Introduction

As part of the Sunnyside Canal Improvement Project in FY 2006, five new canal check structures are proposed at Milepost (M.P.) 43.65, 50.76, 52.63, 55.21, and 57.55. Existing check structures at these locations will be replaced by new check structures. A field exploration program was conducted in April 2006 in order to evaluate subsurface conditions and provide design recommendations for the proposed check structures. This memorandum describes the field exploration program, summarizes the subsurface conditions, presents the laboratory test data, and provides recommendations for use in design of the check structures.

Subsurface Exploration and Conditions

This section summarizes the exploration program including the drilling program, site specific subsurface soil conditions, and groundwater conditions.

Drilling Program

The field subsurface exploration program consisted of drilling a total of five test borings (B-43.65, B-50.76, B-52.63, B-55.21, and B-57.55) on April 10-11, 2006. The borings were drilled along the canal access road near the locations of each proposed check structure. Drilling was conducted by Haz-Tech Drilling of Meridian, Idaho, under subcontract to CH2M HILL. A CME-75 truck mounted rig with a 140-pound autohammer was used for the drilling. Drilling techniques included hollow-stem augers and split-spoon sampling in soil-like conditions, and continuous HQ triple-tube rock coring in fractured and solid bedrock.
Laboratory Testing Program
A limited laboratory testing program was performed to confirm field classifications, evaluate compaction characteristics, and evaluate corrosion properties. The laboratory test methods and test results are presented in Appendix B.

Subsurface Conditions
The subsurface materials observed during the field exploration varied between each proposed check structure location. Subsurface materials encountered during the investigation included sandy silts, silty sands and gravels, moderately to highly weathered basalt bedrock, and non-weathered basalt. The subsurface conditions found at each proposed check structure are described below. Note that because the borings were drilled on the canal access road, the depths to the layers are below the elevation of the access road. The cross sections illustrate how the subsurface conditions relate to the position of the canal inverts (which are typically 6 to 10 feet lower in elevation).

The boring logs are included in Appendix A. Figure 1 shows the locations of the proposed check structures and boring locations. Figures 2 through 6 show a simplified cross section of the canal geometry with soil, rock and groundwater conditions at each proposed structure location.

MP 43.65 (Existing Check #23)
Boring B-43.65 was drilled to a depth of 36.5 feet on the south side of the canal about 100 feet east (downstream) of the existing Check 23. The subsurface at B-43.65 consisted of brown, moist to wet, firm to stiff, silt to silt with sand to a depth of approximately 30 feet. Laboratory testing of this material indicated this material is non-plastic silt, with up to 18 percent fine sand. Compaction testing showed a maximum density of 111 pounds/cubic feet at 15.5 percent moisture. A lens of brown, wet, fine-grained poorly-graded sand was found between 30 and 31 feet in depth. Pinkish-white cemented calcium material (caliche) was found at 31 feet; this is interpreted to overlie bedrock. Below 31 feet in depth the drilling action, combined with weathered basalt recovered in the SPT drive shoe at 35 feet, indicated weathered basalt below 31 feet in depth.

MP 50.76 (Existing Check #26)
Boring B-50.76 was drilled to a depth of 15 feet on the south side of the canal about 20 feet west (upstream) of the existing Check 26. The subsurface at B-50.76 consisted of about 6 feet of gravelly sand with gravel underlying the canal bank. Between 6 and 9 feet in depth the subsurface material was brown silty clay; interpreted to be the original surface soils underlying the canal berm. At 9 feet in depth, a glassy volcanic rhyolite was encountered. This material is described as yellow-tan, glassy, fractured, hard, microcrystalline quartz. Underneath this material, at 12.7 feet in depth, dark brown, vesicular, fine-grained, moderately weathered basalt with RQD of zero was found.

MP 52.63 (Existing Check #27)
Boring B-52.63 was drilled to a total depth of 15 feet on the south side of the canal about 20 feet west (upstream) of the existing Check 27. The subsurface at B-52.63 consisted of 5 feet of sand and gravel canal embankment, overlying weathered basalt. The weathered basalt is between 5 and 10 feet in depth, and could be penetrated (slowly) by the hollow stem auger. At 10 feet,
the basalt appeared to transition to dark gray, fine-grained, moderately weathered basalt with RQD of 9 percent. Fractures in this basalt were weathered and stained with planar surfaces.

**MP 55.21 (Existing Check #28)**

Boring B-55.21 was drilled to a depth of 41.5 feet on the south side of the canal about 100 feet east (downstream) of the existing Check 28. The subsurface at B-55.21 consisted of 8 feet of brown, dry, firm, low-plasticity silt with very fine sand. The silt was underlain by brown, dry to wet, very dense sandy to silty gravels with interbedded medium dense gravelly sands to the total depth. Below approximately 30 feet in depth these materials were wet.

**MP 57.55 (Existing Check #29)**

Boring B-57.55 was drilled to a depth of 15 feet on the east side of the canal about 20 feet south (upstream) of the existing Check 29. Basalt is exposed in the canal at this location. The subsurface at B-57.55 consisted of about 5 feet of silty, sandy gravel, overlying basalt bedrock. The basalt from 5 to 11 feet in depth was described as dark gray with oxidation staining, fine-grained, slightly weathered, with RQD of zero. Below 11 feet in depth the basalt was described as dark gray, vesicular, fresh, with RQD of 78 percent.

**Groundwater Conditions**

No piezometers were installed in the borings to measure groundwater levels. In addition, drilling fluid is circulated during rock coring which makes groundwater measurements impractical. Groundwater was noted in borings 43.65 and 55.21 at approximately 30 feet in depth, based on wetness observed in the samples. However, the borings were drilled very soon after irrigation began in the spring of 2006 and the canals filled with water. It appears that perched, shallow groundwater may be present in the area as a result of irrigation deep percolation and canal leakage. In irrigated areas, typically groundwater levels rise with the onset of irrigation season, and then fall during non-irrigated periods.

**Corrosion Potential**

The laboratory test results indicate the silty soils are slightly basic (pH = 8.5) with resistivity between 3300 and 4900 ohm-cm. These results indicate a mildly to moderately corrosive environment for steel, respectively. The silty soils are not expected to be corrosive toward concrete.

**Seismic Conditions**

This section provides a brief summary of the area seismicity for the check structure sites. This is a cursory summary and the reader is referred to the report titled, *Geotechnical Canal Improvement Project, MP 59.29 Reservoir*, by CH2M HILL (2004) which provides more detail of the regional seismic setting, historical seismicity, and seismic source zones for Central Washington.

**Seismic Hazards**

Seismic hazard areas are defined by Washington State Department of Community, Trade and Economic Development (2003) as those that include areas subject to severe risk of damage as a result of earthquake induced ground shaking, slope failure, settlement, soil liquefaction, or
surface faulting. One indicator of potential for future earthquake damage is a record of earthquake damage in the past. In Washington, ground shaking is the primary cause of earthquake damage and the strength of ground shaking is primarily affected by:

• The magnitude of an earthquake.
• The distance from the source of an earthquake.
• The type of thickness of geologic materials at the surface.
• The type of subsurface geologic structure.

Settlement and soil liquefaction conditions occur in areas underlain by cohesionless soils of low density, typically in association with a shallow ground water table.

Area Seismicity
The project sites are located within the Columbia Basin, which is underlain by the Columbia River Basalt Group (CRBG). The sites are located in a subprovince that has been termed the Yakima Fold Belt, a series of anticlines, synclines, and thrust faults in the Columbia Basin of central Washington (Schuster and Moses, 2002; Lasmanis, 1991; Yeats et al., 1997; Reidel and Campbell, 1989; Campbell, 1989). The development of the folds began during the formation of the CRBG and has continued to the present, leading to the formation of numerous thrust faults within the Yakima Fold Belt (Schuster and Moses, 2002; Lasmanis, 1991; Yeats et al., 1997).

The sites are located in a region of relatively little historic seismic activity (CH2M HILL, 2004) with few earthquakes having occurred within a 60 mile radius of Sunnyside. Three moderate earthquakes occurred within a 60 mile radius of the site (USGS, 2004), one with a magnitude 4.7 occurred in 1893 in Umatilla, Oregon (Myers and price, 1979), located about 55 miles southeast of Sunnyside. Two events occurred with magnitude 4.8 since 1973. One was in 1973, about 60 miles north of Sunnyside, near Othello, Washington and the other was in 1981, about 58 miles west of Sunnyside, near Tieton, Washington. Earthquakes over a magnitude 4.8 have not been reported by USGS within a 60 mile radius of Sunnyside since 1568.

The Yakima County CAO reports that earthquake activity in Yakima County is mostly marked by low magnitude events and thus has a low seismic risk. One exception is an area along Toppenish Ridge where Holocene faulting may have produced earthquakes of as much as magnitude 7. The rest of the county is relatively low risk for earthquake related damage. The Yakima County Critical Areas Ordinance (CAO) reports that the earthquake potential in Yakima County is considered low to moderate.

Design earthquake accelerations have been determined from the probabilistic seismic hazard modeling conducted by the United States Geologic Survey (USGS) (USGS, 2002) and adopted by the Federal Emergency Management Agency (FEMA) (FEMA, 2001), and the International Building Code (IBC) (IBC, 2003). The modeling considers known seismic sources and the maximum credible earthquake and return interval associated with each source, including the Seattle Fault and subduction earthquakes similar to the Nisqually earthquake.

Site-Specific Seismic Design Parameters
The USGS probabilistic study indicates that the peak firm ground acceleration with a 10 percent chance of occurrence over a 50-year period (approximately 500-year return period) is 0.10g, and
0.22g for earthquakes with probabilities of exceedance of 10 percent in 50 years, and 2 percent in 50 years, respectively (USGS 2002). The equivalent return periods for these probabilities are approximately 500 years, and 2,500 years. The mean moment magnitude associated with these return periods is approximately magnitude 6.1.

Ground motions at the site will be a function of both the firm-ground acceleration (given above) and the soil response. The soil site class consists of a weighted average of the soil properties (shear wave velocity or standard penetration resistance) in the upper 100 feet. Soils at the project locations were assigned three different Site Classes, based on the variety of subsurface conditions in the upper 100 feet at each site. The Site Class at MP 43.65 is assumed to be Site Class D, which is a stiff soil profile. This is based on the average properties of the soil profile, which consists of Standard Penetration Resistance (N) of <15 (the average N value in boring B-43.65 is 12) from 0 to 20 feet, N equal to 15 from 20 to 30 feet, and N greater than 50 and rock below a depth of 30 feet. The Site Class at MP 55.21 is assumed to be Site Class D, which is a stiff soil profile, based on Standard Penetration Resistance (N) of between 15 and 50 (the average N value in boring B-55.21 is 40). The Site Class at MP 50.76, 52.63, and 57.55 is assumed to be Site Class B, which is rock with an estimated shear wave velocity between 2,500 and 5,000 feet per second.

The 2003 IBC provides spectral accelerations for an earthquake with a 2 percent probability of occurrence in 50 years or roughly a 2,500-year recurrence interval. The magnitude of acceleration associated with this event is needed for liquefaction analysis. The mean magnitude for short period accelerations during a 2,500-year earthquake is 6.1. The mean magnitude is based on a deaggregation of the PGA for a 2,500-year event taken from the USGS website (USGS, 2002).

Seepage and Uplift Pressure Analysis

Seepage analysis is used to evaluate the potential for piping (internal erosion) of soils and possible undermining of the structures by seeping water. Estimated uplift pressures are required for the structural design of the structures.

Two methods were used to analyze seepage and seepage cutoff for the new check structures at MP 43.65 and MP 55.21: Lane’s Weighted-Creep Method and flow net. The flow net method was also used to analyze uplift pressures on the downstream sides of these check structures. The seepage analyses are based on design maximum water levels upstream of the structures (as indicated on the preliminary drawings and design criteria) with the gates closed and a drained canal downstream. However, it should be noted that this condition is encountered infrequently and is expected to occur only in the rare event of a canal dike failure.

The predominant soil type encountered at the MP 43.65 check structure is non-plastic silt with sand. Soil samples show that more than 80 percent of the material passes the No. 200 sieve size. This soil type is particularly susceptible to piping (erosion). The predominant soil types encountered at the MP 55.21 check structure are gravels and sands, overlain by silts. Again, silts are susceptible to piping.

Seepage and uplift for the new check structures at MP 50.76, MP 52.63, and 57.55 were not analyzed because these structures will be founded directly on or close to basalt bedrock. Hydraulic flow characteristics within the basalt bedrock are expected to vary widely and be
governed by fracture flow. Effective seepage cutoff through basalt bedrock can be attained by pressure grouting, but pressure grouting is not feasible for these check structures because of the shallow depth of the expected seepage zone. The recommended design approach for these three structures is to assume full hydrostatic uplift on the downstream side of the check structure and to use concrete cutoff walls extended through the surficial soils to the top of bedrock at a minimum. With the concrete cutoff walls in intimate contact with the bedrock surface, and preferably notched (trenched) into the bedrock as deep as can be excavated with reasonable effort, piping of the surficial soils is expected to be controlled. These concrete cutoff walls should extend vertically from the structure to bedrock and full-depth from edge of check structure on one bank to edge of check structure on the other bank. In other words, the bottom of the concrete cutoff should extend to the rock surface not only along the canal bottom, but also within the canal banks.

Lane’s Weighted-Creep Method

Lane’s Weighted-Creep method of analysis is described in the Bureau of Reclamation publication Design of Small Canal Structures. This method provides a procedure for calculating the length of the path water would follow as it seeps past a hydraulic structure. The seepage path is the sum of weighted lengths of “creep” (seepage) along the soil/structure interfaces or though the soil. Weighting factors are assigned depending on whether the seepage path is vertical, horizontal, or a “short cut” through soil. This cumulative weighted-creep length is then divided by the difference in water surface elevations upstream and downstream of the structure (the maximum hydraulic head) to determine a weighted-creep ratio. Lane provides recommended minimum allowable weighted-creep ratios for various soil types. For soils classified as “very fine sand or silt” (similar to the soils found at the MP 43.65 and MP 55.21 check structure sites), the recommended ratio is 8.5 to 1.

Cutoff wall depths of 10, 15, and 20 feet were analyzed for the check structures at MP 43.65 and MP 55.21. At the MP 43.65 check structure, a cutoff wall at least 20 feet deep is recommended. The calculated weighted-creep ratio is 6.4, which does not meet the minimum of 8.5 for silts. In order to achieve 8.5, a cutoff wall depth of 30 feet is required. However, as discussed below, the flow net analysis indicates a 20-foot cutoff depth is adequate.

At MP 55.21, a cutoff wall at least 10 feet deep is recommended. The calculated weighted-creep ratio for the 10-foot cutoff wall is 8.6, which exceeds the minimum recommended ratio of 8.5. The primary reason for the different ratios at these two structures is the difference in hydrostatic head.

Steel sheet piles cutoff walls, as opposed to deep concrete cutoff walls, are recommended for these two check structures because of the presence of saturated soils in the canal bottom. Excavation of a narrow, deep trench for a cutoff wall and placement of concrete for a cutoff wall would be difficult and costly given these conditions.

Analysis by Flow Net Method

Flow nets were developed for the MP 43.65 (existing Check No. 23) and MP 55.21 (existing Check No. 28) check structures using SLIDE, a two-dimensional finite element program for groundwater analysis by Rocscience, Inc. The flow nets were used to check the exit gradient
and uplift pressures under the structures for several cutoff depths assuming full differential head across the structure.

Exit gradient is the gradient at the point where seepage water discharges from under the structure. If soil is unconfined or unprotected by properly graded filter material, piping and quick conditions can begin to develop at exit gradients of about 1.0. Table 1 shows a summary of calculated exit gradients for cutoff depths of 10, 15, and 20 feet for the check structures, along with uplift pressures as explained below.

**TABLE 1**
Calculated Exit Gradients and Uplift Pressures

<table>
<thead>
<tr>
<th>Check Structure</th>
<th>Cutoff Depth (ft)</th>
<th>Exit Gradient</th>
<th>Maximum Uplift Pressure (psf)</th>
<th>Minimum Uplift Pressure (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>MP 43.65</td>
<td>10</td>
<td>0.81</td>
<td>316</td>
<td>53</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.77</td>
<td>289</td>
<td>53</td>
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<td></td>
<td>20</td>
<td>0.71</td>
<td>263</td>
<td>26</td>
</tr>
<tr>
<td>MP 55.21</td>
<td>10</td>
<td>0.25</td>
<td>138</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>0.21</td>
<td>107</td>
<td>15</td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>0.18</td>
<td>92</td>
<td>15</td>
</tr>
</tbody>
</table>

*Note:* Maximum uplift pressure occurs at the point closest to the cutoff wall. Minimum uplift pressure occurs at the point furthest from the cutoff wall.

An exit gradient of less than 0.5 is typically recommended and provides a factor of safety of 2. However, in the case of MP 43.65, the exit gradient with a 20-foot cutoff wall is 0.71, which provides a factor of safety of 1.4. This is acceptable considering several factors. First, the full differential head condition is expected to rarely occur; it is not a long-term condition. Second, the hydraulic conductivity of silt soils is relatively low, and the seepage velocity is not sufficient to erode the soils at the exit point. Third, the flow net analysis assumes saturated, steady-state flow. The field exploration data indicate the subsurface soil profile is not saturated throughout the year. Although water infiltrating from the canal will create a seasonal shallow groundwater table, it may be perched and the entire soil column may not become saturated. In other words, the flow net is providing a conservative estimate of the exit gradient.

The uplift pressures shown in Table 1 were also calculated by flow net for cutoff depths of 10, 15, and 20 feet. These uplift pressures act on the bottom of the MP 43.65 and MP 55.21 structures. The pressure distribution varies from the cutoff wall to the downstream toe, and for design purposes a linear pressure distribution between the maximum and minimum can be assumed.

As stated previously, a maximum uplift pressure equal to the full hydraulic head differential should be assumed to act across the entire bottom of the other three check structures.

For all five check structures, the pressure distribution varies linearly from the bottom up the banks to zero at the design upstream water elevation.
Geotechnical Recommendations

This section provides the following geotechnical recommendations for the design and construction of the check structures for the Sunnyside Canal:

- Foundation Subgrade Preparation
- Earthwork and Grading
- Allowable Bearing Pressures
- Lateral Earth Pressures for Buried Structures
- Tiedowns to Resist Uplift
- Corrosion Potential

Foundation Subgrade Preparation

The project site should be cleared prior to construction by removing riprap, concrete, debris, and removing all vegetation, organic soils, and any other deleterious materials.

Site subgrade material exposed will vary depending on location and depth of excavations. In general, concrete should be placed directly against native undisturbed earth. Soft areas should be excavated to firm bearing soils and replaced with suitable backfill material. Under the check structures, suitable backfill should ideally possess low permeability to help control seepage. If a cavity must be backfilled with soil, the zone should be backfilled with densely compacted native silt or clay soils having a minimum of 50 percent passing the No. 200 sieve size. However, this must be balanced against design needs for sliding friction, bearing capacity, material availability, and so forth, which are probably best met by granular backfill. Because some form of seepage cutoff wall will be installed at each check structure, the need for low permeability backfill is lessened.

Earthwork and Grading

Grading or backfill under roads and utilities should consist of granular fill meeting the WSDOT Standard Specifications for Gravel Borrow, Section 9-03.14(1) with gradation altered to limit maximum particle size to 2 inches. The backfill should be placed in uniform layers not exceeding 10 inches in loose thickness. In areas where small hand compaction equipment is required, the backfill should be placed in lifts not exceeding 4 inches in loose thickness. The fill should be compacted to a minimum of 95 percent of the maximum dry density and within 2 percent of optimum moisture content, as determined according to ASTM D698, and should be non-yielding. It is important to note that although this relative compaction is typically easily achieved, it will be difficult if not impossible to achieve if fine-grained subgrade soils are not dewatered, as described below.

Allowable Bearing Pressures

Allowable bearing pressures based on canal invert soil types at the various check structure locations derived from presumptive bearing pressures presented in NAVFAC DM-7.2 and engineering judgment are presented in Table 2.
The depth below the canal invert to the top of rock is estimated at 3 feet at MP 50.76. The depth below the canal invert to the top of gravel at MP 55.21 is 1 foot. The clay and silt overburden at these two locations should be over-excavated and replaced with compacted granular backfill if the higher allowable bearing pressures are needed. The allowable bearing pressure of the compacted granular backfill should be taken as equal to the allowable bearing pressure of the underlying material at these two locations.

**Lateral Earth Pressures for Buried Structures**

For design of buried structures, lateral earth pressures and coefficient of friction, tan δ, vary by soil type, as shown in Table 3. It is assumed that granular backfill will be placed adjacent to structure footings at MP 50.76 and MP 55.21, where relatively thin layers of clay and silt, respectively, overlie fractured rock.

### TABLE 3
Lateral Earth Pressures and Coefficients of Friction

<table>
<thead>
<tr>
<th>Check Structure</th>
<th>Soil Type</th>
<th>$K_s$</th>
<th>$K_o$</th>
<th>$K_p$</th>
<th>Moist Unit Weight (pcf)</th>
<th>tan δ</th>
</tr>
</thead>
<tbody>
<tr>
<td>MP 43.65</td>
<td>Silt</td>
<td>0.4</td>
<td>0.6</td>
<td>2.6</td>
<td>100</td>
<td>0.3</td>
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<tr>
<td>MP 50.76</td>
<td>Granular Backfill</td>
<td>0.3</td>
<td>0.5</td>
<td>3.0</td>
<td>130</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Fractured Rock</td>
<td>0.3</td>
<td>0.5</td>
<td>3.0</td>
<td>165</td>
<td>0.65</td>
</tr>
<tr>
<td>MP 52.63</td>
<td>Fractured Rock</td>
<td>0.3</td>
<td>0.5</td>
<td>3.0</td>
<td>165</td>
<td>0.65</td>
</tr>
<tr>
<td>MP 55.21</td>
<td>Granular Backfill</td>
<td>0.3</td>
<td>0.5</td>
<td>3.0</td>
<td>130</td>
<td>0.55</td>
</tr>
<tr>
<td></td>
<td>Native Gravel</td>
<td>0.3</td>
<td>0.5</td>
<td>3.0</td>
<td>130</td>
<td>0.55</td>
</tr>
<tr>
<td>MP 57.55</td>
<td>Fractured Rock</td>
<td>0.3</td>
<td>0.5</td>
<td>3.0</td>
<td>165</td>
<td>0.65</td>
</tr>
</tbody>
</table>

At MP 43.65 where the soil type is silt, overexcavation of the silt and backfill with granular soil may be preferable to improve the coefficient of friction and passive lateral resistance. If this approach is used in the design analysis, the potential for shearing and sliding along the granular backfill/silt interface should be checked to ensure structural stability. An angle of
internal friction of 26° may be assumed for analyzing shearing and sliding at the granular backfill/silt interface.

Active earth pressures should be used only when the structure is free to move by translation or rotation at least 0.002*H, where H is the total height of soil above the bottom of the wall. Where the wall cannot move, at-rest earth pressures should be used. Movement of about 0.02*H is required to mobilize full passive resistance; a factor of safety of 1.5 should be added to the passive earth pressure to limit the amount of movement. Buoyant unit weights for soil pressures should be used in computing lateral earth pressures, and full hydrostatic pressures should be applied.

A factor of safety against sliding of 1.5 and a factor of safety against overturning of 1.5 are recommended. A factor of safety of 1.0 is recommended for seismic conditions.

**Tiedowns to Resist Uplift**

In order to resist hydraulic uplift pressures on the check structures at MP 50.76, MP 52.63, and 57.55, tiedown anchors grouted into the basalt bedrock are recommended. The primary issues associated with tiedown anchors are general stability, change in loads due to movement, and corrosion protection of the anchor. The stability is evaluated by analyzing the structure weight and the resisting weight of the rock and soil mobilized by the anchor. Change in loads due to movement could be caused by settlement, creep, or heave of the affected soil; however, this is not a concern for these three Sunnyside check structures. In order to protect against corrosion, epoxy-coated bars are recommended.

Additional parameters recommended for use in design of tiedown anchors are:

- Minimum hole diameter of 6 inches
- Uplift capacity of the anchor is the effective (buoyant) weight of a cone-shaped volume of rock. The cone angle should be 60° (in other words, 30° from vertical). Where soil overburden is above the rock, the shape of the soil mass is cylindrical, projected up from the cone at the soil/rock interface. The apex of the cone is assumed to be at the center of the anchor bond zone.
- Overlapping cones of adjacent anchors should not double-count weight.
- Rock and soil shear strength should be ignored.
- Minimum bond length of 10 feet
- Presumptive ultimate bond stress at the grout/rock interface of 250 psi
- Factor of safety of 2 to 3

**Other Construction Considerations**

**Dewatering**

Dewatering of the construction zones is recommended at sites where saturated fine-grained soils, particularly silts, exist at or near the surface. As can be seen in the test results for samples
taken from B-43.65, the seasonally saturated soils are at a moisture content greater than 30 percent while the optimum moisture content is 15.5 percent. The soils are expected to be saturated with perched water for several weeks to months following the end of irrigation season. If the soils are saturated or near saturation, the soils are expected to soften and “pump” when disturbed; trafficking and earthwork, particularly compaction, will become virtually impossible. In order to work on or in these saturated soils, they must first be dewatered.

The least expensive method of dewatering the soils is by excavating dewatering trenches to a depth of 5 or 6 feet upstream and downstream of the work zone. If perched groundwater migrates into the work zone from the canal banks, additional trenches along the banks are recommended. It is recommended that these trenches be shown on the construction drawings in order to ensure the contractor is aware of the need. If it turns out that the soils are not saturated at the time of construction, excavation of the dewatering trenches can be waived by the construction manager. The drawing notes or specifications should also require sufficient time (up to several weeks) in the construction schedule to allow the soils in the construction zone to drain.

Sheet Pile Driving
Driving tips are recommended for driving sheet piles to construct the cut-off walls. At MP 55.21, Z-section sheet piles are recommended to prevent sheet pile damage during driving into the gravels. The determination of whether to drive the sheet piles with a vibratory hammer or an impact hammer should be left up to the contractor.

Construction Access
Construction access to each of the check structures will be from the Sunnyside Canal road.

Stockpile and Staging Areas
Stockpile and staging areas are available nearby each of the check structures. Additional staging areas should be identified if needed for construction.

Waste Area
The Contractor will be responsible for locating suitable disposal sites as required to perform the work.

References


**Attachments**

- Figures 1 – 6
- Appendix A – Boring Logs
- Appendix B – Laboratory Test Results
Figure 2
**PROJECT NUMBER:** 335880.03.CH.03  **BORING NUMBER:** B-43.65  
**SHEET:** 1 of 2  

**PROJECT:** Sunnyside Valley Irrigation District  
**LOCATION:** MP 43.65  
**ELEVATION:** Approximately 838 ft  
**DRILLING CONTRACTOR:** HazTech  
**DRILLING METHOD AND EQUIPMENT:** CME 750 H.S.A.  
**WATER LEVELS:** Not Measured  
**START:** 4/11/06  **FINISH:** 4/11/06  **LOGGER:** G. Warren, P.G.

<table>
<thead>
<tr>
<th>DEPTH BELOW</th>
<th>INTERVAL</th>
<th>SAMPLE</th>
<th>NUMBER and TYPE</th>
<th>RECOVERY (FT)</th>
<th>STANDARD PENETRATION TEST RESULTS</th>
<th>SOIL DESCRIPTION</th>
<th>COMMENTS</th>
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<td></td>
<td></td>
<td></td>
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</tr>
<tr>
<td>5</td>
<td>5.0-6.5</td>
<td>SS-1</td>
<td>1.5</td>
<td>1-2.5 (7)</td>
<td></td>
<td>SILT with Sand (ML), wet, very fine sandy silt lenses, brown, moist, firm, medium plasticity.</td>
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<td>10</td>
<td>10.0-11.5</td>
<td>SS-2</td>
<td>1.4</td>
<td>2-4-10 (14)</td>
<td></td>
<td>SILT (ML), brown, wet, stiff, rapid dilatancy, massive.</td>
<td></td>
</tr>
<tr>
<td>15</td>
<td>15.0-16.5</td>
<td>SS-3</td>
<td>1.5</td>
<td>1-3-5 (8)</td>
<td></td>
<td>SILT (ML), brown, moist, firm, medium dilatancy.</td>
<td></td>
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<tr>
<td>20</td>
<td>20.0-21.5</td>
<td>SS-4</td>
<td>1.5</td>
<td>3-5-10 (15)</td>
<td></td>
<td>Very fine Sandy Silt (ML), interbedded with silty clay (CL), brown, moist, stiff, low plasticity, silt is rapid dilatancy.</td>
<td></td>
</tr>
</tbody>
</table>

**SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY**

**DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION**

Downstream on right bank access road.

Bulk sample collected from 5.0 to 10.0 ft.

PP = 0.75, 1.0, 0.75 tsf. Easy, fast augering.

PP = 1.5, 1.5, 1.5 tsf.

PP = 1.5, 2.0 tsf in clay.
### Soil Boring Log

**Project:** Sunnyside Valley Irrigation District  
**Location:** MP 43.65  
**Elevation:** Approximately 838 ft  
**Drilling Contractor:** HazTech  
**Drilling Method and Equipment:** CME 750 H.S.A.  
**Water Levels:** Not Measured  
**Start:** 4/11/06  
**Finish:** 4/11/06  
**Logger:** G. Warren, P.G.  

#### Soil Description

<table>
<thead>
<tr>
<th>Depth Below</th>
<th>Interval</th>
<th>Sample Number and Type</th>
<th>Recovery (ft)</th>
<th>Standard Penetration Test Results</th>
<th>Soil Name, USCS Group Symbol, Color, Moisture Content, Relative Density or Consistency, Soil Structure, Mineralogy</th>
<th>Comments</th>
</tr>
</thead>
<tbody>
<tr>
<td>25.0 to 26.5</td>
<td>SS-5</td>
<td>1.5</td>
<td>3-6-9 (15)</td>
<td>Interbedded Silt (ML) and Lean Clay (CL), like above.</td>
<td>PP = 2.0, 2.0, 2.5 tsf.</td>
<td>Slower augering (only keep caliche in bag.)</td>
</tr>
<tr>
<td>30.0 to 31.5</td>
<td>SS-6</td>
<td>1.5</td>
<td>3-46-50/3” (R)</td>
<td>30 ft to 31 ft: Sand (SP), brown, wet, very dense, fine-grained. 31 ft: Cemented Calcium (Caliche), pinkish-white, dry, very dense, breaks down to granular material.</td>
<td>31 ft to 35 ft: Driller said feels like &quot;decomposed&quot; bedrock.</td>
<td></td>
</tr>
<tr>
<td>35.0</td>
<td>SS-7</td>
<td>0.1</td>
<td>50/2” (R)</td>
<td>Weathered Basalt in drive shoe, bedrock at 35.0 ft.</td>
<td>T.D. = 35.2 ft.</td>
<td></td>
</tr>
</tbody>
</table>
## ROCK CORE LOG

### PROJECT: Sunnyside Valley Irrigation District
### LOCATION: MP 50.76
### ELEVATION: Approx. 830 ft
### DRILLING CONTRACTOR: HazTech
### DRILLING METHOD AND EQUIPMENT: CME 750 H.S.A.
### WATER LEVEL: Not Measured
### START: 4/11/06
### FINISH: 4/11/06
### LOGGER: G. Warren, P.G.

<table>
<thead>
<tr>
<th>DEPTH BELOW SURFACE (FT)</th>
<th>INTERVAL (ft)</th>
<th>CORE NUMBER</th>
<th>CORE RECOVERY (%)</th>
<th>RQD (%)</th>
<th>FRACTURES PER FOOT</th>
<th>DISCONTINUITIES</th>
<th>LITHOLOGY</th>
<th>COMMENTS</th>
</tr>
</thead>
</table>
|                          |              |             |                   |         |                  | DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS | ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS | SIZE AND DEPTH OF CASING, FLUID LOSS, COREING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC. | Begin 11:45
20 ft upstream on right bank access road. |
|                          | 5.0          | SS-1        | 0.8               | 8-10-7  | (17)             |                 | Silty GRAVEL with Sand (GM), brown, dry, medium dense, fine to coarse, subround gravel, 2-inch minus. | |
|                          | 6.5          |             |                   |         |                  |                 | Silty Clay in cuttings 6 to 8 ft, moist. | |
|                          | 10.0         | SS-2        | --                | 50/0.5" |                  |                 | Bedrock - microcrystalline quartz, yellow-tan, hard but fractured, very poor rock mass, glassy texture, stained fractures. | Hard at 9 ft, slow, smooth augering; appears to be weathered bedrock.
Core 10 ft to 15 ft. |
|                          | 10.05        | HQ-3        | 88%               | 0%      |                  |                 | 12.7 ft: Basalt, dark brown, vesicular, fine-grained, moderately weathered to gravelly pieces, H=R4, very poor rock mass, irregular fractures with silty coat and staining, open, rough, non-planar, moderately to highly fractured rock. | T.D. = 15.0 ft.
Backfill with hole plug. |
## ROCK CORE LOG

**PROJECT:** Sunnyside Valley Irrigation District  
**LOCATION:** MP 52.63  
**ELEVATION:** Approx. 828 ft  
**DRILLING CONTRACTOR:** HazTech  
**WATER LEVEL:** Not Measured  
**START:** 4/11/06  
**FINISH:** 4/11/06  
**LOGGER:** G. Warren, P.G.

### ELEVATION (FT)

<table>
<thead>
<tr>
<th>Depth Below Surface (ft)</th>
<th>Interval (ft)</th>
<th>Core Number</th>
<th>Core Recovery (%)</th>
<th>RQD (%)</th>
<th>Fractures per Foot</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.0</td>
<td>SS-1</td>
<td>0.4</td>
<td>43-50/3&quot; (R)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>10.0</td>
<td>SS-2</td>
<td>--</td>
<td>50/1&quot;</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3.6</td>
<td>HQ-3</td>
<td>72%</td>
<td>9%</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### DISCONTINUITIES

- Depth, type, orientation, roughness, planarity, infilling material and thickness, surface staining, and tightness

### LITHOLOGY

- Rock type, color, mineralogy, texture, weathering, hardness, and rock mass characteristics

### COMMENTS

- Embankment fill: Sand/Gravel.
- Begin 10:00  
  Upstream right bank access road.
- Weathered Basalt, breaks down to sandy granular material, either bedrock or large boulder.
- Refusal, try to auger down through weathered rock.
- Very slow smooth augering, appears to be solid rock.
- No recovery, refusal on bedrock.
- Core 5 ft to verify bedrock.
- Continues basalt, dark gray, fine-grained, tiny vesicles, moderately weathered, very poor rock mass.
- T.D. = 15.0 ft.  
  Backfill with hole plug.
<table>
<thead>
<tr>
<th>DEPTH BELOW</th>
<th>INTERVAL</th>
<th>SAMPLE NUMBER and TYPE</th>
<th>RECOVERY (FT)</th>
<th>STANDARD PENETRATION TEST RESULTS</th>
<th>SOIL DESCRIPTION</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td></td>
<td></td>
<td></td>
<td>6&quot;-6&quot;-6&quot; (N)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>5.0-6.5</td>
<td>SS-1</td>
<td>1.2</td>
<td>2-2-2 (4)</td>
<td>Sandy Silt (ML), brown, dry, firm, low plasticity, very fine sand.</td>
<td></td>
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<tr>
<td>10</td>
<td>10.0-11.5</td>
<td>SS-2</td>
<td>1.3</td>
<td>19-26-23 (49)</td>
<td>Silty Gravel with Sand (GM), brown, dry, dense, 1.5-inch minus angular to subangular pieces, mostly basalt, silty sand matrix.</td>
<td>Slower drilling in gravels.</td>
</tr>
<tr>
<td>15</td>
<td>15.0-16.5</td>
<td>SS-3</td>
<td>1.0</td>
<td>17-35-39 (74)</td>
<td>Sandy Gravel (GP), brown, dry, very dense, subangular to sub-round, silty sand matrix.</td>
<td>Gravels throughout boring have silty coatings.</td>
</tr>
<tr>
<td>20</td>
<td>20.0-21.5</td>
<td>SS-4</td>
<td>0.5</td>
<td>4-5-4 (9)</td>
<td>Silty Gravel with Sand (GM), brown, dry, loose, fine to coarse, 30% 1-inch minus gravels.</td>
<td>Easier augering, cuttings of fine round and subround gravels.</td>
</tr>
</tbody>
</table>

Gravels throughout boring have silty coatings.

Silty Gravel with Sand (GM), brown, dry, loose, fine to coarse, 30% 1-inch minus gravels.

Variable drilling, smooth then gravelly, etc.
### Soil Boring Log

**PROJECT:** Sunnyside Valley Irrigation District  
**LOCATION:** MP 55.21  
**ELEVATION:** Approximately 826 ft  
**DRILLING CONTRACTOR:** HazTech  
**DRILLING METHOD AND EQUIPMENT:** CME 750 H.S.A.  
**WATER LEVELS:** Not Measured  
**START:** 4/11/06  
**FINISH:** 4/11/06  
**LOGGER:** G. Warren, P.G.

<table>
<thead>
<tr>
<th>DEPTH BELOW</th>
<th>SAMPLE</th>
<th>STANDARD PENETRATION TEST RESULTS</th>
<th>SOIL DESCRIPTION</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
<td>INTERVAL</td>
<td>NUMBER and TYPE</td>
<td>6&quot;-6&quot;-6&quot; (N)</td>
<td>SOIL NAME, USCS GROUP SYMBOL, COLOR, MOISTURE CONTENT, RELATIVE DENSITY OR CONSISTENCY, SOIL STRUCTURE, MINERALOGY</td>
<td>DEPTH OF CASING, DRILLING RATE, DRILLING FLUID LOSS, TESTS AND INSTRUMENTATION</td>
</tr>
<tr>
<td>25.0 - 26.5</td>
<td>SS-5</td>
<td>1.0</td>
<td>14-16-13 (29)</td>
<td>Silty gravel (GM), brown, moist, dense, fine to coarse, silty sand matrix, 1.5-inch minus sub-round gravel.</td>
</tr>
<tr>
<td>30.0 - 31.5</td>
<td>SS-6</td>
<td>1.0</td>
<td>7-11-24 (35)</td>
<td>Gravelly sand (SW), brown, wet, dense, coarse-grained, trace silt, 30% sub-round gravels.</td>
</tr>
<tr>
<td>35.0 - 36.5</td>
<td>SS-7</td>
<td>1.3</td>
<td>14-24-29 (53)</td>
<td>Gravelly sand (SW), brown, moist, very dense, fine to coarse-grained, trace silt, 20% sub-rounded gravels.</td>
</tr>
<tr>
<td>40.0 - 41.5</td>
<td>SS-8</td>
<td>1.3</td>
<td>31-37-32 (69)</td>
<td>Gravelly sand (SW), brown, wet, very dense, fine to coarse with traces of silt and clay, 20% fine sub-round gravels.</td>
</tr>
</tbody>
</table>

**Sample Notes:**  
- Piece of gravel stuck in sampler shoe.  
- Sampler wet, groundwater at 40.0 ft.  
- T.D. = 41.5 ft.  
- Backfill with hole plug.
<table>
<thead>
<tr>
<th>DEPTH BELOW SURFACE (FT)</th>
<th>INTERVAL (ft)</th>
<th>CORE NUMBER</th>
<th>CORE RECOVERY (%)</th>
<th>RQD (%)</th>
<th>FRACTURES PER FOOT</th>
<th>DISCONTINUITIES</th>
<th>LITHOLOGY</th>
<th>COMMENTS</th>
</tr>
</thead>
<tbody>
<tr>
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</tbody>
</table>

**DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS**

**LITHOLOGY**
- ROCK TYPE, COLOR, MINERALOGY, TEXTURE, WEATHERING, HARDNESS, AND ROCK MASS CHARACTERISTICS

**GRAPHIC LOG**
- SIZE AND DEPTH OF CASING, FLUID LOSS, CORING RATE AND SMOOTHNESS, CAVING, ROD DROPS, TEST RESULTS, ETC.

**DISCONTINUITIES**
- DEPTH, TYPE, ORIENTATION, ROUGHNESS, PLANARITY, INFILLING MATERIAL AND THICKNESS, SURFACE STAINING, AND TIGHTNESS

**COMMENTS**
- 0 to 5 ft: Embankment fill - silty, sandy, gravel, angular basalt cobbles to small boulder, brown sand-silt matrix.
- Upstream right bank access road.
- Basalt, dark gray with orange iron oxidation staining, slightly vesicular, fine-grained, slightly weathered, H=R3 to R4. Very poor rock mass, moderately fractured.
- Basalt, dark gray, vesicular, fresh, H=R4 to R5, very good rock mass, tiny iron stains from weathered olivine.
- Basalt, dark gray, vesicular, fresh, H=R3 to R4, very poor rock mass, moderately fractured.
- Horizontal break.
- T.D. = 15.0 ft. Backfill with hole plug.

**PROJECT:** Sunnyside Valley Irrigation District
**LOCATION:** MP 57.55
**ELEVATION:** Approx. 819 ft
**DRILLING CONTRACTOR:** HazTech
**WATER LEVEL:** Not Measured
**START:** 4/10/06
**FINISH:** 4/10/06
**LOGGER:** G. Warren, P.G.
## Summary of Test Results

<table>
<thead>
<tr>
<th>Boring</th>
<th>Depth - Feet</th>
<th>Lab Number</th>
<th>Description and remarks (classification)</th>
<th>Maximum Dry Density, pcf</th>
<th>In situ Moisture</th>
<th>Passing No. 200</th>
<th>Atterberg Limits</th>
<th>Fines Class.</th>
</tr>
</thead>
<tbody>
<tr>
<td>B43.65 BK 5 - 10</td>
<td>B6L0613</td>
<td>Silt with Sand</td>
<td>111.0@15.5%</td>
<td>30.7%</td>
<td>82%</td>
<td>NV</td>
<td>NP</td>
<td>ML</td>
</tr>
<tr>
<td>B43.65 SS2 10 - 11.5</td>
<td>B6L0614</td>
<td>Silt w/Trace micaceous sand</td>
<td>33.5%</td>
<td>92%</td>
<td>NV</td>
<td>NP</td>
<td>ML</td>
<td></td>
</tr>
<tr>
<td>B50.76 SS1 5 - 6.5</td>
<td>B6L0615</td>
<td>Silty Gravel with Sand (GM)</td>
<td>8.5%</td>
<td>16%</td>
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<td></td>
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</tr>
<tr>
<td>B55-21 SS1 5 - 6.5</td>
<td>B6L0616</td>
<td>Silt with Sand</td>
<td>15.4%</td>
<td>72%</td>
<td>NV</td>
<td>NP</td>
<td>ML</td>
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</tr>
<tr>
<td>B55.21 SS2 10 - 11.5</td>
<td>B6L0617</td>
<td>Silty Gravel with Sand (GM)</td>
<td>8.8%</td>
<td>17%</td>
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<tr>
<td>B55.21 SS4 20 - 21.5</td>
<td>B6L0618</td>
<td>Silty Gravel with Sand (GM)</td>
<td>10.0%</td>
<td>18%</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Reviewed by: _____________________________

At: Kimball Ohsiek

Project: Sunnyside Valley Irrigation Dist. 335880.03.DH.03
Report to: CH2MHill
File Name: CH2H03 BM06149
Report Date: 4/27/06
MOISTURE-DENSITY RELATIONSHIP CURVE

ASTM D-698
Method A

Project: Sunnyside Valley Irrigation Dist.
Client: CH2MHill
File Name: CH2H03 BM06149
Date Tested: 4/25/06 By: CAK
Sample Number: B6L0613
Sample Location: B43.65 @ 5’ - 10’
Sample Description: Silt with Sand
Atterberg Limits: LL = No Value  PI = Non Plastic
Soil Tempered: Yes
Rammer Type: Manual

Maximum Dry Density, pcf: 111.0
Optimum Moisture Content, %: 15.5

Reviewed By: Ted Clague

STRATA
GEOTECHNICAL ENGINEERING & MATERIALS TESTING
Integrity from the Ground Up
Project: Sunnyside Valley Irrigation District 335880-03.CH.03
Client: CH2M Hill
File: CH2H03 BM06149
Sample No.: B6L0615
Sample Location: B50.76 SS-1 @ 5' - 6.5'
Description: Silty Gravel with Sand (GM)
Date tested: 4/25/06  By: tc

GRADATION ANALYSIS
ASTM C 136/C!17

Reviewed by: [Signature]

[Diagram showing gradation analysis with data points and screen sizes]

[Signet: STRATA]

[Text: Integrity from the ground up]
GRADATION ANALYSIS
ASTM C 136/C117

Project: Sunnyside Valley Irrigation District 335880-03.CH.03
Client: CH2M Hill
File: CH2H03 BM06149
Sample No.: B6L0617
Sample Location: B55.21 SS-2 @ 10' - 11.5'
Description: Silty Gravel with Sand (GM)
Date tested: 4/25/06 By: tc

Reviewed by: __________________________

Strata Geotechnical Engineering & Materials Testing
Integrity from the Ground Up
GRADATION ANALYSIS

ASTM C 136/C117

Project: Sunnyside Valley Irrigation District 335880-03.CH.03
Client: CH2M Hill
File: CH2H03 BM06149
Sample No.: B6L0618
Sample Location: B55.21 SS-4 @ 20' - 21.5'
Description: Silty Gravel with Sand (GM)
Date tested: 4/25/06 By: tc

Reviewed by: ____________________

Inches Screen Sizes
Cobbles Gravel Sand
Coarse Fine Coarse Medium Fine

SOIL GRAIN DIAMETER, millimeters

PERCENT PASSING

0 10 20 30 40 50 60 70 80 90 100

0 0.1

0 1 2 3 4

1/100 1/50 1/40 1/20 1/16 1/10 1/8 1/4 1/2 3/4 1 2 3 4 6
SOIL RESISTIVITY & pH for Corrosion testing
ASTM G57, G51

Report to: CH2M-Hill  File No.: CH2H03-BM06149
Lab No.: B6L06149  Project No.: 335880.03.CH.03

Sample ID: B43.65 Bulk 5 - 10 feet  Soil Type: Silt with Sand

<table>
<thead>
<tr>
<th>ml of water</th>
<th>Voltage, volts</th>
<th>Current, ma</th>
<th>Ohm·cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
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<tr>
<td>25</td>
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<tr>
<td>50</td>
<td>4.89</td>
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<tr>
<td>300</td>
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</tr>
</tbody>
</table>

Minimum Resistivity ohm·cm = 3300

pH of Soil = 8.54  @ 70.2 Degrees F.

Reviewed by: [Signature]

Sunnyside Valley Irrigation Dist.  
ASTM G57, G51

CH2M-Hill  
B6L06149

Silt with Sand