of the more constrained temporary excavations will likely be adjacent to the two structures on the east side of the proposed Franklin Avenue Alternative 2. While rock is visible at the cut faces in significant portions of the existing Franklin Avenue cut, we recommend completing additional explorations in this vicinity of the structures to better evaluate the depth to rock and the potential need for temporary shoring. Rock excavation adjacent to the existing structures will also be a design and construction consideration. The selected rock excavation technique will likely require consideration of vibration impacts to existing structures and improvements and vibration monitoring and precondition surveys of existing structures may be appropriate.

Both alternatives at Franklin Avenue involve primarily fill walls on the downhill side of the proposed path (adjacent to the road) and cuts into the rock toward the adjacent properties. Proposed wall height for Alternative 1 may approach 10 feet. We anticipate soil nail or rock bolt walls may be a practical wall type for the proposed cuts. Some rock excavation will likely be needed to establish either cast-in-place, ultrablock, or mechanically stabilized earth walls for the proposed fills. The preferred wall types should be further evaluated after an alternative is selected.

Seismic Considerations

General

We understand the proposed crossing improvements will be seismically designed in accordance with current AASHTO LRFD BDS and ODOT requirements. The current ODOT Bridge Design and Drafting Manual (BDDM) and the ODOT Geotechnical Design Manual (GDM) require bridges to be designed to withstand seismic loading in accordance with the 2020 AASHTO Guide Specification for LRFD Seismic Bridge Design (AASHTO SBD), 9th edition, except as modified by the ODOT BDDM. Based on the AASHTO LRFD BDS, bridges must be designed for a “low probability of collapse but may suffer significant damage and disruption to service” in response to a 1,000-year return interval earthquake (7% probability of exceedance in 75 years). The ground-motion parameters for the 1,000-year return period design earthquake are based on the 2014 USGS seismic hazard maps (Petersen et al., 2014). The 1,000-year return interval “Life Safety” criteria require bridge foundation and approach fills to withstand the forces and soil displacements caused by the earthquake without collapse of any portion of the bridge structure.

A summary of the seismic parameters, including the zero-period peak ground-surface spectral acceleration, the 0.2- and 1.0-second coefficients for the 1,000-year hazard level for Site Class B conditions are provided for the project site (i.e., site coordinates of 45.0580° N and 121.3074° W) in Table 2 below.

Table 2: SPECTRAL VALUES AND SITE COEFFICIENTS

	Bedrock Spectral Values			Site Class B Site Coefficients			Site Class Adjusted Spectral Values		
Hazard Level	PGA, g	S_s, g	S_1, g	F_{PGA}	F_a	F_v	A_S	S_{DS}	S_{D1}
1,000-year	0.17	0.38	0.20	0.90	0.90	0.80	0.10	0.22	0.08

Other Seismic Design Considerations

Based on available information on subsurface conditions, the risk of damage from liquefaction and cyclic softening is low. It is our opinion that the major seismic hazard is ground shaking. Based on preliminary drawings indicating that the new crossing structures will be founded in the basalt bedrock, the risk of earthquake-induced slope instability and/or lateral spreading is low. Unless occurring on a previously unmapped or unknown fault, it is our opinion the risk of ground rupture at the site is very low. The potential for damage by tsunami and/or seiche at the site is absent.

LIMITATIONS

This memorandum has been prepared to aid the project team in the conceptual design of this project and without new subsurface information. The scope is limited to the specific project and locations described within this memorandum, and our description of the project represents our understanding of the significant aspects of the project relevant to earthwork and design and construction of the foundations. Additional explorations must be completed for the project to develop design-level recommendations. In the event that any changes in the conceptual-level design and location of the project elements as outlined in this memorandum are planned, we should be given the opportunity to review the changes and modify or reaffirm the conclusions and recommendations of this memorandum in writing.

Please contact the undersigned if you have any questions.

Submitted for GRI,

Scott M. Schlechter, PE, GE
Principal

Declan P. Schade, PE
Project Engineer

This document has been submitted electronically.

6637-A BEND CROSSING PRELIMINARY MEMORANDUM

REFERENCES

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