City of St. Joseph, Missouri

St. Joseph Disinfection Facility and Effluent Pump Station

St. Joseph, MO

Internal Geotechnical Design Memorandum

B&V Project 166529
January 2011

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1.0 Site Conditions

1.1 Site Location

The proposed location of the project site is near the eastern bank of the Missouri River, south of Highway 36 in St. Joseph, Missouri. The site is generally gently sloping ground with grass growing 3-4 feet high. The site is bordered by grain silos to the west, and the Water Protection Facility (WPF) to the east side. The site has very little fill, and is mostly comprised of alluvium deposited by the river.

1.2 Proposed Facility

The project involves the design and construction of a new ultra-violet (UV) disinfection facility and effluent pumping station at the WPF, along with a new outfall pipeline to the Missouri River. The UV disinfection facility will have capacity to disinfect 54 million gallons per day (mgd) of treated effluent before discharge into the river. The new effluent pump station and associated pipeline will have the capacity to convey 108 mgd of flow to the existing effluent channel. The project also involves the design and construction of an outfall structure that will discharge to the existing effluent channel.

2.0 Subsurface Conditions

The field subsurface investigation is described in further detail in the Geotechnical Data Report for this project. Three main stratigraphic units were identified. The first consists of 42-45 feet of firm high plasticity clay. The undrained compressive strengths in this layer, as measured with a pocket penetrometer, range from 0.5 to 2.0 tsf. The undrained shear strength, as measured with a hand held torvane ranged from 0.4 to 0.8 tsf. The blow counts in this layer range from 0 to 6, whereas the unconfined compression test results showed strength values ranging from 1173 psf to 2532 psf for five (5) Shelby tube samples that were tested. The clay unit was consistent between the two borings.

The second stratigraphic unit is medium dense poorly graded sand with varying grain sizes, ranging from very fine to gravelly. This layer may be divided into three broad sub-units. The first sub-unit can be classified as a very fine to fine silty sand layer roughly 10-15 feet thick immediately underlying the clay layer. Based on the blow counts, it can be classified as generally medium dense. Beneath this layer lies a sub-layer of gravelly sands, 5 to 10 feet in thickness. This layer is classified as generally medium dense. Underneath lies a layer of mostly medium grained sand, roughly 15 feet in thickness, classified as medium dense.

The bedrock, which is comprised of limestone, was found at a depth of roughly 78 feet (Elev. 734).

2.1 Seismicity

Table 2-1 lists the values for various spectral responses for the site as per the USGS Hazard Mapping Project. Figures 2-1, 2-2, and 2-3 show the latest US National Seismic Hazard Maps from the USGS website.

Table 2-1 Design Response Spectrum for the site, based on latest USGS Seismic Hazard Maps	
Response Spectrum	**Acceleration as a fraction of "g"**
Peak Ground Acceleration	0.04 g
0.2 s Spectral Acceleration	0.06 g
1.0 s Spectral Acceleration	0.04 g

The seismic site class for this site has been determined as "E" per IBC 2006.

3.0 Geotechnical Design Recommendations

The boring logs, laboratory test results and in-situ resistivity surveys included in the Geotechnical Data Report were used to prepare the geotechnical design recommendations.

3.1 Design Groundwater Table

For structures located within the UV/Effluent Pump Station area, a design groundwater elevation 815.6 should be used. This elevation is based on the Missouri River 100 year flood elevation. The normal groundwater level at the WPF should be taken as elevation 800. The 100 year flood elevation should be used to design the UV/Effluent PS for uplift resistance.

3.2 Deep Foundations for the UV/Effluent Pump Station

Based on the subsurface profile in the pump station area (borings B-1 and B-2), depth to limestone rock, potential construction practices (i.e. open cut construction with temporary excavation support systems), and existing foundations at this facility, we recommend that deep foundations be used to support the UV and Effluent Pump Station Facility. Additional pile evaluations such as lateral load analyses may be provided once the pile loads become available.

3.2.1 Piling

Axial Capacity

Three different kinds of pile foundations were evaluated: steel pipe piles, steel H-piles, and augercast piles. The following table summarizes the allowable design loads for each pile type for both compression and uplift.

Table 3-1
Allowable Pile Design Loads

PILE TYPE	COMPRESSION		UPLIFT	
	Q_a (Kips)	Q_a (Tons)	Q_a (Kips)	Q_a (Tons)
Closed end steel pipe pile				
12-inch	128	64	39	19
16-inch	171	85	52	26
24-inch	257	129	78	39
Steel H-Pile				
12 x 53	160	80	55	28
14 x 73	206	103	65	33
Concrete augercast pile				
16-inch	94	47	73	37
24-inch	155	78	110	55

All compression pile capacities are based on piles acting on skin friction and end bearing (mostly end bearing for driven piles), using a factor of safety of 2.25. Uplift capacities are based on skin friction only, using a factor of safety of 3. Uplift pile resistance values are based on skin friction resistance alone and do not include the weight of the pile.

Pile tips for driven piles are based on top of rock Elev. 734. Pile tips for augercast piles are based Elev. 732. The augercast pile tip elevation is raised, since the installation would require at least one pile diameter penetration into rock bearing stratum to utilize the end bearing capacity of the rock. The penetration rate of auger during rock drilling would be slowed, therefore increasing the chance of mining the overburden soils by side loading of the auger above the rock.

If pipe piles are used, we recommend predrilling the piles to be installed in the upper slab level (top slab elev. 801.5) down to Elev. 787. This is to reduce pile driving vibrations and to minimize soil heave. Vibration monitoring will be required during pile driving to ensure that existing infrastructure (i.e. 48-inch and 96-inch RCP pipes) do not experience excessive vibration.

Although several pile alternatives are presented, steel H-piles provide the most cost benefit as far as pile capacity available per pile size. The H-pile configuration also provides flexibility in pile layouts if field conditions would require placement of additional piles within the foundation footprint. If driven piles are selected, indicator pile(s) should be monitored in the field with a Pile Driving Analyzer (PDA). The PDA will help confirm pile capacities and ensure that driving stresses of 0.9 F_y are not exceeded during pile driving and rock refusal, where F_y is the yield strength of the steel. The steel piles should be driven to rock refusal (approximately tip El 734) to obtain these capacities. Static load testing in compression and uplift is recommended.

Compressive axial group effects for end bearing piles on hard rock are negligible. However, a minimum center-to-center distance of 3 pile diameters is recommended to minimize pile installation problems.

Lateral Capacity

The allowable lateral capacity is provided by the lateral resisting forces in the soil and the structural stiffness of the pile. The allowable lateral capacity is determined by either the limiting lateral deflection of the top of the pile or by the structural capacity of the pile. Two pile head conditions can be considered in design: the top of the pile would be allowed to move laterally and rotate under the applied shear load (free head condition) or the top of the pile might be allowed to move laterally, but would not be allowed to rotate under the applied shear load (fixed head condition).

Lateral capacities may be determined using the computer program L-Pile Plus v.5.0 by Ensoft. The L-Pile parameters to be used for analysis are presented in Tables 2 and 3. The boundary conditions (i.e. fixidity, shear, deflection, and bending moment) on top of the pile will be required for L-Pile analysis. The weak axis of H-pile sections should be considered.

Table 3-2
L-Pile Parameters for Undisturbed Subgrade (Earth Retaining System)

STRATUM	EFFECTIVE UNIT WEIGHT (PCI)	ANGLE OF INTERNAL FRICTION (DEGREES)	UNDRAINED COHESION (PSI)	HORIZONTAL SUBGRADE MODULUS (PCI)	UNCONFINED COMPRESSIVE STRENGTH (PSI)
Fat CLAY* Elev. 787-770	.0214	---	3.5	30	6.94
Fine SAND Elev. 770-734	.0289	30	---	60	---
Limestone Elev. 734	.0594	---	---	---	7500**

* For Fat CLAY, use soil strain parameter ε_{50} of 0.02. Use Elev. 787 for upper and lower piles.
** Assumed. No unconfined compression test data available.

3.3 Shallow Foundations for Miscellaneous Structures

Shallow structures near finished grade such as transformer pads bearing on select fill may be designed with spread footings using a net allowable bearing pressure of 1,500 psf. Select fill should be used to raise the site grades below and within 5 feet of structural footprints.

When using the net allowable bearing pressure, the weight of the footing and soil backfill over the footing including the floor slab weight may be excluded; therefore, only the loads applied at or above the top of the finished slab need to be considered for sizing the footings. Based on the net allowable design values given herein, total and differential settlements are estimated at one inch or less.

3.3.1 Uplift

The Facility Drain Sump (Wet Well), Facility Drain Valve Vault, Special Manhole No. 3 and Special Manhole Number 4 will be subject to uplift pressures when empty. The effective unit weight of the soil above the top of the footing extension may be used as additional dead weight to resist uplift forces. An effective unit weight of 57.6 pcf may be used for this purpose.

3.4 Lateral Earth Pressures

Recommended pressures against yielding and nonyielding walls for structure backfill are presented in Table 3-3, which is based on the information obtained from borings B-1 and B-2. The table below provides undrained values, given the floodplain location and high groundwater table.

Below grade walls supported at the top are considered non-yielding and should be designed for the at rest equivalent fluid unit weights recommended in Table 3-3. Active equivalent fluid unit weights should be used for yielding walls.

Table 3-3 STATIC Lateral Earth Pressure			
Equivalent Fluid Pressure*		Fat Clay (CH)**	SAND***
At Rest (pcf)	Undrained	104	91
Active (pcf)	Undrained	94	81

* The force distribution is triangular with the resultant acting at one third height from the base of the wall.
** (CH) compacted to 95% Standard Proctor with a moisture content ranging between 0 to+4 % of optimum.
*** SAND is imported material.

These values do not include load contributions from additional surcharge or heavy compaction equipment against the wall (within one roller width against the wall). A 400 psf compactive lateral pressure envelope may be used at the top of the wall.

The excavation methods to be used for construction of the UV and Effluent Pump Station Facility will require a combination of open cut and braced excavation techniques. Any excavation support systems to be used within 500 feet of the levee centerline must be temporary and will be removed. The USACE also does not permit the use of granular backfill material within the critical area. Most if the UV Disinfection and Effluent Pump Station Facilities and associated structures will fall outside of the 500 ft critical zone. In order to maintain construction excavations outside of the 500 ft critical flood zone, the deeper portion of the UV/PS structure will require braced excavation techniques. Assuming the removal of the bracing elements after construction, the retaining walls will experience earth pressures closer to a clay backfill. We recommend that the deeper retaining walls be designed for at rest, undrained clay wall pressures.

Whenever imported structure backfill material is used (Sand), the upper 3 feet of structure backfill should consist of compacted clay (CH), compacted to 95% Standard Proctor Density with a moisture content ranging between 0 to +4 % of optimum.

3.5 Site Preparation and Earthwork

Earthwork associated with the new UV and Effluent Pump Station Facility including junction boxes, manholes, valve vault manholes, transformer pads, parking areas, etc. is covered in this section.

In general, the site should be graded to provide stormwater drainage away from foundations and equipment. The existing grade southeast of the grain bin and railroad track areas will be raised to El. 817. The borrow source to be used for raising site fills is unknown at this time. Raising the site fill to El. 817 will cause the underlying soft compressible clay to undergo a slow consolidation settlement due to the fill loading. Using consolidation test results, it is estimated that consolidation settlement of one inch will occur. The settlement is expected to occur over a long period of time estimated over 50 years.

Because of the in situ moisture content, significant amount of fines and high liquid limits and plasticity indices of the onsite soils, the use of onsite material is only acceptable for general fill and structure backfill for manhole, valve vault and wet well structures, provided the soil is reconditioned (modified) to meet the compaction requirements. Select fill shall be used for roadway areas and for structures bearing near finished grade. Structures near grade are defined as structures bearing at or above existing grade. As previously stated, select fill should be used to raise the site grades below and within 5 feet of structural footprints. Select fill should consist of imported materials classified as GW, SW, SM, or SC per the Unified Soil Classification System (USGS), with a liquid limit of less than 35 and a plasticity index between 4 and 15.

General fill is material placed as required across the area to achieve the final design grade elevation. Select fill is material placed beneath shallow structure foundations and slabs. Structure backfill is material placed against the walls of below grade structures.

After preparation of the fill and embankment areas, the subgrade shall be leveled and rolled so surface materials will be as compact and well bonded with the first layer of the fill or embankment as indicated for subsequent layers. Compaction characteristics for the in situ and borrow material should be determined based on testing in accordance with ASTM D698 Method A, Standard. For select fill and structure backfill, field compaction should be greater than 95 percent maximum dry density and minus 2 to plus 2 percent of optimum moisture content in accordance with ASTM D698. For general fill, field compaction should be greater than 95 percent maximum dry density and minus 1 to plus 4 percent of optimum moisture content in accordance with ASTM D698.

Excavated clay material from the UV and Effluent Pump Station Facility is only suitable for general fill as described herein. This material will likely require drying, disking and reconditioning. The Contractor will likely require an open area on the site to spread the excavated material to allow proper drying and disking. These recommendations should be addressed in the contract specifications.

Fill for roadways, general fills and embankments, select fills and structure backfill should be placed in uniform, horizontal loose lifts limited to 8 inches or less in uncompacted lift thickness.

4.0 Geotechnical Construction Considerations

We recommend an inspection of the existing 48-inch and 96-inch diameter RCP pipes prior to any pile construction activities. Vibration monitoring near the 48-inch and 96-inch RCP pipes is recommended if displacement (pipe) piles are to be installed. Preconstruction surveys of existing pipes and buildings (i.e. grain silos) is also recommended.

The planned excavation depths range from El. 787 to 800. During our geotechnical investigation, the groundwater elevation was around elevation 800. The 100-year flood elevation corresponding to this site is El. 815.6. Based on the amount of clay material that will be removed for the lower excavation, the soft clay below the structure subgrade (down to El. +/- 770), and the sand strata below the clay (El. +/-770 to 734), the Contractor can anticipate special dewatering techniques prior to the pump station excavation. Dewatering systems such as deep wells or well points should be considered for excavation stability. This will require that the groundwater level remains at least three (3) feet below the bottom of the lowest excavation subgrade at all times during construction. Dewatering recommendations should be addressed in the contract specifications.

All excavations should be designed based on OSHA Standards of safety and care. Temporary shoring systems and/or slopes may be required. Permanent excavation support systems may also be required to maintain excavations outside the critical area. The U.S. Army Corps of Engineers defines the critical area as 500 feet landward and 300 feet riverward of a flood control project (i.e. levee) centerline. The design and implementation of temporary slopes and excavation support systems shall be the responsibility of the Contractor.

5.0 References

1. Design and Construction of Driven Pile Foundations, Workshop Manual – Volume 1, Federal Highway Administration, Report No. FHWA-HI-97-013, November 1998.

2. US Geological Survey website, Java Ground Motion Parameter Calculator, (http://www.earthquake.usgs.gov/hazards/designmaps/javacalc.php).

3. Geotechnical Data Report, St. Joseph Disinfection Facility and Effluent Pump Station, Black & Veatch Corporation, October 2010.

4. City of St. Joseph, Missouri Water Protection Facility Drawings, Black & Veatch Corporation, sixty percent (60%) level drawings.

5. Geotechnical Engineering Procedures for Foundation Design of Buildings and Structures, Department of Defense, Unified Facilities Criteria (UFC), UFC 3-220-01N, August 15, 2005.

6. Foundations and Retaining Structures, Design Manual 7.02, Naval Facilities Engineering Command (NAVFAC), September 1, 1986.