

4.2.5.2.2 SPT Borings

Material encountered in the subsurface at Borings B-12, B-13, and B-14 generally consisted of alluvium including poorly graded, fine- to coarse-grained sand (SP) and to a lesser extent silty sand (SP-SM) and clayey sands (SC). Silt and lean clay zones were encountered in Borings B-13 and B-14 in the upper roughly 10 to 23 ft; these soils were logged as fill and documented as such in various historical geotechnical reports and as-built drawings provided by OPPD.

No voids or very soft/very loose conditions that might be indicative of piping or related material loss or movement were identified through drilling and continuous sampling of the test borings. N-values (uncorrected) indicated that the encountered alluvium ranges from loose to medium dense and that soil conditions were similar between anomalous zones (low velocity features reported by Geotechnology) and non-anomalous zones. The reported low velocity zones are attributed to the inherent variability in the relative density of the granular alluvium that underlies the site. SPT results were compared to similar data from numerous other geotechnical investigations that have been conducted on the FCS site in previous years and for this assessment at other locations across the site. This comparison did not identify apparent differences from soils encountered at other on-site test boring locations nor did it identify changes in soil relative density following the 2011 flood.

4.2.5.2.3 Inclinometer Monitoring

Data from inclinometers through January 24, 2012, compared to the original baseline measurements have not exceeded the accuracy range of the inclinometers. Therefore, deformation at the monitored locations since the installation of the inclinometers has not occurred.

4.2.5.2.4 Survey Monitoring

Survey data points (in the Paved Access Area) through December 29, 2011, compared to the original baseline surveys have not exceeded the accuracy range of the surveying equipment. Therefore, deformation at the monitored locations since the survey baseline was shot has not occurred.

4.2.5.3 Discussion and Conclusions

Forensic investigation, as described above, was performed where observed pavement distress was most prominent, at locations coincident with shallow underground structures and utilities, and where recent seismic surveys identified low velocity features (that is, locations where potential for degradation related to the Triggering Mechanisms and CPFMs associated with KDI #2 was identified).

Excavation and subgrade testing identified no evidence of piping erosion, voids, or subsidence of site fills. Field SCP testing of the exposed subgrade indicated that stiff to very stiff soils were generally encountered in the upper 3 ft below the ground surface or pavement. Based on the observations made and tests results obtained, the fill soils in the locations exposed and tested are compact, cohesive soils that are not susceptible to piping erosion. SPT borings did not identify voids or very soft/very loose conditions that might indicate piping or related material loss nor did they identify changes in soil relative density following the 2011 flood.

Inclinometer and survey monitoring indicates that movement of on-site subsurface soils or structures has not occurred.

Possible Triggering Mechanisms and related CPFMs identified for KDI #2 and the Paved Access Area include:

- Subsurface Erosion and Piping (due to pumping), CPFMs 3a, 3b, and 3c.
- Subsurface Erosion and Piping (due to rapid river drawdown), CPFMs 3d, 3e, and 3f.

Based on the observations and test results, the individual distress indicators that comprise KDI #2 are not attributed to the possible Triggering Mechanisms identified for KDI #2: Subsurface Erosion/Piping (due to pumping) and Subsurface Erosion/Piping (due to river drawdown). Rather, these distress indicators appear related to normal wear and tear typical for pavements that are subjected to heavy loads and summertime expansion-contraction and wintertime freeze-thaw and exposure to de-icing agents. Observed pavement distress following flooding that may have been exacerbated by localized subgrade saturation (especially along expansion joints as was observed during pavement removals and SCP investigation) and softening followed by heavy applied loads as equipment moved into the area for clean-up and site restoration. The singular broken pavement and underlying surficial void observed just north of the Security Building was coincident with the discharges from several large pumps that ran continuously during flooding, and this feature likely was caused by the hydraulic forces of that discharge.

Forensic investigation for KDI #2 also indicates that the Triggering Mechanism of Subsurface Erosion and Piping (due to rapid river drawdown) was not initiated by the 2011 flood and that the CPFMs related to this Triggering Mechanism, including CPFMs 3d, 3e, and 3f, have been ruled out.

However, the Triggering Mechanism of Subsurface Erosion and Piping (due to pumping) and the CPFMs related to this Triggering Mechanism, including CPFMs 3a, 3b, and 3c, cannot be ruled out for all structures associated with the Paved Access Area. Even though this Triggering Mechanism does not appear to have caused the distresses observed in the Paved Access Area, its root cause (damaged Turbine Building sub-floor drain pipes and sump pumping) as identified by investigations in the Turbine Building basement continues. A number of other Priority 1 and Priority 2 structures have been assigned CPFMs that are related to this remaining credible Triggering Mechanism and its related CPFMs. These other structures differ from KDI #2 and the Paved Access Area in that no strong evidence of distress has been identified or documented through assessment observations or ongoing survey monitoring.

Priority 1 Structures in this category include:

- Security BBREs
- Turbine Building South Switchyard
- Condensate Storage Tank
- Underground Cable Trench (Trenwa)
- Circulating Water System
- Demineralized Water System (line)
- Raw Water Piping
- Fire Protection System Piping

- Waste Disposal Piping
- Fuel Oil Storage Tanks and Piping
- Main Underground Cable Bank, Auxiliary Building to Intake Structure
- Blair Water System
- River Bank

Priority 2 Structures in this category include:

- Service Building
- Sanitary Sewer System

The potential for impact on the above Priority 1 and Priority 2 structures from the Triggering Mechanism of Subsurface Erosion and Piping (due to pumping) exists, and the CPFMs related to this Triggering Mechanism remain credible until the recommendations related to KDI #1, as presented in Sections 4.1 and 8.3, are implemented and completed.

However, it can be concluded that the Triggering Mechanism of Subsurface Erosion/Piping (due to pumping) most likely did not extend outside the perimeter of the seismic investigation lines taken around the Power Block (see Figure 4-6). This conclusion supports the ruling out of the Subsurface Erosion/Piping (due to pumping) CPFMs associated with this Triggering Mechanism for the following structures:

- Security Building
- Intake Structure
- River Bank

4.2.5.4 Recommendations

The results of this KDI #2 forensic investigation have ruled out potential Triggering Mechanisms and associated CPFMs that could have been the cause of the observed distress. However, it could not be used to entirely rule out CPFMs associated with KDI #1, which is associated with the uncontrolled drainage of the groundwater into the broken Turbine Building basement drainage system piping. These CPFMs will be ruled out only when the physical modifications presented for KDI #1 are implemented. Discussion of KDI #1 began in Section 4.1 and continues in Section 8.0.

4.3 KDI #3 – Column Settlement in Maintenance Shop

KDI #3 is the settlement of Column MG-15 in the Maintenance Shop. The column is on the first floor of the Maintenance Shop outside the men's restroom adjacent to the northern side of the Turbine Building. OPPD staff indicated that the column had begun settling prior to the 2011 flood and that the settlement increased during the flood. As of October 7, 2011, the column had settled 2.2 in. In addition to the settled column, cracks in the wall were observed near the beam adjacent to the men's restroom, and the doors on the restroom no longer operate properly.

4.3.1 Physical Observations

A number of physical observations made during the facility assessments have been grouped under this KDI #3:

- Significant settlement of building column (2.2 in.)
- Significant settlement of floor slab
- Cracking of masonry partition walls in the southwest corner of this building immediately adjacent to the Turbine Building

4.3.2 Triggering Mechanisms

Two possible triggering mechanisms that might be the root cause of this KDI are discussed as follows.

4.3.2.1 Subsurface Erosion and Piping (Due to Pumping)

Multiple connected seepage paths have the potential to exist in the soil backfill at the FCS site. This potential network of seepage paths could be connected to several pumping sources: the sump in the Turbine Building, Manhole MH-5, and a series of surface pumps inside the perimeter of the Aqua Dam. The dewatering pumps inside the Aqua Dam were operated for an extended period, maintaining a head differential on any potential seepage path networks. Gradient might have been sufficient to begin erosion of surrounding soil.

Unfiltered seepage into the Turbine Building sump provided the potential for subsurface erosion/piping and would continue until that seepage is completely stopped. The potential subsurface erosion/piping caused by the Turbine Building sump pumping could extend underneath the Maintenance Shop.

4.3.2.2 Soil Collapse (First Time Wetting)

The most recent flood elevation prior to the 2011 flood was 1003.3 ft, which occurred in 1993. The maximum flood elevation in 2011 was approximately 1006.9 ft. The foundation of the Maintenance Shop has footings at el. 1000.5 ft and subgrade below the flooring slab of approximately el. 1006 ft. Therefore, it is possible that up to 3 ft of soil were saturated for the first time as a result of the 2011 flood. This alone could not cause settlement of the foundation footings due to first time wetting because the footing elevation of 1000.5 ft had likely experienced first time wetting in 1993.

4.3.3 Structures and CPFMs Associated with Triggering Mechanisms

The Triggering Mechanisms outlined could apply to the following structures and CPFMs:

- Security Building
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Security BBREs
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).

- Turbine Building South Switchyard
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
 - CPFM 7a through 7c – Soil Collapse (first time wetting). Cracked slab, differential settlement of shallow foundation, loss of structural support; displaced structure/broken connections; and general site settlement.
- Condensate Storage Tank
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Underground Cable Trench
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Demineralized Water System
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Raw Water Piping
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Fire Protection System Piping
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Waste Disposal Piping
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Fuel Oil Storage Tanks and Piping
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Main Underground Cable Bank, Auxiliary Building to Intake Structure
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Blair Water System
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
 - CPFM 3c – Subsurface Erosion/Piping. Undermined buried utilities (due to pumping).
- Camera Towers and High Mast Lighting
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).
- Service Building
 - CPFM 3a – Subsurface Erosion/Piping. Undermining and settlement of shallow foundation/slab (due to pumping).

4.3.4 Assessment Methods and Procedures

Initial assessments were made by OPPD staff and are described in OPPD reports.

An additional investigation was performed on August 2, 2011, to further characterize the subsurface at the areas where conditions indicative of potential flood-related impacts or damage were observed. A subsurface survey using GPR was performed by Ground Penetrating Radar Systems, Inc. as a subconsultant to OPPD. The report is titled "Ground Penetrating Radar (GPR) survey to locate sub surface voids at the Ft. Calhoun Nuclear Facility in Blair, NE." The GPR survey identified potential voids in the soil beneath the column and along the length of the corridor from 8 to 12 in. below the surface. The voids are referred to as "small in thickness in most areas" and slightly thicker nearest the settled column.

4.3.5 Previous Investigations and Baseline Information

In 1977, prior to construction of the Maintenance Shop addition, Geotechnical Services, Inc., performed an investigation documented in its report titled "Report of Subsoil Investigation for Proposed Maintenance Shop Addition to the OPPD Fort Calhoun Station" (July 14, 1977). Four borings were completed to assess soil conditions. The borings recorded 7 to 9.5 ft of fill material consisting of clayey silt on the south side of the proposed structure area and fine sand on the north side of the proposed structure area. SPT N-values of the fill material range from 9 to 20. Elevations of the borings were not recorded.

Maintenance Shop drawings indicate that the floor elevation is 1007.5 ft and the elevation of the bottom of the foundation footings is 1000.5 ft. Therefore, based on the depth of fill material below existing grade established in the previously mentioned report, the foundation footings are placed on fill material.

4.3.6 KDI #3 Forensic Investigation

Forensic investigation consisting of concrete floor slab drilling and field and laboratory subgrade testing was completed in the Maintenance Shop to evaluate subsurface conditions near KDI #3. This KDI consists of differential settlement of Column MG-15, presumed differential settlement of the nearby floor slab, and cracked nearby masonry partition walls. These building distresses were observed at the southwest corner of the Maintenance Shop immediately adjacent to the north side of the Turbine Building during facility assessments. Additional details on the column and floor settlement, including the time period over which these settlements occurred, are presented in Section 6.4.

Possible Triggering Mechanisms identified for KDI #3 include:

- Subsurface Erosion and Piping (due to pumping)
- Soil Collapse (due to first time wetting)

These Triggering Mechanisms and related structures/CPFMs are discussed in detail in Sections 4.3.2 and 4.3.3.

The purpose of this investigation was to determine the presence and extents of potential voids and soft zones beneath the floor slab and to evaluate which of the Triggering Mechanisms identified for KDI #3 were responsible for the observed building distresses.

As discussed in Section 6.4, the Maintenance Shop is a single-story building. Its roof is supported by a pre-engineered steel frame that transfers the load to each structure's perimeter spread footings, and an interior mezzanine is supported by interior columns that transfer load to the interior spread footings. The top of the interior footings is approximately el. 1004 ft, and the top of the exterior footings is approximately el. 1002 ft. The main floor elevation is approximately el. 1007.5 ft. The surrounding grades are at approximately el. 1004 ft. The building is founded on about 20 ft of fill along the Turbine Building and 10 to 12 ft of fill in other areas, including along the Technical Support Center, Service Building, and CARP Building. Design plans reviewed for this forensic investigation indicate that the floor slabs and underlying construction consist of a 6-in.-thick concrete slab with a 6 mil poly vapor barrier placed between the floor slab and a 6-in.-thick granular base layer.

4.3.6.1 Scope of Work for 2011 Testing

Forensic investigation of the Maintenance Shop was conducted from November 9 through December 2, 2011. A total of 22 1-in.-diameter floor slab locations were drilled and the underlying subgrade evaluated during the investigation. This included 16 holes at locations nearby and surrounding Column MG-15 (drill-hole locations 1-1 through 1-16), 4 holes investigated to the east of Column MG-15 (drill-hole locations EW-1 through EW-4), and 2 holes investigated to the north of Column MG-15 (drill-hole locations NS-1 and NS-2). A total of two 4-in.-diameter holes were drilled and investigated with Shelby tube sampling (ST-1 and ST-2). These investigation locations are illustrated in Figure 4-13.

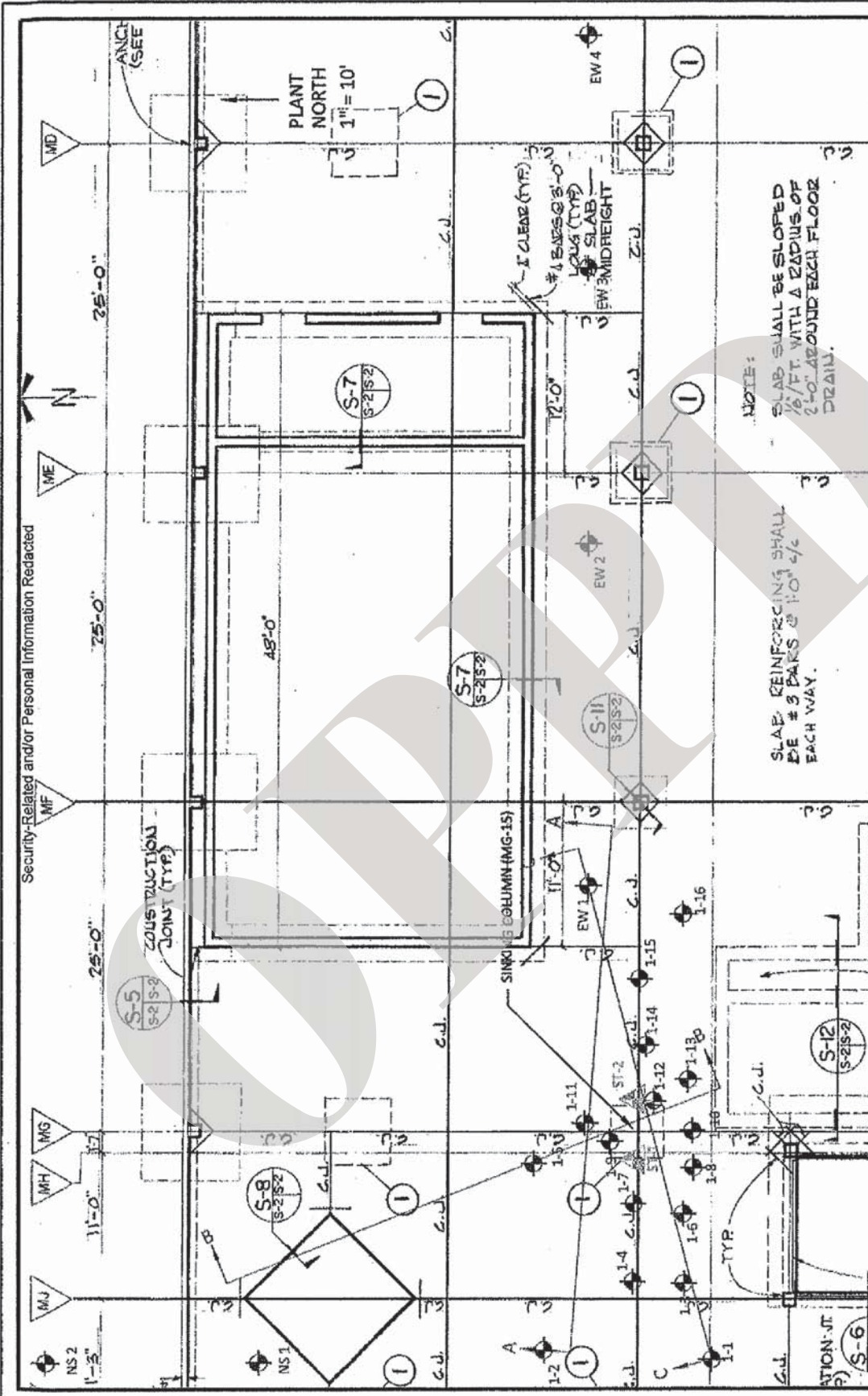
Drilling of the concrete for the 22 1-in.-diameter holes was accomplished by Lueder Construction Company under contract to OPPD using a hammer drill. All of the 1-in.-diameter drill-holes were protected immediately following drilling and before and after subgrade evaluations using temporary plastic caps that were flush with the surrounding floor surface. Concrete drilling for the two Shelby tube test borings was accomplished by Omaha Concrete Sawing under contract to Lueder Construction Company using a hand drill fitted with a 4-in.-diameter core bit. The 4-in.-diameter drill-holes were protected after subgrade evaluations using temporary expanding plugs. After testing was completed, the holes were filled with non-shrink grout by Lueder Construction Company.

Subgrade evaluation included observing conditions below the floor slab, in-situ field testing at each drilled location, and laboratory testing on Shelby tube samples of the subgrade material at the two test boring locations. Observations were made by HDR and Thiele Geotech. Subgrade field and laboratory testing was performed solely by Thiele Geotech as directed by HDR.




Observation of the subgrade below the floor slab included direct visual observation through the open holes with the aid of a flashlight; close-up visual observations using a lighted, water-proof borescope lowered through the open drill-holes; and measurement of the floor slab thickness and depth to subgrade using a hooked probe made from #9 tie wire.

In-situ field tests on the subgrade consisted of SCP tests at each drilled location and a DCP test performed at one drill-hole location (1-12). Laboratory tests included moisture content and unit weight (wet and dry).

Security-Related and/or Personal Information Redacted



Note: Background is OPPD Drawing #16892 Drawings for Maint. Shop Expansion not legible at this scale.

 Drilled Test Holes
 Drilled Test Holes with Shelby Tube Sampling
 Sections - See Figures 4-14, 4-15, 4-16



Maintenance Shop Drilling Locations
Cross Section Location Plan
Sections A-A, B-B, C-C
Fort Calhoun Station
 Plant and Facility Geotechnical and Structural Assessment

DATE April 2012
 FIGURE 4-13

HDR

Thiele Geotech used a Humboldt Model H-4210A Portable Static Cone Penetrometer to perform the SCP tests. This penetrometer uses a 1.5 square cm, 60 degree cone at the tip and was hand pushed. A pressure gauge provides a direct reading of the cone resistance, or pressure, as the penetrometer is pushed. The cone penetrometer was advanced into the soil by hand in continuous 6-in. vertical intervals until refusal or the maximum vertical reach of the device (3.0 ft) was reached, whichever occurred first. Peak resistance readings were observed and recorded for each 6-in. interval. Measurements of the depth into the subgrade at each location tested were made using gradation markings on the cone penetrometer shaft.

The H-4210A Product Manual for the SCP test device provides a coefficient of 0.25 for correlating the direct read value of cone index (Q_c) in kilograms per square cm to N-value. The H-4210A Product Manual states that the correlation was determined through extensive field use, but is not absolute, and should be verified for local soil types (Humboldt Mfg. Co., 2009). Because of the hydro-excavation required in the upper 10 ft of all test borings completed during the geotechnical investigation, direct correlation with on-site soils was not possible. As such, the N-values provided in Table 4-2, below, are not based on correlations with site soils, and were used only for qualitative comparison and evaluation in this investigation.

Thiele Geotech used a Humboldt Model H-4219 Heavy-Duty Dual-Mass Dynamic Cone Penetrometer, which uses a sliding 10.1 pound or 17.6 pound drop weight to drive a 20-millimeter-diameter, 60 degree cone, to perform the DCP tests. The number of blows and resulting penetration depths were recorded to provide a correlation to bearing pressure and CBR. The results were also used to evaluate relative changes in soil density/stiffness as the penetrometer was advanced. This testing was performed in accordance with ASTM D6951/D6951M, Standard Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications. Although not directly applicable to determining density of non-cohesive soils as would be the case with the SPT, the DCP was used due to limited access and space in the Maintenance Shop hallway. The data obtained can be used to identify zones of relatively low density or consistency compared to the surrounding subgrade as stated in ASTM D6951/D6951M, Note 1.

Laboratory test samples were collected by Thiele Geotech using 3-ft-long, thin-walled Shelby tubes at two test boring locations: one advanced about 2.3 ft west of Column MG-15 (identified as Boring No. ST-1) and one advanced about 2.3 ft east of Column MG-15 (identified as Boring No. ST-2). Borings were initiated using a 3-in.-diameter hand auger to advance through about 6 in. of gravel comprising the uppermost portion of the floor slab subgrade. Below the gravel layer, continuous Shelby tube samplers were advanced manually using a fence post-type slide hammer and soil samples collected to depths of about 4 ft below the top of subgrade. At Boring ST-1, a total of four Shelby tube samples, ranging in length from 8 to 12 in., were collected. At Boring ST-2, a total of three Shelby tube samples, ranging in length from 10 to 18 in., were collected. Refusal was encountered on coarse gravel at about 4 ft at both Shelby tube borings.

The investigation also included review of a previous geotechnical investigation report prepared in support of design of the original Maintenance Shop structure (Geotechnical Services, Inc., July 14, 1977).

4.3.6.2 Results of 2011 Testing

Forensic investigation results including field observations, SCP and DCP test results, laboratory test results from the 2011 portion of the investigation, and previous geotechnical investigation results are summarized below. Test reports by Thiele Geotech and the previous geotechnical investigation report are included in Attachment 6.

4.3.6.2.1 Observation Results

Visual observations and measurements were made as described above. Data obtained at each drill-hole are summarized below in Table 4-1.

Table 4-1 - Maintenance Shop Forensic Investigation Observations						
Drill-hole Number	Floor Slab Thickness ^A (in.)	Depth to Subgrade from Top of Slab ^A (in.)	Void Space Depth ^A (in.)	Straight-Line Distance from Column MG-15 ^B (ft)	Cardinal Direction Relative to Column MG-15 (based on plant north)	Comments
1-1	5.0	7.0	2.0	18.0	WSW	upper 6 in. granular fill ^C
1-2	5.3	6.0	0.8	18.0	WNW	upper 6 in. granular fill ^C
1-3	5.0	8.0	3.0	12.0	WSW	upper 6 in. granular fill ^C
1-4	5.0	8.5	3.5	11.5	W	fine-grained fill
1-5	5.5	7.5	2.0	9.5	NW	upper 6 in. granular fill ^C
1-6	5.0	9.5	4.5	7.5	SW	upper 6 in. granular fill ^C
1-7	5.5	8.0	2.5	5.5	W	upper 6 in. granular fill ^C
1-8	4.3	12.5	8.3	5.5	SSW	upper 6 in. granular fill ^C
1-9	5.0	11.5	6.5	2.0	NNW	upper 6 in. granular fill ^C
1-10	5.5	10.0	4.5	4.5	S	upper 6 in. granular fill ^C
1-11	5.0	13.5	8.5	3.5	N	upper 6 in. granular fill ^C
1-12	5.5	9.0	3.5	2.5	SE	upper 6 in. granular fill ^C
1-13	5.0	9.5	4.5	5.5	SE	upper 6 in. granular fill ^C
1-14	5.5	10.0	4.5	6.5	ESE	upper 6 in. granular fill ^C
1-15	6.0	8.5	2.5	11.5	ESE	upper 6 in. granular fill ^C
1-16	5.5	7.0	1.5	16.5	ESE	upper 6 in. granular fill ^C
EW-1	6.0	7.0	1.0	19.0	ENE	upper 6 in. granular fill ^C
EW-2	5.0	6.0	1.0	45.0	ENE	upper 6 in. granular fill ^C
EW-3	5.0	5.0	0.0	65.0	ENE	upper 6 in. granular fill ^C
EW-4	5.5	5.8	0.3	83.0	ENE	upper 6 in. granular fill ^C
NS-1	4.8	5.0	0.2	33.0	NNW	upper 6 in. granular fill ^C
NS-2	5.0	5.3	0.3	47.0	NNW	upper 6 in. granular fill ^C

Notes:
^A - Approximate value, rounded to the nearest 1/10-in., based on probe measurements using a tape measure.
^B - Approximate value, rounded to the nearest 0.5 ft, based on scaled plan drawings; not field measured.
^C - Subjective apparent material encountered based on CPT probe action during advancement through subgrade is believed to consist of fine-grained fill.
N = north, S = south, E = east, W = west

As indicated by the data above, floor slab thickness at the locations drilled ranged from about 5 to 6 in. at the testing locations. Drilling completed to access the floor slab subgrade resulted in penetration slightly below the slab. Observations during drilling indicated that the drill usually advanced beyond the slab bottom about 1 in. Measured void space of less than about 1 in. was not considered significant or representative of a void space.

Significant void space (greater than about 1 in.) was detected below the floor slab at all but one of the 16 locations drilled in the open area surrounding Column MG-15. Away from the settled column in the adjacent hallways, no significant void space was detected below the floor slab in any of the locations drilled (drill-hole locations EW-1 through EW-4, NS-1, and NS-2).

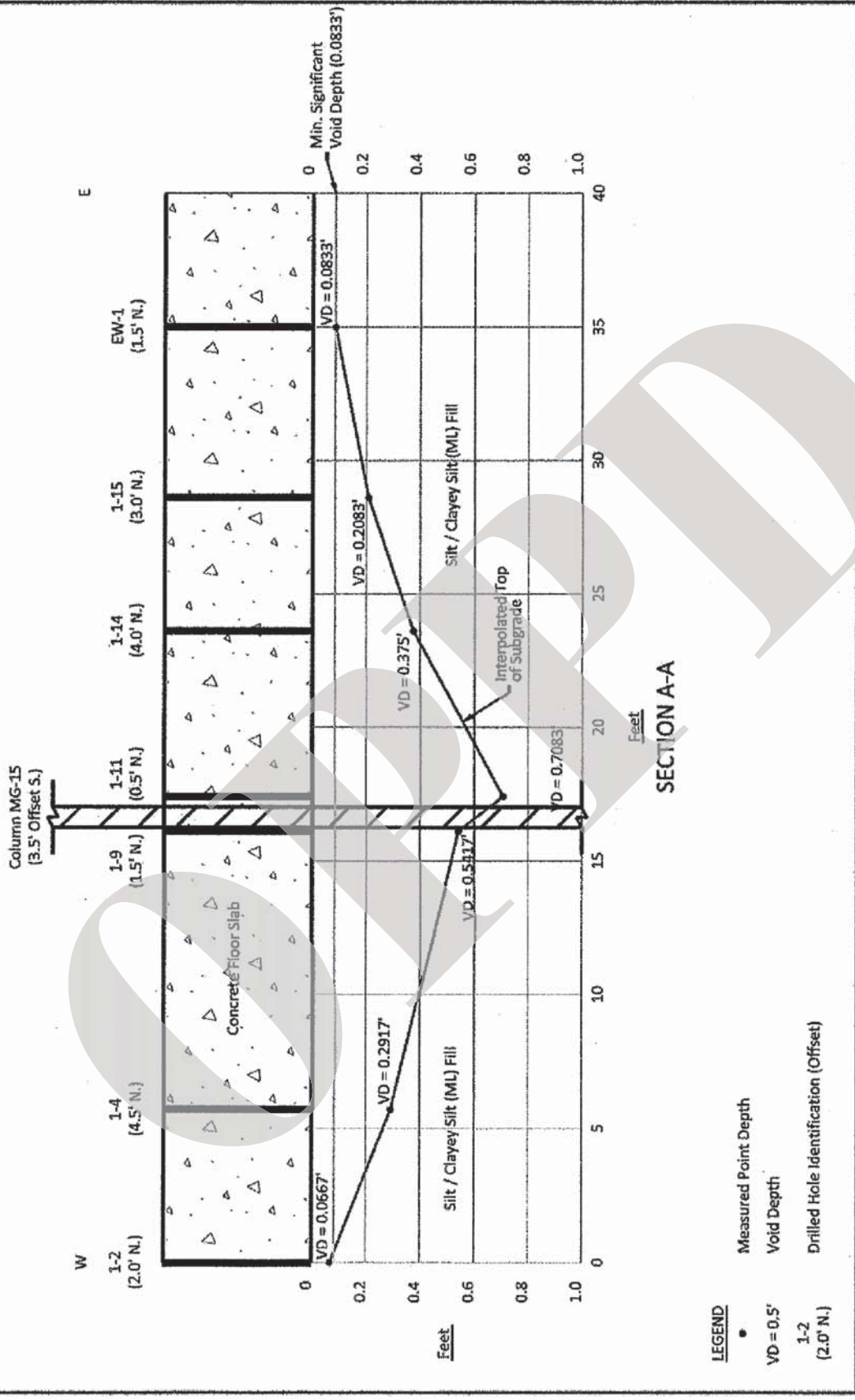
Overall, void depths ranged from zero to about 8.5 in. and averaged about 3.0 in. In the area surrounding the settled column, void depths were greater ranging from about 0.8 to 8.5 in. and averaging about 3.9 in.

The data also indicate that void space below the floor slab in the area investigated is greater nearer to the settled column, as follows:

- Within about 3.5 ft of the settled column, void space averaged 6.2 in.
- Within about 4.5 ft of the settled column, void space averaged 5.8 in.
- Within about 5.5 ft of the settled column, void space averaged 5.5 in.
- Within about 7.5 ft of the settled column, void space averaged 5.3 in.
- Beyond about 7.5 ft from the settled column, void space averaged 1.4 in.
- Beyond about 18 ft from the settled column, void space averaged 0.5 in.

The above statements related to void space are illustrated in cross sections A-A', B-B', and C-C', presented as Figures 4-14, 4-15, and 4-16. The cross section locations are shown in Figure 4-13.

Security Related and/or Personal Information Redacted



SECTION A-A

- LEGEND**
- Measured Point Depth
 - VD = 0.5' (2.0' N.)
 - 1-2 (2.0' N.)
 - Drilled Hole Identification (Offset)

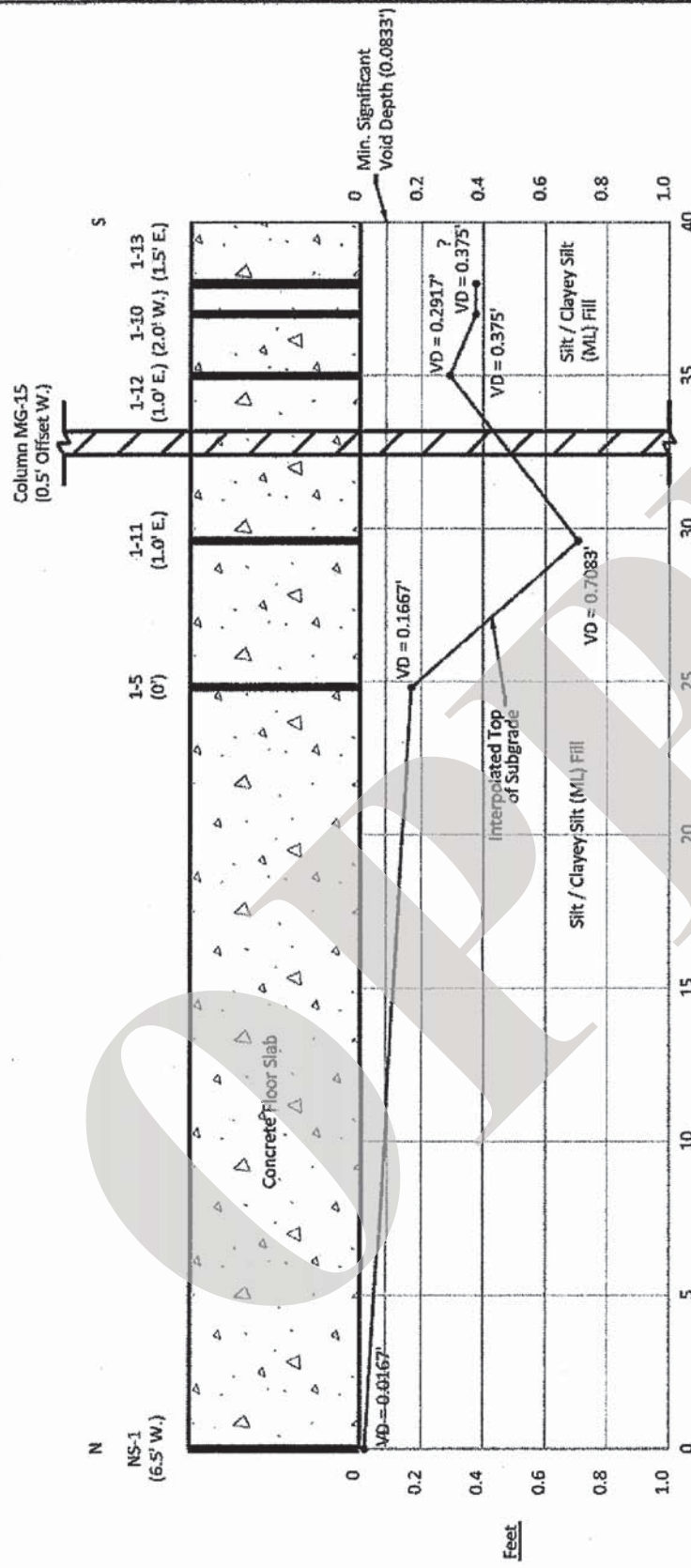
NOTES

- Nominal floor slab thickness is 5.5".
- Vertical Scale is 1" = 0.5'.
- Horizontal Scale is 1" = 5'.

DATE	Mar 2012
FIGURE	4-14

Maintenance Shop Drilling Locations
Cross Section A-A
Fort Calhoun Station
 Plant and Facility Geotechnical and Structural Assessment






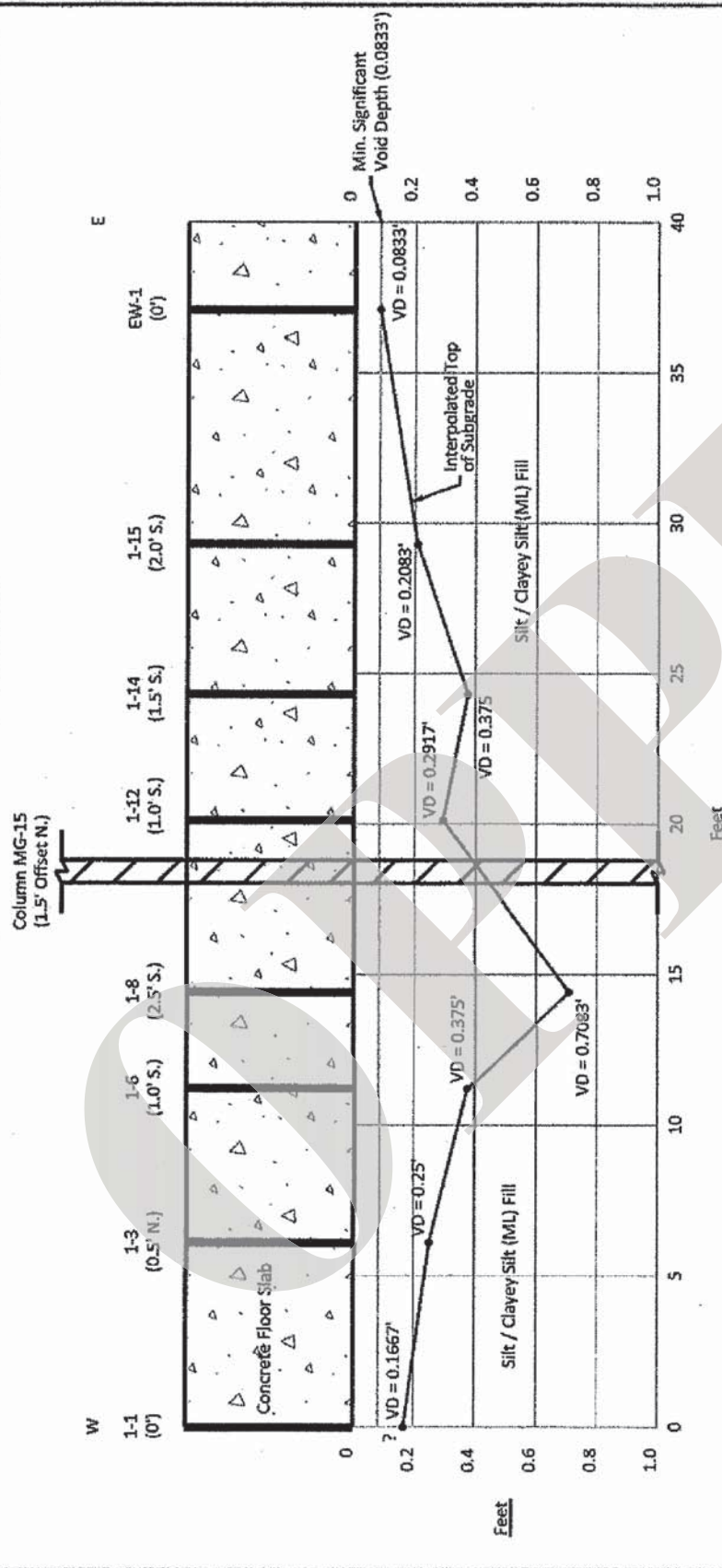
SECTION B-B

- LEGEND**
- Measured Point Depth
 - VD = 0.5'
 - 1-2 (2.0' N.)
 - ? Extent of Void Unknown

NOTES

- Nominal floor slab thickness is 5.5".
- Vertical Scale is 1" = 0.5'.
- Horizontal Scale is 1" = 5'.

 <p>Omaha Public Power District</p>	<p>Maintenance Shop Drilling Locations Cross Section B-B Fort Calhoun Station</p> <p>Plant and Facility Geotechnical and Structural Assessment</p> <p style="font-size: 2em; font-weight: bold; letter-spacing: 0.5em;">HR</p>	<p>DATE Mar 2012</p> <p>FIGURE 4-15</p>
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LEGEND

- Measured Point Depth
- Void Depth
- 1-2 (2.0' N.)
- Drilled Hole Identification (Offset)
- Extent of Void Unknown

NOTES

- Nominal floor slab thickness is 5.5".
- Vertical Scale is 1" = 0.5".
- Horizontal Scale is 1" = 5'.

DATE	Mar 2012
FIGURE	4-16

Maintenance Shop Drilling Locations
Cross Section C-C
Fort Calhoun Station
 Plant and Facility Geotechnical and Structural Assessment



OPPD00000205

4.3.6.2.2 SCP Test Results

SCP tests were performed as described above. Data obtained at each drill-hole are summarized below in Table 4-2.

Drill-hole Number	Depth from Top of Subgrade (ft)	Gauge Reading (kg/cm ²)	Approx. Correlated N-value (uncorrected, blows/ft)	Relative Consistency or Firmness ^A	Straight-Line Distance from Column MG-15 ^B (ft)	Cardinal Direction Relative to Column MG-15 (based on plant north)	Comments
1-1	0.0-0.5	24.5	6	soft	18.0	WSW	refusal 9-in. BTOS
	0.5-1.0	40	10	stiff			
1-2	0.0-0.5	20	5	soft	18.0	WNW	refusal 7.5-in. BTOS
	0.5-1.0	51	13	stiff			
1-3	0.0-0.5	6	2	soft	12	WSW	refusal 7-in. BTOS
	0.5-1.0	53	13	stiff			
1-4	0.0-0.5	45	11	stiff	11.5	W	refusal 36-in. BTOS
	0.5-1.0	48	12	stiff			
	1.0-1.5	14	4	soft			
	1.5-2.0	36.5	9	stiff			
	2.0-2.5	31.5	8	med. stiff			
1-5	0.0-0.5	35	9	stiff	9.5	NW	refusal 7-in. BTOS
	0.5-1.0	55	14	stiff			
1-6	0.0-0.5	8	2	soft	7.5	SW	refusal 8-in. BTOS
	0.5-1.0	53	13	stiff			
1-7	0.0-0.5	28	7	med. stiff	5.5	W	refusal 8-in. BTOS
	0.5-1.0	53	13	stiff			
1-8	0.0-0.5	54	14	stiff	5.5	SSW	refusal 4-in. BTOS
1-9	0.0-0.5	9	2	soft	2	NNW	refusal 23-in. BTOS
	0.5-1.0	5	1	very soft			
	1.0-1.5	16	4	soft			
	1.5-2.0	30	8	med. stiff			
1-10	0.0-0.5	9	2	soft	4.5	S	refusal 10-in. BTOS
	0.5-1.0	54	14	stiff			
1-11	0.0-0.5	52	13	stiff	3.5	N	refusal 1.5-in. BTOS
1-12	0.5-1.0	52	13	stiff	2.5	SE	refusal 6-in. BTOS
1-13	0.0-0.5	0	0	NA	5.5	SE	refusal 10-in. BTOS
	0.5-1.0	50	13	stiff			
1-14	0.0-0.5	0	0	NA	6.5	ESE	refusal 10-in. BTOS
	0.5-1.0	52	13	stiff			
1-15	0.0-0.5	30	8	med. stiff	11.5	ESE	refusal 6-in. BTOS

Drill-hole Number	Depth from Top of Subgrade (ft)	Gauge Reading (kg/cm ²)	Approx. Correlated N-value (uncorrected, blows/ft)	Relative Consistency or Firmness ^A	Straight-Line Distance from Column MG-15 ^B (ft)	Cardinal Direction Relative to Column MG-15 (based on plant north)	Comments
1-16	0.0-0.5	9	2	soft	16.5	ESE	refusal 8.5-in. BTOS
	0.5-1.0	54	14	stiff			
EW-1	0.0-0.5	39	10	stiff	19.0	ENE	refusal 7-in. BTOS
	0.5-1.0	55	14	stiff			
EW-2	0.0-0.5	58	15	stiff	45.0	ENE	refusal 6-in. BTOS
EW-3	0.0-0.5	59	15	stiff	65.0	ENE	refusal 5-in. BTOS
EW-4	0.0-0.5	56	14	stiff	83.0	ENE	refusal 5-in. BTOS
NS-1	0.0-0.5	52	13	stiff	33.0	NNW	refusal 5-in. BTOS
NS-2	0.0-0.5	50	13	stiff	47.0	NNW	refusal 14.5-in. BTOS
	0.5-1.0	32	8	med. stiff			
	1.0-1.5	46	12	stiff			

Notes:
^A - AASHTO Manual on Subsurface Investigations, 1988.
^B - Approximate values based on scaled investigation plan drawings; not field measured.
 BTOS = below top of subgrade.
 N = north, S = south, E = east, W = west

No voids were detected below the top of subgrade at the test locations surrounding the settled building column. Based on N-values (uncorrected) correlated from SCP test cone index results, medium stiff to stiff fine-grained fills were encountered at all test locations. Very soft to soft soils were sometimes encountered in the upper 6 in. of subgrade.

4.3.6.2.3 DCP Test Results

DCP testing was performed at drill-hole location I-12. The DCP test was performed from about 4.3 to 24.2 ft below the top of subgrade. Materials encountered ranged from medium stiff to stiff. No voids or very soft to soft soil zones were detected by the DCP test. Cone-index-correlated CBR values ranged from about 6.5 to 50. Cone-index-correlated bearing capacity ranged from about 2,000 to 7,000 psf (Humboldt Mfg. Co., 2003). Anomalously high CBR and bearing capacity values were obtained for soil at about 60 in. below the top of subgrade; these values are related to unusually high blow counts believed to be the result of the cone tip encountering a particle of gravel and are not included in the CBR and bearing capacity ranges mentioned above.

4.3.6.2.4 Shelby Tube Samples and Laboratory Test Results

Soils encountered during Shelby tube sampling were generally logged as silt and lean clay fill. Shelby tube advancement/recovery ranged from 8 to 18 in. Refusal was encountered at both test borings at about 4 ft below top of subgrade on coarse gravel (crushed limestone) believed to be at the previous plant grade and placed during original plant construction to stabilize the ground surface for heavy equipment traffic.

Moisture contents of the sampled soils ranged from 17.2 to 24.8 percent and averaged 20.6 percent. Void ratio (based on an assumed specific gravity of 2.7) ranged from 0.591 to 0.731 and averaged 0.600. Percent saturation (based on an assumed specific gravity of 2.7) ranged from 78 to 96 and averaged 86.

Wet unit weight ranged from 118.8 to 125.9 pounds per cubic foot (pcf) and averaged 123.6 pcf. Dry unit weight ranged from 97.3 to 105.9 pcf and averaged 102.5 pcf. Based on an assumed Standard Proctor (ASTM D698) dry density of 104 pcf, estimated relative compaction ranged from 94 to 100 percent and averaged 99 percent.

4.3.6.2.5 Previous Geotechnical Investigation Report

In 1977, a geotechnical investigation and report was completed by Geotechnical Services, Inc. to support the foundation design of the Maintenance Shop (July 14, 1977). The investigation included four test borings (one of which was advanced in the immediate vicinity of Column MG-15), field SPTs, collection of subsurface soils using thin-walled Shelby tube and standard split-spoon samplers, and laboratory testing. The investigation indicated that the southern portions of the planned building footprint (including soils upon which Column MG-15 is founded) consisted of 7 to 9 ft of loess derived fill that classified as medium stiff, low plasticity clayey silt (ML). Fine sand fill was encountered along northern portions of the planned building footprint. Below the fill, medium dense to dense stratified alluvium including sandy silty clays, fine sands, and thin clay seams were encountered. The report concluded the following:

- The building could be supported on shallow foundations.
- Existing site fills are suitable.
- Cohesive soils would provide a safe net (FS=3) allowable bearing pressure of 2,000 psf.
- Total settlement of 0.75 to 1 in. would be expected.
- Settlement would be rapid and differential settlements would not be a problem.

The forensic investigation for KDI #3 continued in 2012. The description of the scope and results of that activity as well as conclusions and recommendations resulting from that work is described in Section 8.0.

4.4 Comparative Evaluation of Geotechnical Analyses

The purpose of this comparative evaluation is to assess the potential impacts of the 2011 flood on the overall geotechnical conditions at the FCS site. This assessment included a comparative evaluation of new and existing geotechnical data in an attempt to assess whether the foundation soils have been disturbed or weakened from the sustained high water.

The primary basis of comparison for this evaluation was provided by 1) the penetration resistance data recorded during drive sampling and seismic refraction surveys completed as a part of the pre-flood investigations, and 2) the subsurface investigations conducted for this assessment. The penetration resistance data from these investigations provide an indirect but useful indication of the relative strength and stiffness of the subsurface soils and bedrock at the FCS site. The seismic refraction surveys provide an estimation of the p-wave (compression) and s-wave (shear) wave velocities, which can be an additional indication of the relative strength and stiffness of these materials.

4.4.1 Site Conditions

Site grades before construction at the FCS site generally ranged from about el. 995 to 1000 ft. At the boring locations for the post-flood investigations, the site grades varied from el. 1000 to 1005 ft.

For reference, the generalized subsurface profile at the FCS site consists of the following strata in descending order:

- A 1- to 10-ft-thick layer of existing earth fill, most of which was placed at the time of the original construction
- An intermittent layer of soft to firm, fine alluvium (silts and clays) that varies in thickness from 0 to 20 ft
- A 50- to 60-ft-thick layer of loose to dense, coarse granular alluvium (primarily silty to poorly graded sands with some clay seams)
- Limestone/shale bedrock at depth of about 75 ft below present grades, or at about el. 930 ft

The granular alluvium at the FCS site is composed of a loose to medium dense layer of recent alluvium extending to about el. 960 ft and underlain by a dense older alluvium extending to the top of rock.

Groundwater levels at the times of the pre-flood and post-flood investigations varied in elevation from about 986 to 1001 ft. River levels during the 2011 flood reached a high water elevation of approximately 1006.9 ft.

Additional discussions of the geological and geotechnical conditions at the FCS site are provided in Section 2.0 and by Dames & Moore (March 30, 1967, and January 30, 1968).

4.4.2 Pre-Flood Investigations

4.4.2.1 Dames & Moore Drive Sampling

The majority of the geotechnical data obtained from the pre-flood investigations was derived from the subsurface investigation completed by Dames & Moore (March 30, 1967, and January 30, 1968) of New York, New York. This investigation consisted of advancing 73 test borings in the area of the main facility using 3.25-in.-diameter hollow stem augers and drive sampling at 5-ft intervals in the overburden soils and BX-size rock coring in the bedrock. The depths of the borings ranged from 50 to 150 ft below grade.

The Dames & Moore "Type U" sampler and the SPT sampler were used to collect samples and measure driving resistance as the number of hammer blows per foot of sampler penetration. The Dames & Moore sampler retrieved 2.42-in.-diameter drive samples using a 300- to 350-pound weight falling a vertical distance of 24 in. (energy = 600 to 700 foot-pounds). The SPT

sampler retrieved 1.375-in.-diameter drive samples using a 140-pound weight falling from a vertical distance of 30 in. (energy = 350 foot-pounds).

4.4.2.2 Dames & Moore Seismic Refraction Surveys

Three deep and one shallow seismic refraction surveys were also conducted as a part of the 1967 Dames & Moore investigation. According to Dames and Moore, the deep geophysical refraction surveys, which ranged in length from 600 to 1000 ft, were conducted to: "indicate the depth to bedrock, the compressional wave velocities in the bedrock, and the compressional wave velocities in the soil overburden." Plate II C-1 in the Dames and Moore report lists s-wave velocities as well as p-wave velocities. Since the discussion in the report does not indicate that s-wave velocity measurements were made, it appears that the s-wave velocities which are reported in the table are estimated from the P-wave velocities and the assumed Poisson's ratio values in the table. In addition to the deep refraction surveys, a 60-foot long shallow survey was conducted to assist in evaluating the velocity of compressional waves in the upper soils. A short line was also used to investigate an anomaly identified in the deep surveys that was determined to be a group of timber piles.

The results of the seismic refraction surveys from the Dames & Moore investigation are presented in Attachment 5A. A summary of the p-wave velocities and estimated s-wave velocities is presented in Attachment 5A, Plate II C-1. Between the depths of 0 to 46 ft (with 46 ft below grade corresponding to roughly elevation 950 to 960 ft) the p-wave velocities were found to vary from 1000 to 2000 feet per second (fps) and the estimated s-wave velocities vary from 300 to 900 fps. Between the depths of 46 and 70 ft (with 70 ft below grade corresponding to roughly elevation 930 ft) the p-wave velocities were found to be 3000 fps and the estimated s-wave velocities were found to be 1400 fps. The p-wave velocity was 15,300 fps and the estimated s-wave velocity was 9000 fps in the bedrock.

4.4.2.3 Other Pre-Flood Investigations

The results of other pre-flood investigations conducted at the FCS site were also used in this evaluation, including the following studies:

- The 1987 investigation by Woodward-Clyde Consultants, consisting of 21 borings at the site of the Training Center (April 7, 1987).
- The 1989 investigation by Woodward-Clyde Consultants, consisting of 14 borings at the site of the Administration Building (September 26, 1989).
- The 2003 investigation completed by Shaw Stone & Webster, consisting of 9 test borings and 10 CPTs at the site of ISFSI (April 28, 2004).
- The 2003 investigation by Bechtel Corporation, consisting of 32 borings across the entire site (October 21, 2003).

4.4.2.4 Boring Location Plans and Logs

Boring Location Plans for the pre-flood investigations are provided in Attachment 5B, Figures 1 and 1A. The logs of the borings from the pre-flood investigations are not included as a part of this Assessment Report. However, a tabular summary of the pertinent penetration data from the pre-flood investigations is provided in Attachment 5C.

4.4.3 Post-Flood Investigation

4.4.3.1 General

The post-flood subsurface investigation was completed by Thiele Geotech in September 2011. This investigation consisted of advancing 9 test borings and 12 CPTs across the entire site. The locations of the borings and CPTs for the post-flood investigation are shown in Attachment 5B, Figure 1.

Prior to commencing each boring and CPT, OPPD required that the upper 10 ft of soil be hydro-excavated (soil removed by jetting and vacuuming) due to the potential for encountering shallow utilities across the site. As a result, no soil samples or geotechnical data were retrieved from the present ground surface to a depth of 10 ft at these locations.

Detailed discussions of these investigations are provided in Sections 4.1, 4.2, 4.3, and 8.3.

4.4.3.2 SPT Sampling

The test borings were advanced to depths of 46 to 76 ft (refusal on rock) below grade. Sampling of the overburden soils was completed using 3.25-in.-diameter hollow stem augers and drive sampling using the SPT sampler (ASTM D 1586-08a). Stability of the borehole was maintained during drilling with the use of bentonite slurry.

Prior to drilling and sampling, the SPT hammer systems on all drill rigs to be used in this investigation were energy calibrated by Foundation Testing and Consulting of Overland Park, Kansas. The calibration was performed to determine the actual efficiency in transferring the energy of the hammer blow to the drive sampler. The results of this calibration indicated that the efficiency of the SPT hammers used in the post-flood investigation ranged from 77 to 83 percent (Foundation Testing and Consulting, LLC, September 12, 2011).

4.4.3.3 Cone Penetration Testing

The CPTs completed by Thiele Geotech were advanced using an acoustic piezocone rig manufactured by Geoprobe Systems. The procedures for the CPT were performed in accordance with ASTM D 5778-07. The CPTs were advanced to depths of 16 to 47 ft below existing grade. Some of the CPTs reached refusal to further penetration in the dense alluvium at about el. 960 ft.

The CPT field data for tip resistance (q_c), sleeve friction (f_s) and pore pressure (U) are provided in CPT-Pro software format in Attachment 5C.

4.4.3.4 Geophysical Investigations

The post-flood investigation also included several geophysical testing methods conducted by Geotechnology. These methods included five seismic refraction lines and a series of GPR and spontaneous resistivity grids. The purpose of the geophysical testing, including the seismic refraction surveys, was to investigate the overburden soils in an attempt to identify the presence of soft or loose zones of soil or voids that may have developed at the FCS site from the flooding.

Locations of the seismic refraction surveys are provided in Attachment 5B, Figure 2. A full version of the report is provided in Attachment 6C.

The graphical results of the seismic refraction surveys from Geotechnology are provided in Attachment 5D and consist of plots of p-wave and s-wave velocity versus depth along each of the seismic lines. The plots display contours of the p-wave velocities that range from about 1000 fps near the surface to about 6000 fps at the base of the alluvium, and s-wave velocities that range from about 400 fps near the surface to about 2000 fps at the base of the alluvium. In general, the magnitude of both the p-wave and s-wave velocities was found to increase with depth, except some isolated zones of lower velocity material were encountered at depths of 40 to 75 ft below existing grade.

4.4.4 Interpretation of Penetration Resistance Data

Because the drive sampling in the pre-flood and post-flood investigations was completed under variable site conditions and with different equipment and technology, the penetration resistance values (in number of sampler blows per foot of penetration) had to be adjusted for these differences to allow a reasonable basis for comparison. Correction factors were applied to the field blowcount values to account for these differing conditions in accordance with ASTM D 6066-96. These factors included corrections for:

- *Overburden pressure at the time of drive sampling* – The grade of the site was raised about 10 ft since the 1967 Dames & Moore investigation, and groundwater elevation and resulting effective stress condition varied at the time of each investigation.
- *Hammer energy* – Drive sampling with the Dames & Moore sampler applied a greater amount of energy than the SPT procedure (600 to 700 foot-pounds versus 350 foot-pounds).
- *Borehole diameter* – Sampling in larger diameter holes is slightly less efficient in transferring the energy to the sampler than in smaller holes.
- *Drill rod length* – Drive samples taken with shorter lengths of drill rod are more efficient than sampling with longer sections.
- *Thin-walled liners* – The use of liners in the sampling process is less efficient than sampling without liners.

Each of the variables described above affects the magnitude of the field-measured blowcount value recorded at the time of the investigation. Following the application of these correction factors, a normalized resistance value is developed for a hammer efficiency of 60 percent. This normalized value is referred to as $(N_1)_{60}$. An efficiency of 60 percent was selected for the normalized value because the commonly used safety hammer with a rope-cathead has an efficiency of about 60 percent. In addition, many of the published correlations of SPT values and soil properties have been developed using blowcount data from this hammer system. Newer equipment that utilizes an automatic trip hammer typically has a higher efficiency (70 to 80 percent) than the rope-cathead system. Donut-type hammers typically have a lower efficiency that ranges from about 40 to 50 percent.

Calculations of the $(N_1)_{60}$ values and background information for the correction factors are provided in Attachment 5C. Plots of the $(N_1)_{60}$ values versus elevation of the recorded blowcount for the pre-flood and post-flood investigations are presented in Attachment 5C, Figure A-1. Using the correlations and procedures recommended by Robertson et al. (1986) and Lunne et al. (1997), estimates of $(N_1)_{60}$ were derived from the CPT data, and these data points have been included in these plots.

As depicted in Attachment 5C, Figure A-1, the $(N_1)_{60}$ values from the pre-flood and post-flood investigations show a similar pattern and scatter of blowcounts that range from 2 to 60 blows per foot along the full depth of the subsurface profile. The mean and standard deviation for the pre-flood and post-flood $(N_1)_{60}$ values are plotted in Attachment 5C, Figures A-2 and A-3, along the full depth of the profile. These plots also display a similar range and scatter of values.

4.4.5 Findings and Conclusions

Comparison of the computed $(N_1)_{60}$ values for the pre-flood and post-flood investigations indicates that there was no observable difference in the overall geotechnical conditions at the FCS site and that the foundation materials have not been disturbed or significantly weakened from the flood inundation. In addition, comparison of the seismic refraction data from the pre-flood and post-flood investigations reveals similar magnitudes of p-wave and s-wave velocities over the full depth of the overburden soils, and no observable differences were identified from this work. The presence of loose to medium dense zones with lower p-wave and s-wave velocities interbedded within denser materials confirms the inherent variation in the resistance data retrieved in the pre-flood and post-flood investigations.

Based on these findings and evaluations, it appears that the overall geotechnical conditions at the FCS site have not been significantly altered due to the sustained high water. The observed scatter of data points in both plots is consistent with the relatively wide range of strength and stiffness and corresponding blowcounts typically encountered in the alluvial soils within the Missouri River valley.

It should be noted that the findings and conclusions from the SPT comparative analysis are considered applicable only to those existing soils below a depth of 10 ft at the FCS site, or below an elevation of about 995 ft, because these soils were hydro-excavated to avoid damaging buried utilities. The upper 10 ft of soil may have been disturbed from underseepage and high exit gradients beneath the temporary levees during high water as minor flows were observed from pavement cracks at several locations. Additionally, disturbance to the site could have resulted from the settlement of utility backfill during drawdown of the river level and groundwater.

Additionally, it should be noted that these findings and conclusions are not applicable to the potential impacts that may have occurred due to the presence of the broken piping and groundwater flow into the Turbine Building Sump since the time of the site visits and post-flood investigations.

4.4.6 Limitations

This Assessment Report presents the findings and conclusions for an engineering evaluation of the potential impacts of the 2011 flood on the geotechnical conditions at the FCS site. It has been prepared in accordance with generally accepted engineering practice and in a manner consistent with the level of care and skill for this type of project within this geographical area. No warranty, expressed or implied, is made.

Geotechnical engineering and the geologic sciences are characterized by uncertainty. The professional judgments presented herein are based on HDR's review of available design and construction information provided to HDR, the results of field exploration and laboratory materials testing by others, the results of engineering evaluations, our general experience and the state-of-the-practice at the time of this writing.