

#### **4.0 GEOTECHNICAL TESTING, KEY DISTRESS INDICATORS, AND GEOTECHNICAL COMPARATIVE ANALYSIS**

##### **4.01 Introduction**

During the FCS site visual assessments, three problem areas were observed that potentially indicated that the 2011 flood had changed the FCS site's geotechnical and physical character. These observed problem areas are referred to as Key Distress Indicators (KDIs). This section of the Assessment Report will describe the geotechnical testing methods employed for the assessment, the testing accomplished in 2011, and the KDIs identified during the course of the assessment (KDIs #1, #2, and #3). In addition, this section presents the results of 2011 analyses related to the KDIs and the results of the geotechnical comparative analysis conducted for this assessment.

All investigations and analyses related to KDI #2 were complete in 2011, and the conclusions and recommendations described in Section 4.2.5 are considered final. Additional investigations were necessary in 2012 to finalize the conclusions and recommendations for KDIs #1 and #3. The 2012 investigations and final conclusions and recommendations for KDIs #1 and #3 are presented Section 8.0.

##### **4.02 Testing Methods – 2011 and 2012**

###### **4.02.1 Geotechnical, Geophysical, and Pipe Video Inspection Work**

In order to better understand how the recent flooding affected the geotechnical conditions of the site, it was necessary to gather subsurface information. The type of information to be gathered, the project schedule, and access limitations led to the conclusion that multiple investigative methods would be necessary to gather these data. The following four specialty subcontractors assisted HDR in gathering information to help evaluate various physical aspects of the site:

- Thiele Geotech, Inc., from Omaha, Nebraska
- American Engineering Testing, Inc., from St. Paul, Minnesota
- Geotechnology, Inc., from St. Louis, Missouri
- Elite Pipeline Services, from Grand Rapids, Michigan

Thiele Geotech, Inc. (Thiele Geotech) specializes in geotechnical drilling, sampling, and field and laboratory testing. This information was used to compare current soil conditions with past conditions, which are evidenced in previous borings and geotechnical studies of the site.

American Engineering Testing, Inc. (AET) specializes in nondestructive pavement testing, including GPR and falling weight deflectometer (FWD) testing. AET's information was used to evaluate pavement and subgrade conditions.

Geotechnology, Inc. (Geotechnology) specializes in geophysical ground evaluation surveys, including GPR, seismic refraction, refraction microtremor (ReMi), and other methods. Geotechnology's test results were used to evaluate areas of the site for underlying voids or wet, soft subgrade conditions.

Elite Pipeline Services (EPS) provides hydrocleaning and video inspections of buried pipes. EPS's data were used to evaluate the condition of suspected leaking drain pipes as well as other pipes.

These four specialty subcontractors made trips to the FCS site and performed services that were subsequently reported. Additional discussion of the work done is presented in the following paragraphs.

In addition, a survey and monitoring program was executed by Lamp Rynearson & Associates, Inc. (LRA), and groundwater monitoring well readings and Missouri River elevations were provided by OPPD staff.

#### 4.02.1.1 Thiele Geotech, Inc.

##### 4.02.1.1.1 Test Borings

Thiele Geotech drilled test borings, performed geotechnical laboratory testing on selected soil samples, performed cone penetrometer tests (CPTs), performed portable static cone penetrometer (SCP) testing, performed dynamic cone penetrometer (DCP) testing, installed groundwater monitoring wells, and installed inclinometers. This work was performed between the months of September 2011 and March 2012.

The test borings, CPTs, portable SCP testing, and DCP testing were performed to gather soil stratigraphy and engineering properties data. These data were used to evaluate soil conditions and to compare existing soil conditions with past conditions using information from previous geotechnical studies. In areas of the site where a drill rig could not be used due to access limitations, test borings were drilled with hand augers. Where a drill rig could be used, the test borings were advanced to depths ranging from 40 to 76 ft below existing grade with a Central Mine Equipment (CME) 55 truck-mounted drill rig and a CME 750 all-terrain drill rig using hollow-stem augers. The augers were kept full of bentonite slurry during drilling to stabilize the drill-hole and to prevent "blow-in" of sand into the augers. The hole depths were selected based on their intended evaluation purpose; the shallower borings were of sufficient depth to apply to heavy transformers supported on a mat foundation or a several-story structure supported on shallow foundations, while the deeper borings extended to bedrock and provided a soil profile from the ground surface to the bedrock. The deeper borings apply to deep-foundation-supported structures and provided a means to evaluate changes to the soil formation since the initial geotechnical investigations for the plant were performed.

At every exterior boring location, including test borings, CPT test holes, monitoring wells, and inclinometers, OPPD required Thiele Geotech to hydro-excavate to a 10-ft depth to clear any underlying utilities. Therefore, it was not possible to recover soil samples in the upper 10 ft. At the end of the drilling operation, each exterior boring was backfilled with a bentonite-cement grout above the depth at which the soil in the boring caved in. The grout was pumped through a tremie pipe to the bottom of the hole, displacing the water and drilling fluid as it filled the hole. The grout was a USACE-approved mix proportioned with a 94-pound sack of Type I Portland cement with 5 pounds of powered bentonite and 8.5 gallons of water.

##### 4.02.1.1.2 Sampling

Soil samples were obtained at the sampling intervals shown on the boring logs. Undisturbed samples, designated "U" samples, were obtained with thin-walled, 3.0-in.-outside-diameter tube samplers that were hydraulically pushed in general accordance with ASTM D1587-94, Thin Walled Tube Sampling of Soils. Undisturbed samples were typically taken in cohesive soils. Split-barrel samples, designated "S" samples, were obtained while performing standard

penetration tests (SPTs) with a thick-walled, 1.5-in.-inside-diameter sampler driven in accordance with ASTM D1586-84, Penetration Test and Split-Barrel Sampling of Soils. The N value, reported in blows per foot, is the number of blows required to drive the sampler for the last 1 ft of the sampling interval. SPT samples were taken in granular deposits.

The field boring logs were prepared in accordance with ASTM D2488-84, Description of Soils (Visual-Manual Procedure). Stratification lines represent the approximate boundary between soil types, and the transition between materials might be more or less gradual than indicated on the boring logs. Water-level readings were made in the borings at times and under conditions stated on the boring logs. The field boring logs were reviewed to outline the depths, thickness, and extent of the soil strata, and a testing program was established to evaluate the engineering properties of the recovered samples.

#### 4.02.1.1.3 Laboratory Tests

Specific laboratory tests that were performed include moisture content and density determinations, unconfined compression tests, hydrometer and sieve grain size distribution tests, Atterberg limits classification tests, specific gravity tests, and one-dimensional consolidation testing. Moisture content and density determinations were used to determine the existing moisture-density state of the soils. Unconfined compression tests were used to define the stress-strain characteristics and related shear strength of the soils. Grain size analyses and Atterberg limits were used to classify the soils under the Unified Soils Classification System. One-dimensional consolidation testing was done to evaluate the compressibility and past stress history of the soil. All tests were conducted in accordance with current ASTM test procedures.

Soil encountered in the test borings consisted of man-placed fill and alluvium. The man-placed fill was described as firm, moist to wet, lean clay and silt. The alluvium consisted primarily of poorly graded sand, poorly graded sand with silt, poorly graded sand with clay, and silty sand, with some clay and silt in the upper zones. Limestone/shale bedrock was encountered at depths between 69 and 76 ft below grade. Based on field observations including the drilling difficulty, and observation of the recovered samples, some weathering of the limestone/shale surface was present.

#### 4.02.1.1.4 CPT Testing

CPT testing was performed with a Geoprobe Systems acoustic cone penetration testing system. The test probe is temperature compensated and is equipped with individual sensors for point resistance, sleeve friction, and pore pressure. The cone was pushed with a CME 55 drill rig. Due to OPPD requirements that every hole be hydro-excavated to a 10-ft depth, anchoring the rig using screw-in anchors was not permitted. Refusal of the probe (as attained by lifting of the drill rig) was achieved in the denser sand layers at depths of about 16 to 47 ft. The CPT logs provide soil classification, point resistance, skin friction, pore pressure, and SPT  $N_{60}$  correlation data. The CPT results provide a means to correlate soil conditions with the test borings, and they provide a continuous record of the soil properties.

#### 4.02.1.1.5 Portable SCP Testing

Portable SCP testing was performed with a Humboldt H-4210 Portable Static Cone Penetrometer. This penetrometer uses a 1.5 square centimeter (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. Gauge readings were recorded at 6-in. intervals. Humboldt Manufacturing provides a correlation between the gauge reading and cohesion of the tested soils. This correlation was checked in 2009 by Thiele Geotech for Peoria loess clayey silt soils, which are similar to the some of the FCS site soils.

#### 4.02.1.1.6 DCP Testing

DCP testing was performed with a Humboldt H-4218 Dual-Mass Dynamic Cone Penetrometer, which uses a sliding 10.1 or 17.6 pound drop weight to drive a 20-millimeter-diameter, 60 degree cone. The number of blows and resulting penetration depths are recorded and provide a correlation to bearing pressure and California bearing ratio (CBR). The results can also be used to evaluate relative changes in soil density/stiffness as the penetrometer is advanced. DCP testing was conducted in accordance with ASTM D6951.

#### 4.02.1.1.7 Groundwater Monitoring Wells

Groundwater monitoring wells were installed at three locations. The primary purpose of the wells was to provide information on the response time for the saturated upper soils to drain back to static groundwater levels as the Missouri River level receded. The monitoring wells consist of 2-in. PVC slotted-screen and solid-riser pipe. A sand filter pack surrounds the screen, and a bentonite plug fills the boring annulus above the sand. Each well has a concrete pad surrounding a steel stickup protective cover, with steel protective bollards around the cover.

#### 4.02.1.1.8 Inclinerometers

Inclinometers were installed at five locations. Three of the locations were near the river bank. The purpose of these inclinometers was to provide a means to observe signs of lateral ground movement (specifically slope movement) in the area of the river bank, which can occur as river levels drop. Two of the inclinometers were located landward a significant distance from the river bank. These inclinometers were intended to serve as baseline references. The inclinometers were read by dropping an inclinometer measurement device down the casing and pulling it back up. The device reads deviations from vertical, allowing any change in orientation to be observed.

Each inclinometer consists of a grooved casing that is installed vertically. The tip of each casing is socketed 10 ft into bedrock. The casing was installed through a 4.25-in.-diameter hollow-stem auger after first drilling to depth and coring into the rock. After the auger was removed, the annulus of the hole was filled with a non-shrink bentonite-cement grout mix. A locking steel stickup cover was installed to protect the above-grade portion of the inclinometer casing (approximately 2 ft will protrude above grade). A 2-ft-diameter concrete pad was placed around the stickup cover, and steel bollards were installed to protect the stickup.

#### 4.02.1.2 American Engineering Testing, Inc.

AET performed nondestructive pavement testing, including GPR and FWD testing, on various driving and parking pavements. The purpose of this testing was to check for large-scale void development under the pavements and to evaluate the condition of the pavement and subgrade material following prolonged submergence under the floodwaters. GPR testing was performed with a bumper-mounted Horn Model 4105 2 gigahertz (GHz) air-coupled antenna and an SIR-20 control and data acquisition processor following the requirements of ASTM D4748. FWD testing was done using a Dynatest 8000 FWD Test System following the requirements of ASTM D4694.

#### 4.02.1.3 Geotechnology, Inc.

Geotechnology performed GPR testing and seismic survey, including seismic refraction and ReMi methods, at the FCS site. The purpose of this testing was to check for large-scale void development under pavements and slabs, and over utility trenches.

GPR is a geophysical technique in which a broadband electromagnetic (EM) signal is transmitted into the ground. The EM signal travels through underlying materials and is reflected by subsurface features. The reflected signal is recorded with respect to the time required for the signal to travel down and back. Data are collected along survey lines, and the results are presented in profiles representing reflected radar signals from beneath the survey line. A GSSI SIR 3000 GPR system with a 400 MHz antenna was used for data collection. This survey method was performed in accordance with ASTM D6432. After the GPR data was collected and analyzed, "ground truthing" was performed. For the GPR data, ground truthing consisted of drilling small diameter access holes for probing high amplitude GPR features with a simple hand-pushed probe rod.

The seismic refraction method involves generating compressional seismic waves (p-waves) at the ground surface using an impact source. The resulting profile represents p-wave velocities of the soil and bedrock directly beneath the survey line. This survey method was performed in accordance with ASTM D5777. Ground truthing (performed by Thiele Geotech) of the identified low-velocity features consisted of drilling SPT test borings through the features and performing SPT tests at closely spaced intervals.

The ReMi method is used to develop shear wave velocity profiles by passively recording background surface waves (microtremors) that are generated by, for example, passing vehicles, equipment, and airplanes. Shear wave velocity profiles are then constructed by analyzing surface wave phase velocities and frequencies, and performing inversion modeling.

#### 4.02.1.4 Elite Pipeline Services

EPS, a subconsultant to OPPD, performed video inspection of buried 6-in.-diameter floor drain pipes and 10-in.-diameter equipment drain pipes under the Turbine Building basement floor, a sanitary sewer line and floor drain line under the Maintenance Shop floor, and a sanitary sewer pipe under the floor in the Technical Support Center mechanical equipment room. The report prepared by EPS includes written descriptions of the testing as well as photos and videos taken inside the pipes, as discussed in Section 4.03.1.4.

#### 4.02.2 Survey and Monitoring Program

Following the 2011 Missouri River flood, LRA, a subconsultant to OPPD, conducted a general site survey and implemented a monitoring program at the FCS site to continuously collect and record elevations of reference point locations identified by HDR personnel and marked by LRA. General site survey work consisted of the following:

- Survey pumping locations (internal and external, permanent and temporary), including sump pumps used during the flood.
- Capture high water marks on a 200-ft +/- grid and limits of maximum flooding on the shoreline.
- Locate and verify site benchmarks (referenced to NGVD29 vertical datum) and clarify monitoring well locations.

The ongoing monitoring program at the FCS site consisted of monitoring structures for stability (that is, movement or lack thereof) and flagging monitoring reference points with apparent changes in elevation. On average, elevation data were collected each week, which exceeded the recommendations presented in the long-term monitoring plans in Section 5.0 of the Assessment Report.

#### 4.02.3 Groundwater versus River Level Mapping

OPPD provided HDR with groundwater monitoring well readings and Missouri River elevations. These readings were taken on site by OPPD personnel. HDR used these data to summarize the relationship between the groundwater elevation and the river elevation.

### 4.03 Testing Accomplished in 2011

#### 4.03.1 Geotechnical, Geophysical, and Pipe Video Inspection Work

As described in Section 4.02, five specialty subcontractors and OPPD staff assisted HDR in gathering information to help evaluate various physical aspects of the FCS site in order to better understand how the recent flooding affected the geotechnical conditions of the site. The following is a general description of the testing accomplished in 2011.

##### 4.03.1.1 Thiele Geotech, Inc.

Thiele Geotech drilled 15 test borings, performed 12 CPTs, performed portable SCP testing at multiple locations in protected and non-protected areas and inside the Maintenance Shop, performed DCP testing at 26 locations in the Turbine Building basement, installed 3 groundwater monitoring wells, and installed 5 inclinometers. This work was performed from September through December 2011. The locations of these borings, CPTs, monitoring wells, inclinometers, and some of the portable SCP tests are shown in Attachment 6A, Figure 1. Boring logs, laboratory test results, CPT logs, and test reports for portable SCP, DCP, and inclinometer testing are also included in Attachment 6A.

#### 4.03.1.2 American Engineering Testing, Inc.

AET performed nondestructive pavement testing, including GPR and FWD testing, during two trips to the FCS site. This testing was reported on September 14, 2011. Pavement that was tested during the first trip included Power Lane from the railroad tracks to the Training Center entry road, the access road to the Training Center and Administration Building parking lots, the Training Center parking lot, and portions of the Administration Building parking lot that were not under water at that time. The tested areas are depicted in AET's Report of Pavement Testing and Evaluation, Figure 1, Testing Locations, provided in Attachment 6B.

Pavement that was tested during the second trip consisted of a proposed haul route for removal of a circulating water pump motor from the Intake Structure. The haul route consisted of the Paved Access Area between the Service Building and the Intake Structure, from the north end of the Intake Structure southward toward the Security Building, then west between the security fence and the south side of the building, exiting at the southwest gate in the security fence, and then south to Power Lane. The purpose of this testing was to check the pavement integrity after sunken pavement panels and a void under a panel were identified by OPPD (see Section 4.2).

#### 4.03.1.3 Geotechnology, Inc.

Geotechnology services included two phases of work: a methods testing phase (report dated September 2, 2011) and a production surveying phase (report dated October 24, 2011). These reports are contained in Attachment 6C. The purpose of this work was to evaluate areas of the FCS site for underlying voids or wet, soft subgrade conditions.

The methods testing phase consisted of comparing four different geophysical test methods: GPR, EM surveying, spontaneous potential surveying (SP), and resistivity profiling. Geotechnology indicated that geophysical tests can be negatively influenced by the presence of shallow saturated soils, and that EM and SP testing are negatively influenced by buried utility lines.

Of the four test methods, GPR was found to be the most effective in identifying shallow subsurface anomalies. Three separate areas were chosen to evaluate the effectiveness of GPR at the site. One area was located on the main entry road at the service driveway into the Training Center; one area was located in the exterior corridor near the southeast corner of the Service Building; and one area was located in the Maintenance Shop near column MG-15. These areas were selected because they either had known anomalies (Maintenance Shop column) or were located where the presence of anomalies would be most likely, and were accessible for verification testing. Anomalous areas which were identified as voids had been previously found by Ground Penetrating Radar Systems, Inc., during an investigation conducted on August 2, 2011. For the methods testing phase of Geotechnology's work, testing known or suspected anomalous areas was more efficient than simply testing random areas and hoping to find an anomaly. To evaluate whether GPR would be effective at the site, it was important to test actual anomalous areas.

The GPR depth of penetration within the test areas was 5 to 7 ft. The mapped anomalies were described as either soft, wet clay or possible voids. As a means to investigate the potential anomalies identified near the Training Center, two small-diameter cores were advanced through the pavement within the surveyed area. As a result of the coring, the near-surface anomalous readings were identified to be wet/saturated soft clay at one location and wet/saturated soft clay with gravel at the other location. No voids were identified from this coring.

The production surveying phase was performed between September 6 and 30, 2011. The onsite activities were scheduled to coincide with a visit to the facility by U.S. Nuclear Regulatory Commission inspectors. The GPR grid locations were designated by OPPD and HDR, and included both areas within the PA surrounding the main plant building and within the basement of the Turbine Building. Two weeks of GPR data collection were followed by one week of drilling small diameter access holes for probing high amplitude GPR features. A total of 54 GPR grids were collected throughout the facility, and high amplitude GPR features were observed in all of the grids. Most high amplitude reflectors were observed at depths less than approximately 3 ft, and some were observed at depths greater than 3 ft. A total of 76 drill-holes were performed within 39 GPR grids. Eleven GPR grids were collected in the Turbine Building but could not be drilled by Geotechnology per the direction of the onsite OPPD contact. Drilling inside buildings could not be conducted within the project schedule due to requirements for obtaining the necessary plant clearances and the amount of time required to drill through 3 ft or more of concrete. However, GPR was ground truthed by subsequent forensic investigation conducted by HDR with the assistance of Thiele Geotech. Many of the GPR features drilled by Geotechnology were found to be areas of soft soils or large gravel. In one instance, a 22-in. void was observed during probing; however, the void was later discovered to be the Underground Cable Trench (Trenwa). The limited amount of ground truthing and use of the more simplistic probing method was chosen due to the project schedule and the requirement that every test boring be hydro-excavated.

Seismic survey methods, including seismic refraction and ReMi, were also performed at the FCS site. The seismic refraction method involves generating compressional seismic waves (p-waves) at the ground surface using an impact source. The resulting profile represents p-wave velocities of the soil and bedrock directly beneath the survey line. This survey method was performed in accordance with ASTM D5777. The ReMi method is used to develop shear wave velocity profiles by passively recording background surface waves (microtremors) that are generated by, for example, passing vehicles, equipment, and airplanes. Shear wave velocity profiles are then constructed by analyzing surface wave phase velocities and frequencies, and performing inversion modeling. Five seismic lines were laid out and tested between September 19 and 22, 2011. Low-velocity features that were identified within the alluvium were ground truthed by drilling SPT test borings through the features and performing SPT tests at closely spaced intervals.

GPR grids and seismic lines are shown in the Geotechnology report titled "Geophysical Survey for Void Detection," Plates 2 and 3, provided in Attachment 6C.

#### 4.03.1.4 Elite Pipeline Services

EPS, a subconsultant to OPPD, performed video inspection of buried 6-in.-diameter floor drain pipes and 10-in.-diameter equipment drain pipes in the Turbine Building basement. After initial video inspection, EPS determined that inspection of these pipes would not be possible without hydrocleaning. The only way to effectively remove debris from the buried pipes was to work from the sump with a hydrojet to pull all the debris in the pipes back into the sump. When EPS entered the sump, it found clean sand on the floor up to 3 ft high in some areas. After cleaning the pipes, the video inspection was completed.

Five separate pipes that enter the sump were video inspected. All of these pipes are either floor or equipment drain pipes, which would not normally carry water unless water was being discharged into them from various plant processes, including flushing of equipment (this water is termed "plant water"). According to OPPD personnel, no plant water was being discharged into the pipes at the time of inspection, yet water was flowing through the pipes into the sump. The video inspection found that all pipes contained water, with the majority having substantial flow. Both a pipe break and a joint separation were identified along with other areas of possible water infiltration. The inspected pipes deviated from the as-built mechanical drawings in some places, including an additional 101 ft of pipe not shown on drawings that HDR reviewed.

Additional video inspections took place in the Maintenance Shop building from a sanitary cleanout located between the sinking column (MG-15) and the restrooms. A low area was identified in this line. An effort was also made to inspect floor drain piping on the north side of the Maintenance Shop after removing several cleanout covers, but EPS found the entire system to be blocked and backed up with water. A sanitary sewer pipe in the Technical Support Center mechanical equipment room was also inspected, beginning at a cleanout near water heater PW-26 and extending 92 ft. The pipe was open, and surface aggregate was reported to be visible at 11 ft from the starting point.

The EPS report is presented in Attachment 6D.

#### 4.03.2 Survey and Monitoring Program

Following the 2011 Missouri River flood, LRA, a subconsultant to OPPD, conducted a general site survey and implemented a monitoring program at the FCS site to continuously collect and record elevations of reference point locations identified by HDR personnel and marked by LRA. General site survey work consisted of the following:

- Survey pumping locations (internal and external, permanent and temporary), including sump pumps used during the flood.
- Capture high water marks on a 200-ft +/- grid and limits of maximum flooding on the shoreline.
- Locate and verify site benchmarks (referenced to NGVD29 vertical datum) and clarify monitoring well locations.

The ongoing monitoring program at the FCS site consisted of monitoring structures for stability (that is, movement or lack thereof) and flagging monitoring reference points with apparent changes in elevation. On average, elevation data were collected each week, which exceeded the recommendations presented in the long-term monitoring plans in Section 5.0 of the Assessment Report.

From the end of August through December 2011, 264 monitoring reference points were identified and data were compiled into tabular form (see Attachment 6E). The total elevation difference (maximum minus minimum) for each reference point is shown in the table under column "ΔEL Range." The ΔELs exceeding the established accuracy and reliability range as specified by LRA are highlighted in red. The accuracy and reliability range is based on an aggregate of setup and equipment precision error totaling ±0.02 ft. Where practical, reference points were grouped by similar location and graphically represented in multiple figures to reveal trends (see Attachment 6E).

Based on the field data provided by LRA, 38 reference points appear to be outside of the accuracy and reliability range. For example, reference point TW29 (South Switchyard 161 kV Power Pole) has a total elevation difference of 0.24 ft. The principal difference occurred on the first reading. LRA's survey group leader reviewed the data and determined that no computational error was apparent in the elevation for this first point. However, there could have been a procedural error on the first reading, which cannot be confirmed by evaluating the data available. Moreover, the LRA survey crew chief stated that he does not believe that reference point TW29 has moved based on visual observations of the surroundings. Subsequent surveying of TW29 has not shown any indication of further movement.

Reference points outside of the accuracy and reliability range were reviewed by HDR staff as part of its assessment of the various FCS structures. The results of this analysis can be found in Sections 5.0 and 6.0 of this Assessment Report. Discrepancies in the data may be the result of windy conditions, human error, or some other means. LRA specifically indicated that there were some days when windy conditions may have impacted its ability to collect survey information.

Elevation surveys continued through December 2011.

#### 4.03.3 Groundwater versus River Level Mapping

OPPD provided HDR with groundwater monitoring well readings and Missouri River elevations. These readings were taken on site by OPPD personnel. HDR used these data to summarize the relationship between the groundwater elevation and the river elevation. Supporting documentation is presented in Attachment 6F.

OPPD has 22 existing monitoring wells, 20 of which are spread across the eastern portion of the FCS site (see Attachment 6F, Figure 1). Monitoring Wells (MW) 12A and 12B are located to the west and were not read in conjunction with this Assessment Report. Twelve of the monitoring wells were installed in August 2007 by Terracon; documentation and typical construction details are provided in Attachment 6F. The remaining wells were installed at an earlier date. Three additional monitoring wells were installed by Thiele Geotech in September 2011, as discussed in Section 4.02.1.1.7.

The OPPD Chemistry department sent HDR weekly readings of the monitoring wells from September 1 through October 7, 2011 (see Attachment 6F). A spreadsheet of pre-flood groundwater data was also provided by OPPD (see Attachment 6F). This pre-flood data contained readings from October 2007 to June 2011. Readings taken every 3 months are available starting in March 2008. The data provided by OPPD were measurements from the top of casing to water level. These were subtracted from top of casing elevations surveyed by LRA (shown in Attachment 6F, Figure 1) for the groundwater elevations used in the figures. The LRA survey information has been provided to OPPD.

OPPD also provided HDR with Missouri River elevations from a river gage located in the Intake Structure (see Attachment 6F). This includes elevations from February 28, 2005, to October 24, 2011. Data from January 1, 2007, to May 30, 2008, are missing as acknowledged by OPPD.

Both the groundwater and river data were plotted together using standard Microsoft Excel graphing features. The following four graphs were developed to summarize the relationship between the groundwater elevation and river elevation, and they are presented in Attachment 6F:

- A historical graph showing all data available from October 2007 to October 2011
- A graph showing data over the last 2 years (2010 and 2011)
- A graph showing the 2011 data, both pre- and post-flood
- A graph showing data during the flood drawdown from July to October 2011

#### 4.04 Key Distress Indicators

The three KDIs identified during the site visual assessments are:

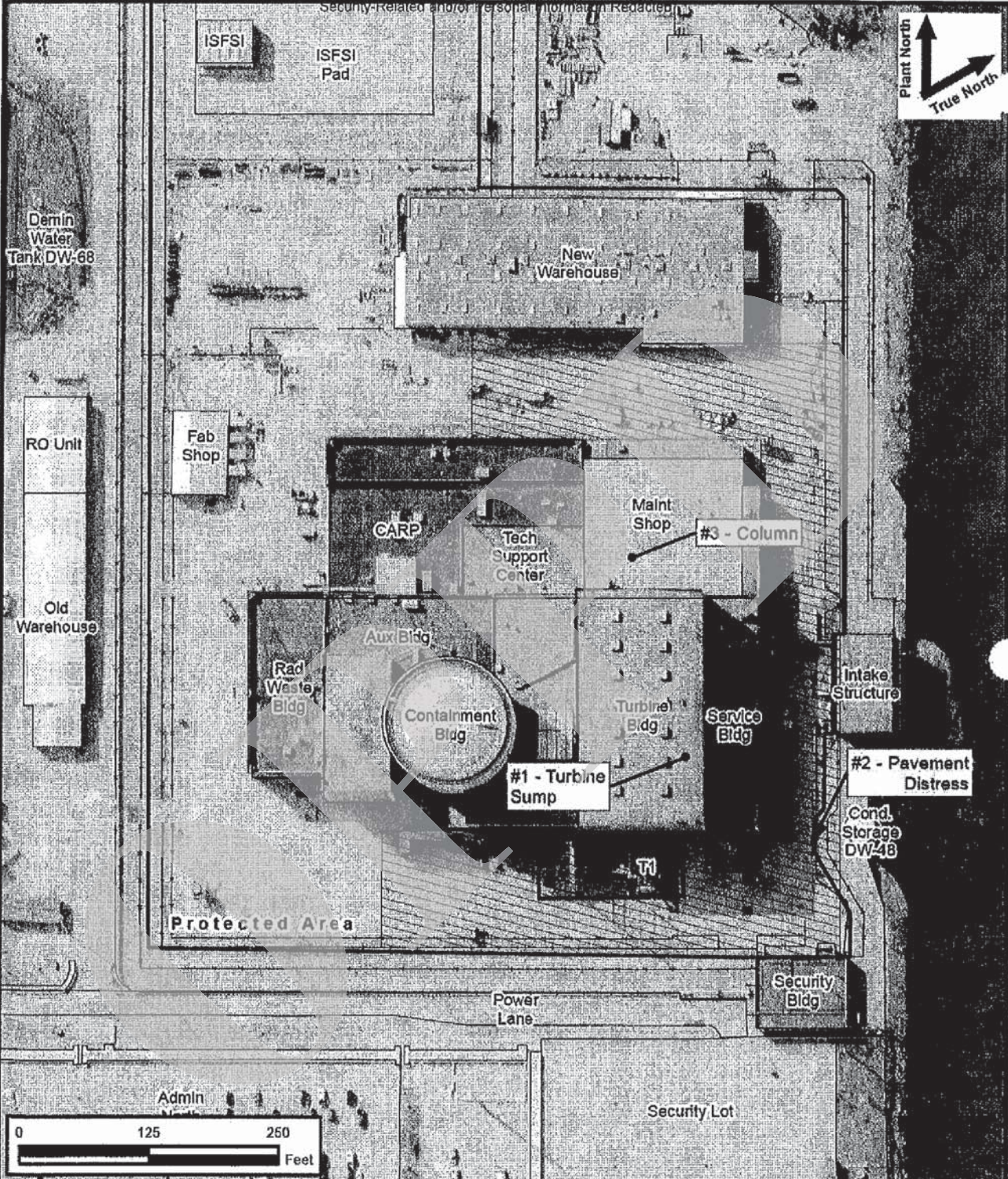
1. Increased flow into the Turbine Building sump
2. Pavement failure and surficial void in the Paved Access Area between the Intake Structure and Service Building
3. Column settlement in the Maintenance Shop

The locations of these KDIs are shown in Figure 4-1. Each of the observed KDIs was evaluated using PFM analysis to determine the associated Triggering Mechanism, to identify the CPFMs, to identify other structures that could be affected by the same PFM, and to recommend remedial measures intended to restore the KDIs to their pre-flood condition.

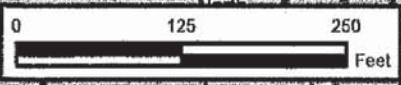
#### 4.1 KDI #1 – Increased Flow into the Turbine Building Sump

KDI #1 was the large amount of water observed flowing into the Turbine Building sump during the 2011 flood. It was determined that this large amount of flow could not be caused entirely by the intended function of the drainage system that leads to the sump, which is to drain water from the Turbine Building basement floor and from equipment housed in the Turbine Building. Review of records and a video of the drain pipe system confirmed that this large amount of flow was caused by groundwater flowing into the drain system through broken floor and equipment drain piping under the Turbine Building basement floor. This unintended, unfiltered flow of groundwater into the drainage system allowed soil to be transported from under the Turbine Building floor slab into the sump, where it was pumped out to the Missouri River. The Triggering Mechanism associated with KDI #1 is Subsurface Erosion/Piping (due to pumping). The three CPFMs associated with this Triggering Mechanism are the following:

- CPFM 3a – Undermining and settlement of shallow foundation/slab (due to pumping)
- CPFM 3b – Loss of lateral support for pile foundation (due to pumping)
- CPFM 3c – Undermined buried utilities (due to pumping)



Z:\Projects\OPPD\164565\_FCS\_2011\_Flood\_Services\Map\_Docs\Figures\SA\_Report\Figure 4\_1\_FortCalhoun\_DistressIndicators.mxd\_11/28/2011



- Fence
- Area of Concern



### Distress Indicators Fort Calhoun

Plant and Facility Geotechnical and Structural Assessment



DATE  
Sep 2011

FIGURE  
4-1

#### 4.1.1 Physical Observations

The Turbine Building has a documented history of a void below its basement floor slab dating back to 1997. This void was discovered via cored holes in the floor slab and video recordings of broken drain piping that lies under the floor slab. Conversations with OPPD personnel indicate that groundwater has been flowing at varying rates through these broken pipes into the sump from 1993 to the present day. Observations during and after the 2011 flood and historic records confirmed that the rate of flow into the sump is directly related to the hydraulic head of the groundwater; the observed flow rates have increased as the floodwater elevation increased across the facility. This drainage system was designed as a closed system; therefore, the pipes are not surrounded by appropriate filter systems to preclude the transportation of soils from under the slab. It is logical to assume that because the groundwater moves below the floor slab and into the broken piping, some movement of the soil has occurred. Video taken inside the drainage pipes confirmed the existence of breaks in the piping, and soil particles could be seen suspended in the water flowing in the pipes and traveling along the bottom of the pipes as bedload. There was also sand observed in the sump that is assumed to be material transported through the underslab drain piping.

The structures potentially affected by the CPFMs associated with the Triggering Mechanism (Subsurface Erosion/Piping) for KDI #1 are presented in Section 4.1.3. A complete description of each structure is presented in its respective subsection of Sections 5.0 and 6.0.

The following information was taken from a summary report prepared by OPPD dated March 24, 2009, regarding broken drain pipes:

CR2009-1365 was created 03/24/09 when Operations observed the floor drains backing up and flooding the floor when they were backwashing the condenser waterboxes. There are two drain lines that run parallel to each other: the 6-in. floor drain and the 10-in. waterbox drain. The drain lines are not cross-connected, so both lines must have a piping break if the 10-in. line is causing the floor drains to back up.

A vendor was brought in to visually inspect the drain lines. The vendor found a break in the 10-in. drain at the branch tee from the VD-193 drain valve but could not inspect the 6-in. floor drain because the line does not have a cleanout connection in this area, and accessibility through floor drains is restricted by a drain trap at each location.

A review of system files shows that a break in the waterbox drain line has been known about since at least 1993. In 1997, a repair was attempted by core drilling holes in the vicinity of the leak and pressure grouting to seal the leak. Per the "Water Systems Report Card for Report Period April 1 Through June 30, 1997" (memo PED/EOS SYE 97-123):

Repair of the Turbine Building Basement Drain line header was attempted during this period. The repair procedure consisted of core drilling holes in the vicinity of the leak and pressure grouting to seal the leak.

Approximately 10 holes were drilled and it was estimated that a void of approximately 10x8x1 feet existed under the concrete slab. The void was filled with cement grout but the leak could not be stopped. Boroscope inspection of the pipe exterior performed through the core drills showed considerable pipe damage, in more than one location. The extent of the damage and concern over collapsing the line were determining factors in terminating the pressure grouting operation. FC ECN 97-213 was originated to request that a new drain header be installed.

The grout was injected in the area by the VD-193 (FW-1A south return box tail valve). At some time later, the Turbine Building sump was cleaned out, and a slab of hardened grout was found in the sump, confirming that the grout had flowed through the drain system into the sump. A recent inspection of the floor drains revealed a considerable amount of grout in the floor drain south of the FW-3 Condensate Cooler. The drain looks to be almost fully restricted. This grout most likely came from the 1997 effort, indicating that both lines were broken at that time too.

#### 4.1.2 Triggering Mechanism

As discussed previously, the Triggering Mechanism associated with KDI#1 is Subsurface Erosion/Piping (due to pumping). Multiple potentially connected seepage paths could exist in the soil backfill at the site, including soil backfill in utility trenches, granular trench bedding, and building floor drains with open/broken joints. The paths could be exposed at some locations to the river floodwater and high groundwater. This network of seepage paths could be connected to the sump in the Turbine Building. The breaks in the piping have been documented for an extended period of time (dating back to at least 1993), maintaining a head differential on the potential seepage path networks. The gradient during the 2011 flood was increased, which most likely caused higher flows through the seepage path networks. The unfiltered seepage condition will continue until the breaks in the piping system are repaired, which means that the potential for further erosion remains. Subsurface erosion/piping could extend out, negatively affecting soils and/or creating voids under other structures.

Review of video from the sump and visual observations indicate groundwater flowing from all five drains. Drain lines are located below the floor slab. OPPD personnel indicated that the drain lines were cleaned in 2011.

Three soil borings (B-22, B-24, and B-70) were completed within the Turbine Building footprint as part of the Dames & Moore January 30, 1968, report titled "Foundation Studies." B-24 logged fine sand with clayey silt and silty clay lenses and SPT N-values of 7 and 11; B-22 logged fine sand with some medium sand and SPT N-values of 11 and 7; and B-70 logged fine sand with some medium sand and SPT N-values of 26 and 15. The fine sand is susceptible to piping if water velocity is sufficient. The zones of silty clay and clayey silt encountered in B-24 are the materials most susceptible to piping. Excavation beneath the Turbine Building footprint is shown in the Dames & Moore January 30, 1968, report in the drawing "Excavation and Grading Cross Sections" to extend to an approximate elevation of 984 ft for the overall foundation. Elevations of 979.2 ft were reached for the Sump Pit. Soil density tests reported by Nebraska Testing in 1968 during foundation preparation show density test elevation for the Turbine-Generator Foundation Mat as low as el. 977 to 980.2 ft and ranging from 97 to 100 percent of modified (specifications require 95 percent modified) using the American Association of State Highway and Transportation Officials (AASHTO) test method T-147-54 for the Turbine-Generator Foundation Mat. Material was described as brown sand. The test elevations below the excavation level of 984 ft likely indicate a zone between the piles overexcavated due to the presence of loose material.

The portion of the drain pipe located below the Turbine Building base slab consists of asbestos-cement sewer pipe. The material placed around the pipe is assumed to be sand from the excavation. No density tests of the backfill around the drain pipes are available.

This review of the data associated with the Turbine Building floor slab/foundation and drain piping confirm that the Triggering Mechanism of Subsurface Erosion/Piping (due to pumping) is a plausible scenario.

#### 4.1.3 Structures and CPFMs Associated with Triggering Mechanism

An initial list of structures that could have been negatively affected by the Triggering Mechanism of Subsurface Erosion/Piping (due to pumping) associated with KDI #1 was developed. The following is a list of those structures and the CPFMs associated with that Triggering Mechanism:

- Auxiliary Building
  - CPFM 3b – Loss of lateral support for pile foundation.
- Containment
  - CPFM 3b – Loss of lateral support for pile foundation.
- Rad Waste Building
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- Technical Support Center
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- Turbine Building
  - CPFM 3b – Loss of lateral support for pile foundation.
- Security BBREs
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- Turbine Building South Switchyard
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- Condensate Storage Tank
  - CPFM 3c – Undermined buried utilities.
- Circulating Water System
  - CPFM 3b – Loss of lateral support for pile foundation.
- Demineralized Water System
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- Raw Water Piping
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Fire Protection System Piping
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Waste Disposal Piping
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Fuel Oil Storage Tanks and Piping
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Main Underground Cable Bank, Auxiliary Building to Intake Structure (MH-5, MH-31)
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Blair Water System
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Main Underground Cable Bank, MH-1 to Auxiliary Building (MH-1, MH-2, MH-3, MH-4)
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.
- Service Building

- CPFM 3b – Loss of lateral support for pile foundation.
- Maintenance Shop
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- PA Paving, PA Sidewalks, and Outdoor Drives
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
- Sanitary Sewer System
  - CPFM 3a – Undermining and settlement of shallow foundation/slab.
  - CPFM 3c – Undermined buried utilities.

The influence of this Triggering Mechanism might be somewhat limited by the fact that vibroflotation of subsoils was conducted during construction of the Containment and the Auxiliary Building. However, subsurface erosion/piping cannot be completely ruled out because that Triggering Mechanism could extend to the Containment and the Auxiliary Building.

#### 4.1.4 KDI #1 Forensic Investigation

A forensic investigation program consisting of concrete floor slab drilling and subgrade testing was completed in the Turbine Building basement and surrounding structures to evaluate subsurface conditions for KDI #1. KDI #1 was the observation of increased flow of water into the Turbine Building sump that has entered the drain pipes through existing breaks in those pipes. The Triggering Mechanism associated with KDI #1 is Subsurface Erosion Piping (due to pumping) and the related CPFMs are 3a, 3b, and 3c, which are all “due to pumping.” The flow into the broken drain pipes has caused a cone-of-depression in the groundwater similar to what would have occurred due to the pumping of groundwater from a well (see Figure 4-2). The resulting flow through the subsurface soils into the broken pipes and then into the sump resulted in the piping of soil material out from under the floor slab in the basement of the Turbine Building, and possibly from the subsurface below adjacent structures. While system files indicate that these drain pipes have been broken since 1993, voids under the Turbine Building basement base slab were first observed in 1997, and remedial actions were taken by OPPD to grout the voids. These grouting efforts were unsuccessful in that the entrance of significant amounts of groundwater into the drainage pipes was not arrested. The purpose of this forensic investigation of KDI #1 was to obtain data to establish the degree and significance of piping and erosion of foundation materials, estimate the location and possible extent of the void or subgrade disturbance beneath the floor slab, and to ascertain their significance related to the CPFMs identified for the Turbine Building itself and other structures that may be affected by the piping and erosion features.

#### 4.1.4.1 Scope of Work for 2011 Testing

Initial investigation of KDI #1 began in August 2011 with estimates of flow into the sump, GPR surveys performed by Geotechnology of the floor slab and subgrade, and drainage pipe video investigation by EPS. The GPR surveys noted some anomalous zones of seemingly lower density material. Review of the drainage pipe video revealed six possible breaks in the 6- and 10-in.-diameter drainage pipe. Drill-hole locations 1-1 through 1-8, as presented in Figure 4-3, were located to investigate these lower density zones and pipe break locations. The Geotechnology report and EPS report are presented in Attachments 6C and 6D, respectively.

The 2011 portion of the sub-slab geotechnical forensic investigation of the Turbine Building basement subgrade began on November 11, 2011. A total of 27 floor slab locations were designated for drilling and subsequent underlying subgrade evaluation (see Figure 4-3 for drill-hole locations). One of the locations, 1-7, was not drilled due to OPPD plant safety concerns. Drill-hole locations 2-1 through 2-19 were located to provide as much data as possible near the edges of the foundation slab in order to assess the extent of subsurface erosion and piping (negatively affected soils or voids) away from the drainage pipes and sump locations.

Drilling through the Turbine Building basement floor was accomplished by Omaha Concrete Sawing and Lueder Construction Company, under contract to OPPD, using a hammer drill to advance 1-in.-diameter holes in the floor slab through which subgrade evaluations were performed. All drill-holes were covered immediately following drilling and before and after subgrade evaluations using temporary plastic caps that were flush with the surrounding floor surface. After testing was completed, the holes were filled with non-shrink grout by Lueder Construction Company.