IGES®
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July 1, 2016

Richard Naylor
8106 South 1460 West, Suite 1
West Jordan, UT 84088

IGES, Inc. Project No. 02316-001

RE: Geotechnical Investigation Report
Monte Meadows Subdivision
1614 West 13400 South
Riverton, Utah

Mr. Naylor,

As requested, IGES conducted a geotechnical investigation for the proposed Monte Meadows Subdivision to be constructed at 1614 West 13400 South, Riverton, Utah. The approximate location of the property is shown on the Site Vicinity Map (Figure A-1 in Appendix A). The proposed site consists of four lots being serviced by a private lane. There were two existing homes and one home under construction on three of the lots at the time of our investigation; leaving only one lot vacant that was considered as part of this study. The private lane connects the lots with 13400 South Street (see Site Map, Figure A-2 in Appendix A). Based on direct communication with client, the proposed single family home will have a basement with the foundation elevation at approximately 4.5 to 5.5 feet below existing site grade.

The purposes of this investigation were to assess the nature and engineering properties of the subsurface soils at the proposed site and to provide recommendations for the design and construction of foundations for the single family home and pavement for the private lane. The scope of work completed for this study included subsurface exploration, engineering analyses and preparation of this letter.

FIELD INVESTIGATION

The vacant lot considered as part of this study is relatively flat with easy access via a private lane connected to 13400 South Street. There is an existing home east of the vacant lot and two other existing homes accessed by the private lane. The vacant lot is covered with weeds which are approximately 1 to 3 feet in height. The private lane is unpaved, covered with approximately 1 to 2 inches of gravel road base.

Subsurface soils were investigated by excavating two test pits 8.5 to 9 feet below the existing site grade and completing four Dynamic Cone Penetrometer (DCP) tests along the private lane. DCP testing was completed by driving a cone-shaped tip through soils using a 17.6 pound weight that is lifted and dropped 22.6 inches. The number of blows required to drive the cone a minimum distance of approximately 2 inches and incremental displacement are recorded. The data is then plotted and compared to California Bearing Ratio (CBR) and allowable bearing capacity correlations as presented in US Army Corps of Engineers, Technical Report No. GL-94-17 and *Design of Concrete Airport Pavement* (Portland Cement Association, 1955). The approximate location of the test pit and DCP tests is illustrated on the Site Map included with this report as Figure A-2 in Appendix A. The soil types and conditions were visually logged at the time of the excavation in general accordance with the Unified Soil Classification System (USCS, ASTM 2487). Subsurface soil classifications and descriptions are included on the Test Pit Log included as Figure A-3 to 4 in Appendix A. A key to USCS symbols and terminology is included as Figure A-5.

The soil at the surface of the site consists of dark brown Gravelly Lean CLAY (CL) with sand topsoil that is stiff and moist. The topsoil is approximately 6 to 18 inches deep with frequent fine roots and root holes. Below the topsoil was a layer of gray Silty GRAVEL (GM) with sand that is dense and moist or brown Gravelly Lean CLAY (CL) with sand that was stiff and moist to very moist, which extends to a depth of approximately 5 to 6 feet below the surface. Below this layer was a layer of yellowish brown Silty SAND (SM) that was dense and moist or very moist, which extends to the maximum depths of the test pits.

Groundwater was not encountered at the maximum depth explored (9 feet below existing grade). Based on this observation and the proposed foundation bearing elevation (4.5 to 5.5 feet below existing grade), groundwater is not anticipated to adversely impact the proposed construction. However, groundwater levels could rise based on several factors including recent precipitation, on- or off-site runoff, irrigation, and seasonal fluctuation. Should the groundwater become a concern during the proposed construction, IGES should be contacted to provide dewatering recommendations.

ENGINEERING ANALYSES

Engineering analyses were performed using soil observation during our field investigation and empirical correlations based on material density, depositional characteristics and classification. Appropriate factors of safety were applied to the results consistent with industry standards and the accepted standard of care.

Seismicity and Faulting

An active fault is defined as a fault that has experienced movement with the Holocene (11,000 years before present). No active faults are mapped through or immediately adjacent to the site (Black et al., 2003). The closest mapped active fault is the Salt Lake City segment of the Wasatch Fault Zone, located about 5 miles east of the site. The most recent documented event occurred in the latest Quaternary (<15 ka). The Salt Lake Segment of the Wasatch Fault Zone has a slip rate of approximately 1-5mm/yr and an overall length of 43 km. The site is also mapped approximately 10½ miles south of the Granger Fault portion of the West Valley fault zone, a north-south trending series of faults that are mapped within the middle of the Salt Lake Valley. The last event reportedly occurred on the West Valley Fault Zone <12,000 years ago, and has a recurrence interval of 6,000 to 12,000 years. The West Valley Fault Zone trends in a north-south orientation and is located in the central portion of the Salt Lake Valley (Keaton and Curry, 1993). While the West Valley Fault Zone is reported to be active and probably seismically independent of the Wasatch Fault Zone, sympathetic movement on the West Valley Fault Zone resulting from major earthquakes on the Wasatch Fault Zone is a possibility. The site is also located approximately 14 miles east-southeast of the Oquirrh Fault Zone.

Following the criteria outlined in the 2012 International Building Code (IBC, 2012), 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 *U.S. Seismic Design Maps,* USGS website: http://earthquake.usgs.gov/designmaps/us/application.php; this tool incorporates seismic hazard maps depicting probabilistic ground motions and spectral response data developed for the United States by the U. S. Geological Survey as part of NEHRP/NSHMP (Frankel et al., 1996). These maps have been incorporated into both *NEHRP Recommended Provisions for Seismic Regulations for New Buildings and Other Structures* (FEMA, 1997) and the *International Building Code* (IBC) (International Code Council, 2012).

To account for site effects, site coefficients that vary with the magnitude of spectral acceleration and *Site Class* are used. Site classification is based on the upper 100 feet of the site soil profile; based on our field exploration and our understanding of the geology in this area, the subject site can be estimated as Site Class D (*stiff soil*). Based on IBC criteria, the short-period (F_a) and long-period (F_v) site coefficients are 1.000 and 1.536, respectively. Based on the design spectral response accelerations for a *Building Risk Category* of I, II or III, the site's *Seismic Design Category* is D. The Risk-Targeted Maximum Considered Earthquake (MCE_R) *Spectral Response Accelerations* are presented in Table 1; a summary of the *Design Maps* analysis is presented in Appendix C.

The *peak ground acceleration* (PGA) may be taken as 0.584g (ASCE 7-10, see Appendix C). It should be noted that to more accurately determine the site classification, geotechnical investigation to a minimum depth of 100 feet is needed.

Table 1
Short and 1-Second Period Spectral Accelerations

Parameter	Short Period (0.2 sec)	Long Period (1.0 sec)
MCE_R Spectral Response Acceleration Site Class B (g)	$S_S = 1.383$	$S_1 = 0.464$
MCE_R Spectral Response Acceleration Site Class D (g)	$S_{MS} = 1.383$	$S_{M1} = 0.713$
Design Spectral Response Acceleration Site Class D (g)	$S_{DS} = 0.922$	$S_{D1} = 0.475$

Liquefaction

Liquefaction is the loss of soil strength or stiffness due to a buildup of excess pore-water pressure during strong ground shaking. Liquefaction is associated primarily with loose (low density), granular, saturated soil. Effects of severe liquefaction can include sand boils, excessive settlement, bearing capacity failures, and lateral spreading.

The geologic hazards map titled *Liquefaction-Potential Map for a Part of Salt Lake County, Utah*, dated August 1994, indicates that the subject property is located within an area designated as having a *very low* liquefaction potential. No groundwater was encountered during our investigation in the upper 9 feet of the site; therefore, it is our opinion that the upper 9 feet of the site have a very low liquefaction potential. A liquefaction hazard study, which would include multiple borings and/or CPT soundings to depths of 50 feet, was not performed and is beyond our scope of services for this project.

CONCLUSIONS AND RECOMMENDATIONS

Based on the results of the field investigations, DCP testing and engineering analysis, the site is suitable for the proposed construction provided that the recommendations contained in this report are complied with.

General Site Preparation and Grading

Below proposed structures, fills, and man-made improvements, all vegetation, topsoil, debris, and undocumented fill soils should be removed. 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 2 feet beyond footings, flatwork, pavements and slabs-on-grade. 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, compacted in place or replaced with structural fill. All excavation bottoms should be observed by an IGES representative prior to placement of structural fill to evaluate whether soft, loose, or otherwise deleterious earth materials have been removed and to assess compliance with the recommendations presented herein.

Cut and Excavation Stability

The contractor is responsible for site safety, including all temporary cuts and 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. Soil types encountered in the field and laboratory investigations are consistent with *Type B* soils. 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, a trench-shield or shoring should be used as a protective system to workers in the trench. Sloping the sides at 1 horizontal to 1 vertical (1H:1V) (45 degrees) in accordance with OSHA Type B soils may be used as an alternative to shoring or shielding.

Structural Fill and Compaction

All fill placed for the support of structures, flatwork or pavements, should consist of structural fill. Structural fill may consist of onsite native soils or imported granular soils. Imported structural fill (if used) should be a granular material with less than 20 percent fines having an Expansion Index less than 20. Prior to use, all structural fill should be approved by IGES. Soils not meeting the aforementioned criteria may be suitable for use as structural fill; however, such material should be evaluated on a case-by-case basis and should be approved by IGES prior to use. In all cases structural fill should be relatively free of vegetation and debris, and contain no rocks larger than 4 inches in nominal size (6 inches in greatest dimension). All structural fill should be 1-inch minus material when within 1 foot of any base coarse material.

All structural fill should be placed in maximum 6-inch loose lifts if compacted by small hand-operated compaction equipment, maximum 8-inch loose lifts if compacted by light-duty rollers, and maximum 10-inch loose lifts if compacted by heavy duty compaction equipment that is capable of efficiently compacting the entire thickness of the lift. These lift thicknesses are maximums; the Contractor should understand that thinner lifts may be necessary to achieve the required compaction. We recommend that all structural fill be compacted on a horizontal plane, unless otherwise approved by IGES. Structural fill placed beneath footings and pavements should be compacted to at least 95 percent of the maximum dry density (MDD) as determined by ASTM D-1557. During the compaction process, the moisture content should be at, or slightly above, the optimum moisture content (OMC) for all structural fill. Prior to placing any fill, the subgrade should be observed by IGES to assess whether unsuitable materials have been removed and/or the subgrade has been properly prepared.

Foundation Backfill

Backfill around foundations should consist of native soils placed in maximum 12-inch loose lifts compacted to 90 to 95 percent of the MDD and within 2 percent of the OMC (Modified Proctor, ASTM D 1557). Compacting by means of injecting water or "jetting" is not recommended. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

Utility Trench Backfill

Utility trenches can be backfilled with the onsite soils that are substantially free of debris, organic and oversized material. Prior to backfilling the trench, pipes should be bedded in and covered with a uniform granular material that has a Sand Equivalent (SE) of 30 or greater. Pipe bedding should *not* be water-densified in-place (jetting). Alternatively, pipe bedding and shading may consist of clean ¾-inch gravel, which generally does not require densification. Native earth materials can be used as backfill over the pipe bedding zone. All utility trenches backfilled below pavement sections, curb and gutter, and sidewalks, should be backfilled with structural fill compacted to at least 95 percent of the MDD with the moisture content at or slightly above the OMC as determined by ASTM D-1557. All other trenches, including landscape areas, should be backfilled and compacted to approximately 90 percent of the MDD with a moisture content that is within 2 percent of the OMC (ASTM D-1557). Backfill around foundations should consist of native soils placed in maximum 12-inch loose lifts compacted to 90 to 95 percent of the maximum dry density and within 2 percent of the optimum moisture content (Modified Proctor, ASTM D 1557). Compacting by means of injecting water or "jetting" is not recommended. Specifications from governing authorities having their own precedence for backfill and compaction should be followed where applicable.

Foundations

It is anticipated that the proposed foundation elements will consist of conventional continuous and/or spread footings. Based on the encountered soil conditions, the foundation elements may be placed directly on undisturbed native soils and should be proportioned for a maximum net allowable bearing pressure of **1,500 psf** for dead load plus live load conditions. All foundations exposed to the full effects of frost should be established at a minimum depth of 30 inches below the lowest adjacent final grade. Interior footings, not subjected to the full effects of frost, may be established at higher elevations, however, a minimum depth of embedment of 12 inches is recommended for confinement purposes. The minimum recommended footing width is 20 inches for continuous wall footings and 36 inches for isolated spread footings.

Undocumented fill, vegetation, debris, heavy-rooted, organic soils and any otherwise unsuitable materials located below the proposed structure should be removed prior to placement of footings. Any soft areas identified during excavation should be removed and replaced with structural fill in accordance with the *structural fill and compaction* section of this report. Other than localized soft areas, the foundation elements should bear entirely on competent, undisturbed native soils or entirely on structural fill (i.e., the structure should not be founded partially on native earth materials and partially on structural fill).

Prior to placing any fill or constructing the foundation elements, all excavations should be observed by IGES to observe that all fill or otherwise unsuitable materials have been removed and the site is suitable for the placement of the footings.

Moisture Protection

Moisture should not be allowed to infiltrate into the soils in the vicinity of the foundations. The following construction practices should be implemented to minimize water ponding and infiltration in areas adjacent to the proposed building:

- Roof runoff devices should be installed around the entire perimeter of the home to collect and discharge all runoff a minimum of 10 feet from the foundation elements or beyond the limits of backfill around the foundation walls; whichever distance is greater.
- The ground surface within 10-feet of the foundations should be sloped a minimum of 5 percent to drain away from the structure.
- Pavement sections should be constructed to adequately divert water into storm water disposal systems.
- Parking lot and roadway shoulder areas should be constructed and maintained to prevent infiltration of water underneath the pavements and building pad.

Settlement

Settlement of properly designed and constructed conventional foundations, founded as described above and in accordance with the moisture protection recommendations provided, are anticipated to be less than 1 inch. Differential settlement should be on the order of half the total settlement over a distance of 30 feet.

Lateral Earth Pressures

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 0.44 for native soils should be used.

Ultimate lateral earth pressures from on-site native soils used as backfill acting against retaining walls and buried structures may be computed from the lateral pressure coefficients or equivalent fluid densities presented in the following table:

Table 2
Lateral Earth Pressure

Condition	Level Backfill	
	Lateral Pressure Coefficient	Equivalent Fluid Density (pcf)
Active (Ka)	0.28	35
At-rest (Ko)	0.44	55
Passive (Kp)	3.54	442

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. If imported backfill will be used, the values presented in Table 2 can be re-evaluated by IGES upon request and subsequently modified as appropriate.

Walls and structures allowed to rotate slightly should use the active condition. If the element is 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.

Slabs-on-Grade

To minimize settlement and cracking of slabs, all concrete slabs should be founded on a 4-inch layer of compacted gravel. The slab may be designed with a Modulus of Subgrade Reaction of **200 psi/inch**. The gravel should consist of free draining gravel or road base with a 3/4-inch maximum particle size and no more than 12 percent passing the No. 200 mesh sieve. The layer should be compacted to at least 95 percent of the maximum dry density as determined by ASTM D 1557 (Modified Proctor). 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. Reinforcement should be designed by the structural engineer.

Pavements

For the pavement design, we completed four Dynamic Cone Penetrometer (DCP) tests, which can be correlated to the California Bearing Ratio (CBR) within the area of the proposed new pavement section. The DCP tests extended approximately 3 feet in depth below existing site grade. DCP test results are presented in Appendix B. The approximate locations of our DCP tests are shown on the *Site Map* (Figure A-2). The CBR values based on the DCP correlation are summarized in Table 3.

Table 3
DCP Test Data

DCP ID	Maximum Depth Tested (in.)	Representative CBR Value Based on DCP Analysis
DCP-1	35	15
DCP-2	36	20
DCP-3	37	25
DCP-4	38	20

Anticipated traffic volumes were not available at the time this report was prepared; however, based on our understanding of the project development we assume traffic within the private lane would consist primarily of passenger cars with occasional heavy vehicles associated with construction, municipal waste collection and similar. The following pavement designs have been developed for a 20-year design life assuming a 0 percent annual growth rate and an assumed equivalent single axle load (ESAL) value not exceeding 50,000 ESALs. Asphalt has been assumed to be a high stability plant mix, base course material should be composed of crushed stone with a minimum CBR of 70 and granular borrow should consist of a pit-run type of material with a minimum CBR of 30. Asphalt should be compacted to a minimum density of 96% of the Marshall value; base course and granular borrow should be compacted to at least 95% of the MDD as determined by ASTM D-1557.

The following pavement design recommendations have been prepared based on a representative CBR value of 20 and the assumptions listed previously, we recommend the following pavement section presented in Tables 4 be constructed on properly prepared subgrade.

Table 4
Flexible (Asphalt) Pavement Section

Asphalt Concrete (in.)	Untreated Base Course (in.)	Reworked Native Soil (in.)
3	6	6

It is our experience that pavement in areas where vehicles frequently turn around, backup, or load and unload, including round-a-bouts and exit and entrance areas, often experience more distress. If the owner wishes to prolong the life of the pavement in these areas, consideration should be given to using a Portland cement concrete (rigid) pavement in these areas. For these conditions, the following rigid pavement section presented in Table 5 is recommended:

Table 5
Rigid Pavement Section

Concrete (in.)	Untreated Base Course (in.)	Reworked Native Soil (in.)
5	5	6

Concrete should consist of a low slump, low water cement ratio mix with a minimum 28-day compressive strength of 4,000 psi. The base course should be compacted to at least 95% of the MDD as determined by ASTM D-1557.

If traffic conditions vary significantly from our stated assumptions, IGES should be contacted so we can modify our pavement design parameters accordingly. Specifically, if the traffic counts are significantly higher or lower, IGES should be contacted to revise the pavement section design as necessary. The pavement sections presented assume that the majority of construction traffic including cement trucks, cranes, loaded haulers, etc. has ceased. If a significant volume of construction traffic occurs after the pavement section has been constructed, the owner should anticipate a reduced life and increased maintenance in some areas of the pavement.

The pavement section thicknesses presented assume that there is no mixing over time between the road base/granular borrow and the softer native subgrade below. In order to prevent mixing or fines migration, and thereby prolonging the life of the pavement section, we recommend that the owner give consideration to placing a non-woven filter fabric between the native soils and the road base. We recommend that a product such as TenCate Mirafi 160N, or an IGES-approved equivalent be used for separation.

Soil Corrosion Potential

Soils with a high level of soluble sulfates may react with concrete, resulting in the breakdown of concrete elements in contact with site soils. Chemical testing was not completed on the near-surface site soils. However, IGES anticipates that conventional type I/II cement will be suitable based on test results from nearby projects and experience in the area. IGES recommends that a corrosion engineer be consulted for recommendations concerning corrosion protection of ferrous metals in contact with the site soils.

CLOSURE

The recommendations contained in this letter are based on limited field exploration and a general understanding of the proposed construction. The subsurface data used in the preparation of this letter were obtained from the explorations made for this investigation. It is possible that variations in the soil and groundwater conditions could exist between the points explored. The nature and extent of variations may not be evident until construction occurs. If any conditions are encountered at this site that are different from those described in this report, IGES should be immediately notified so that any necessary revisions to recommendations contained in this report may be made. In addition, if the scope of the proposed construction changes from that described in this report, IGES should also be notified.

This report was prepared in accordance with the contract and scope of work dated May 24, 2016. No warranty, expressed or implied, is made.

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.

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 should be on site to assess compliance with these recommendations. These tests and observations should include, but not necessarily be limited to, the following:

- Observations and testing during site preparation, earthwork and structural fill placement.
- Consultation as may be required during construction.
- Quality control testing of cast-in-place concrete

- Review of plans and specifications to assess compliance with our recommendations.

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 contact the undersigned at (801) 748-4044.

Respectfully submitted,
IGES, Inc.

LICENSED PROFESSIONAL ENGINEER
No. 184125
KENT A.
HARTLEY
7/8/16
STATE OF UTAH

Shun Li, P.E.I.
Staff Engineer

Kent A. Hartley, P.E.
Principal Engineer

Appendices:

Appendix A
- Figure A-1 – Site Vicinity Map
- Figure A-2 – Site Map
- Figure A-3 to 4 – Test Pit Logs
- Figure A-5 – Key to Soil Symbols and Terminology

Appendix B – DCP Test Data

Appendix C – MCE_R Response Spectra

References:

ASCE 7 Standard, 2010, Minimum Design Loads for Buildings and Other Structures

Black, B.D., Hecker, S., Hylland, M.D., Christenson, G.E., and McDonald, G.N.,2003, Quaternary Fault and Fold Database and Map of Utah, Utah Geological Survey Map 193DM.

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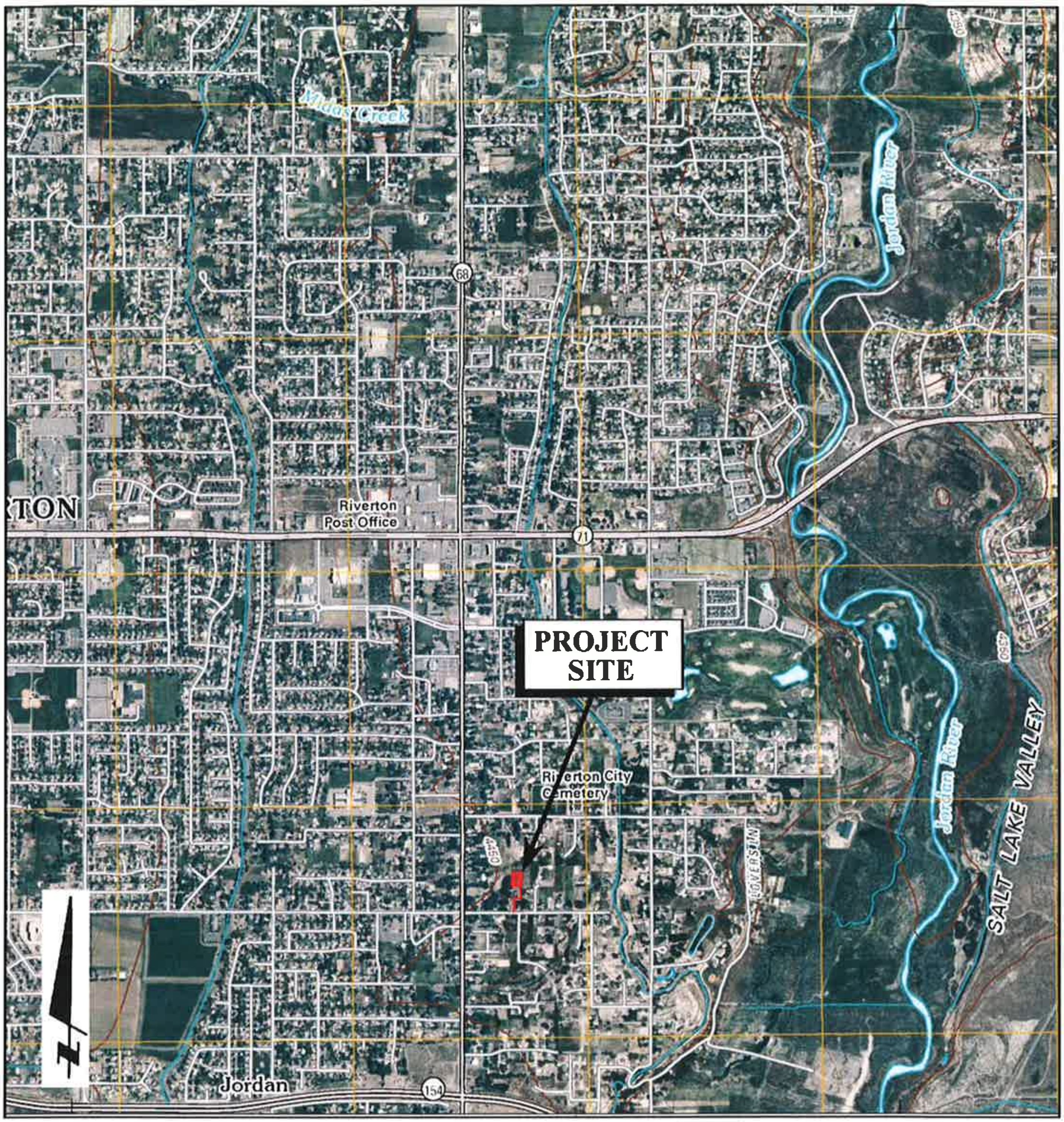
APPENDIX A



BASE MAP:
USGS Midvale, Utah
7.5-Minute Quadrangle Topographic Maps (2011)

MAP LOCATION

0 1000' 2000'
SCALE 1:24,000

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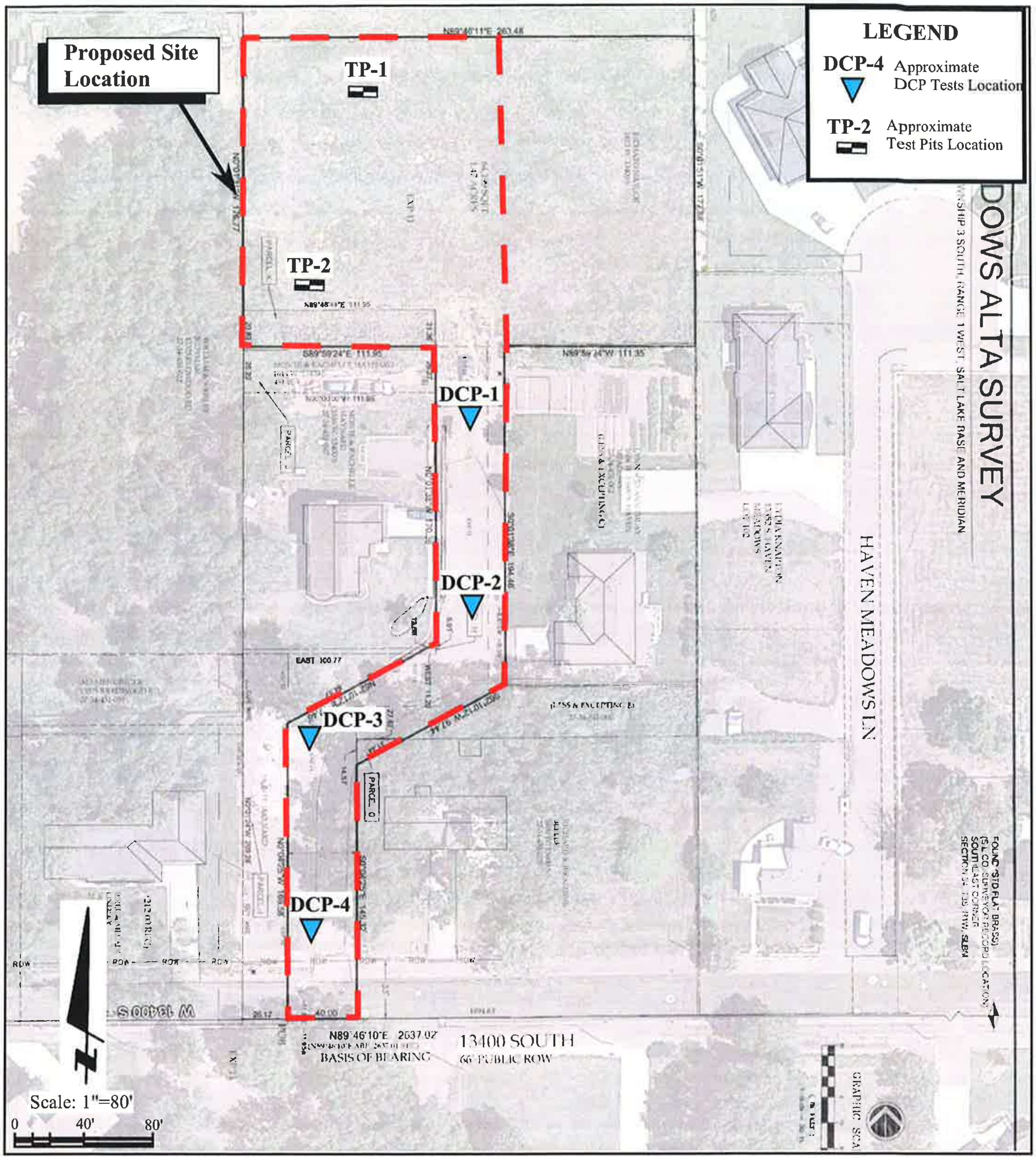
Project No. 02316-001

Geotechnical Investigation
Monte Meadows Subdivision
1614 West 13400 South
Riverton, Utah

SITE VICINITY MAP

Figure
A-1



BASE MAP:
Flint Enterprises, 2015, Monte Meadows Alta, Deed Swap Exhibit, Project Number 150602, Sheet 2 of 3, Dated June 28, 2015

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Geotechnical Investigation
Monte Meadows Subdivision
1614 West 13400 South
Riverton, Utah

SITE MAP

Figure
A-2

DATE		
STARTED:	6/3/16	
COMPLETED:	6/3/16	
BACKFILLED:	6/3/16	

Geotechnical Investigation
Monte Meadows Subdivision
1614 West 13400 South
Riverton, Utah

Project Number 02306-001

IGES Rep: SL

Rig Type: JCB 8018CTS

TEST PIT NO:

TP-1

Sheet 1 of 1

LOCATION

LATITUDE | LONGITUDE | ELEVATION

ELEVATION	DEPTH (FEET)	SAMPLES	WATER LEVEL	GRAPHICAL LOG	UNIFIED SOIL CLASSIFICATION	MATERIAL DESCRIPTION	Dry Density(pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index	Moisture Content and Atterberg Limits (Plastic Limit, Moisture Content, Liquid Limit) 10 20 30 40 50 60 70 80 90
	0					Topsoil - Gravely Lean CLAY with sand - stiff, moist, dark brown, frequent fine roots and root holes						
					GM	Native - Silty GRAVEL with sand - dense, moist, gray, matrix supported, gravel is sub-angular to sub-rounded, typical diameter 1/2 to 2 inches						
		Grab sample										
	5				SM	Silty SAND - dense, moist, yellowish brown, some iron staining						
		3" O.D. thin-walled hand sampler										
		Grab sample										
						No groundwater encountered						
						Bottom of test pit @ 8.5 Feet						

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Figure

A-3

DATE		
STARTED:	6/3/16	
COMPLETED:	6/3/16	
BACKFILLED:	6/3/16	

Geotechnical Investigation
Monte Meadows Subdivision
1614 West 13400 South
Riverton, Utah

Project Number 02306-001

IGES Rep: SL

Rig Type: JCB 8018CTS

TEST PIT NO: **TP-2**

Sheet 1 of 1

LOCATION

LATITUDE | LONGITUDE | ELEVATION

Depth (Feet)	Samples	USCS	Material Description	Dry Density (pcf)	Moisture Content %	Percent minus 200	Liquid Limit	Plasticity Index
0			Topsoil - Gravelly Lean CLAY with sand - stiff, moist, dark brown, frequent fine roots and root holes					
		CL	Native - Gravelly Lean CLAY with sand - stiff, moist to very moist, brown, no pinholes observed					
5	Grab sample; 3" O.D. thin-walled hand sampler	GM	Native - Silty GRAVEL with sand - dense, moist, gray, matrix supported, gravel is sub-angular to sub-rounded, typical diameter 1/2 to 2 inches					
	Grab sample	SM	Silty SAND - dense, very moist, yellowish brown, some iron staining					

No groundwater encountered

Bottom of test pit @ 9 Feet

Moisture Content and Atterberg Limits

Plastic Limit — Moisture Content — Liquid Limit

10 20 30 40 50 60 70 80 90

IGES

SAMPLE TYPE
- GRAB SAMPLE
- 3" O.D. THIN-WALLED HAND SAMPLER

WATER LEVEL
- MEASURED
- ESTIMATED

NOTES:

Figure A-4

UNIFIED SOIL CLASSIFICATION SYSTEM

MAJOR DIVISIONS			USCS SYMBOL	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS (More than half of material is larger than the #200 sieve)	GRAVELS (More than half of coarse fraction is larger than the #4 sieve)	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	WELL-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
			GP	POORLY-GRADED GRAVELS, GRAVEL-SAND MIXTURES WITH LITTLE OR NO FINES
		GRAVELS WITH OVER 12% FINES	GM	SILTY GRAVELS, GRAVEL-SILT-SAND MIXTURES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES
	SANDS (More than half of coarse fraction is smaller than the #4 sieve)	CLEAN SANDS WITH LITTLE OR NO FINES	SW	WELL-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
			SP	POORLY-GRADED SANDS, SAND-GRAVEL MIXTURES WITH LITTLE OR NO FINES
		SANDS WITH OVER 12% FINES	SM	SILTY SANDS, SAND-GRAVEL-SILT MIXTURES
			SC	CLAYEY SANDS SAND-GRAVEL-CLAY MIXTURES
FINE GRAINED SOILS (More than half of material is smaller than the #200 sieve)	SILTS AND CLAYS (Liquid limit less than 50)		ML	INORGANIC SILTS & VERY FINE SANDS, SILTY OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS & ORGANIC SILTY CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS (Liquid limit greater than 50)		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILT
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS & ORGANIC SILTS OF MEDIUM-TO-HIGH PLASTICITY
HIGHLY ORGANIC SOILS			PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

LOG KEY SYMBOLS

BORING SAMPLE LOCATION

TEST-PIT SAMPLE LOCATION

WATER LEVEL (level after completion)

WATER LEVEL (level where first encountered)

CEMENTATION

DESCRIPTION	DESCRIPTION
WEAKELY	CRUMBLES OR BREAKS WITH HANDLING OR SLIGHT FINGER PRESSURE
MODERATELY	CRUMBLES OR BREAKS WITH CONSIDERABLE FINGER PRESSURE
STRONGLY	WILL NOT CRUMBLE OR BREAK WITH FINGER PRESSURE

OTHER TESTS KEY

C	CONSOLIDATION	SA	SIEVE ANALYSIS
AL	ATTERBERG LIMITS	DS	DIRECT SHEAR
UC	UNCONFINED COMPRESSION	T	TRIAXIAL
S	SOLUBILITY	R	RESISTIVITY
O	ORGANIC CONTENT	RV	R-VALUE
CBR	CALIFORNIA BEARING RATIO	SU	SOLUBLE SULFATES
COMP	MOISTURE/DENSITY RELATIONSHIP	PM	PERMEABILITY
CI	CALIFORNIA IMPACT	-200	% FINER THAN #200
COL	COLLAPSE POTENTIAL	Gs	SPECIFIC GRAVITY
SS	SHRINK SWELL	SL	SWELL LOAD

MODIFIERS

DESCRIPTION	%
TRACE	<5
SOME	5 - 12
WITH	>12

MOISTURE CONTENT

DESCRIPTION	FIELD TEST
DRY	ABSENCE OF MOISTURE, DUSTY, DRY TO THE TOUCH
MOIST	DAMP BUT NO VISIBLE WATER
WET	VISIBLE FREE WATER, USUALLY SOIL BELOW WATER TABLE

STRATIFICATION

DESCRIPTION	THICKNESS	DESCRIPTION	THICKNESS
SEAM	1/16 - 1/2"	OCCASIONAL	ONE OR LESS PER FOOT OF THICKNESS
LAYER	1/2 - 12"	FREQUENT	MORE THAN ONE PER FOOT OF THICKNESS

GENERAL NOTES

1. Lines separating strata on the logs represent approximate boundaries only. Actual transitions may be gradual.
2. No warranty is provided as to the continuity of soil conditions between individual sample locations.
3. Logs represent general soil conditions observed at the point of exploration on the date indicated.
4. In general, Unified Soil Classification designations presented on the logs were evaluated by visual methods only. Therefore, actual designations (based on laboratory tests) may vary.

APPARENT / RELATIVE DENSITY - COARSE-GRAINED SOIL

APPARENT DENSITY	SPT (blows/ft)	MODIFIED CA. SAMPLER (blows/ft)	CALIFORNIA SAMPLER (blows/ft)	RELATIVE DENSITY (%)	FIELD TEST
VERY LOOSE	<4	<4	<5	0 - 15	EASILY PENETRATED WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
LOOSE	4 - 10	5 - 12	5 - 15	15 - 35	DIFFICULT TO PENETRATE WITH 1/2-INCH REINFORCING ROD PUSHED BY HAND
MEDIUM DENSE	10 - 30	12 - 35	15 - 40	35 - 65	EASILY PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
DENSE	30 - 50	35 - 60	40 - 70	65 - 85	DIFFICULT TO PENETRATED A FOOT WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER
VERY DENSE	>50	>60	>70	85 - 100	PENETRATED ONLY A FEW INCHES WITH 1/2-INCH REINFORCING ROD DRIVEN WITH 5-LB HAMMER

CONSISTENCY - FINE-GRAINED SOIL

CONSISTENCY	SPT (blows/ft)	TORVANE UNTRAINED SHEAR STRENGTH (tsf)	POCKET PENETROMETER UNCONFINED COMPRESSIVE STRENGTH (tsf)	FIELD TEST
VERY SOFT	<2	<0.125	<0.25	EASILY PENETRATED SEVERAL INCHES BY THUMB. EXUDES BETWEEN THUMB AND FINGERS WHEN SQUEEZED BY HAND.
SOFT	2 - 4	0.125 - 0.25	0.25 - 0.5	EASILY PENETRATED ONE INCH BY THUMB. MOLDED BY LIGHT FINGER PRESSURE.
MEDIUM STIFF	4 - 8	0.25 - 0.5	0.5 - 1.0	PENETRATED OVER 1/2 INCH BY THUMB WITH MODERATE EFFORT. MOLDED BY STRONG FINGER PRESSURE.
STIFF	8 - 15	0.5 - 1.0	1.0 - 2.0	INDENTED ABOUT 1/2 INCH BY THUMB BUT PENETRATED ONLY WITH GREAT EFFORT.
VERY STIFF	15 - 30	1.0 - 2.0	2.0 - 4.0	READILY INDENTED BY THUMBNAIL.
HARD	>30	>2.0	>4.0	INDENTED WITH DIFFICULTY BY THUMBNAIL.

IGES®

Key to Soil Symbols and Terminology

Figure A-5

APPENDIX B

IGES

DCP TEST DATA (DCP-1)

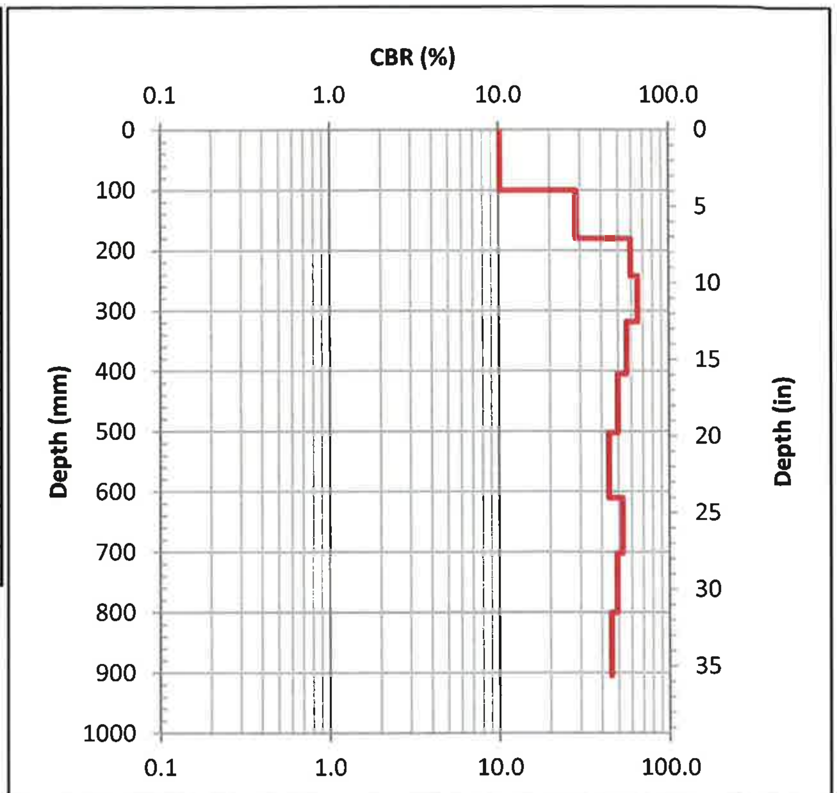
Project: 02316-001 Monte Meadows

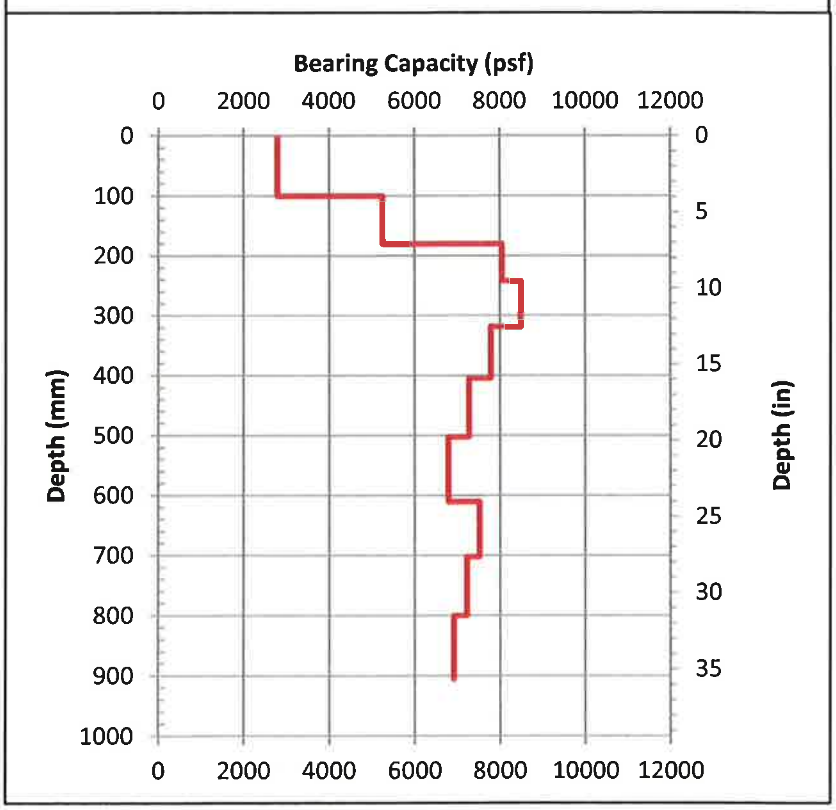
Soil Type: CLG (USCS)

Location: Riverton

Date: 3-Jun-16

No. of Blows	Penetraion Reading (mm)	Hammer Blow Factor *
0	0	1
5	100	1
10	180	1
15	242	1
20	318	1
20	405	1
20	502	1
20	610	1
20	702	1
20	800	1
20	905	1





* Enter 1 = 17.6 lb hammer;
2 = 10.1 lb hammer

CBR Values from - US Army Corps of Engineers, Technical Report No. GL-94-17

Bearing Capacity - Design of Concrete Airport Pavement, Portland Cement Association, 1955

IGES

DCP TEST DATA (DCP-2)

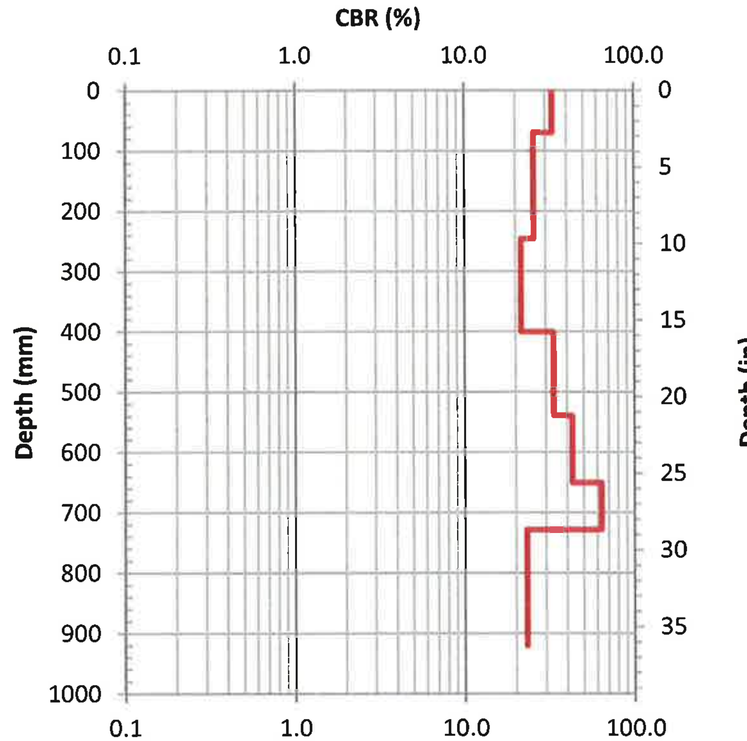
Project: 02316-001 Monte Meadows

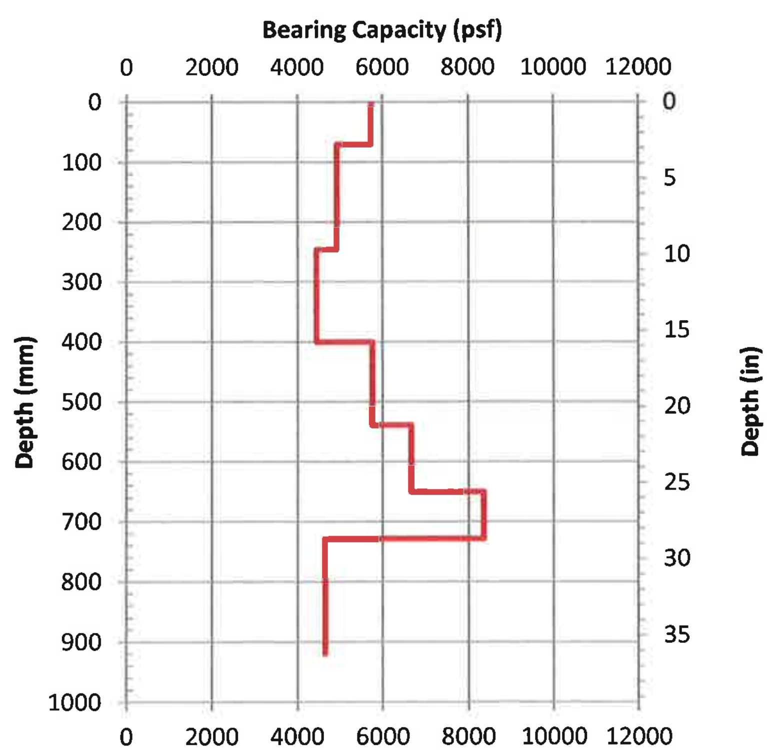
Location: Riverton

Date: 3-Jun-16

Soil Type: CLG (USCS)

No. of Blows	Penetraion Reading (mm)	Hammer Blow Factor *
0	0	1
10	70	1
20	246	1
15	400	1
20	539	1
20	650	1
20	728	1
20	920	1





* Enter 1 = 17.6 lb hammer;
2 = 10.1 lb hammer

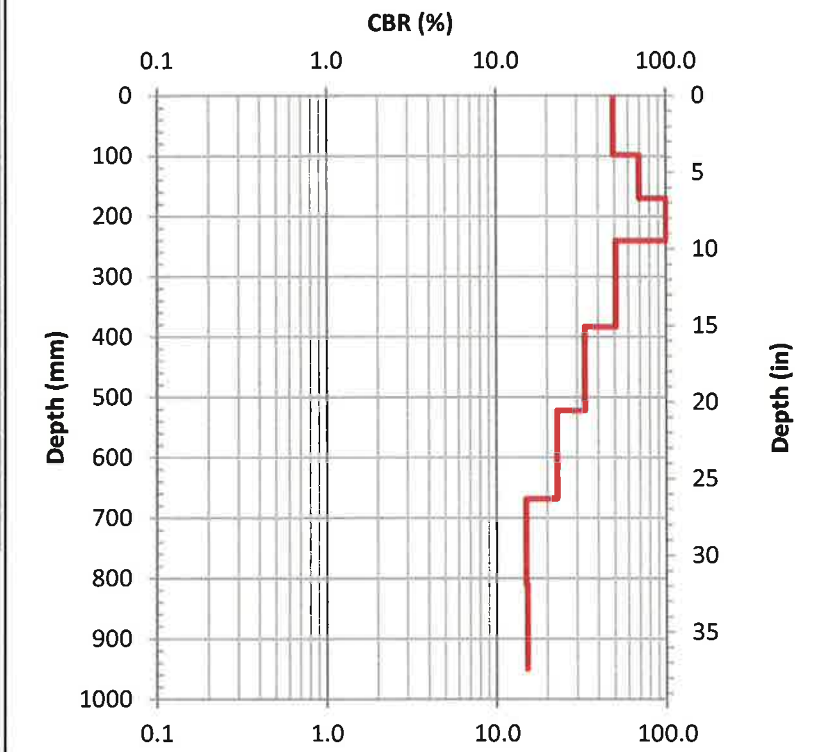
CBR Values from - US Army Corps of Engineers, Technical Report No. GL-94-17
Bearing Capacity - Design of Concrete Airport Pavement, Portland Cement Association, 1955

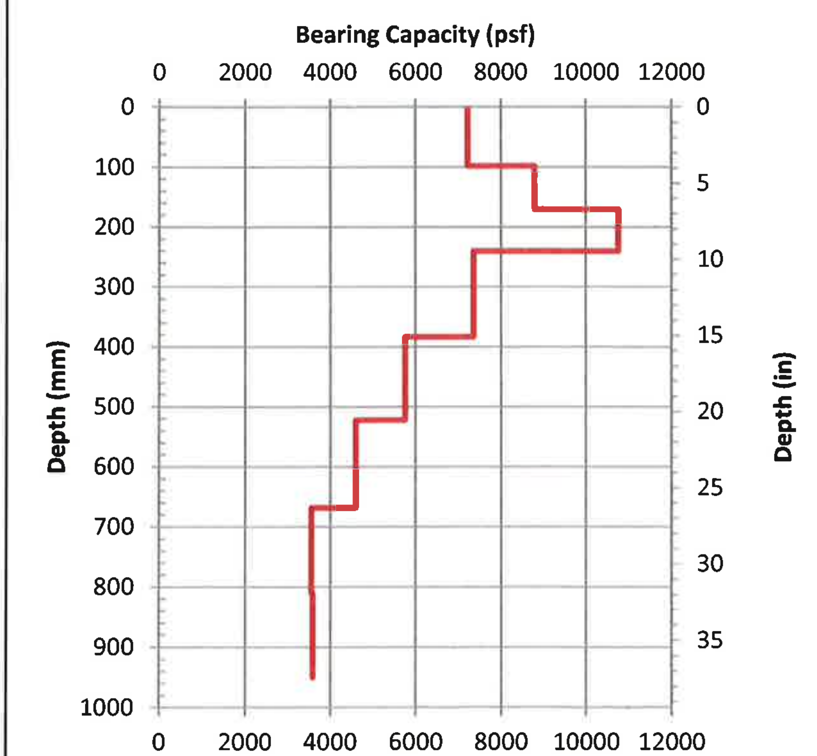
DCP TEST DATA (DCP-3)

Project: 02316-001 Monte Meadows
Location: Riverton
Date: 3-Jun-16

Soil Type: CLG (USCS)

No. of Blows	Penetraion Reading (mm)	Hammer Blow Factor *
0	0	1
20	98	1
20	170	1
30	240	1
30	383	1
20	522	1
15	668	1
10	810	1
10	950	1





* Enter 1 = 17.6 lb hammer;
2 = 10.1 lb hammer

CBR Values from - US Army Corps of Engineers, Technical Report No. GL-94-17
Bearing Capacity - Design of Concrete Airport Pavement, Portland Cement Association, 1955

IGES

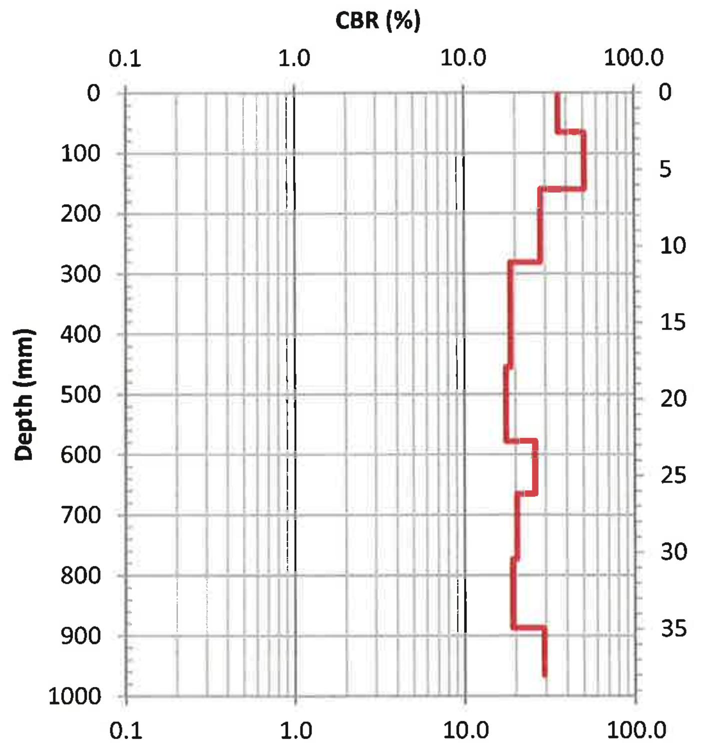
DCP TEST DATA (DCP-4)

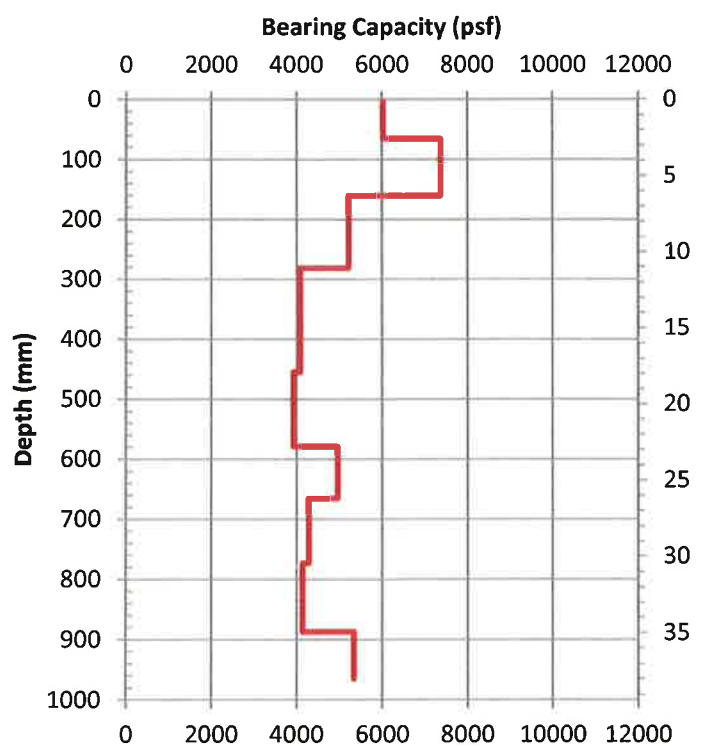
Project: 02316-001 Monte Meadows   Soil Type: CLG (USCS)

Location: Riverton

Date: 3-Jun-16

No. of Blows	Penetraion Reading (mm)	Hammer Blow Factor *
0	0	1
10	65	1
20	160	1
15	281	1
15	455	1
10	578	1
10	665	1
10	773	1
10	887	1
10	965	1





* Enter 1 = 17.6 lb hammer;
2 = 10.1 lb hammer

CBR Values from - US Army Corps of Engineers, Technical Report No. GL-94-17
Bearing Capacity - Design of Concrete Airport Pavement, Portland Cement Association, 1955

APPENDIX C

USGS Design Maps Detailed Report

ASCE 7-10 Standard (40.50895°N, 111.93602°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1] $S_S = 1.383$ g

From Figure 22-2 [2] $S_1 = 0.464$ g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_S$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index $PI > 20$, • Moisture content $w \geq 40\%$, and • Undrained shear strength $\bar{s}_u < 500$ psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk-Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.383 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.464 g, F_v = 1.536

Equation (11.4-1): $S_{MS} = F_aS_S = 1.000 \times 1.383 = 1.383$ g

Equation (11.4-2): $S_{M1} = F_vS_1 = 1.536 \times 0.464 = 0.713$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

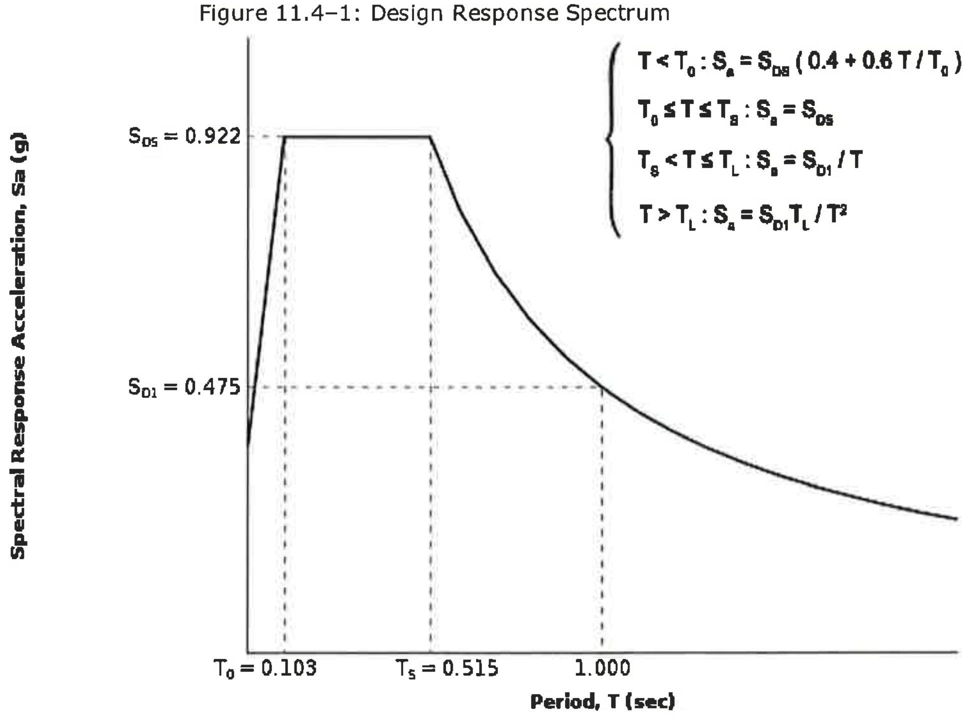
Equation (11.4-3): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.383 = 0.922$ g

Equation (11.4-4): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.713 = 0.475$ g

Section 11.4.5 — Design Response Spectrum

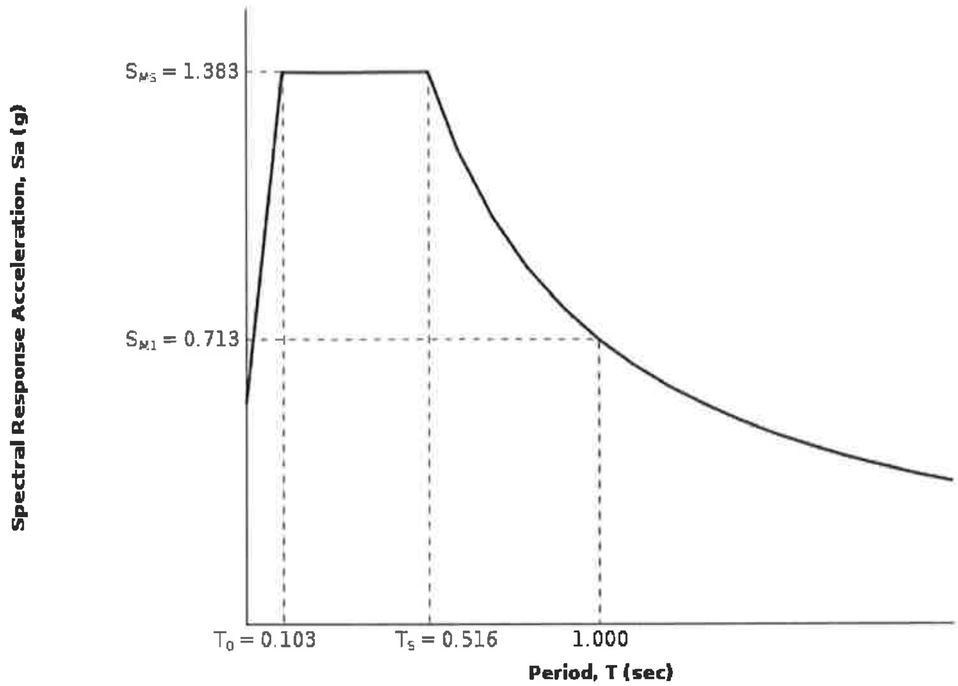
From [Figure 22-12](#) [3] $T_L = 8$ seconds

Figure 11.4-1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.584

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.584 = 0.584$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = D and PGA = 0.584 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 0.808

From Figure 22-18 [6] C_{R1} = 0.811

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \leq S_{DS} < 0.33g$	B	B	C
$0.33g \leq S_{DS} < 0.50g$	C	C	D
$0.50g \leq S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.922 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \leq S_{D1} < 0.133g$	B	B	C
$0.133g \leq S_{D1} < 0.20g$	C	C	D
$0.20g \leq S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.475 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 11.6-1 or 11.6-2" = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

USGS Design Maps Detailed Report

2012 International Building Code (40.50895°N, 111.93602°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1) [1] S_S = 1.383 g

From Figure 1613.3.1(2) [2] S_1 = 0.464 g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class D, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_S$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \leq 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \geq 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	1.0
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = D and S_S = 1.383 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1–s Period				
	$S_1 \leq 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \geq 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	1.3
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = D and S_1 = 0.464 g, F_v = 1.536

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 1.383 = 1.383$ g

Equation (16-38): $S_{M1} = F_vS_1 = 1.536 \times 0.464 = 0.713$ g

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔ S_{MS} = ⅔ \times 1.383 = 0.922$ g

Equation (16-40): $S_{D1} = ⅔ S_{M1} = ⅔ \times 0.713 = 0.475$ g

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.922 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.475 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf