Appendix E

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GEOTECHNICAL DESIGN REPORT

Closure of North, East, and West Ash Ponds
Bremo Power Station
Bremo Bluff, Virginia

GEOTECHNICAL DESIGN REPORT

Dominion

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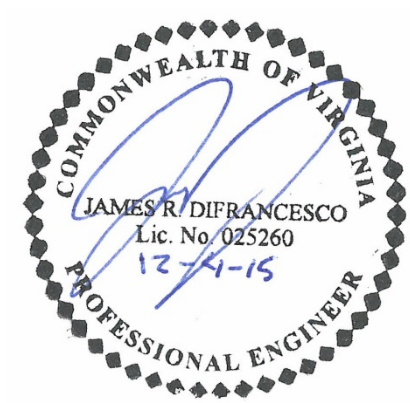

November 2015 **15-20347**

Golder Associates

November 30, 2015 1520347

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**RE: GEOTECHNICAL DESIGN REPORT
CLOSURE DESIGN FOR THE NORTH, EAST, AND WEST ASH PONDS
BREMO POWER STATION - BREMO BLUFF, VIRGINIA**

Dear Mr. Glagola:

Golder Associates Inc. (Golder) is pleased to submit this Geotechnical Design Report relating to the engineered closure systems proposed for the inactive Coal Combustion Residuals (CCR) impoundments at the Bremo Power Station in Bremo Bluff, Virginia. This report presents results and recommendations from the geotechnical analysis performed on the proposed closure plans to confirm design adequacy for stability, liquefaction, and settlement.

The data used for this report are presented under separate cover and included as Attachment 1 to this report. This report was prepared for use in regulatory permitting and to complete final design drawings for construction. Recommendations presented herein are based on the results of the specific analyses performed and known data. Should different field conditions exist than those presented in this report, certain geotechnical analyses may need to be re-evaluated to ensure design adequacy.

Golder appreciates the opportunity to assist you on this project. Please contact us at 804-358-7900 if you have any questions regarding this report or if we can be of any further service.

Sincerely,
GOLDER ASSOCIATES INC.

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PJD/GLH/JGM/JRD/glh

EXECUTIVE SUMMARY

The following are salient conclusions and recommendations from our geotechnical analysis of the three Coal Combustion Residuals (CCR, in this case, ash) ponds at Dominion's Bremo Power Station in Bremo Bluff, Virginia. Please review the text, tables, figures, and attachments to this report for details. Bremo contains three ash ponds (specifically, the North, East, and West Ash Ponds, or NAP, EAP, and WAP). The NAP and EAP will be closed in place by capping. The WAP will be clean-closed, with removal of CCR and conversion of a portion of the pond to a lined process wastewater treatment pond. These ponds have been in operation for 30 to 60 years.

1. This report addresses issues of concern for regulatory closure of the facilities based on the Virginia Solid Waste Management Regulations (VSWMR) and Virginia Department of Environmental Quality (DEQ) guidance.

   - **Geologically Stable** - The Bremo site lies in a geologically stable area with no active (Holocene) faults, karst (limestone, dolomite, or marble) potential, or other geologic conditions of concern.
   - **Seismicity** - Seismicity is low to moderate. The site lies within a seismic impact zone, but the design considers and addresses seismicity effects on stability and settlement of the pond closures including dikes, caps, and drainage as well as stormwater systems.
   - **Flood Plain** - The EAP and WAP lie within the 100-year floodplain, but perimeter dike crests are above the 100-year flood elevation. The 100-year floodwater levels do not impinge on the cover system.
   - **Groundwater Conditions** - Current groundwater levels are influenced by the ponds where impounded water produces a local rise in the surrounding groundwater levels. After the ponds are closed, which will include draining standing water, capping, and adding surface drainage improvements so that water will no longer be impounded the groundwater regime should fall to near pre-development levels, with an overall trend downward from the hills in the northeast toward the James River. There are no public or private drinking water sources between the ponds and the James River.
   - **Slopes -** The proposed closure slopes of the EAP, NAP, and WAP closures were evaluated and found to meet Virginia regulatory requirements, including factors of safety above 1.5 for static conditions and above 1.2 for seismic conditions (dam safety regulations). Additionally, slopes were evaluated for instability from liquefaction and a range of water level conditions that may apply to the closure design.
   - **Cover** - The NAP and EAP will have a permanent cover over the CCR consisting of a vegetative support layer, soil cover, geocomposite drainage layer, and geomembrane. The proposed cover meets the requirements of 9VAC20-81-160.D.2.f of the VSWMR.

- **Seepage** - While seepage from the ponds following closure is expected to decrease over time, Dominion plans to collect and discharge seepage from the ponds via the approved VPDES permit.

2. **Settlement** - Settlement of the ash was considered with respect to potential impacts on closure, notably changes in drainage of the cap and surface water ditches. Settlement is expected to occur due to additional load resulting from grading the ash and placing cover material during closure, as well as drainage from the ash during and after closure.

   - Laboratory test results and conventional consolidation theory show settlements up to several feet (up to 6 feet for portions of the NAP and up to 3.5 feet for portions of the EAP).
   - The grading design has taken these settlements into account, such that adverse slopes should not occur on the cover or in drainage ditches in the event such settlement occurs.

3. **Stability** - Stability of the dikes is critical to providing safe long-term CCR storage.

   - Existing dikes at the NAP and WAP provide acceptable factors of safety in their current configurations, with only minor improvements and routine maintenance.
   - Existing sections of the East Ash Pond require re-grading to ensure global stability. These improvements are included in the design and meet stability requirements.

4. **Liquefaction** - Liquefaction can occur in saturated, non-cohesive soils for a short time following seismic events. When it occurs, liquefaction causes loss of shear strength that can impact settlement and slope stability.

   - EAP ash, residual soils, alluvial soils, and fills derived from them are not calculated to liquefy in the design earthquake.
   - Some of the NAP ash, to the extent it remains saturated could liquefy in the design earthquake (2% probability in 50 years). For higher frequency events, such as the 10% in 50 years event, liquefaction of NAP ash is not anticipated based on our calculations.
   - Capping and closure will substantially reduce infiltration into the NAP ash, so the saturation level of the ash will drop well below the surface and liquefaction risk of ash near the surface will be substantially reduced over time.
   - Even if all ash liquefies, the NAP dikes will remain stable. As stated above, liquefaction is not calculated to occur for the EAP materials.
   - If the design earthquake occurs while ash near the surface is saturated, there is a risk of lateral spreading that could damage internal drainage channels that are cut into the ash and the surrounding cover in the NAP. This risk was identified and engineering controls in the form of a proposed buttress were implemented to provide adequate factors of safety against this potential instability mode.

- Measures to reduce potential for lateral spreading include additional soil or densified ash buttresses on either side of the ditches, and/or improvements to the drainage design to address the potential for movement near the ditches.

5. **Grading Ash** - The upper surface of the ash will need to be re-graded to accommodate stormwater drainage.

   - **Access** - Ash should form a "crust" if dewatered to several feet below the surface. The unsaturated crust of several feet thickness should support construction equipment. Procedures to evaluate the crust prior to operating equipment should be developed to limit risk to workers and equipment.
   - **Dewatering** - Ash can be dewatered using conventional methods including trenching and well points, similar to natural silty and fine sand soils.
   - **Handling** - Once drained, ash should become workable with conventional equipment, though some additional drying may be required for compaction. Double handling of ash, with excavation to stockpiles to allow drainage, has been effective at other sites. In dry weather, stockpile drainage is rapid, generally about 1 week to reach moisture contents acceptable for transport and compaction.
   - **Slopes** - Cut slopes of 4 horizontal to 1 vertical drained by trenching during excavation should provide acceptable stability. Steeper slopes of 3H:1V will be acceptable in drained, compacted ash.
   - Special care to ensure ash containment and stability during dike improvements will be required.

6. **Grading Dikes and Spillway** - Other grading associated with the closure should be routine as for other earthwork projects in the area. Some difficult excavation (rock and disintegrated rock) is possible in spillways in the existing dike abutments.

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1.0 INTRODUCTION

The Bremo Power Station (Station) is located in Fluvanna County, Virginia at 1038 Bremo Bluff Road, Bremo Bluff, Virginia. As of October 19, 2015, the Station contains three inactive Coal Combustion Residual (CCR) surface impoundments (ash ponds) as defined by the Federal Hazardous and Solid Waste Management System; Disposal of Coal Combustion Residuals from Electric Utilities; Final Rule (40 CFR 257; the CCR rule): the North Ash Pond (NAP), East Ash Pond (EAP), and West Ash Pond (WAP).

The three impoundments are being closed as inactive CCR surface impoundments under the CCR rule provisions at 40 CFR 257.100. Closed inactive surface CCR impoundments are not subject to further requirements in the CCR rule, but will be regulated under the Virginia Solid Waste Management Regulations (VSWMR). The closure plan has been prepared in accordance with the VSWMR; however, it also incorporates several of the measures described in the CCR rule. The Closure Plan generally follows the format guidelines for Solid Waste Disposal Facilities as described in Submission Instruction No. 6.

Golder has conducted geotechnical subsurface exploration in support of this effort to assess the subsurface conditions within the ponds and containment dikes. This report presents the engineering interpretations, analysis, results, and recommendations. Data collected have been presented under separate cover, and are also included as Attachment 1 to this report.

1.1 Existing Conditions

The Bremo Power Station is constructed on an alluvial terrace along the north side of the James River, east of U.S. Highway 15 (James Madison Highway), about 50 miles west of Richmond (Figure 1, below). The terrace is relatively level at about 212 feet elevation (NGVD, a local datum of NGVD less approximately 122 feet is used in many historic station documents). The river flows from west-northwest to east-southeast, but to simplify descriptions, we refer to the river orientation as west to east and perpendicular to the river as north to south. Along the north edge of the alluvial terrace, the ground rises moderately to steeply into rolling hills with well-developed dendritic drainage valleys, typical of the Piedmont Physiographic Province in which the site lies. Drainage from these valleys flows or flowed by natural meandering streams and/or man-made ditches and channels across the terrace to the river.

The Bremo Power station has three ash ponds: the NAP, WAP, and EAP. From startup in the 1930's until about 1972, the Bremo Station did not capture fly ash, so only bottom ash was placed in the ponds. From about 1972 until 2013, the Station used fly ash capture techniques, and considerably more ash, mostly fine-grained fly ash, was placed in the various ponds. The Bremo Station has converted to natural gas such that no additional ash production and storage are anticipated. Further description of the ponds and vicinity, based on site reconnaissance and document review, is included in the sections below. More detailed descriptions can be found in Attachment 1 - Design Geotechnical Data Report.

1.1.1 West Ash Pond (WAP)

The WAP covers about 17 acres in an area roughly bounded by the railroad to the south, Virginia secondary road 656 to the north, Spring Garden Creek and then the Bremo Power Station to the east, and a "metals cleaning pond" and undeveloped wooded land to the west. The WAP lies entirely on the alluvial terrace. The rising hillside is generally north of the road. The alluvial terrace was apparently sloping from about 220 feet near the road to about 215 feet south of the WAP. The WAP dike crest elevation is about 234 feet, so the dike is about 14 to 19 feet high. Existing dike slopes are about 2 horizontal to 1 vertical, except for a short segment on the southeast, which is indicated on the flown aerial topography as slightly steeper than 2H:1V. No significant indications of instability or erosion issues with the WAP dikes were noted during Golder's initial geotechnical investigation in March 2015. Dikes were constructed by borrowing alluvial soils from within the pond, thus deepening the pond.

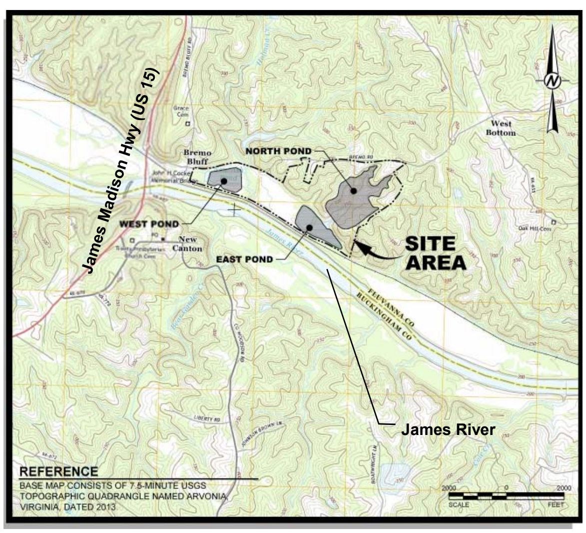

Figure 1: Bremo Power Station Site Location Map

1.1.2 East Ash Pond (EAP)

The EAP covers about 27.4 acres on the alluvial terrace east of the plant and former coal pile. It is bounded to the south by railroad main track, spurs, and a related drainage ditch, on the north by rising natural ground and the NAP, and on the west by the former coal pile. The EAP is roughly triangular in shape and defined by an earth dike that begins at a steep left or east abutment and extends about 1,900 feet to the west before turning north about 700 feet to meet the rising ground in the right or northwest abutment. Like the WAP, materials for the dikes were borrowed from within the pond, so are primarily alluvial soils from the terrace, and the pond is deeper than the surrounding terrace level.

1.1.3 North Ash Pond (NAP)

The NAP covers about 67.5 acres, northeast of the plant and immediately north of the EAP. Unlike the EAP and WAP, which were constructed over the relatively level alluvial terrace, the NAP was constructed by damming a steeper drainage feature in the rising natural hillside. The dike extends primarily across the mouth of the feature (about 1,000 feet long) and is over 100 feet high. The dike extends along the western ridge of the drainage, where it is much lower, generally less than 20 feet high. Dike materials were borrowed from several large borrow areas within the pond, exposing disintegrated rock and possibly rock in some areas now covered with CCR. Dry CCR fills were placed to the northeast of the pond (the microwave tower hill).

1.2 Proposed Improvements

The project includes the following improvements to the WAP, EAP, and NAP.

1.2.1 West Ash Pond

The WAP (and adjacent metals pond) will be “clean closed”; that is, the ash will be removed. Following ash removal, most of the existing perimeter dikes will be removed, with only the eastern and small portions of the northern and southern dikes to remain. Material from the perimeter dikes along with imported soils will be used to:

- Backfill the former WAP area with a gentle draining slope, essentially restoring the pre-development conditions, Build a new western dike parallel to and a short distance (about 200 feet) away from the existing eastern dike to form a smaller pond to be lined and used for plant process wastewater unrelated to CCR. The new western dike will have 3H:1V slopes on the interior and exterior, and;
- Re-grade the existing interior dike slope to 3H:1V matching the new dike above, which will enhance stability.

The exterior slopes of the dike will be confirmed to be 2H:1V or re-graded to that slope. After re-grading, a permanent liner will be placed within the new pond, which will be renamed the West Treatment Pond (WTP).

Due to access limitations, no exploration was conducted within the proposed new western dike alignment. The dike will be founded on the existing pond bottom on alluvial soils at the bottom of the former borrow excavation used to construct the original WAP dikes. While the condition of the foundation soils is not known, we anticipate that the upper part of these soils may have been or will be degraded due to disturbance by the original excavation, routine maintenance dredging operations, and final CCR removal operations. These foundation soils will be saturated when the pond is dewatered, and the schedule and need for the WTP will likely not allow for long-term natural drainage. Therefore, degraded alluvial soils will likely remain in this dike foundation, to be stabilized in place or bridged. A reduced strength of alluvium (indicated as degraded alluvium) was accounted for as a possibility in the WTP foundation analysis.

1.2.2 East Ash Pond

The EAP will be re-graded to improve stability and will include:

- Clean closure of the eastern portion of the pond (about 2 acres) for use as a stormwater dry detention pond for the NAP and EAP.
- Construction of a soil dike between the proposed dry detention pond and the remainder of the ash.
- Improvement of the existing perimeter dike by re-grading to a maximum slope of 2.5H:1V, as well as removing trees, and providing exterior seepage control. These modifications will require temporary dewatering and re-grading of ash behind the existing dike.
- Dewatering, excavating and replacing (with compaction) ash in and below proposed ditch slopes to improve stability.
- Abandoning, removing, or grouting existing pipe spillways and outlet structures.
- Re-grading ash within the EAP to improve surface drainage.
- Capping the ash.

1.2.3 North Ash Pond

The NAP dike provides acceptable stability factors of safety, and no modifications are necessary. Ash in the NAP will be re-graded to provide surface drainage for the cap and to convey stormwater via ditches to a new spillway to be excavated through the right (west) dike abutment. Proposed grading will eliminate stormwater storage in the NAP. Existing spillway structures will be removed and/or abandoned in place by grouting. The dry ash fill area to the northeast will have minor grading improvements for drainage, and will be capped with the rest of the NAP.

1.2.4 Stormwater Handling

The new NAP spillway will convey water from the NAP and upgradient drainage area down the right abutment of the NAP dike, via an armored channel, to a plunge pool near the northwestern corner of the EAP. Water will combine with most of the drainage from the EAP surface and flow along a ditch over the north edge of the EAP to the new stormwater dry detention pond (clean closed eastern end of the EAP). The dry detention pond will release water through a control structure into the stream adjacent to the CSX railroad embankment, then out through the existing brick arched culvert under the railroad embankment to the James River.

The WAP footprint will largely be returned to a state similar to the original grades, with sheet flow to the northeast towards Holman Creek, then to the James River. The remaining WAP pond (West Treatment Pond) will not receive any stormwater except direct precipitation.

During flood events, the James River waters will rise along the EAP and WAP dikes. Both dikes have proven in the past to be able to sustain flood waters without damage to the embankment from flood water velocities or rapid drawdown conditions. Therefore, no additional armoring protection is proposed at this time.

2.0 ANALYSIS AND RESULTS

Principal concerns regarding closure of the ash ponds are the dikes and cover. Dike concerns include hydrologic controls (spillway capacity), stability, seepage, erosion, and maintenance issues. The main concerns with the cover are erosion, settlement (resulting in ponding or reversal of flow of surface drainage), and maintenance.

Our analysis focused on stability of key slopes in the existing and proposed conditions as well as expected settlement to address these principal concerns. Seepage was considered and addressed through maintaining existing toe drain controls on the NAP and installing a toe drain for the EAP. Because the WTP will be a lined pond, seepage is not a principal concern. Site seismicity and response was also considered and incorporated into our analyses.

2.1 Parameter Selection

Subsurface materials include CCR, alluvial soils, residual soils, disintegrated rock, and rock, as well as existing and proposed fills derived from alluvial and residual soils. Except for rock and disintegrated rock, each material was further subdivided with potential different properties for materials at various depths in different locations and using accepted methods to extrapolate properties around and between exploratory points. Figure 2 illustrates schematically the subsurface materials from the James River to the north end of the NAP. Strength, compressibility, permeability, and dynamic properties for these materials were developed based on field and laboratory data, and parameters were selected for the various analytical models used and described in the following paragraphs. Attachment 2 - Geotechnical Material Properties outlines the properties, selection process, and methods for each material.

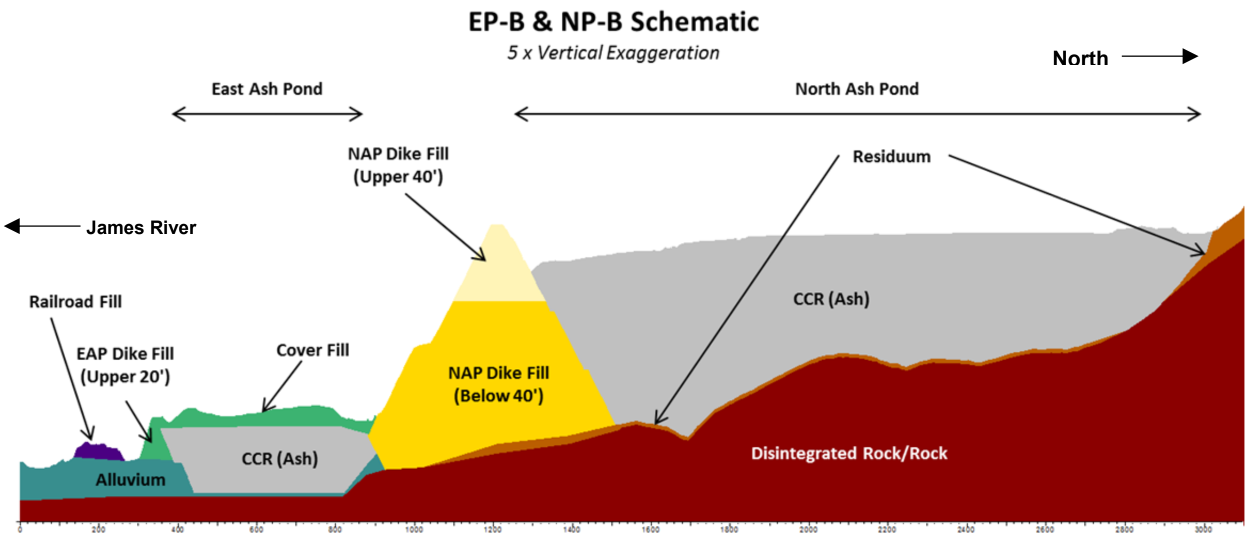

Figure 2: Schematic cross-section of existing (2015) subsurface conditions at EAP and NAP

2.2 Seismic Issues

The Bremo Site is located within a seismic impact area that can affect the stability and settlement analysis. Therefore, seismic events were considered in the design and analyses, and it was determined that the design addresses potential ground movements and meets required factors of safety.

2.2.1 Site Seismicity

The Bremo Site is in a seismic impact zone, defined as an area with a 10 percent or greater probability that the maximum horizontal acceleration in lithified earth material (generally, bedrock), expressed as a percentage of the earth's gravitational pull (g), will exceed 0.10g in 250 years (10%/250 years, or about a 2,370-year return period) (40 CFR 258.14). The design earthquake for the Bremo Site was taken from the 2008 USGS National Seismic Hazard Map. The design earthquake has a probability of occurrence of 2 percent in 50 years (2%/50 years, or a 2,475-year return period) and produces a peak ground acceleration (PGA) of 0.14g. A higher frequency was also considered (10%/50 years, or a 450-year return period) with a PGA of 0.03g. The PGA is the expected motion of the bedrock, so amplification through the soil/ash profile was considered as appropriate, meeting required factors of safety.

2.2.2 Liquefaction

Liquefaction is a phenomenon that can occur under seismic shaking, in loose, saturated, non-cohesive soils. Liquefaction occurs when relatively severe earthquake accelerations cause temporary increases in pore pressure that exceed confinement and/or the ability of the soil structure to resist such pressures. Liquefied soils lose significant strength, but not all strength, and can flow like a viscous liquid. Effects are temporary, beginning near the end of the seismic event and ending shortly after the cessation of the seismic event. Typical issues that can arise from liquefaction include:

- Sand boils or volcanoes – These occur when liquefied materials flow to the surface and form characteristic features that resemble small volcanoes. The movement of material from the subsurface to the surface can result in significant settlement.
- Lateral spreading – Liquefaction can weaken soils and create slope instability such that relatively flat slopes can fail, graded surfaces can flatten, and ditches can close.
- Increased compressibility – Liquefied soils may lose microstructure and become more compressible. While generally causing much smaller settlement issues than sand boils, this can increase settlement of overlying structures.

Liquefaction potential of various soils at the Bremo site was calculated based on shear wave velocities correlated from Cone Penetrometer Tests (CPT) tip resistance and the design earthquake. The published bedrock peak ground acceleration assumed amplification through the ash/soil profile. Attachment 3 provides details of the calculations.

2.2.2.1 Liquefaction of Natural Soils and Soil Fills

The natural alluvial and residual soils at the Bremo site, as well as fills derived from those soils, are generally cohesive and/or have sufficient consistency [based on CPT and Standard Penetration Tests (SPT) data] to resist liquefaction in the design earthquake.

2.2.2.2 Ash Liquefaction

CPT data from the EAP indicate that ash will not liquefy in the design earthquake. Analyses indicate that, under saturated conditions, portions of the ash in the NAP could liquefy in the design earthquake, but not in the 10/50 event.

For liquefaction of ash to occur, the ash must be nearly saturated. Closure and capping of the NAP and EAP will substantially reduce infiltration of surface water into those ponds, allowing the saturation level of the ash to decrease over time. After closure, a new equilibrium condition will develop with the saturation level of the ash well below the current levels. As such, liquefaction will occur only deep within the ash. Since the earthquake duration is limited, breakout of ash to the surface in the form of sand boils and/or lateral spreading of surficial ash is unlikely. Liquefaction effects on settlement and stability are addressed in the sections below.

2.3 Slope Stability Analysis

Slope stability analysis was conducted on selected sections of perimeter dikes, as well as proposed interior slopes (ditches and other slopes in the ponds), for the existing condition and design configurations. The purpose was to assess compliance with regulatory requirements for stability under static and seismic conditions, including with liquefied ash. Stability analyses were completed using the computer program SLIDE 6.0 Version 6.036 by RocScience. SLIDE computes potential failure surfaces using a general limit equilibrium (GLE) method developed by Morgenstern and Price (Abramson *et al*., 2002). The method is based on the principle of limit equilibrium (i.e., the method calculates the shear strengths that would be required to maintain equilibrium, and then calculates a factor of safety by dividing the available shear strength by the shear strength required to maintain stability). A "grid" circular failure surface search-method was used in this study and checked with SLIDE's "auto refine" circular failure surface search method. For these iterations, safety factors in excess of 1.0 indicate stability, and those less than 1.0 indicate a potential for instability. Figure 3 illustrates the results of a typical slope stability analysis using a grid search and SLIDE 6.0. See Attachment 4 for details and reference information.

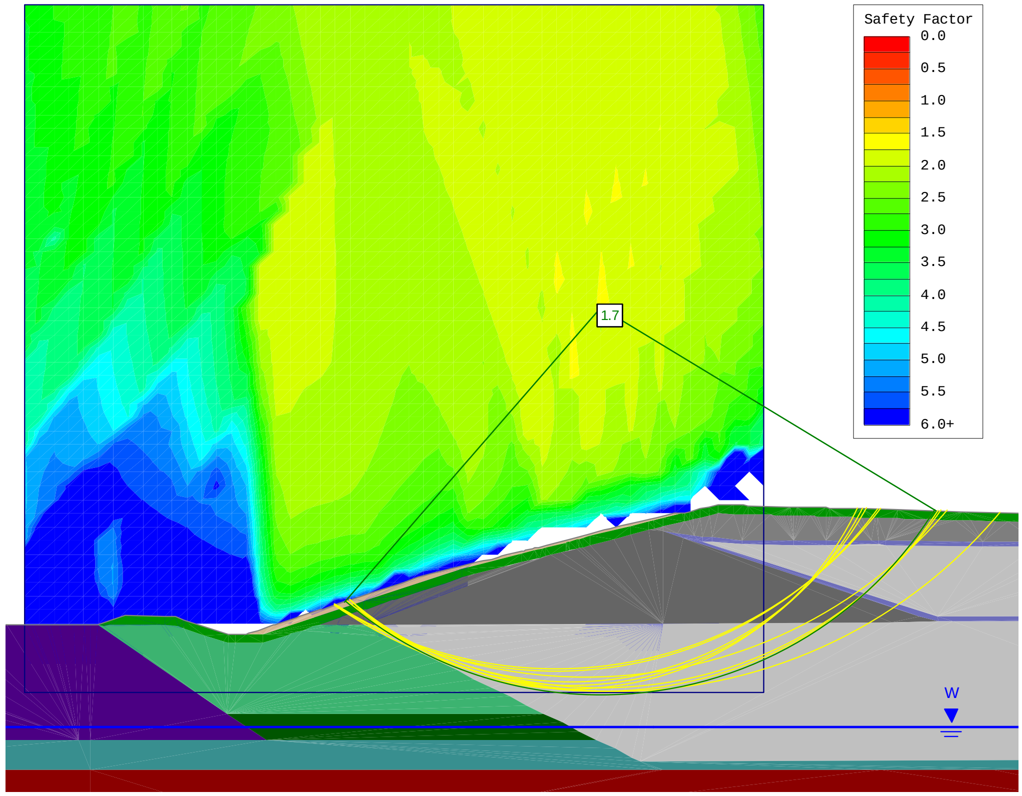

Figure 3: Example of slope stability analysis results from SLIDE 6.0

2.3.1 Factors of Safety Selection

In Virginia, impounding structures (such as the Bremo dikes) are regulated under the Virginia Dam Safety Act, Article 2, Chapter 6, Title 10.1 of the Code of Virginia and the Dam Safety Impounding Structure Regulations (4VAC50-20), established and published by the Virginia Soil and Water Conservation Board (VSWCB). The Impounding Structure Regulations do not specifically identify minimum factors of safety for design and analysis; however, the Virginia Department of Mines, Minerals, and Energy's Water Impoundment Regulations (4VAC25-31-500) do. The Water Impoundment Regulations require that slopes meet a static safety factor of 1.5 under steady state seepage conditions and a seismic safety factor of 1.2. These values represent the standard of care for the design factors of safety, and are in alignment with accepted sound engineering practice.

2.3.2 Slope Geometry

Slope geometries selected for analyses were based on the proposed design grades and existing surrounding topography. Critical sections were selected for analysis based on slope length, steepness, and surrounding terrain and physical features such as ditches, streams, or other abrupt changes in topography.

2.3.3 Seismic Inputs

Stability under seismic conditions is calculated using the pseudo-static method to model horizontal seismic forces as the product of a seismic coefficient (k) and the weight of the sliding mass (vertical seismic forces are typically neglected). The seismic coefficient is estimated from the peak ground acceleration (PGA) expected at the site. Hynes-Griffin and Franklin (1984) recommend using a seismic coefficient equal to half the PGA with a 20% shear strength reduction, where the shear strength is based on a composite of the total and the average of the total and effective shear strength envelopes.

2.3.4 Liquefied Ash Sections

Liquefaction of ash is expected to occur only in low frequency earthquakes (i.e., the 2,475-year return period design earthquake) and only in uncompacted, saturated ash in the NAP. The NAP is expected to drain substantially within a matter of a few years to a decade after closure. Liquefaction of deeper ash zones that exist away from the drainage channels is calculated to not impact stability of the ditch slopes. Therefore, within a few years after closure, the liquefaction risk is negligible.

The design incorporates improvements along ditches in the NAP to prevent localized liquefaction that could cause lateral spreading displacements of the ditch slopes. To demonstrate stability, a zone of shallow saturation extending about 100 feet from critical ditch slopes was analyzed and found to have acceptable factors of safety.

2.3.5 Strength Parameter Selection

Material properties for the various strata were interpreted based on subsurface data and site reconnaissance presented in Attachment 2 and documented in the Design Geotechnical Data Report in Attachment 1. These material properties are presented in the table below. Drained strength parameters were chosen for long-term stability analysis and undrained strength parameters for short-term stability and seismic analysis.

Table 1: Summary of Geotechnical Strength Properties

Summary of Geotechnical Strength Properties East, North, and West Ash Ponds				
Material	***Total Unit Weight (pcf)***	***Strength Properties***		
		Peak φ' (°)	**Cohesion (psf)**	**Su (tsf)**
Uncompacted CCR	90	28	0	0.5
Liquefied CCR	90	N/A	N/A	0.04xVertical Stress
Dike Fill Soils- NAP	125	0 - 40 ft: 31 > 40 ft: 28	50	2.0
Dike Fill Soils- EAP and WAP	125	0 - 20 ft: 31 > 20 ft: 28	50	1.5
Alluvium	115	28	50	1.0
Degraded Alluvium (applicable to excavated areas of WAP)	115	N/A	N/A	0.25
Residuum	125	28	50	1.5
Clay Liner (EAP vertical expansion)	125	26	50	0.25
Disintegrated Rock	140	31	1000	50

2.3.6 Stability Analysis Results

Summary results of the stability analyses that include both static and seismic loadings are presented in this section. The complete analysis is included in Attachment 4.

The global stability factors of safety for some of the existing condition scenarios for the EAP dikes did not pass the standard of care for permanent closure. The proposed design condition of the EAP addresses this low factor of safety concern and achieves passing short- and long-term factors of safety for the final closure configuration.

The proposed design condition incorporates improvements to ensure acceptable factors of safety for the EAP dike. These improvements include flattening the exterior slope of the EAP dike and specifying compacted CCR or soil within 15 feet of the EAP's southern perimeter drainage channel.(see Drawing GD-4B - Design – Geotechnical Exploration Plan (East Pond).

The NAP dikes meet acceptable factors of safety in their current and post-closure condition; therefore, no modifications or improvements are needed. CCR interior to the dike will, however, require improvement to ensure acceptable factors of safety against post-closure seismic liquefaction. Similar to the EAP, compacted CCR or soil is required within 15 feet of the perimeter drainage channels to maintain adequate factors of safety.

The WAP dikes, except for a short segment on the southeast corner, meet acceptable factors of safety. The short segment shows slightly steeper than 2H:1V slopes, which, if confirmed during closure activities, should be flattened to conform to the rest of the dike at 2H:1V to ensure acceptable short- and long-term factors of safety. The interior slopes of the dikes and a new dike to be constructed across the existing WAP will be constructed with 3H:1V slopes to ensure acceptable post-closure factors of safety. Table 2 below presents a summary of the Slope Stability results for existing and post-closure conditions for the three ponds.

Table 2: Summary of Slope Stability Results

		A	B	C	D	E	F
	Grading	Existing	Design	Design	Design	Design	Design
	Type	Steady-State	Steady-State	Steady-State	Undrained	Seismic	Seismic
	Water Level	Existing	Max	Design	Max	Max	Design
	Required FS	N / A	1.5	1.5	1.5	1.2	1.2
Figure	**Section**	**Factors of Safety - West Ash Pond**					
WP-3	WP-C, West, Right	N/A	1.7	N/A	1.8	1.2	N/A
WP-4	WP-C, East, Left	2.3	2.1	N/A	2.2	1.8	N/A
WP-5	WP-C, East, Right	1.6	1.6	N/A	1.6	1.4	N/A
WP-6	WP-E, South, Left	1.4	1.6	N/A	1.6	1.4	N/A
WP-7	WP-E, South, Right	1.2	2.1	N/A	2.2	1.8	N/A
WP-8	WP-E, North, Left	2.1	2.0	N/A	2.4	1.9	N/A
WP-9	WP-E, North, Right	1.7	1.7	N/A	1.7	1.5	N/A
Figure	**Section**	**Factors of Safety - East Ash Pond**					
EP-4	EP-B, South	1.2	1.9	2.0	1.8	1.2	1.2
EP-5	EP-B, North	N/A	1.9	2.1	2.0	1.3	1.3
EP-6	EP-D, South	1.3	1.7	2.0	2.0	1.3	1.3
EP-7	EP-D, North	2.4	1.8	2.1	2.1	1.7	1.7
EP-8	EP-G, West	1.2	1.5	1.8	1.8	1.5	1.5
EP-9	EP-G, East	1.3	1.5	1.8	1.9	1.4	1.4
EP-10	EP-H, South	1.3	1.8	2.1	2.1	1.7	1.7
EP-11	EP-I, South, Left	1.3	1.7	N/A	1.9	1.6	N/A
EP-12	EP-I, South, Right	2.1	1.7	N/A	2.0	1.6	N/A
Figure	**Section**	**Factors of Safety - North Ash Pond**					
NP-2	NP-B, South, Left	1.6	1.6	1.6	1.6	1.3	1.3
NP-3	NP-B, South, Right	2.1	2.2	2.2	2.2	1.7	1.7
NP-4	NP-H, East	1.7	2.1	2.0	2.0	1.7	1.7

2.4 Settlement Analysis

Settlement of the ash ponds was assessed considering two approaches: a relatively conservative method using one-dimensional consolidation based on laboratory data, and an elastic model based on shear wave velocity. The latter method, based on experience, will provide a better prediction of anticipated settlement. The complete analysis is provided in Attachment 5. Drawings GD-6A through 9A present isopachs of settlements derived from the available subsurface data using these two approaches for the EAP and NAP.

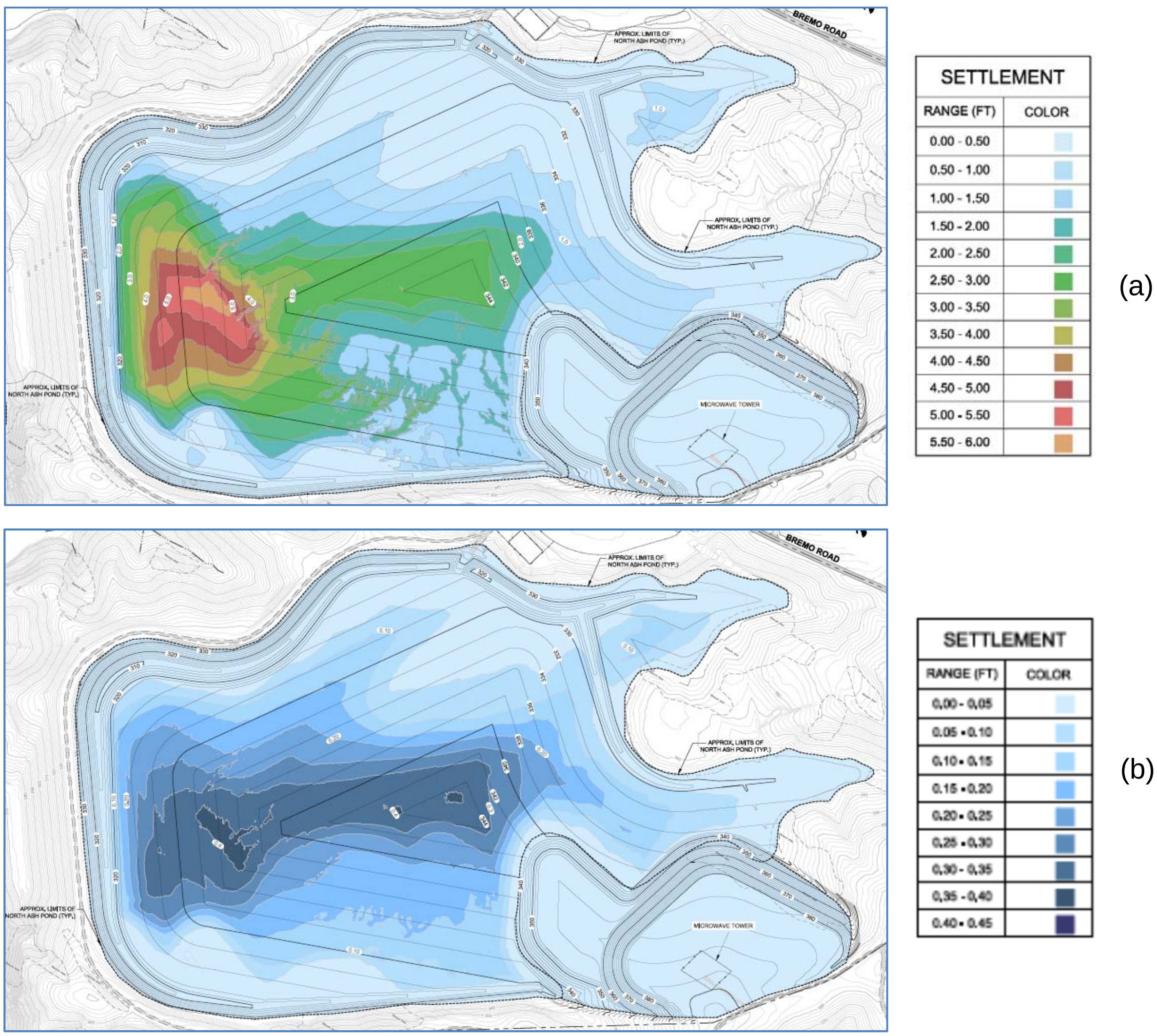

Figure 4: Example of results from (a) conservative and (b) predicted settlement approaches.

Drawings GD-6B through -9B present potential effects of the settlement on the proposed design grades. The consolidation approach is termed “Conservative” and the elastic approach termed “Predicted” to differentiate the approaches and methods used. Figure 4 above illustrates results from the conservative and predicted approaches. The WAP was not included in the analysis since the ash is planned to be removed. Note that there are variations in the ash between the exploratory locations as well as variations in the thickness and bottom elevations of the ash that can impact these settlement analyses.

2.4.1 Consolidation Approach (Conservative)

A common method of settlement analysis is to collect relatively undisturbed samples in the field and test them in a one-dimensional settlement mode in accordance with ASTM D2435. Because the microstructure and weak cementation properties of ash are very difficult to preserve through the process of sampling, transport, and lab preparation, the results generally show a greater compressibility than that observed in the field. Therefore, Golder considers the consolidation approach to be a conservative estimate of potential settlement.

2.4.2 Elastic Approach (Predicted)

The predicted settlement analysis models the settlement of the ash considering the known light cementation properties that exists within the ash using elastic theory. Elastic theory was applied using properties obtained from *in situ* (CPT) investigation and was applied to calculate predicted settlements at each discrete location within the pond. The results generally show more realistic compressibility to that observed in the field. Therefore, Golder considers the predictive approach a more appropriate method to estimate the potential settlement.

2.4.3 Loading Conditions

The closure is expected to change vertical stresses in the ash and underlying soils due to grading of the ash and placement of the cover soils. During construction, ash will be graded to develop the final drainage patterns of the final cover system. This grading will involve cutting and filling the ash over the NAP and EAP. The ash surface will be capped with at least 2 feet of soil, and in some locations, with drainage channel armoring materials (i.e., stone gabions). The density of the excavated ash will vary, but partially saturated densities will likely average about 90 pcf at removal. Compacted densities (also partially saturated) will be on the order of 110 pcf for ash and 125 pcf for the cover soil and protective armoring.

During grading operations, ash will be dewatered to at least several feet below the planned grades to facilitate access and compaction. Dewatering will increase the vertical stress on the materials below the initial water level by 62.4 pcf, the weight of water, for each foot the groundwater is lowered due to the loss of buoyancy. For example, if the groundwater is lowered by 10 feet, the vertical stress on the ash and underlying soils below the current water level will be increased by 62.4 psf for the first foot up to 624 psf for the tenth foot and all materials below that.

Currently, stormwater flows into the ash ponds, where it can infiltrate into the ash. The water level in the ponds is controlled by the elevation of the outlet structures, which are near the crest elevations of the dikes. Due to the seepage gradient in the ash, water levels away from the outlet structures are somewhat higher. The ash surface on average is generally more permeable than the average permeability of the pond bottom soils. Stormwater inflows generally exceed losses due to seepage into the underlying formations, keeping the ash saturated to above the free water levels in the pond. Closure will involve redirecting the stormwater over the capped surface, which will substantially reduce infiltration. This drainage control will allow the seepage losses in the bottom and sides of the dikes of the NAP and EAP to exceed the infiltration rate through the cap, resulting in an eventual decrease in the saturated water level in the ponds.

2.4.4 Method

The proposed final loading on the NAP and EAP was calculated based on a grid of points across each pond. The settlement at each of those grid points was then calculated along the surface based on each of the two approaches (conservative and predictive) with the changes in vertical stress due to grading, dewatering, and parameters derived from the exploration. The analysis at each point is one-dimensional and assumes infinite area loading in all directions.

The difference between the existing grade and proposed final grade at each point provided the height for calculating the proposed change in loading. The loading was assumed to be single stage and immediate. Drawings GD-6 through GD-9 present contours of calculated total settlement over the NAP and EAP.

2.4.5 Settlement Results

Settlement of the ash was considered with respect to potential impacts of closure. Settlement is expected to occur due to additional load resulting from grading of ash and final cover placement during closure activities, as well as drainage of the ash during and after closure. Laboratory test results and conventional consolidation theory show settlements up to several feet (up to 6 feet for portions of the NAP and up to 3.5 feet for portions of the EAP). The grading design has taken these settlements into account, such that adverse slopes should not occur on the final cover or in drainage ditches in the event such settlement occurs. The anticipated settlement amounts are not expected to inhibit the proper functioning of the proposed final cover or stormwater conveyance systems. The complete analysis of predicted settlement is found in Attachment 5 - Settlement Analysis. Results are outlined in Drawings GD-6 through -9 and discussed in the conclusions section below.

2.5 Seepage

Preferential seepage paths through the EAP dike include interfaces of fill layers, tree roots, animal burrows, and thin layers of coarser or higher permeability soils in the dike fill and underlying alluvium. Exploration could not practically detect or define all such features, but the large variation in water level in borings along the dike indicate that such features are present and more significant to overall seepage than flow through the mass of dike fill. Planned reworking of the dikes should substantially reduce such preferential pathways and produce lower and more uniform seepage conditions.

The NAP dike is more homogeneous than the EAP dike, but preferential seepage paths occur through the disintegrated rock and rock in the NAP bottom and abutments of the dike, as indicated by controlled seepage drains in the abutments and variation in water levels in wells screened through the dike.

The NAP dike is provided with seepage controls (Figure 5: Outlet of NAP Seepage Drains), in the form of blanket drains that protect the embankment fill from seepage erosion and soil piping.

Planned improvements to the EAP dikes will include construction of similar controls through the installation of a toe drain to collect potential seepage. Post-closure water levels in the ash contained in both ponds will decrease due to the reduction of infiltration, which will reduce seepage pressures through the dikes.

Figure 5: Outlet of NAP Seepage Drains

2.6 Veneer Stability

Stability of the proposed cap over the planned ash and fill slopes was evaluated to assess the veneer stability of the final cover materials including the geomembrane, drainage composite, and cover soil placed over CCR and fill materials. Results are included in Attachment 6 and demonstrate that adequate factors of safety are achieved with the proposed final cover design.

3.0 CONCLUSIONS AND RECOMMENDATIONS

Based on Golder's review of historical documentation, completed field explorations, engineering analysis, and understanding of regulatory requirements, the following design recommendations for the closure of the NAP, EAP, and WAP are presented:

3.1 Data Confirmation

Through additional field investigations performed subsequent to those completed as part of the Design Geotechnical Data Report (Attachment 1), it was concluded that those prior investigations were representative of conditions where access and schedule constraints did not allow data to be collected initially. These areas that were confirmed through subsequent field investigations include:

- West soil dike of the EAP near the new gas line and coal pile cover
- Southwest soil dike of the EAP near the railroad spur and wooden piles
- Toe of the south dike of the EAP due to the railroad right of way
- Northwest soil dikes of the EAP due to the gas line
- Splitter dike buried in the EAP's ring dike expansion

- Proposed spillway location at the southwest corner of the NAP
- Southeast and east portions of the NAP
- North and east sides of stacked ash near the microwave tower in the northeast corner of the NAP
- New west dike footprint in the WAP/West Treatment Pond

3.2 Liquefaction

It has been concluded that potential risks resulting from liquefaction will decrease after closure as the water level within the ash decreases. This decline in water levels will also enhance dike stability.

3.3 Stability

It has been concluded that portions of the EAP dikes require improvements as part of the closure activities. Improvements have been incorporated into the closure design of the EAP to mitigate stability risks and provide adequate factors of safety. It has further been concluded that the WAP and NAP dikes do not require dike improvements, and have been shown to have adequate factors of safety.

3.3.1.1 East Ash Pond

As previously mentioned, the existing EAP dike does not provide adequate long-term stability without re-grading and reshaping. To improve stability, the design has incorporated flattening the perimeter dike slopes to a maximum of 2.5H:1V.No ash will remain beneath the downstream slope or crest of the re-graded dike. Slope flattening will be accomplished through a combination of extending the toe of the dike with a buttress of compacted soil fill and cutting into the existing dike at the crest. Some challenges to be considered during buttressing and cutting activities are as follows:

- Surface Water Controls - Free water shall be drained from the pond prior to re-grading activities. Stormwater handling and diversion measures shall be installed prior to re-grading activities to prevent refilling of the pond during construction.
- Water Level and Saturated Ash Controls - Maintaining temporary stability during the re-grading activities will require lowering the water level in the ash. Water levels should be lowered to at least 15 feet below the lowest dike crest elevation and/or areas of ash re-grading. Ash dewatering through surface ditching should be sloped no steeper than 4H:1V. If ash is dewatered using well points or larger

wells, steeper slopes may be adequate based on field conditions. Measures shall be in place to contain ash from exposed surfaces prior to construction.

- Dike Excavation – Construction activities may encounter soft, wet, or deleterious zones within the existing dike that will need to be removed or stabilized in order to receive new fill. Should such zones require removal by additional excavation, the ash and water level behind the dike area should be lowered to maintain a separation of 5 feet.
- Dike Re-grading – Re-grading activities may encounter ash or come within 10 feet of the impounded ash. It is recommended that a minimum of 10 feet of compacted soil is placed between ash and finished dike surface slopes where practical. This may require over-excavation of ash and replacement with soil fill behind the existing dike in some areas.

Closure of the EAP, including capping and stormwater controls, will substantially reduce infiltration into the ash and allow the water level in the impounded ash to lower significantly over time after closure. This reduction of water levels in the dikes will improve stability factors of safety as compared to conditions considered in our analysis.

3.3.1.2 West Ash Pond

The WAP dikes present acceptable factors of safety for stability and show no evidence of instability in their current use. The WAP will no longer contain CCR in the post-closure plans, so risks of CCR movement are eliminated. The stability of exterior slopes of the existing dikes to remain will be acceptable, with some limited areas in need of repair to re-establish the original design grades.

Interior slopes of the new and existing dikes and exterior slopes of the new dike will be constructed on the bottom of the former WAP borrow pit. The condition of these foundation soils has not been directly investigated due to access limitations. Bottom soils may have been disturbed in subsequent grading and maintenance dredging efforts. Therefore, it is anticipated that these bottom soils will be in a highly disturbed and weakened state to some depth below the pond bottom. The fine-grained alluvial soils may be difficult to dewater, especially as natural groundwater levels associated with the James River will likely be near the surface. Rim trenching and sumps will take much longer to dry the fine-grained alluvium than the ash and may prove to be ineffective.

For the general backfill of most of the WAP area, stability and settlement will not be of concern. In these areas, a bridge lift of 2-4 feet of relatively dry soil pushed out over the soft bottom will "bridge" soft zones. Compaction of fill should be limited until well above the water level, to avoid saturation and softening of the bridge lift. In any case, limited compaction in this area should be acceptable to restore grades, though some settlement may occur within a few months after completion. At that time, additional fine grading to restore positive grading where local depressions develop may be required, but should be limited.

For support of dikes and slopes, bridge lifts may be acceptable and foundation soils should improve under the planned dike loads. However, more aggressive improvement, such as stabilization by various methods (such as lime stabilization), may be required if soils have degraded such that they impact stability. Over-excavation to more competent material is preferred, but may be difficult due to shallow groundwater. It is recommended that the foundation of new proposed dikes be inspected by the design engineer prior to construction should soft soils be present that do not pass proof rolling.

3.3.1.3 North Ash Pond

The NAP dike in its current configuration presents acceptable factors of safety for stability, and shows no evidence of past instability. As with the EAP, closure of the NAP will allow gradual reduction in water levels, which will result in gradual improvement in stability factors of safety compared to conditions considered in our analysis.

NAP dike stability could be impacted by excavation in the EAP near the toe of the NAP dike. It is recommended that no excavation be completed beyond a line extending outward and downward at a 4H:1V slope from the north bank of the existing ditch at the NAP dike toe without detailed analysis to determine if additional shoring or other measures are required. If additional excavation is necessary in this area beyond the recommended 4H:1V projection, the design engineer should be consulted prior to any further excavation activities in this area.

3.3.2 Spillway Capacity

Spillway capacity is critical to protecting earth dikes from overtopping, which can lead to rapid erosion, breaching, and failure. Spillway hydraulic and civil design is provided in other reports and analysis related to the closure design. Spillways should be designed with sufficient capacity and maintenance controls to not cause the erosion and loss of containment dikes that could subsequently impact stability.

3.3.3 Seepage Controls

As previously discussed, the existing seepage controls for the NAP are adequate and shall be maintained during and after the closure activities. The WAP will be clean-closed of CCR material and the majority of the dike removed. The remaining portion of the existing dike and the new proposed dike necessary for the new West Treatment Pond will not require seepage controls because the new pond will lined with a geomembrane.

Seepage controls are recommended and provided for in the closure design the EAP dike. These proposed controls shall be maintained after the closure activities are complete. Seepage control improvement to the EAP dike should be in the form of blanket, rake, and/or toe drains to maintain and control seepage above the downstream toe of the embankment. This seepage control will limit the risk of piping failures and softening of the embankment surface that could make maintenance difficult and/or increase erosion and stability concerns.

Drains may consist of man-made geocomposites or filtered gravel. Principal elements of these drains should include:

- A small diameter (typically 4 to 6 inches) perforated pipe to collect and convey flow longitudinally along the drain. Typically, the drain and pipe will slope at least ½ percent for drainage.
- A zone of open-graded gravel, geonet, or other high void and incompressible material to collect water from the overlying soils and convey it to a longitudinal pipe.

A filter, either geotextile or properly designed graded aggregate or sand, is recommended to allow water into the drain but prevent migration of soils. Non-woven geotextiles are recommended and should be selected considering survivability during installation and compatibility with the overlying soils with respect to filtering.

3.4 Grading

Construction grading and re-grading activities will entail a number of construction operations that should be monitored for signs of instability. These operations include:

- Ash accessibility
- Ash dewatering
- Excavation

- Re-working existing ash materials
- Fill placement
- Work on slopes

3.4.1 Ash Accessibility

Since ash is both fine-grained and non-cohesive, it is subject to bearing capacity failure when loaded while saturated. Equipment may be at hazard operating on ash that is saturated at or near the surface. When the saturation or water level in the ash is lowered to about 3 feet or more below the surface, low pressure (lightweight tracked) equipment should be able to access the ash surface to begin grading activities. Continuous lowering of the water level will be required to access the ash with larger equipment. It is recommended that water levels be lowered to a least 15 feet below the working surface to ensure stable access with large equipment.

Accessibility conditions may vary with weather and changes in water level, or local variations in the ash such that, even after initial lowering, problems may develop if water levels are allowed to rise or seepage is encountered from adjacent ash fills. Therefore, drainage measures will likely need to be operated continuously during the construction, and procedures to evaluate the ash stability should be employed following rainfall or shutdown of drainage measures. Such measures may include shallow hand auger holes to assess water levels periodically, and use of low-ground-pressure equipment with an experienced operator to traverse areas prior to accessing these areas with heavier equipment. Operator training should include awareness of potential ash instability, and measures to recover bogged equipment should be in place.

Special care is required near ash slopes, as traffic can create liquefaction of saturated ash that could lead to slope failures and hazards to equipment and workers. Generally, only ditching equipment with experienced operators trained to recognize the potential for slope failures and taking measures to avoid rollover or other hazards should be used to excavate ash dewatering ditches. Construction traffic should stay away from dewatering ditch edges.

3.4.2 Ash Dewatering

Ash is similar to natural silts, with moderate permeability. Experience indicates dewatering of ash is feasible with conventional methods such as rim ditching, although well points may have local application where ditching equipment cannot initially access the ash surface. Dewatering volumes are expected to produce about 20 percent of the total saturated ash volume, plus any additional infiltration that occurs during the time the ash and dewatering system is exposed. Dewatering shall follow the progression of grading activities to provide a stable foundation on which to work.

3.4.3 Excavation

Excavation related to the pond closures should generally be accomplished with conventional earth moving equipment. Difficulties may arise with ash and saturated embankment fills and weak foundation materials, as well as in areas where disintegrated rock and rock are encountered, such as in and around dike abutments.

3.4.3.1 Ash

Excavation in ash materials should generally be accomplished with conventional excavators. Pan scrapers and loaders are more likely to encounter access difficulties over newly dewatered areas. Recently dewatered ash will remain wet and difficult to work and transport for some time after lowering the water level. Additional handling and/or draining may be required. Working with ash during winter months or periods of wet weather may be challenging and impractical without additional stabilization measures such as lime addition.

3.4.3.2 Existing Embankment Fill and Foundation Soils

Fill materials in the embankment and foundations soils should be readily excavated with conventional equipment, though some larger boulders or debris may require special handling if encountered. Excavated embankment materials, especially those below the water level, are likely to be excessively wet and require special handling and drying prior to re-use. Soils will generally be more difficult to dry than ash.

3.4.3.3 Abutment Materials

Excavation for the NAP spillway through the right (west) abutment may encounter disintegrated rock and rock, based on observations of rock in the abutment. Further, the steep abutment will require deep excavation and produce tall excavation slopes. Rock and disintegrated rock in Piedmont profiles is irregular, and the excavation may encounter inter-bedding of rock and soil or adjacent areas of rock and soil at the spillway channel bottom elevation. Rock excavation, including ripping and blasting, is likely to be required.

Ripping typically requires single-tooth rippers pulled by large crawler tractors (Cat D-8 or larger) or powerful track hoes (Cat 326 or larger). Ripped disintegrated rock can typically be pulverized into soil by repeated trafficking with heavy equipment and re-used as structural fill. Ripping may take advantage of fracture patterns in hard rock, producing large, jagged boulders that would require special crushing. Rock crushing operations for the volumes anticipated are unlikely to be economic. The boulder-sized rock pieces may be usable as rip-rap or may need to be wasted either on-site or off-site.

Rip-rock excavation is typically marginally more costly than earth excavation, but blast-rock excavation is typically substantially more costly. Therefore, careful measurement of blast-rock quantities is important to limiting costs. We recommend ripping all possible material to expose and allow inspection and measurement of the blast-rock prior to resorting to blasting. Blast mats or loose fill shall be placed over the blast-rock to control fly-rock.

3.4.4 Reworking Existing Ash Materials

Most of the existing ash in the EAP and NAP may remain in place under new grades with no need for disturbance, excavation, or dewatering. However, at ditches in the NAP and EAP (see Drawings GD-4B and -5B), existing ash may not provide acceptable factors of safety for seismic stability and liquefaction, and will need to be reworked within 15 feet of proposed cap slopes (see Figure 6 below). Reworking will entail:

- Dewatering - Ash will need to be dewatered to a depth of at least 3 feet below planned excavation grades to allow low-ground-pressure equipment access, and up to 15 feet to allow access larger equipment.
- Excavation - The zone(s) identified for re-compaction will need to be excavated. Excavated ash may be used elsewhere or staged to be reused in the same location. If the same ash is reused, some shrinkage due to compaction will likely occur and additional material may be required. Existing dike fill (not underlain by additional ash), undisturbed residual soils, disintegrated rock, or rock will not require excavation unless in a weak or soft condition.
- Subgrade preparation - The exposed bottom of excavations should be observed by the design engineer and stabilized as necessary to allow fill placement.
- Replacement – The excavated volume will need to be replaced with compacted ash or soil meeting embankment structural fill requirements and benched into excavated slopes to mitigate veneer failures.

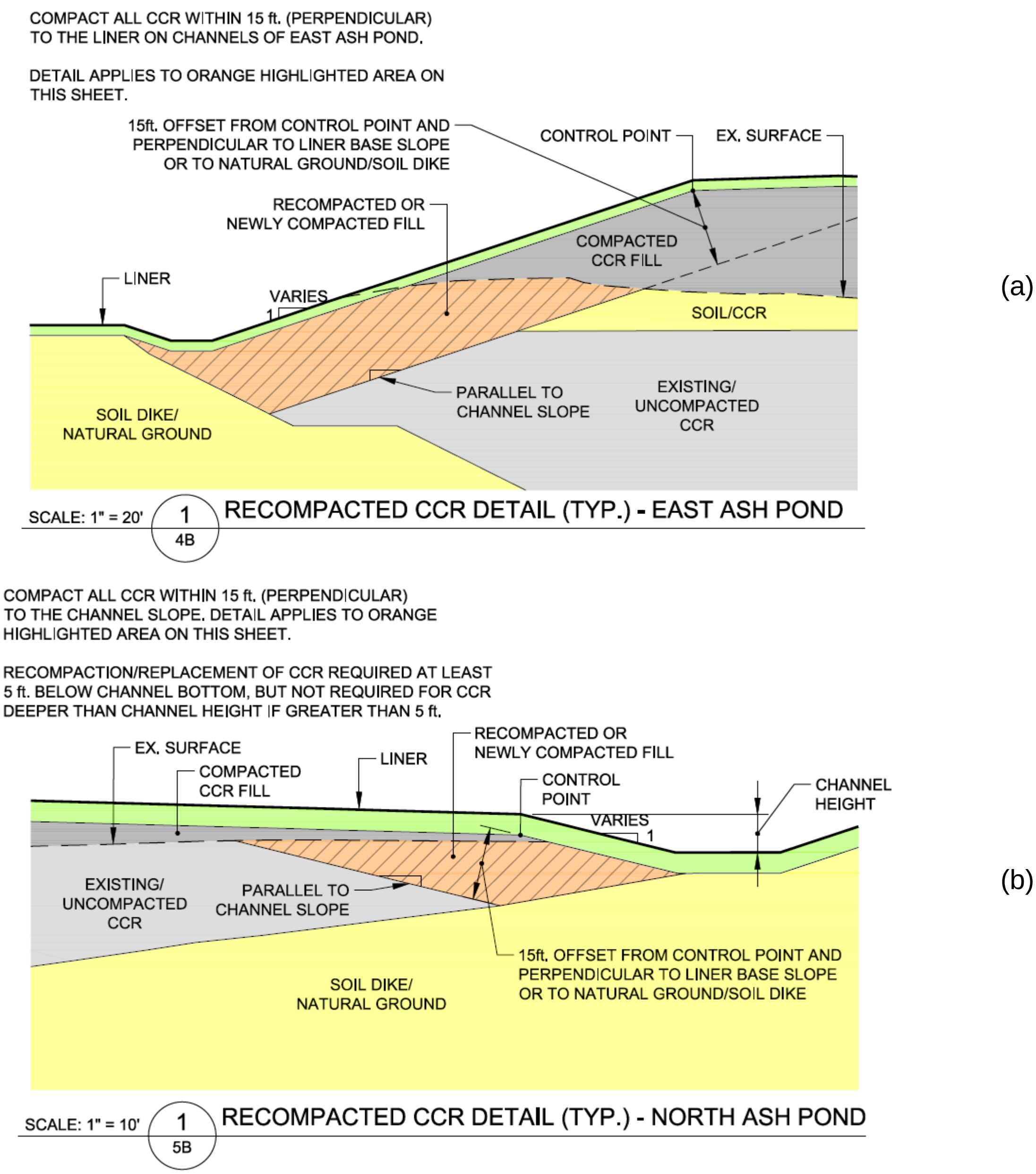

Figure 6: Re-compaction / Replacement Detail for (a) EAP slopes and (b) NAP Slopes

3.4.5 Fill Placement

The following fill placement recommendations (for both ash and soil fills) should be followed during construction.

3.4.5.1 Subgrade Preparation

After excavation, areas to receive fill should be observed by the engineer for stability. Stabilization, which may entail dewatering, proof rolling with compaction equipment or other methods, undercutting and replacement, or use of stabilization stone or geotextiles, may be required to provide a surface acceptable for fill placement. Fill compaction is not practical on an unstable surface. Bridging with thick lifts of fill can impact stability and settlement, and is generally not acceptable for critical areas of construction.

3.4.5.2 Fill Placement On and In Slopes

Fill placed on slopes steeper than 6H:1V should be benched into the slope to reduce the potential for sliding along the former slope surface. Figure 7, below, shows a general benching concept. Immediately prior to fill placement, a near vertical cut up to 4 feet high should be made in the existing slope. Depending on the existing slope, such a cut will typically require excavation of more than 4 feet horizontally into the slope. This cut will provide a horizontal "bench" for fill placement.

Fill lifts shall be placed and compacted in accordance with the construction specifications on the horizontal surface produced until the top of the vertical cut is reached. At that point, the process should be repeated for the next bench. If excavated material from the bench is acceptable for use as fill, it may be incorporated into the fill placement.

Acceptable compaction is difficult to achieve on sloping surfaces or near the slope surface due to lack of confinement. Therefore, when creating a fill slope, fill lifts should generally extend beyond the planned fill slope such that acceptable compaction is achieved at the planned slope and the edge of the fill lift, which may not be acceptably compacted if beyond the planned fill (overfill). This excess fill material may then be graded off to leave the planned fill slope.

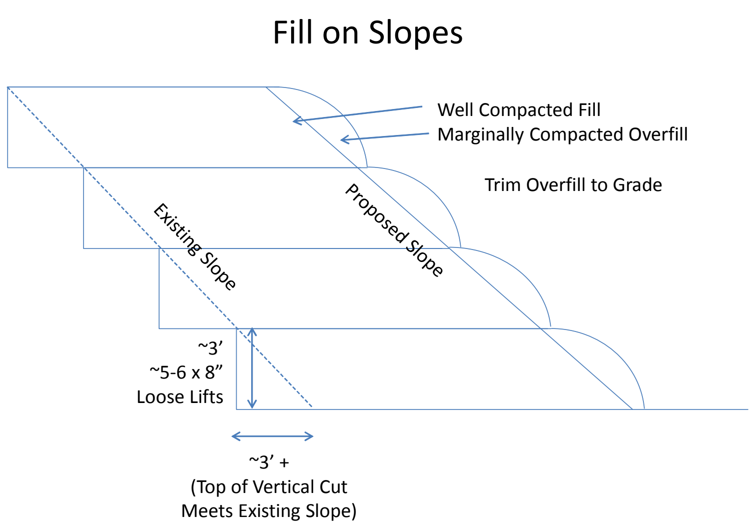

Figure 7: Schematic of recommended slope fill benching and edge compaction

3.4.5.3 Fill Materials

Fill materials should be low to moderate plasticity soil (PI less than 30), generally free of organic matter and other deleterious materials and rock fragments larger than 2 inches in any dimension. Except for organic soils, blasted rock and hard rock boulders, most site materials, including ash, residual soils, pulverized disintegrated rock, and alluvial soils, should meet these requirements, though some raking to remove larger rocks, roots, and stumps may be required.

Soil fill and ash should not be mixed, nor should lifts of soil fill be placed between lifts of ash fill in grading. Such mixing/inter-bedding could have negative effects on drainage and produce perched water conditions. If soil fill is placed over the ash at any point, soil fill should be continued to the design base grades of the final cover system.

3.4.5.4 Compaction

Fill soils and ash excavated and replaced should be compacted to the following minimum specifications:

Table 3: Minimum Compaction and Moisture Requirements

Material Type / Use	Compaction Requirement	Moisture Content
Structural Fill / Liner Subgrade	90%	-2% to +4%
Structural Fill / Road Subgrade	95%	-2% to +2%
Structural Fill / Trench Backfill & Stockpile	90%	-2% to +4%
CCR / Liner Subgrade	90%	-4% to +4%
Protective Cover Soil	90%	-2% to +4%
Structural Fill / Embankments	95%	-2% to +4%
Structural Fill / Foundations	95%	-2% to +4%
Structural Fill / All other uses	95%	-2% to +4%
Vegetative Soil / Final Cover	Do not compact	n/a

Fill lifts shall be placed and compacted in accordance with the construction specifications, raked as necessary to remove larger rocks and debris, moisture conditioned, and compacted with large pad foot or sheep's foot rollers. Vibratory rollers can be effective on well drained ash, but should be used with caution, as they may cause water to move up into the ash and soften the ash surface, or cause liquefaction of the material.

3.4.5.5 Protection of Work

Fill (either soil or ash) will be adversely impacted by excessive wetting. Positive site drainage should be maintained at all times during grading. Excavations that cannot be drained by gravity should be pumped dry expeditiously, using sumps and pumps or other methods, to minimize infiltration. Disturbed surfaces should be rolled smooth at the end of each work shift to encourage runoff. Moisture control, including drying following wet weather and wetting in dry weather, may be required. Fill placement during periods of wet weather, especially during winter months, will be difficult slow drying times. Trafficking over wetted surfaces may degrade previously compacted materials.

3.4.6 Work on Slopes

Work on steep slopes shall be performed with caution, with slopes not being graded steeper than the following the minimum recommendations:

Table 4: Maximum Slope Steepness Requirements

Condition	Allowable Temporary Slopes	Allowable Permanent Slope
Existing Fill, Well Drained[3]	2H:1V	2.5H:1V
Compacted Soil Fill, Well Drained, In Dikes (not over ash)	2H:1V	2.5H:1V
Residual Soil, Well Drained	2H:1V	2.5H:1V
Disintegrated Rock/Rock	2H:1V1	2.5H:1V[1]
Ash, Well Drained, Sluiced Placement	3H:1V	N/A[2]
Ash, Well Drained, Compacted	3H:1V	3H:1V

NOTES

[1] Steeper slopes may be acceptable in rock or disintegrated rock. The design engineer should observe materials exposed to confirm the material type and assess whether weaknesses (joints, fractures, seams, etc.) in adverse orientations, groundwater conditions, or other conditions that might reduce stability are present.

[2] In no case should sluiced ash be left at surface (below cover) in permanent slopes. Ash should be reworked near the slope surface to promote stability.

[3] Drainage is critical to stability of slopes in soil and ash. In no case should water be allowed to seep through the face of temporary or permanent slopes.

3.5 Erosion Control

For earthwork activities, acceptable erosion control is typically accomplished by managing surface water flows with slopes and grading to produce sheet flow on relatively flat slopes, or for short distances on steeper slopes, with benches and armored channels or pipes to safely collect and convey concentrated flows. A thick turf of grass typically provides acceptable resistance to sheet flow and low velocity flow in the upper parts of ditches, and should be established and maintained over the cover and dikes. Natural Piedmont and alluvial soils will generally support such grassing with minor amendment. Temporary erosion control matting is recommended to facilitate establishment of grass on slopes. Use of topsoil or lower compaction targets may simplify establishing grass, but may erode on slopes before grass becomes established if not protected with mulch or other erosion control measures.

Armor for ditches and spillways should be designed considering the flow velocity, flow depth, slope, and curves or impingements into the flow. A variety of flexible and rigid armor methods is available and can be effective if properly designed, constructed, and maintained. Armor maintenance must be considered in design. Clearing limits should extend to 25 feet beyond the edges and toe of the proposed cover and dikes where practicable, and this area should be maintained with turf grass.

3.6 Dike and Cover Maintenance

Frequent and ongoing maintenance will be required to prevent degradation of the dikes and final cover systems. Typical maintenance activities that should be anticipated include:

1. <u>Mowing and Vegetation Control</u> - Allowing brush, trees, clumping of weeds, etc. on the dike, cover, ditches, channels, spillways, or slopes will concentrate flow and lead to the development of erosion rills. Mowing at least twice per growing season should maintain the turf grass and prevent growth of clumps and woody vegetation. Because mowing will likely be the most frequent maintenance activity, mower personnel or supervisors should be trained to observe the embankment, cover, and ditches for other needed maintenance.

2. <u>Turf Repair</u> - Expeditious repair of bare or sparse areas by irrigation, over-seeding, fertilizing, and/or placing erosion control matting will limit the formation of erosion rills that will require greater repair effort.

3. Erosion Repair - If erosion rills or gullies develop, they should be repaired by excavating soft or loosened soil and re-compacting soil fill. Repair of rills in areas intended for sheet flow by placing rip-rap or gravel is not recommended, as it precludes the reestablishment of grass and will often cause erosion rills to form beside or below the rock-filled rill.

4. Cleaning Ditches - Ditches should be kept free of sediment and other debris that may interfere with drainage.

5. Armor Repair - Armor in ditches, spillways, dike toes, etc. should be inspected periodically (at least annually) and after major storms (2-year or higher return period) for evidence of damage, movement, undermining, etc. and repaired expeditiously. Mowers may damage geosynthetic flexible armor, especially at changes in slope. Trees and woody vegetation can grow through rip-rap as well as joints in concrete or other armor, and should be removed at least annually to prevent regrowth.

6. Pipe and Structure Maintenance - Pipes on slopes have the potential to cause significant damage. Small leaks can rapidly undermine the pipe and cause failure, which can allow high-volume concentrated flow. Pipe systems should be monitored and inspected periodically for clogging, leaks, erosion around the pipes, movements, or other indications of problems, and be repaired expeditiously.

7. Minor Grading - Over time, minor grading may be required to repair areas where the cover has thinned from erosion or where rocks have been exposed on the surface. These areas shall be re-graded and seeded as soon as practicably possible.

8. Drains - Drain flows should be measured and recorded periodically, and drains should be cleaned out at least every 5 years. Records of drain flows should indicate a gradual reduction in flow for some years after closure before stabilizing to a new steady state. Drain flows should not be significantly impacted by short-term weather conditions, such as rainstorms, though long droughts or particularly wet periods of long duration may have noticeable impacts on flow. Rapid increases or decreases in flow likely indicate issues that need to be investigated and addressed by the engineer.

As part of a regular maintenance program, the engineer or a competent person under their direction should make thorough inspections of the cover and dikes at least annually to confirm continued performance. Issues such as seepage or wet areas, bulging, cracking, exposed geocomposite or geomembrane; damaged drainage structures or spillways, etc. should be investigated and repaired expeditiously

4.0 LIMITATIONS

The conclusions drawn and recommendations made in this report are based on the site characteristics described herein, the data obtained in Golder's field investigations, and Golder's experience with similar subsurface conditions, slope stability, liquefaction, and settlement analysis.

This report has been prepared for the exclusive use of Dominion, and applies to the subject site only, as depicted in this report. No other warranty, expressed or implicit, is made.

Drawings

DOMINION

BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
GEOTECHNICAL DESIGN

FORK UNION MAGISTERIAL DISTRICT
FLUVANNA COUNTY, VIRGINIA
NOVEMBER 2015

DRAWING INDEX

DRAWING NO.	DRAWING TITLE
1	GEOTECHNICAL DESIGN COVER SHEET
2	GENERAL SITE PLAN - DESIGN GRADES
3A	E□ISTING CONDITIONS - GEOTECHNICAL E□PLORATION PLAN (WEST POND)
3B	DESIGN - GEOTECHNICAL E□PLORATION PLAN (WEST POND)
4A	E□ISTING CONDITIONS - GEOTECHNICAL E□PLORATION PLAN (EAST POND)
4B	DESIGN - GEOTECHNICAL E□PLORATION PLAN (EAST POND)
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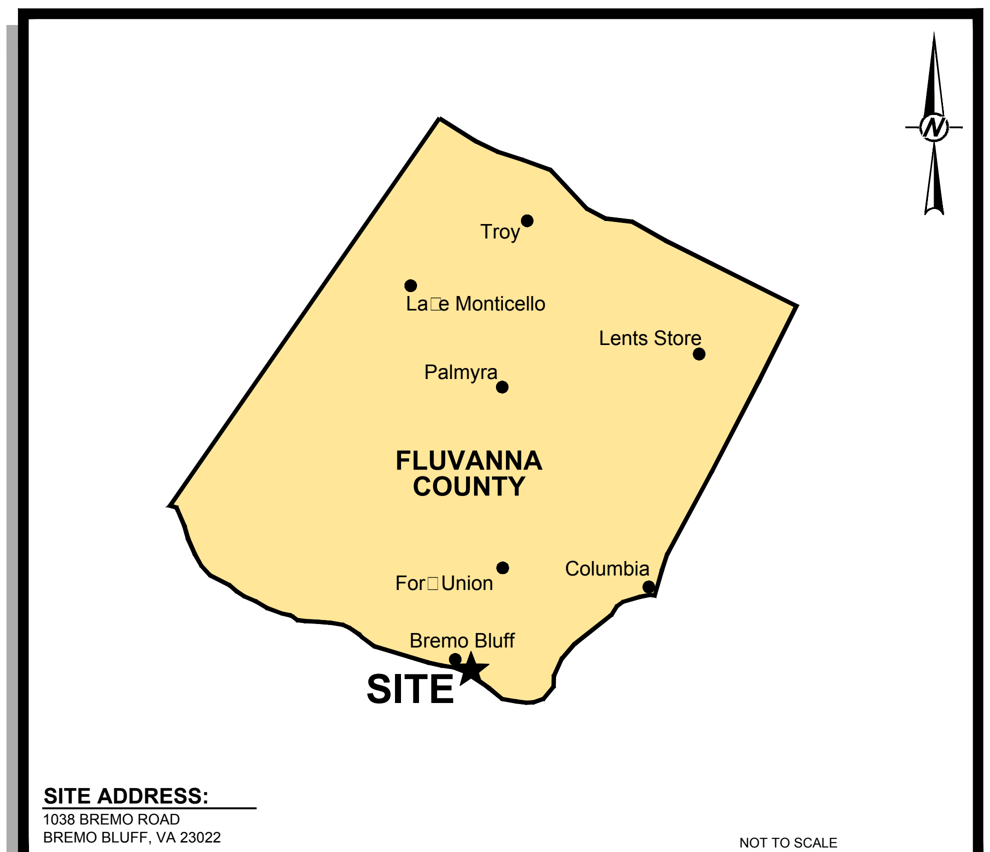
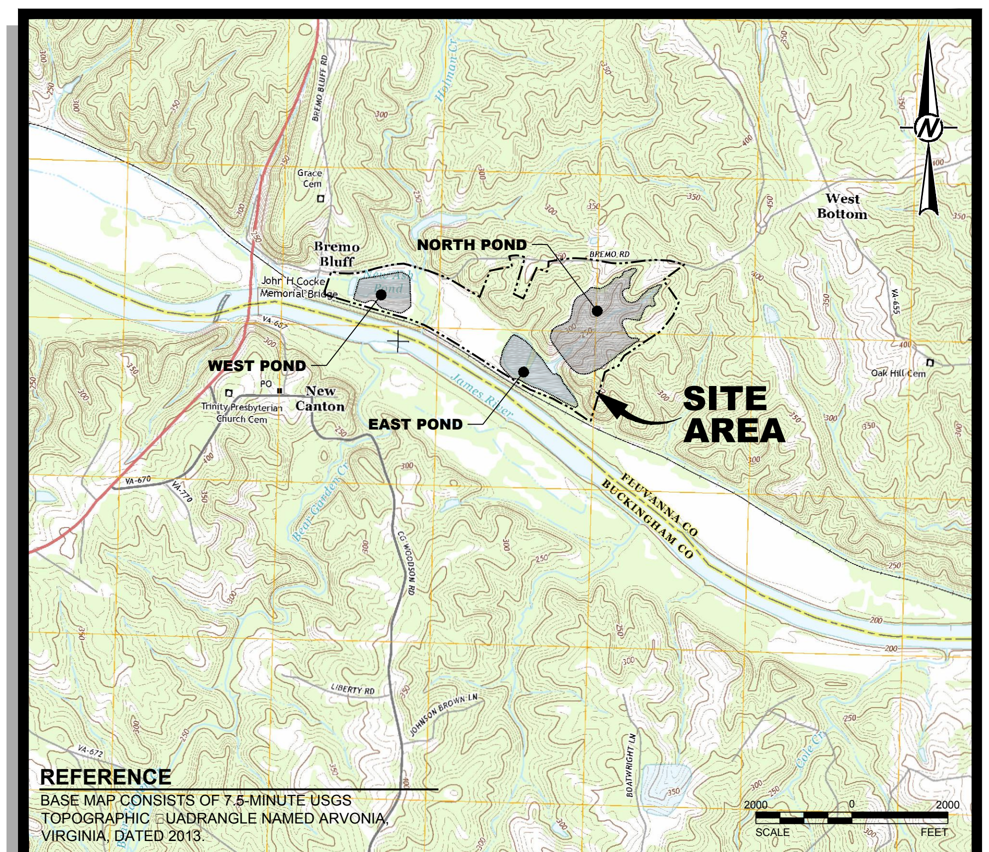
□A	PREDICTED SETTLEMENT ISOPACHS (EAST POND)
□B	PREDICTED POST SETTLEMENT SURFACE (EAST POND)
7A	CONSERVATIVE SETTLEMENT ISOPACHS (EAST POND)
7B	CONSERVATIVE POST SETTLEMENT SURFACE (EAST POND)
8A	PREDICTED SETTLEMENT ISOPACHS (NORTH POND)
8B	PREDICTED POST SETTLEMENT SURFACE (NORTH POND)
9A	CONSERVATIVE SETTLEMENT ISOPACHS (NORTH POND)
9B	CONSERVATIVE POST SETTLEMENT SURFACE (NORTH POND)



VICINITY MAP



SITE LOCATION MAP

GENERAL NOTES

1. E□ISTING CONDITIONS COMPILED FROM:
   a. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKEN□IE SNYDER, INC., DATE OF AERIAL PHOTO: 1/1□/15 □CONTROL PREPARED BY H□B SURVEYING □ MAPPING (H□B)□
   b. BOUNDARY SURVEY PREPARED BY H□B SURVEYING AND MAPPING, LLC DATED 04/27/15.
   c. BATHYMETRIC SURVEYS PREPARED BY H□B, SURVEYS PERFORMED IN FEBRUARY 2015
   d. HISTORICAL DATA FOR THE DEVELOPMENT OF THE WEST POND, EAST POND AND NORTH POND
   e. USGS □UADRANGLE TOPOGRAPHY FROM 19□7.
2. SITE DATUM: NAD83 / NAVD88
3. GOLDER GEOTECHNICAL E□PLORATIONS FROM AUGUST 2014 (GB-1 TO GB-5) AND MARCH 2015.
4. E□PLORATION DATA BY OTHERS WAS REVIEWED AND ARE SHOWN WHEN APPLICABLE:
   a. D'APPLONIA CONSULTING ENGINEERS, INC (1981) / SHORT-TERM WASTE DISPOSAL FACILITY REPORT.
   b. SCHNABEL (1982) / NORTH ASH POND DAM DESIGN REPORT.
   c. SCHNABEL (2010) / STABILITY EVALUATION OF WEST ASH POND DIKES.
   d. GROUNDWATER AND ENVIRONMENTAL SERVICES, INC. (GES) (2013) / MONITORING WELL INSTALLATION REPORT.
   e. HALEY □ ALDRICH, INC. (2015) / MONITORING WELL INSTALLATION REPORT.
5. DRAWINGS AND CALCULATIONS BASED ON PROPOSED 90% DESIGN GRADES BY GOLDER DATED SEPTEMBER 2015.

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PREPARED BY:

Golder Associates

PREPARED FOR:

Dominion

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
GEOTECHNICAL DESIGN COVER SHEET

PROJECT No.		15-20347	FILE No.	1520347AD01
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CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		

Golder Associates

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE

GENERAL SITE PLAN DESIGN GRADES

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD02-05B
DESIGN	-	-	SCALE	AS SHOWN
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CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		

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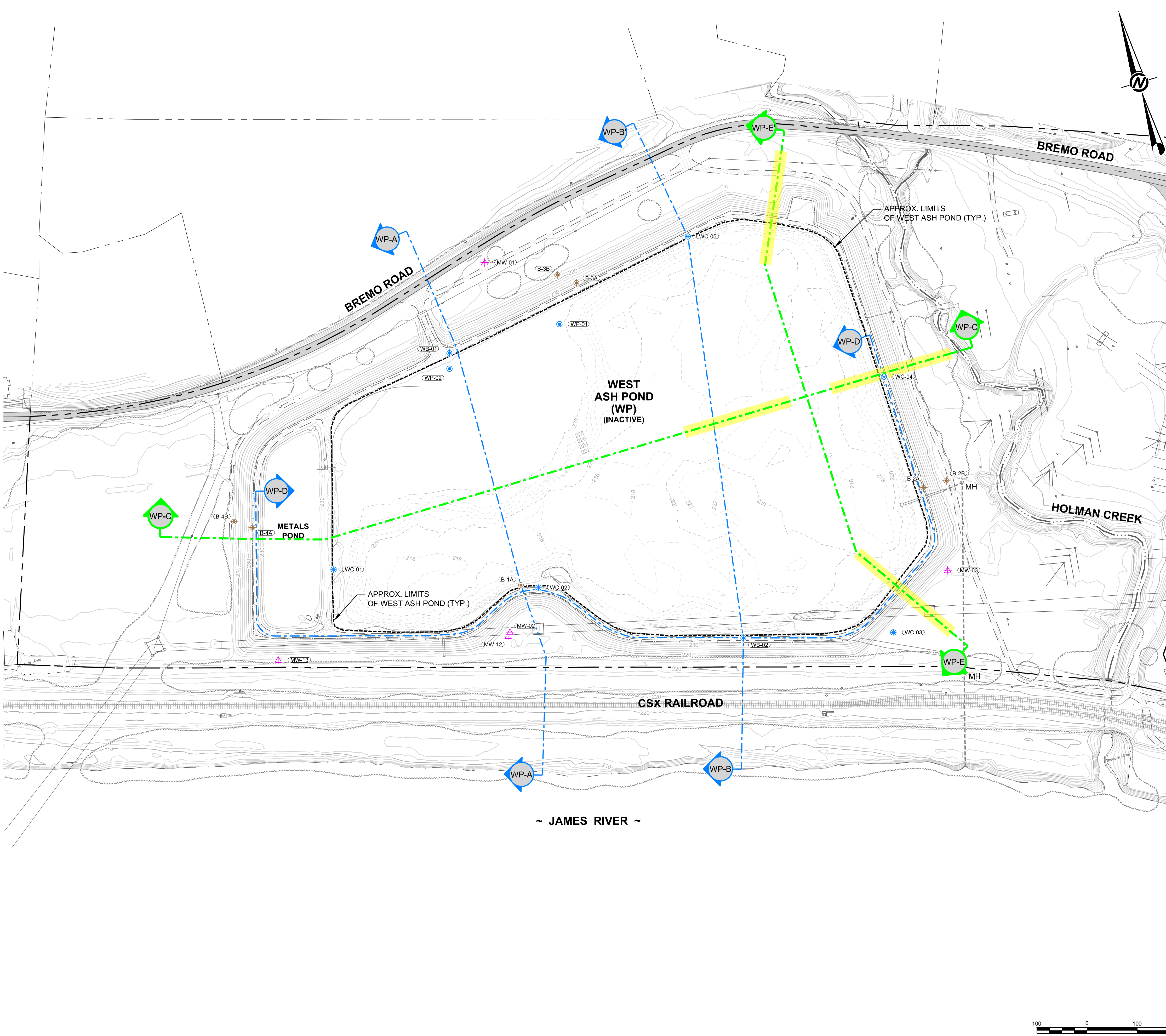

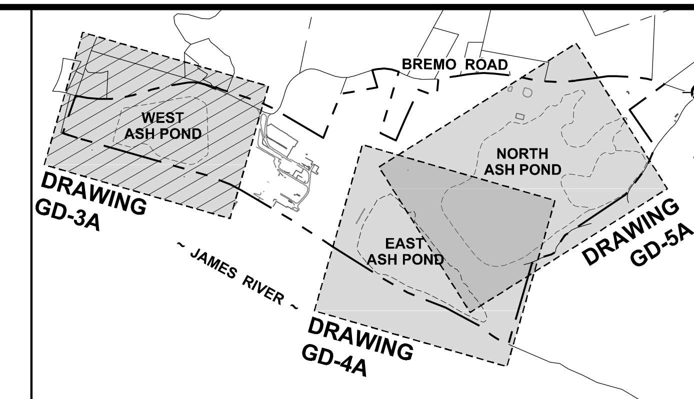

SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- EXISTING BATHYMETRIC SURFACE CONTOURS (2' INTERVALS)
- DESIGN SURFACE CONTOURS (2' INTERVALS)
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- GOLDER CONE PENETRATION TEST (CPT)(2015)
- GOLDER PROBE HOLE (2015)
- GOLDER HAND AUGER (2015)
- EP-A GEOTECHNICAL SECTIONS (SEE GOLDER 2015 GEOTECH DATA REPORT)
- EP-B SLOPE STABILITY SECTIONS
- SLOPE STABILITY DRAWING LOCATIONS - SEE APPENDIX D

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].

2. BATHYMETRIC SURVEYS PREPARED BY H&B, SURVEYS PERFORMED IN FEBRUARY 2015.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
EXISTING CONDITIONS GEOTECHNICAL EXPLORATION PLAN (WEST POND)

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD03A-05A
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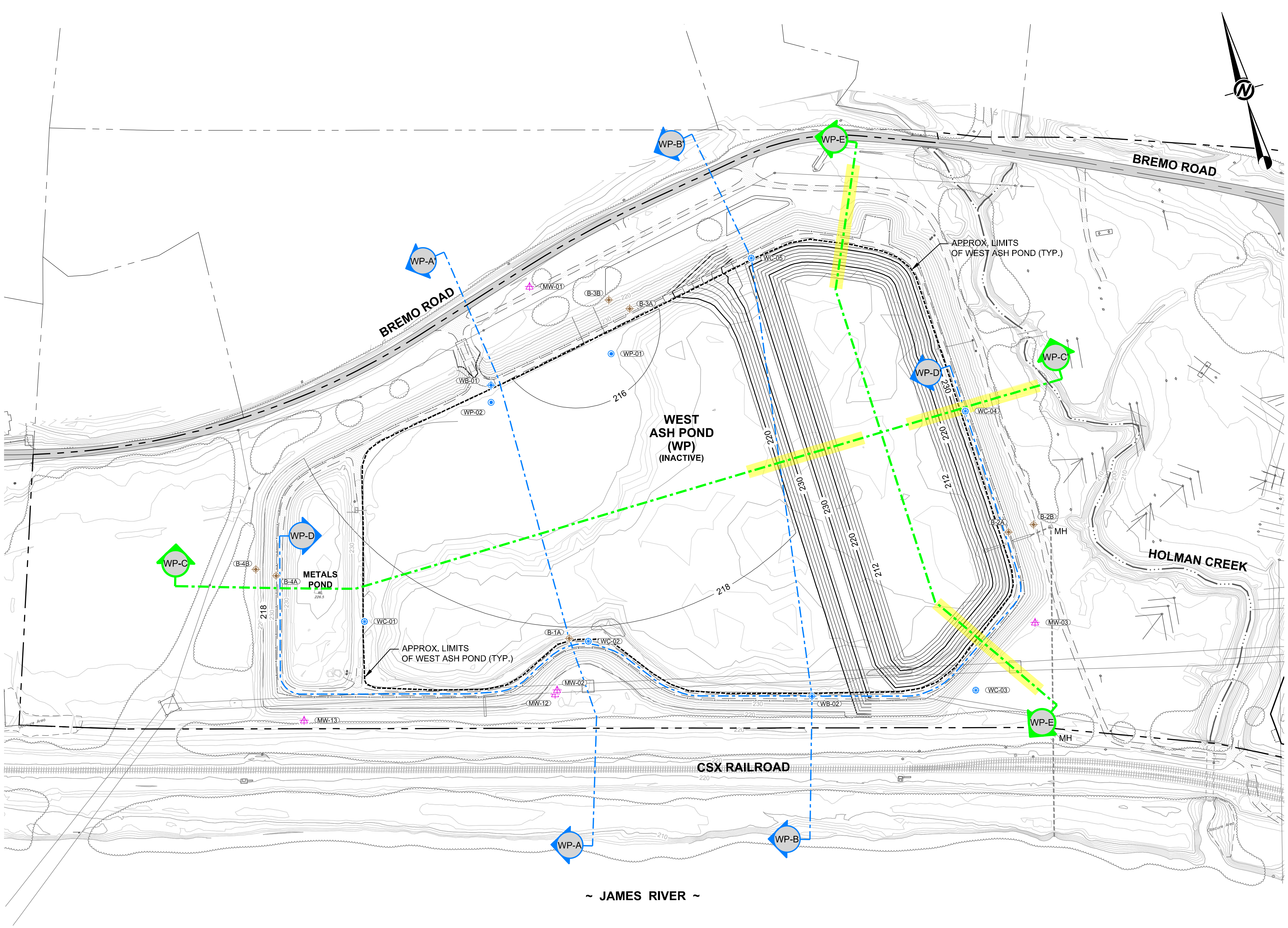

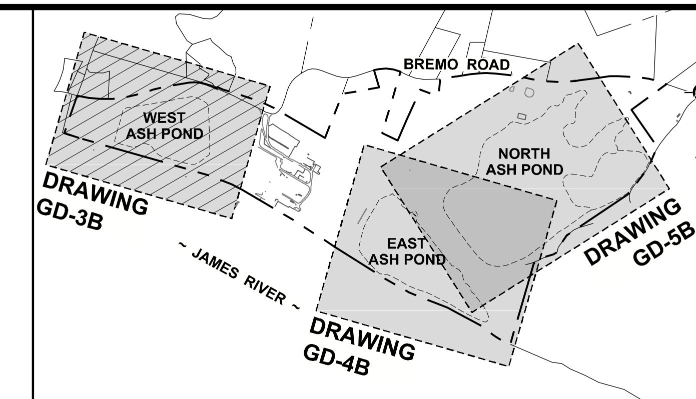

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- SLOPE STABILITY DRAWING LOCATIONS - SEE APPENDIX D

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REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
DESIGN GRADES GEOTECHNICAL STABILITY PLAN (WEST POND)

Golder Associates

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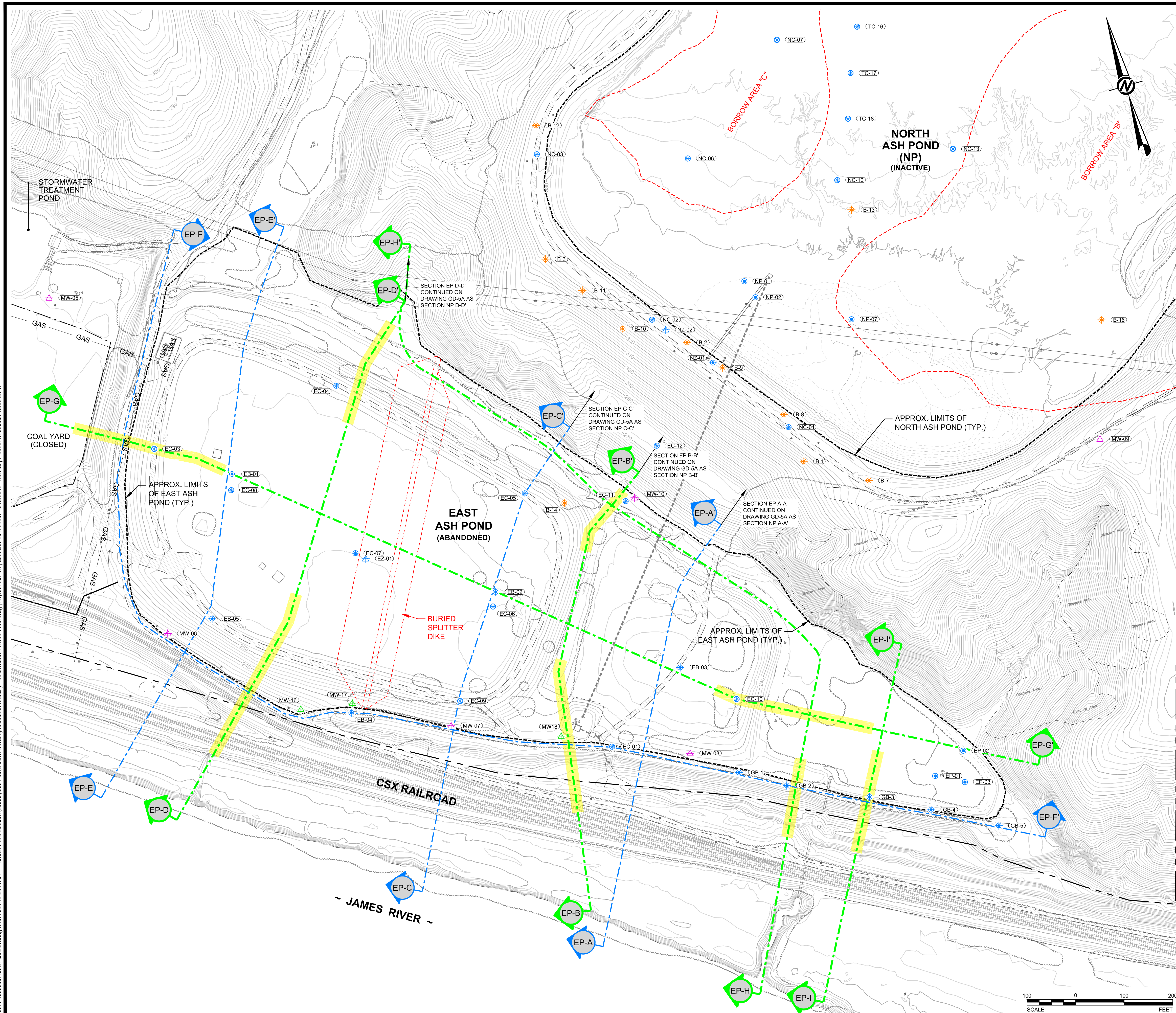

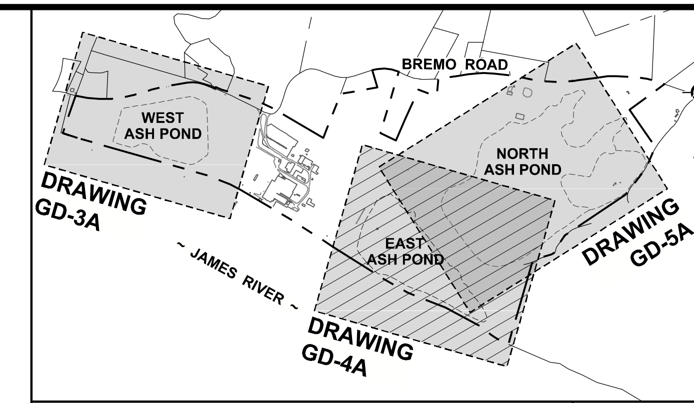

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2. BATHYMETRIC SURVEYS PREPARED BY H&B, SURVEYS PERFORMED IN FEBRUARY 2015.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE

EXISTING CONDITIONS GEOTECHNICAL EXPLORATION PLAN (EAST POND)

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD03A-05A
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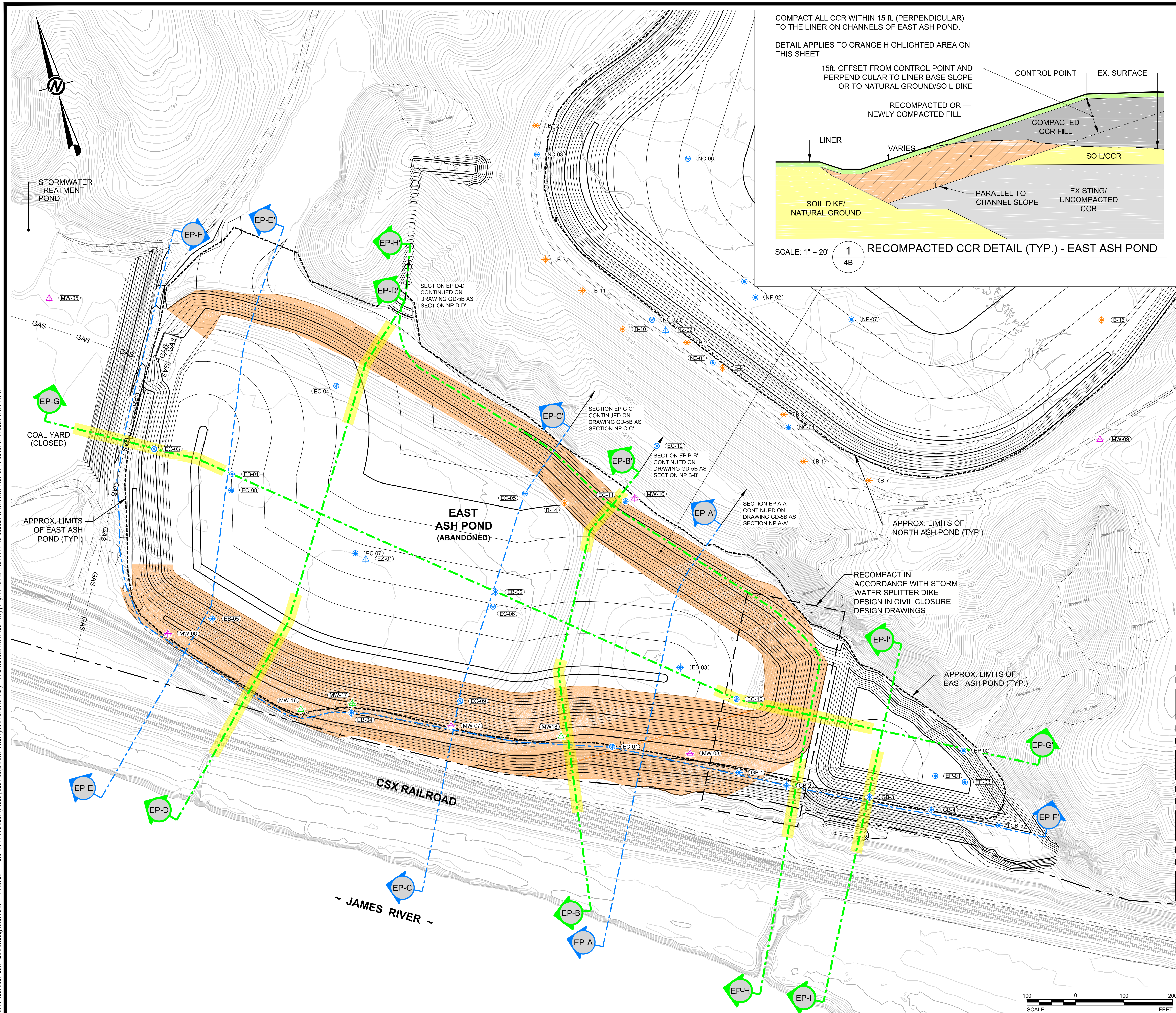

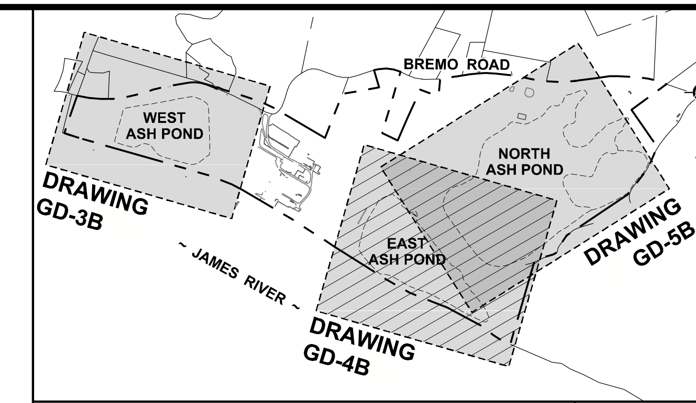

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- SLOPE STABILITY DRAWING LOCATIONS - SEE APPENDIX D

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2. BATHYMETRIC SURVEYS PREPARED BY H&B, SURVEYS PERFORMED IN FEBRUARY 2015.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE

DESIGN GRADES GEOTECHNICAL STABILITY PLAN (EAST POND)

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD02-05B
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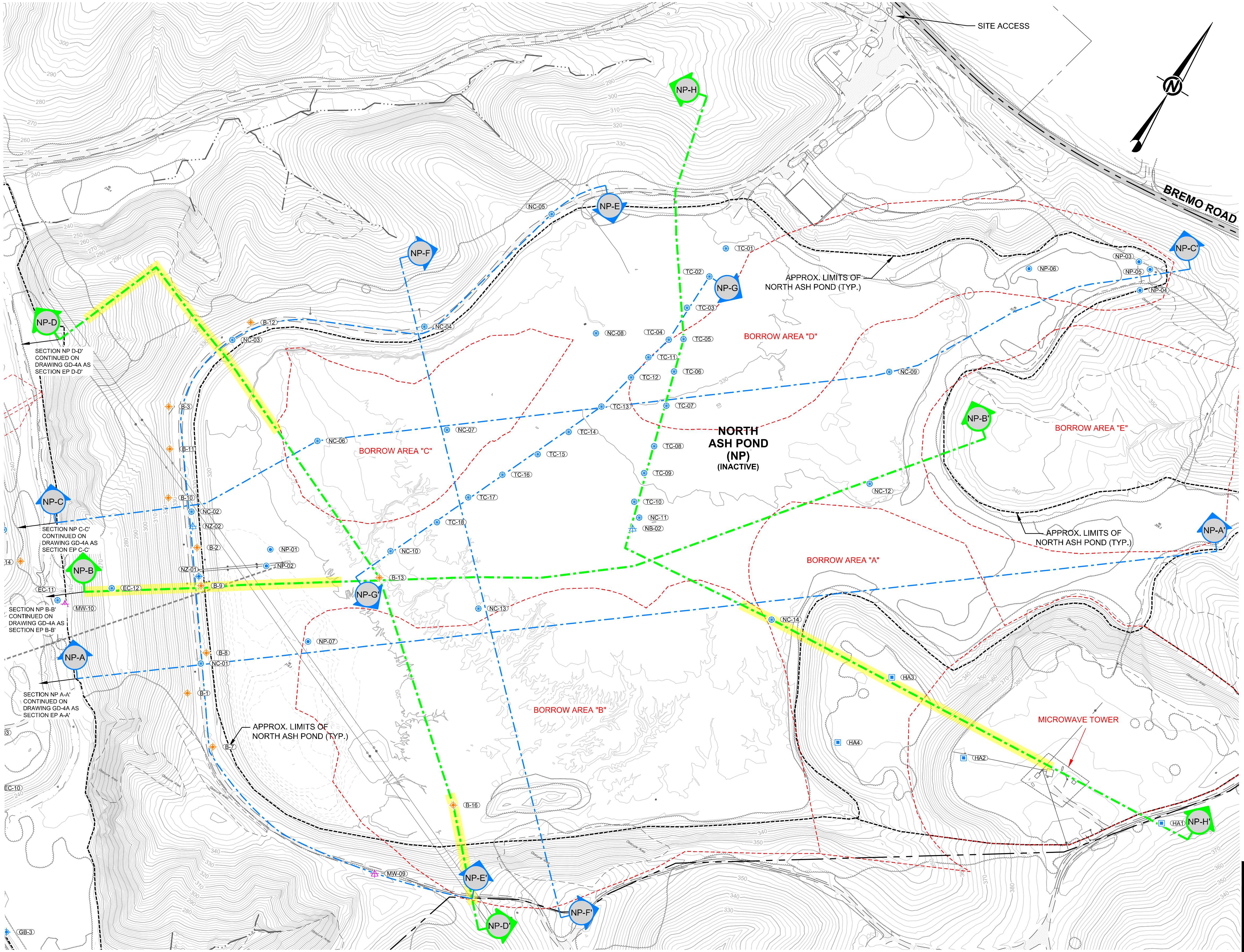

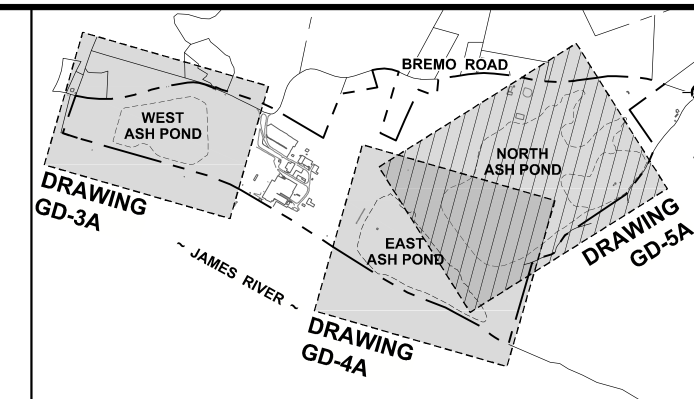

SITE KEY

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REFERENCES

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REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
EXISTING CONDITIONS GEOTECHNICAL EXPLORATION PLAN (NORTH POND)

PROJECT No.		15-20347	FILE No.	1520347AD03A-05A
DESIGN	-	-	SCALE	AS SHOWN
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CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		

Golder Associates

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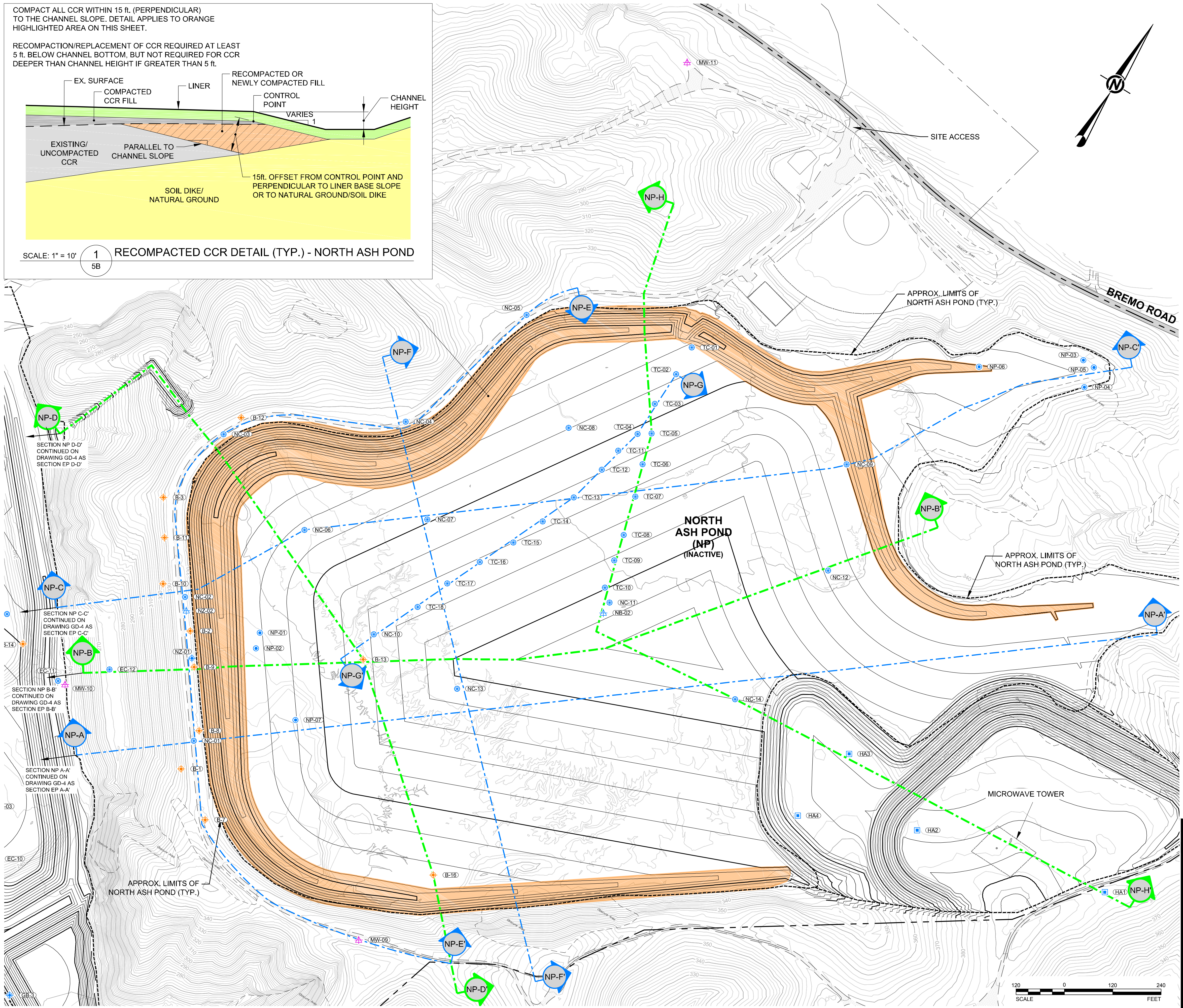

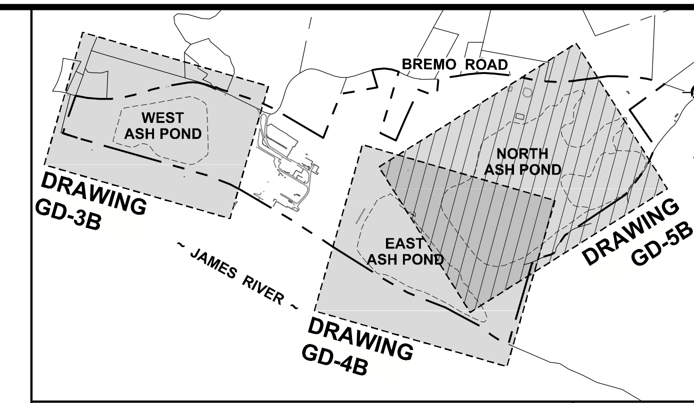

SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- EXISTING BATHYMETRIC SURFACE CONTOURS (2' INTERVALS)
- DESIGN SURFACE CONTOURS (2' INTERVALS)
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING RAILROAD
- CREEK CENTERLINE
- EXISTING TREE LINE
- EXISTING FENCE
- GAS — EXISTING GAS LINE
- DENOTES AREAS OF TOPOGRAPHY THAT DO NOT MEET MINIMUM ACCURACY STANDARDS FOR AERIAL SURVEYING
- SCHNABEL BORING (1982)
- SCHNABEL BORING (2010)
- GES MONITORING WELL (2013)
- HALEY AND ALDRICH BORING (2015)
- GOLDER BORING (2014 / 2015)
- GOLDER PIEZOMETER (2015)
- GOLDER CONE PENETRATION TEST (CPT)(2015)
- GOLDER PROBE HOLE (2015)
- GOLDER HAND AUGER (2015)
- EP-A — GEOTECHNICAL SECTIONS (SEE GOLDER 2015 GEOTECH DATA REPORT)
- EP-B — SLOPE STABILITY SECTIONS
- SLOPE STABILITY DRAWING LOCATIONS - SEE APPENDIX D

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].
2. BATHYMETRIC SURVEYS PREPARED BY H&B, SURVEYS PERFORMED IN FEBRUARY 2015.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
DESIGN GRADES GEOTECHNICAL STABILITY PLAN (NORTH POND)

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD02-05B
DESIGN	-	-	SCALE	AS SHOWN
CADD	SEP	11/30/15	DRAWING GD-5B	
CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		

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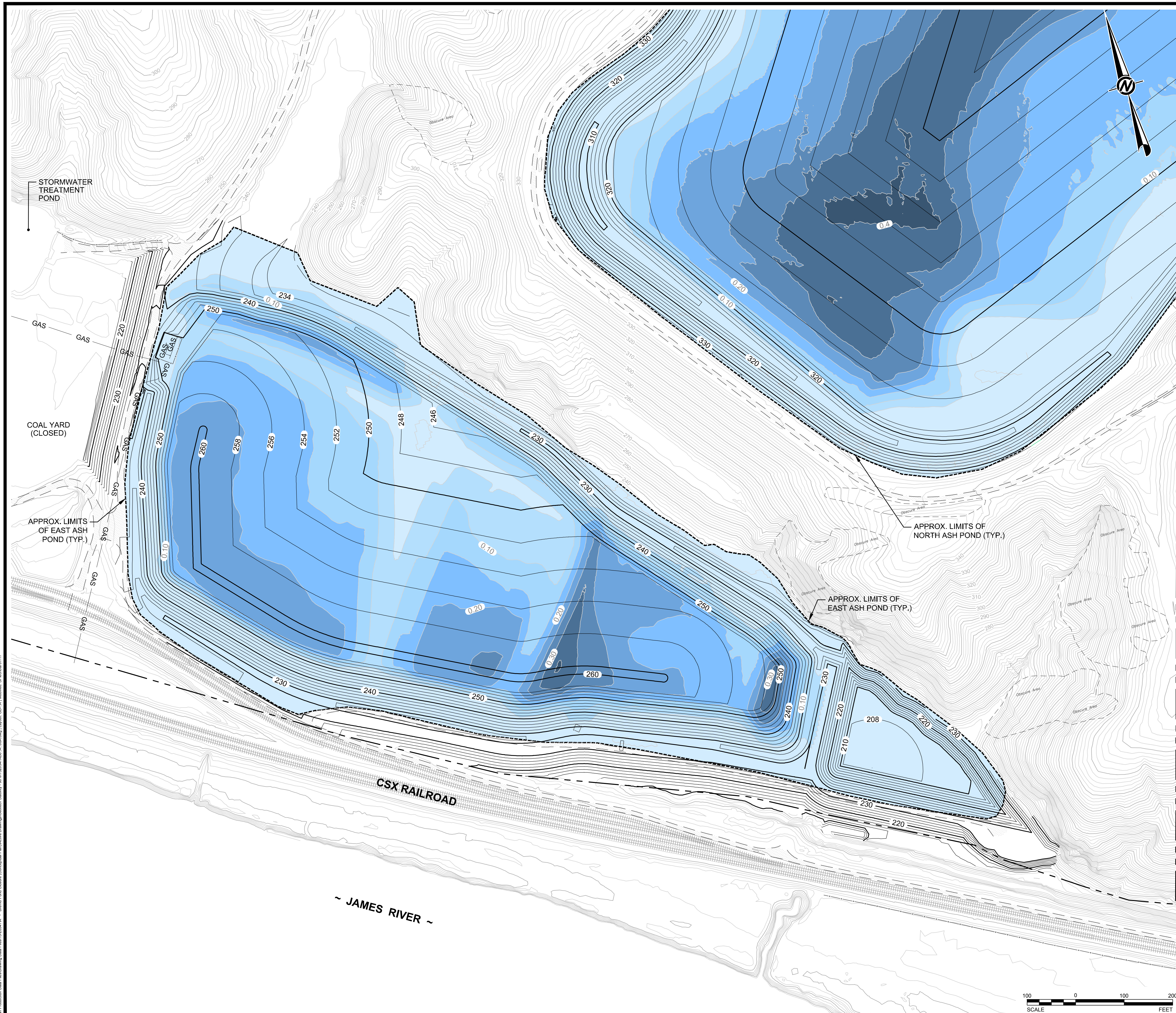

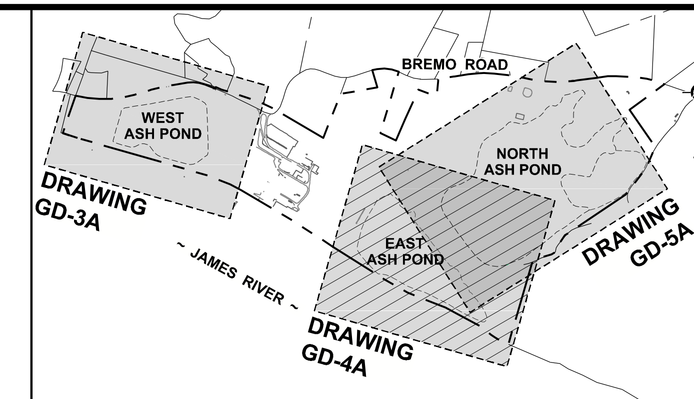

SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- DESIGN SURFACE CONTOURS (2' INTERVALS)
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING RAILROAD
- CREEK CENTERLINE
- EXISTING TREE LINE
- EXISTING FENCE
- GAS — EXISTING GAS LINE
- DENOTES AREAS OF TOPOGRAPHY THAT DO NOT MEET MINIMUM ACCURACY STANDARDS FOR AERIAL SURVEYING

SETTLEMENT

RANGE (FT)	COLOR
0.00 - 0.05	
0.05 - 0.10	
0.10 - 0.15	
0.15 - 0.20	
0.20 - 0.25	
0.25 - 0.30	
0.30 - 0.35	
0.35 - 0.40	
0.40 - 0.45	

NOTE

PREDICTED SETTLEMENT BASED ON DEGRADED ELASTIC MODULUS BASED ON IN-SITU TESTS; ACTUAL SETTLEMENT MAY VARY BETWEEN OR AWAY FROM CPT LOCATIONS DUE TO CHANGES IN MATERIAL PROPERTIES. PREDICTED CONTOURS HAVE NOT BEEN SMOOTHED, NOTING ACTUAL SETTLEMENT WILL BE SMOOTHER.

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].
2. 90% DESIGN GRADES PREPARED BY GOLDER ASSOCIATES INC., DATED 09/15/15.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
PREDICTED SETTLEMENT ISOPACHS EAST POND

Golder Associates

PROJECT No.		15-20347	FILE No. 1520347AD0_A_08A
DESIGN	-	-	SCALE AS SHOWN
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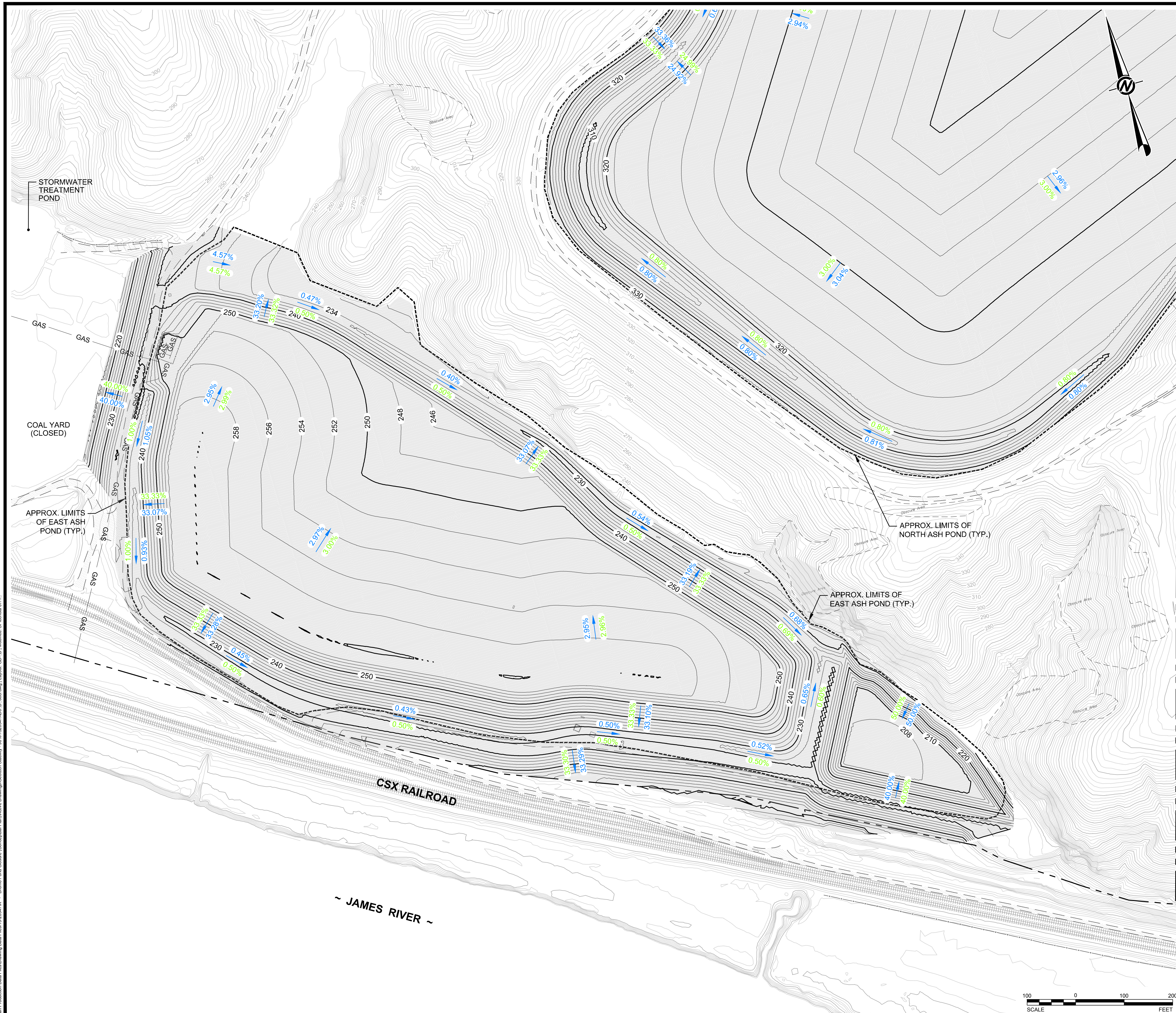
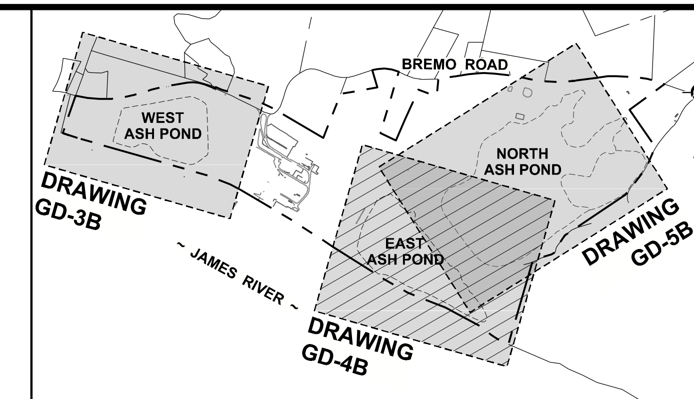
CHECK	JGM	11/30/15	
REVIEW	GLH	11/30/15	





SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- DESIGN SURFACE CONTOURS (2' INTERVALS)
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING RAILROAD
- CREEK CENTERLINE
- EXISTING TREE LINE
- EXISTING FENCE
- GAS — EXISTING GAS LINE
- DENOTES AREAS OF TOPOGRAPHY THAT DO NOT MEET MINIMUM ACCURACY STANDARDS FOR AERIAL SURVEYING
- POST-SETTLEMENT SLOPE (1.79%)
- SURFACE GRADE ARROW
- DESIGN SLOPE (1.82%)

NOTE

PREDICTED SETTLEMENT BASED ON DEGRADED ELASTIC MODULUS BASED ON IN-SITU TESTS; ACTUAL SETTLEMENT MAY VARY BETWEEN OR AWAY FROM CPT LOCATIONS DUE TO CHANGES IN MATERIAL PROPERTIES. PRESENTED CONTOURS HAVE NOT BEEN SMOOTHED, NOTING ACTUAL SETTLEMENT WILL BE SMOOTHER.

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].

2. 90% DESIGN GRADES PREPARED BY GOLDER ASSOCIATES INC., DATED 09/15/15.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE

PREDICTED POST SETTLEMENT SURFACE EAST POND

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD0□B□08B
DESIGN	-	-	SCALE	AS SHOWN
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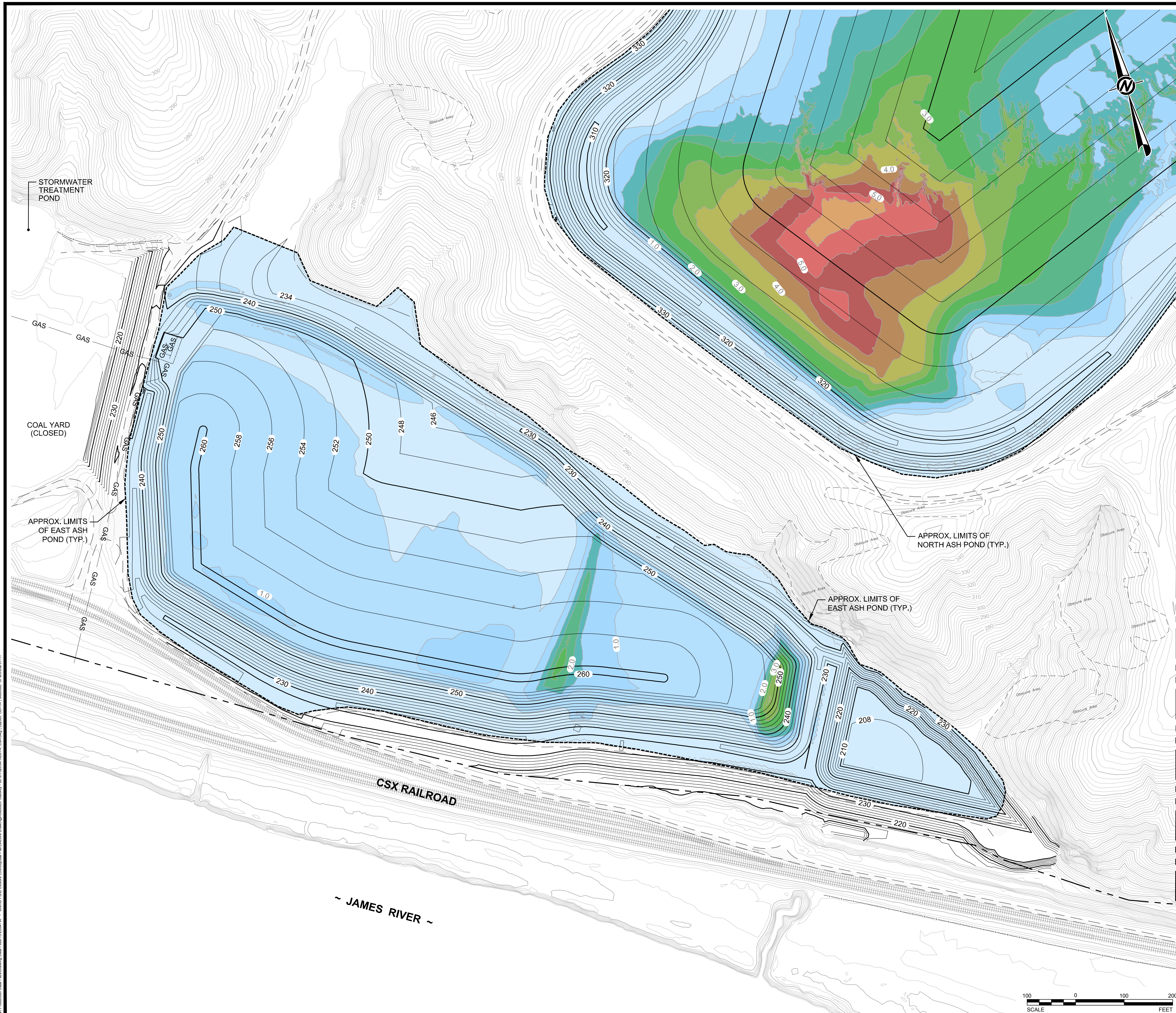
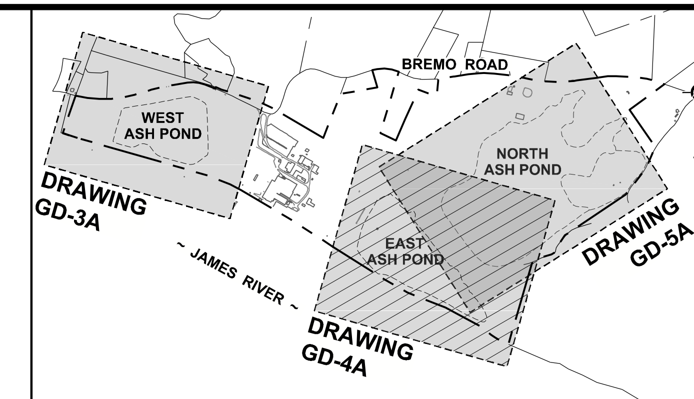
CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		





SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- DESIGN SURFACE CONTOURS (2' INTERVALS)
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING RAILROAD
- CREEK CENTERLINE
- EXISTING TREE LINE
- EXISTING FENCE
- EXISTING GAS LINE
- DENOTES AREAS OF TOPOGRAPHY THAT DO NOT MEET MINIMUM ACCURACY STANDARDS FOR AERIAL SURVEYING

SETTLEMENT	
RANGE (FT)	COLOR
0.00 - 0.50	
0.50 - 1.00	
1.00 - 1.50	
1.50 - 2.00	
2.00 - 2.50	
2.50 - 3.00	
3.00 - 3.50	
3.50 - 4.00	
4.00 - 4.50	
4.50 - 5.00	
5.00 - 5.50	
5.50 - 6.00	

NOTES

PREDICTED SETTLEMENT BASED ON CONSOLIDATION THEORY CONSIDERING UNIFORM DRAINAGE DRAWDOWN, AND CONSERVATIVELY NOT CONSIDERING CEMENTATION, AGING & OTHER DIAGENESIS. SEE PREDICTED PLOTS FOR PRESENTATION OF ESTIMATED SETTLEMENTS.

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].
2. 90% DESIGN GRADES PREPARED BY GOLDER ASSOCIATES INC., DATED 09/15/15.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT
DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE
CONSERVATIVE SETTLEMENT ISOPACHS EAST POND

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD07A_09A
DESIGN	-	-	SCALE	AS SHOWN
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CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		

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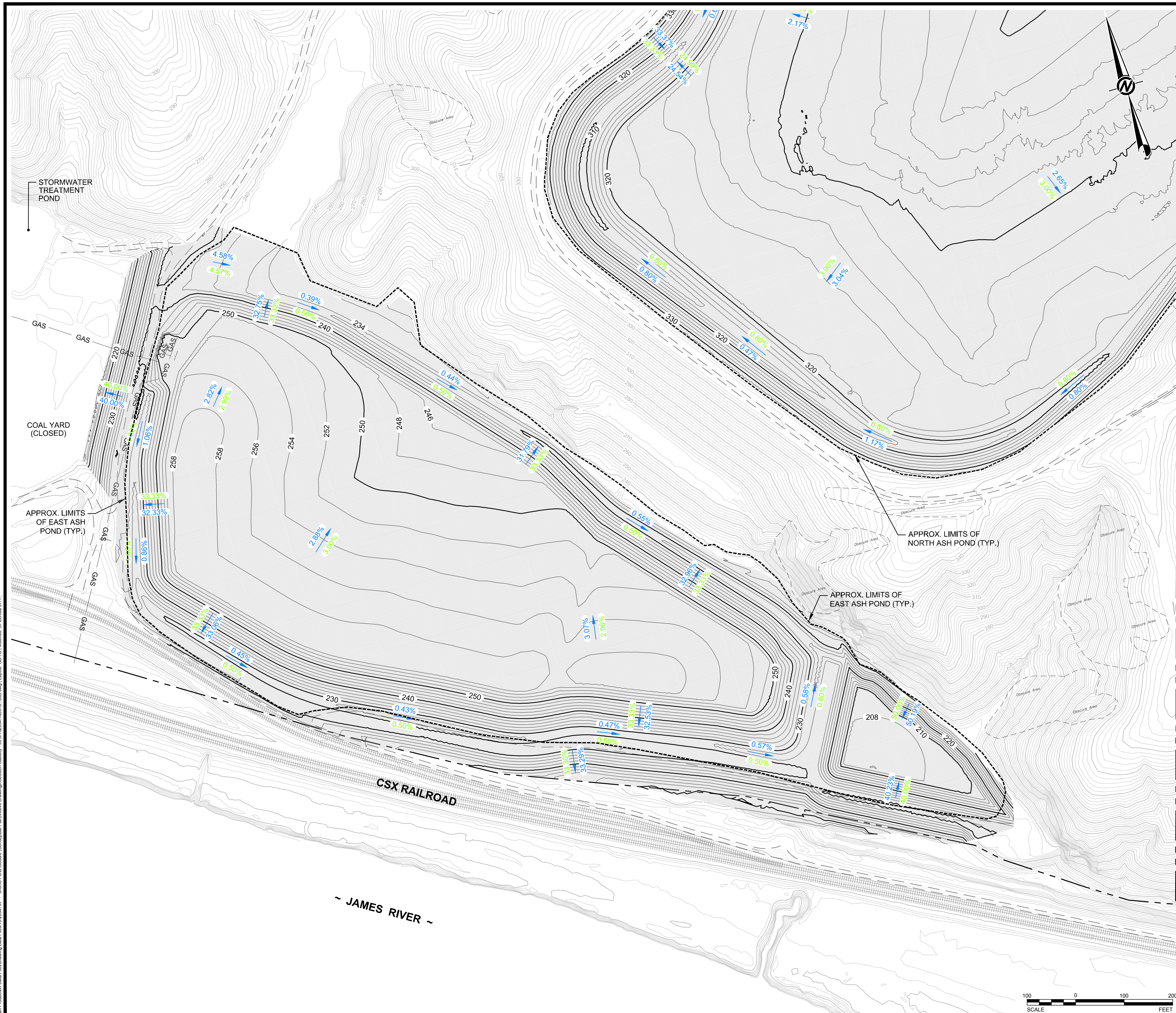

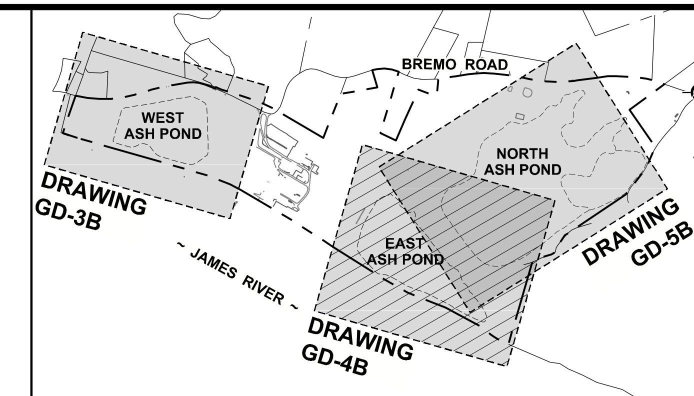

SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- 300 — EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- 220 — DESIGN SURFACE CONTOURS (2' INTERVALS)
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING RAILROAD
- CREEK CENTERLINE
- EXISTING TREE LINE
- EXISTING FENCE
- GAS — EXISTING GAS LINE
- Obscure Area — DENOTES AREAS OF TOPOGRAPHY THAT DO NOT MEET MINIMUM ACCURACY STANDARDS FOR AERIAL SURVEYING
- 1.79% / 1.82% — SURFACE GRADE ARROW (POST-SETTLEMENT SLOPE; DESIGN SLOPE)

NOTES

PREDICTED SETTLEMENT BASED ON CONSOLIDATION THEORY CONSIDERING UNIFORM DRAINAGE DRAWDOWN, AND CONSERVATIVELY NOT CONSIDERING CEMENTATION, AGING & OTHER DIAGENISIS. SEE PREDICTED PLOTS FOR PRESENTATION OF ESTIMATED SETTLEMENTS.

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].
2. 90% DESIGN GRADES PREPARED BY GOLDER ASSOCIATES INC., DATED 09/15/15.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE

CONSERVATIVE POST SETTLEMENT SURFACE EAST POND

Golder Associates

PROJECT No.		15-20347	FILE No.	1520347AD07B09B
DESIGN	-	-	SCALE	AS SHOWN
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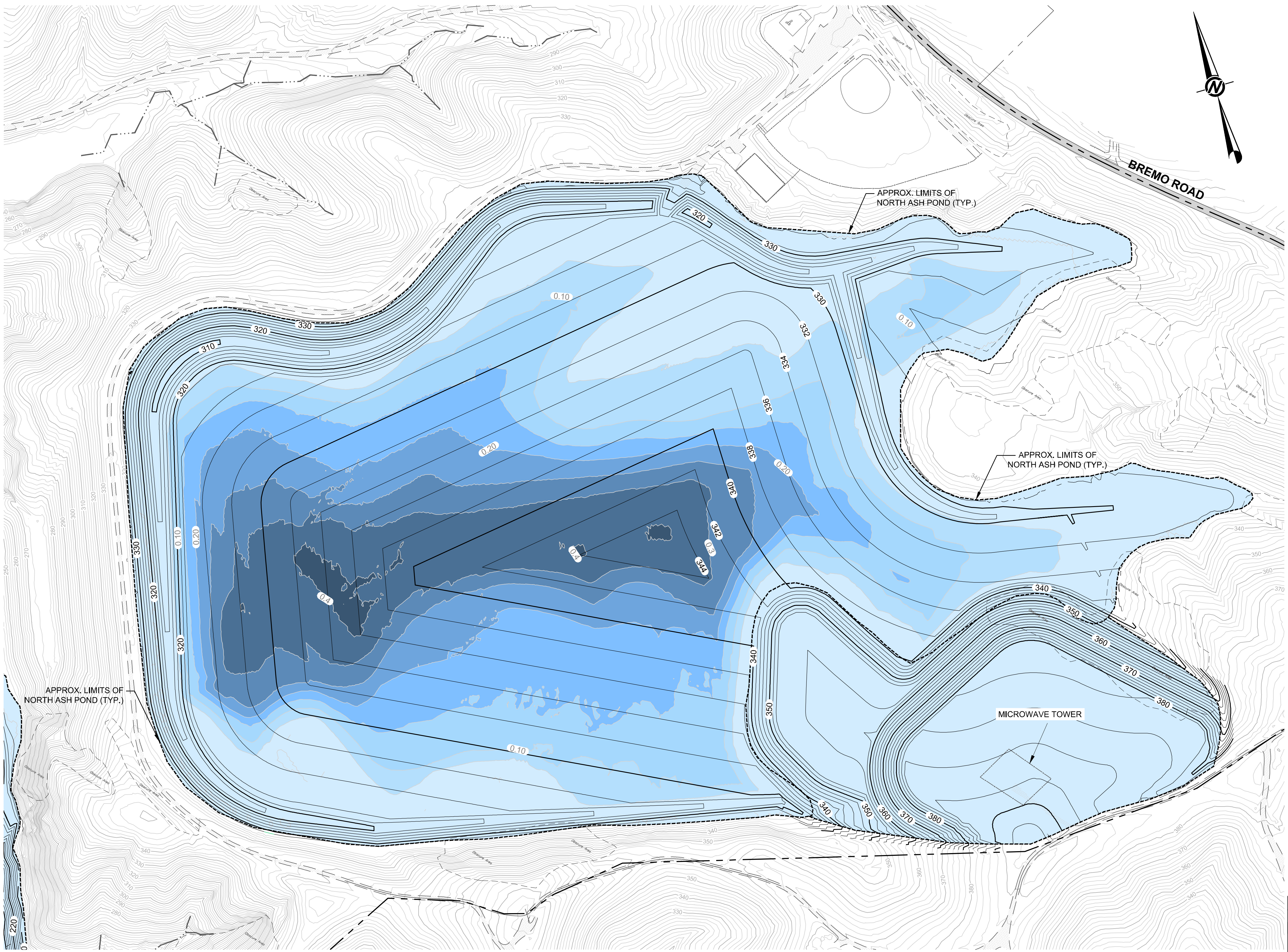
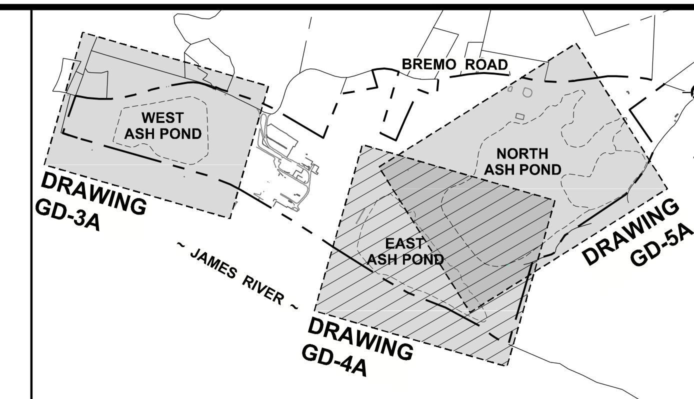
CHECK	JGM	11/30/15		
REVIEW	GLH	11/30/15		





SITE KEY

NOT TO SCALE

LEGEND

- DOMINION PROPERTY BOUNDARY
- ADJACENT PROPERTY BOUNDARY
- APPROXIMATE LIMITS OF EXISTING ASH PONDS
- 300 — EXISTING TOPOGRAPHIC CONTOURS (2' INTERVALS)
- 220 — DESIGN SURFACE CONTOURS (2' INTERVALS)
- EXISTING PAVED ROAD
- EXISTING UNPAVED ROAD
- EXISTING RAILROAD
- CREEK CENTERLINE
- EXISTING TREE LINE
- EXISTING FENCE
- GAS — EXISTING GAS LINE
- Obscure Area — DENOTES AREAS OF TOPOGRAPHY THAT DO NOT MEET MINIMUM ACCURACY STANDARDS FOR AERIAL SURVEYING

SETTLEMENT	
RANGE (FT)	COLOR
0.00 - 0.05	
0.05 - 0.10	
0.10 - 0.15	
0.15 - 0.20	
0.20 - 0.25	
0.25 - 0.30	
0.30 - 0.35	
0.35 - 0.40	
0.40 - 0.45	

NOTE

PREDICTED SETTLEMENT BASED ON DEGRADED ELASTIC MODULUS BASED ON IN-SITU TESTS; ACTUAL SETTLEMENT MAY VARY BETWEEN OR AWAY FROM CPT LOCATIONS DUE TO CHANGES IN MATERIAL PROPERTIES. PRESENTED CONTOURS HAVE NOT BEEN SMOOTHED, NOTING ACTUAL SETTLEMENT WILL BE SMOOTHER.

REFERENCES

1. AERIAL TOPOGRAPHIC SURVEY PREPARED BY McKENZIE SNYDER, INC., DATE OF AERIAL PHOTO: 01/16/15 [CONTROL PREPARED BY H&B SURVEYING & MAPPING (H&B)].
2. 90% DESIGN GRADES PREPARED BY GOLDER ASSOCIATES INC., DATED 09/15/15.

REV	DATE	REVISION DESCRIPTION	DES	CADD	CHK	RVW

PROJECT

DOMINION
BREMO POWER STATION
CCR IMPOUNDMENT CLOSURE
FLUVANNA COUNTY, VIRGINIA

TITLE

PREDICTED SETTLEMENT ISOPACHS NORTH POND

Golder Associates

PROJECT No.		15-20347
DESIGN	-	-
CADD	SEP	11/30/15
CHECK	JGM	11/30/15
REVIEW	GLH	11/30/15

FILE No. 1520347AD0□A□08A

SCALE AS SHOWN

DRAWING GD-8A

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