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GEOTECHNICAL AND GEOLOGIC HAZARD INVESTIGATION
Lot 80R of Summit Eden Phase 1C
8483 E. Spring Park
Summit Powder Mountain Resort
Weber County, Utah

IGES Project No. 03092-001

July 12, 2019

Prepared for:

Mr. Ryan Byrne

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE OF WORK

This report presents the results of a geotechnical and geologic hazards investigation conducted for Lot 80R of Summit Eden Phase 1C, part of the currently on-going expansion at the Powder Mountain Ski Resort in Weber County. The purpose of our investigation was to assess the nature and engineering properties of the subsurface soils at the project site and to provide recommendations for the design and construction of foundations, grading, and drainage. In addition, geologic hazards have been assessed for the property. The scope of work completed for this study included literature review, site reconnaissance, subsurface exploration, engineering analyses, and preparation of this report.

Our services were performed in accordance with our proposal dated May 6, 2019, and your signed authorization. The recommendations presented in this report are subject to the limitations presented in the "Limitations" section of this report (Section 6.1).

1.2 PROJECT DESCRIPTION

Our understanding of the project is based primarily on our previous involvement with the Summit Powder Mountain Resort project, which included two geotechnical investigations for the greater 200-acre Powder Mountain Resort expansion project (IGES, 2012a and 2012b), as well as a number of lot-specific and site-specific geotechnical and geologic hazard investigations in various locations across the greater Powder Mountain Resort expansion area. The project site is located within the Summit Powder Mountain Resort, illustrated on the *Site Vicinity Map*, Figure A-1 in Appendix A.

The Summit Powder Mountain Resort expansion project is located southeast of SR-158 (Powder Mountain Road), south of previously developed portions of Powder Mountain Resort, in unincorporated Weber County, Utah. The Summit Powder Mountain project area is accessed by Powder Ridge Road. Lot 80R is located within Phase 1C of the Powder Mountain expansion project (Summit Eden), on the south side of Spring Park – the street address is 8483 E. Spring Park. The 0.347-acre residential lot has an approximate buildable area (building envelope) of 4,300 square feet. The proposed improvements will include a single-family home, presumably a high-end vacation home, with associated improvements such as utilities and hardscape. Construction plans were not available for our review; however, based on the architectural drawings provided by Scandinavian, the new home will be a three-level structure, the lowest level consisting of a partial walk-out basement, founded on conventional spread footings.

2.0 METHODS OF STUDY

2.1 LITERATURE REVIEW

2.1.1 Geotechnical

The earliest geotechnical report for the area is by AMEC (2001), which was a reconnaissance-level geotechnical and geologic hazard study. IGES later completed a geotechnical investigation for the Powder Mountain Resort expansion in 2012 (2012a, 2012b). Our previous project-wide work included twenty-two test pits and one soil boring excavated at various locations across the 200-acre development. IGES has performed single-lot geotechnical and geologic hazard investigations for nearby projects, the closest being Lot 79R (IGES, 2018a), located just west of Lot 80R. As a part of this current study, the logs from relevant nearby test pits and other data from our previous reports were reviewed.

2.1.2 Geological

Several pertinent publications were reviewed as part of this assessment. Sorensen and Crittenden, Jr. (1979) provides 1:24,000 scale geologic mapping of the Huntsville Quadrangle, and Crittenden, Jr. (1972) provides 1:24,000 scale geologic mapping of the Brown's Hole Quadrangle. Coogan and King (2001) provide more recent geologic mapping of the area, but at a 1:100,000 scale. An updated Coogan and King (2016) regional geologic map (1:62,500 scale) provides the most recent published geologic mapping that covers the project area. Western Geologic (2012) conducted a reconnaissance-level geologic hazard study for the greater 200-acre Powder Mountain expansion project, including the Lot 80R area. The Western Geologic (2012) study modified some of the potential landslide hazard boundaries that had previously been mapped at a regional scale (1:100,000) by Coogan and King (2001) and Elliott and Harty (2010). The corresponding United States Geological Survey (USGS) topographic maps for the Huntsville and Brown's Hole Quadrangles (2017) provide physiographic and hydrologic data for the project area. Regional-scale geologic hazard maps pertaining to landslides (Elliott and Harty, 2010; Colton, 1991), faults (Christenson and Shaw, 2008a; USGS and Utah Geological Survey (UGS), 2006), debris-flows (Christenson and Shaw, 2008b), and liquefaction (Christenson and Shaw, 2008c; Anderson et al., 1994) that cover the project area were also reviewed. The Quaternary Fault and Fold Database (USGS and UGS, 2006), was reviewed to identify the location of proximal faults that have had associated Quaternary-aged displacement.

Stereo-paired aerial imagery for the project site, recent and historic Google Earth imagery, and lidar imagery was also reviewed to assist in the identification of potential adverse geologic conditions. The aerial photographs reviewed are documented in the *References* section of this report.

2.2 FIELD INVESTIGATION

Subsurface soils were investigated by excavating a single test pit within the property boundary. The approximate location of the test pit is illustrated on the *Geotechnical & Geology Map* (Figure A-2 in Appendix A). The soil types were visually logged at the time of our field work in general accordance with the *Unified Soil Classification System* (USCS). Soil classifications and descriptions are included on the test pit log, presented as Figure A-3 in Appendix A. A key to USCS symbols and terminology is included as Figure A-4, and a key to physical rock properties is included as Figure A-5.

2.3 LABORATORY TESTING

Samples retrieved during the subsurface investigation were transported to the IGES laboratory for evaluation of engineering properties. Specific laboratory tests included:

- Grain-Size Distribution (ASTM D6913)
- Fines Content (ASTM D1140)
- Moisture Content (ASTM D7263)
- Atterberg Limits (ASTM D4318)
- Direct Shear (ASTM D3080)

Results of the laboratory testing are discussed in this report and presented in Appendix B.

3.0 GEOLOGIC CONDITIONS

3.1 GENERAL GEOLOGIC SETTING

The Lot 80R property is situated in the western portion of the northern Wasatch Mountains, approximately 4 miles northeast of Ogden Valley. The Wasatch Mountains contain a broad depositional history of thick Precambrian and Paleozoic sediments that have been subsequently modified by various tectonic episodes that have included thrusting, folding, intrusion, and volcanics, as well as scouring by glacial and fluvial processes (Stokes, 1987). The uplift of the Wasatch Mountains occurred relatively recently during the Late Tertiary Period (Miocene Epoch) between 12 and 17 million years ago (Milligan, 2000). Since uplift, the Wasatch Front has seen substantial modification due to such occurrences as movement along the Wasatch Fault and associated spurs, the development of the numerous canyons that empty into the current Salt Lake Valley and Utah Valley and their associated alluvial fans, erosion and deposition from Lake Bonneville, and localized mass-movement events (Hintze, 1988).

The Wasatch Mountains, as part of the Middle Rocky Mountains Province (Milligan, 2000), were uplifted as a fault block along the Wasatch Fault (Hintze, 1988). Ogden Valley itself is a fault-bounded trough that was occupied by Lake Bonneville (Sorensen and Crittenden, Jr, 1979) before being cut through by the Ogden River and subsequently dammed to form the Pineview Reservoir.

The Wasatch Fault and its associated segments are part of an approximately 230-mile long zone of active normal faulting referred to as the Wasatch Fault Zone (WFZ), which has well-documented evidence of late Pleistocene and Holocene (though not historic) movement (Lund, 1990; Hintze, 1988). The faults associated with the WFZ are almost all normal faults, exhibiting block movement down to the west of the fault and up to the east. The WFZ is contained within a greater area of active seismic activity known as the Intermountain Seismic Belt (ISB), which runs approximately north-south from northwestern Montana, along the Wasatch Front of Utah, through southern Nevada, and into northern Arizona. In terms of earthquake risk and potential associated damage, the ISB ranks only second in North America to the San Andreas Fault Zone in California (Stokes, 1987).

The WFZ consists of a series of ten segments of the Wasatch Fault that each display different characteristics and past movement and are believed to have movement independent of one another (UGS, 1996). The Lot 80R property is located approximately 10.15 miles to the northeast of the Weber Segment of the Wasatch Fault, which is the closest documented Holocene-aged (active) fault to the property and trends north-south along the Wasatch Front (USGS and UGS, 2006).

3.2 SURFICIAL GEOLOGY

According to Sorensen and Crittenden, Jr. (1979), the property is entirely underlain by the undivided Tertiary/Cretaceous Wasatch and Evanston Formations (map unit TKwe), described as "unconsolidated pale-reddish-brown pebble, cobble, and boulder conglomerate, forms boulder-covered slopes. Clasts are mainly Precambrian quartzite and are tan, gray, or purple; matrix is mainly poorly consolidated sand and silt." A generalized bedding attitude shows this unit striking due north and dipping 10 degrees to the east. This map forms the basemap for the *Regional Geology Map 1* (Figure A-6). Coogan and King (2001) shows the property to be underlain by mass-movement deposits, described as "slides, slumps, and flows, as well as colluvium, talus, and alluvial fans that are mostly debris flows." Western Geologic (2012) identified a number of landslide deposits contained within the Powder Mountain Resort expansion area (*Regional Geology Map 2,* Figure A-7). In this map, the property is located within mapped landslide deposits described as "mixed slope colluvium, shallow landslides, and talus." A large Holocene to Late Pleistocene-aged landslide deposit is also mapped immediately south of the southern margin of the property. Finally, Coogan and King (2016) updated their 2001 map, which shows the property to be entirely located within the northeastern end of a large lobe of landslide deposits (map unit Qms), described as "poorly sorted clay- to boulder-sized material; includes slides, slumps, and locally flows and floods; generally characterized by hummocky topography, main and internal scarps, and chaotic bedding in displaced blocks" (see *Regional Geology Map 3*, Figure A-8). The northern margin of the property is mapped as at or very near the contact between the landslide deposits (to the south) and the Wasatch Formation (to the north; map unit Tw). A nearby bedding attitude shows the Wasatch Formation to be striking nearly due north and dipping at 5 degrees to the east.

Previous geotechnical and geologic hazard investigations have been performed by IGES for nearby lots, including the adjacent Lot 79R (IGES, 2018a) and Lot 82R (IGES, 2017a). The test pit excavated for Lot 79R found a thin (~4 to 6-inch thick) topsoil forming upon a 1 to 1.5-foot thick foot thick loose, cobbly colluvium unit, which was in turn underlain by poorly consolidated Wasatch Formation consisting of clayey sand with gravel and clayey gravel with sand.

3.3 HYDROLOGY

The USGS topographic maps for the Huntsville and Brown's Hole Quadrangles (2017) show that the Lot 80R project area is situated on a slope, with the local topographic gradient down to the southwest towards a larger west-trending ephemeral drainage[1] locally known as Lefty's Canyon (see Figure A-1). A small ephemeral stream drainage passes northeast-southwest along the southern margin of the property, which passes downslope to the southwest and empties into Lefty's Canyon. No springs are known to occur on the property, though it is possible that springs may occur on various parts of the property during peak runoff. Groundwater seepage is known to occur at the base of the slope at the road cut along the southern margins of Lots 74R and 75R (IGES, 2017b).

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[1] Ephemeral stream: A stream or reach of a stream that flows briefly only in direct response to precipitation in the immediate locality and whose channel is at all times above the water table. (AGI, 2005)

Baseline groundwater depths for the Lot 80R property are currently unknown, but are anticipated to fluctuate both seasonally and annually. A known spring is located approximately 600 feet south of the property (see Figure A-1); it is possible that the Lot 74R and 75R excavations have intersected the hydrologic pathway for this spring. Groundwater seepage was encountered in the test pit excavated in this investigation at a depth of 8.5 feet below existing grade from the northern end of the pit, with a possible potentiometric surface[2] (water table) encountered at 16 feet below existing grade when potholing the southern portion of the test pit.

3.4 GEOLOGIC HAZARDS FROM LITERATURE

Based upon the available geologic literature, regional-scale geologic hazard maps that cover the Lot 80R project area have been produced for landslide, fault, debris-flow, and liquefaction hazards. The following is a summary of the data presented in these regional geologic hazard maps.

3.4.1 Landslides

Two regional-scale landslide hazard maps have been produced that cover the project area. Colton (1991) does not show the property to be underlain by or adjacent to landslide deposits, though south and west-trending landslide deposits are noted nearby to the west and south. Elliott and Harty (2010) show the property to be located within mapped landslide deposits described as "Landslide undifferentiated from talus and/or colluvial deposits." As noted above, both Western Geologic (2012; Figure A-7) and Coogan and King (2016; Figure A-8) show the property to be located within mapped landslide deposits.

Notably, in more site-specific studies, landslide deposits were not observed in the test pits excavated for the nearby Lot 79R and Lot 82R properties (IGES, 2018a; IGES, 2017a), though possible landslide deposits had been identified across Lot 80R based on surficial morphology (IGES, 2017a).

3.4.2 Faults

Neither Christenson and Shaw (2008a) nor the Quaternary Fault and Fold Database of the United States (USGS and UGS, 2006) show any Quaternary-aged (~2.6 million years ago to the present) faults to be present on or projecting towards the subject property. The Weber County Natural Hazards Overlay Districts defines an active fault to be "a fault displaying evidence of greater than four inches of displacement along one or more of its traces during Holocene time (about 11,000 years ago to the present)" (Weber County, 2015). The closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 10.15 miles southwest of the western margin of the property (USGS and UGS, 2006).

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[2] Potentiometric Surface: A surface representing the total head of groundwater and defined by the levels to which water will rise in tightly cased wells. The water table is a particular potentiometric surface. (AGI, 2005)

3.4.3 Debris Flows

Christenson and Shaw (2008b) do not show the project area to be located within a debris-flow hazard special study area.

3.4.4 Liquefaction

Anderson, et al. (1994) and Christenson and Shaw (2008c) both show the project area to be located in an area with very low potential for liquefaction.

3.5 REVIEW OF AERIAL IMAGERY

A series of aerial photographs that cover project area were taken from the UGS Aerial Imagery Collection (UGS, 2019) and analyzed stereoscopically for the presence of adverse geologic conditions across the property. This included a review of photos collected from the years 1946, 1952, and 1963. A table displaying the details of the aerial photographs reviewed can be found in the *References* section at the end of this report.

No geologic lineaments, fault scarps, landslide headscarps, or landslide deposits were observed on the subject property in the aerial photography.

Google Earth imagery of the property from between the years of 1993 and 2018 was also reviewed. No landslide or other geological hazard features were noted in the imagery. Preceding the installation of Spring Park Road, the property was observed to be densely covered low-lying bushes. A southwest-northeast trending two-track road was put through the north-central portion of the lot between December of 2005 and July of 2006. No notable changes to the property, either human or natural, were observed in the aerial imagery between July of 2006 and when Spring Park Road was cut in between September of 2011 and October of 2014. During this time, some of the northern portion of the property was disturbed as part of the excavation and covered in fill.

UGS 2015-2017 0.5-meter LiDAR data that covers the project area was reviewed. This imagery showed the human disturbance across the property in the form of Spring Park Road and the northeast-trending two-track road that passes through the northern part of the property. The ephemeral drainage along the southern margin of the property was clearly discernible. No distinct landslide deposits or other adverse geologic conditions were observed on the property, though irregular, possibly hummocky topography was observed to be present across and immediately to the southwest of the property.

3.6 SEISMICITY

Following the criteria outlined in the 2018 International Building Code (IBC, 2018), spectral response at the site was evaluated for the *Maximum Considered Earthquake* (MCE) which equates to a probabilistic seismic event having a two percent probability of exceedance in 50 years (2PE50). Spectral accelerations were determined based on the location of the site using the *ASCE-*

7 Hazard Tool; this software incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey. These maps have been incorporated into the *International Building Code* (IBC) (International Code Council, 2018).

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site Class is a parameter that accounts for site amplification effects of soft soils and is based on the average shear wave velocity of the upper 100 feet (30 meters, Vs_{30}); site classifications are identified in Table 3.6a.

Table 3.6a
Site Class Categories

Site Class	Earth Materials	Shear Wave Velocity Range (Vs_{30}) m/s
A	Hard Rock	>1,500
B	Rock	760-1,500
C	Very Dense Soil/Soft Rock	360-760
D	Stiff Soil	180-360
E	Soft Soil	<180
F	Special Soils Requiring Site-Specific Evaluation (e.g. liquefiable)	n/a

Based on our field exploration and our understanding of the geology in this area, including explorations made for other nearby sites (IGES, 2017a and 2018a), the site is underlain by older landslide deposits derived from poorly consolidated Tertiary-aged conglomeratic bedrock of the Wasatch Formation and at depth by the Calls Fort Shale Member of the Bloomington Formation, and would reasonably be expected to classify as Site Class C or possibly B. IGES has reviewed shear wave velocity measurements performed for the greater Summit Powder Mountain project (PSI, 2012); this data was obtained in similar geologic conditions just west of the project site. The shear wave velocity data indicates that the B/C boundary is located between 25 and 50 feet below existing grade across much of the Powder Mountain area, with a maximum recorded shear wave velocity of 3,000 fps below this interface. Based on this information and considering that the proposed home could conceivably be underlain by as much as 10 feet of surficial soils overlying bedrock, the site is appropriately categorized as Site Class C (measured). Based on the assumed Site Class C site coefficients, the short- and long-period *Design Spectral Response Accelerations* are presented in Table 3.6b. For geotechnical practice, the geo-mean peak ground acceleration (PGA_M) is presented in Table 3.6c. A summary of the ASCE-7-16 data output is presented in Appendix C.

Table 3.6b
Spectral Accelerations for MCE, Risk-Targeted Values (Structural)

Mapped B/C Boundary S_a (g)		Site Coefficient (Site Class C)		Design Sa (g)		
Ss	S_1	F_a	F_v	PGA	S_{DS}	S_{D1}
0.802	0.277	1.2	1.5		0.642	0.277

1) T_L=8
2) Cv=1.051
3) Seismic Design Category D for Risk Categories I, II, and III

Table 3.6c
Spectral Accelerations for MCE, Geo-Mean Values (Geotechnical)

Mapped B/C Boundary PGA (g)	Site Coefficient F_{PGA} (Site Class C)	PGA_M (g)
0.349	1.2	0.419

3.7 GEOLOGIC HAZARDS ASSESSMENT

Geologic hazards assessments are necessary to determine the potential risk associated with particular geologic hazards that are capable of adversely affecting a proposed development area. As such, they are essential in evaluating the suitability of an area for development and provide critical data in both the planning and design stages of a proposed development. The geologic hazard assessment discussion below is based upon a qualitative assessment of the risk associated with a particular geologic hazard, based upon the data reviewed and collected as part of this investigation.

A “low” hazard rating is an indication that the hazard is either absent, is present in such a remote possibility so as to pose limited or little risk, or is not anticipated to impact the project in an adverse way. Areas with a low-risk determination for a particular geologic hazard do not require additional site-specific studies or associated mitigation practices with regard to the geologic hazard in question.

A “moderate” hazard rating is an indication that the hazard has the capability of adversely affecting the project at least in part, and that the conditions necessary for the geologic hazard are present in a significant, though not abundant, manner. Areas with a moderate-risk determination for a particular geologic hazard may require additional site-specific studies, depending on location and construction specifics, as well as associated mitigation practices in the areas that have been identified as the most prone to susceptibility to the particular geologic hazard.

A “high” hazard rating is an indication that the hazard is very capable of or currently does adversely affect the project, that the geologic conditions pertaining to the particular hazard are present in abundance, and/or that there is geologic evidence of the hazard having occurred at the area in the historic or geologic past. Areas with a high-risk determination always require additional site-specific hazard investigations and associated mitigation practices where the location and construction specifics are directly impacted by the hazard. For areas with a high-risk geologic hazard, simple avoidance is often considered.

The following is a summary of the geologic hazard assessment for the Lot 80R property.

3.7.1 Landslides/Mass-Movement

According to the several most recent geologic maps produced that cover the property, the property is situated within mapped landslide deposits (Coogan and King, 2016; Western Geologic, 2012; Elliott and Harty, 2010). Previous site-specific mapping of the area had identified possible landslide deposits across the Lot 80R property (IGES, 2017a), and an irregular surficial morphology was observed in the lidar imagery.

Additionally, landslide deposits were observed in the subsurface, extending to a depth of at least 20 feet below existing grade and likely to around 25 feet below existing grade. Notably, these deposits were clast-rich and gradational between sandy fat clay with gravel and clayey gravel with sand and appear to be derived from weathered Wasatch Formation bedrock or colluvium. No slickensides or other evidence of shear was observed in the upper portion of the unit, though the unit displayed a heterogeneity in composition and appearance. The basal 3 feet of the exposed portion of the deposits was a distinct light brownish gray fat clay seam with a very high plasticity. This high plasticity fat clay material was very similar to material observed by IGES in the subsurface in the more southerly part of the Powder Mountain Resort area in other investigations, being identified as the uppermost, highly weathered portion of the Calls Fort Shale Member of the Bloomington Formation in those areas. This clay may represent a basal slide plane along which the landslide material moved in the past, and notably this clay was saturated by groundwater at the time of the subsurface investigation.

However, the landslide deposits are considered to be older (Pleistocene-aged), and part of a smaller lobe that terminates to the west in the northern portion of Lot 79R (see Figure A-2). This age determination was made based upon the fact that a small drainage has developed along the eastern margin of the deposits, with some of the alluvial deposits overlying the landslide deposits (see Figure A-3). Additionally, the landslide deposits appear to be overlain by a thin (3 to 3.5-feet thick) veneer of colluvium, and the surficial morphology is more subdued than what is typical of young landslide deposits. The scale and areal extent of the landslide deposits is limited by the fact that the landslide deposits were not observed in the test pit excavated on the adjacent Lot 79 property (IGES, 2018a).

Given the geologic data alone, the risk associated with landslides is considered to be moderate to high. However, the site is located on a largely gentle grade (~7H:1V) (Horizontal:Vertical), and slope stability modeling performed as part of our assessment indicates that the slope is stable under current *static* conditions, although some slope deformation can be expected under *seismic* conditions (see Section 4.3). The slope stability modeling therefore reduces the landslide hazard risk classification for the property to be moderate.

3.7.2 Rockfall

Though the property is on a slope, no bedrock outcrops are exposed upslope of the property. As such, the rockfall hazard associated with the property is considered to be low.

3.7.3 Surface-Fault Rupture and Earthquake-Related Hazards

No faults are known to be present on or project across the property, and the closest active fault to the property is the Weber Segment of the Wasatch Fault Zone, located approximately 10.15 miles to the west of the property (USGS and UGS, 2006). Given this information, the risk associated with surface-fault-rupture on the property is considered low.

The entire property is subject to earthquake-related ground shaking from a large earthquake generated along the active Wasatch Fault. Given the distance from the Wasatch Fault, the hazard associated with ground shaking is considered to be moderate. Proper building design according to appropriate building code and design parameters can assist in mitigating the hazard associated with earthquake ground shaking.

3.7.4 Liquefaction

The site is underlain in part by the Wasatch Formation, a poorly consolidated sedimentary rock unit (conglomerate), and likely the Calls Fort Shale Member of the Bloomington Formation. Rock units such as these are not considered susceptible to liquefaction; as such, the potential for liquefaction occurring at the site is considered low.

3.7.5 Debris-Flows and Flooding Hazards

The property contains an ephemeral drainage that passes northeast-southwest along the southern margin of the lot. However, there are no debris-flow source areas upslope of the property, site grading is anticipated to utilize the preexisting drainage to funnel stormwater away from the proposed residence. Given these conditions, the debris-flow and flooding hazard associated with the property is considered to be low.

3.7.6 Shallow Groundwater

Groundwater seepage was observed at a depth of approximately 8.5 feet below existing grade on the north end of the test pit. A possible potentiometric surface was encountered at a depth of approximately 16 feet below existing grade when the test pit was potholed to a depth of 20 feet in the southern part of the test pit. The test pit was excavated in early June, and the groundwater level was likely to be at or near its annual high. However, no springs were observed on the property, and no plants indicative of persistent shallow groundwater conditions were observed on the property.

Given the existing data, it is expected that groundwater levels will fluctuate both seasonally and annually, and the risk associated with shallow groundwater hazards is considered moderate to high. Spring thaw and runoff are likely to significantly contribute to elevated groundwater conditions (localized perched conditions). However, shallow groundwater issues can be mitigated through appropriate grading measures and/or the avoidance of the construction of basement levels, or constructing basements with foundation drains.

4.0 GENERALIZED SITE CONDITIONS

4.1 SITE RECONNAISSANCE

Mr. Peter E. Doumit, P.G., C.P.G., of IGES conducted reconnaissance of the site and the immediate adjacent properties on June 7, 2019. The site reconnaissance was conducted with the intent to assess the general geologic conditions present across the property, with specific interest in those areas identified in the geologic literature and aerial imagery reviews as potential geologic hazard areas. Additionally, the site reconnaissance provided the opportunity to geologically map the surficial geology of the area. Figure A-2 is a site-specific geologic map of the Lot 80R property and adjacent areas.

At the time of the site reconnaissance, the property was observed to be gently sloping downhill to the south. The ground surface was observed to be irregular, and patchily covered in small bushes and grasses. The northernmost approximately 10 to 15 feet of the lot was observed to consist of a fill slope extending north to Spring Park Road. The southwestern side of the lot was still covered in approximately 6 to 8 inches of snow at the time of the site reconnaissance. The southwest-trending ephemeral drainage was observed along the southern margin of the property, and a cluster of aspen trees was observed on the southern side of the drainage that exhibited strong soil creep downslope to the southwest. The drainage was not actively flowing with water during the site visit.

Variously-sized boulders and cobbles were found scattered across the surface of the property. These were typically subrounded to subangular, and were found to be as large as 7.5 feet in diameter, though were most commonly between 6 and 8 inches in diameter. The rock clasts[3] were found to be comprised entirely of pale yellowish orange to medium gray, granular to amorphous quartzite. The clasts were interpreted at the time to be part of a surficial colluvial geologic unit derived from weathered Wasatch Formation.

No springs, seeps, or running water were observed on the property at the time of the site visit. The ground surface appeared to be have been in part disturbed by human activity, especially in the northern half of the lot. No adverse geologic conditions were observed on the property at this time.

4.2 SUBSURFACE CONDITIONS

On June 7, 2019, one exploration test pit was excavated in the south-central portion of the lot (see Figure A-2). The test pit was excavated to a depth of 11.5 feet below existing grade and subsequently potholed to a depth of 20 feet below existing grade on the southern end of the test pit with the aid of a Doosan DX 340 LC-HD tracked excavator. Upon completion of logging, the test pit was backfilled without engineered compaction controls. A detailed log for the test pit is

[3] Clast: An individual constituent, grain, or fragment of a sediment or rock, produced by the mechanical or chemical disintegration or a larger rock mass. (AGI, 2005)

displayed in Figure A-3. Four distinct geologic units were encountered in the subsurface. The soil and moisture conditions encountered during our investigation are discussed in the following paragraphs.

4.2.1 Earth Materials

A/B Soil Horizon: This topsoil unit was found to be between approximately 1½ and 2 feet thick. The unit was a dark yellowish brown to brownish black, medium stiff, moist, lean CLAY with gravel (CL), with gravel and larger-sized quartzite clasts comprising approximately 20% of the unit. The topsoil contained abundant plant and tree roots and was found to be forming upon the underlying colluvium or alluvium unit.

Colluvium (Qc): This unit was approximately 3 to 3½ feet thick. The unit consisted of a dark yellowish brown, medium stiff, moist, sandy lean CLAY with gravel (CL). Gravel and larger-sized subrounded to subangular quartzite clasts comprised between approximately 30 and 40% of the unit. Individual clasts were as much as 14 inches in diameter, though the mode clast size was approximately 4 to 6 inches in diameter. On the northern end of the test pit, this unit was noted to have an increased clast concentration, with some voids observed between clasts. It is possible that this unit may represent a weathered portion of the underlying older landslide deposits, or be the remnant of an old talus slope.

Alluvium (Qac): This unit was approximately 2½ to 3 feet thick. The unit consisted of a dark yellowish brown, medium dense, moist, clayey GRAVEL with sand (GC). Gravel and larger-sized subrounded quartzite clasts comprised between approximately 40 and 60% of the unit. Individual clasts were as much as 2 feet in diameter, though the mode clast size was approximately 8 inches in diameter. This unit coincided with the transition into the ephemeral drainage in the southern part of the lot.

Older Landslide (Qlso): This unit was at least 13.5 feet thick and extended to the maximum depth of exploration within the test pit, including when potholing to a depth of 20 feet below existing grade. The uppermost approximately 10 feet of the unit consisted of a heterogeneous mixture of moderate reddish brown to dark reddish brown, medium dense, moist to wet clayey GRAVEL with sand (GC) gradational to sandy fat CLAY with gravel (CH). Gravel and larger-sized subrounded to subangular quartzite clasts comprised between approximately 30 and 50% of the unit, with individual clasts up to 14 feet in diameter and a mode clast size of 4 to 6 inches in a wide range of clast sizes. The basal 3 feet exposed after potholing was a wet, light brownish gray fat CLAY with gravel (CH) with a very high plasticity. This clay seam contained occasional subrounded to subangular quartzite clasts up to several inches in diameter.

The clay seam is interpreted to be the weathered uppermost portion of the Calls Fort Shale Member of the Bloomington Formation, though no Calls Fort Shale rock was excavated from the test pit. It is considered likely that the top of the unaltered Calls Fort Shale is approximately 25 feet below the existing grade.

4.2.2 Groundwater

Groundwater seepage was encountered in the north end of the test pit at a depth of approximately 8.5 feet below existing grade. Saturated materials were observed below a depth of approximately 16 feet below existing grade in the southern part of the test pit, which may be representative of the potentiometric surface.

4.2.3 Strength of Earth Materials

To assess the shear strength of native earth materials, a direct shear test (ASTM D6913) was performed on a representative soil sample of the upper portion of the older landslide deposits. Since the prevailing native soils are fairly coarse, generally classifying as clayey gravel with sand (GC), a remolded specimen was tested, with the coarse fraction removed to accommodate the limitation of a 2.5" oedometer. The test results indicated that the soils have a friction angle of 27 degrees and a cohesion of 283 psf (peak strengths). These test results are considered somewhat conservative, as the sample tested initially had a significant coarse fraction; accordingly, the in-situ friction angle would be reasonably expected to be higher than reported on the test result. A summary of the direct shear test results is presented in Appendix B.

Considering the basal shear of the landslide could be in a residual, or near-residual condition, it would be reasonable to assume residual shear strengths along the basal landslide shear surface (where the landslide comes in contact with the underlying bedrock). Residual shear strength values are required to model any pre-existing sheared earth material, typically isolated along a roughly planar surface along the base of a landslide. Residual shear strength testing was not performed for this project; however, residual shear strength testing was performed on a nearby project on earth materials very similar to those identified within the landslide mass, in particular the soils identified as fat clay (IGES, 2018b). As a part of that investigation, a ring shear test (ASTM D6467) was completed on a remolded clay sample obtained from what was interpreted to be a landslide basal shear surface. The tests were conducted under drained conditions. The test results indicated a residual friction angle of 14.4°. For ease of review, the test results are included in Appendix B.

4.3 SLOPE STABILITY

4.3.1 Global Stability

The lot and the surrounding area is relatively flat; however, much of the lot is underlain by landslide deposits. Accordingly, the stability of the existing landslide mass has been assessed in accordance with methodologies set forth in Blake et al. (2002) and AASHTO LRFD for Bridge Design Specifications with respect to two representative cross-sections, illustrated on Figures D-1 and D-2 in Appendix D (the sections are identified in plan-view on Figure A-2). Section A-A' represents the steepest section with respect to the buildable area, the section line being roughly perpendicular to the prevailing topographic lines. Section B-B' is intended to specifically evaluate the landslide mass and is drawn roughly parallel with the inferred sense of direction of the landslide mass.

The stability of the slope was modeled using SLIDE, a computer application incorporating (among others) Spencer's Method of analysis. Calculations for stability were developed by searching for the minimum factor of safety for a failure occurring along the basal shear of the landslide mass, just above the underlying bedrock – this analysis is presumed to be the most conservative case, as residual shear strength can be reasonably assumed along the shear surface. Analysis was performed for both static and seismic (pseudo-static) cases.

Groundwater, e.g. a piezometric groundwater surface, was not specifically encountered during our subsurface investigation; however, considering that we did encountered wet/saturated conditions at a depth of about 16 feet, and considering the history of localized springs occurring early in the year at nearby locations, a groundwater depth of 16 feet was adopted for this model.

Spring Park Road is located at the top of the slope; accordingly, a traffic surcharge of 250 psf has been modeled for static conditions. The new home is expected to be founded on deep foundations; therefore, the majority of the load from the home will be transferred to deeper stratum, hence a surcharge load from the home was not included in the analysis.

Soil strength parameters were selected based on soil types observed, local experience, correlation with index properties (Atterberg Limits, clay content), site-specific strength testing (direct shear test), and comparisons with soil strength laboratory data from nearby sites. Based on this assessment, the following soil strength parameters were selected for this analysis:

Table 4.3.1a
Soil Strength Parameters

Earth Materials	Friction angle (degrees)	Cohesion (psf)	Unit Weight (pcf)
Colluvium (Qc)	36	0	125
Alluvium/Colluvium (Qac)	35	0	120
Landslide (Qlso)	27 (ϕ_R=14.4°)	250	110
Bedrock (Tw)	38	150	135
Bedrock (Cbc)	3	1,000	130
Embankment Fill (Af)	30	100	125

Pseudo-static (seismic screening) analysis of the proposed slope was performed in general conformance with Blake et al. (2002), ASCE 7-16 and AASHTO LRFD for Bridge Design Specifications. The design seismic event was taken as the ground motion with a 2 percent probability of exceedance in 50 years (2PE50). Based on information provided the ASCE-7-16

Seismic Hazard Tool, the geometric mean Peak Ground Acceleration (PGA_M) associated with a 2PE50 event is estimated to be 0.419g. Half of the PGA, (0.21g), was taken as the horizontal seismic coefficient (k_h) (Hynes and Franklin, 1984), and used in the pseudo-static seismic screen analysis. The results of the analyses have been summarized in Table 4.3.1b.

Table 4.3.1b
Results of Slope Stability Analyses

Section	Static Factor of Safety	Pseudo-Static Factor of Safety
A-A'	2.23	0.88
B-B'	2.29	0.77

The results of the analysis indicate the existing conditions meet the minimum required factor-of-safety of 1.5 for static conditions; however, for the seismic (pseudo-static) case, the factor-of-safety is less than the minimum 1.0 that is generally allowed. A summary of the slope stability analysis is presented in Appendix D.

4.3.2 Slope Deformation Analysis

Based on our analysis, the landslide deposits underlying the subject property meet the minimum acceptable factor-of-safety of 1.5 for static conditions; however, the factor-of-safety for the seismic case was less than 1.0. As such, a slope deformation analysis was performed in accordance with the simplified screening procedure developed by Bray and Travasarou (2007). For our slope deformation analysis, the following parameters were adopted:

- Shear Wave Velocity: Vs = 360 m/s (C/D boundary, estimated)
- Sa(T=1.5s): 0.277g
- Mw=7.0
- Yield Coefficient (k_y): see Table 4.3.2 and Appendix D
- Height of Slope, H: see slope stability analysis in Appendix D

A summary of our slope deformation analysis is presented in Table 4.3.2. The screening procedure suggests slope deformations on the order of 1 to 2 cm could occur during a design-level seismic event.

Table 4.3.2
Slope Deformation Analysis - Summary

Section	Yield (g)	Estimated Deformation (cm)
A-A'	0.165	1.0
B-B'	0.138	1.6

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL CONCLUSIONS

Based on the results of the field observations, literature review, and slope stability analyses, **the subsurface conditions are considered suitable for the proposed development provided that the recommendations presented in this report are incorporated into the design and construction of the project.** The property is underlain by older (Pleistocene-aged) landslides deposits; these deposits are expected to remain stable under static conditions; however, under seismic conditions, some ground deformation should be anticipated. Accordingly, the foundations should be designed to accommodate some ground deformation during an earthquake; for design, a differential settlement of 3 inches over a distance of 40 feet may be assumed.

Supporting data upon which the following conclusions and recommendations are based have been presented in the previous sections of this report. The recommendations presented herein are governed by the physical properties of the earth materials encountered in the subsurface explorations. If subsurface conditions other than those described herein are encountered in conjunction with construction, and/or if design and layout changes are initiated, IGES must be informed so that our recommendations can be reviewed and revised as deemed necessary.

5.2 GEOLOGIC CONCLUSIONS AND RECOMMENDATIONS

Based upon the data collected and reviewed as part of the geologic hazard assessment, IGES makes the following conclusions regarding the geological hazards present at the Lot 80R project area:

- **The Lot 80R project area appears to have geological hazards in the form of landslides that are capable of adversely impacting the development as currently proposed under the existing conditions. However, the preexisting gentle slope combined with the implementation of engineered mitigation practices are capable of reducing the landslide hazard risk to an acceptable level.**

- Recent geologic mapping shows the site to be located within young landslide deposits, though surficial morphology and subsurface observations indicate that the property is underlain by older (Pleistocene-aged) landslide deposits. A high plasticity fat clay seam may represent the basal slide plane for these deposits at a depth of approximately 20 feet below existing grade, and this seam is likely below the water table depth, further increasing the hazard risk. Slope stability modeling indicates the underlying landslide deposits are stable under static conditions, however these deposits may not be stable under seismic conditions – some slope deformation can be expected during a design-level seismic event. Therefore, the risk of landslide hazards is considered to be moderate. The risk associated with seismically-induced ground deformation can be mitigated by designing a foundation system capable of surviving some level of ground deformation (see Section 5.5.2).

- Earthquake ground shaking may potentially affect all parts of the project area and is considered to pose a moderate risk.

- Shallow groundwater conditions were observed in the test pit, with seepage at a depth of 8.5 feet and a likely water table at a depth of 16 feet below existing grade. These represent groundwater levels at or near the annual high levels and are following a wet winter and spring. Additionally, groundwater seepage has been observed in test pits and springs on nearby properties; therefore, shallow groundwater hazards are considered to be moderate to high for the property.

- Rockfall, surface-fault-rupture, liquefaction, debris-flow, and flooding hazards are considered to be low for the property.

Given the conclusions listed above, IGES makes the following recommendations:

- To maintain slope stability, it is imperative to keep the landslide slide plane from daylighting or being exposed to conditions that could increase the susceptibility to downslope movement. It is considered largely impractical to over-excavate the landslide deposits across the building footprint; therefore, it is recommended that footings be founded upon the clayey gravel with sand found within the upper portion of the landslide deposits. In an effort to minimize loading the head of the landslide, which would serve to reduce the stability of the site, IGES recommends that the site grading be 'balanced' such that additional fill is not brought to the site, or a net loss of soil is achieved during the earthwork for the foundations (e.g., earth materials are exported from the site).

- Effort should be made to limit the introduction of water into the subsurface near the proposed residence. Appropriate grading and drainage away from the home and xeriscape or natural landscaping will assist in minimizing the introduction of water into the subgrade, thereby reducing the risk of landsliding.

- Because landslide deposits are noted near the property, an IGES engineering geologist or geotechnical engineer should observe the foundation excavation to assess the absence (or presence) of landslide-induced shearing and to ensure that footings are founded on appropriate native materials.

5.3 EARTHWORK

5.3.1 General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris and undocumented fill should be removed. Any existing utilities should be re-routed or protected in place. The exposed native soils should then be proof-rolled with heavy rubber-tired equipment such as a scraper or loader*. Any soft/loose areas identified during proof-rolling should be removed and replaced with structural fill. All excavation bottoms should be observed by an IGES representative during proof-rolling or otherwise prior to placement of engineered fill to evaluate whether soft, loose, or otherwise deleterious earth materials have been removed, and to assess compliance with the recommendations presented in this report.

*not required where bedrock is exposed in the foundation subgrade

5.3.2 Excavations

Soft, loose, or otherwise unsuitable soils beneath structural elements, hardscape or pavements may need to be over-excavated and replaced with structural fill. If over-excavation is required, the excavations should extend ½ foot laterally for every foot of depth of over-excavation. Excavations should extend laterally at least two feet beyond flatwork, pavements, and slabs-on-grade. Structural fill should consist of granular materials and should be placed and compacted in accordance with the recommendations presented in this report.

Prior to placing structural fill, all excavation bottoms should be scarified to at least 6 inches, moisture conditioned as necessary at or slightly above optimum moisture content (OMC), and compacted to at least 90 percent of the maximum dry density (MDD) as determined by ASTM D-1557 (Modified Proctor). Scarification is not required where hard bedrock is exposed.

5.3.3 Excavation Stability

The contractor is responsible for site safety, including all temporary trenches excavated at the site and the design of any required temporary shoring. The contractor is responsible for providing the "competent person" required by Occupational Safety and Health (OSHA) standards to evaluate soil conditions. For planning purposes, Soil Type C is expected to predominate at the site (sands and gravels). Close coordination between the competent person and IGES should be maintained to facilitate construction while providing safe excavations.

Based on OSHA guidelines for excavation safety, trenches with vertical walls up to 5 feet in depth may be occupied. Where very moist soil conditions or groundwater is encountered, or when the trench is deeper than 5 feet, we recommend a trench-shield or shoring be used as a protective system to workers in the trench. As an alternative to shoring or shielding, trench walls may be laid back at one and one-half horizontal to one vertical (1½H:1V) (34 degrees) in accordance with OSHA Type C soils. Trench walls may need to be laid back at a steeper grade pending evaluation of soil conditions by the geotechnical engineer. Soil conditions should be evaluated in the field on a case-by-case basis. Large rocks exposed on excavation walls should be removed (scaled) to minimize rock fall hazards.

5.3.4 Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements should consist of structural fill. Structural fill should consist of granular native soils, which may be defined as soils with less than 25% fines, 10-60% sand, and contain no rock larger than 4 inches in nominal size (6 inches in greatest dimension). Structural fill should also be free of vegetation and debris. All structural fill should be 1-inch minus material when within 1 foot of any base coarse material. Soils not meeting these criteria may be suitable for use as structural fill; however, such soils should be evaluated on a case by case basis and should be approved by IGES prior to use.

All structural fill should be placed in maximum 4-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 6-inch loose lifts if compacted by light-duty rollers, and maximum 8-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. Additional lift thickness may be allowed by IGES provided the Contractor can demonstrate sufficient compaction can be achieved with a given lift thickness with the equipment in use. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill underlying all shallow footings and pavements should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557. **The moisture content should be at, or slightly above, the OMC for all structural fill**. Any imported fill materials should be approved prior to importing. Also, prior to placing any fill, the excavations should be observed by IGES to confirm that unsuitable materials have been removed. In addition, proper grading should precede placement of fill, as described in the General Site Preparation and Grading subsection of this report.

Specifications from governing authorities such as Weber County and/or special service districts having their own precedence for backfill and compaction should be followed where more stringent.

5.3.5 Oversize Material

Based on our observations, there is a significant potential for the presence of oversize materials (larger than 6 inches in greatest dimension). Large rocks, particularly boulders up to 18 inches in diameter, may require special handling, such as segregation from structural fill, and disposal.

5.3.6 Utility Trench Backfill

Utility trenches should be backfilled with structural fill in accordance with Section 5.3.4 of this report. Utility trenches can be backfilled with the onsite soils free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and shaded with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding may be waterdensified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, and hardscape, should be backfilled with structural fill compacted to at least 95 percent of the MDD as determined by ASTM D-1557. All other trenches should be backfilled and compacted to approximately 90 percent of the MDD (ASTM D-1557). However, in all cases the pipe bedding and shading should meet the design criteria of the pipe manufacturer. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where they are more stringent.

5.4 FOUNDATION RECOMMENDATIONS

Based on our field observations and considering the presence of relatively competent native earth materials, the proposed new home may be founded on conventional shallow foundations. The footings may be founded either *entirely* on competent native soils or *entirely* on structural fill. Native/fill transition zones are not allowed. Where soft, loose, or otherwise deleterious earth materials are exposed on the foundation subgrade, IGES recommends a minimum over-excavation of two feet and replacement with structural fill. Alternatively, the foundations may be extended such that the foundations bear directly on competent earth materials (Wasatch Formation, e.g. conglomerate bedrock, or medium dense granular surficial soils). It should be noted that older landslide deposits were identified within the test pit, although Wasatch Formation (conglomeratic bedrock) was identified on the adjacent lots (IGES, 2017a and 2018a). Thus, this landslide deposit is expected to be highly localized and relatively stable. However, part of the buildable area of the lot consists of a fill embankment associated with Spring Park Road, hence undocumented fill will be encountered in conjunction with the existing road embankment. We recommend that IGES assess the bottom of the foundation excavation prior to the placement of steel or concrete, or structural fill, to identify the competent native earth materials as well as any unsuitable soils or transition zones. Additional over-excavation may be required based on the actual subsurface conditions observed.

Shallow spread or continuous wall footings constructed entirely on structural fill, or entirely on competent, uniform native earth materials may be proportioned utilizing a maximum net allowable bearing pressure of **2,800 pounds per square foot (psf)** for dead load plus live load conditions. The net allowable bearing values presented above are for dead load plus live load conditions. The allowable bearing capacity may be increased by one-third for short-term loading (wind and seismic). The minimum recommended footing width is 20 inches for continuous wall footings and 30 inches for isolated spread footings.

All conventional foundations exposed to the full effects of frost should be established at a minimum depth of 42 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost (i.e., *a continuously heated structure*), may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes.

5.5 SETTLEMENT

5.5.1 Static Settlement

Static settlements of properly designed and constructed conventional foundations, founded as described in Section 5.4, are anticipated to be on the order of 1 inch or less. Differential settlement is expected to be half of total settlement over a distance of 30 feet.

5.5.2 Dynamic Settlement

Dynamic settlement (or seismically-induced settlement) consists of dry dynamic settlement of unsaturated soils (above groundwater) and liquefaction-induced settlement (below groundwater). During a strong seismic event, seismically-induced settlement can occur within loose to moderately dense sandy soil due to reduction in volume during, and shortly after, an earthquake event. Settlement caused by ground shaking is often non-uniformly distributed, which can result in differential settlement. In addition to settlement, ground *deformation* may occur as a result of earthquake-induced ground displacement – this is not settlement, however ground deformation can result in differential ground movement under a structure, thereby causing damage.

Based on the subsurface conditions encountered, dynamic settlement of conventional spread footings arising from an MCE seismic event is expected to be low; however, *ground deformation* arising from a seismic event reactivating the underlying landslide deposits could occur (see Section 4.3.2). Accordingly, for design purposes, differential ground movement on the order of 3 inches over 40 feet may be assumed.

5.6 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 0.48 for sandy/gravelly native soils or structural fill should be used.

Ultimate lateral earth pressures from *granular* backfill acting against retaining walls, temporary shoring, or buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in Table 5.6. These lateral pressures should be assumed even if the backfill is placed in a relatively narrow gap between a nearly vertical soil cut and the foundation wall. These coefficients and densities assume no buildup of hydrostatic pressures. The force of 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. Backfill should consist of native granular soil with an Expansion Index (EI) less than 20.

Table 5.6
Lateral Earth Pressure Coefficients

Condition	Level Backfill		2H:1V Backfill	
	Lateral Pressure Coefficient	**Equivalent Fluid Density (pcf)**	**Lateral Pressure Coefficient**	**Equivalent Fluid Density (pcf)**
Active (Ka)	0.33	41.7	0.53	66.5
At-rest (Ko)	0.50	55	0.80	85
Passive (Kp)	3.0	375	—	—
Seismic Active	0.12	15.1	0.38	47.4
Seismic Passive	-0.33	-40.8	—	—
Seismic At-rest	0.18	22.5	0.57	71.7

Walls and structures allowed to rotate slightly should use the active condition. If the element is to be constrained against rotation (i.e., a basement wall), 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.

For seismic analyses, the *active* earth pressure coefficient provided in the table is based on the Mononobe-Okabe pseudo-static approach and only accounts for the dynamic horizontal thrust produced by ground motion. Hence, the resulting dynamic thrust pressure *should be added* to the static pressure to determine the total pressure on the wall. The pressure distribution of the dynamic horizontal thrust may be closely approximated as an inverted triangle with stress decreasing with depth and the resultant acting at a distance approximately 0.6 times the loaded height of the structure, measured upward from the bottom of the structure.

5.7 CONCRETE SLAB-ON-GRADE CONSTRUCTION

To minimize settlement and cracking of slabs, and to aid in drainage beneath the concrete floor slabs, all concrete slabs should be founded on a minimum 4-inch layer of compacted gravel overlying properly prepared subgrade. The gravel should consist of free-draining gravel or road base with a 3/4-inch maximum particle size and no more than 5 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the MDD as determined by ASTM D-1557.

All concrete slabs should be designed to minimize cracking as a result of shrinkage. Consideration should be given to reinforcing the slab with a welded wire fabric, re-bar, or fibermesh. Slab reinforcement should be designed by the structural engineer; however, as a minimum, slab reinforcement should consist of 4''×4'' W2.9×W2.9 welded wire mesh within the middle third of the slab. We recommend that concrete be tested to assess that the slump and/or air content is in compliance with the plans and specifications. We recommend that concrete be placed in general accordance with the requirements of the American Concrete Institute (ACI). A Modulus of Subgrade Reaction of **280 psi/inch** may be used for design.

A moisture barrier (vapor retarder) consisting of 10-mil thick Visqueen (or equivalent) plastic sheeting should be placed below slabs-on-grade where moisture-sensitive floor coverings or equipment is planned. Prior to placing this moisture barrier, any objects that could puncture it, such as protruding gravel or rocks, should be removed from the building pad. Alternatively, the subgrade may be covered with 2 inches of clean sand**.**

5.8 MOISTURE PROTECTION AND SURFACE DRAINAGE

Surface moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. As such, design strategies to minimize ponding and infiltration near the structures should be implemented.

We recommend roof runoff devices be installed to direct all runoff a minimum of 10 feet away from foundations. The builder should be responsible for compacting the exterior backfill soils around the foundation; failure to properly compact the basement backfill can result in excessive settlement and damage to exterior improvements such as pavement or other flatwork. Additionally, the ground surface within 10 feet of the structures should be constructed so as to slope a minimum of **five** percent away from the structure. Irrigation valves should be placed a minimum of 5 feet from foundation walls and must not be placed within the basement backfill zone. Over-watering near the foundation walls is discouraged; use of Xeriscape and/or a drip irrigation system should be considered. Pavement sections should be constructed to divert surface water off the pavement into storm drains, curb/gutter, or another suitable location.

Foundation drains should be installed around below-ground foundations (e.g., basement walls) to minimize the potential for flooding from shallow groundwater or seepage, which may be present at various times during the year, particularly spring run-off. The foundation perimeter drain be should constructed in accordance with the latest edition of the International Residential Code (IRC).

5.9 SOIL CORROSION POTENTIAL

Laboratory testing of a representative soil sample obtained from a nearby lot (Lot 82, IGES, 2017) indicated that the soil sample tested had a sulfate content of 12.8 ppm. Accordingly, the soils are classified as having a 'low potential' for deterioration of concrete due to the presence of soluble sulfate. As such, conventional Type II Portland cement may be used for all concrete in contact with site soils.

To evaluate the corrosion potential of ferrous metal in contact with onsite native soil, laboratory test results for nearby Lot 82 were also reviewed; a sample from that lot was tested for soil resistivity, soluble chloride and pH. The tests indicated that the Lot 82 soil tested has a minimum soil resistivity of 2,613 OHM-cm, soluble chloride content of 13.2 ppm and a pH of 5.5. Based on this result, the onsite native soil is considered to be *moderately corrosive* to ferrous metal.

The soils are mildly acidic - to address the acidic soil conditions, we recommend a lower water/cement ratio, ~0.4, for reinforced concrete. The lower water/cement ratio will reduce permeability of the concrete and reduce the susceptibility of the reinforcing steel to acidic corrosion.

5.10 CONSTRUCTION CONSIDERATIONS

5.10.1 Over-Size Material

Large boulders (up to 18 inches in diameter) were observed on the surface and within the test pits; as such, excavation of the basement may generate an abundance of over-size material that may require special handling, processing, or disposal.

5.10.2 Groundwater

Some seepage was identified in the test pit at a depth of approximately 8 feet below grade. At a depth of about 16 feet below grade, the prevailing soils appeared wet and saturated. Based on these observations, some groundwater issues may be present during the construction of the home's foundations, particularly if a basement level is planned. Temporary dewatering, temporary diversion structures, or shoring may be needed during construction, particularly in the spring during snow run-off.

6.0 CLOSURE

6.1 LIMITATIONS

The concept of risk is a significant consideration of geotechnical analyses. The analytical means and methods used in performing geotechnical analyses and development of resulting recommendations do not constitute an exact science. Analytical tools used by geotechnical engineers are based on limited data, empirical correlations, engineering judgment and experience. As such the solutions and resulting recommendations presented in this report cannot be considered risk-free and constitute IGES's best professional opinions and recommendations based on the available data and other design information available at the time they were developed. IGES has developed the preceding analyses, recommendations and designs, at a minimum, in accordance with generally accepted professional geotechnical engineering practices and care being exercised in the project area at the time our services were performed. No warrantees, guarantees or other representations are made.

The information contained in this report is based on limited field testing and our understanding of the project. The subsurface data used in the preparation of this report were obtained largely from the exploration made on Lot 80R. It is very likely that variations in the soil, rock, and groundwater conditions exist between and beyond the point explored. The nature and extent of the variations may not be evident until construction occurs and additional explorations are completed. If any conditions are encountered at this site that are different from those described in this report, IGES must be immediately notified so that we may make any necessary revisions to recommendations presented in this report. In addition, if the scope of the proposed construction or grading changes from those described in this report, our firm must also be notified.

This report was prepared for our client's exclusive use on the project identified in the foregoing. Use of the data, recommendations or design information contained herein for any other project or development of the site not as specifically described in this report is at the user's sole risk and without the approval of IGES, Inc. It is the client's responsibility to see that all parties to the project including the designer, contractor, subcontractors, etc. are made aware of this report in its entirety. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

We recommend that IGES be retained to review the final design plans, grading plans and specifications to determine if our engineering recommendations have been properly incorporated in the project development documents. We also recommend that IGES be retained to evaluate construction performance and other geotechnical aspects of the project as construction initiates and progresses through its completion.

6.2 ADDITIONAL SERVICES

The recommendations made in this report are based on the assumption that an adequate program of tests and observations will be made during the construction. IGES staff or other qualified personnel should be on site to verify compliance with these recommendations. These tests and observations should include at a minimum the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction.
- Quality control on concrete placement to verify slump, air content, and strength.

We also recommend that project plans and specifications be reviewed by us to verify compatibility with our conclusions and recommendations. Additional information concerning the scope and cost of these services can be obtained from our office.

We appreciate the opportunity to be of service on this project. Should you have any questions regarding the report or wish to discuss additional services, please do not hesitate to contact us at your convenience at (801) 748-4044.

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AERIAL PHOTOGRAPHS

Data Set	Date	Flight	Photographs	Scale
1947 AAJ	August 10, 1946	AAJ_1B	88, 89, 90	1:20,000
1953 AAI	September 14, 1952	AAI_4K	34, 35, 36	1:20,000
1963 ELK	June 25, 1963	ELK_3	57, 58, 59	1:15,840

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