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May 6, 2013

Summit, LLC
c/o Mr. Rick Everson, P.E.
1335 North 5900 East
Eden, Utah 84310

IGES Project No. 01628-005

Subject: Preliminary Geotechnical Recommendations (Revised)
Skier Bridge No. 1
Powder Mountain Resort
Weber County, Utah

Reference: IGES, Inc., 2012, Design Geotechnical Investigation, Powder Mountain Resort, Weber County, Utah, Project No. 01628-003, dated November 9, 2012

Mr. Everson:

As requested, IGES has prepared the following preliminary geotechnical recommendations for the proposed skier bridge to be constructed on a proposed new public road (Summit Pass) at approximate Station No. 96+00 within the Powder Mountain Resort, Weber County, Utah. In preparation of these preliminary recommendations we have reviewed the geotechnical engineering report for the 200-acre resort expansion project, of which the bridge is a part (IGES, 2012).

This report presents preliminary recommendations for construction of the bridge structure *for planning purposes only*; the recommendations presented herein are subject to validation and possible revision based on future site-specific subsurface exploration, which will presumably occur once the site becomes accessible after the snow has melted at the site. This report has been revised from the original document dated May 2, 2013 to incorporate new information about the bridge foundation loads and dimensions.

Project Description

Our understanding of the project is based on the plan set prepared by Mulholland Development Solutions titled “Skier Bridge 1” (Sheets 1 through 7), dated April 23, 2013, and information provided by The Client. We understand that a new bridge will be constructed over the proposed Summit Pass roadway to accommodate a proposed ski run that will pass over the road. Based on our review of the plans we understand that the new bridge will consist of a precast CON/SPAN arch founded on conventional spread footings. The approximately 36-foot wide bridge is expected to have a 14-foot tall clearance to accommodate most types of automobiles and common service-type vehicles. The bridge abutments will be supported by conventional spread footings; we understand that the footings will be 43 feet long and 11½ feet wide and will induced a net bearing pressure of 3,500 psf.

The opening of the undercrossing will be flanked on either side by MSE walls; we understand that the MSE walls will be designed by others. There will also be several rockeries associated with the bridge; IGES will provide recommendations for the rockeries in a separate submittal. We also understand that existing grade will be lowered approximately 5 feet to attain final design grade under the bridge.

Review of Subsurface Data

In preparation of these preliminary recommendations we have reviewed the geotechnical engineering report for the 200-acre resort expansion project, of which the bridge is a part (IGES, 2012). The referenced study included two test pits completed within a few hundred feet of the proposed bridge structure. The subsurface conditions are summarized below:

- TP-17 is located a few hundred feet south/downhill from the project site; the test pit log indicates that the upper 10 feet consists largely coarse, medium dense Clayey GRAVEL (GC). Underlying the gravel (below 10 feet), the soils transition to medium stiff Fat CLAY (CH). No groundwater was encountered.

- TP-18 is located a few hundred feet east of the project site; the test pit log indicates that highly weathered bedrock (limestone) was encountered approximately 6 feet below existing grade. No groundwater was encountered.

Foundation Recommendations

Considering the potential presence of compressible clays at the foundation subgrade, the footings for the proposed bridge structure should be founded on a minimum of 2 feet of structural fill. We recommend that IGES inspect the bottom of the foundation excavation prior to the placement of structural fill, steel or concrete to identify any unsuitable or otherwise deleterious soils – additional over-excavation may be necessary based on actual subgrade conditions. Over-excavation of the foundation subgrade should extend 1 foot laterally for every foot of depth.

If competent limestone bedrock, or dense granular soil, is exposed across the entire foundation excavation, then the minimum section of structural fill may be reduced to 1 foot (subject to written approval by IGES).

All foundations exposed to the full effects of frost should be established at a minimum depth of 42 inches below the lowest adjacent final grade.

The preceding recommendations are intended to limit total static settlement to 1 inch or less.

Earth Pressures and Lateral Resistance

Lateral forces imposed upon conventional foundations due to wind or seismic forces may be resisted by the development of passive earth pressures and friction between the base of the footing and the supporting soils. In determining the frictional resistance against concrete, a coefficient of friction of 0.45 for granular structural fill should be used.

Ultimate lateral earth pressures from *granular* backfill acting against buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table.

Table 1
Lateral Earth Pressure Coefficients

Condition	Level Backfill	
	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.33	40
At-rest (Ko)	0.5	60
Passive (Kp)	3	360

These coefficients and densities assume no buildup of hydrostatic pressures. The force of the water should be added to the presented values if hydrostatic pressures are anticipated.

Clayey soils drain poorly and may swell upon wetting, thereby greatly increasing lateral pressures acting on earth retaining structures; therefore, clayey soils should not be used as retaining wall backfill or backfill against foundation walls. Backfill should consist of either native granular soil or sandy imported material with an Expansion Index (EI) less than 20.

Wall-type structures allowed to rotate slightly should use the active condition. If the element is constrained against rotation, the at-rest condition should be used. These values should be used with an appropriate factor of safety against overturning and sliding. A value of 1.5 is typically used. Additionally, if passive resistance is calculated in conjunction with frictional resistance, the passive resistance should be reduced by ½.

Seismic Considerations

The spectral accelerations presented in Table 2 are calculated based on the site's approximate latitude and longitude of 41.3685˚ and -111.7602˚ respectively. For AASHTO bridge design, a spectral acceleration corresponding to a 7PE75 event is typically prescribed without further modification (consult the appropriate AASHTO controlling document for guidance).

Table 2
Spectral Acceleration Design Parameters

Design Seismic Event	Source	Class C Site Coefficients			Spectral Acceleration (g)		
		F_a	F_v	F_{pga}	As	0.2 sec.	1.0 sec.
7PE75	AASHTO 2009[a]	1.156	1.587	1.148	0.289	0.705	0.338

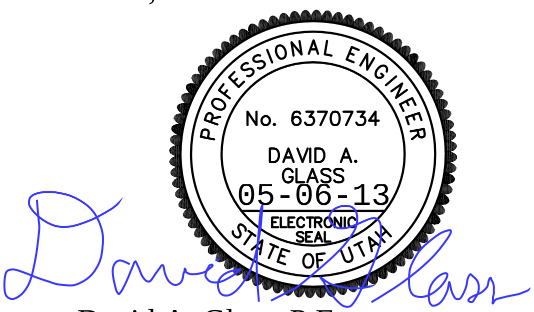
[a]AASHTO 2009, U.S. Design Maps online ground motion calculator, available at the USGS website: http://geohazards.usgs.gov/designmaps/us/application.php, based on the USGS 2002 fault database.

Closure

The recommendations presented herein supersede the recommendations presented in our referenced geotechnical report (IGES, 2012). All other recommendations presented in our referenced report remain valid and should be implemented into the design and construction of the project.

We appreciate the opportunity to provide you with our services. If you have any questions please contact the undersigned at your convenience (801) 748-4044.

Respectfully Submitted,
IGES, Inc.



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