Geocal, Inc.

April 1, 2011

Scott Waterman, P.E.
Wilson & Company, Inc., Engineers & Architects
999 18th Street
Denver, CO 80202

RE: Preliminary Geotechnical Recommendations
Soil & Foundation Investigation
6th Avenue Bridge over BNSF, Denver, Colorado

Dear Scott:

This letter provides preliminary recommendations for the referenced 6th Avenue bridge project over the BNSF Railroad in CDOT's Region 6, Denver, Colorado. The recommendations contained are based on subsurface information detailed in drawings provided by the Division of Highways, dated February 1988 and for investigations conducted for Ramp H planned to be constructed north of this bridge project. Our recommendations are subject to change once the site specific subsurface investigation is completed.

Subsurface Conditions

The subsoils graphically shown on the drawing indicate that a layer of fill consisting of clayey sands to sandy clays was over relatively clean natural sands with trace gravel. Claystone Bedrock was encountered at about elevation 5180 feet. Ground water was encountered at about elevation 5192 feet. Ground surface was at about 5210 feet.

Preliminary Foundation Recommendations

Based on the subsurface conditions described a drilled shaft or driven pile foundation may be used for support of the structure. The following design and construction recommendations should be observed.

Drilled Shafts

1. Drilled shafts should be designed for ultimate end bearing pressure and side shear values as shown below for that portion of the shaft in competent claystone bedrock.

Ultimate End Bearing (psf)	Ultimate Side Shear (psf)
130,000	13,000

Using LRFD methodology, a resistance factor of 0.50 should be applied to end bearing, 0.55 should be applied to side shear, and 0.45 should be applied to side shear for uplift resistance. The ultimate capacity recommendations assume that a weighted load factor of 1.6 is used.

The above ultimate strength parameters are intended to correspond to the following Allowable Stress Design (ASD) capacities.

Allowable End Bearing (psf)	Allowable Side Shear (psf)	Allowable Side Shear Uplift (psf)
40,000	4,400	3,600

2. Some variation in the bedrock surface should be anticipated. Drilled shafts should penetrate at least 20 feet into claystone bedrock and have a minimum length of at least 25 feet for the upper capacities to be valid. These are geotechnical parameters, greater penetration depths may be needed based on the structural requirements.

3. The minimum spacing requirements between drilled shafts should be 3 diameters from center to center. At this spacing, no reduction in axial design parameters is required. Drilled shafts grouped less than 3 diameters center to center should be studied on an individual basis to evaluate the appropriate reduction in axial capacity.

4. Drilled shaft holes should be properly cleaned prior to placement of reinforcing steel or concrete. A maximum length to diameter ratio of 25 is recommended to facilitate cleaning and observation of the shaft hole.

5. Concrete utilized in the drilled shafts should be a fluid mix with sufficient slump so it will fill the voids between reinforcing steel and the shaft hole. Concrete with a slump in the range of 5 to 7 inches is recommended.

6. Casing and mud slurry will be required to reduce water infiltration and to help control caving. If water cannot be removed prior to placement of concrete, then concrete should be placed with an approved tremie method. The drilling contractor should be aware that water may be encountered in the bedrock as well as overburden soils. Concrete placement should occur after the hole has been well cleaned and approved. Concrete should not be placed through more than 2-inches of water.

7. A sufficient head of concrete should be maintained inside the casing during casing extraction to prevent voids being formed in the concrete upon casing removal. The concrete level should not be allowed to rise during casing removal. If it becomes apparent that voids may have formed during shaft installation, the contractor should be required to perform non-destructive tests to evaluate the continuity and integrity of the shaft. Tests may include sonic echo tests or other tests.

8. Bedrock penetration should be measured down from the bottom of the casing or top of competent bedrock, whichever is the lower elevation.

9. Care should be taken to prevent forming mushroom shapes at the top of the drilled shafts.

10. Concrete should be placed in the holes the same day they are drilled. The presence of water will most likely require concrete to be placed immediately after the shaft hole is completed. Failure to place concrete the day of drilling will result in degradation of bedrock capacity and a requirement for additional bedrock

penetration. The amount of additional bedrock penetration will be a function of how long the hole is left open and whether or not water accumulates during the inactive period. If holes are left open over night, this office should be contacted for additional bedrock penetration requirements.

11. The drilling contractor should mobilize equipment of sufficient size and operating condition to penetrate the materials and to achieve the required bedrock penetration.

12. Installation of drilled shafts should be observed by a representative of Geocal, Inc.

Driven Pile Foundation

Recommendations presented in this section are based on the "AASHTO LRFD Bridge Design Specifications" manual, the subsurface data described, our experience, and local geotechnical engineering practice. Installation of driven piles should be in accordance with Section 502 of *Standard Specifications for Road and Bridge Construction (2011)*, by the Colorado Department of Transportation.

1. Piles should consist of heavy steel H-sections driven into and supported by the underlying bedrock. A Pile Driving Analyzer (PDA) should be used to establish the pile driving refusal criteria. For preliminary design purposes, a combined side shear friction and end bearing ultimate capacity of 30,000 pounds per square inch (30 ksi) times the cross sectional area of the pile may be used for the Load and Resistance Factor Design (LRFD) method. The ultimate capacity assumes a weighted load factor of 1.6. A resistance factor of 0.45 should be applied. For the Allowable Stress Design (ASD) method, an allowable load equal to 9 ksi times the cross sectional area of the pile may be used for piles driven into the underlying bedrock. The above values are for f_y = 36 ksi steel H-piles.

2. For H-piles consisting of f_y = 50 ksi steel driven into bedrock, a combined side shear friction and end bearing ultimate capacity of 42,000 pounds per square inch (42 ksi) times the cross sectional area of the pile may be used for the Load and Resistance Factor Design (LRFD) method. The ultimate capacity assumes a weighted load factor of 1.6. A resistance factor of 0.45 should be applied. For the Allowable Stress Design (ASD) method, an allowable load equal to 12 ksi times the cross sectional area of the pile may be used.

3. Settlement of properly constructed driven piles is expected to be nominal, on the order of ½ inch or less.

4. H-piles are expected to encounter refusal within about 2 feet to 5 feet of the bedrock surface, although some variation in the bedrock surface elevation and penetration should be expected.

5. Penetration into the bedrock may vary and could be nominal. Therefore, uplift resistance should be limited to the soil-pile side shear above bedrock. Side shear capacity should be assumed zero "0" in the upper three feet to account for frost activity and surface disturbance. At three feet the side shear can be assumed to be an ultimate value of 500 psf. The ultimate value of 500 psf may be assumed constant for the remaining depth. A Resistance Factor of 0.25 should be applied. Pile and pile cap weights may be included in dead weight resistance to uplift forces.

6. Pile groups will require appropriate reductions of the axial capacities based on the effective envelope of the pile group. For axial and uplift, this reduction can be avoided by spacing the piles no closer than 3

diameters from center to center. Piles spaced closer than 3 diameters should be evaluated on an individual basis to establish the appropriate reduction in the design parameters.

7. The pile hammer should be operated at the manufacturer's recommended stroke when measuring penetration resistance. The pile capacity should be verified during construction by using a Pile Driving Analyzer (PDA). A minimum of two piles per structure should be monitored using a PDA, each at a separate foundation element (abutment or pier foundation).

8. The pile driving operation should be observed by qualified personnel on a full-time basis. Piles should be observed and checked for buckling, crimping and alignment in addition to recording penetration resistance and general pile driving operations.

Retaining Structure Earth Pressures

For this preliminary report, we have assumed that walls will be cast-in-place concrete and cantilevered. Retaining structures which are laterally supported and can be expected to undergo only a slight amount of deflection should be designed for lateral earth pressures based on the "at-rest" earth pressure condition. Cantilevered or gravity retaining structures which rotate and/or deflect sufficiently to mobilize the internal soil strength of the wall backfill may be designed for the "active" earth pressure condition. The following ultimate earth pressure coefficients are recommended for imported Class 1 material. Fine grained soils (i.e. clays and silts) produce excessive earth pressures on walls and are not recommended for use as structure backfill.

The following values assume placement and compaction in accordance with the CDOT standard specifications.

Material or location	Active (K_a)	At-Rest (K_o)	Passive (K_p)	γ_T – Unit Weight (pcf)	Friction Angle (ϕ), degrees
Imported Class 1	0.28	0.44	3.54	135	34

For granular backfill, lateral wall movements or rotation equal to 1% of the wall height is typically required to develop the full active case, whereas lateral movement equal to at least 2% of the wall height is normally required to establish full passive resistance. Suitable factors of safety should be applied to the above ultimate values to limit strain needed to reach ultimate strength, particularly in the case of passive resistance where large strains are needed to mobilize resistance. Imported material should meet CDOT Class 1 structure backfill grading requirements. Equivalent fluid unit weights may be taken as follows:

Above ground water: $\gamma_{eq} = \gamma_T \times K_{a,o,p}$

Below ground water: $\gamma_{eq} = (\gamma_T - 62.4) \times K_{a,o,p}$

where γ_T = soil total unit weight
$K_{a,o,p}$ = appropriate earth pressure coefficient

The above parameters are for a horizontal backfill and no surcharge load to the backfill. Retaining structures should be designed for appropriate surcharge pressures such as from traffic, etc. The buildup of water behind a wall or an upward sloping backfill surface will increase the lateral pressure imposed on retaining structures and should be accounted for. An under-drain should be provided to help reduce hydrostatic pressure buildup, unless the wall is designed to accommodate the additional pressure.

Limitations

This ***preliminary*** report has been prepared in accordance with generally accepted geotechnical engineering practices used in this area, and has been prepared for planning purposes. The conclusions and recommendations are based upon the data obtained from a review of previous information collected by others. The nature and extent of the variations adjacent to the borings may not become evident until our investigation is done or excavation is performed. If during construction, soil, bedrock, fill, or ground water conditions appear to be different from those described, this office should be advised so that re-evaluation of our recommendations may be made. Onsite observation of foundation bearing materials and testing of fill placement by a representative of this office is recommended.

If you have any questions, or if we can be of further service, please feel free to give me a call.

Sincerely,

GEOCAL, INC.

Ronald J. Vasquez, P.E.
Principal Engineer

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