July 25, 2022

Central Iron County Water Conservancy District
Brent Hunter
88 E. Fiddlers Canyon Road, Suite A
Cedar City, UT 84721

Subject: Geotechnical Investigation Report
Cedar Highlands Water Tank
Iron County, Utah
Landmark Project No.: 220353

Brent,

As requested, we have completed our Geotechnical Investigation for the above noted project. Our geotechnical recommendations, along with our field and laboratory data are presented in this report.

Our field investigation consisted of 2, 26.5- to 50.0-foot deep borings within the proposed tank footprint. Grouted micropiles are recommended for structural support of the proposed structure to control settlement. The uphill side of the tank will need to be designed to resist a lateral forces to resist movement of the surficial landslide deposits in the case of a seismic event.

Based on our findings, this site presents serious design and construction challenges which may not be able to be mitigated. Consideration should be given to selecting an alternative site.

Please feel free to contact our office at (435) 986-0566 if you have any questions.

Sincerely,
LANDMARK TESTING AND ENGINEERING

Steven Wells, P.E.
Geotechnical Manager

TABLE OF CONTENTS

GEOTECHNICAL INVESTIGATION REPORT
CEDAR HIGHLANDS TANK
IRON COUNTY, UTAH

TABLE OF FIGURES

GEOTECHNICAL INVESTIGATION REPORT
CEDAR HIGHLANDS TANK
IRON COUNTY, UTAH

1.0 INTRODUCTION

This report presents the results of Landmark Testing & Engineering's geotechnical investigation for Cedar Highlands Water Tank to be constructed in Iron County, Utah. Figure A-1 is a Vicinity Map showing the project location relative to surrounding features. Figure A-2 is a Site Map showing the proposed project layout and the approximate locations of the borings completed for this investigation.

This investigation was completed to assist in developing opinions and recommendations concerning site earthwork and foundation design.

2.0 PROPOSED CONSTRUCTION

We understand that a reinforced concrete culinary water tank is planned for construction on the site. The tank will be 65 feet in diameter, and approximately 20 feet in height.

Loading for the structure was not provided; however, we have estimated the following loads:

- Exterior Wall Footing: 6.0 klf
- Interior Spot Footings: 100 kips
- Tank Water Load: 3.0 ksf

Any significant changes to the anticipated development should be reviewed by Landmark to evaluate the continued applicability of the recommendations contained in this report.

3.0 SITE SETTING

3.1 SURFACE CONDITIONS

The site for the tank is located on a side hill, and the elevation change across the proposed tank pad area is approximately 20 feet. Due to the topography, we understand that current plans call for the tank to be partially to fully buried. The site is covered by a thick ground covering of low grasses, and moderately dense scrub oak.

3.2 GEOLOGIC SETTING

According to the Utah Geological Survey,[1] the project site is mapped as located on:

Qms **Landslides (Historical to middle[?] Pleistocene)** – Very poorly sorted, locally derived material deposited by rotational and translational movement; composed of clay- to boulder-size debris as well as large, partly intact, bedrock blocks; characterized by hummocky topography, numerous internal scarps, chaotic bedding attitudes, and small ponds, marshy depressions, and meadows; the largest landslide complexes involve the Tropic Shale and Dakota Formation (Ktd) and are several square miles in size; undivided as to inferred age because even landslides having subdued morphology (suggesting that they are older, weathered, and have not experienced recent large-scale movement) may continue to exhibit slow creep or are capable of renewed movement if stability thresholds are exceeded; age

1 Interactive Geologic Map Portal, Retrieved July 11, 2022, from Utah Geological Survey, https://geology.utah.gov/apps/intgeomap.

and stability determinations require detailed geotechnical investigations.

The surface deposits found on the site all correlate well with this formation. Landslide deposits were found in our Boring 2.

The underlying formations shown in the vicinity of the site, which are buried by the landslide deposits, consisted of:

Kcm **Tibbet Canyon Member (Upper Cretaceous)** – Yellowish-brown, medium- to thick-bedded, generally planar bedded, fine- to medium-grained quartzose sandstone and interbedded gray mudstone, carbonaceous shale, and thin to thick beds of oyster coquina; generally forms bold cliffs, although from east to west across the quadrangle, the component of interbedded mudstone, shale, and coquina increases, resulting in several alternating slopes and cliffy ledges; represents initial progradational (overall regressive) strata of the Greenhorn Cycle deposited in shoreface, beach, lagoonal, and estuarine environments adjacent to a coastal; about 650 feet thick.

Ktd **Tropic Shale and Dakota Formation, undivided (Upper Cretaceous)** – Interbedded, slope- and ledge-forming sandstone, siltstone, mudstone, claystone, carbonaceous shale, coal, and marl; sandstone is yellowish brown, thin to very thick bedded, fine to medium grained; mudstone and claystone are gray to reddish brown and commonly smectitic; oyster coquina beds, clams, and gastropods are common; 5 to 12 feet of dark gray and yellowish-brown sandy mudstone, coal, and shale near the top of the map unit represent a thin (0 to 8 feet thick) Tropic Shale and underlying Upper Culver coal zone; Tropic and Dakota strata are typically poorly exposed and involved in large landslide complexes; deposited in marginal-marine environments including floodplain, river, estuarine, lagoonal, and swamp environments for the Dakota Formation and a shallow-marine environment dominated by fine-grained clastic sediment for the Tropic Shale; the Dakota Formation is about 950 feet thick in Cedar Canyon.

Kcm **Cedar Mountain Formation (Cretaceous)** – Grayish-brown, poorly cemented, basal conglomerate overlain by brightly colored variegated mudstone; conglomerate ranges from 0 to 10 feet (0–3 m) thick and contains subrounded to rounded, pebble- to small-cobble-size quartzite, chert, and limestone clasts; mudstone is variegated gray, purplish red, and reddish brown; clay is smectitic and weathers to "popcorn-like" soils; upper contact is poorly exposed and corresponds to a color and lithologic change, from comparatively brightly colored smectitic mudstone below to gray and light-yellowish-brown mudstone.

The geologic map package includes a cross section which crosses the vicinity of the site. The bedrock formations in the area are bent and deformed due to the presence of the Hurricane fault. The layers are curved, with a dip angle of approximately 45 degrees where the formation layers surface.

A detail of the geologic map is included as Figure A-3, and includes the cross section.

3.3 GEOLOGIC HAZARDS

Cedar City lies within the transitional zone between the Colorado Plateau to the east and the Basin and Range Province to the west. Southwestern Utah is located on a structural block proximate to the southern segment of the Intermountain Seismic belt, which is characterized by high-angle normal faults that tend to step down to the west. The Hurricane fault with an offset of 6,000 to

8,000 feet forms the eastern edge of the transition zone. The Grand Wash-Reef Reservoir-Gunlock fault system with displacement of about 1,500 to 3,000 feet forms the western edge.

Fault Rupture

The trace of the Hurricane fault is located approximately 2 miles northwest of the site. Higgins and Willis (1995),[2] indicate that the Hurricane fault displaces late Quaternary sediments and is considered active. Strong ground motion associated with movement along the Hurricane or other faults associated with the Intermountain Seismic Belt is possible, however, the potential for surface fault rupture is considered low.

Liquefaction

Liquefaction is the sudden loss of shear strength in the soil due to the build-up of excess pore water pressure.[3] This can occur when the soil is subjected to intense shaking such as during a seismic event. The soils that are most susceptible to liquefaction are loose, saturated sandy soils with a low fines content (material passing the #200 sieve).

Soils encountered in the borings were soft to medium stiff clays. These types of soils are generally not considered susceptible to liquefaction in the presence of shallow ground water. A defined groundwater table was not encountered in our borings during our investigation and therefore liquefaction potential may be considered low. A liquefaction study does not appear to be warranted.

Rockfall

During our field investigation, our field geologist conducted a cursory survey of the slopes. Photographs of the slope and the boulders present on the slope are included in Appendix A, Figures A-7 and A-8.

Rockfall is generally not considered to be hazard if the slope from the source area is shallower than 22 degrees. The slope angle of the steepest potion of the site was approximately 18 degrees. Based on the lack of a defined source zone, and the shallowness of the slope, it does not appear that rockfall is a significant hazard on this site.

Landslides

The site is shown on the Geologic Map, Figure A-3 as being located on mapped landslide deposits. A mapped head scarp is shown on the map above the dirt road to the east of the site.

Defined layers consistent with historical landsliding were encountered in our borings. This evidence included debris, and possibly defined slide planes. Boring 2 extended to 50.0 feet, and evidence of landsliding was observed to 21.0 feet. Landslide deposits were not definitively seen in Boring 1, at the bottom side of the tank site. Please see Section 4.3 for local slope stability analysis for the tank site.

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2 Higgins, J.M. and Willis, G.C., 1995, Interim Geologic Map of the St. George Quadrangle, Washington County, Utah; Utah Geological Survey Open-File Report 323

3 Coduto, Donald P. (1999), Geotechnical Engineering: Principles and Practices, Prentice Hall, Upper Saddle River, NJ

The area in which the site is located is area known for largescale landslide activity. Such large-scale activity cannot be discounted and cannot be practically analyzed or mitigated.

Expansive soils

Expansive soils are clays and claystone which changes volume due to moisture content changes. Expansive soils were encountered in our borings, and recommendations for to mitigate this hazard are included in this geotechnical investigation report.

3.4 SEISMICITY

Seismicity at the site was determined using the Seismic Maps Tool from seismicmaps.org website. The following values are presented to assist with seismic design:

- Latitude = 37.63279° North, Longitude = 113.03507° West
- Risk Category = IV (public water supply)
- Site Class = D – Stiff Soil, based on ASCE 7 as referenced in 2018 IBC

Period (sec)	Sa (g)	Site Class
0.2	$S_S = 0.746$	B/C
1.0	$S_1 = 0.241$	B/C
0.2	$S_{DS} = 0.598$	D
1.0	S_{D1} = Null*	D

* See ASCE Section 11.4.8. S_{D1} is dependent upon the fundamental period of the structure.

As per Section 20.1 of ASCE 7-16, "The soil shall be classified in accordance with Table 20.3-1 and Section 20.3 based on the upper 100 feet of the site profile." However, Section 20.1 continues, "Where site specific data are not available to a depth of 100 feet, appropriate soil properties are permitted to be estimated by the registered design professional preparing the soil investigation report based on known geologic conditions." Based on our engineering experience in the area, mapped geology and the soils encountered in the borings, it is the opinion of Landmark Testing and Engineering that the soils on site classify as Site Class D.

4.0 INVESTIGATION

4.1 FIELD INVESTIGATION

Two (2) borings were completed to a maximum depth of 50.0 feet within the anticipated tank site footprint in order to characterize subsurface conditions at the site for the structure. The borings were drilled with a CME 55 drill rig utilizing 8.0-inch O.D. hollow-stem augers. Samples were obtained with a 3.0- inch O.D., split barrel, sampler driven with a 140-lb auto hammer dropping 30 inches. Depending on subsurface conditions, bag or block samples of soil were obtained from the borings. A Landmark geologist, Mr. Michael Meyers, supervised drilling and sampling operations. Approximate boring locations are shown on Figure A-2.

Soils encountered in the borings were soft to medium stiff sandy clays and weak claystone. In Boring 2, which was located at the upper side of the tank site, 2 distinct layers of debris, which appear to be slide planes, were encountered at 7.5 and 20.0 feet respectively. The layers were approximately 2 feet in thickness, and contained gravel and boulders, along with folded, spindled and mutilated variable-colored clays.

Groundwater was not encountered in the borings at the time of investigation. However, the soils had relatively high moisture contents.

The Logs of Borings are presented on Figures A-4 and A-5. A key to the soil classifications used on the logs is presented on Figure A-6.

4.2 LABORATORY TESTING

Soil samples from the borings were taken to our St. George, Utah laboratory for testing. Tests performed on the samples included moisture content and unit weight for density determination, sieve analysis and Atterberg Limits for soil classification, one-dimensional consolidation, and a corrosion suite.

Two one-dimension consolidation tests were conducted on samples of the clays. A sample from Boring 1 at 21.0 feet swelled 1.8 percent when saturated under a constant load of 2,000 psf. A second sample from Boring 1 at 26.0 feet collapsed 0.8 percent when saturated under a constant load of 2,000 psf. The unit weights of the samples were 102.7 and 116.0, respectively. The moisture content of the samples was 9.1 and 8.8 percent, respectively.

Processing the data presented for the sample from Boring 1 at 26.0 feet, yielded a Compression Index of 0.145. The preconsolidation pressure of the material was 3,000 psf, indicating a normally to underconsolidated clay.

Gradation and Atterberg limits tests were conducted on samples of the soils. The soils were relatively similar. The fines content of the samples tested ranged from 63.3 to 74.3 percent. The liquid limit of the samples tested were non-plastic and 23, and plasticity index of non-plastic and 8. These test results indicate low plasticity clays (CL) and silts (ML). The plasticity of the clays was on the bottom end of the range, just above the boundary with the sandy clays.

A standard corrosion suite was conducted on a sample of the subsurface soils from Boring 1 at 15.0 feet.

Parameter	Result	Score (AWWA C105)
Chlorides	ND	0
Sulfates	30 ppm	3.5
pH	8.3	0
Resistivity	1,380 Ω-cm	2
Moisture	Wet	2

The total AWWA score is 7.5, which is less than 10, indicating external corrosion protection is not critical. The concentration of soluble sulfates is negligible according to ACI 318.

The results of the laboratory tests are shown on the Boring Logs on Figures A-4 and A-5 and on the Summary of Laboratory Test Results on Table B-1. Individual test results are also included in Appendix B.

4.3 SLOPE STABILITY STUDY

The area which the site is located is area known for largescale landslide activity. Such large-scale activity cannot be discounted and cannot be practically analyzed or mitigated.

Due to the presence of clays and existing slide deposits, we conducted a slope stability study on the slope in the existing and proposed construction states. The profile of the slope was taken from publicly available LIDAR data. The LIDAR data and the cross section used for the profile are shown on Figure C-1. Material properties are based on data from this investigation, and shear strengths from tests conducted on similar materials. The following table:

Soil Type	Moist Unit Weight (pcf / kg/m^3)	Saturated Unit Weight (pcf / kg/m^3)	Cohesion (psf / kPa)	Internal Angle of Friction (degrees)
Slide Material	116.0 / 1,860	126.0 / 2,020	105.0 / 5.00	28.0
Sandy Clay (CL)	115.5 / 1,900	134.0 / 2,150	210.0 / 10.00	18.0
Sandy Silt (ML)	116.0 / 1,860	125.0 / 2,000	21.0 / 1.00	28.0
Weak Claystone	125.0 / 2,000	134.0 / 2,150	525.0 / 25.00	20.0

To assess the stability of the slope, the slope was modeled with FLAC/Slope 8.1. FLAC/Slope creates a finite difference model of the slope. The method divides the cross-section into a series of relatively uniform blocks in a grid. The model solves the differential equations which are used to describe the interaction of the forces within the soil profile. The seismic performance of the slope was modeled based on pseudo-static methods, using a horizontal seismic acceleration of 0.5*PGA, or 0.12g. The pseudo static method takes the seismic force as having gravity acting at an angle with a horizontal component equal to this value, or 9.64 degrees.

Industry standards recommend that the Factor of Safety (FoS), the percentage stronger than equilibrium, is 1.4 to 1.5 for the static condition, and 1.0 to 1.1 for the seismic condition. The higher values indicate a greater resistance to failure, and are recommended for conditions which require greater safety. Landslides typically, but not always, have a circular sliding or failure surface.

For the existing conditions, the seismic FoS was determined to be 0.99. This value indicated that the slope is likely to failure and slide under the design seismic event. The failure plane is a shallow surface extending through the existing slide materials and into the undisturbed clays below. The introduction of a defined water table will reduce this value. Based on the intended use of the site as part of a public water supply system, we believe a minimum seismic FoS of 1.1 is appropriate. Based on this limit, the slope cannot be considered stable. See Figure C-2 for the model output.

Construction of the pad for the tank will require the excavation of a building pad. As the construction of the tank can be considered a temporary condition, the seismic case does not apply, and a lower FoS can be used for analysis. If the back slope is cut at a slope of 2 horizontal to 1 vertical (2H:1V), the FoS was determined to be 0.96. See Figure C-3 for the model output for a 2H:1V slope. At a slope of 3H:1V, the FoS was determined to be 1.10. The 3H:1V slope overlays the failure plane of the 2H:1V slope. See Figure C-4 for the model output for a 3H:1V slope.

For the completed tank, seismic forces apply. A distributed load of 1,300 psf (62.0 kPa) was applied over the proposed tank footprint. The FoS for the slope above the tank was negligible, indicating that the slope will fail, and introduce a load on the rear side of the tank. The failure plane mirrors the failure wedge predicted in the passive case according to Rankine earth pressure theory. See Figure C-5 for the model output for the unsupported case.

To produce a stable slope with a seismic FoS of 1.0, the rear wall of the tank will require a structural design to resist a lateral load of 1,500 psf (72.0 kPa). To meet a FoS of 1.1, the tank wall will need to resist a lateral load of 2,500 psf (120.0 kPa) will be required. See Figures C-6 and C-7 for the model output for the supported case.

If the upper slope is excluded, and only the tank pad and load from the tank is considered, the FoS was determined to be 1.44. See Figure C-8 for the model output for the tank only case.

4.4 ANALYSIS

Settlement

Based on the results of our consolidation testing, parameters were obtained for bearing capacity and settlement analysis. The compression Index ($C_{\varepsilon C}$) of the clays was 0.145. This value was used to determine the expected consolidation settlement under the expected loading.

$$\delta_C = \sum_{i=1}^{n} C_C * H_i * \log(\frac{\sigma_v + \Delta\sigma_v}{\sigma_v})$$

Where:

n =	Layer
H =	Layer thickness (ft)
σ_v =	Effective stress at layer midpoint (psf)
$\Delta\sigma_v$ =	Increase in stress due to load (psf)

Under the proposed slab loading (water weight) we expect a total settlement under the center of the tank to be on the order of 1.32 feet (15.8 inches).

Under the proposed outer ring footing, the displacement settlement, the settlement due to punching shear and soil heaving, is expected to be 0.25 feet (3.0 inches). The consolidation settlement under the expected loading is predicted to be 1.62 feet (19.4 inches) for a total estimated settlement of 1.87 feet (22.4 inches).

Bearing Capacity

Based on the soil parameters listed for the slope stability analysis, bearing capacity analysis was conducted for the soft to medium stiff, sandy clays (CL). The recommended allowable bearing capacity of the sandy clays is 1,600 psf.

4.5 CONCLUSIONS

Based on the available data, the site appears to be marginally suitable for the intended purpose. The combination of low bearing capacity and large expected settlements will require ground improvement to achieve satisfactory performance. Due to the low stability of the onsite soils which form the hill above the site, the potential ground improvement methods available are limited. For example, if the soils in the slope were not sensitive to seismic loading (vibration) we would recommend rammed aggregate piers which can improve the bearing capacity of the footing areas, and control settlement for both the footings and the distributed loading of the tank floor. However, this method requires aggregate (road base) to be hammered into a drilled pier hole with a hoe-ram powered mandrel. Such vibrations are not suitable for this site.

Therefore, we recommend that the tank be supported on a grouted, drilled displacement micropile, such as Stelcore micropiles. The micropiles should be designed according to FHWA design recommendations[4] for a Type B micropile.

The presence of the existing landslide deposits and the low stability of the slope during the construction phase pose significant risks to the safety of the workers and the construction site. The structural requirements for long-term stability are significant in terms of cost and scheduling. We proposed two alternatives for mitigation of the slope stability of the site:

- A horizontal load can be applied to the rear side of the tank during structural design. A distributed load of 1,500 psf will increase the overall stability of the slope to a FS of 1.0. A distributed load of 2,500 psf will increase the stability of the slope to a FoS of 1.1.
- The uphill slope can be constructed as a soil nail retaining wall. This wall can be constructed as part of the excavation. The wall can be constructed as either a standalone wall, or as an integral part of the tank structure.

Corrosive soils do not appear to be present at this site.

Design and construction recommendations for the site are presented in Sections 5.0 and 6.0.

5.0 SITE GRADING AND EARTHWORK

5.1 GENERAL GRADING

Site preparation should initially consist of grubbing and removal of vegetation. Grubbing is expected to extend from 6 inches to 1 foot.

The proposed pad should extend a minimum of 10 feet beyond the front of the proposed tank. Due the recommendation for grouted micropiles, a ring wall footing is not required.

The temporary slope for the pad should be cut at a maximum grade of 2 horizontal to 1 vertical (2H:1V), preferably at a 3 horizontal to 1 vertical (3H:1V). 2H:1V is marginally stable under static conditions, but may become unstable under seismic conditions or increased moisture.

Prior to micropile installation, the pad should be over-excavated a minimum of 2 feet below the proposed finished pad elevation. The exposed subgrade should be scarified a minimum of 8 inches, moisture conditioned to within 2 percent of the optimum for compaction, and compacted

4 Micropile Design and Construction Reference Manual", December, 2005, FHWA NHI-05-039

to a firm, non-yielding surface. Due to the risk of pumping and movement of the pad, the pad should them be topped with 12 inches of imported, select granular fill.

Due to the risk posed by saturation of the subsurface soils, the tank should be underlain by a drainage system. The drainage system should consist of 12 inches of select granular fill, under 12 inches of free-draining gravel. The granular fill should be sloped toward the exterior of the tank at a 2% grade, and be collected in a 4-inch perforated pipe. A proposed detail of the sub-drain is included in Appendix D.

5.2 FILL PLACEMENT AND COMPACTION

Imported, select granular fill, should meet UDOT specification 02056, Granular Borrow. Specifically, be well-graded, and have a maximum particle size of 3 inches. Spec roadbase meets this definition, but other materials may also meet these requirements.

Free-draining gravel should meet specifications for coarse concrete aggregate, either #57 or #67 gravel.

Onsite soils are suitable for use as backfill behind the rear of the tank. These soils will require moisture conditioning (drying) prior to placement, and placement and compaction recommendations should be understood and followed.

Material not meeting the above requirements may be suitable for use as structural fill at the discretion of the geotechnical engineer. Samples of structural fill should be submitted for testing prior to transporting to the site.

Any on-site soils used as structural fill or imported, granular fill should be compacted to the following specifications.

FILL PLACEMENT AND COMPACTION	
Maximum lift thickness	8-inch (loose)
Minimum compaction	95% ASTM D-1557
Compacted Moisture Content	within 2% of optimum

Compaction of structural fill should be completed with equipment suitable for the conditions encountered in the field such that compaction requirements are met, including those areas that may be inaccessible to large rolling compactors. All structural fill should be evenly spread on a horizontal plane in eight-inch loose lifts. Each eight-inch lift of structural fill material placed at the site should be tested for compliance with the required relative compaction and moisture content prior to proceeding with additional lifts.

5.3 CUT AND FILL SLOPES

Permanent slopes in fill slopes constructed in the existing clays should be maintained at a maximum slope of 2 horizontal to 1 vertical (2H:1V) unless improved. This recommendation assumes that the material is compacted to structural fill standards. Uncompacted fill should be maintained a maximum slope of 3H:1V.

Cut slopes in the existing clays should be maintained at a maximum slope of 3H:1V.

All exposed slopes will require permanent erosion control to prevent erosion.

Grading of both cut and fill slopes should be such that surface water is directed away from the slopes and not concentrated on slopes or in unprotected channels. Construction procedures should ensure adequate compaction of slope faces. All excavations should conform to OSHA standards. All existing soils should be designated a Type C soil for excavation safety. Due to the low stability of the cut slopes for construction of the tank, clear zones and escape routes should be strictly enforced.

6.0 FOUNDATION & CONSTRUCTION CONSIDERATIONS

Micropiles should be designed according to the FHWA Micropile Design and Construction Reference Manual (NHI-05-039). Micropiles should be displacement piles, and designed as a Type B micropile. Micropile design for spacing and location is typically a deferred submittal by the micropile contractor.

1. Micropiles should be installed with methods which do not induce vibrations. Percussive methods should be avoided. Our borings were drilled with an 8-inch hollow-stem auger.
2. Micropiles should be installed a minimum of 30 feet into the clay. A minimum pile diameter of 8-inches is recommended to allow for insertion of the rebar and sleeve and to allow adequate grout cover. However, smaller pile diameter piles are acceptable based on structural engineer design.
3. Micropiles should be designed for an ultimate skin friction value of 1,400 psf for the portion of pile extending below 6 feet.
4. Micropiles should be designed and spaced for a minimum deadload of 10,000 psf based on the pile cross-sectional area.
5. Micropiles should be reinforced the full length to resist tension forces in the event of swelling of adjacent soils. Potential tension forces may be calculated using at least 6 feet of pile length with an ultimate skin friction of 1,400 psf. The minimum percent of steel should be 1% of the grout area. Reinforcement should extend into grade beams and foundation walls.
6. Laterally loaded piles should be at least three diameters apart and may be designed utilizing the parameters in the following table.

Soil Type	Total Unit Weight (pcf)	ε_{50} (in/in)	K value (lb/in^3)	Friction Angle	Cohesion (psf)	Unconfined Compressive Strength (psf)	Modulus of Elasticity (psi)
Medium Stiff Sandy Clay	115.5	0.01	400	18	210	425	5,500

7. The grout placed in the micro-piles should consist of Type I/II, neat cement, mixed in a high-shear mixer. Structural grout is generally mixed to between 15.3 and 16.2 lbs of cement per gallon of water. Grout should be checked frequently with a mud balance.
8. Pile installation should be observed by competent personnel to assure that adequate bearing

is achieved, clean out is performed, and the re-bar is properly placed.

7.0 LATERAL EARTH PRESSURES

7.1 LATERAL EARTH PRESSURES

Lateral loads imposed on footings may be resisted by the development of passive earth pressures against the sides of footings and friction between the base of the footing and the supporting soils. Lateral earth pressure values are presented in the following table.

Case Evaluated	Soil Type	Value
Active	On-site Sandy Clay	66 psf/ft
		122 psf/ft (with seismic)
At-Rest	On-site Sandy Clay	86 psf/ft
Passive	On-site Sandy Clay	237 psf/ft
		155 psf/ft (with seismic)
Seismic Coefficient	IBC 1610.1.1	0.236
Coefficient of friction tan(φ*0.6) where φ = 18°	On-site Sandy Clay	N/A

The values listed assume that the tank is backfilled with the onsite spoils, compacted as described below. Cohesion was neglected in the calculation. Lateral forces for the tank should be resisted by the drilled micropiles.

The lateral earth pressures presented do not include any safety factors except where the friction angle (φ) used to determine the coefficient of friction has been multiplied by 0.6 to account for smooth contact conditions. The pressures also assume horizontal backfill and that the backfill is in a drained condition with no build-up of hydrostatic pressure. The additional effects of sloping backfill, surcharge, structural loads and groundwater conditions should be included in calculating lateral earth pressures. Backfill should be placed in accordance with the requirements of structural fill except that backfill in landscape and areas that will not be subject to structural loadings may be reduced to 90 percent of the maximum dry density as determined by ASTM D-1557.

7.2 BEHIND WALL DRAINAGE

The portion of the tank which is backfilled should have a drainage system installed as part of the backfill of the tank. A moisture barrier should be installed on the tank. This barrier should meet AWWA/APWA standards for the type of tank planned. The drain should consist of free-draining gravel, as described in Section 5.2, and separated from the tank backfill by a minimum 4-oz, non-woven filter fabric. The free-draining rock should extend to the drainpipe for the base of the tank. If groundwater seepage is encountered, the pipe size needs to be increased. A detail of the proposed drain is included in Appendix D.

7.3 BACKFILL

The rear tank wall should be designed to resist the lateral earth pressured listed above, in addition to the lateral loads required for slope stability. Backfill should be placed subsequent to roof construction. Otherwise, the walls should be braced to prevent lateral movement and cracking. Backfill should be placed in accordance with the requirements of structural fill except that the compaction requirement for the backfill may be reduced to 90 percent of the maximum dry density as determined by ASTM D-1557. It is recommended that relatively light, manually propelled compactors be used within 5 feet of the walls and that compactors used beyond 5 feet be limited in weight to 3,000 pounds.

8.0 SOIL CORROSIVITY

Soils from the project site were tested for water-soluble sulfates content. The percent of water-soluble sulfates in the soil is 30 ppm, which is considered negligible according to ACI 318. We recommend that concrete mixes used on the project be designed in accordance with ACI 318 Table 4.3.1 for Sulfate Exposure Class S1.

9.0 FOUNDATION REVIEW AND TESTING

This report has been prepared to assist in project design and construction. Variations from the conditions portrayed in the exploratory investigations may occur which are sometimes sufficient to require modifications to the design. In order to incorporate recommendations provided into actual field conditions and to confirm that the project specifications are implemented, we recommend that observation and testing be performed during construction to monitor over-excavation, grading, and preparation of soils upon which foundations elements or structural loads may be established.

10.0 LIMITATIONS

The exploratory data presented in this report were collected to provide geotechnical design recommendations for this project and subsurface site descriptions represent conditions observed at the time and at the locations explored. The investigations may not be indicative of subsurface conditions beyond the investigation location and conditions may change with passage of time. If subsurface conditions are encountered that are significantly different than those reported herein, Landmark should be contacted immediately for the continued applicability of the recommendations. In the event changes to the project are made that differ from those presented in this report, Landmark should be made aware of the changes. Landmark will provide written verification that the recommendations and conclusions remain valid or that modifications are required.

This report has been prepared to assist in project design and construction. We respectfully request the opportunity to review the final design drawings and specifications in order to determine whether the assumptions and recommendations presented herein are applicable to the anticipated designs.

This report is not intended to be used as a bid document. Any information concerning the environmental conditions of the site is beyond the scope of this geotechnical study. This geotechnical report has been prepared to meet the specific needs of our client and may not be appropriate to satisfy the needs of other users.

LANDMARK TESTING & ENGINEERING

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Reviewed by:

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