Enclosure 2

Construction Dewatering Design Report
No. SL-009655, Revision 2

Sargent & Lundy LLC

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UniStar Nuclear
Bell Bend Nuclear Power Plant

Deliverable: Construction Dewatering Design Report
Schedule ID: BBPP020504129

Mr. Stephen Geier
VP New Nuclear Projects
750 East Pratt St., 16th Floor
Baltimore, MD 21202

Dear Mr. Geier:

Sargent & Lundy LLC (S&L) is submitting this letter to document transmittal of the following deliverable for approved for USE.

- Construction Dewatering Design Report No. SL-009655, Rev. 2, 11-23-10.

Also, posted on the ftp server; BBNPP Documents for Approval/Deliverables/Sargent & Lundy/Part 02 FSAR/_ Second Submittal - For OAR, are three (3) CD's that contain our consultants report (Weaver Boos) and the data run from Weaver Boos.

If you have any questions, please contact me at 312.269.6482.

Yours very truly,

R. A. Hameetman
Senior Project Manager

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Transmittal Coversheet

UniStar NUCLEAR ENERGY

1. UniStar Purchase Order Number: Bell Bend: 300458			
2. UniStar Purchase Order Short Title: Bell Bend			
3. (Check one): ☒ ***First Time Submittal*** ☐ ***Resubmitted***			
4. Reason for Submittal (check any that apply): ☐ ***For Final Approval*** ☐ ***For Comments (OAR)*** ☐ ***For Information*** ☒ ***Conceptual Design for Use***			
5. Vendor: ☐ **AREVA** ☐ **Black & Veatch** ☐ **Rizzo** ☒ **Sargent & Lundy** ☐ **Other:**			
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8. Schedule Activity ID: BBPP020504129			
9. Related Section of COLA: FSAR 2.5.4			
10. Submittal Date: 11/23/10			
	PRINT NAME	**DATE**	**SIGNATURE**
11. **Preparer:**	Tawana Barnett	11/23/10	Tawana Barnett
12. **Supplier:** NOTE: Signature indicates the supplier offering the Document for Approval for acceptance has verified that the Document for Approval being furnished complies with the requirements in the UniStar procurement documents.	Robert A. Hameetman	11/232/10	Robert A. Hameetman

13. **Administrator**			
14.(Check one): ☐ **Approved As Is** ☐ **Not Approved (Comments Attached)** ☐ **Review Not Required**			

15. ***Reviewers:***	
UniStar Engineering *(name/date)*	***UniStar Quality Assurance*** *(name/date)*
UniStar Regulatory Affairs *(name/date)*	***Client*** *(name/date)*
Other *(name/ date)*	***Other*** *(name/ date)*

16. ***Final Acceptance:***	
UniStar Contract Manager *(name/date)*	***Other*** *(name/date)*

FORM 2(Rev 06.2008)

UniStar NUCLEAR

Construction Dewatering Design

Bell Bend Nuclear Power Plant

UniStar Nuclear Energy

Non-Safety-Related

Report No. SL-009655

Revision 2

November 23, 2010

Sargent & Lundy LLC

Approval Page

Construction Dewatering Design

Non-Safety-Related

Revision Summary

Revision 0	**For OAR Review**
Revision 0	**For Use**
Revision 0A	**For OAR Review**
Revision 1	**For Use**
Revision 2A	**For OAR Review**
Revision 2	**For Use**

Prepared By: ____________________ Date: 11/23/2010
D. E. Nielson

Reviewed By: Daniel C. Kocunik Date: 11/23/2010
D. C. Kocunik

Approved By: R. A. Hameetman Date: 11/23/2010
R. A. Hameetman

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1. PURPOSE/OBJECTIVE

The purpose of this report is to evaluate the existing groundwater conditions around the proposed

- Nuclear Island (NI), which includes:
    - Reactor Building;
    - Fuel Building;
    - Reactor Auxiliary Building;
    - Safeguard Buildings;
    - Radioactive Waste Building;
    - Emergency Diesel Buildings;
    - Ultimate Heat Sink Buildings;
    - Turbine Building;
    - Essential Service Water Emergency Makeup System (ESWEMS) pond and pump house;
- ESWEMS Pipeline between the pump house and NI; and
- Two Cooling Towers,

at the Bell Bend Nuclear Power Plant (BBNPP) and provide recommendations for the temporary construction dewatering system during the construction of the power plant. Attachment A depicts a conceptual layout of the major elements of the plant.

This information will be used to support the Combined Operating License Application (COLA) for the BBNPP. This evaluation will be the basis for the discussion of the construction dewatering system and the disposal of the extracted water as addressed in the Environmental Report (ER).

The purpose of Revision 2 is to evaluate the existing groundwater condition around the NI based on its new location 972 feet north and 300 feet west of the original plant location (Reference 10.1). Revision 2 is a comprehensive revision and thus, revision bars are not indicated.

2. BACKGROUND

The explored site conditions and plant layout result in two distinct conditions. The NI and cooling towers will be located in an area of unsaturated granular soils above the shale bedrock. In the vicinity of the ESWEMS pond and pump house, the lower granular soils that overlie the bedrock are saturated.

For structural and seismic design considerations, the safety related structures, will be supported on bedrock or engineered fill (concrete or granular) extending from the bearing elevation down to the top of competent rock. Although considered safety related, the ESWEMS pond will be supported on cohesive fill extending from the pond floor down to the competent rock.

This construction detail and the site geologic setting requires an approximate maximum of 56 feet of water-bearing sands and gravels to be excavated (References 10.2, 10.3 and 10.4) for the ESWEMS pond. Proper placement of the backfill requires the work be performed in a dry condition. As such, an active construction dewatering system will be implemented prior to construction to maintain dry conditions and it will continue until the subgrade portions for these structures are completed and the excavation is backfilled. The dewatering system will be decommissioned as the structures are completed and the backfill is placed to a level above the groundwater and up to the final grade.

3. DESIGN INPUTS

The following design inputs and assumptions are used in this report:

a. Three months of available groundwater levels are provided in the Paul C. Rizzo and Associates (Rizzo) response to RFI SL-BBNPP-111 (References 10.3).

b. Twelve months of groundwater levels for monitoring wells installed in 2008 as documented in FSAR Section 2.4.12 (Reference 10.5).

c. Locations of the monitoring wells, subsurface soil and rock descriptions, and top of rock elevations as provided by Rizzo in their responses to RFI SL-BBNPP-132 and RFI SL-BBNPP-111 (References 10.2 and 10.3, respectively). Attachment C presents the locations of the groundwater monitoring wells.

d. Dewatering system criteria, groundwater levels with various dewatering approaches and comments as provided by Weaver Boos Consultants North Central, LLC (Weaver Boos) (Reference 10.6 – included as Appendix A of this report).

e. Potential construction reuse of groundwater pumped from the excavation (Reference 10.7).

f. General site layout per the Reduced Scale Standard Utilization Plot Plan (SUPP) (Reference 10.1), which is provided as Attachment A.

g. Final yard high-point finished grade is established at Elevation 719 feet per the North American Vertical Datum of 1988 (NAVD 88), with the design finished floor elevation of the NI structures at 720 feet (Reference 10.8). Note, all elevations in this report refer to the NAVD 88 and are in feet.

h. The approximate elevation of the invert of the pipes from the ESWEMS pump house to the NI is 694 feet (Reference 10.13).

i. Water quality information from on site wells and water sampling (Reference 10.10).

j. Conceptual Excavation Plan as prepared by Rizzo and provided in their response to RFI SL-BBNPP-149 (Reference 10.4). Attachment B presents a compilation of the data presented in Reference 10.4.

4. ASSUMPTIONS

Design inputs 3a, 3b, 3c, 3e, 3h, 3i and 3j listed in Section 3 (above) are the latest available information based on responses to Requests For Information (RFI) and are considered as verified information for the conceptual design of the construction dewatering system. Design inputs 3f and 3g are the site layout and grading drawings and are the latest information for the conceptual design.

Integration of the Site conceptual model with a mathematical computer code to simulate flow requires several simplifying assumptions. The following assumptions and idealizations apply to the model utilized herein (from Reference 10.6):

a. The model domain is underlain by a conductive overburden aquifer extending through the basin lowlands and restricted in its horizontal extent by surrounding rises in the less conductive bedrock.

b. The complex natural flow system may be represented using a system of seven discrete layers, while the natural conditions likely result in a more gradual variation in hydrogeologic properties.

c. The baseline groundwater flow system is in equilibrium and is modeled on a steady-state basis.

Assumptions 4a, 4b, and 4c, listed above, are consistent with the available data and do not need further verification for this evaluation. Adjustments can be made during construction of the dewatering system to account for any subsurface discrepancies, which may be encountered during construction. All assumptions for this report are considered to be verified for its current use. However, this report will be reviewed after receipt of the 6-month and 12-month water levels in the piezometers currently being monitored and updated as necessary.

5. METHODOLOGY AND CRITERIA

The groundwater modeling and calculations discussed in this report were primarily performed by Weaver Boos based on field data obtained and evaluated by Rizzo. The Weaver Boos report is documented as Reference 10.6, which is attached as Appendix A to this report. The Rizzo findings from the field investigations and testing are documented in References 10.2, 10.3, 10.5, and 10.10.

Prior to assessing applicable dewatering technologies, Weaver Boos developed a conceptual model, which to the extent practicable, incorporates natural hydrogeologic boundaries for the flow system of interest. Preparation of the conceptual model included the following general steps:

- Defining hydrostratigraphic units based on the data presented in References 10.2, 10.3, 10.5, and 10.10;
- Defining the flow system; and
- Preparing a water budget of flows into and out of the area of interest.

Evaluation of groundwater flow was performed by Weaver Boos utilizing a seven-layered conceptual model implemented using Visual MODFLOW Version 2009, by Schlumberger Water Services. This software is a widely used implementation of the USGS's globally recognized MODFLOW-2000 program. Weaver Boos selected this software for its capability to reliably model groundwater flow in three dimensions and relative ease of use offered by its integrated graphical user interface. The modeling reflected two principal groundwater dewatering strategies:

- Open excavation and water table depression without groundwater flow barriers; and,
- Dewatering and excavation using a slurry wall, diaphragm wall, or other type of subsurface flow barrier to mitigate potential off-site water level drawdown and subsequent impact to potentially sensitive areas around the ESWEMS pond.

Since the ESWEMS pump house is contiguous with the ESWEMS pond dike alignment, the term ESWEMS pond incorporates the excavation for the pump house.

5.1 Model Domain

The digital model domain is based on a rectangular block-centered grid network that covers a 1.8-square mile flow domain representing the local drainage basin. The grid includes 316 rows and 245 columns, with their spacing refined as needed to assess small-scale effects in the area where dewatering is needed. In the areas where the greatest detail was desired, the grid node spacing is approximately 22 by 22 feet and provides site-scale detail without creating a computationally excessive number of model nodes.

5.2 Model Calibration

Calibration of the baseline flow model consisted of initial simulations, with the model-estimated groundwater elevations compared to the adjusted target values discussed in Section 6.5. To adjust for discrepancies in the model predicted and actual groundwater elevations, adjustment of selected model elements such as recharge flux and distribution, river boundary condition parameters along the edges of the model domain and along Walker Run were made. The calibration is iterative to allow a suitable set of recharge and boundary conditions to be formulated.

An evaluation of the calibration for all layers indicates a correlation coefficient of 0.90, which is considered reasonable given the distribution of the groundwater monitoring wells and the relatively short duration of groundwater monitoring in the recently installed wells. The calibration indicated the model estimate groundwater elevations were typically slightly lower than the target values.

The model calculated heads are considered a reasonable match with the observed values given the objective of the flow model (Reference 10.6).

This calibrated digital model was then used to simulate the dewatering program parameters as presented in the following sections. Additionally traditional hand calculations were used to check the results of the computer modeling, determine near well hydraulics, and to determine the well spacing. These methods are presented in Appendix D of Reference 10.6.

There are no acceptance criteria for this evaluation since its purpose is to provide recommendations for the need of a flow barrier to mitigate the drawdown effects of dewatering.

6. EVALUATION

6.1 Topography

The topography of the site is gently rolling with an east-west trending set of ridges. At the BBNPP, ground elevations range from 650 feet along Walker Run in the southwest corner of the site up to elevations slightly higher than 800 feet on the hilltop located in the vicinity of the NI and cooling towers (Reference 10.8). North of Beach Grove Road (north of the site), the elevation rises sharply upward to elevations of 1,100 to 1,150 feet along the crest of the ridge. Thus, total topographic relief in the immediate vicinity of BBNPP is approximately 500 feet. The ground surface elevation in the area of the NI generally ranges from approximately 700 to 800 feet. The ground surface elevation in the area of the ESWEMS pond ranges from approximately 680 to 740 feet. The existing grade elevation in the area of the cooling towers varies from 700 to 800 feet. Attachment A provides a general site layout for the BBNPP along with the general topography.

6.2 Geology

The geologic conditions described below are generally based on FSAR Sections 2.4.12 (Reference 10.5), 2.5.4 (Reference 10.9) and the recent boring logs (Reference 10.2).

Subsurface conditions beneath the site are characterized by sand and gravel deposits that are underlain by shale bedrock.

The Pleistocene Age overburden soils range in thickness from 0 to 100 feet with the thinner overburden encountered on the ridges and hills. With the exception of some loose sand pockets, the overburden consists of over-consolidated, brown silty sand and sand containing gravel and large rounded cobbles and boulders. The frequency of the boulders increases with depth.

The bedrock generally consists of folded, jointed and fractured Devonian-Age shales of the Mahantango Formation extending approximately 1000 to 1200 feet beneath the site. The Mahantango Formation consists primarily of dark gray, silty to very silty claystone. Frequent joints and intense cleavage development causes the claystone to become splintery, and fragmented upon weathering. The Mahantango Formation has low to moderate resistance to weathering.

6.3 Hydrogeology

The hydrogeologic conditions described below are generally based on FSAR sections 2.4.12 (Reference 10.5) and 2.5.4 (Reference 10.9), the Rizzo Monitoring Well Test Data Report (Reference 10.3) and the Weaver Boos report (Reference 10.6).

Generally, borings in the vicinity of the NI and cooling towers did not encounter groundwater in the overburden soils based on the field exploration performed by Rizzo (References 10.3 and 10.5). Although not encountered, it is not uncommon for groundwater to become perched in granular soils at the soil-bedrock interface. The occurrence and quantity of this perched groundwater is often seasonally affected and highly variable in areas of a sloped interface between the granular overburden and the less permeable bedrock. The conceptual design for the temporary construction dewatering system considers this potential source of water, which must be controlled to facilitate the planned work. In the vicinity of the ESWEMS pond and pump house, groundwater levels in the overburden typically range from 2 to 15 feet below the ground surface (bgs). Along the south side of the proposed ESWEMS pond, the depth to water was approximately 2 to 8 feet bgs, and flows generally southerly and westerly towards Walker Run. In the vicinity of the ESWEMS pond and pump house, the overburden aquifer is recharged by downward percolating precipitation and upflow from the deeper bedrock aquifer. Groundwater discharges from the surficial aquifer as springs and seeps into ponds, the wetlands along the southern border of the site, and into Walker Run.

The underlying Mahantango Shale Formation is also considered an aquifer. There are no extensive aquitards in the vicinity of the BBNPP site. Vertical groundwater flow in the upland areas to be developed as the power block and cooling towers is generally downward. Vertical groundwater flow is generally upward from the bedrock aquifer to the overburden aquifer in the area to be developed as the ESWEMS pond.

6.4 Hydraulic Properties by Layer

Groundwater flow is simulated in seven layers. Walker Run, the site wetlands and the excavations for the ESWEMS pond are generally located within Layer 1, which is a relatively high conductivity zone. The base of the ESWEMS pond excavation is generally at the top of the competent rock, which is within Layer 2 and exhibits a lower conductivity than Layer 1. The excavations for the NI and cooling towers extend through Layer 1 and well into Layer 2. Layer 2 includes the upper weathered rock zone, the transition zone and the upper extent of the competent rock. Layer 2 is also the primary component of the highland ridges. Layers 3 through 7 are deeper shale bedrock.

6.4.1 Layer 1

Layer 1 exhibits a varying thickness across the model domain, since its upper surface is based on the topography of the site and the lower surface is based on the interface with weathered bedrock (Reference 10.2). Rizzo performed and documented (References 10.3 and 10.5) both slug and pump tests to quantify the horizontal hydraulic conductivity of Layer 1.

The horizontal hydraulic conductivity values calculated from slug tests conducted in the overburden aquifer ranged from 1.19×10^{-5} cm/s to 3.4×10^{-2} cm/s, with a geometric mean of 3.63×10^{-3} cm/s (Reference 10.5, Section 2.4.12.3.2.1). A single slug test in the overburden was completed in observation well MW410 during 2010, indicating a horizontal hydraulic conductivity of 1.72×10^{-3} cm/s (Reference 10.3). Slug tests of the kind implemented during the site investigation measure horizontal hydraulic conductivity only near a test well, and may reflect influences by filter pack storage or low-conductivity borehole skins remaining after conventional rotary drilling using mud.

The horizontal hydraulic conductivity values calculated based on a 24-hr pump test at approximately 60 gpm ranged from 3.63×10^{-2} cm/s to 1.26×10^{-1} cm/s, with a geometric mean of 5.93×10^{-2} cm/s (Reference 10.5, Section 2.4.12.3.2.1). A pump test of the kind implemented during the site investigation stresses a much broader area of the aquifer than a slug test, and is therefore considered more representative than the slug test results.

S&L considers the geometric mean value obtained during the pump test, which is the highest mean value for the site, as a representative (yet conservative) value for the horizontal hydraulic conductivity in the overburden aquifer. Because sand and gravel deposits comprising the overburden aquifer are horizontally stratified as described in the boring logs, the deposit is likely anisotropic, and the vertical hydraulic conductivity (which has not been measured) is considered to be 1/10th of the horizontal value obtained during the pump test. The specific yields computed for the pump test indicated values ranging between 0.253 and 0.500, with a geometric mean of 0.344, and a median value of 0.322. For a well- to fairly well-graded material such as the overburden, the median value of 0.322 appears reasonable and is therefore considered appropriate for use in the model. Thus, conservative values to quantify the hydraulic properties of the overburden aquifer were selected for use in this conceptual dewatering evaluation.

6.4.2 Layers 2 Through 7

Layer 2 also varies in thickness, since it extends from the interface with the overburden down to elevation 600 feet and is also referred to as the shallow shale bedrock. Layers 3 through 7 are considered to be uniform in thickness and extend from elevation 600 feet down to elevation 0 feet.

The horizontal hydraulic conductivity values calculated from slug tests conducted in the shallow bedrock aquifer during 2007 and 2008 ranged from 3.70 x 10^{-4} cm/s to 1.36 x 10^{-2} cm/s, with a geometric mean of 1.41 x 10^{-3} cm/s (Reference 10.5, Section 2.4.12.3.2.2). The horizontal hydraulic conductivity values calculated from slug tests conducted in the shallow bedrock during 2010 ranged from 4.69 x 10^{-5} cm/s to 1.32 x 10^{-3} cm/s, with a geometric mean of 2.86 x 10^{-4} cm/s (Reference 10.3, Table 3).

The horizontal hydraulic conductivity values calculated from slug tests conducted in the deep bedrock aquifer ranged from 1.15 x 10^{-5} cm/s to 1.51 x 10^{-3} cm/s, with a geometric mean of 1.18 x 10^{-4} cm/s (Reference 10.5, Section 2.4.12.3.2.2). No further slug testing of wells screened in the deep bedrock was reported during 2010.

Packer tests were performed in 56 intervals of the shale bedrock during 2007 and 2008. Of these tests, nearly one-half (26) indicated impermeable rock. In the other 30 tests, the horizontal hydraulic conductivity ranged from 2.39 x 10^{-7} to 1.63 x 10^{-4} cm/s (Reference 10.5, Section 2.4.12.3.2.2). Packer tests were performed in 34 additional intervals of shale bedrock during 2010. In these most recent tests, seven (7) tests indicated impermeable rock. In the other 27 tests, the horizontal hydraulic conductivity ranged from 3.99 x 10^{-7} cm/s to 3.82 x 10^{-4} cm/s (Reference 10.3).

The horizontal hydraulic conductivity values calculated based on a 24-hr pump test at approximately 6 gpm ranged from 1.93 x 10^{-5} cm/s to 7.23 x 10^{-4} cm/s, with a geometric mean of 1.64 x 10^{-4} cm/s (Reference 10.5, Section 2.4.12.3.2.2) during 2007 to 2008. This pump test was completed in wells screened from elevations ranging from 502 to 582 feet. Additional pump tests were performed using wells screened in the bedrock during 2010. The horizontal hydraulic conductivities calculated on the most recent pump tests ranged from 6.42 x 10^{-6} cm/s to 2.88 x 10^{-4} cm/s, with a geometric mean calculated equal to 5.43 x 10^{-4} cm/s (Reference 10.3, Table 4). The recent pump tests utilized wells screened in the bedrock at elevations ranging from 618 to 670 feet.

For the reasons discussed in Section 6.4.1, S&L considers the geometric mean values obtained during the pump tests as more representative than values obtained by slug testing. However, results from the packer testing program are also considered representative for the intervals that were tested. Of the values reported for the shale bedrock, the geometric mean horizontal hydraulic conductivities from the pump tests are selected as conservatively high values for use in the dewatering evaluation. S&L selected the geometric mean pump test conductivity value of 5.43 x 10^{-4} cm/s as representative and conservative for shallow bedrock occurring above an elevation of approximately 600 feet (based on 2010 pump test). The geometric mean pump test conductivity value of 1.64 x 10^{-4} cm/s is similarly selected as representative and conservative for deep bedrock occurring below an elevation of approximately 600 feet (based on 2007 and 2008 pump tests). These values are regarded as conservative because their selection is likely to over-predict rather than under-predict the flow of groundwater to be yielded by temporary dewatering systems. As was selected for the overburden, S&L also considered the vertical conductivities of the bedrock (shallow and deep) to equal $1/10^{th}$ of their respective horizontal values obtained during the pump tests.

Although the shale bedrock is correctly described as an aquifer in Reference 10.5 (Section 2.4.12.3.2.2), the conductivity of Layer 2 is about $1/1{,}100^{th}$ of the overburden aquifer conductivity, while Layers 3 through 7 are about $1/3{,}700^{th}$ of the overburden aquifer. The contrast in conductivity between the overburden and bedrock aquifers means the preferential flow path is through the overburden rather than the bedrock aquifers.

6.5 Groundwater Level Observations

Monthly water table elevations in the overburden and the head elevations in the bedrock were previously measured between October 2007 and September 2008 (Reference 10.5, Table 2.4-44). The groundwater elevation data obtained in 2007 and 2008 indicate a slight seasonal variation in groundwater elevation has been observed during the monitoring period. Generally, the groundwater elevation is at a minimum in autumn, followed by gradually increasing levels in winter, peak groundwater elevations are noted in the early spring and then decreasing elevations through the summer. For the overburden monitoring wells, the differences between the annual high and low elevations for each well ranged from 1.67 to 5.49 feet. Elevations measured on January 26, 2008 appear to represent "average" conditions, and elevations measured on March 24, 2008 are taken to represent "high" water levels.

Monthly groundwater levels were most recently reported for the period between May 2010 and July 2010 (Reference 10.3). Measurements taken during 2010, which include measurements from the initial round of wells (MW300 series) and the recently installed MW400-series observation wells (MW401 through MW410), are generally somewhat lower than the "average" levels measured during January 2008. In order for this evaluation to conservatively consider the reasonably foreseeable maximum future groundwater elevations that may occur in the 2010 – 2011 12-month monitoring period, two feet was added to the "high" groundwater elevations previously reported during March 2008. These higher values were then selected as flow model calibration targets. The highest recently measured water levels in the MW400-series wells were "corrected" to reasonably foreseeable maximum future groundwater levels by adding 5.0 feet to the new rock wells and 4.6 feet to the new overburden well for the calibration targets to evaluate the groundwater model simulations. The resulting conservatively high groundwater elevations selected for use herein are listed in Table 2 of Reference 10.6, which is Appendix A of this report.

6.6 Excavation Approaches and Dewatering Implications

6.6.1 Collection Ponds for Dewatering Output

Prior to initiating dewatering activities, preparations must be made to receive the water discharged from the dewatering systems. Final selection of the site ponds, which will receive the flow from the dewatering system, as well as the precipitation that falls in the excavation, should be based on the location of the ponds, piping routes, pond volumes and the sequence of construction of the ponds. The following paragraphs address the possible ponds which may be selected to receive the waters.

6.6.1.1 Temporary Groundwater Storage Pond

Effluent from the dewatering system could be routed through the Temporary Groundwater Storage Pond (TGWSP), which is to be located between the ESWEMS and the NI. Thus, it would be beneficial to construct this pond prior to excavation activities in order to use it as a collection area for the dewatering system. The design elevation of the bottom of the TGWSP is anticipated to be approximately elevation 665 to 670 feet (to be determined during final design), while the current ground surface is at approximately elevation 672 feet. Table 5 of Reference 10.3 documents the range of measured groundwater elevations from 656.58 to 658.91 feet in this area. Given this information, it is anticipated that dewatering for the construction of the TGWSP will not be needed.

6.6.1.2 Combined Waste Water Retention Pond

The Combined Waste Water Retention Pond (CWWRP) is located east of the ESWEMS Pond. Reference 10.16 indicates the design elevation of the bottom of the CWWRP is 686.5 feet. The current ground surface elevation in the vicinity of the CWWRP ranges from approximately 676 to 728 feet (Reference 10.8). The estimated groundwater level is estimated to be below elevation 675 feet (Reference 10.3), except where the groundwater may be perched on top of the bedrock. Although the field exploration program by Rizzo did not encounter groundwater perched on the bedrock, some perched groundwater can be anticipated. Given this information, it is anticipated that dewatering for the construction of the CWWRP will not be required. However, if any groundwater is encountered at the soil/rock interface, this can be controlled by utilizing diversion trenches and sumps around the periphery of the excavation to maintain a dry condition. Where soil is present beneath the pond floor, the groundwater is estimated to be below the excavation limits.

The CWWRP could be used for storage and exfiltration of the dewatering effluent provided it is constructed early enough and that the lining designed for the permanent pond is not installed until after the dewatering is complete. However, the excavation for the placement of engineered fill below the ESWEMS pond (as depicted in the drawing attachments to the Response to RFI SL-BBNPP-149 [Reference 10.4]) appears to intersect the western portion of the CWWRP. Thus, the final excavation plan for the ESWEMS pond construction or the final design of the CWWRP will require some slight modifications to allow the use of this waste pond for dewatering effluent while the ESWEMS pond excavation and dewatering system is active.

6.6.1.3 Other Ponds

In addition to the TGWSP and the CWWRP, other impoundments such as the Temporary Sediment Basin (TSB) and the ESWEMS Pond could receive dewatering system output, provided they are operable prior to completion of the dewatering activities for the NI and the cooling towers.

6.6.2 ESWEMS Pond

The ESWEMS pond excavation is expected to fully penetrate the overburden soils and the upper weathered bedrock to establish the bearing surface (subsurface information from References 10.2 and 10.4) on the competent rock at elevations ranging from 610 to 640 feet. The excavation in the vicinity of the southern portion of the ESWEMS pond will extend approximately 56 feet through saturated granular deposits. Existing groundwater elevations are discussed in Section 6.3.

To facilitate quality construction methods in the ESWEMS area, the excavations should be performed in a dry condition. A dewatering system consisting of deep wells surrounding the excavation is conceptually designed. The excavation can proceed as the dewatering takes place provided the dewatering system maintains the groundwater level below that of the excavation. As the excavation advances, a series of groundwater monitoring wells will be monitored to verify the effectiveness of the dewatering system in reducing the groundwater level.

6.6.3 Nuclear Island and Cooling Towers

The final plant grade in the vicinity of the nuclear island and cooling towers is at elevation 719 and 699 feet (Reference 10.8), respectively, while the current ground surface ranges from approximately 700 to 800 feet (Reference 10.8) in these areas. These structures and/or the fill supporting these structures will extend down to the upper surface of the competent rock, which is at an approximate minimum elevation of 650 feet in the NI and elevation 565 feet in the vicinity of the cooling towers, as indicated by References 10.2 and 10.4. Thus, the excavations associated with construction of the NI and cooling towers will extend from the current ground surface, through the surficial soils and into the bedrock. Based on References 10.2 and 10.3, the overburden soils are not saturated. General groundwater conditions for the site are discussed in Section 6.3. Thus, an active dewatering system for the upper soils is not likely to be required. It is expected that groundwater inflows from localized water bearing zones in the overburden and from the bedrock (weathered and unweathered) may be controlled using trench drains at the soil bedrock interface as well as some trenches cut into the bedrock excavation slopes and floor. The trench drains can be sloped to sumps where the water can be pumped out if a proper slope cannot be attained to drain the trenches to the groundwater storage pond by gravity.

An area of uncertainty is located at the northwest portion of the cooling towers excavation, where the available boring data is limited. Specifically, Rizzo developed excavation plans (Reference 10.4) for the cooling towers based on a boring located at the proposed center of each tower. From these two borings, they extrapolated the bedrock surface elevation and likely excavation depth. Reference 10.4 indicates the northwest quadrant of the cooling towers excavation will extend down to elevation 646 feet. Figure 2.4-33 of Reference 10.5 indicates that the stream bed of Walker Run near the intersection of Market Street and Beach Road is at an approximate elevation of 675 feet. If the overburden extends below the elevation of Walker Run, it is likely that the overburden will be saturated. Figure 5 of Reference 10.3 did not indicate groundwater in the overburden soils in this area. If the overburden soils are saturated, it is likely that excessive groundwater pump rates and subsequent dewatering of the adjacent wetlands and possibly Walker Run will occur. The groundwater pumping rates and subsequent drawdown determined and presented in this evaluation does not consider this potential outcome since the Rizzo groundwater data does not indicate the overburden in this area to be water-bearing. It should be noted that if the overburden extends below the wetlands, this condition could be mitigated by installing a flow barrier wall as discussed for the ESWEMS pond and pump house. No further discussion is provided for the cooling towers since this condition and the extent of the cooling towers excavation will be determined during the subsurface exploration and construction phases for the cooling towers.

6.6.4 ESWEMS Pipeline

The ESWEMS pipeline from the ESWEMS pump house to the NI will have an approximate invert elevation of 694 feet, with the pipe bedding supported on the natural soils (Reference 10.13). Since the construction activities for the ESWEMS pipeline is above the groundwater elevation of 665 feet (Figure 5 of Reference 10.3), construction dewatering will not be needed.

6.6.5 Groundwater Flow Barrier

Dewatering for the ESWEMS excavation can be performed either with or without a flow barrier as discussed later. Based on the inputs for this work, the ESWEMS excavation is the only area where a flow barrier was considered. However, based on the site conditions (which may be identified when additional exploratory borings and wells are performed), a flow barrier may also be considered for the northwestern area of the cooling towers excavation.

If a flow barrier, such as a slurry wall, is constructed, a significant reduction in the required pumping rate and aerial extent of drawdown will be achieved. Reference 10.6 considered the effect of implementing a flow barrier along a preliminary alignment. If the final design of the flow barrier is combined with a construction phase excavation support to minimize the pond excavation, the alignment may be adjusted inward (made smaller). An open excavation (sloped sidewalls not structurally supported) is currently planned for construction of the ESWEMS pond as indicated in the response to RFI SL-BBNPP-149 (Reference 10.4). The extent of the excavation with this approach is quite wide. If a construction phase excavation support (earth retention system) is used, the planned dimension of the excavation will be smaller since the cut slope out of the excavation will be eliminated. Since the dimension of the excavation is now smaller, the barrier can be closer to the pond, making the overall area to be dewatered smaller.

6.7 Definition of the Flow System

Weaver Boos reviewed the available information and formulated the following definition of the flow system as presented in Reference 10.6. This review indicates that the BBNPP site may be viewed as located within a small groundwater basin storing water mostly in the overburden aquifer. The overburden aquifer basin is defined to the north by the system of higher ridges, to the east by a bedrock ridge and groundwater flow divide corresponding approximately to the route of Confers Lane, to the south by a bedrock ridge forming in the knolls, and to the west by a bedrock ridge forming in the uplands west of Walker Run. Surface water and groundwater enter the overburden basin from the north and exit the basin via Walker Run, its small tributary located on the BBNPP site.

Deeper groundwater flows through the bedrock are less constrained than in the overburden basin and are assumed to reflect high upland recharge occurring to the north, followed by upward flow just south of the site, and deeper horizontal southerly and southeasterly flow towards the Susquehanna River.

6.7.1 Water Flow Budget (Initial Steady State Conditions)

The groundwater digital model is presented in Reference 10.6 (included as Appendix A of this report), and is summarized in the following sections. Based on the baseline flow budget presented in Reference 10.6, the basin receives and discharges groundwater from three potential sources of groundwater flow:

- The first is groundwater discharge, assumed equal to groundwater recharge, reported in Table 2.4-42, of Reference 10.5 for the Wapwallopen Creek Basin as ranging from 6.6 to 21.8 inches per year, with an average equal to approximately 14.2 inches per year.
- The second is groundwater exchange with Walker Run that flows along the west side of the model domain.
- The third is groundwater inflow originating in the ridge that rises to elevations as high as 1,100 feet directly north of the site. This source cannot be directly measured, yet its significance is inferred from the upward vertical flow of groundwater in the lowland areas south of the proposed power block and ESWEMS pond.

Potential discharges of groundwater originating beneath the site include bank and bottom discharge to Walker Run and subsurface outflow to the south (much of which likely occurs in overburden deposits beneath Walker Run), with eventual discharge to the Susquehanna River. Additional southerly and southeasterly discharges of groundwater through the shallow and deep bedrock are also inferred from the bedrock potentiometric surfaces provided by Reference 10.3.

6.7.2 Water Flow Budget and Drawdown Forecast for Dewatering Without a Flow Barrier

The mass flow budget for this model includes drains that represent the collective withdrawal of groundwater by multiple dewatering wells to temporarily (about three years) depress the groundwater to facilitate construction of the ESWEMS excavation and drains to represent dewatering trenches and/or well points to dewater the minor inflows in the NI and cooling towers excavations.

Zone budgets were set in the model to separately account for the dewatering system outflows from the power block, cooling towers, and ESWEMS excavation. The dewatering system under this scenario is suggested to remove water at a rate of 0.11 cfs (50 gpm) at the power block excavation and 0.16 cfs (70 gpm) from the cooling towers excavation. The total flow from the ESWEMS excavation is 2.0 cfs (920 gpm), which is the sum of approximately 0.56 cfs (250 gpm) from the ESWEMS drains, and 1.5 cfs (670 gpm) from the ESWEMS dewatering wells. The total pump flow rate of 2.3 cfs (1040 gpm) will be required to maintain steady state conditions in all three excavations. These rates are steady state and will be much higher when dewatering is first initiated. The flow rates when the dewatering program is implemented will be dependent upon the desired schedule to achieve the target groundwater elevations.

The digital model results of drawdowns in Layer 1 for the dewatering system without the use of flow barriers are illustrated in Attachment D. The drawdowns are shown in feet, and represent water table depression from the steady state head calculated by the calibrated model. Review of Attachment D indicates deep water table depression (5 to 40 feet) in the areas extending west, south, and east of the proposed ESWEMS pond. The model predicts an area of up to 25 feet of groundwater table depression extending approximately 400 feet south and east of the ESWEMS pond. This pumping scheme would most likely result in extensive dewatering of the wetlands south of the ESWEMS pond.

6.7.3 Water Flow Budget and Drawdown Forecast for Dewatering With a Flow Barrier

Installation of a flow barrier, such as a soil-bentonite slurry wall, or diaphragm wall substantially reduces the steady-state outflow from the ESWEMS pond excavation dewatering system.

Considering the preliminary alignment of the flow barrier depicted on Attachment H, the model calculated (Reference 10.6) the steady state flow rate required to dewater the ESWEMS excavation to be approximately 0.51 cfs (230 gpm) (0.35 cfs from the rock drains and 0.16 cfs from the wells) as compared with 2.0 cfs (920 gpm) without the barrier. Total dewatering system outflow for the NI, cooling towers and ESWEMS excavations is approximately 0.78 cfs (350 gpm) considering a flow barrier around the ESWEMS and approximately 2.3 cfs (1040 gpm) without a flow barrier. The model also indicates that with the flow barrier around the ESWEMS, the flow from the drains in the rock portion of the three excavations (NI, cooling towers and ESWEMS pond) will yield approximately 0.62 cfs (280 gpm). However, the actual flow may be less due to the wide range of hydraulic conductivities reported in References 10.3 and 10.5. Numerous packer tests conducted in the shale during the site investigation indicate hydraulic conductivity values much lower than considered in the model, and in approximately one-half of the tests, the hydraulic conductivity was effectively zero. Thus, these hydraulic conductivities and the resultant flow values are considered to be conservatively high.

The flow rates discussed herein are steady state and will be higher when dewatering is initiated. The initial rates of dewatering within the flow barrier are dependent upon the schedule allocated to achieve the target groundwater elevation and the volume of water stored in the pore space of the soils within the barrier wall. As the alignment of the barrier wall is adjusted, the initial flow rate and/or schedule of achieving the target groundwater elevation will need to be reconsidered.

As before, the flow model (modified to include a groundwater barrier wall around the ESWEMS pond and pump house, wells and drains) used the initial heads computed by the baseline flow model and the expected drawdowns are plotted on Attachment E. Review of this figure again shows the deep drawdown required at the ESWEMS pond. However, the simulated drawdown elsewhere in the basin is very much less than the simulation without the flow barrier. Drawdown greater than 5 feet is focused immediately west and southwest of the flow barrier. This effect is likely not primarily due to the withdrawal of water from within the flow barrier, but rather due to the partial cutoff of natural westerly flow of groundwater through the position of the barrier. Groundwater levels are expected to diminish on the down-gradient side of a flow barrier and possibly build along the upgradient side. The close proximity of the wetland to the flow barrier wall at the northwest corner of the ESWEMS pond (near the 50-foot buffer zone) and construction of the ESWEMS pumphouse structure directly south of the wetland may result in some mounding of groundwater upgradient of these impermeable barriers. This groundwater mounding may result in a rise in the groundwater level and subsequent expansion of the wetlands into the 50 foot buffer zone. Thus, there will be a need to monitor the water level fluctuation in this wetland area.

6.8 Conceptual Dewatering Design

In general, the dewatering system should be designed to remove the flows suggested by the flow budgets and to evacuate the precipitation that falls into the excavation during construction. The flows discussed herein, only consider those flows originating from the groundwater and not those associated with evacuation of precipitation into the excavation. However, due to the conservatism used in this conceptual design, as noted later, the dewatering system should be capable of extracting most of the precipitation that falls within the limits of the excavation. Considering that sound construction practice dictates the area around the excavations will be graded to prevent stormwater from flowing into the excavation, the only additional water to be evacuated will be the direct precipitation that falls into the excavations. The approximate cumulative aerial extent of the excavations is 53.7 acres (from the plans provided in RFI No SL-BBNPP-149). Considering the storm water report (Report No. SL-009446 [Reference 10.15]), the 100 year storm event is 7.49 inches in a 24 hour period. The increased flow from this storm event is 10,921,000 gallons per day (10.9 mgd) or 7580 gpm. This flow, combined with seepage into the excavations equates to a flow of 7,930 gpm. This flow should be within the capacity of the pumps for the sumps which collect and discharge the flow. These pumps will be sized in final design.

Dewatering wells could be drilled at this site using direct rotary, reverse-circulation rotary, cable tool, or other methods such as Rotosonic drilling. Reverse-circulation rotary will provide wells with the greatest efficiency and should therefore be considered. The other methods listed might tend to compact the aquifer formation, or leave low-conductivity borehole skins that cannot be completely removed during development. Because the overburden aquifer contains boulders, it may be necessary to use a chisel, or other methods to remove or penetrate them.

6.8.1 Conceptual Design Without Flow Barrier

Deep dewatering wells may be located around the perimeter of the ESWEMS excavation to implement the first stage of water table depression. Because wells cannot depress the water table to the base of the aquifer in areas between the wells, a level of approximately 10 feet above the shale is selected as a target for use in computing cumulative drawdowns. By inspection of the drawdown curves presented in Appendix D of Reference 10.6, an inter-well spacing of approximately 100 feet will provide for a cumulative drawdown of slightly more than 50 feet at locations between the wells. Dewatering wells may be located as shown on Attachment F, based on this conceptual design criterion. A total of approximately 28 dewatering wells appear to be appropriate for conditions at the ESWEMS pond excavation. Given the large number of wells required and potentially very large initial flows that such a system might develop, individual pumps should be sized for maximum flows of approximately 100 to 150 gpm each. The discharge lines should be fitted with throttling valves to control the overall flow rate of the system and avoid overwhelming the body receiving the discharge. A schematic diagram showing a typical dewatering well considered appropriate for conditions at this site is provided as Attachment I.

The ESWEMS excavation will likely require a method to control groundwater at the interface of the overburden and weathered shale in the form of a system of vacuum well points positioned as shown on Attachment F. Each of the headers shown will draw water from well points that are typically 2-in. diameter that may be drilled, driven or jetted in if conditions allow. Each header will need to be connected to its own vacuum pump. Individual vacuum pumps will need to be sized based on conditions encountered and the length of each header.

Final stages of the dewatering conceptual design for the ESWEMS excavation include the installation of trench drains and sumps into the exposed bedrock surface at the base of the ESWEMS excavation. Such trenches might be excavated 3 to 5 feet wide, and 2 to 3 feet deep, and sloped to collection sumps for ejection from the excavation. Groundwater flow from the bedrock is expected to vary over a wide range, and additional trenches or sumps might be needed at locations to be determined. Three such trenches were incorporated into the digital flow model at the ESWEMS pond as shown on Attachment F.

Groundwater observations at the NI and cooling towers excavations suggest that little saturated overburden is present in either area. It is therefore expected that groundwater inflows may be controlled using trench drains cut into the bedrock at the locations and elevations suggested on Attachments F (NI) and G (cooling towers). The trench drains can be sloped to sumps where the water can be pumped to the TGWSP or other disposal points if gravity drainage to the ponds cannot be established.

The effectiveness of the dewatering system should be monitored to compare observed drawdown with the estimates described herein (or more detailed design estimates developed prior to implementation). Water levels may be monitored for this dewatering strategy using existing monitoring well clusters that have been drilled at the site. Additional monitoring wells or piezometers should be installed at select locations to provide further points for comparison. A typical schematic diagram for monitoring wells or piezometers is provided on Attachment J.

Operation of this conceptual dewatering system will require an uninterrupted source of power for electrically operated submersible pumps and vacuum pumps, and an uninterrupted source of fuel for internal combustion vacuum pumps if selected for use. Provisions for convenient maintenance should be included for all system elements as needed for a project duration approaching 3 years.

6.8.2 Conceptual Design of a Flow Barrier

Temporary construction dewatering of the site was simulated to evaluate the potential benefits of a flow barrier encompassing the proposed ESWEMS pond excavation (See Paragraph 6.7.3). Wall boundaries considered in the flow model were a 3-feet thick flow barrier characterized by a hydraulic conductivity of 1×10^{-6} cm/s. The wall boundaries form a continuous flow barrier around the proposed excavation and extend from top to bottom in Layer 1 of the model. This model simulation utilized 14 pump wells, located inside the flow barrier wall to achieve dry conditions in the ESWEMS pond. The preliminary alignment of the flow barrier and the wells is presented on Attachment H.

As discussed in Paragraph 6.6.3 an area of uncertainty is located at the northwest portion of the cooling towers excavation, where the available boring data is limited. If the overburden extends below the elevation of Walker Run or the associated wetlands, it is likely that the overburden soils are saturated it is likely that excessive groundwater pump rates and subsequent dewatering of the adjacent wetlands and possibly Walker Run occur. If these conditions are present the installation of a flow barrier wall should be considered in the area of the cooling towers excavation where the overburden extends below the groundwater table.

The NI excavation will not require a flow barrier.

If a soil-bentonite (S-B) slurry wall is selected for use as a flow barrier, it might be installed along an alignment as shown on Attachment H, and should reflect the following guidelines in its final design:

- The slurry wall will be a minimum of three feet thick, and will be at least ½-foot-thick for each 10 feet of hydraulic head across the wall.
- The slurry wall will be keyed into competent shale such that the flow underneath the wall through the shale is less than or equal to the flow directly through the soil-bentonite slurry wall. The minimum depth of penetration of the slurry wall key will be two feet into the shale below any permeable lenses or weathered shale zones.
- The slurry will consist of 4 to 7 percent bentonite in water, and the backfill will contain bentonite at a rate of 3 percent. If the groundwater barrier is also designed to act as a temporary excavation support wall, Portland cement may also be incorporated into the slurry.

- The slurry wall will have a designed in-situ permeability less than or equal to 1 x 10^{-7} cm/s. A value of 1 x 10^{-6} cm/s is used to account for any minor imperfections in the wall. Some plastic fines may need to be imported to meet this criterion.

- The slurry wall will have a minimum of a five-foot overlap at corners.

- The slurry wall will be constructed vertically.

- Slurry levels will be maintained at least seven feet above the groundwater table during construction. Depending upon the groundwater levels along the southern leg of the wall for the ESWEMS pond, this will likely require the construction of a berm to raise the ground level at several locations along the specified alignment.

- Extensive quality control measures should be taken to assure that the S-B slurry wall is constructed without gaps or windows.

- Because the overburden aquifer contains boulders, it may be necessary to use an orange peel, clamshell, chisel, or other methods to remove or penetrate through them.

If final design incorporates the flow barrier wall into an excavation support structure, sheet piling, concrete diaphragm walls, intersecting caissons or secant piles, or cofferdams should be considered. All aspects of ground support and excavation stability will require extensive additional evaluation and detailed designs beyond the scope of this evaluation.

Appendix D of Reference 10.6, which is attached to this report as Appendix A, estimates potential flux rates through the flow barrier wall when the maximum gradient is established. Assuming that the in-situ hydraulic conductivity will achieve 1 x 10^{-6} cm/s, flux across the wall is estimated at approximately 8 gpm. If the design criterion of 1 x 10^{-7} cm/s is achieved, the corresponding flux rate is about 1 gpm. If the barrier wall is discontinuous over 1 percent of its vertical surface area due to gaps or windows, excess inflows approaching 5,000 gpm might occur. This finding underscores the need for adequate quality assurance and quality control (QA/QC) during construction. Furthermore, it indicates that if the wall is discontinuous, the presence of discontinuities should be obvious shortly after the initiation of interior dewatering as the excavation proceeds downward.

Operation of the barrier wall and interior dewatering system should include a piezometric monitoring program to compare expected groundwater withdrawals and drawdown rates with those calculated in advance. This program should include continuous monitoring of the existing and proposed monitoring wells or piezometers at select locations. Data logging pressure transducers with remote telemetry are recommended for this purpose so that head levels may be continuously monitored during initial drawdown and later during the extended phase of construction activity. If any windows or gaps in the flow barrier are indicated by the piezometric monitoring program, then pressure grouting or other remedial measures will be necessary to correct these deficiencies. Additional groundwater monitoring wells may be warranted in the immediate vicinity of significant repairs to the flow barrier wall.

6.8.3 Conceptual Dewatering System Design With a Flow Barrier

When determining the spacing between wells within the flow barrier for the ESWEMS pond, they can be spaced at greater distances than without a barrier, since the flow barrier will effectively prevent inflows.

Considering the use of the flow barrier along the preliminary alignment, dewatering wells may be located as shown on Attachment H. A total of approximately 14 dewatering wells appear appropriate when the flow barrier is utilized. Given the number of wells required and the potential flows (steady state total in flow of 350 gpm, for the three excavations evaluated) that such a system might develop, individual pumps can be sized for a maximum flow rate of 150 gpm each.

The model of this dewatering strategy suggests that interior dewatering might require a steady-state flow on the order of 230 gpm at the ESWEMS pond excavation; however, the actual flow may be less as discussed in paragraph 6.7.3.

Appendix D of Reference 10.6 (which is attached to this report as Appendix A), determined the volume of groundwater contained in the saturated pore space of the soils within the ESWEMS flow barrier to be approximately 166 acre-feet. Approximately 85 days are required to remove this stored water (not considering inflow from upward flow through the soil rock interface or through the barrier wall), at a flow rate of 600 gpm (Reference 10.6). During the initial dewatering, the inflow through the rock interface and flow barrier can be estimated as one half the steady state flow rate. Thus if 85 days are scheduled to drain the saturated soils within the ESWEMS flow barrier, the average flow rate during initial dewatering is approximately 715 gpm (600 gpm + (0.5 x 230 gpm) = 715 gpm [1.6 cfs]).

A second stage of water table depression to the shale surface or near the shale surface may require the use of vacuum well points positioned as shown on Attachment H. Each of the headers shown will draw water from well points that are typically 2-in. diameter that may be drilled, driven, or jetted in if conditions allow. Each header will need to be connected to its own vacuum pump. Individual vacuum pumps will need to be sized based on conditions encountered and the length of each header.

Final stages of the dewatering conceptual design include the excavation of trench drains and sumps into the exposed bedrock surface in front of the toe of the slope at the base of the ESWEMS excavation. Such trenches might be excavated 3 to 5 feet wide, and 2 to 3 feet deep, and sloped to collection sumps for ejection from the excavation. Groundwater flow from the bedrock is expected to vary over a wide range, and additional trenches or sumps might be needed at locations to be determined. Three such trenches were incorporated into the digital flow model at the ESWEMS pond as shown on Attachment H.

Groundwater observations at the NI and cooling towers excavations suggest that little saturated overburden is present in either area. It is therefore expected that groundwater inflows may be controlled using trench drains cut into the bedrock at the locations and elevations suggested on Attachments H (NI) and G (cooling towers). The trench drains can be sloped to sumps where the water can be pumped to the TGWSP or other disposal points, if gravity drainage to the ponds cannot be established.

Operation of this conceptual dewatering system should be less sensitive to brief interruptions in electrical power because the flow barrier will retard inflows to the excavation. However, provisions for convenient maintenance should still be included for all system elements as needed for a project duration approaching 3 years.

6.9 Disposal of Groundwater

As stated in Section 6.7.3 above, the steady state discharge from a dewatering system without the use of a seepage cutoff wall would be approximately 1040 gpm (approximately 1.5 million gallons per day [mgd]). Considering the use of a seepage cutoff wall around the ESWEMS excavation, the discharge will be reduced to an estimated flow of 350 gpm (0.5 mgd). For this report, an average value of 350 gpm (0.5 mgd) will be considered as the average daily quantity of water that will be discharged with the installation of a competent seepage cutoff wall and after steady state conditions are established.

There are several options for the disposal of the groundwater pumped from the excavations. PPL may or may not choose to implement any one or more of these options. They include:

- Discharge into the Susquehanna River.
- Temporary storage/sedimentation in the temporary groundwater storage pond (TGWSP) or other discharge ponds, with or without infiltration into the overburden prior to release.
- Injection / infiltration into the overburden (away from the excavation) to replenish the drawdown in groundwater levels.
- Treatment for human consumption.
- Used for various construction activities, such as:
    - Dust control;
    - Water for compaction control of fill and backfill; and
    - Concrete mixing.

The use of injection wells to replenish the drawdown in the groundwater level in the overburden soils can be considered, but these wells have a tendency to clog due to sedimentation or fowling and may require extensive maintenance. Therefore, the potential use of injection wells to maintain the groundwater levels in the nearby wetlands is not feasible or recommended.

Water obtained from the dewatering activities will not be used for human consumption. A potable water line would be constructed from a local municipality (Reference 10.7, Section A4.2.1.3).

There is the possibility that the amount of water extracted during dewatering will trigger the need for a Susquehanna River Basin Commission (SRBC) Groundwater Withdrawal Permit. Also, Pennsylvania DEP Regulation §110.201 has a requirement: "The following persons shall register the information specified in §110.203 (relating to content of registration) with the Department: (3) Each person whose total withdrawal from a point of withdrawal, or from multiple points of withdrawal operated as a system either concurrently or sequentially, within a watershed exceeds an average rate of 10,000 gallons per day in any 30 day period."

Of these disposal options, the most likely beneficial uses are for construction activities and to aid in recharge of the overburden soils and associated wetlands in the vicinity of the ESWEMS excavation. These likely uses are discussed in Section 6.10.

Even with the installation of the seepage cutoff around the excavation, there will be some drawdown of the water within the wetlands south of the NI as noted in Reference 10.6. The use of the pumped water to restore the groundwater level in this wetlands area would be beneficial. The surface water present in the wetlands at the site is hydraulically connected to the groundwater. Therefore, the water chemistry is very similar (Reference 10.10). The various water quality components tested from the shallow bedrock wells also indicated similar values for these components. Thus, the direct discharge of any groundwater pumped from the excavation would not have any detrimental chemical effect on the water in the wetlands. However, direct discharge would require permits, a sedimentation basin, a suitable area with erosion protection measures, and a controlled outlet. If the discharge water is pumped directly into the temporary groundwater storage pond (TGWSP) to be constructed on the southeast side of the NI, then the outlet facilities of the pond would provide the necessary controlled outlet and erosion protection. Since the in situ soils are granular and permeable, the water pumped from the excavation would naturally infiltrate through the bottom of the pond and replenish the wetlands naturally. Additionally, waters discharged from the TGWSP into Walker Run (if allowed) will aid in the recharge of the wetlands since Walker Run has a granular bottom. It is important to construct this TGWSP as one of the first construction activities for this project.

It was stated in Reference 10.7, Section A4.2.1.3, that the water obtained from the dewatering activities would not be used for human consumption and is no longer a consideration for water reuse. A potable water line would be constructed from a local municipality.

The anticipated maximum flow which may be discharged to the Susquehanna River during dewatering activities (with proper permitting) could be considered to be the average value of steady state discharge from the dewatering systems for all areas of 0.5 mgd (350 gpm). This flow is well within the design parameters for the 24 inch CWWRP blowdown discharge drain (if used) which will have a flow capacity of 9356 gpm (Reference 10.14, Section 4.1).

6.10 Beneficial Water Reuse

The most beneficial uses of the groundwater pumped from the excavations would be reuse as a source of non-potable water for construction use and replenishment of the wetlands.

Construction uses for non-potable water include dust control for the construction roads and water to be used for moisture conditioning of fill during placement and compaction. Approximately 40,000 gallons of water per day will be required for dust control (Reference 10.7, Table 4.2-1, Note d).

Approximately 1.3 million cubic yards (cy) of granular and cohesive backfill will be placed from the top of competent rock to the bottom of foundations or plant grade, where applicable (Reference 10.4, Table 3). This fill volume does not include fill placed around the site for general site grading operations or the concrete fill beneath select safety related structures. Estimating an addition of 2 percent (approximately 2.5 pounds of water per cubic foot of material) moisture to material for soil placement and compaction, a total of 10.5 million gallons will be required ($1.3x10^6$ cy x 27 cf/cy x 2.5 lbs/cf / 8.34 lbs/gal = $10.5x10^6$ gallons). Considering 180 days per year for 3 years of work, the daily usage would be approximately 19,000 gallons per day [$10.5x10^6$/(180 x 3) = 19,000].

Concrete mixing requires the use of potable water to preclude the addition of impurities to the concrete that may result in improper strength in the concrete. Based on the groundwater quality data available from on-site pumping tests (Reference 10.10, Table 2.3-41), the water to be extracted during dewatering appears to be acceptable for concrete mix water; however, test batches should be performed per ASTM C 1602 (Reference 10.11) when non-potable water is used. It is estimated that 2,220,000 gallons of water will be required per year to mix and cure concrete (Reference 10.7, Table 4.2-1). Considering concrete placement 250 days per year, this equates to 8,900 gallons per day (2,220,000 / 250 = 8,900).

The anticipated daily average beneficial water use in construction activities is approximately 68,000 (40,000 [dust control] + 19,000 [soil compaction] + 8,900 [concrete mixing and curing] = 67,900 say 68,000 gallons per day), which is substantially less than the anticipated average daily flow of 500,000 gallons per day anticipated from the dewatering systems. The remaining 432,000 gallons per day could, with proper evaluation and permits, be used to recharge the wetlands near the ESWEMS excavation.

The surface water present in the wetlands at the site is hydraulically connected to the groundwater. Therefore, the water chemistry is very similar (Reference 10.10). The various water quality components tested from the shallow bedrock wells also indicated similar values for these components. Thus, the direct discharge of any groundwater pumped from the excavation would not have any detrimental chemical effect on the water in the wetlands. However, direct discharge would require permits, a sedimentation basin, a suitable area with erosion protection measures, and a controlled outlet.

With proper design and construction, the TGWSP (1.5 acre pond - Reference 10.8) could act as a natural recharge facility to the wetlands near the ESWEMS excavation. Since the steady state dewatering system flow rate (with a flow barrier) minus the anticipated average beneficial use for construction is approximately 432,000 gallons per day, approximately 1.3 acre-feet/day is available for recharge to the wetlands (432,000 gallons/day / 7.48 gallons/cf / 43,560 sf/acre = 1.3 acre-feet/day). This indicates that if exfiltration rates through the pond floor are established and maintained in excess of 10.4 inches/day (1.3 acre feet/day x 12 inches/foot / 1.5 acres = 10.4 inches/day), under average conditions the dewatering system effluent would not discharge into the wetlands via the discharge structure. If the exfiltration rate is less than 10.4 inches/day, the excess effluent from the dewatering systems, with proper permits, could be released into the adjacent wetlands via the discharge structure.

The final design of the TGWSP should consider both the steady state flow from all three excavations as well as peak flows from the ESWEMS dewatering system startup combined with the flows from the other excavations to the extent they will have concurrent flows based on the construction sequencing. It is important to construct this TGWSP as one of the first construction activities for this project.

In summary, the most prudent approach for the disposal of the water pumped from the excavation would be to pump it directly into the TGWSP located southeast of the NI. This pond could act as a natural recharge facility to the wetlands near the ESWEMS excavation. Water for beneficial use in construction (dust control, fill conditioning and concrete mixing and curing) could be extracted from the TGWSP. A pumping facility could easily be established adjacent to this detention pond for ease of extraction. No additional storage facilities (tanks) would need to be constructed. However, the use of a storage tank for water, if it was to be used for concrete mixing, may be prudent for ease of testing. The excess water from the TGWSP could then flow through the controlled outlet structure and into the wetlands and Walker Run.

6.11 Environmental Effects

Infiltration may be required for the disposal of water produced from dewatering activities. However, if disposal into ponds is allowed, some of the discharge from the construction dewatering system will potentially directly enter the surrounding environment through overflow from these detention ponds (sedimentation basins).

As such, prior to land disturbance and construction, an NPDES Stormwater Discharge Permit (PAG-2) will be required. The major components of the permit include:

- Notice of Intent (NOI);
- Erosion and Sediment (E&S) Control Plan;
- Pennsylvania Natural Diversity Inventory (PNDI) Search;
- Post Construction Stormwater Management (PCSM) Plan;
- Thermal Impact Analysis; and

- Antidegradation Analysis.

Walker Run is classified as a wild trout stream by the PA Fish and Boat Commission (PFBC). The wetlands associated with such a stream are considered "exceptional value" by the PA Department of Environmental Protection (PADEP). It may not be possible to obtain a "General NPDES Permit". An Individual NPDES Permit will be required, as referenced in 25 Pennsylvania. Code Chapter 92. Coordination with the Luzerne Conservation District would most likely be required. Water sampling and testing will most likely be required as part of this permit to ensure that the water contains no material detrimental to the environment (Reference 10.12).

The Pennsylvania Department of Environmental Protection does not specify a limit on the flow rate of the discharge. However, they do specify that "Best Management Practices (BMPs) be implemented to maximize infiltration technologies, eliminate (where possible) or minimize point source discharges to surface waters, preserve the integrity of stream channels, and protect the physical, biological and chemical qualities of the receiving surface water." Therefore, high discharge rates that would not preserve the integrity of the stream channel or the physical qualities of the receiving surface water may be restricted. This permit will also require the use of proper erosion control measures and other BMP, such as hay bales and silt fences for any discharges to the surface bodies of water.

Since the groundwater in the overburden aquifer and the shallow bedrock have water quality parameters similar to the existing surficial water in the wetlands and Walker Run, no detrimental effects are anticipated from disposing the pumped water into the wetlands and Walker Run or reusing it for dust control or water content control during compaction operations.

6.11.1 <u>Possible Impacts of Dewatering Without Flow Barrier</u>

The extent and magnitude of groundwater drawdown projected for dewatering without a flow barrier is shown on Attachment D (Reference 10.6, Figure 15). Review of this figure indicates deep drawdown (25 feet or more) at distances of up to approximately 800 feet south and east of the ESWEMS pond. The extent and magnitude of groundwater drawdown projected from dewatering using the flow barrier is shown on Attachment E (Reference 10.6, Figure 15), which indicates drawdowns of 5 feet extend no further than approximately 400 feet west of the ESWEMS pond. However, groundwater recharge from the groundwater storage pond (if unlined) will reduce both the magnitude and aerial extent of drawdown.

The majority of residents near the site obtain water from domestic wells. Several industries including the Susquehanna Steam Electric Station (SSES) obtain water from wells. There are six domestic use wells and one commercial use well within one-half to three-quarters of a mile from the site. Given the drawdown projected to occur during dewatering without a flow barrier, some potential exists for negative impact on nearby domestic and industrial water supply wells.

In the case where the flow barrier is utilized, little or no impact to nearby wells is anticipated.

Numerous and extensive wetlands are located both on the BBNPP site and in adjoining areas, particularly to the west, south, and east. Such features are often expressions of the natural water table at or near the surface, and are therefore quite sensitive to impact via water table depression.

If dewatering is implemented without the flow barrier, substantial adverse impact is expected on the levels of surface water and groundwater in the wetland south of the ESWEMS pond. A very small area to the northwest of the ESWEMS pond is shown with a drawdown of 5 feet, suggesting a minor potential for adverse impact to the wetland at that location. As stated in Section 6.7.3, the presence of the flow barrier may counteract this drawdown due to a slight mounding effect. A very small area of drawdown of 5 feet is also shown immediately west of the proposed power block excavation. This very small area of drawdown does not appear to extend to the wetland located west of the power block.

If dewatering is implemented utilizing flow barrier(s) around the ESWEMS and any other areas where the overburden soils are saturated, the potential for adverse impact on the wetland is significantly reduced. The actual impact is likely to be less than indicated by the model (Attachment E) because the flow barrier will be keyed several feet into bedrock. The digital model can only simulate the extension of the flow barrier to the top of the bedrock. Potential drawdown to the northwest of the ESWEMS pond appears to be nearly eliminated. Potential drawdown immediately west of the power block excavation remains unchanged since no flow barrier is used for the power block and is not expected to affect the wetland to the west.

6.11.2 Mitigation of Potential Impact

Potential impacts due to water table drawdown may be mitigated by any method that reduces or eliminates drawdown in areas beyond the excavation. Aquifer recharge is one potential method to reduce drawdown in areas where drawdown of the groundwater is not desired. This might be implemented using injection wells or by allowing exfiltration from the TGWSP if constructed without a lining. It will be difficult; however, to control extensive drawdown using these means alone if dewatering is undertaken without the flow barrier around the ESWEMS pond.

Given the physical constraints posed by the location of the site and adjoining wetlands, a vertically-oriented flow barrier, such as a S-B slurry wall, or diaphragm wall appears to be a viable and effective means to mitigate potential impacts due to projected water table drawdown. Drawdown outside the flow barrier extends mostly west and south of the ESWEMS pond as shown on Attachment E.

If the overburden soils in the northwestern quadrant of the cooling towers excavation extend below the groundwater level an additional flow barrier wall should be implemented to reduce the adverse impacts of the planned excavation.

7. CONCLUSIONS AND RECOMMENDATIONS

The following **conclusions** are based on this evaluation of the conceptual dewatering system for the construction of the BBNPP:

a. An active dewatering system will be required to lower the groundwater for the excavation to allow for construction of the foundations for the ESWEMS structures to be performed under dry conditions. The dewatering system will consist of deep wells penetrating the overburden soils down to the top of the bedrock and collector trenches or well points near the interface of the soil overburden and weathered rock.

b. A passive dewatering system (collection trenches) will be required to excavate the area where the NI and two cooling towers are to be located. Extensive excavation of both overburden soils and bedrock will be required. Based on the available data, trenches and ditches at the soil/rock interface and at select locations in the rock excavations can be designed to collect and divert any groundwater from the NI and cooling towers excavations.

c. The radius of influence of dewatering wells for the ESWEMS excavation would extend significant distances to the south and east from the site. Anticipated drawdown of 25 feet being experienced approximately 400 feet from the wells if a flow barrier is not utilized. This would result in a significant impact on the nearby wetlands. Some of the nearby wetlands could become fully dewatered.

d. The use of a flow barrier, such as a soil-bentonite slurry wall, around the ESWEMS excavation would greatly reduce the drawdown effect of the dewatering wells since the wells would be located within the limits of the flow barrier. Considering a groundwater barrier wall around the ESWEMS pond and pump house, the model forecasts drawdowns will be much less than the simulation without the flow barrier. Drawdown greater than 5 feet is focused immediately outside (west and southwest) the flow barrier. These impacts should be characterized in the Environmental Assessment and the Permanent and Temporary Wetland Impact Report.

e. There is the potential for significant water seepage through the bedrock in the bottom of the NI, cooling towers and ESWEMS excavations. The numeric groundwater model calculated the flow collected from the rock portion of the three excavations to be approximately 0.62 cfs (280 gpm). However, this calculated flow rate is based on the mean value of hydraulic conductivity from pump tests, which were considered conservative and resulted in higher forecast flow rates than if the values for hydraulic conductivity had been chosen. Trenches and ditches will most likely be required in the bottom of the excavation to direct this upward flow through the rock away from the center of the excavation to the perimeter ditches. Sumps and pumps will be utilized to remove this water from the excavation.

f. With a competent flow barrier around the ESWEMS excavation and no barrier around the NI and cooling towers excavations, inflow into the three excavations considered (through the flow barrier and up through the bedrock) is anticipated to be 0.78 cfs (350 gpm). The initial flow rate, to remove the groundwater from within the flow barrier, will be contingent upon the time period allowed. If 85 days are scheduled to remove the water from within the flow barrier of the ESWEMS (not considering initial flow from NI and cooling towers excavations), an average flow rate of approximately 1.6 cfs (715 gpm) would be required from within the ESWEMS barrier wall.

g. Direct discharge of the groundwater into Walker Run will most likely not be permitted. The use of a detention/sedimentation pond and the use of Best Management Practices to reduce the total solids in the runoff will be required. Disposal of water produced from dewatering activities will most likely be accomplished by allowing infiltration from the TGWSP and possibly other ponds provided bottom liners are not installed to prevent infiltration. During periods of excessive flows from the excavations due to precipitation or at the start of pumping, the excess water will likely be allowed to settle and thermally stabilize before discharge directly into the Susquehanna River via the Combined Waste Water Retention Pond blowdown pipeline if the pipeline has been installed. If disposal in surface water or wetlands is allowed, an NPDES permit will be required at a minimum."

h. There is the possibility that the amount of water extracted during dewatering will trigger the need for a Susquehanna River Basin Commission (SRBC) Groundwater Withdrawal Permit. Also, Pennsylvania DEP Regulation §110.201 defines the filing requirements.

i. The water removed from the excavation should be suitable for reuse as dust control, soil compaction, and concrete mixing and curing based on the available water quality information. Some testing of the water will be required if it is to be used for concrete mixing.

j. The ESWEMS pipeline will be constructed above the groundwater level, thus a dewatering system is not required.

k. The Temporary Groundwater Storage Pond will most likely be constructed above the groundwater level, thus a dewatering system will not be required.

l. The Combined Waste Water Storage Pond will most likely be constructed above the groundwater level, thus a dewatering system will not be required. Trenches to divert the groundwater in the northwest corner where rock is present may be needed.

The following **recommendations** for a dewatering system are based on this evaluation of the conceptual dewatering system for the construction of the BBNPP:

a. A flow barrier, such as a soil-bentonite slurry wall should be installed around the ESWEMS excavation, which includes the pump house. One continuous wall is recommended for the portions of the excavation where water bearing overburden (sand and gravel) will be encountered. The flow barrier would be installed by keying it into the underlying bedrock.

The minimum design permeability of the flow barrier is 1 x 10^{-7} cm/s with an approximate thickness of three feet.

b. With a flow barrier around the ESWEMS excavation, a total of 14 dewatering wells, as shown on Attachment H, will be required to create and maintain a dry condition at the bottom of the excavation. These wells should have a capacity of up to 150 gpm. If a build up of groundwater occurs on the north side of the ESWEMS excavation or extreme levels of seepage are encountered, additional pumping wells can be integrated into the pumping system. To control seepage at the interface of the soil and rock, a series of well points is also shown on Attachment H.

c. Sufficient ditches and trenches should be installed at the soil/rock interface in the NI and cooling towers excavations to preclude groundwater from flowing into the excavations. Based on the available data, flow barriers are not required for the NI and cooling towers excavations.

d. Trenches will be required in the underlying bedrock in the bottom of the NI, cooling towers and ESWEMS excavations to direct any up flow of groundwater through the rock to the perimeter ditches where it can be removed through the use of sumps and pumps.

e. The Temporary Groundwater Storage Pond, to be located south east of the NI should be constructed prior to any dewatering activity. This pond can be utilized as the detention and release point for the discharge from the dewatering systems established for the ESWEMS, NI and cooling towers.

f. The Combined Waste Water Retention Pond, the Temporary Sediment Basins and possibly the Essential Service Water Emergency Makeup System Pond could be used as depositories for dewatering outflow, if they are constructed prior to the completion of all on site dewatering activities.

g. The existing monitoring wells should be utilized to monitor the effectiveness of the temporary construction dewatering program. Additional monitoring wells should also be installed to provide adequate monitoring on all four sides of each excavation. The monitoring program should include recording water levels on both the inside and outside of the flow barrier at the ESWEMS excavation.

h. If the monitoring wells indicate an open window within the flow barrier, remedial measures, such as pressure grouting, will be required to mitigate this condition.

i. Prior to implementation of dewatering using the conceptual designs provided with this evaluation, the subsurface conditions along the alignment of the proposed flow barrier and along the horizontal limits of the planned excavations should be better defined using soil borings advanced several feet into the underlying competent bedrock. Such borings should be advanced on 100 foot centers (or less) along the flow barrier alignment for the ESWEMS excavation and at 200 foot centers (or less) along the perimeter of the excavations for the NI and cooling towers, and if significant variations in bedrock elevation or groundwater conditions are encountered, additional borings or wells should be advanced to assess conditions in such areas.

j. Groundwater conditions at the northwest corner of the cooling towers excavation should be defined by advancing additional borings and by installing monitoring groundwater monitoring wells in the overburden and upper bedrock. The required extent of excavation for the cooling towers should also be reevaluated once the additional data is available.

k. The groundwater model was constructed using the available data. Since the exploratory testing to date is based on low flow pump and packer tests along with slug tests, this testing may not have stressed the aquifer sufficiently to allow a complete understanding of the flow regime in the fractured rock. To further evaluate the potential fractured flow regime and the potential aerial extent of dewatering in the fractured rock, a long-term high-flow-rate pump test program can be implemented.

l. Conceptual evaluations presented herein should be reviewed to consider additional data and information as it becomes available at the end of the 12-month monitoring period and the conceptual designs further refined and developed to provide final designs suitable for use in construction.

8. LIMITATIONS

This conceptual construction dewatering evaluation was performed consistent with the principles of hydrogeology in accordance with the prevailing standards for professionals practicing under similar circumstances in the same geographical area. This warranty is in lieu of all other warranties either expressed or implied.

This evaluation is conceptual in nature, and the conceptual evaluations presented herein will require confirmation and refinement prior to development of final designs for the purposes stated herein. The input data and information considered during this evaluation were developed primarily by others. The soil and groundwater conditions in areas between soil borings and wells are interpolated or extrapolated, and the actual soil and groundwater conditions may differ from those considered in this report.

The following specific technical qualifications and limitations should be considered by the users of this report:

a. This evaluation was prepared using subsurface characterization data that are limited in several respects. Relatively few exploratory borings were drilled in the area of the cooling towers and ESWEMS pond. Actual subsurface conditions, including the depth to bedrock, are therefore uncertain in these areas and may differ significantly from the interpolations and extrapolations used to develop the excavation plans and groundwater potentiometric surface maps (prepared by others), which were used in this evaluation.

b. Groundwater mass budgets, flow rates, projected drawdowns, and projected dewatering system yields are estimated based on digital flow models and manual calculations using available hydraulic conductivity and specific yield data. The actual groundwater flow system may therefore differ from the conceptual models used in the digital and manual calculations.

c. The dewatering operations, without a flow barrier and to a lesser extent with a flow barrier, evaluated herein will locally stress the groundwater flow system. The aquifers' actual response to such stress (e.g., actual dewatering system flow rates, basin drawdown, and changes in the mass flow budgets) has not been verified at high rates of test pumping and may therefore vary significantly from the estimates projected herein.

9. ATTACHMENTS AND APPENDICES

This report includes the following Attachments and Appendices.

Attachment A – Reduced Scale SUPP, Reference 10.1 (1 Page)

Attachment B – Construction Excavation Plan, Reference 10.6 - Figure 3, which is based on Reference 10.4 (4 Pages)

Attachment C - Groundwater Monitoring Wells (Location Plan), Reference 10.3 - Figure 1 (1 Page)

Attachment D – Drawdown in Overburden Aquifer Without Flow Barrier at ESWEMS, Reference 10.6 - Figure 15 (1 Page)

Attachment E – Drawdown in Overburden Aquifer With Flow Barrier at ESWEMS, Reference 10.6 - Figure 16 (1 Page)

Attachment F – Conceptual Dewatering Strategy Power Block and ESWEMS Without Flow Barrier, Reference 10.6 - Figure 20 (1 Page)

Attachment G – Conceptual Dewatering Strategy Cooling Towers, Reference 10.6 - Figure 21 (1 Page)

Attachment H – Conceptual Dewatering Strategy Power Block and ESWEMS With Flow Barrier, Reference 10.6 - Figure 22 (1 Page)

Attachment I – Typical Dewatering Well Schematic, Reference 10.6 - Figure 23 (1 Page)

Attachment J – Typical Monitoring Well (Piezometer) Schematic, Reference 10.6 - Figure 24 (1 Page)

Appendix A – Weaver Boos Consultants North Central, LLC, "Evaluation of Temporary Construction Dewatering Strategies Proposed Bell Bend Nuclear Power Plant Berwick, Pennsylvania", Dated October 20, 2010, on 3 CDs.

CD-1 Evaluation of Temporary Construction Dewatering Strategies

CD-2 Visual MODFLOW Project Files (Disc 1 of 2)

CD-3 Visual MODFLOW Project Files (Disc 2 of 2)

10. REFERENCES

10.1 Sargent & Lundy LLC drawing SK-12198-400-015, Rev. 4, "Reduced Scale SUPP".

10.2 Paul C. Rizzo Associates, Inc. 2010. Response to RFI SL-BBNPP-132, Approved for Use by UniStar August 12, 2010 (Final Boring Logs).

10.3 Paul C. Rizzo Associates, Inc., Response to RFI SL-BBNPP-111, Approved for Use by UniStar August 31, 2010 (3-Month Groundwater Monitoring data Report).

10.4 Paul C. Rizzo Associates, Inc., Response to RFI SL-BBNPP-149, Approved for Use by UniStar September 13, 2010 (Excavation Plans).

10.5 BBNPP, Final Safety Analysis Report, Section 2.4.12 – Groundwater, Rev. 2.

10.6 Weaver Boos Consultants North Central, LLC, "Evaluation of Temporary Construction Dewatering Strategies Proposed Bell Bend Nuclear Power Plant Berwick, Pennsylvania", Dated October 20, 2010.

10.7 Areva, Response to RFI SL-BER-069, Approved for Use by UniStar August 19, 2008 (Water Use).

10.8 Sargent & Lundy LLC drawings:

- SK-12198-400-015, Sheet 1, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 1".
- SK-12198-400-015, Sheet 2, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 2".
- SK-12198-400-015, Sheet 5, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 5".
- SK-12198-400-015, Sheet 6, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 6".

10.9 BBNPP, Final Safety Analysis Report, Section 2.5.4 – Stability of Subsurface Materials and Foundations, Rev. 2.

10.10 BBNPP, Environmental Report, Section 2.3 – Water, Rev. 2.

10.11 ASTM International C 1602 – 06, "Standard Specification for Mixing Water Used in the Production of Hydraulic Cement Concrete".

10.12 Paul C. Rizzo Associates, Inc., Response to RFI SL-BER-070, Approved for Use by UniStar September 9, 2008 (Water Discharge).

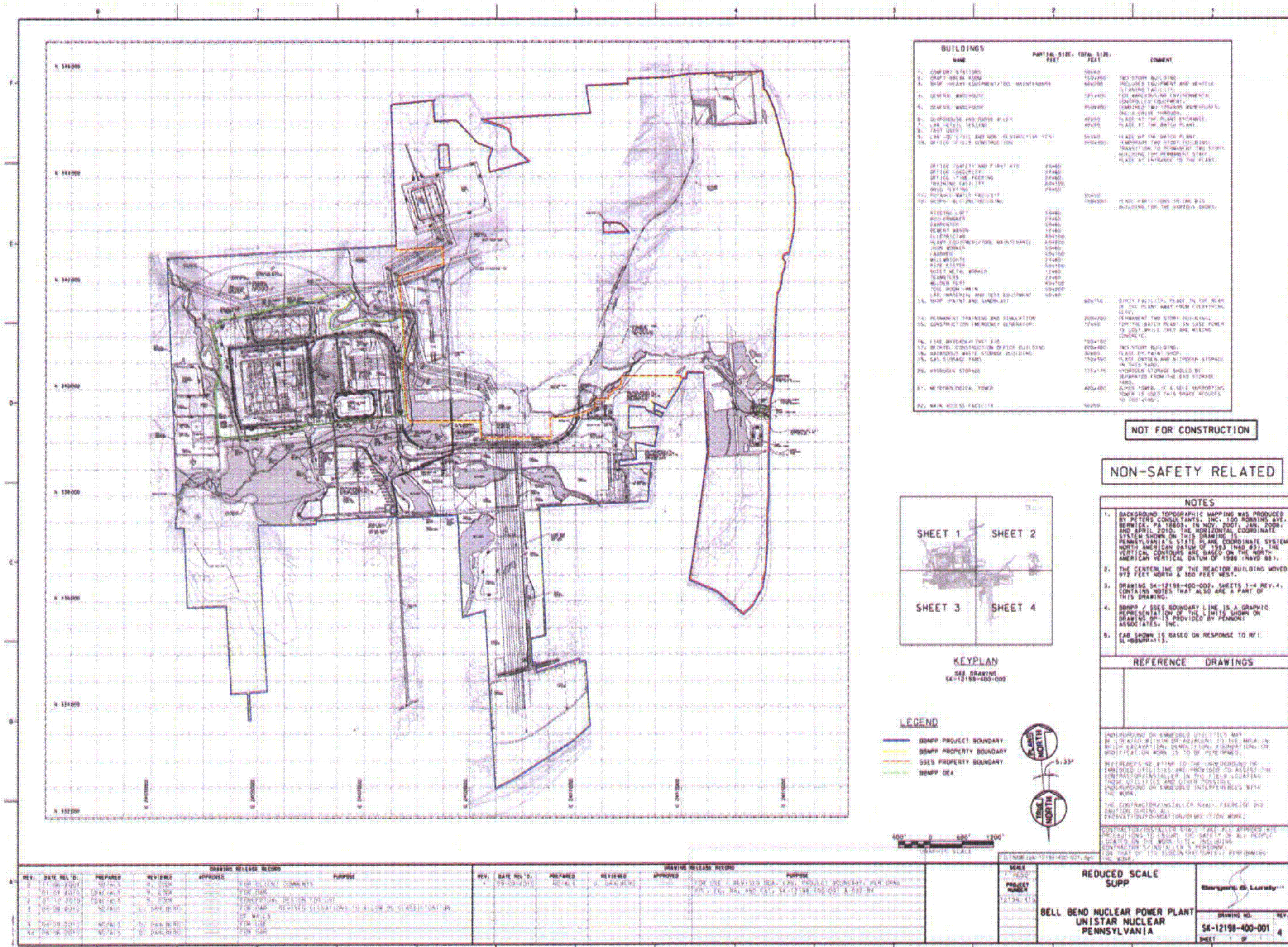
10.13 Black and Veatch, 2010. Response to RFI SL-BBNPP-143, Approved for Use by UniStar July 27, 2010 (ESWEMS Pipeline).

10.14 Sargent & Lundy, LLC, Report No. SL-0009498, "Conceptual Design of the Circulating Water System, Bell Bend Nuclear Power Plant, UniStar Nuclear Energy", Dated October 28, 2010, Revision 6.

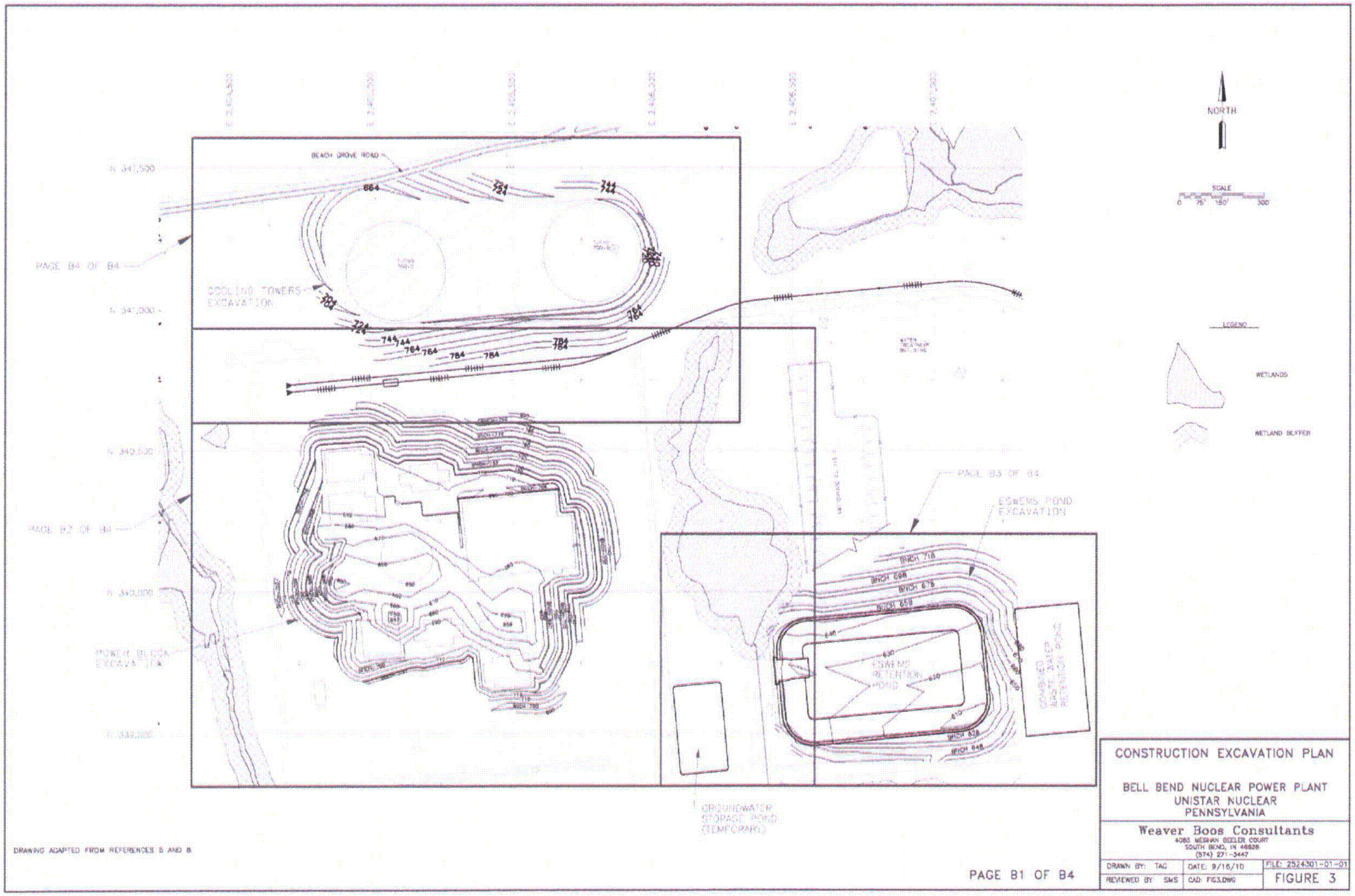
10.15 Sargent & Lundy, LLC, Report No. SL-0009446,"Conceptual Design of Stormwater Management, Bell Bend Nuclear Power Plant, UniStar Nuclear Energy", Dated July 28, 2010, Revision 5.

10.16 Sargent & Lundy LLC drawing SK-12198-400-01512198-400-CWS-003, Rev. 0, "Conceptual Combined Waste Water Retention Pond General Arrangement".

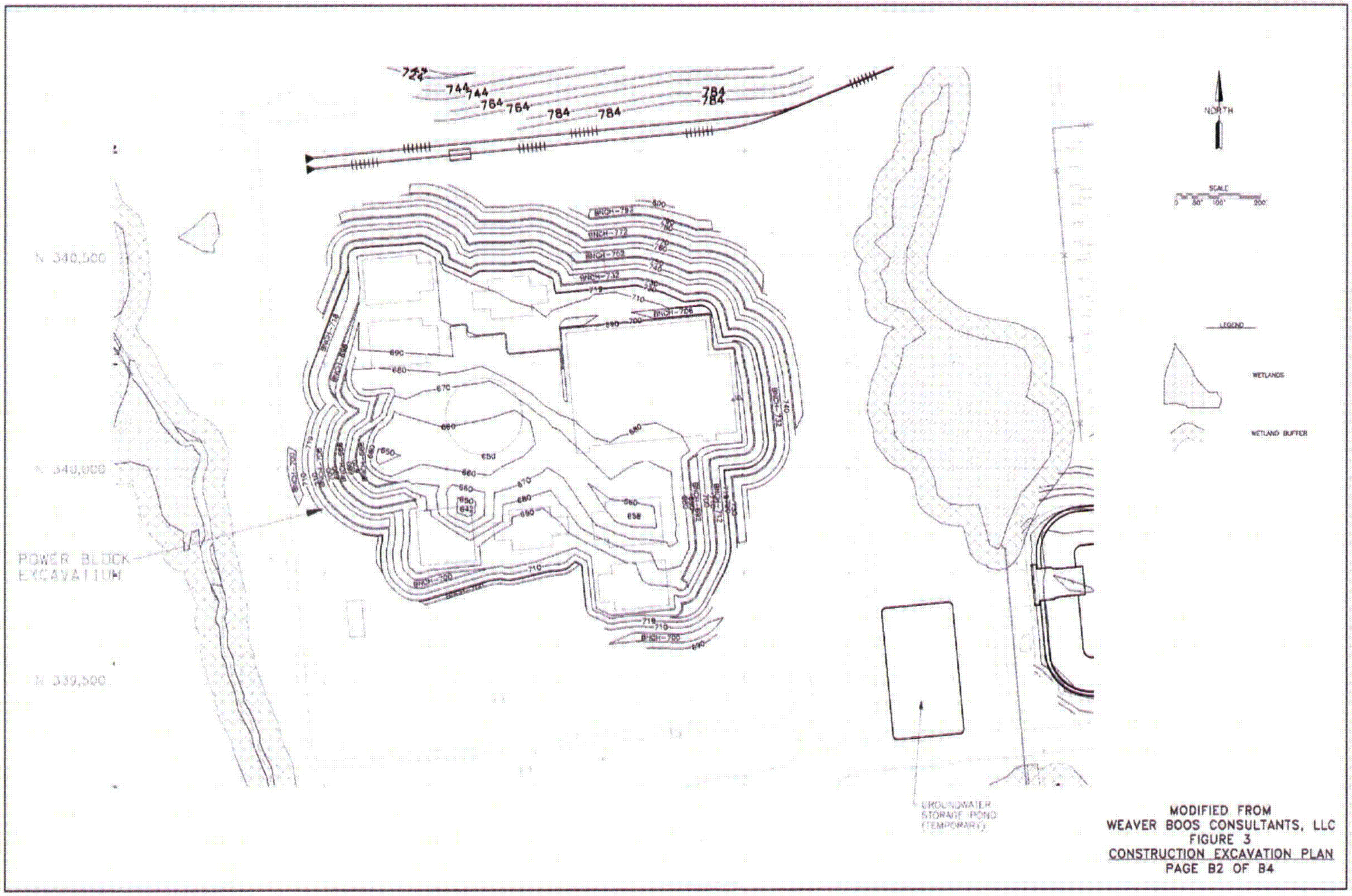
ATTACHMENT A



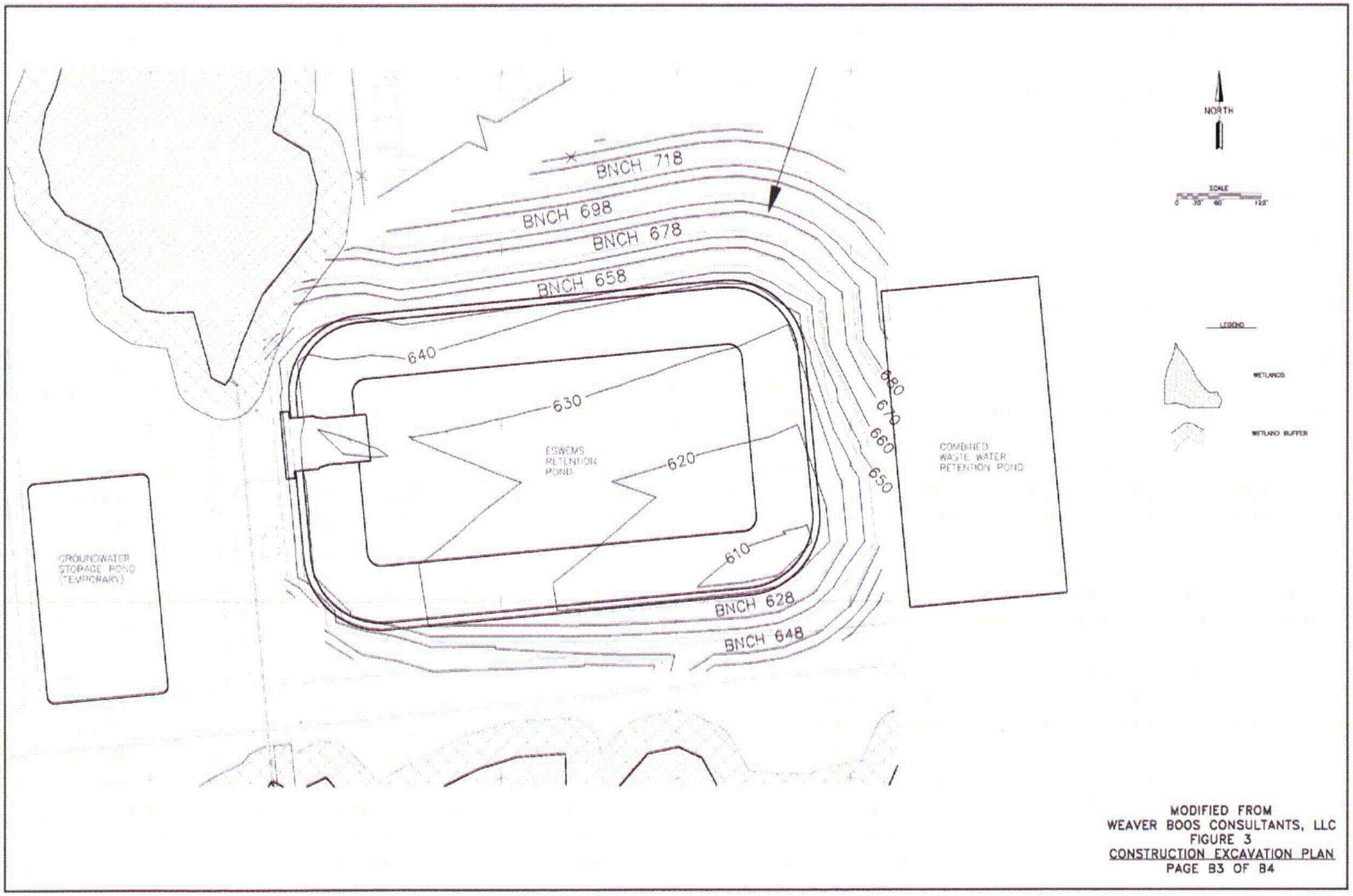
ATTACHMENT B
B1 of B4



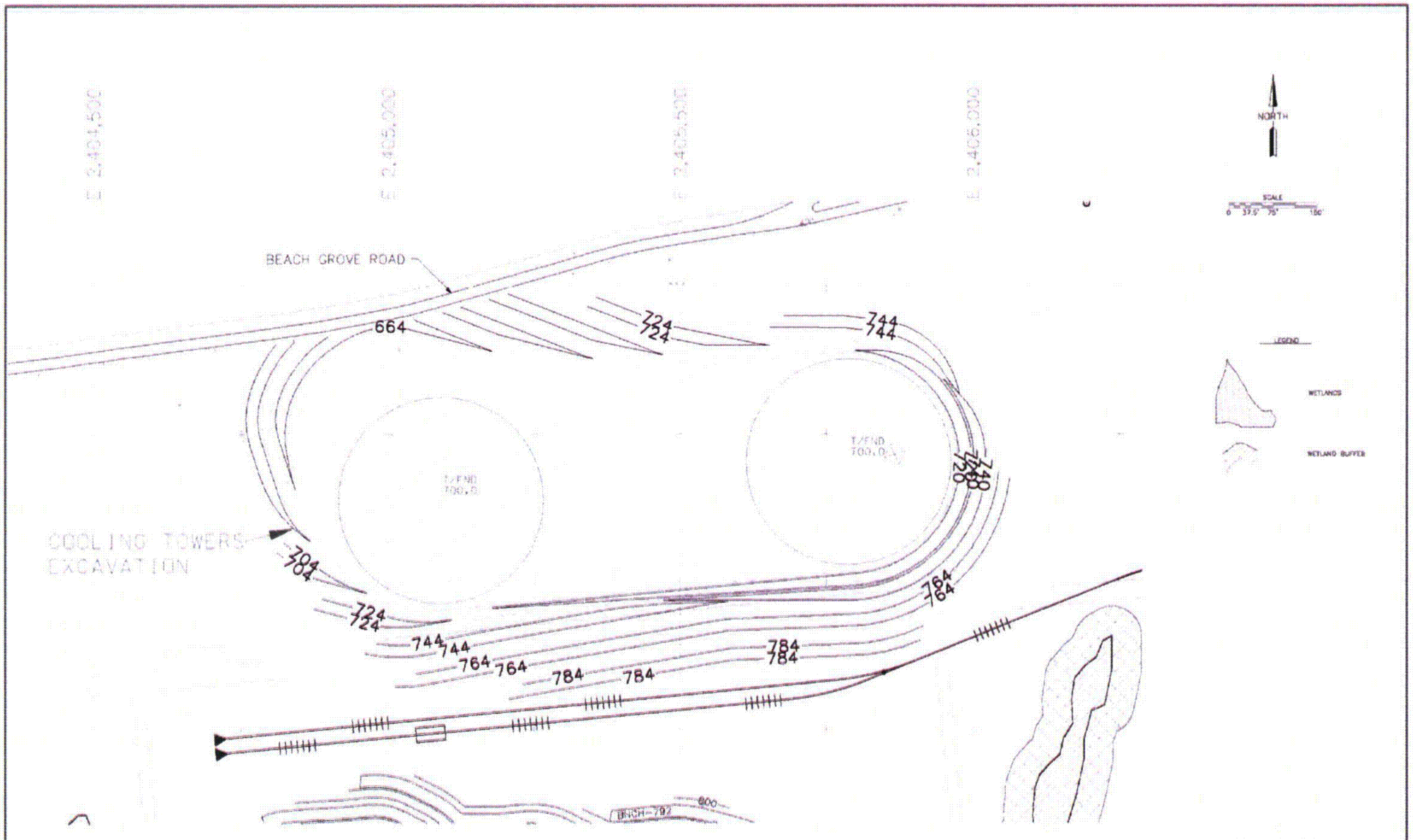
ATTACHMENT B
B2 of B4



ATTACHMENT B
B3 of B4

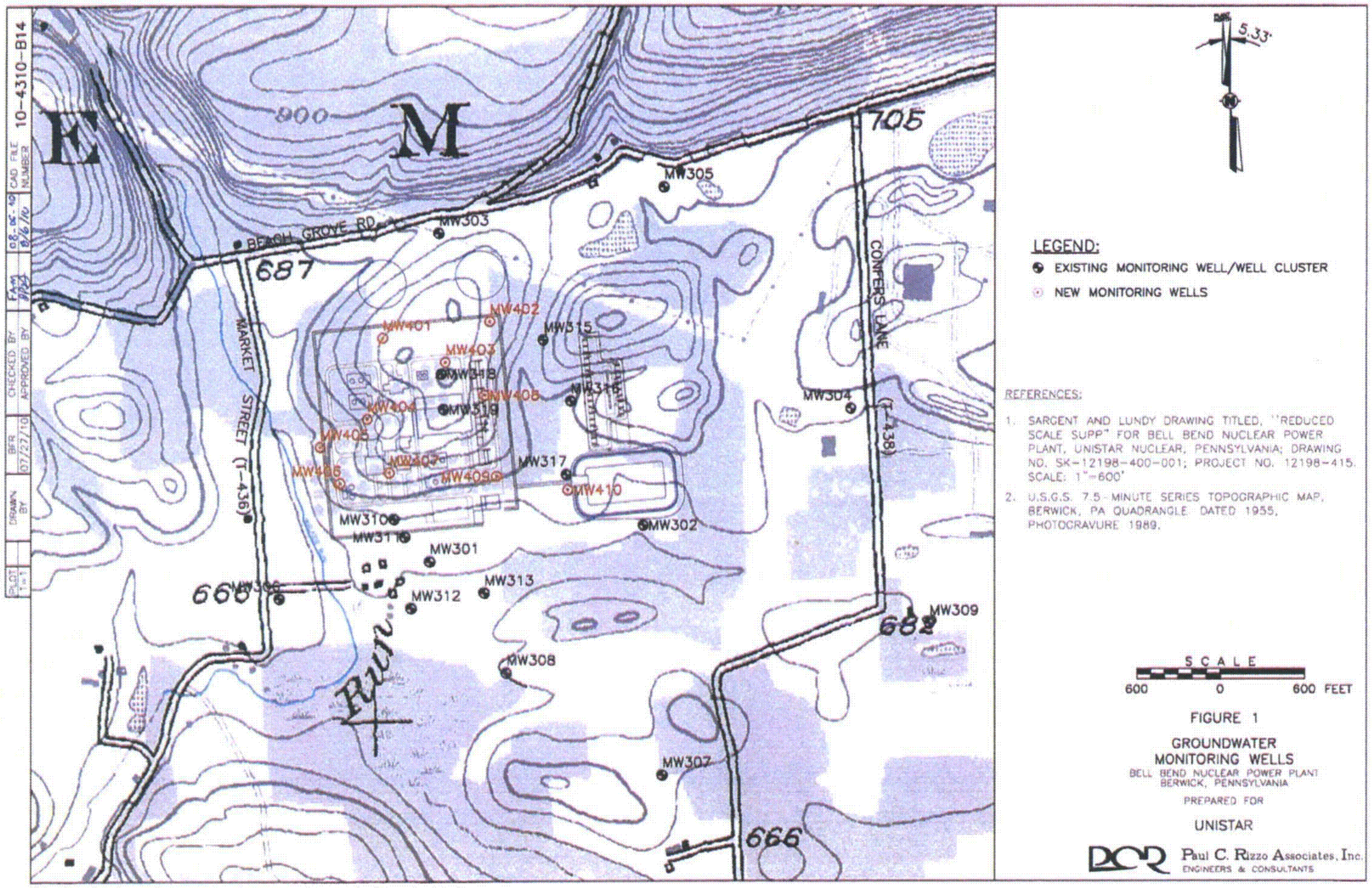


ATTACHMENT B
B4 of B4



MODIFIED FROM
WEAVER BOOS CONSULTANTS, LLC
FIGURE 3
CONSTRUCTION EXCAVATION PLAN
PAGE B4 OF B4

ATTACHMENT C



ATTACHMENT D

FIGURE 15
Drawdown in Overburden Aquifer Without Flow Barrier at ESWEMS

WEAVER BOOS CONSULTANTS
4085 Meghan Beeler Court
South Bend, Indiana 46628

Bell Bend Nuclear Power Plant - Non Safety Related
Berwick, Pennsylvania

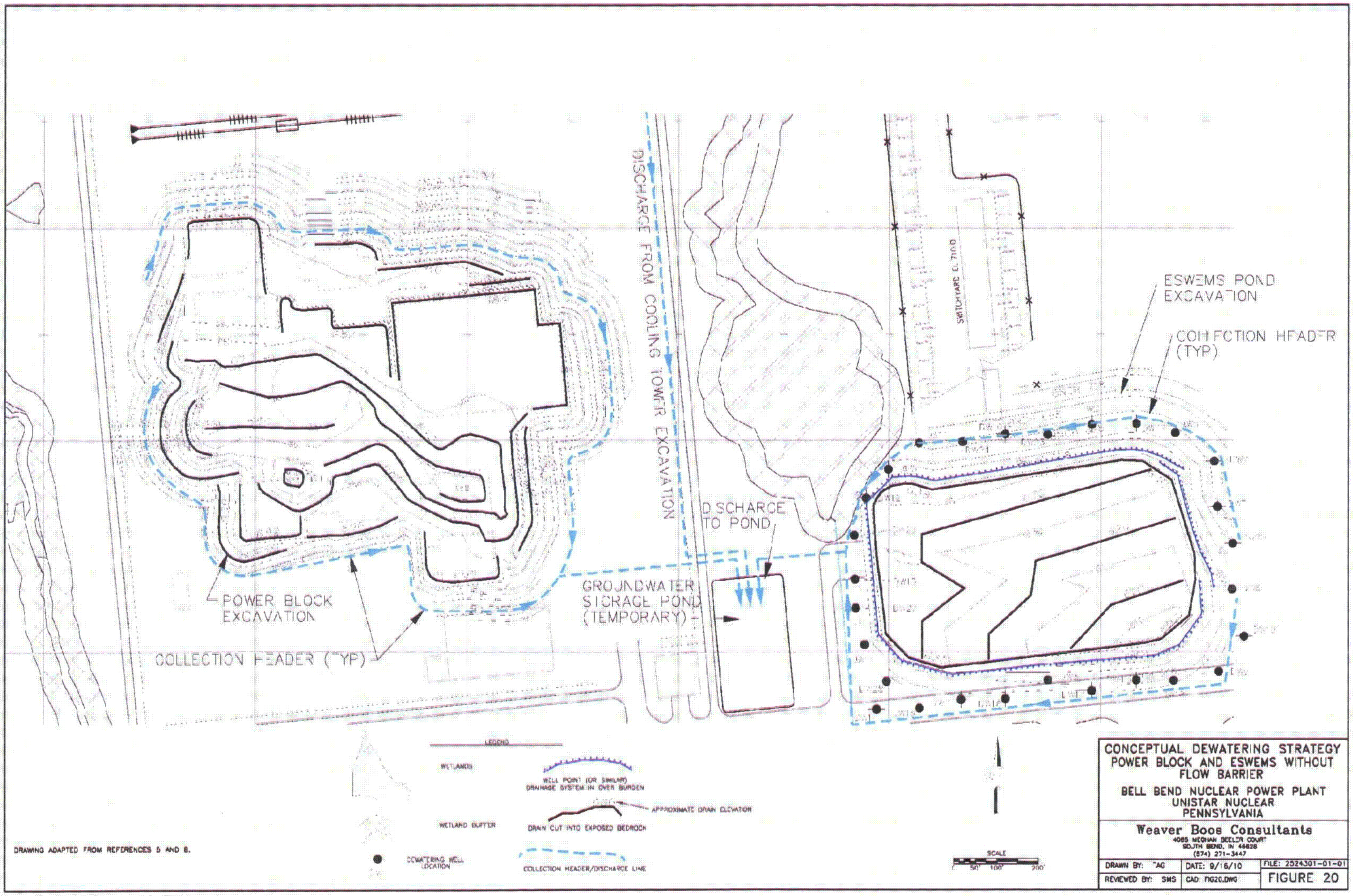
ATTACHMENT E

FIGURE 16
Drawdown in Overburden Aquifer With Flow Barrier at ESWEMS

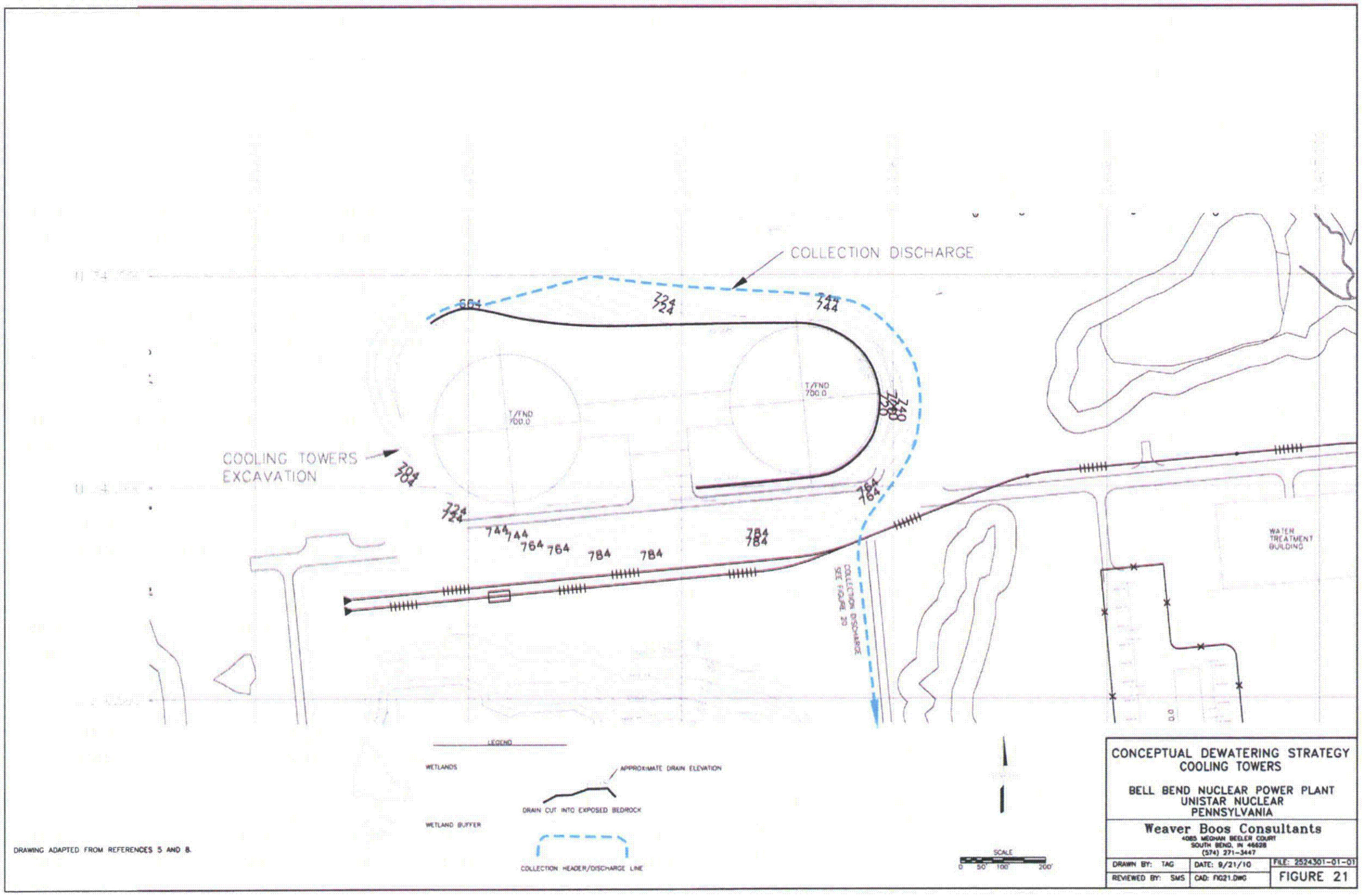
WEAVER BOOS CONSULTANTS
4085 Meghan Beeler Court
South Bend, Indiana 46628

Bell Bend Nuclear Power Plant - Non Safety Related
Berwick, Pennsylvania

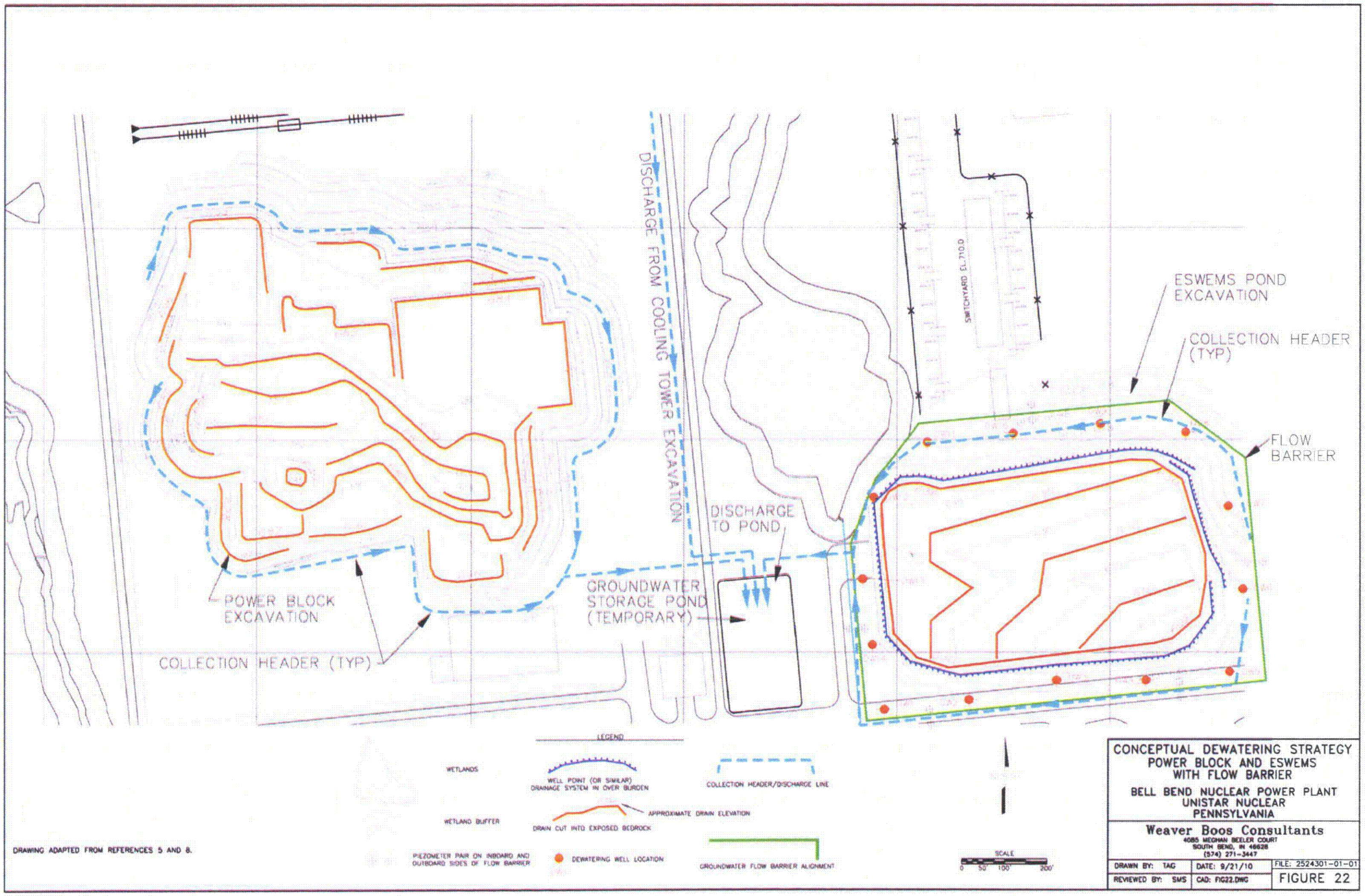
ATTACHMENT F



ATTACHMENT G

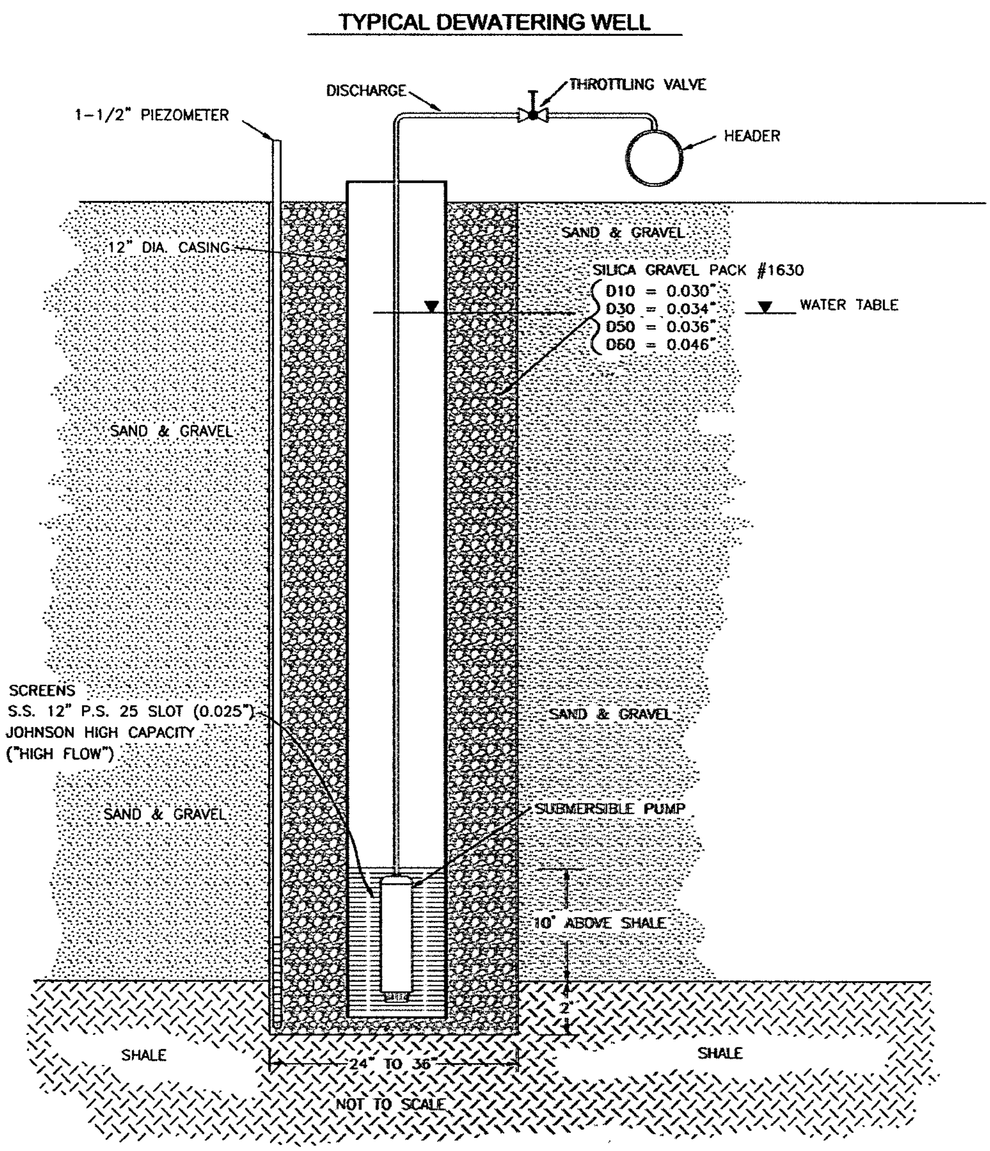


ATTACHMENT H



ATTACHMENT I

TYPICAL DEWATERING WELL



NOTES:
1) SUBMERSIBLE PUMP RATED AT APPROXIMATELY 100 TO 150 GPM
2) 1" DIA. SAMPLING PORT AT EACH PIPE DISCHARGE

TYPICAL
DEWATERING WELL SCHEMATIC
BELL BEND NUCLEAR POWER PLANT
UNISTAR NUCLEAR
PENNSYLVANIA

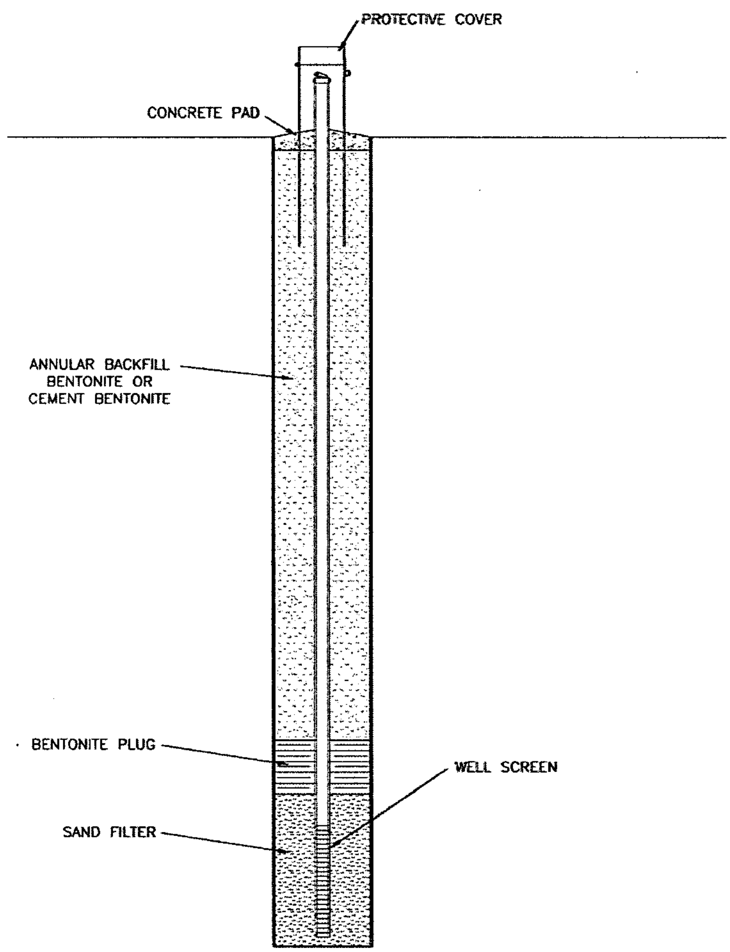
WEAVER BOOS CONSULTANTS
NORTH CENTRAL, LLC

DRAWN BY: TAG	DATE: 09/21/10	FILE: 2524301-01-01
REVIEWED BY: SMS	CAD: WELLSCHEM.DWG	FIGURE 23

K:\WBCNC\CLIENT INFORMATION\2500-2599\2524\301\01\P100133 - S&L BELL BEND 2\WELLSCHEM.DWG

ATTACHMENT J

TYPICAL 2-INCH MONITORING WELL SCHEMATIC



NOT TO SCALE

TYPICAL MONITORING WELL
(PIEZOMETER) SCHEMATIC
BELL BEND NUCLEAR POWER PLANT
UNISTAR NUCLEAR
PENNSYLVANIA

WEAVER BOOS CONSULTANTS
NORTH CENTRAL, LLC

DRAWN BY: TAG	DATE: 09/21/10	FILE: 2524301-01-01
REVIEWED BY: SMS	CAD: PIEZSCHEM.DWG	FIGURE 24

K:\WBCM\CLIENT INFORMATION\2500-2599\2524\301\01\01\P100133 - S&L BELL BEND 2\PIEZSCHEM.DWG

APPENDIX A

Weaver Boos Consultants North Central, LLC
Evaluation of Temporary Construction Dewatering Strategies
Proposed Bell Bend Nuclear Power Plant Berwick, Pennsylvania
Dated: October 20, 2010

(On Three CDs)

CD-1 Evaluation of Temporary Construction Dewatering Strategies

CD-2 Visual MODFLOW Project Files (Disc 1 of 2)

CD-3 Visual MODFLOW Project Files (Disc 2 of 2)

October 20, 2010
Project No. 2524301-01-02

EVALUATION OF TEMPORARY CONSTRUCTION DEWATERING STRATEGIES PROPOSED BELL BEND NUCLEAR POWER PLANT
BERWICK, PENNSYLVANIA

NON-SAFETY RELATED

Prepared For:

Sargent & Lundy LLC

Sargent & Lundy, LLC
55 E. Monroe Street
Chicago, IL 60603

Prepared By:

WEAVER
BOOS
CONSULTANTS

Weaver Boos Consultants North Central, LLC
4085 Meghan Beeher CT
South Bend, IN 46628

EVALUATION OF TEMPORARY CONSTRUCTION DEWATERING STRATEGIES PROPOSED BELL BEND NUCLEAR POWER PLANT BERWICK, PENNSYLVANIA

NON-SAFETY RELATED

Prepared: October 20, 2010

The above titled evaluation and report has been prepared, reviewed, and approved consistent with Weaver Boos Consultants North Central, LLC standard operating procedures.

Steven M. Stanford

Prepared and Approved by: Steven M. Stanford, Senior Project Manager

Michael B. Maxwell

Reviewed by: Michael B. Maxwell, Senior Project Manager

EVALUATION OF TEMPORARY CONSTRUCTION DEWATERING STRATEGIES PROPOSED BELL BEND NUCLEAR POWER PLANT

NON-SAFETY RELATED

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PROPOSED BELL BEND NUCLEAR POWER PLANT

NON-SAFETY RELATED

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FIGURES

EVALUATION OF TEMPORARY CONSTRUCTION DEWATERING STRATEGIES PROPOSED BELL BEND NUCLEAR POWER PLANT

NON-SAFETY RELATED

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TABLES

APPENDICES

1.0 INTRODUCTION

1.1 General

Weaver Boos Consultants North Central, LLC (Weaver Boos) has performed this evaluation of temporary construction dewatering strategies for use at the proposed Bell Bend Nuclear Power Plant (BBNPP) site located near Berwick, Pennsylvania, for UniStar. This assignment was undertaken consistent with the scope of services described in Weaver Boos Proposal No. P10013, dated June 2, 2010, as well as Sargent & Lundy, LLC authorizing Purchase Order No. 27290 dated June 16, 2010.

1.2 Background Information and Purpose

Weaver Boos understands that construction of the Bell Bend Nuclear Power Plant (BBNPP) facility will require excavation as needed for the placement of foundation systems for the power block, cooling towers, and the Essential Service Water Emergency Makeup System (ESWEMS) pond and pump house. These structures generally need to be founded on competent bedrock or on structural fill extending from the structural foundation down to the competent rock. Overlying soils in the area of the ESWEMS pond comprise a conductive overburden aquifer with a saturated thickness of up to about 56 ft (Reference 3). Overlying soils in the area of the power block and cooling towers are generally thinner and likely contain only minor groundwater perched atop the weathered surface of the bedrock (Reference 4). The underlying weathered bedrock and the competent rock also comprise aquifers, although they are much less conductive than the overburden soil.

Temporary groundwater control will be required during excavation and construction in the three areas of the site considered herein (powerblock, cooling towers, ESWEMS pond and ESWEMS pump house) (Reference 5). Groundwater will be pumped from the aquifers as needed to dewater the excavations to facilitate below grade construction activities, for the duration of below grade construction. Weaver Boos evaluated alternative dewatering strategies to assess the effectiveness and potential impacts of temporary construction dewatering both with and without the use of a flow barrier in the area of the ESWEMS pond. This approach was utilized to assess whether a slurry wall, diaphragm wall, or other type of subsurface flow barrier is needed around the ESWEMS pond to minimize groundwater intrusion into the excavation and to mitigate detrimental effects such as drainage of nearby wetlands. The scope of work for this assignment includes the following general tasks:

- Review of site characterization data;
- Preparation of dewatering system conceptual strategies;
- Estimates of groundwater flow rates, drawdown rates, and radius of influence;
- Evaluation of offsite impacts of dewatering strategies;
- Preparation of groundwater effluent discharge conceptual strategy; and
- Report submittal preparation.

The overall purpose and objective of this work is to evaluate site conditions, identify applicable dewatering strategies, evaluate their potential impacts to the nearby wetlands and neighboring well owners, and to provide conceptual strategies for dewatering under the following two general scenarios:

- Open excavation and water table depression with no groundwater flow barrier; and
- Dewatering and excavation using a subsurface flow barrier to reduce the quantity of groundwater removal and to mitigate potential impacts to potentially sensitive nearby areas.

This evaluation is based largely upon the data and information contained in the BBNPP Final Safety Analysis Report (FSAR), Section 2.4.12 – Groundwater, Rev. 2 (Reference 9, which presents data and information developed during 2007 and 2008, and more recent data and information reported by Paul C. Rizzo Associates, Inc. (Rizzo) for monitoring well test data obtained during May, June, and July 2010 (Reference 4). Weaver Boos also reviewed and considered selected additional information obtained from published sources. Weaver Boos has not visited the site or completed any physical work at the site. The following report is therefore based upon our review of the available information and our evaluation of relevant site characterization data developed by others.

Weaver Boos notes that we previously completed a similar evaluation of temporary construction dewatering strategies for this site dated September 8, 2008. This current report provides our re-evaluation of dewatering strategies after shifting the power block and cooling towers approximately 972 ft north and 300 ft west of the previously proposed location (Reference 7). Elevation changes and several other variations from the original site plan are also considered. This evaluation supersedes our September 8, 2008 report, yet retains much of the relevant data and information we previously considered.

1.3 Report Organization

Section 1.0 introduces the facility and states the technical challenge(s) to be addressed. **Section 2.0** describes the physical setting, including geology and hydrogeology of the site and surrounding area. **Section 3.0** reviews data applicable for developing the dewatering strategies presented herein. **Section 4.0** describes a conceptual model of the site developed from the information reviewed. **Section 5.0** describes the implementation and results obtained from a three-dimensional groundwater flow model developed to evaluate applicable dewatering strategies and to assess their potential impact to nearby sensitive areas. **Section 6.0** provides conceptual strategies for temporary dewatering systems both with and without the use of a flow barrier at the ESWEMS pond. **Section 7.0** evaluates potential off-site impacts of the dewatering strategies. A conceptual strategy for dewatering effluent discharge is provided in **Section 8.0**. Conclusions and recommendations are provided in **Section 9.0**. Qualifications and limitations in connection with this work are discussed in **Section 10.0**. References cited are listed in **Section 11.0**. Selected supporting data and information are appended, as calculations supporting the conceptual dewatering strategies presented herein.

2.0 PHYSICAL SETTING

2.1 Site Location and Setting

The proposed BBNPP facility site is located in Luzerne County, Pennsylvania as shown on **Figure 1**, and is about three miles east northeast of Berwick, a borough of Columbia County, Pennsylvania. Land use at and around the site is primarily rural agricultural, except to the east, where the adjoining land is occupied by the Susquehanna Steam Electric Station (SSES). The site and immediate surrounding area is situated on an upland terrace above the North Branch of the Susquehanna River. The surface elevation at the site ranges from approximately 650 ft above the North American Vertical Datum of 1988 (NAVD88) to about 800 ft (NAVD88) (Reference 9, Section 2.4.12.3).

Development plans for the site, including the power block, cooling towers, and ESWEMS pond and pump house, are illustrated on **Figure 2** (Reference 8). Construction excavation plans for these features are illustrated on **Figure 3** (Reference 5).

2.2 Physiography

The site lies in the northeastern end of the Ridge and Valley Province in northeastern Pennsylvania. Within the Ridge and Valley Province, the site lies in the Susquehanna Lowland Section, close to the North Branch of the Susquehanna River (Reference 9, Section 2.4.12.1.1).

The northeast-southwest trending Valley and Ridge Physiographic Province extends from West Virginia and Maryland to northeastern Pennsylvania. This Province is characterized by layered Paleozoic sedimentary rocks that have been faulted and folded. Elongated mountain ridges are formed by well-cemented sandstones and conglomerates that are resistant to weathering. These ridges are typically the remnant flanks of breached anticlines. Limestone, dolomite, and shale are more easily weathered and eroded and, as a result, form the intervening valleys between the ridges. The BBNPP site is located on the north limb of the Berwick Anticlinorium (Reference 9, Section 2.4.12.1.2).

Current land use appears to be mostly agricultural, and includes fields and forested areas (Reference 12). Several areas of wetland are located on and around the site (Reference 8). Wetlands mapped nearest to the power block, cooling towers, and ESWEMS pond and pump house, are illustrated on **Figures 2** and **3**.

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2.3 Regional Geology and Hydrogeology

In the northeastern and northwestern corners of Pennsylvania, the bedrock is overlain by a variable thickness of glacial till, outwash, colluvium, kame, and kame terrace deposits of Pleistocene age. A large percentage of these surficial glacial materials were deposited during the last major glacial advance (Wisconsinan Stage). The BBNPP site lies at the edge of where the Wisconsinan glacier made its farthest advance. As a result, end moraine deposits have been mapped at the site (Reference 9, Section 2.4.12.1.1).

The total thickness of Paleozoic sedimentary rocks overlying the Precambrian crystalline basement is approximately 33,000 ft in the vicinity of the BBNPP site. The Paleozoic sedimentary rocks form a wedge that is thickest in eastern Pennsylvania and gradually thins to the north and west across the state. The sedimentary rocks include sandstone, siltstone, shale, and limestone units, with lesser amounts of coal and conglomerate of Cambrian to Pennsylvanian age (Reference 9, Section 2.4.12.1.1).

The glacial outwash and kame terrace deposits constitute some of the most permeable aquifers in the region. The outwash deposits in the Susquehanna River Valley are especially thick and permeable in some places. The highest groundwater levels generally occur in the winter and spring months each year (Reference 9, Section 2.4.12.1.2.10).

Groundwater in the Ridge and Valley Province of Pennsylvania is found and produced in almost all of the rock formations, including shales and clay shales. The area is dependent on groundwater for domestic purposes because the major public water supplies are few and generally separated by large distances. Groundwater in the bedrock formations is present primarily in secondary openings, including fractures, joints, and bedding plane separations. The size and frequency of water-bearing zones generally decreases with depth because the confining pressure increases and the fractures close as the weight of rock above increases. Thus, the hydraulic conductivities of the rock formations are expected to decrease with depth (Reference 9).

2.4 Site Investigations

Geotechnical and hydrogeological investigations completed by Paul C. Rizzo Associates, Inc. provide information to depths of 600 ft beneath the BBNPP site. Subsurface information was collected from over 110 borings and 50 groundwater observation wells drilled across the site during 2007, 2008 (References 9 and 10), and 2010 (Reference 4). The observation wells range from 1-inch to 4-inches in diameter. The locations for observation wells or clusters of observation wells are illustrated on **Figure 4** (Reference 4, Figure 1).

Of the 50 observation wells installed, 15 are screened in the glacial overburden deposits and the rest are screened in shallow and deeper bedrock. Clustered observation wells were installed at select locations to measure vertical gradients. A total of ten well clusters were installed around the BBNPP site. Upward flow of groundwater from the bedrock is apparent in roughly half of the well clusters. There are two areas of suspected upward flow from bedrock. The first area lies along Beach Grove Road along the northern boundary of the site and extends westward to Walker Run. The second area covers all of the wetlands south of the proposed power block and ESWEMS pond locations (Reference 9, Figure 2.4-97).

The hydraulic properties of the BBNPP site lithologies were characterized by several means (References 4 and 9). Observation wells were generally slug tested. Bedrock intervals in many of the borings were packer tested. Finally, pumping tests were performed in glacial overburden wells, shallow bedrock wells, and in deep bedrock wells. Results for the pumping tests are summarized in **Table 1** and are considered by Weaver Boos as most representative for use in the evaluation of dewatering strategies involving the pumping of groundwater during temporary construction dewatering. Selected site investigation data, including groundwater and surface water measurements, groundwater flow directions, slug tests and packer tests, developed by Rizzo are provided as **Appendix A**.

2.5 Site Geology and Hydrogeology

Subsurface conditions beneath the site are characterized by Pliestocene Age sand and gravel deposits ranging from 0 to 100 ft thick that are underlain by folded, jointed and fractured Devonian-Age shales of the Mahantango Formation extending approximately 1,000 to 1,200 ft beneath the site. The Mahantango Formation consists primarily of dark gray, silty to very silty claystone. Frequent joints and intense cleavage development causes the claystone to become splintery, chippy, and fragmented upon weathering. The Mahantango Formation has low to moderate resistance to weathering (Reference 9, Section 2.4.1.2.4).

Borings in the vicinity of the powerblock and cooling towers generally did not encounter groundwater in the overburden soils (Reference 3). In the vicinity of the ESWEMS pond and pump house, groundwater levels in the glacial overburden typically range from 2 to 23 ft below the ground surface (bgs) (derived from Reference 4, Table 5, and Reference 9, Table 2.4-44). Seasonal fluctuations in groundwater elevations measured in the overburden aquifer are typically less than 5 ft (Reference 9, Section 2.4.12.3.1.1). Along the south side of the proposed ESWEMS pond in the MW302-series observation wells, the depth to water is reported to be about 2 to 8 ft bgs (Reference 9, Table 2.4-44), and flows generally westerly towards Walker Run (**Appendix A Figure 5**). In the vicinity of the ESWEMS pond and pump house, the overburden aquifer is charged by downward percolating precipitation and upflow from the deeper bedrock aquifer (Reference 9, Section 2.4.12.3.1.4). Groundwater discharges from the surficial aquifer as springs and seeps into Pond G8 (shown in **Appendix A Figure 2**), the wetlands along the southern border of the site, and into Walker Run.

Hydrogeological investigations completed at the site indicate that the sand and gravel deposits comprise a conductive "overburden" aquifer with a hydraulic conductivity (K) of approximately 5.9×10^{-2} cm/s (geometric mean from pump test) and a median specific yield of 0.32 (Reference 9, Section 2.4.12.3.2.1). Generally for sand and gravel deposits, the specific yield is nearly the same as effective porosity. Therefore, the effective porosity (n) for the overburden aquifer is considered to be 0.32. Utilizing the aforementioned hydraulic conductivity and effective porosity values in conjunction with an average hydraulic gradient (i) of 0.0081, a groundwater velocity (V) of 4.2 ft per day can be considered for the overburden material by Darcy's law: $V = Ki/n$.

The greatest anticipated thickness of conductive water-bearing overburden (approximately 56 ft) requiring temporary dewatering during construction occurs near the southeast corner of the proposed ESWEMS pond. With the exception of some loose sand pockets, the overburden consists of over-consolidated, brown silty sand and sand containing gravel and large rounded cobbles and boulders. The frequency of the boulders is reported to increase with depth (Reference 9, Section 2.4.12.1.3.1.1).

The underlying Mahantango Shale Formation is also considered an aquifer. There are no extensive aquitards in the vicinity of the BBNPP site. Vertical groundwater flow in the upland areas to be developed as the power block and cooling towers is generally downward. Vertical groundwater flow is generally upward from the bedrock aquifer to the overburden aquifer in the area to be developed as the ESWEMS pond. As stated by Rizzo (Reference 9, Section 2.4.12.3.2.2), results from slug and packer tests do not conclusively support a hypothesis that there is a significant difference between the hydraulic conductivity values of the shallow and deep bedrock. However; the pumping test results (**Table 1**) suggest that hydraulic conductivity is greater in the shallow bedrock than in the deeper bedrock beneath the site.

3.0 REVIEW OF HYDRAULIC PARAMETERS FOR EVALUATION

3.1 General Requirements

The saturated overburden materials will be excavated to depths of 56 ft bgs in the area of ESWEMS pond and pump house (Reference 5). Mostly unsaturated overburden and some partly saturated shallow bedrock will be excavated in the areas of the power block and cooling towers. Excavation will generally extend downward to the surface of the Mahantango Shale Formation and deeper at selected locations to remove weathered and transitional zone bedrock. Several safety-related structures such as portions of the power block and ESWEMS pond will be founded on engineered fill (including concrete fill) bearing directly on the competent bedrock. The construction excavation plans for the power block, cooling towers, and ESWEMS pond based upon available information (Reference 5) are illustrated on **Figure 3**. These relatively deep excavations may be advanced concurrently or sequentially.

Extensive control of groundwater in the overburden aquifer will be required to allow excavation to in "dry" conditions in the area of the ESWEMS pond. Some control of groundwater found in the shallow bedrock beneath the power block and cooling towers is also expected to be required. Relevant parameters for dewatering system evaluation under steady state flow conditions include the hydraulic conductivity (K) of the deposits to be dewatered. Specific yield (S_y) of the overburden and the storage coefficient (S) for the aquifer are relevant for evaluating transient flow conditions.

3.2 Review of Available Data

3.2.1 Overburden Aquifer Material

The horizontal hydraulic conductivity values calculated from slug tests conducted in the overburden aquifer ranged from 1.19 x 10^{-5} cm/s to 3.4 x 10^{-2} cm/s, with a geometric mean of 3.63 x 10^{-3} cm/s (Reference 9, Section 2.4.12.3.2.1). A single slug test in the overburden was completed in observation well MW410 during 2010, indicating a horizontal hydraulic conductivity of 1.72 x 10^{-3} cm/s (**Appendix A**) (Reference 4). Slug tests of the kind implemented during the site investigation measure horizontal hydraulic conductivity only near a test well, and may reflect influences by filter pack storage or low-conductivity borehole skins remaining after conventional rotary drilling using mud.

The horizontal hydraulic conductivity values calculated based on a 24-hr pump test at approximately 60 gpm ranged from 3.63 x 10^{-2} cm/s to 1.26 x 10^{-1} cm/s, with a geometric mean of 5.93 x 10^{-2} cm/s (Reference 9, Section 2.4.12.3.2.1). A pump test of the kind implemented during the site investigation stresses a much broader area of the aquifer than a slug test, and is therefore considered more representative than the slug test results. Weaver Boos considers the geometric mean value obtained during the pump test, which is the highest mean value for the site, as a representative (yet conservative) value for the horizontal hydraulic conductivity in the overburden aquifer. Because sand and gravel deposits comprising the overburden aquifer are horizontally stratified as described in the boring logs, the deposit is likely anisotropic, and the vertical hydraulic conductivity (which has not been measured) is considered to be 1/10th of the horizontal value obtained during the pump test.

The specific yields computed for the pump test indicated values ranging between 0.253 and 0.500, with a geometric mean of 0.344, and a median value of 0.322. For a well- to fairly well-graded material such as the overburden, the median value of 0.322 appears reasonable and is therefore assumed representative. Conservative values for the overburden aquifer selected for use in conceptual dewatering evaluation are summarized in **Table 1**.

3.2.2 Shale Bedrock

The horizontal hydraulic conductivity values calculated from slug tests conducted in the shallow bedrock aquifer during 2007 and 2008 ranged from 3.70 x 10^{-4} cm/s to 1.36 x 10^{-2} cm/s, with a geometric mean of 1.41 x 10^{-3} cm/s (FSAR Section 2.4.12.3.2.2). The horizontal hydraulic conductivity values calculated from slug tests conducted in the shallow bedrock during 2010 ranged from 4.69 x 10^{-5} cm/s to 1.32 x 10^{-3} cm/s, with a geometric mean of 2.86 x 10^{-4} cm/s (Reference 4, Table 3) (**Appendix A**).

The horizontal hydraulic conductivity values calculated from slug tests conducted in the deep bedrock aquifer ranged from 1.15 x 10^{-5} cm/s to 1.51 x 10^{-3} cm/s, with a geometric mean of 1.18 x 10^{-4} cm/s (FSAR Section 2.4.12.3.2.2). No further slug testing of wells screened in the deep bedrock was reported during 2010.

Packer tests were performed in 56 intervals of the shale bedrock during 2007 and 2008. Of these tests, nearly one-half (26) indicated impermeable rock. In the other 30 tests, the horizontal hydraulic conductivity ranged from 2.39 x 10^{-7} to 1.63 x 10^{-4} cm/s. Packer tests were performed in 34 additional intervals of shale bedrock during 2010. In these most recent tests, seven (7) tests indicated impermeable rock. In the other 27 tests, the horizontal hydraulic conductivity ranged from 3.99 x 10^{-7} cm/s to 3.82 x 10^{-4} cm/s.

The horizontal hydraulic conductivity values calculated based on a 24-hr pump test at approximately 6 gpm ranged from 1.93×10^{-5} cm/s to 7.23×10^{-4} cm/s, with a geometric mean of 1.64×10^{-4} cm/s (FSAR Section 2.4.12.3.2.2) during 2007 to 2008. This pump test was completed in wells screened from elevations ranging from 502 ft to 582 ft (NAVD88). Additional pump tests were performed using wells screened in the bedrock during 2010. The horizontal hydraulic conductivities calculated on the most recent pump tests ranged from 6.42×10^{-6} cm/s to 2.88×10^{-4} cm/s, with a geometric mean calculated equal to 5.43×10^{-4} cm/s (Reference 4, Table 4) (**Appendix A**). The recent pump tests utilized wells screened in the bedrock at elevations ranging from 618 ft to 670 ft (NAVD88).

For the reasons discussed in **Section 3.2.1**, Weaver Boos considers the geometric mean values obtained during the pump tests as more representative than values obtained by slug testing. However, results for the packer testing program are also considered representative for the intervals that were tested. Of the values reported for the shale bedrock, the geometric mean horizontal hydraulic conductivities are selected as conservatively high values for use in the dewatering evaluation. Weaver Boos selected the geometric mean pump test conductivity value of 5.43×10^{-4} cm/s as representative and conservative for shallow bedrock occurring above an elevation of approximately 600 ft (NAVD88). The geometric mean pump test conductivity value of 1.64×10^{-4} cm/s is similarly selected as representative and conservative for deep bedrock occurring below an elevation of approximately 600 ft (NAVD88). These values are regarded as conservative because their selection is likely to over-predict rather than under-predict the flow of groundwater to be yielded by temporary dewatering systems. As was selected for the overburden, Weaver Boos also considered the vertical conductivities of the bedrock (shallow and deep) to equal $1/10^{th}$ of their respective horizontal values obtained during the pump tests. Conservative values for the bedrock selected for use in this conceptual dewatering evaluation are summarized in **Table 1**.

3.2.3 Groundwater Water Table and Head Elevation

Monthly water table elevations in the overburden and the head elevations in the bedrock were previously measured between October 2007 and September 2008 (FSAR Section 2.4.12, Table 2.4-44). The data indicate seasonal fluctuations. Elevations measured on January 26, 2008 appear to represent "average" conditions, and elevations measured on March 24, 2008 are taken to represent "high" water levels. Monthly groundwater levels were most recently measured between May 2010 and July 2010. Measurements taken during 2010, which include measurements for the recently installed MW400-series observation wells (MW401 through

MW410), are generally somewhat lower than the "average" levels measured during January 2008.

In order for this evaluation to conservatively consider the reasonably foreseeable maximum future groundwater elevations that may occur during construction, calibration targets for the groundwater flow model were calculated in several steps. Firstly, Weaver Boos added 2.0 ft to the "high" groundwater elevations previously reported during March 2008 and selected these values as conservatively high flow model calibration targets in the MW300-series observation wells that were in the ground during 2008. The MW400-series observation wells were not installed until 2010, so the highest recently measured water levels in the MW400-series wells were corrected to estimate elevations expected to have been measured if they had been in the ground during March 2008. For MW410 screened in the overburden aquifer an adjustment of +2.6 ft was added to estimate an elevation that might have been measured during March 2008. This corrected maximum was further adjusted by +2.0 ft to account for possible fluctuations during consutruction. The total adjustment applied to MW410 was therefore +4.6 ft. High water elevations measured in MW400-series wells screened in the shallow bedrock (MW401 through MW409) were adjusted upward by +3.0 ft to first correct the observed levels back to March 2008 and the result was further adjusted by +2.0 ft to account for future fluctuations. The total adjustment applied to MW401 through MW409 was therefore +5.0 ft. The resulting groundwater elevations listed in Table 2 were then used herein as conservatively high calibration targets for the groundwater flow model described in **Section 5.5**.

3.3 Bedrock Surface Elevation

Bedrock surface elevations were obtained from soil borings advanced during the site investigation as reported in FSAR Section 2.5.4, Table 17. Bedrock surface points are also inferred from soil boring depths reported for overburden monitoring wells in FSAR Table 2.4-43, Rev. 2. Finally, bedrock surface points were collected by direct review of Rizzo's final boring logs (Reference 3) for the B400-series geotechnical borings and MW400-series monitoring wells advanced during 2010. The consolidated data are listed in **Table 3**.

4.0 CONCEPTUAL MODEL OF HYDROGEOLOGY

4.1 General Approach

Prior to assessing applicable dewatering technologies, Weaver Boos developed a conceptual model that to the extent practicable, incorporates natural hydrogeologic boundaries for the flow system of interest. Preparation of the conceptual model included the following general steps:

- Defining hydrostratigraphic units;
- Preparing a water budget; and
- Defining the flow system.

4.2 Definition of the Flow System

Weaver Boos defined the flow system based on review of the 7.5-minute series USGS topographic map of the Berwick Quadrangle (Reference 12), review of the Pennsylvania Department of Environmental Resources (1978) Map of Surface Deposits of the Berwick Quadrangle (FSAR Figure 2.4.12-3), and review of the information and data contained in References 3, 4, and 9).

Review of the available information indicates that the BBNPP site may be viewed as located within a small groundwater basin storing water mostly in the overburden aquifer. The overburden aquifer basin is defined to the north by the ridge formed in till-draped bedrock high upland north of Beech Grove Road, to the east by a bedrock ridge and groundwater flow divide corresponding approximately to the route of Confers Lane, to the south by a bedrock ridge forming in the knolls, and to the west by a bedrock ridge forming in the uplands west of Walker Run. Surface water and groundwater enter the overburden basin from the north and exit the basin via Walker Run, its small tributary located on the BBNPP site, and to a limited extent, southeastward through the narrow pass shown at the bottom of **Figure 4** between easting coordinates 2,404,800 ft and 2,405,500 ft. **Figure 5** depicts the boundary of the basin and groundwater flow model domain.

Deeper groundwater flows through the bedrock are less constrained than in the overburden basin and are assumed to reflect high upland recharge occurring to the north, followed by upward flow just south of the site, and deeper horizontal southerly and southeasterly flow towards the Susquehanna River.

4.3 Definition of Hydrostratigraphic Units

The principal hydrostratigraphic units beneath the site include the conductive overburden aquifer and the less conductive underlying shale bedrock. The shale bedrock is subdivided as "shallow" and "deep", with the dividing line occurring at an elevation of about 600 ft (NAVD88).

4.4 Water Budget

Three potential sources of groundwater flow inputs to the conceptual flow system are identified.

The first is groundwater recharge, assumed equal to groundwater discharge, reported in Reference 9, Table 2.4-42 (**Appendix A**) for the Wapwallopen Creek Basin as ranging from 6.6 to 21.8 inches per year, with an average equal to approximately 14.2 inches per year. The Wapwallopen Creek Basin is located closest to the site (Reference 9, Section 2.4.12.1.2.9) and is therefore considered the most representative of the watersheds listed in Reference 9, Table 2.4-42 (**Appendix A**).

The second is groundwater exchange with Walker Run that flows along the west side of the model domain.

The third is groundwater inflow originating in the ridge that rises to elevations as high as 1,100 ft (NAVD88) north of Beach Grove Road. This source cannot be directly measured, yet its significance is inferred from the upward vertical flow of groundwater in the lowland areas south of the proposed power block and ESWEMS pond.

Potential discharges of groundwater originating beneath the site include bank and bottom discharge to Walker Run and subsurface outflow to the south (much of which likely occurs in overburden deposits beneath Walker Run), with eventual discharge to the Susquehanna River. Additional southerly and southeasterly discharges of groundwater through the shallow and deep bedrock are also inferred from the bedrock potentiometric surfaces provided by Rizzo (**Appendix A**).

5.0 FLOW MODEL IMPLEMENTATION

5.1 Objective and General Approach

A groundwater flow model was implemented to first simulate the basin-scale groundwater flow system, and then to estimate the flows expected from site-scale dewatering systems at the power block, cooling towers, and ESWEMS pond excavations. The baseline model was prepared first and then compared with observed groundwater head and known basin inflows and outflows to check its calibration to the selected targets. Groundwater heads predicted by the baseline model were then used as initial conditions in subsequent models incorporating dewatering system elements (dewatering wells, drains, and constructed flow barriers) to simulate active dewatering, estimate groundwater withdrawals, and evaluate resulting drawdown.

Following development of the baseline flow model, it was modified to reflect two principal groundwater dewatering strategies:

- Open excavation and water table depression with no groundwater flow barriers; and
- Dewatering and excavation using a slurry wall, diaphragm wall, or other type of subsurface flow barrier at the ESWEMS pond to mitigate potential off-site groundwater level drawdown and subsequent impact to potentially sensitive areas.

5.2 Simplifying Assumptions

Integration of the Site conceptual model with a mathematical computer code to simulate flow requires several simplifying assumptions. The following assumptions and idealizations apply to the model utilized herein:

- The model domain is partly formed by a conductive overburden aquifer extending through the basin lowlands and restricted in its horizontal extent by surrounding rises in the less conductive bedrock.
- The complex natural flow system may be represented using a system of seven discrete layers, while the natural conditions likely result in a more gradual variation in hydrogeologic properties.
- The baseline groundwater flow system is in equilibrium and is modeled on a steady-state basis.

These simplifying assumptions do not require validation since they are the basis of modern groundwater modeling techniques.

5.3 Software Selection

Evaluation of groundwater flow was undertaken utilizing a seven-layered conceptual model implemented using Visual MODFLOW Version 2009.1, by Schlumberger Water Services. This software is a widely used implementation of the USGS's globally-recognized MODFLOW-2000 program. Weaver Boos selected this software for its capability to reliably model groundwater flow in three dimensions and relative ease of use offered by its integrated graphical user interface. Detailed information regarding the MODFLOW-2000 program can be found in Harbaugh et al. (2000).

5.4 Baseline Flow Model Design

5.4.1 Model Domain

The general area of the site is shown on **Figure 4**, which includes the locations of existing observation wells. Many of the observation well locations represent nested or clustered wells screened at varying elevations. The digital model domain is based on a rectangular block-centered grid network that covers a 1.8-square mile flow domain representing the local drainage basin as shown in **Figure 5**. The grid includes 316 rows and 245 columns, with their spacing refined as needed to assess small-scale effects in the area where dewatering is needed. In the areas where the greatest detail was desired, the grid node spacing is approximately 22 ft by 22 ft and provides site-scale detail without creating a computationally excessive number of model nodes.

5.4.2 Model Layers and Hydrogeologic Properties

Groundwater flow is simulated in seven layers.

Layer 1 is divided into conductivity zones delineated by narrow gridlines shown on **Figure 6**. The overburden aquifer is vertically limited to Layer 1. The lower surface of Layer 1 in the vicinity of the site was interpolated from the bedrock elevations reported on **Table 3**. Since the upper surface of Layer 1 is based on the topography of the site and the lower surface is based on the interface with weathered bedrock, this layer exhibits a varying thickness across the model domain. The lateral limits of the Sand/Gravel Zone of high horizontal conductivity (5.9 x 10^{-2} cm/s) illustrate the extent of the overburden aquifer. The lateral limits of the overburden aquifer are represented by the conductivity zone border, outside of which Layer 1 is assigned the conductivity for shallow bedrock (5.4×10^{-4} cm/s). Walker Run is represented by a river boundary located wholly within the high conductivity zone of Layer 1.

Layer 2 (not graphically shown) is uniformly assigned the conductivity for shallow bedrock. Within the upland rise to be occupied by the power block and cooling towers, Layers 3 and 4 (not graphically shown) are also assigned the shallow bedrock conductivity so that bedrock above an elevation of about 600 ft (NAVD88) is represented as the more conductive shallow bedrock. Layers 3 through 7 are assigned the conductivity for deep bedrock (1.64×10^{-4} cm/s) in other areas of the model domain at elevations below approximately 600 ft (NAVD88). The vertical conductivity in all zones is assigned as $1/10^{th}$ of the corresponding horizontal conductivity for overburden, shallow bedrock, and deep bedrock, respectively.

The upper surface of Layer 1 is a recharge boundary with three zones illustrated on **Figure 7**. Lowland areas within the model domain are assigned a recharge of 14.2 in/yr. The transition zone between the lowlands and uplands is assigned an intermediate recharge of 21.8 in/yr. The uplands are assigned a high recharge of 29 in/yr. High recharge was focused in the upland areas to simulate the groundwater mounding reported by Rizzo in the shallow bedrock forming the upland part of the site to be occupied by the power block (**Appendix A, Figure 6**). The high upland recharge values were also selected to conservatively model the groundwater flow during very wet times, thus tending to maximize predicted groundwater withdrawals during dewatering.

Several fixed head boundaries as shown on **Figure 8** were incorporated into Layers 2 through 7 of the model to simulate observed deep bedrock head levels. These include constant head boundaries north and southeast of the site and a general head boundary along the southern limit of the model domain. The constant head boundaries are intended to simulate generally southerly and southeasterly flow originating in the highlands north of the site towards the Susquehanna River. The general head boundary at the southern limit of the model domain is intended to simulate the discharge of groundwater directly to the river. The entire model domain is encompassed by a no-flow boundary in all layers. The upper surface of the model domain was interpolated from the ground surface contours included in the USGS 7.5-minute series topographic map for Berwick as shown on the block view illustrated in **Figure 9**.

5.5 Baseline Flow Model Calibration

Calibration of the baseline flow model consisted of initial simulations followed by adjustment of selected model elements such as recharge flux and distribution, river boundary conductance along Walker Run, the placement of constant head boundaries in Layers 2 through 7 to simulate inflow from the upland ridge north of the site, constant head boundaries to simulate deep bedrock outflows to the east and southeast of the site, and the use of a general head boundary to simulate outflow along the southern edge of the model domain towards the Susquehanna River. Calculated baseline heads in the relevant model layers are compared with the target heads listed in **Table 2**. Results for these comparisons are provided as several plots in **Appendix B**.

The first calibration plot includes all target head values for observation wells screened in Layers 1 through 6. As shown on the first plot "Baseline Head Calibration, All Layers", a correlation coefficient of 0.903 is reported. The 95% confidence interval falls slightly to the right of the ideal line, suggesting that the modeled heads are generally lower than their targets. Subsequent plots for Layers 2, 3, 4, 5, and 6; however, shows that the under-prediction of heads is limited to Layer 3, which contains most of the MW400-series observation wells located on the power block upland. This distribution of under-predicted heads on the power block highland is further illustrated as the cluster of blue circles on the "Calibration Residual Map, All Wells" also provided in **Appendix B**. Calibration trials using upland recharge as great at 36 in/yr increased the mass budget discrepancy between total inflows and total outflows to excessive levels, yet failed to significantly even out the residual distribution. Such unrealistic recharge was therefore omitted from the model. The baseline head calibration is nevertheless considered reasonable as it shows significant groundwater mounding on the power block upland as needed to evaluate dewatering. Additionally, some of the high head levels observed on the upland may represent perched conditions, and may not be entirely representative of the groundwater flow system in this area.

Global list file output is also included in **Appendix B** to provide additional details as to the baseline flow model. Included is a compact disc containing all of the files comprising the baseline flow model in connection with the primary project file named "BBNPPBaseline0.vmf".

5.6 Baseline Flow Model Results

Simulated baseline model heads for Layer 1, which includes the overburden aquifer, are shown on **Figure 10**. The upland area of the power block shows no head contours because the layer is dry across the top of the upland as observed in borings advanced in this area. Head contours in the lowland areas are generally similar to those mapped by Rizzo as shown for the overburden aquifer (**Appendix A, Figure 5**). Simulated baseline heads for the shallow bedrock in Layer 2 are illustrated in **Figure 11**. Significant upland groundwater mounding is shown in the model results consistent with observation of the shallow bedrock potentiometric surface provided in

Appendix A, Figure 6. Although not shown, the simulated heads in Layers 3 through 7 also successfully simulate observations in the deep bedrock. Considering the reasonable simulation of observed heads, the steady-state simulation values computed by the baseline flow model were stored and then utilized as an initial condition for two further models simulating dewatering of the site, both with and without the use of a flow barrier at the ESWEMS pond.

5.7 Flow Model Simulating Dewatering with No Flow Barrier

Temporary construction dewatering of the site was first simulated without the benefit of a flow barrier at the ESWEMS pond. Dewatering was simulated using a combination of pumping wells and drains. Pumping wells DW1 through DW28 are shown around the perimeter of the ESWEMS pond excavation as shown on **Figure 12**. Pumping wells are expected to form the most effective first stage of dewatering in this area where excavation through a thick section of saturated sand and gravel is required. The pumping wells were input as multi-node wells (MNWs) and set to draw their casing water levels down near the top of the bedrock. Their rates were specified to vary between 10 and 300 gpm, with the actual rate computationally determined by the software using the specified drawdown elevations.

Lower stages of dewatering at the ESWEMS pond, as well as the dewatering of bedrock at the power block and cooling towers, was simulated using drain cells. The drain cells simulate either lines of vacuum well points if located in overburden, or trenches cut into the exposed bedrock surface drained to sumps. The shallowest anticipated drains reside in Layer 1 of the model and shown as gray cells on **Figure 12**. The drainage elevation at each drain cell is set approximately 2 ft below the nearest excavation contour elevation and is placed on a proposed bench where present. Drains at deeper depths were placed in Layer 2 of the model as shown on **Figure 13**. Drains at the deepest depths to be excavated are shown on **Figure 14** and are limited to the power block and cooling towers excavations.

The modified flow model named "BBNPPNoBarrier0.vmf" was then run on a steady state basis utilizing the output from the baseline flow model to establish initial heads. This procedure allows the modified flow model to compute the drawdowns expected to result when dewatering has reached steady-state conditions. The resulting drawdowns in Layer 1 of the modified flow model are illustrated in **Figure 15**. The drawdowns are shown in feet, and represent water table depression that is likely to result in the overburden aquifer if this dewatering strategy is used. Review of **Figure 15** indicates deep water table depression (5 to 40 ft) in the areas extending

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west, south, and east of the proposed ESWEMS pond. This would most likely result in extensive dewatering of the wetlands south of the ESWEMS pond.

5.8 Flow Model Simulating Dewatering with a Flow Barrier

Temporary construction dewatering of the site was then simulated to evaluate the potential benefits of a flow barrier encompassing the proposed ESWEMS pond excavation. Wall boundaries were inputted to the dewatering flow model to simulate a 3-ft thick flow barrier. As discussed in **Section 6.4.2.1**, it is recommended that the flow barrier be designed for a hydraulic conductivity of 1×10^{-7} cm/s. Recognizing that construction quality control for a subsurface flow barrier is difficult; the flow model was implemented assuming a hydraulic conductivity of 1×10^{-6} cm/s through the wall to account for potential imperfections in its construction. The wall boundaries form a continuous flow barrier around the proposed excavation and extend from top to bottom in Layer 1 of the model that simulates the conductive overburden aquifer. The modified dewatering flow model was named "BBNPPFlowBarrier1.vmf" and is also provided on the compact disc in **Appendix B**.

As before, the modified flow model was run using the initial heads computed by the baseline flow model and the expected drawdowns are plotted on **Figure 16**. Review of this figure again shows the deep drawdown required at the ESWEMS pond as needed for construction. However, the simulated drawdown elsewhere in the basin is very much less than for the simulation without the flow barrier. Drawdown ranging from 5 ft to 10 ft is focused immediately west and south of the flow barrier. Weaver Boos believes that this effect is not primarily due to the withdrawal of water from within the flow barrier, but rather due to the partial cutoff of natural westerly flow of groundwater through the position of the barrier. Groundwater levels are expected to diminish on the downgradient side of a barrier obstructing horizontal flow.

5.9 Dewatering Results

The effects of dewatering included in the evaluation for the power block are illustrated on **Figure 17**, which shows the predicted head in the shallow bedrock at the bottom of the proposed excavation (Layer 3 of the model). The predicted dewatered heads are lower than the bottom of the excavation at almost all locations. The effects of dewatering included in the evaluation for the cooling towers excavation are illustrated on **Figure 18**, which shows similar results in Layer 2 of the model. The effects of dewatering at the ESWEMS pond excavation are illustrated on **Figure 19**, which shows head levels in the bedrock at the base of the excavation (Layer 2 of the model). Predicted heads are lower than the base of the excavation except for a small area along the west side of the excavation floor. This exception appears to be connected with the model discretization of the interface between Layers 1 and 2 and the high contrast between their respective hydraulic conductivities. This exception is not regarded as computationally significant. Additionally, the keying of the flow barrier into the competent bedrock is likely to reduce actual dewatered heads to levels lower than those predicted by the model.

5.10 Mass Flow Budgets

Baseline flow conditions and projected drawdown discussed above provide considerable information regarding the likely influence of the projected dewatering strategies. Mass flow budgets provide additional information as to the inflows and outflows from the model domain and provide an additional means to assess the representativeness of the simulation. Groundwater mass flow budgets from the three flow models are summarized in **Table 5**. Mass budget output from the flow models are also provided in **Appendix C**.

5.10.1 Baseline Flow Model

The mass flow budget for the baseline flow model includes terms for the following sources and sinks:

- *Constant Head Boundary* - Represents inflows to the model domain originating in the bedrock ridge to the north.
- *River Leakage* - Represents exchange of water between Walker Run and the Aquifer.
- *Head Dependent Boundary* - Represents outflow from the southern edge of the model domain to the Susquehanna River.
- *Recharge* - Represents percolation of recharge over the entire 1.8 mi^2 model domain.

Review of the baseline mass flow budget summarized in **Table 5** suggests that the model domain is receiving subsurface groundwater inflow at about 2.1 cfs from the ridge and highlands to the north and losing about 1.5 cfs through deep bedrock pathways to the east and southeast. Walker Run is suggested to lose about 1.4 cfs to the aquifer, but also to gain about 2.0 cfs from the aquifer, indicating a net gain of 0.6 cfs. The basin receives recharge from precipitation at about 2.5 cfs. Water is lost through the head-dependent general head boundary at the south edge of the model domain towards the Susquehanna River at about 2.5 cfs. Total inflow and total outflow are suggested to be about 6.1 to 6.0 cfs, respectively, indicating a discrepancy of about 1.7 percent.

5.10.2 Mass Budget for Dewatering Without a Flow Barrier

The mass flow budget for this model includes terms for *Drains* and *Dewatering Wells* positioned at the locations shown on **Figures 12, 13**, and **14**. Zone budgets were set in the model to separately account for the dewatering system outflows from the power block, cooling towers, and ESWEMS excavation. As shown on **Table 5**, the dewatering system under this scenario is suggested to remove water at a rate of 0.11 cfs (about 50 gpm) at the power block excavation, 0.16 cfs (about 70 gpm) from the cooling towers excavation, about 0.56 cfs (about 250 gpm) from the ESWEMS drains, and 1.5 cfs (about 670 gpm) from the ESWEMS dewatering wells. Dewatering outflows from the site without a flow barrier at the ESWEMS pond excavation total 2.33 cfs (about 1,040 gpm). These rates are steady state and will be much higher when dewatering is initiated.

Other terms of the mass flow budget for this model indicate that inflows from the northern ridge will increase, and southerly/southeasterly bedrock outflow will decrease. Inflow from Walker Run is indicated to increase from 1.4 cfs to 1.9 cfs, and outflow is 1.7 cfs instead of 2.0 cfs as in the baseline model. This result suggests that Walker Run will be changed from a gaining stream to a losing stream (-0.2 cfs) near the site. The total aquifer throughput suggested by this model is 7.1 to 7.5 cfs as compared with 6.0 to 6.1 cfs as indicated by the baseline model. The mass flow discrepancy for this model is considerably higher than the baseline model at about 5 percent.

5.10.3 Mass Budget for Dewatering with a Flow Barrier

Installation of a flow barrier, such as a soil-bentonite slurry wall, or diaphragm wall substantially reduces the steady-state outflow from the ESWEMS pond excavation dewatering system to a rate suggested to be about 230 gpm as compared with 920 gpm without the barrier. Total dewatering system outflows for the site are about 0.78 cfs (about 350 gpm) in this scenario. These rates are steady state and will be higher when dewatering is initiated, as the pore space of the soils within the barrier wall is drained.

Walker Run also appears to remain a gaining stream in this scenario at about 0.2 cfs. Total aquifer throughput in this scenario is 6.4 cfs to 6.5 cfs as compared with 6.0 to 6.1 cfs in the baseline flow model. Total aquifer throughput is therefore expected to increase by only about 8.2 percent. The mass flow discrepancy for this model is about 1.6 percent, which is about equal to the baseline model.

6.0 CONCEPTUAL EVALUATION FOR DEWATERING SYSTEMS

6.1 Objective and General Approach

A digital groundwater flow model was used to simulate basin-scale groundwater flow and the effects of various elements of dewatering systems as discussed in **Section 5.0**. Site-scale dewatering systems for the proposed excavations are further evaluated using traditional manual calculations as described in this section. The general approach included manual calculation of probable flow rates and type drawdown curves for dewatering wells for two representative saturated thickness values, and then the application of the hydraulic principal of superposition to select appropriate dewatering well locations and spacing under the two primary scenarios at the ESWEMS pond.

6.2 Dewatering Well Hydraulics

The calculation of water table drawdown in an unconfined aquifer of the type present at the ESWEMS pond first requires an estimate of flow from a typical dewatering well. Radial flow to a frictionless (100 percent efficient) well in an unconfined aquifer may be estimated using the following equation (Reference 6, page 103):

$$Q = \frac{\pi K (H^2 - h_w^2)}{\ln(R_o / r_w)} \qquad \text{Eqn 1}$$

Where:

- Q = well yield or pumping rate (*Length*^{3}*/time*) assuming 100 percent efficiency
- K = hydraulic conductivity of water bearing formation (*L/time*)
- H = initial height of water above the lower confining layer (*L*)
- h_w = height of water above the lower confining layer in the well (*L*)
- R_o = radius of influence of pumping well, historically estimated at this facility using Sichart's empirical relation (Reference 11) (*L*).
- r_w = distance from the pumping well (*L*)

Once the flow rate, Q, is estimated assuming an ideally efficient well, it should be down-rated to account for the hydraulic inefficiency characterizing even properly drilled water wells. A well efficiency of 80 percent is assumed for this evaluation.

The height of the water table surface at distance r from the well, where r is greater than $1.5H$, where H is the original saturated thickness, may be estimated as follows (Reference 6, page 103):

$$h = \sqrt{H^2 - \frac{Q_w}{\pi K} \ln \frac{R_o}{r}} \qquad \text{Eqn 2}$$

Where:

Q_w = down-rated flow from well ($L^3/time$)

h = height of the phreatic surface above the lower confining layer (L)

r = distance from pumping well or group of pumping wells (L)

Appendix D provides a series of solutions calculated using Equations 1 and 2 to estimate the flows from dewatering wells and to predict the drawdown expected at selected distances. As shown in the calculations, a typical dewatering well installed in a saturated zone 55 ft thick is expected to extend its radius of influence to a distance of approximately 3,300 ft from the well. Accounting for an estimated efficiency of 80 percent, such a well is expected to yield water at about 790 gallons per minute (gpm) if pumped to a level approximately 10 ft above the bottom of the well. Weaver Boos notes that this flow rate is for a single well and does not consider the interaction and reduced flows that will result for individual wells located in a closely spaced group or line.

The second typical case considered in **Appendix D** is for a dewatering well installed where the saturated thickness of the aquifer is 27.5 ft. In this case, the radius of influence is expected to extend to a distance of approximately 1,300 ft, and the 80 percent efficient flow rate for a single well acting alone is estimated at 210 gpm.

Included in **Appendix D** is a semi-logarithmic plot of drawdown expected for the typical wells as described. These drawdown plots are the basis for well location and spacing selection using the hydrogeologic principal of "cumulative drawdowns" stating that the water table drawdown at any point in the vicinity of a well array will be the sum of the individual drawdowns that would have been caused by each well operating alone.

6.3 Conceptual Evaluation for Dewatering with No Flow Barrier

6.3.1 Performance Criteria

Performance criteria for dewatering include depression of the ambient water table to depths at or below the surface of the bedrock. Water level observations listed in **Table 2** and bedrock surface elevations listed in **Table 3** indicate that during high groundwater conditions selected as flow model calibration targets, the saturated portion of the overburden aquifer at the ESWEMS pond ranges from about 26 ft to 56 ft in thickness Depression of a water table to the surface of a low permeability layer such as the shale beneath the site will therefore require several stages.

6.3.2 Conceptual Evaluation

Deep dewatering wells may be located around the perimeter of the excavation to implement the first stage of water table depression. Because wells cannot depress the water table to the base of the aquifer in areas between the wells, a level of approximately 10 ft above the shale is selected as a target for use in computing cumulative drawdowns. By inspection of the drawdown curves computed in **Appendix D**, an inter-well spacing of approximately 100 ft will provide for a cumulative drawdown of slightly more than 50 feet at locations between the wells. Dewatering wells may be located as shown on **Figure 20** based on this conceptual design criterion. A total of approximately 28 dewatering wells appear to be appropriate for conditions at the ESWEMS pond excavation. Given the large number of wells required and potentially very large initial flows that such a system might develop, individual pumps should be sized for approximately 100 to 150 gpm each. The discharge lines should be fitted with throttling valves to control the overall flow rate of the system and avoid overwhelming the body receiving the discharge. A schematic diagram showing a typical dewatering well considered appropriate for conditions at this site is provided as **Figure 23**.

Dewatering wells might be installed using direct rotary, reverse-circulation rotary, cable tool, or other methods such as Rotosonic. Reverse-circulation rotary will provide wells with the greatest efficiency and should therefore be considered. The other methods listed might tend to compact the aquifer formation, or leave low-conductivity borehole skins that cannot be completely removed during development. Because the overburden aquifer contains boulders, it may be necessary to use an orange peel, chisel, or other methods to remove or penetrate through them.

A second stage of water table depression to the shale surface or near the shale surface may require the use of vacuum well points positioned as shown on **Figure 20**. Each of the headers shown will draw water from well points that are typically 2-in. diameter that may be drilled, driven, or jetted in if conditions allow. Each header will need to be connected to its own vacuum pump. Individual vacuum pumps will need to be sized based on conditions encountered and the length of each header.

Final stages of the dewatering conceptual design include the excavation of trench drains and sumps into the exposed bedrock surface in front of the toe of the slope at the base of the EWSEMS excavation. Such trenches might be dug 3 to 5 ft wide, and 2 to 3 ft deep, and sloped to collection sumps for ejection from the excavation. Groundwater flow from the bedrock is expected to vary over a wide range, and additional trenches or sumps might be needed at locations to be determined based on actual field conditions. Three such trenches were incorporated into the digital flow model at the ESWEMS pond as shown on **Figure 20**.

Groundwater observations at the power block and cooling towers excavations suggest that little saturated overburden is present in either area. It is therefore expected that groundwater inflows may be controlled using trench drains cut into the bedrock at the locations and elevations suggested on **Figure 20** (power block) and **Figure 21** (cooling towers). The trench drains can be sloped to sumps where the water can be pumped out if a proper slope cannot be attained to drain the trenches to the groundwater storage pond.

The modeled solution in the vicinity of the northwest corner of the cooling tower is limited by the available boring data. Specifically, the excavation plan (Reference 5) for the cooling towers considers a boring located at the proposed center of each tower. The response to RFI SL-BBNPP-149 indicates that the northwest quadrant of the cooling tower excavation will extend down to elevation 646 ft (NAVD88). The site grading plan (Reference 8) indicates wetlands associated with Walker Run are present at an approximate elevation of 670 ft (NAVD88) near the intersection of Market Street and Beach Road. If the overburden extends below the nearby wetlands, it is likely that the overburden will be saturated. Figure 5 in Reference 4 (**Appendix A**) does not show groundwater in overburden soils in this area. If the overburden soils are saturated it is likely that excessive groundwater pump rates and subsequent dewatering of the adjacent wetlands will occur. The groundwater pumping rates and subsequent drawdown determined and presented in this evaluation does not consider this potential outcome since the site groundwater data does not indicate that the overburden in this area is water-bearing. It should be noted that if the overburden extends below the wetlands, this condition could be mitigated by installing a flow barrier wall in accordance with **Section 6.4.2**.

The effectiveness of the dewatering system should be monitored to compare observed drawdown with the estimates described herein (or more detailed design estimates developed prior to implementation). Water levels may be monitored for this dewatering strategy using existing monitoring well clusters that have been drilled at the site. Additional monitoring wells or piezometers at the locations near to the excavation would provide further points for comparison and are therefore recommended. A typical schematic diagram for monitoring wells or piezometers is provided on **Figure 24**.

Digital modeling of dewatering suggests that the total steady state discharge from ESWEMS system will be somewhat more than 1,000 gpm if no flow barrier is used to control horizontal inflows from the overburden aquifer. The trench drains at the power block excavation are estimated to discharge about 50 gpm at steady state. The trench drains at the cooling towers excavation are estimated to discharge about 70 gpm at steady state. These figures are also summarized in **Table 5**. Larger flows on the order of 2,000 to 4,000 gpm might be incurred during the initial phase of water table depression with the specified dewatering well pump sizing at the ESWEMS pond excavation.

Operation of this conceptual dewatering system will require an uninterrupted source of power for electrically operated submersible pumps and vacuum pumps, and an uninterrupted source of fuel for internal combustion vacuum pumps if selected for use. Provisions for convenient maintenance should be included for all system elements as needed for a project duration approaching 3 years.

6.4 Conceptual Evaluation of Dewatering with Flow Barrier

Dewatering system yields and impacts to nearby wetlands may be mitigated using a subsurface flow barrier. The digital flow model indicates that substantial benefits may be obtained by using a flow barrier at the ESWEMS pond excavation.

6.4.1 Performance Criteria

Performance criteria for this dewatering strategy are the same as discussed in **Section 6.3.1**. However, the use of a flow barrier, such as a slurry wall or diaphragm wall, will assist in controlling groundwater flow towards the ESWEMS pond excavation and likely allow further water table depression using deep wells to be located on the interior of the flow barrier.

6.4.2 Conceptual Design

6.4.2.1 <u>Flow Barrier</u>

If a soil-bentonite (S-B) slurry wall is selected for use as a flow barrier, it might be installed along an alignment as shown on **Figure 22**, and should reflect the following guidelines in its final design:

- The slurry wall will be a minimum of three feet thick, and will be at least ½-foot-thick for each 10 feet of hydraulic head across the wall.

- The slurry wall will be keyed into competent shale such that the flow underneath the wall through the shale is less than or equal to the flow directly through the soil-bentonite slurry wall. The minimum depth of penetration of the slurry wall key will be two feet into the shale below any permeable lenses or weathered shale zones.

- The slurry will consist of 4 to 7 percent bentonite in water, and the backfill will contain bentonite at a rate of 3 percent. If the groundwater barrier is also designed to act as a temporary excavation support wall, Portland cement may also be incorporated into the slurry.

- The slurry walls will have a designed in-situ permeability less than or equal to 1×10^{-7} cm/s. Some plastic fines may need to be imported to meet this criterion. A value of 1×10^{-6} cm/s was used in this evaluation to account for any minor imperfections in the wall.

- Slurry walls will have a minimum of a five-foot overlap at corners.

- Slurry wall will be constructed vertically.

- Slurry levels will be maintained at least seven feet above the groundwater table during construction. This will require the construction of a berm to raise the ground level at several locations along the specified alignment.

- Extensive quality control measures should be taken to assure that the S-B slurry wall is constructed without gaps or windows.

- Because the overburden aquifer contains boulders, it may be necessary to use an orange peel, clamshell, chisel, or other methods to remove or penetrate through them.

If ground support is required, sheet piling, concrete diaphragm walls, intersecting caissons or secant piles, or cofferdams should be considered. All aspects of ground support and excavation stability will require extensive additional evaluation and detailed designs beyond the scope of this evaluation.

Calculations included in **Appendix D** estimate potential flux rates through the flow barrier wall when the maximum gradient is established. Assuming that the in-situ hydraulic conductivity will achieve 1×10^{-6} cm/s, flux across the wall is estimated at approximately 7 gpm. If the design criterion of 1 x 10^{-7} cm/s is achieved, the corresponding flux rate is about 1 gpm. Calculation is also provided assuming that the barrier wall is discontinuous over 1 percent of its vertical surface area. Assuming a 1 percent area of gaps or windows, excess inflows approaching 4,400 gpm might occur. This finding underscores the need for adequate quality assurance and quality control (QA/QC) during construction. Furthermore, it indicates that if the wall is discontinuous, the presence of discontinuities should be obvious shortly after the initiation of interior dewatering as the excavation proceeds downward.

Operation of the barrier wall and interior dewatering system should include a piezometric monitoring program to compare expected groundwater withdrawals and drawdown rates with those calculated in advance. This program should include continuous monitoring of the existing and proposed monitoring wells or piezometers to be located as shown on **Figure 22**. It is recommended that monitoring wells or piezometers be installed in pairs at the five locations shown on **Figure 22** to allow monitoring of groundwater levels inboard and outboard of the flow barrier wall. Data logging pressure transducers with remote telemetry are recommended for this purpose so that head levels may be continuously monitored during initial drawdown and later during the extended phase of construction activity. If a window or gap in the flow barrier is indicated by the piezometric monitoring program, then pressure grouting or other remedial measures will be necessary. Additional groundwater monitoring wells may be warranted in the immediate vicinity of significant repairs to the flow barrier wall.

6.4.3 Interior Dewatering

Deep dewatering wells drilled at the locations shown on **Figure 22** may be used to depress the water table inside the flow barrier. Although the wells will not depress the water table completely to the base of the aquifer in areas between the wells, greater drawdown is expected with this strategy owing to the effective prevention of inflows by the flow barrier. Additionally, the presence of the flow barrier will amplify the drawdown within the enclosed area consistent with the concepts of image theory (Reference 1, page 330). Image well theory states that wells pumping near an impermeable barrier such as a flow barrier wall induce drawdown comprising the sum of the drawdown caused by the real well(s) plus additional drawdown attributable to image well(s) located on the opposite side of the flow barrier.

Considering the use of the flow barrier, dewatering wells may be located as shown on **Figure 22** based on this conceptual design criterion. A total of approximately 14 dewatering wells appear to be appropriate if the flow barrier is utilized. Given the number of wells required and potentially very large initial flows that such a system might develop, individual pumps should be sized for no more than approximately 100 to 150 gpm each. Included in **Appendix D** is calculation of the time expected to drain the groundwater stored within the conceptual flow barrier. As shown therein, it is assumed that the dewatering system inside the flow barrier is pumped at a rate of about 600 gpm. Approximately 226 acre-ft of water will be removed over a period of approximately 85 days. Digital flow modeling of this dewatering strategy suggests that interior dewatering might require a steady-state flow on the order of 230 gpm at the ESWEMS pond excavation. The majority of this water is expected to flow upward through the shale, but the actual flow may be less. Numerous packer tests conducted in the shale during the site investigation indicate hydraulic conductivity values much lower than assumed in the model, and in approximately one-half of the tests, the hydraulic conductivity was effectively zero.

A second stage of water table depression to the shale surface or near the shale surface may require the use of vacuum well points positioned as shown on **Figure 20**. Each of the headers shown will draw water from well points that are typically 2-in. diameter that may be drilled, driven, or jetted in if conditions allow. Each header will need to be connected to its own vacuum pump. Individual vacuum pumps will need to be sized based on conditions encountered and the length of each header.

Final stages of the dewatering conceptual design include the excavation of trench drains and sumps into the exposed bedrock surface in front of the toe of the slope at the base of the EWSEMS excavation. Such trenches might be dug 3 to 5 ft wide, and 2 to 3 ft deep, and sloped to collection sumps for ejection from the excavation. Groundwater flow from the bedrock is expected to vary over a wide range, and additional trenches or sumps might be needed at locations to be determined. Three such trenches were incorporated into the digital flow model at the ESWEMS pond as shown on **Figure 20**.

Digital modeling suggests that the total steady state discharge from ESWEMS system will be about 230 gpm if the flow barrier is used to control horizontal inflows from the overburden aquifer in this area of the site. Dewatering discharges from the power block and cooling towers excavations are not expected to change because flow barriers are not indicated based on the evaluation of available data and information. The trench drains at the power block excavation are therefore estimated to discharge about 50 gpm at steady state. The trench drains at the cooling towers are therefore remain estimated to discharge about 70 gpm at steady state. These figures are also summarized in **Table 5**. Larger flows on the order of 1,400 to 2,100 gpm might be incurred during the initial phase of water table depression with the specified dewatering well pump sizing at the ESWEMS pond.

Operation of this conceptual dewatering system should be less sensitive to brief interruptions in electrical power because the flow barrier will retard inflows to the excavation. However, provisions for convenient maintenance should still be included for all system elements as needed for a project duration approaching 3 years.

6.4.4 Time Domain Analysis Using Transient Digital Model

Manual calculation of the time to drain the enclosed volume of the ESWEMS pond flow barrier suggests that it will take about 85 days. This estimate was also further evaluated using a transient implementation of the computer flow model incorporating the flow barrier. The transient implementation of the model is included on the compact disc in **Appendix B**, and is titled "BBNPPWithBarrier1TR.vmf". A single stress period of 1,095 days, representing the 3-yr duration of dewatering, was subdivided into 10 time steps to estimate the flows from the various dewatering system elements (drains and pumping wells). Graphical output of the results for the ESWEMS pond, power block, and cooling towers excavations are provided at the end of **Appendix C**.

The transient implementation of the model indicates that outflows from all dewatering system elements closely approach their steady state values after about 90 days of pumping. This estimate is nearly equal to the manual calculation for the ESWEMS pond system. Steady state is reached for all dewatering system elements at the power block, cooling towers, and ESWMES pond excavations after about 300 days of pumping according to the transient model.

7.0 EVALUATION OF OFF-SITE IMPACTS

7.1 Objective and General Approach

This section evaluates potential off-site impacts that might develop as a result of temporary construction dewatering as described in **Section 6.0**. The general approach is to consider the extent and magnitude of groundwater drawdown projected to occur in off-site areas in response to dewatering. The following potential impact points and sensitive areas are considered:

- Water wells;
- Wetlands;
- Lakes and other surface waters; and,
- Structures.

7.2 Extent and Magnitude of Drawdown

The extent and magnitude of groundwater drawdown projected for dewatering without a flow barrier is shown on **Figure 15**. Review of this figure indicates deep drawdown (greater than 25 ft) at distances of up to about 800 feet south of the ESWEMS pond. The extent and magnitude of groundwater drawdown projected from dewatering using the flow barrier is shown on **Figure 16**. No deep drawdown (greater than 25 ft) is indicated in areas outside of the flow barrier. Drawdown indicated to range from 5 to 10 ft extends no further than about 500 ft west and about 250 ft south of the ESWEMS pond. However, groundwater recharge from the groundwater storage pond (if unlined) is expected to reduce both the magnitude and aerial extent of such drawdown.

7.3 Potential Impact

7.3.1 Water Wells

The majority of residents near the site obtain water from domestic wells. Several industries including SSES obtain water from wells. There are six domestic use wells and one commercial use well within one-half to three-quarters of a mile of the site. Given the drawdown projected to occur during dewatering without a flow barrier, some potential exists for negative impact on nearby domestic and industrial water supply wells.

In the case where the flow barrier is utilized, little or no impact to nearby wells is anticipated.

7.4 Wetlands

Extensive wetlands are located both on the BBNPP site and in adjoining areas, particularly to the west, south, and east. Such features are often expressions of the natural water table at or near the surface, and are therefore quite sensitive to impact via water table depression.

If dewatering is implemented without the flow barrier, substantial adverse impact is expected on the levels of surface water and groundwater in the wetland south of the ESWEMS pond. A very small area to the northwest of the ESWEMS pond is shown with a drawdown of 5 ft, suggesting a minor potential for adverse impact to the wetland at that location. A very small area of drawdown at 5 ft is also shown immediately west of the proposed power block excavation. This very small area of drawdown does not appear to extend to the wetland located west of the power block.

If dewatering is implemented with flow barriers around the ESWEMS pond excavation and any other areas where the overburden soils are saturated, the potential for adverse impact on the wetlands is significantly reduced. The actual impact is likely to be less than indicated by the model (**Figure 16**) because the flow barrier will be keyed several feet into competent rock. The MODFLOW program cannot easily simulate the partial penetration of the flow barrier into the top of the bedrock. Potential drawdown to the northwest of the ESWEMS pond appears to be nearly eliminated. Potential drawdown immediately west of the power block excavation remains unchanged since no flow barrier is used for the power block and is not expected to affect the wetland to the west.

7.5 Lakes and Other Surface Waters

Several ponds, Walker Run, and a short section of stream channel are located within the basin as shown on **Figure 4**. These water features are expected to be connected with the water table to varying degrees. If connected with the water table, such features will be sensitive to water table drawdown during dewatering.

Given the relatively widespread deep drawdown projected to occur during dewatering without a flow barrier, considerable potential exists for negative impact to surface water features near the site. The mass flow budgets for both dewatering scenarios (**Table 5**) suggest that Walker Run may change from a gaining stream to a losing stream, if dewatering is undertaken without the flow barrier.

In the case where the flow barrier is utilized, projected drawdown is much less, and potential impacts to surface waters, if any, will also be substantially less. The mass flow budget (**Table 5**) indicates that Walker Run should remain a gaining stream in this instance.

7.6 Structures

The sensitivity of structures to changes in water level results from potential changes in effective stress on underlying foundation soils and possible loss of ground. Structures founded on firm bearing soils or on rock are not likely to be impacted by either of the dewatering scenarios. However, structures that bear in organic or compressible soils might experience undesirable settlement or damage due to consolidation resulting from increased effective stress. Potential impacts to structures will tend to be minimized by the use of the flow barrier.

7.7 Mitigation of Potential Impact

Potential impacts due to water table drawdown may be mitigated by any method that reduces or eliminates drawdown in areas beyond the excavation. Aquifer recharge is one potential method to reduce drawdown in areas where drawdown of the groundwater is not desired. This might be implemented using injection wells or by allowing exfiltration from the groundwater storage pond if constructed without a lining. It will be difficult; however, to control extensive drawdown using these means alone if dewatering is undertaken without the flow barrier around the ESWEMS pond.

Given the physical constraints posed by the location of the site and adjoining wetlands, a vertically-oriented flow barrier, such as a S-B slurry wall, or diaphragm wall appears to be a viable and effective means to mitigate potential impacts due to projected water table drawdown. Drawdown outside the flow barrier extends mostly west and south of the ESWEMS pond as shown on **Figure 16**. Exfiltration from the groundwater storage pond might effectively mitigate the limited drawdowns predicted by the model incorporating the flow barrier.

If the overburden soils in the northwestern quadrant of the cooling tower excavation extend below the water table, an additional flow barrier wall should be implemented to reduce the adverse impacts of the planned excavation.

8.0 CONCEPTUAL EFFLUENT DISCHARGE PLAN

8.1 General Approach

This section provides a conceptual plan for groundwater effluent discharge. The following potential discharges are considered:

- Discharge to surface waters;
- Re-injection; and,
- On-site beneficial use for dust control, concrete mix water, street cleaning, and soil compaction.

8.2 General Assumptions

It is considered for purposes of this conceptual plan that groundwater to be pumped from beneath the BBNPP site will generally be the calcium-magnesium-bicarbonate water type of moderate to high hardness, generally depressed dissolved oxygen content, generally elevated dissolved carbon dioxide content, and a production temperature on the order of 11° C (52° F). It is further considered that the water will be free from petroleum products, hazardous substances, and sources of ionizing radiation at significant concentrations (this consideration should be verified prior to initiating off-site discharges).

8.3 Discharge to Surface Water

Effluent from temporary construction dewatering is typically directed either to the nearest natural surface water body or storm water conveyance with sufficient capacity to accept the required rate of flow. Walker Run is the nearest water of the Commonwealth of Pennsylvania, and is designated as a cold water fishery in Chapter 93 of the Pennsylvania Code. According to information obtained during a non-specific inquiry with the Pennsylvania Department of Environmental Protection (PADEP) Northeast Regional Office, discharge of dewatering effluent to a designated cold water fishery may require an individual permit under National Pollutant Discharge Elimination System (NPDES) rules. PADEP also suggested that coordination with the county conservation district would be appropriate.

Under similar circumstances the discharge of dewatering effluent to a surface water stream has, in the experience of Weaver Boos, required that the water be conditioned by settling of suspended solids and aeration. Settling might be efficiently accomplished by routing the discharge through the temporary groundwater storage pond included in the site layout as shown on **Figures 20** and **22**. Aeration and temperature adjustment might be implemented using constructed rapids, falls, or use of spray aeration in the pond. Periodic monitoring of the influent and effluent water may be appropriate to mitigate potential impacts to the fishery. Additional or more specific requirements may be identified by regulatory authorities with federal, state, or local jurisdiction.

8.4 Re-Injection

Aquifer recharge by injection well(s) or recharge ponds may be considered. Without the use of a flow barrier, however, the volume of water requiring management is likely to be too great to manage in the vicinity of the site. If the flow barrier is utilized, the effluent, or a portion of the effluent, might be selectively re-injected or recharged to the aquifer as a means to mitigate impacts to the nearby wetlands. The temporary groundwater storage pond west of the ESWEMS pond excavation appears well located as a means to counter the limited drawdown projected in this area when the flow barrier is used during dewatering. Consideration should therefore be given to designing this temporary structure to recharge the groundwater at a rate appropriate for countering the indicated drawdown. However; groundwater recharge over extended periods can be difficult due to sedimentation or other fouling of the recharge structure. Provisions for maintenance of recharge facilities should therefore be included in final designs.

8.5 On-Site Beneficial Re-Use

Provided that the groundwater characteristics discussed in **Section 8.2** are met, a portion of the dewatering effluent may be diverted and should be readily useable for dust control, street cleaning, and soil compaction with little or no conditioning. Moreover, the water may be near potable quality, and could be suitable for use as concrete mix water. Prior to use as concrete mix water, the effluent should be tested for chloride, sulfate, alkalies, and total solids and checked against the requirements listed in ASTM C 1602. So long as the water is clear and non-turbid, it should meet applicable and relevant requirements.

9.0 CONCLUSIONS AND RECOMMENDATIONS

9.1 Conclusions

Weaver Boos has evaluated alternative dewatering strategies to assess the effectiveness and potential impacts of temporary construction dewatering both with and without the use of a flow barrier at the ESWEMS pond excavation. This approach was utilized to assess whether a slurry wall, diaphragm wall, or other type of subsurface flow barrier is appropriate to be installed around the ESWEMS pond before excavation is initiated to minimize groundwater intrusion into the excavation and to mitigate possible detrimental off-site effects of dewatering. The results of the evaluation support the following principal conclusions in accordance with hydrogeological principles and practice:

- Temporary dewatering implemented without the benefit of a flow barrier is likely to deeply depress (25 ft or more) the groundwater table at distances extending approximately 800 ft south of the proposed ESWEMS pond location. Significant adverse impacts may develop under this dewatering strategy, which include but may not be limited to:

    a. Impairment of nearby domestic and industrial water supply wells;

    b. Loss of stream flow in Walker Run;

    c. The unintended drainage of existing wetlands adjoining the site and elsewhere throughout the basin; and,

    d. The potential settlement of existing structures if founded in compressible soils.

    e. Management of dewatering system effluent flows of approximately 4,000 gpm during the initial dewatering and of approximately 1,000 gpm during steady state dewatering operations.

- Temporary dewatering implemented with the benefit of the flow barrier around the ESWEMS pond and pump house will likely require initial dewatering pump rates of approximately 600 gpm and steady state pumping of approximately 230 gpm. The groundwater model indicates these dewatering system pump rates may tend to depress the groundwater table by 5 to 10 ft in the areas immediately west and southwest of the flow barrier. It therefore remains possible with the proposed flow barrier that nearby wetlands may be drained or partially drained. Given the much lesser drawdown projected during dewatering using the flow barrier; however, the return of dewatering effluent to Walker Run and exfiltration through the temporary groundwater storage pond may be sufficient to counter stream losses and assist in maintaining water levels in the adjoining wetlands.

- Flow barriers should not be needed around the power block excavation. The groundwater model indicates dewatering of the temporary construction excavation will yield approximately 50 gpm (included in flow rates discussed in prior bullets), which can be controlled using ditches located at the toe of the excavation, at the soil rock interface and on the benches of the excavation.

- Based on the groundwater observations, flow barriers should not be needed around the cooling towers excavation. The model indicates the groundwater inflow into the excavation will be on the order of 70 gpm, which can also be controlled through appropriately placed ditches. However, if the overburden soils to be excavated from the northwest corner of the cooling towers extend below the level of the wetlands along Walker run west of the cooling tower, a flow barrier wall and active dewatering system may be required. An alternate solution to the planned excavation is the use of deep soil improvement techniques to reduce or eliminate the deep excavation of potentially water bearing overburden soils.

9.2 Recommendations

Prior to implementation of dewatering using the conceptual designs provided with this evaluation, Weaver Boos recommends the following:

- Subsurface conditions along the alignment of the proposed flow barrier and along the horizontal limits of the planned excavations should be better defined using soil borings advanced several feet into the underlying competent bedrock. Such borings should be advanced on 100 ft centers (or less) along the flow barrier alignment and at 200 ft centers (or less) along the perimeter of the excavations, and if significant variations in bedrock elevation are encountered, additional borings should be advanced to assess conditions in such areas.

- Groundwater conditions at the northwest corner of the cooling tower excavation should be defined by installing and monitoring groundwater monitoring wells in the overburden and upper bedrock.

- Conceptual evaluations presented herein should be reviewed to consider additional data and information as it becomes available and the conceptual designs further refined and developed to support development of final designs suitable for use in construction.

10.0 QUALIFICATIONS AND LIMITATIONS

Weaver Boos has performed this conceptual construction dewatering evaluation consistent with the principles of hydrogeology in accordance with the prevailing standards for professionals practicing under similar circumstances. This warranty is in lieu of all other warranties either expressed or implied.

This evaluation is conceptual in nature, and the conceptual evaluations presented herein will require confirmation and refinement prior to development of final designs for the purposes stated herein. This evaluation was prepared for our client and may not be adequate for use by other parties for other purposes. Data and information considered during this evaluation was developed primarily by others, and is assumed correct and complete as it was received by Weaver Boos. Independent validation of design parameters utilized in this assessment was specifically considered beyond the scope of services for this assignment by the client. It is noted that conditions at the time of site explorations completed by others may be subject to change. Soil and groundwater conditions in areas between soil borings and wells are interpolated or extrapolated, and may in fact differ from those assumed. Moreover, the application of subsurface hydraulic assessment methods applied herein is characterized by inherent uncertainty owing to a necessarily incomplete understanding of subsurface hydrological conditions. If during construction or at any other time, actual conditions are found to differ from those assumed in this report, Weaver Boos should be notified immediately so that the impact of the new information to our conclusions and recommendations contained herein may be properly considered.

The following specific technical qualifications and limitations should be considered by the users of this report:

- This evaluation was prepared using subsurface characterization data that are limited in several respects. Relatively few exploratory borings were drilled in the area of the cooling towers and ESWEMS pond. Subsurface conditions, including the depth to bedrock, are therefore uncertain in these areas and may differ significantly from the interpolations and extrapolations used to develop the excavation plans and groundwater potentiometric surface maps (prepared by others), which were used by Weaver Boos in this evaluation.
- Projected groundwater mass budgets, flow rates, drawdowns, and dewatering system yields are estimated based on digital flow models and manual calculations using available hydraulic conductivity and specific yield data. The actual groundwater flow

system may therefore differ from the conceptual models used in the digital and manual calculations.

- The dewatering operations, without a flow barrier and to a lesser extent without a flow barrier, evaluated herein will locally stress the groundwater flow system to a great degree. The aquifers' actual response to such stress (e.g., actual dewatering system flow rates, basin drawdown, and changes in the mass flow budgets) has not been verified at high rates of test pumping and may therefore vary significantly from the estimates projected herein.

11.0 REFERENCES CITED

1. Freeze, R.A., and J.A. Cherry. 1979. Groundwater. Prentice-Hall, Inc. Englewood Cliffs, NJ 07632.

2. Harbaugh, A.W., Banta, E.R., Hill, M.C., and McDonald, M.G., 2000, MODFLOW-2000, the U.S. Geological Survey modular ground-water model -- User guide to modularization concepts and the Ground-Water Flow Process: U.S. Geological Survey Open-File Report 00-92, 121 p.

3. Paul C. Rizzo Associates, Inc. 2010. Response to RFI SL-BBNPP-132, Approved for Use by UniStar August 12, 2010 (Final Boring Logs).

4. Paul C. Rizzo Associates, Inc., Response to RFI SL-BBNPP-111, Approved for Use by UniStar August 31, 2010 (3-Month Groundwater Monitoring data Report).

5. Paul C. Rizzo Associates, Inc., Response to RFI SL-BBNPP-149, Approved for Use by UniStar September 13, 2010 (Excavation Plans).

6. Powers, J.P. 1981. Construction Dewatering – A Guide to Theory and Practice. John Wiley & Sons, Inc.

7. Sargent & Lundy LLC, Conceptual Design of Stormwater Management, Bell Bend Nuclear Power Plant Unistar Nuclear Energy, Report SL-009446, Revision 5, July 28, 2010.

8. Sargent & Lundy LLC drawings:
   - *12198-400-015, Sheet 1, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 1".*
   - *12198-400-015, Sheet 2, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 2".*
   - *12198-400-015, Sheet 5, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 5".*
   - *12198-400-015, Sheet 6, Rev. 5, "Conceptual Grading & Drainage Plan, Sheet 6".*

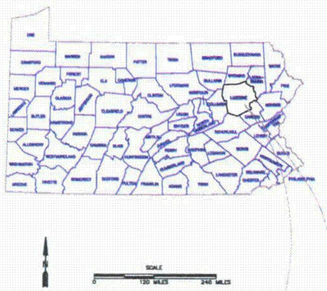
9. BBNPP, Final Safety Analysis Report, Section 2.4.12 – Groundwater, Rev. 2.

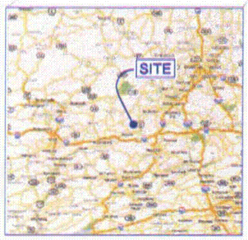
10. BBNPP, Final Safety Analysis Report, Section 2.5.4 – Stability of Subsurface Materials and Foundations, Rev. 2.

11. Departments of the Army, Navy, and Air Force. 1971. *Dewatering and Groundwater Control for Deep Excavations, Army TM 5-818-5, Navy NAVFAC No. P-418, Air Force AFM 88-5, Chap 6.* April.

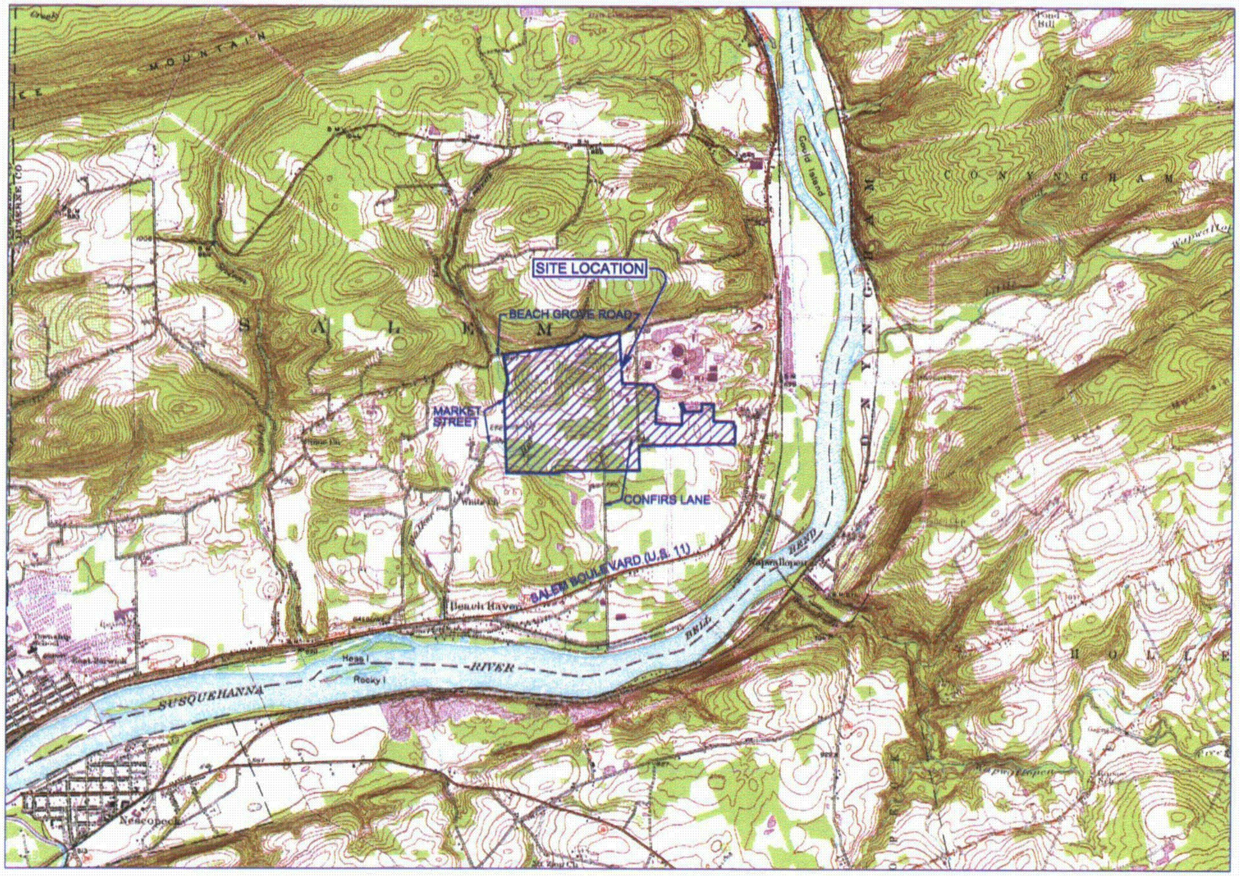
12. USGS 7.5 Minute Series Topographic Map, Berwick, Pa Quadrangle, 1955, Photorevised 1989.

STATE OF PENNSYLVANIA





VICINITY MAP, LUZERNE COUNTY



N

SCALE
0 4000' 8000'

NOT FOR CONSTRUCTION

NON-SAFETY RELATED

SOURCE: U.S.G.S. 7.5 MINUTE SERIES QUADRANGLE MAPS OF
BERWICK AND SYBERTSVILLE, PA. MAPS PHOTOREVISED 1989.

SITE LOCATION PLAN

BELL BEND NUCLEAR POWER PLANT
UNISTAR NUCLEAR
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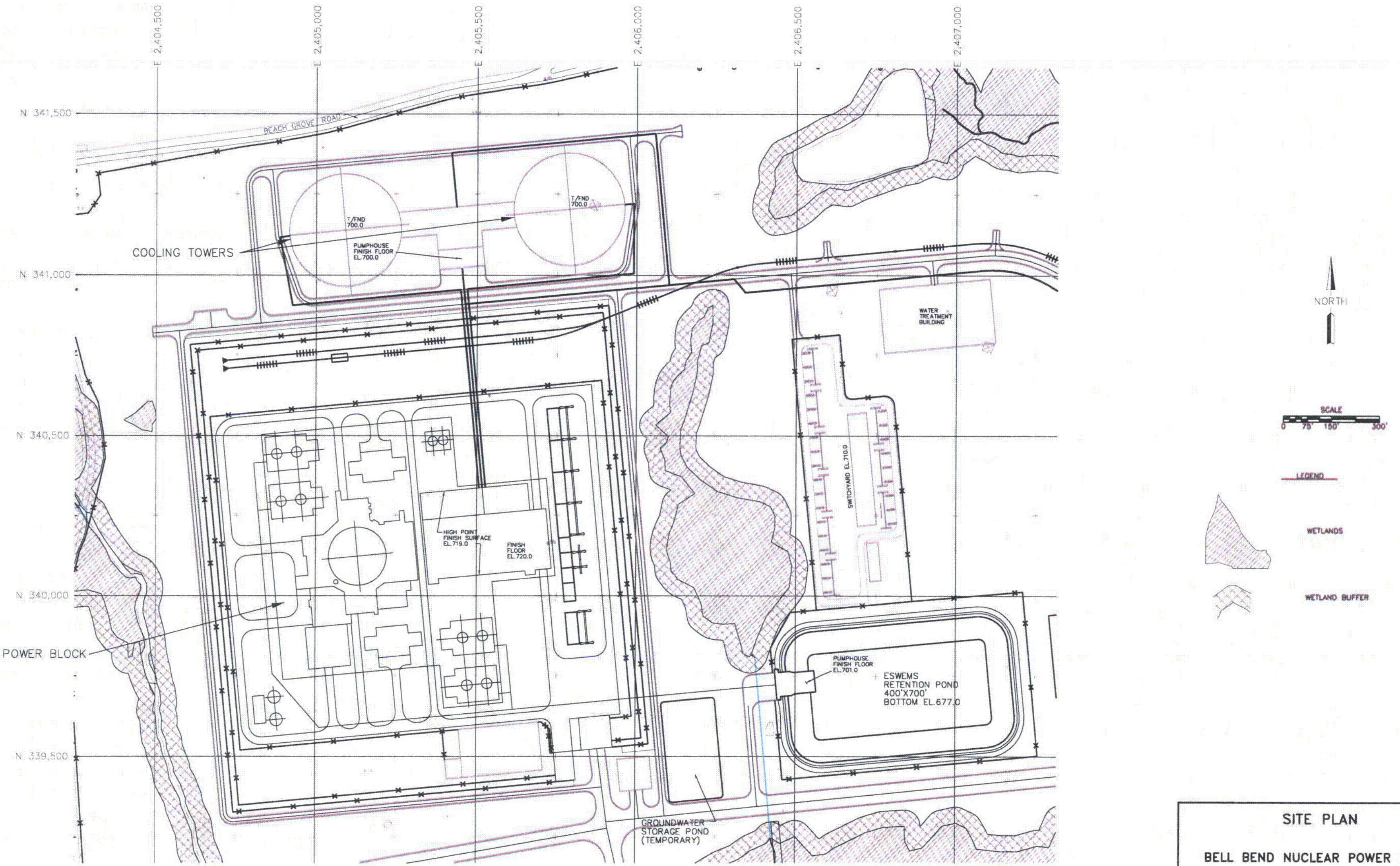
DRAWN BY: TAG	DATE: 9/22/10	FILE: 2524301-01-01
REVIEWED BY: SMS	CAD: DRAWING1.DWG	FIGURE 1



DRAWING ADAPTED FROM REFERENCE 8.

SITE PLAN

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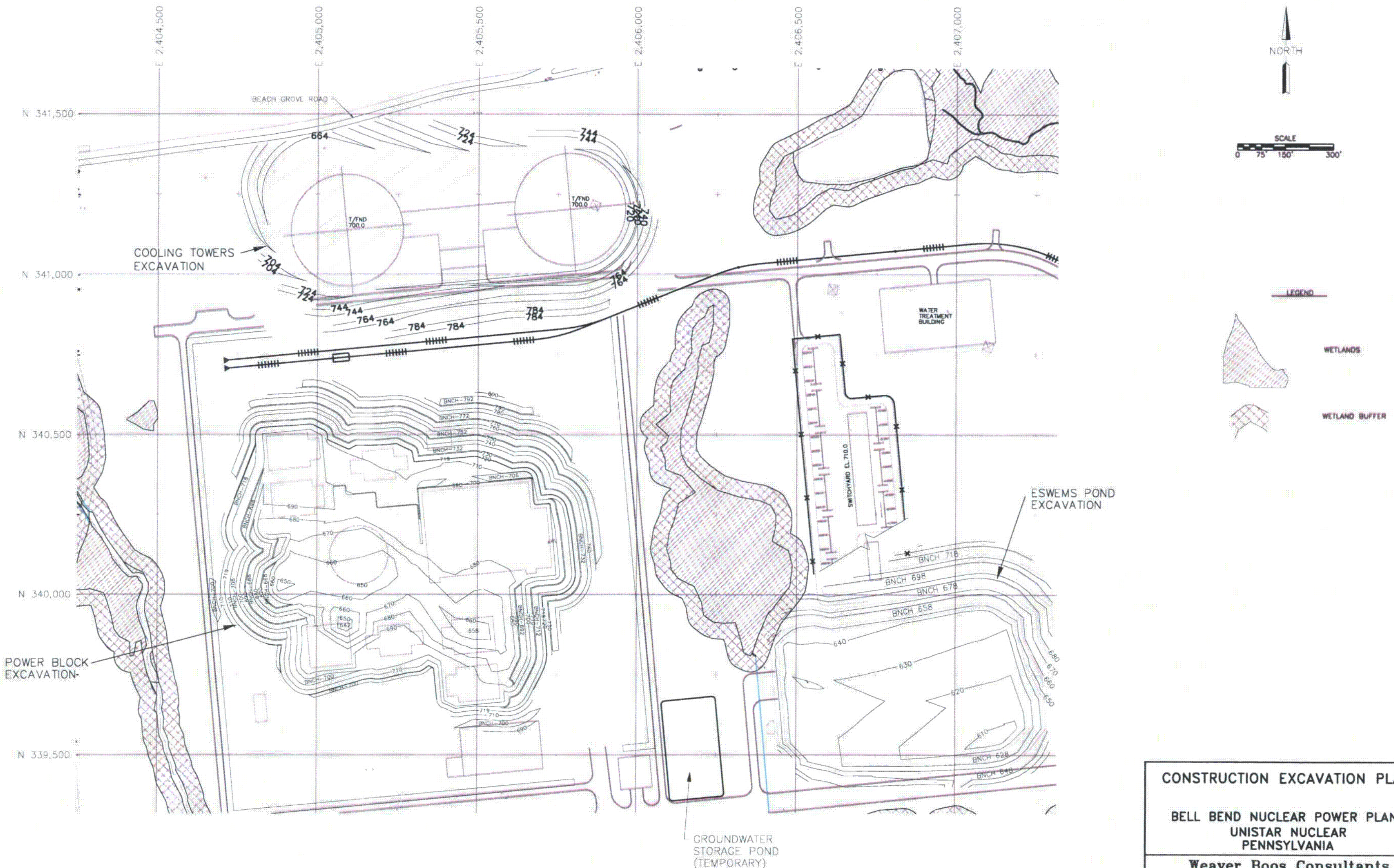
DRAWN BY: TAG	DATE: 9/16/10	FILE: 2524301-01-01
REVIEWED BY: SMS	CAD: FIG2.DWG	FIGURE 2



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CONSTRUCTION EXCAVATION PLAN

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REVIEWED BY: SMS	CAD: FIG3.DWG	FIGURE 3

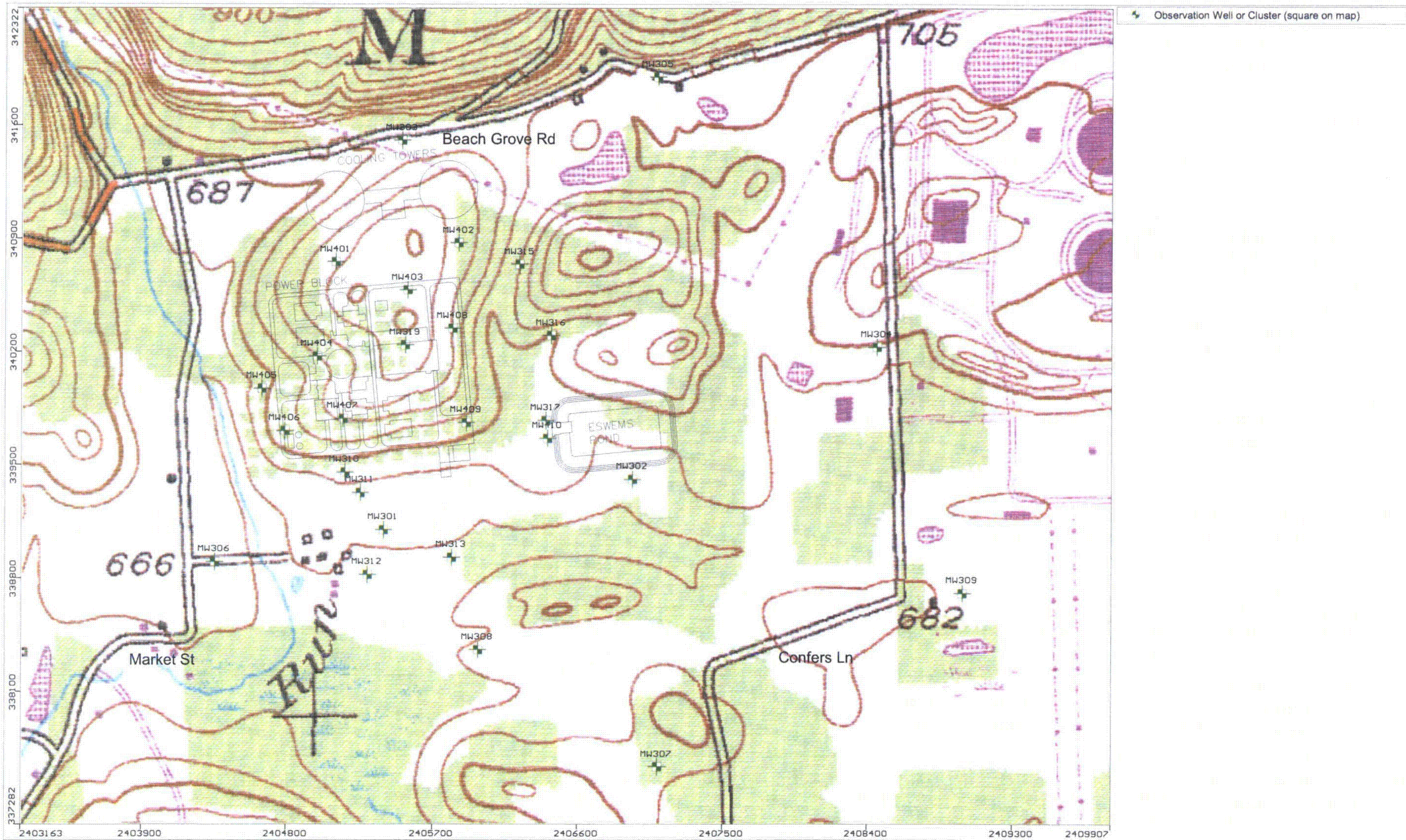

FIGURE 4
Groundwater Observation Well Locations
(References 4 and 9)

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FIGURE 5
Groundwater Flow Model Domain

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FIGURE 6
Layer 1 Conductivity Zones and River Boundaries

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FIGURE 7
Layer 1 Recharge Boundary Distribution (in/yr)

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FIGURE 8
Layers 2 - 7 Fixed Head Boundaries

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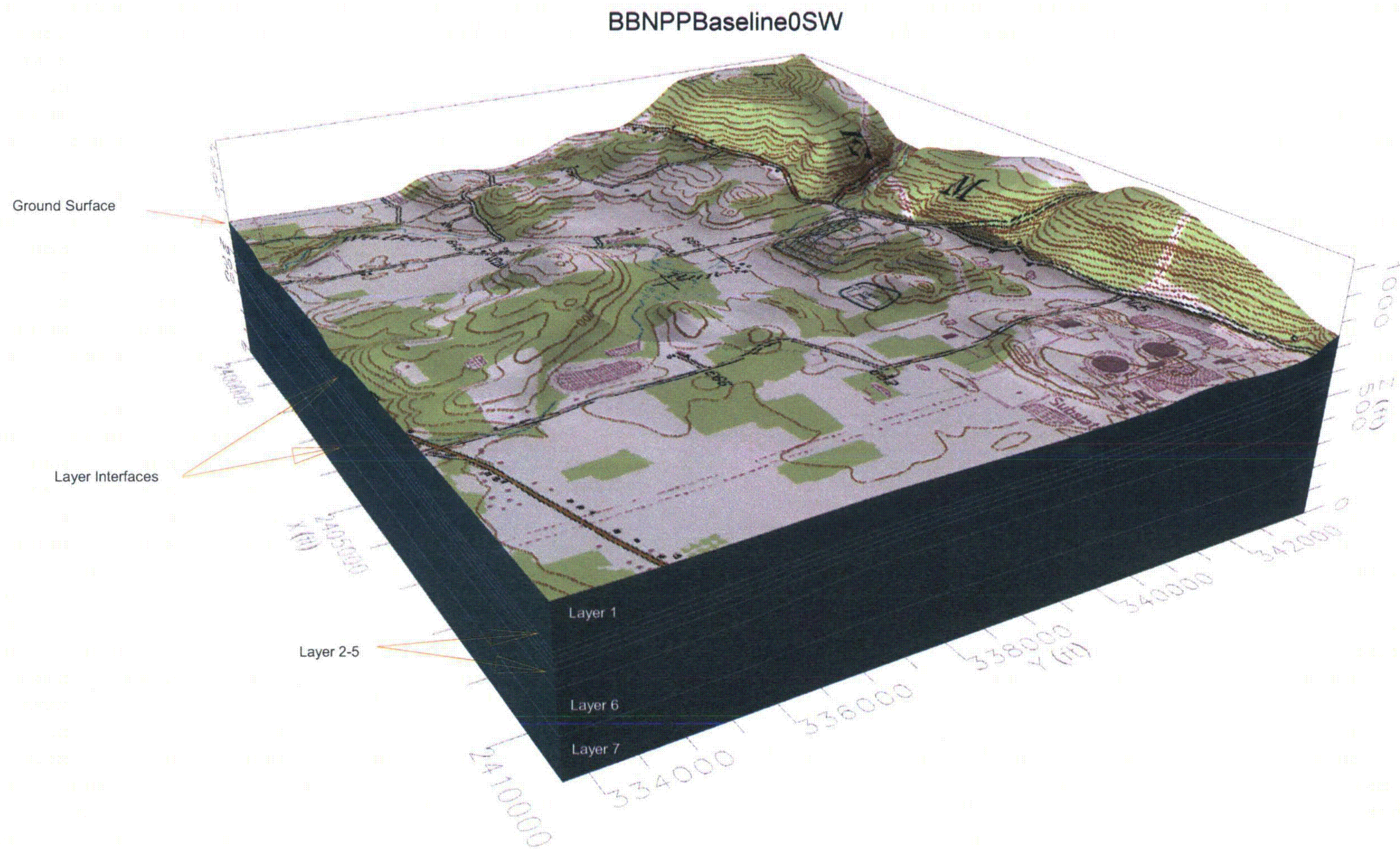

FIGURE 9
Block View of Model Area

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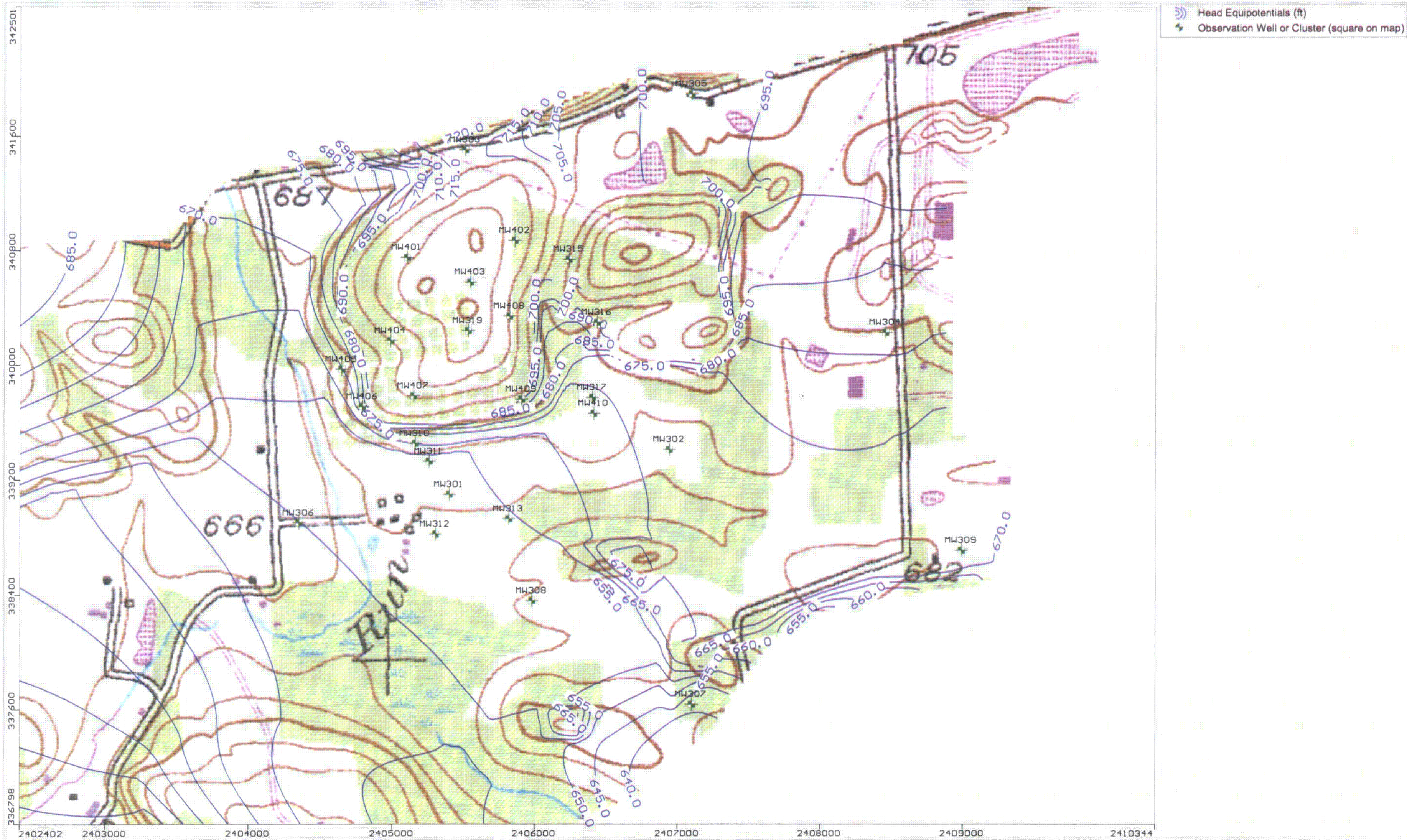

FIGURE 10
Layer 1 Baseline Simulated Head

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FIGURE 11
Layer 2 Baseline Simulated Head

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FIGURE 12
Layer 1 Dewatering Plan (Without Flow Barrier at ESWEMS)

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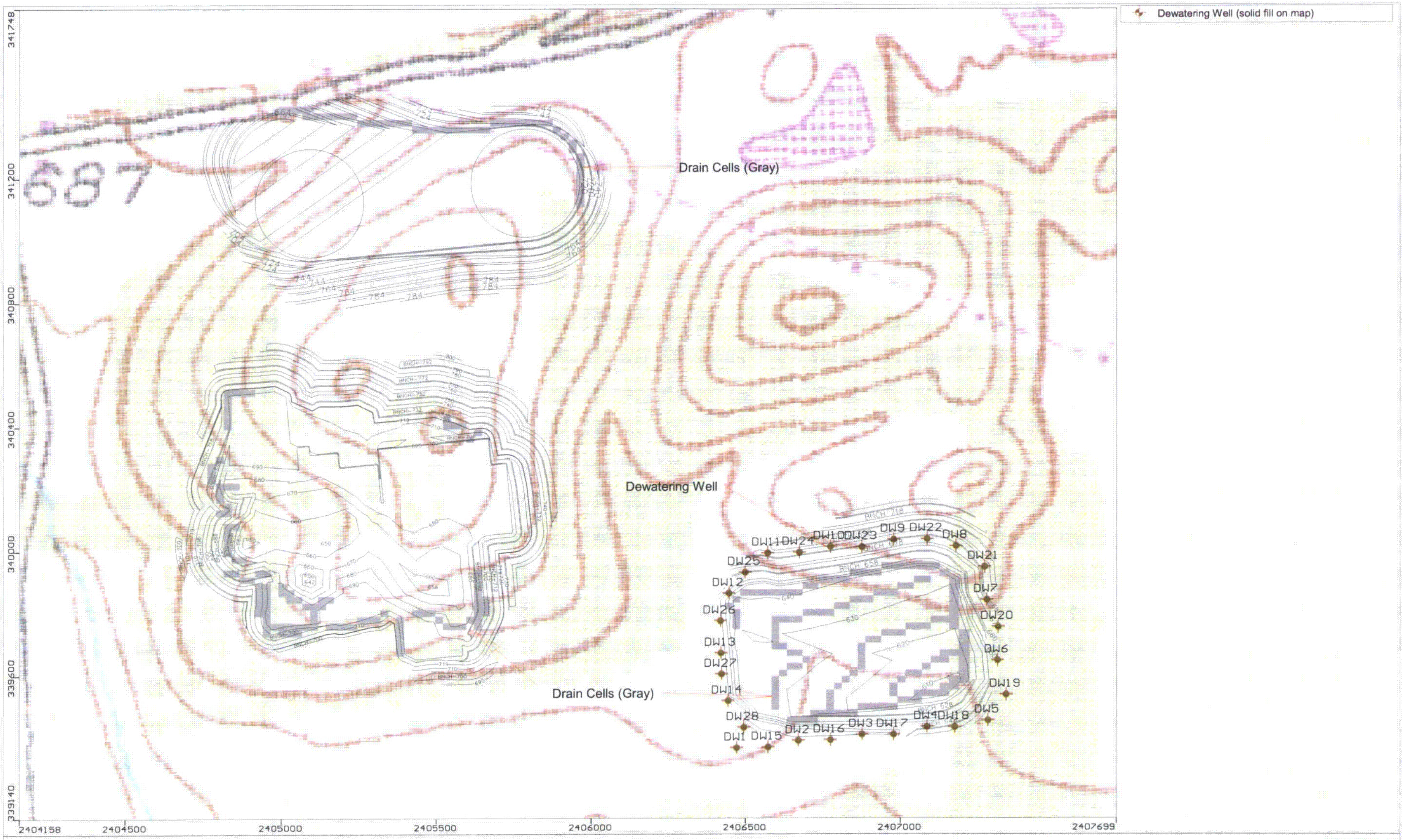

FIGURE 13
Layer 2 Dewatering Plan (Without Flow Barrier at ESWEMS)

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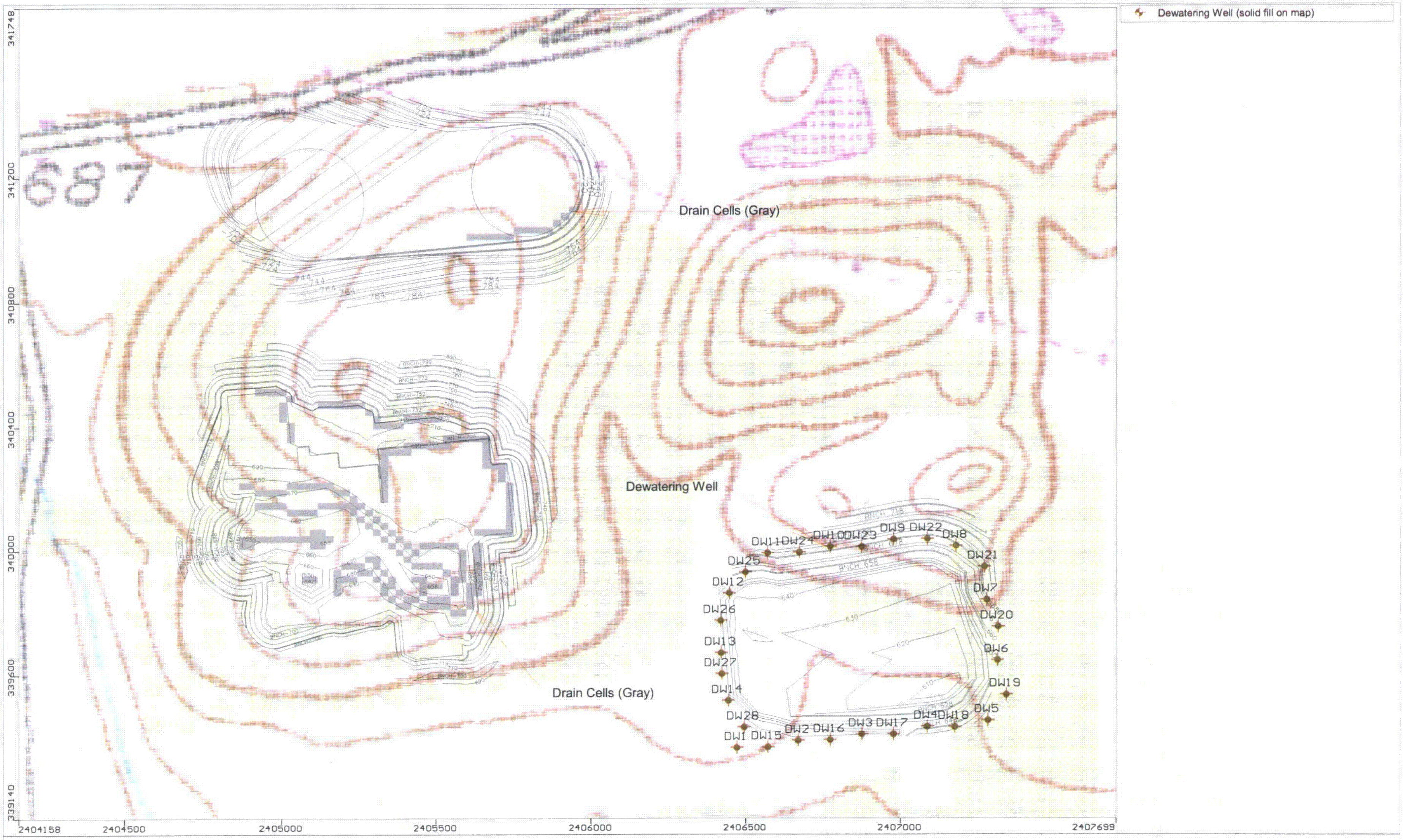

FIGURE 14
Layer 3 Dewatering Plan (Without Flow Barrier at ESWEMS)

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FIGURE 15
Drawdown in Overburden Aquifer Without Flow Barrier at ESWEMS

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FIGURE 16
Drawdown in Overburden Aquifer With Flow Barrier at ESWEMS

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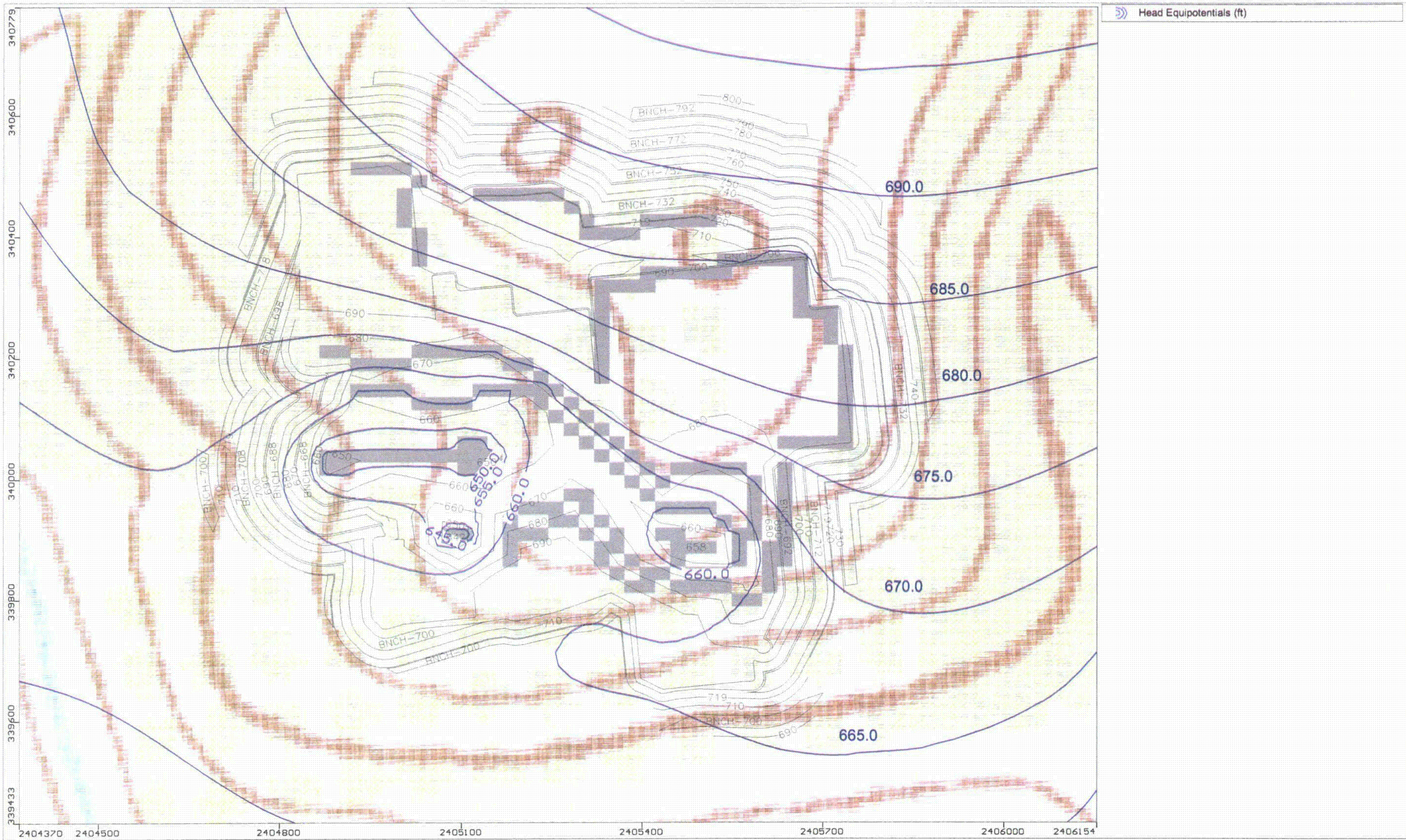

FIGURE 17
Power Block Dewatered Head, Layer 3

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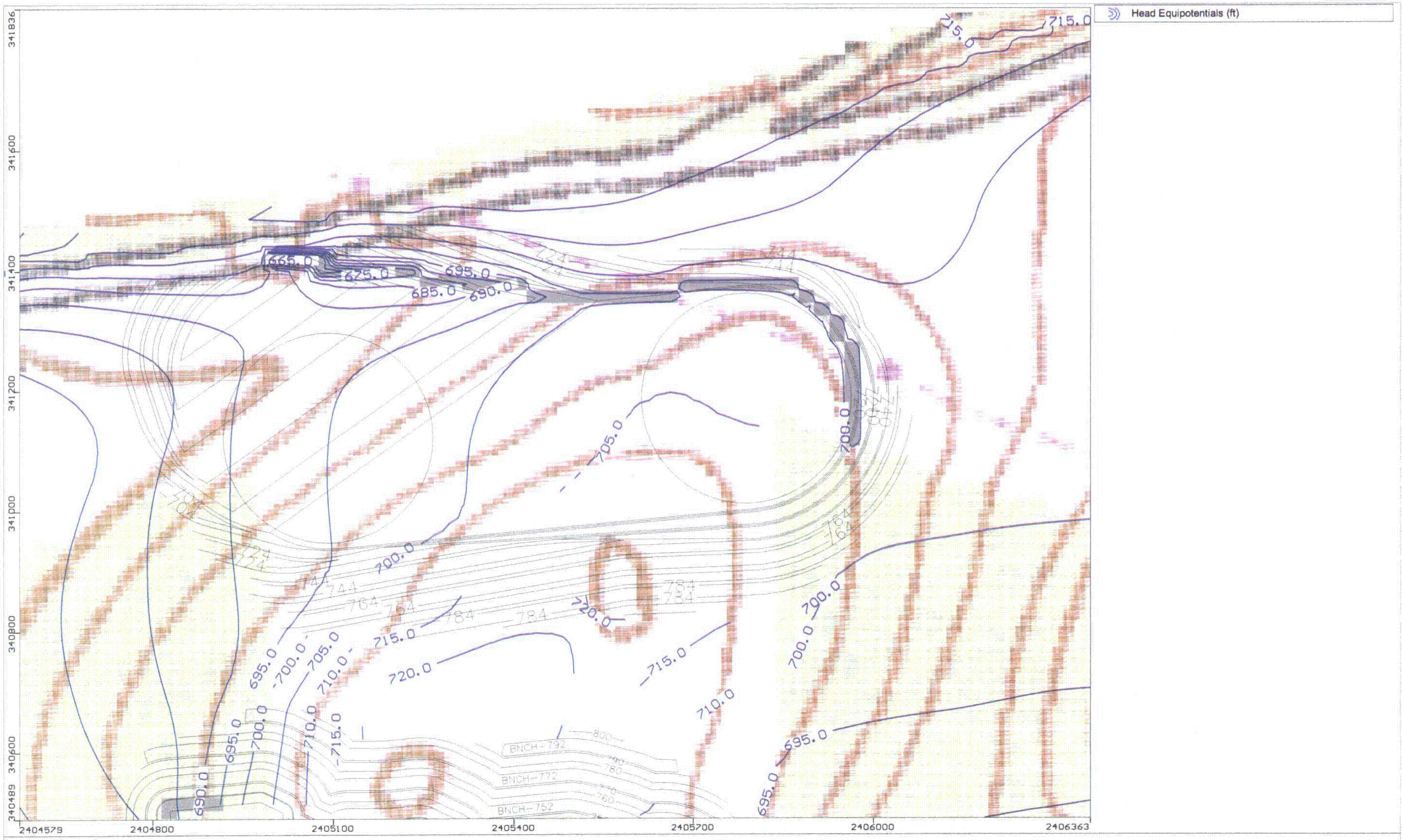

FIGURE 18
Cooling Towers Dewatered Head, Layer 2

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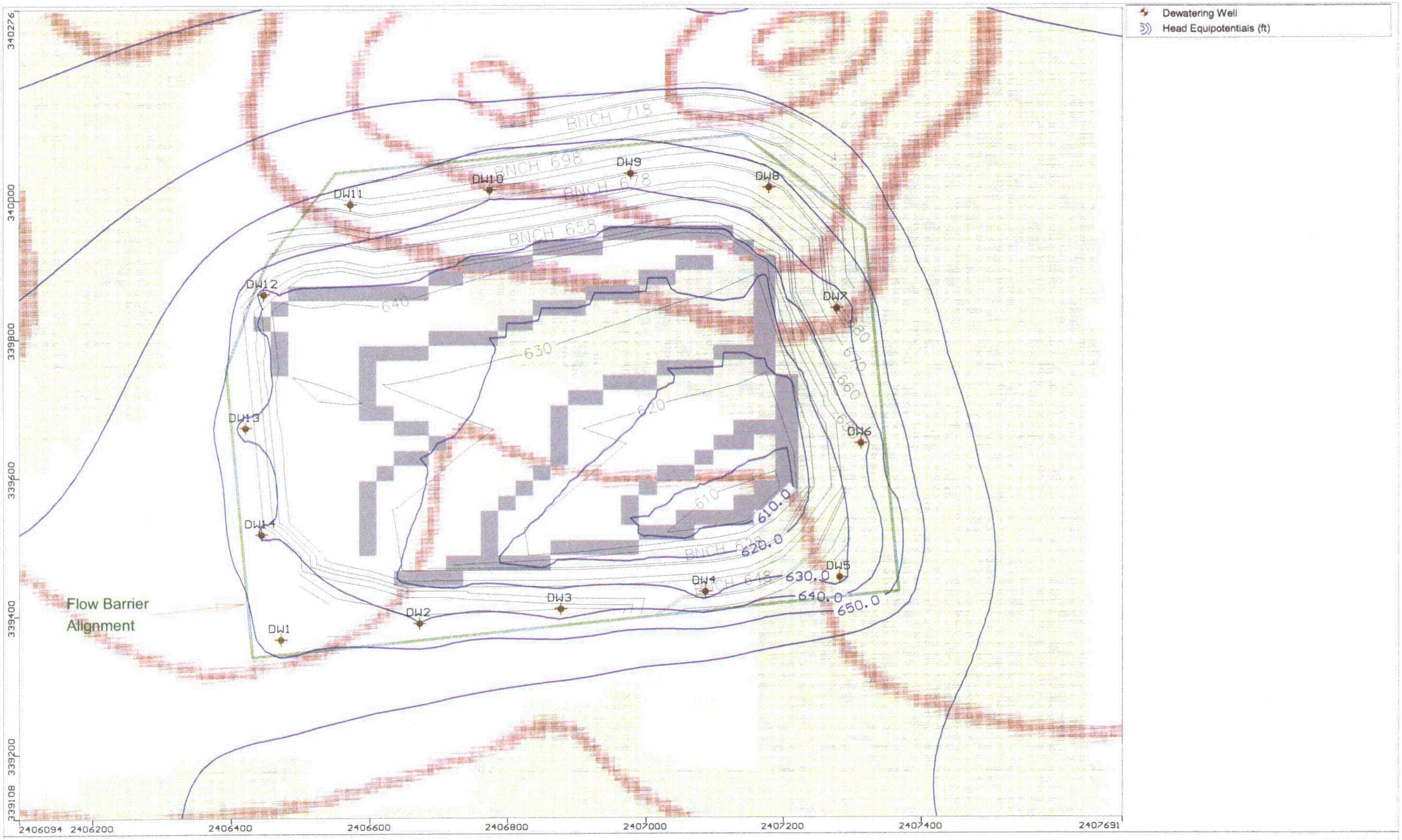

FIGURE 19
ESWEMS Pond Dewatered Head, Layer 2

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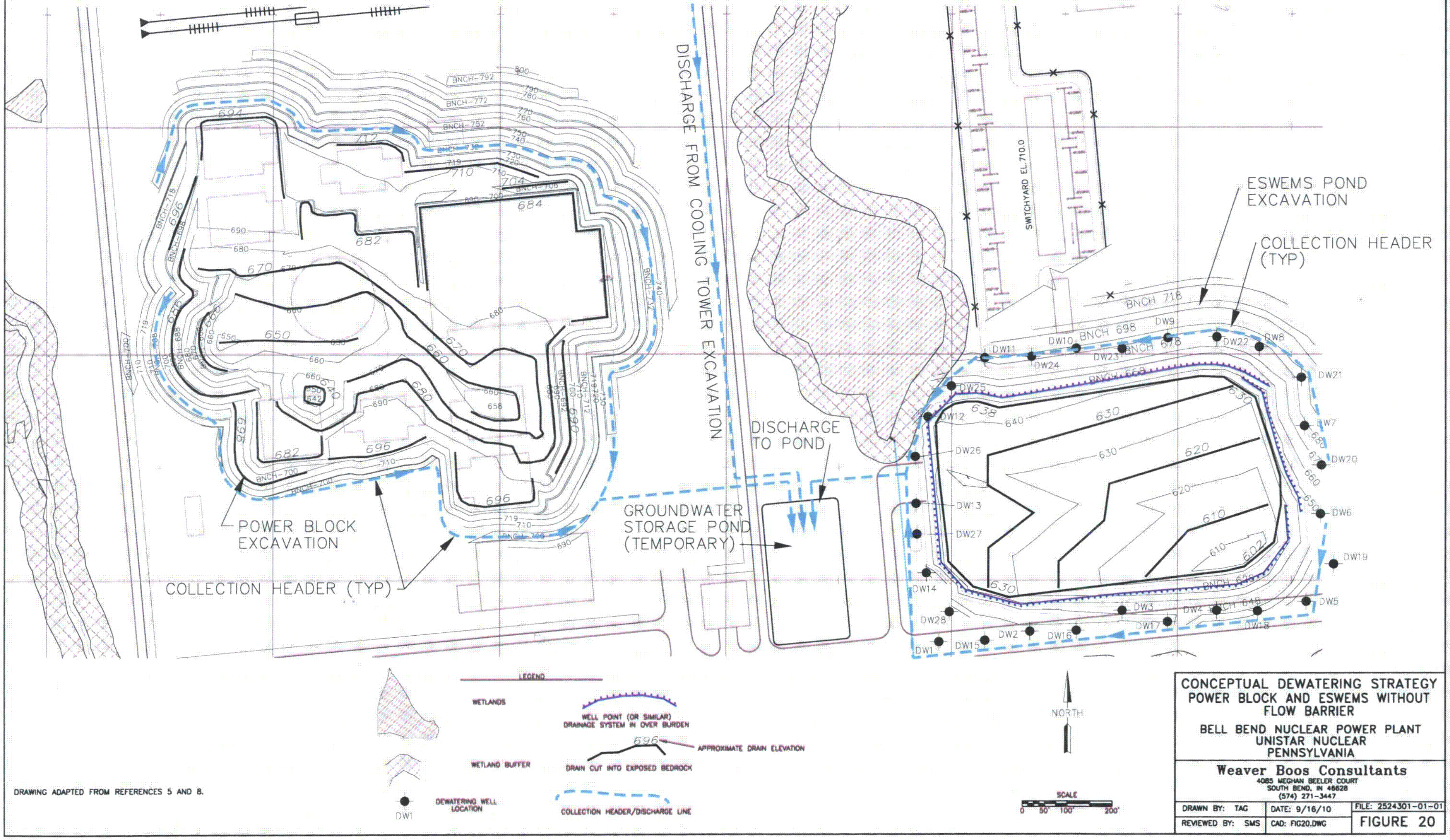

CONCEPTUAL DEWATERING STRATEGY
POWER BLOCK AND ESWEMS WITHOUT FLOW BARRIER

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UNISTAR NUCLEAR
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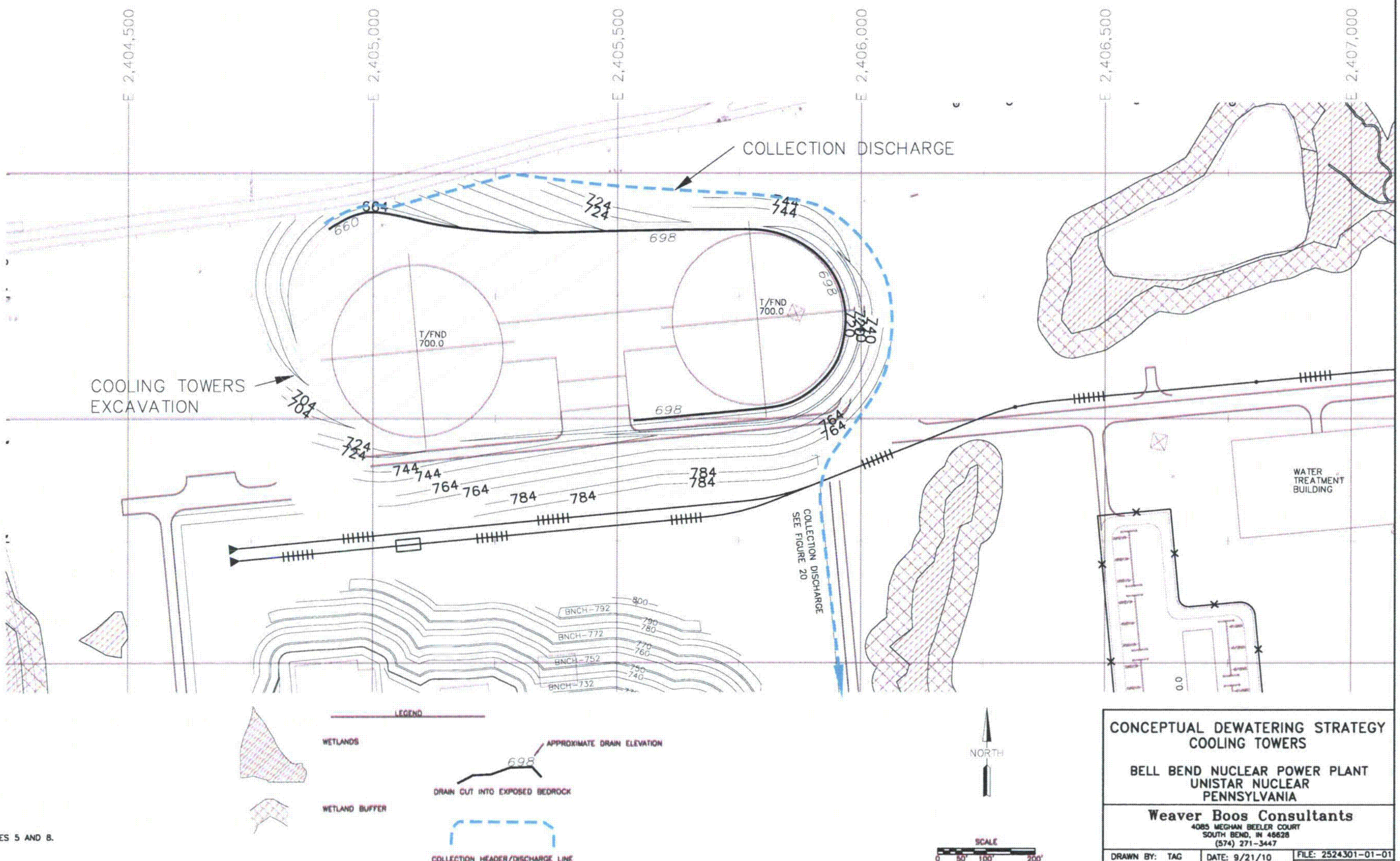
REVIEWED BY: SMS	CAD: FIG20.DWG	FIGURE 20



DRAWING ADAPTED FROM REFERENCES 5 AND 8.

CONCEPTUAL DEWATERING STRATEGY
COOLING TOWERS

BELL BEND NUCLEAR POWER PLANT
UNISTAR NUCLEAR
PENNSYLVANIA

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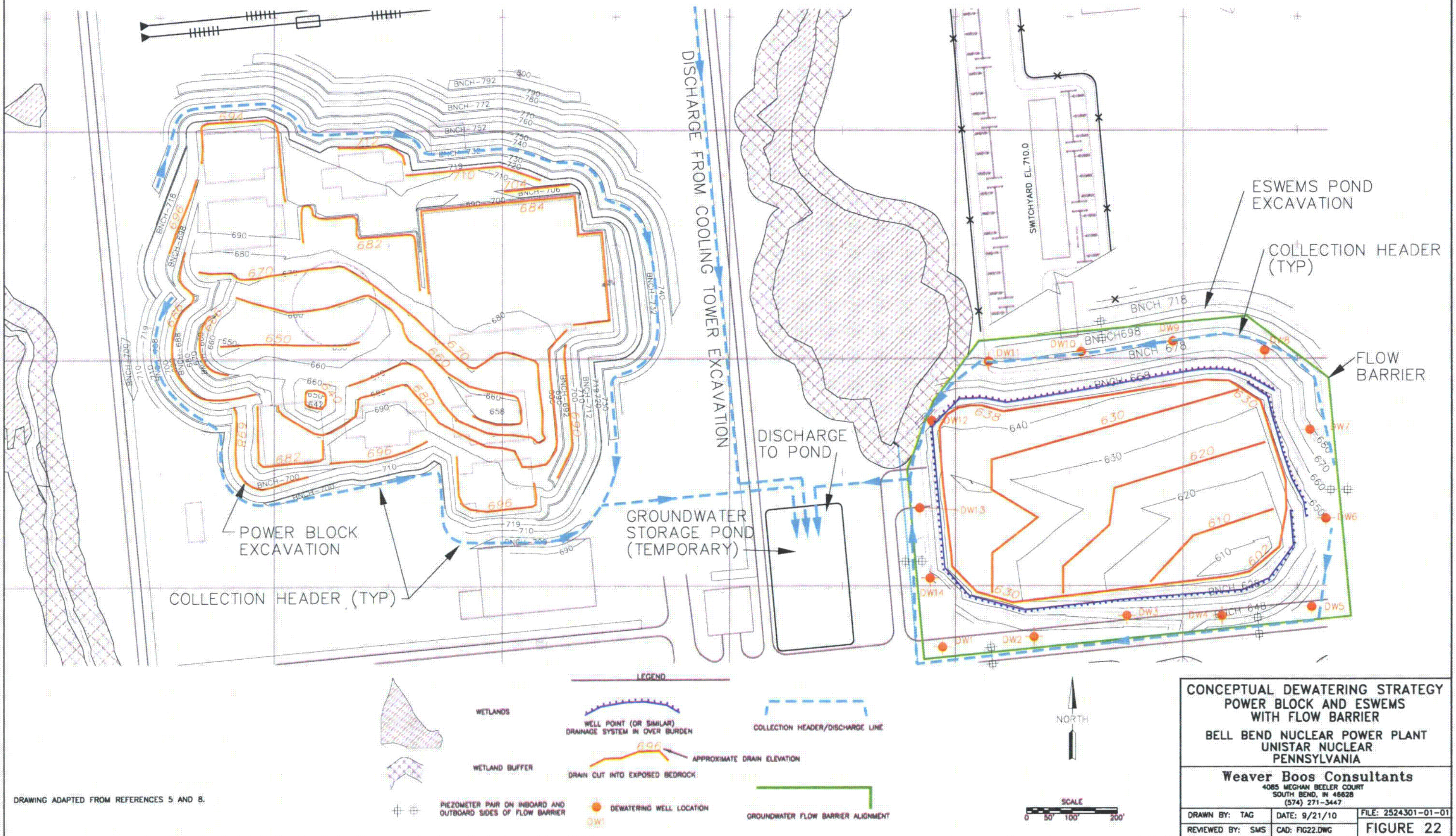
REVIEWED BY: SMS	CAD: FIG21.DWG	FIGURE 21



DRAWING ADAPTED FROM REFERENCES 5 AND 8.

CONCEPTUAL DEWATERING STRATEGY POWER BLOCK AND ESWEMS WITH FLOW BARRIER BELL BEND NUCLEAR POWER PLANT UNISTAR NUCLEAR PENNSYLVANIA	
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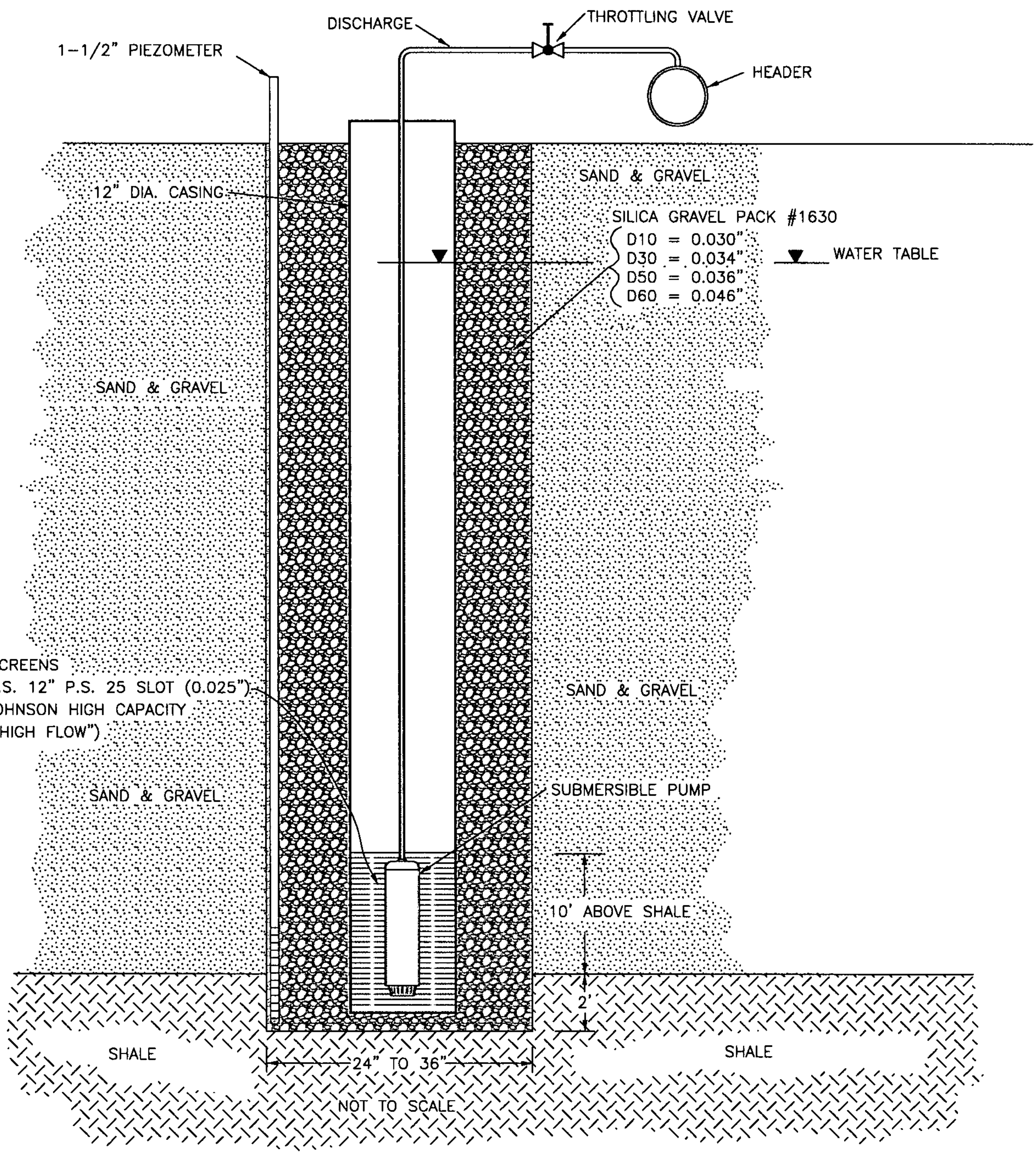
REVIEWED BY: SMS / CAD: FIG22.DWG	FIGURE 22

TYPICAL DEWATERING WELL



NOTES:
1) SUBMERSIBLE PUMP RATED AT APPROXIMATELY 100 TO 150 GPM
2) 1" DIA. SAMPLING PORT AT EACH PIPE DISCHARGE

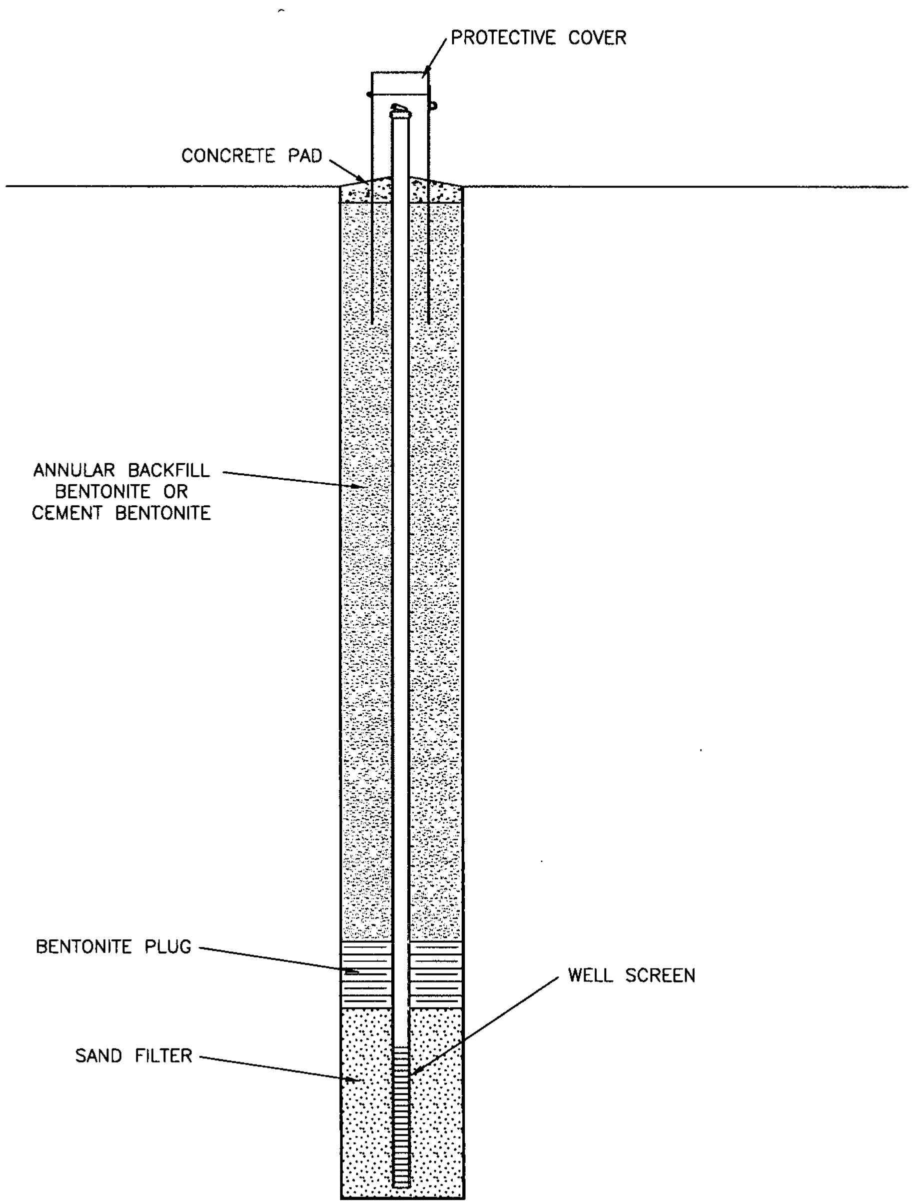
TYPICAL
DEWATERING WELL SCHEMATIC
BELL BEND NUCLEAR POWER PLANT
UNISTAR NUCLEAR
PENNSYLVANIA

WEAVER BOOS CONSULTANTS
NORTH CENTRAL, LLC

DRAWN BY: TAG	DATE: 09/21/10	FILE: 2524301-01-01
REVIEWED BY: SMS	CAD: WELLSCHEM.DWG	FIGURE 23

K:\WBGM\CLIENT INFORMATION\2500-2599\2524\301\01\01\P100133 - S&L BELL BEND 2\WELLSCHEM.DWG

TYPICAL 2-INCH MONITORING WELL SCHEMATIC



NOT TO SCALE

TYPICAL MONITORING WELL (PIEZOMETER) SCHEMATIC
BELL BEND NUCLEAR POWER PLANT
UNISTAR NUCLEAR
PENNSYLVANIA

WEAVER BOOS CONSULTANTS NORTH CENTRAL, LLC

DRAWN BY: TAG	DATE: 09/21/10	FILE: 2524301-01-01
REVIEWED BY: SMS	CAD: PIEZSCHEM.DWG	FIGURE 24

K:\WBGM\CLIENT INFORMATION\2500-2599\2524\301\01\01\P100133 - S&L BELL BEND 2\PIEZSCHEM.DWG

TABLE 1
Hydraulic Properties Summary
Proposed BBNPP Facility
Berwick, Pennsylvania

Observation Well ID	Test Type	Hydraulic Conductivity		Storage Coefficient, S	Specific Yield, Sy
		ft/day	cm/s	(unitless)	(unitless)
Glacial Overburden Pumping Test (Pumping Well = MW302A1)[1] (EL. 645 to 630)					
MW302A2	Pumping	1.10E+02	3.88E-02	NA	5.00E-01
	Recovery	1.67E+02	5.89E-02	NA	NA
MW302A3	Pumping	1.03E+02	3.63E-02	NA	2.53E-01
	Recovery	3.57E+02	1.26E-01	NA	NA
MW302A4	Pumping	1.13E+02	3.99E-02	NA	3.22E-01
	Recovery	2.92E+02	1.03E-01	NA	NA
Geometric Mean[2]		1.68E+02	5.92E-02	NA	3.44E-01
Median		1.40E+02	4.94E-02	NA	3.22E-01

Shallow Bedrock Pumping Test (Pumping Well = MW404)[3] (EL. 670 to 618)					
MW405	Pumping	3.45E+00	1.22E-03	2.60E-04	NA
	Recovery	1.75E+00	6.17E-04	1.84E-04	NA
MW407	Pumping	3.25E+00	1.15E-03	1.15E-04	NA
	Recovery	2.42E+00	8.54E-04	2.14E-04	NA
Shallow Bedrock Pumping Test (Pumping Well = MW405)[3] (EL. 670 to 618)					
MW404	Pumping	2.45E+00	8.64E-04	2.88E-04	NA
	Recovery	2.78E-01	9.81E-05	1.43E-04	NA
MW407	Pumping	1.47E+00	5.19E-04	2.33E-04	NA
	Recovery	6.59E-01	2.32E-04	1.91E-04	NA
Shallow Bedrock Pumping Test (Pumping Well = MW407)[3] (EL. 670 to 620)					
MW404	Pumping	2.53E+00	8.93E-04	1.79E-04	NA
	Recovery	1.08E+00	3.81E-04	1.76E-04	NA
MW409	Pumping	1.64E+00	5.79E-04	7.27E-06	NA
	Recovery	1.26E+00	4.45E-04	6.42E-06	NA
Geometric Mean[2]		1.54E+00	5.43E-04	1.10E-04	NA
Median		1.64E+00	5.79E-04	1.79E-04	NA

Deep Bedrock Pumping Test (Pumping Well = MW301B1)[4] (EL. 582 to 502)					
MW301B2	Pumping	2.38E-01	8.40E-05	8.37E-05	NA
	Recovery	2.51E+00	8.85E-04	5.50E-04	NA
MW301B3	Pumping	2.58E-01	9.10E-05	5.37E-05	NA
	Recovery	2.05E+00	7.23E-04	2.52E-04	NA
MW301B4	Pumping	5.46E-02	1.93E-05	1.25E-05	NA
	Recovery	5.77E-01	2.04E-04	7.41E-05	NA
Geometric Mean[2]		4.64E-01	1.64E-04	9.12E-05	NA
Median		4.18E-01	1.48E-04	7.89E-05	NA

(yellow box) - Design value for non-safety related temporary construction dewatering.

(1) - Reference 9, Table 2.4-51.

(2) - Hydraulic conductivities measured by pump test are horizontal (K_h). Vertical (K_v) is considered to be $K_h/10$.

(3) - Paul C. Rizzo Associates, Inc., August 24, 2010, per SL-BBNP-111 Response (Reference 4).

(4) - Reference 9, Table 2.4-54.

TABLE 2
Head Observations Used in Dewatering Evaluation
Proposed BBNPP Facility
Berwick, Pennsylvania

Well ID	X (Easting)	Y (Northing)	Screen Center Point Elevation (ft) (NAVD88)	Head on 01/26/08 (ft) (NAVD 88)	Head on 03/24/08 (ft) (NAVD88)	Head on 5/6-7/2010 (ft) (NAVD88)	Head on 05/20/2010 (ft) (NAVD88)	Head on 06/29/2010 (ft) (NAVD88)	Head on 07/27/2010 (ft) (NAVD88)	Flow Model Calibration Value (ft) (NAVD88)
Overburden Aquifer										
MW301A	2,405,396.73	339,097.64	633.5	657.68	659.33	656.81	NM	NM	654.77	661.3 (1)
MW302A1	2,406,939.74	339,410.17	637.7	661.57	663.85	660.24	NM	658.47	657.67	665.9 (1)
MW302A2	2,406,925.67	339,410.07	637.8	661.58	663.84	660.22	NM	658.44	657.65	665.8 (1)
MW302A3	2,406,899.92	339,410.16	637.1	661.53	663.79	660.14	NM	658.38	657.58	665.8 (1)
MW302A4	2,406,939.42	339,495.31	635.6	661.57	663.86	660.27	NM	658.49	657.70	665.9 (1)
MW303A	2,405,505.31	341,504.72	711.1	714.32	717.11	713.99	NM	713.22	712.68	719.1 (1)
MW304A	2,408,455.38	340,228.16	653.6	671.05	672.16	670.45	NM	668.92	668.16	674.2 (1)
MW305A1	2,407,090.85	341,896.43	682.3	706.86	708.01	706.02	NM	704.50	704.02	710.0 (1)
MW305A2	2,407,096.81	341,888.61	648.6	706.44	707.23	705.57	NM	704.24	703.76	709.2 (1)
MW306A	2,404,351.67	338,899.63	632.0	655.93	657.07	655.05	NM	653.98	653.59	659.1 (1)
MW307A	2,407,085.99	337,632.51	659.1	684.65	685.82	684.69	NM	682.22	680.55	687.8 (1, 5)
MW308A	2,405,979.80	338,355.50	637.9	656.21	657.02	655.69	NM	654.45	653.88	659.0 (1)
MW309A	2,408,989.20	338,707.94	657.5	669.25	670.57	667.44	NM	665.33	664.47	672.6 (1)
MW310A	2,405,156.30	339,453.78	660.3	659.25	661.09	657.83	NM	NM	655.86	663.1 (1)
MW410	2,406,412.50	339,662.11	650.5	NI	NI	NM	658.91	657.55	656.58	663.5 (2)
Shallow Bedrock Aquifer										
MW301B1	2,405,384.28	339,098.94	517.4	659.37	660.62	659.05	NM	NM	656.99	662.6 (1)
MW301B2	2,405,338.53	339,142.99	524.2	660.69	659.28	656.83	NM	655.50	654.78	661.3 (1)
MW301B3	2,405,288.63	339,069.30	572.1	657.22	658.64	656.38	NM	654.94	654.58	660.6 (1)
MW301B4	2,405,444.97	338,987.79	568.5	657.80	658.98	652.90	NM	NM	654.94	661.0 (1)
MW303B	2,405,493.42	341,504.61	646.5	717.64	720.27	716.94	NM	715.85	715.28	722.3 (1)
MW304B	2,408,443.45	340,245.01	510.3	670.60	671.56	669.58	NM	668.55	667.48	673.6 (1)
MW305B	2,407,108.09	341,880.51	584.1	706.35	707.09	705.48	NM	704.20	703.71	709.1 (1)
MW308B	2,405,969.62	338,356.71	592.0	600.48	588.69	606.77	NM	607.24	610.66	612.7 (1)
MW309B	2,408,999.09	338,708.71	523.2	666.61	667.33	665.17	NM	663.52	662.79	669.3 (1)
MW310B	2,405,176.41	339,454.71	595.3	664.81	666.24	668.54	NM	667.17	666.40	668.4 (1)
MW311B	2,405,252.94	339,328.29	578.9	659.47	661.17	658.25	NM	NM	656.02	663.2 (1)
MW312B	2,405,297.70	338,820.62	564.4	656.99	658.20	655.77	NM	654.65	654.11	660.2 (1)
MW313B	2,405,815.58	338,927.92	567.7	658.24	659.97	656.89	NM	655.55	654.84	662.0 (1)
MW313C	2,405,754.79	338,922.54	537.2	658.24	658.01	656.84	NM	655.54	654.84	660.0 (1)
MW315B	2,406,234.46	340,738.30	660.1	718.67	719.79	717.15	NM	715.42	714.83	721.8 (1)
MW316B	2,406,433.93	340,298.18	632.4	693.54	693.78	696.68	NM	692.84	691.55	698.7 (1)
MW317B	2,406,401.48	339,772.49	621.2	660.78	662.91	659.47	NM	657.79	656.92	664.9 (1)
MW318B	2,405,516.32	340,493.18	741.3	759.15	761.04	757.47	NM	754.66	751.35	763.0 (1, 5)
MW319B	2,405,528.14	340,239.46	700.6	719.19	721.71	715.26	NM	709.51	707.11	723.7 (1)
MW401	2,405,097.68	340,753.25	645.4	NI	NI	NM	696.70	693.78	692.47	701.6 (3)
MW402	2,405,855.94	340,870.66	683.2	NI	NI	NM	720.59	719.54	718.79	725.5 (3)
MW403	2,405,542.37	340,579.28	650.0	NI	NI	NM	700.27	699.08	696.65	705.2 (3)
MW404	2,404,985.30	340,170.50	655.4	NI	NI	NM	700.16	696.92	695.55	705.1 (3)
MW405	2,404,646.35	339,970.47	633.4	NI	NI	NM	680.51	680.08	679.99	685.5 (3)
MW406	2,404,789.81	339,710.35	637.5	NI	NI	NM	669.70	668.99	668.36	674.6 (3)
MW407	2,405,144.25	339,784.93	634.8	NI	NI	NM	697.34	693.99	692.59	702.3 (3)
MW408	2,405,819.88	340,342.30	652.0	NI	NI	NM	705.86	703.68	702.25	710.8 (3)
MW409	2,405,905.35	339,760.65	635.8	NI	NI	NM	696.92	693.64	692.23	701.9 (3)
Deep Bedrock Aquifer										
MW302B	2,406,954.17	339,409.88	460.3	667.42	667.42	666.95	NM	665.19	664.42	669.4 (1)
MW303C	2,405,483.36	341,503.54	492.9	704.18	704.70	698.39	NM	698.59	697.65	706.7 (1)
MW304C	2,408,449.59	340,236.49	300.6	670.43	671.14	670.45	NM	669.26	668.23	673.1 (1)
MW306C	2,404,353.48	338,889.03	357.5	656.79	657.82	657.42	NM	656.40	655.84	659.8 (1)
MW307B	2,407,096.69	337,632.75	428.3	621.15	637.52	625.42	NM	616.57	613.54	639.5 (1)
MW310C (4)	2,405,233.06	339,452.09	590.9	678.35	678.35	678.35	NM	678.35	678.35	680.4 (1)
MW311C	2,405,413.69	339,313.21	476.1	528.81	534.15	597.64	NM	600.54	601.99	604.0 (1, 5)

- Observed maximum head.
- Design value for non-safety related temporary dewatering (observed maximum, or observed maximum corrected to 2008, plus 2.0 ft).

(1) - Maximum observed head plus 2.0 ft.
(2) - Maximum value measured during 2010 plus 4.6 feet to correct the observed value to the March 2008 maximum.
(3) - Maximum value measured during 2010 plus 5.0 feet to correct the observed value to the March 2008 maximum.
(4) - Artesian pressure causes this well to flow. The groundwater head is assumed equal to the top of the well casing (EL. 678.35).
(5) - MW-307A in different basin, omitted. MW318B likely a perched condition, omitted. MW311C is non-responsive, omitted.
NM - Not measured.
NI - Not Installed.

K:\Wbgm\Client Information\2500-2599\2524\301\01\01\Report\Tables\T2 Head Levels.xls

TABLE 3

Mahantango Shale Formation Elevation

Proposed BBNPP Facility

Berwick, Pennsylvania

Boring I.D.	X (Easting)	Y (Northing)	Top of Shale Formation El. (ft) (NAVD88)
B-301	2,405,430.7	339,151.8	625.4 (1)
B-302	2,405,420.6	339,243.1	640.3 (1)
B-303	2,405,338.5	339,143.0	637.7 (1)
B-304	2,405,438.6	339,060.2	617.8 (1)
B-305	2,405,520.6	339,160.2	623.3 (1)
B-306	2,405,413.7	339,313.2	649.1 (1)
B-307	2,405,276.1	339,193.3	638.5 (1)
B-308	2,405,288.6	339,069.3	623.4 (1)
B-309	2,405,333.7	338,998.8	614.6 (1)
B-310	2,405,445.0	338,987.8	607.5 (1)
B-311	2,405,592.5	339,099.7	615.4 (1)
B-312	2,405,582.0	339,230.1	634.3 (1)
B-313	2,405,379.3	338,917.2	602.7 (1)
B-314	2,405,288.2	338,916.5	596.3 (1)
B-315	2,405,297.7	338,820.6	596.9 (1)
B-316	2,405,513.4	338,882.2	602.8 (1)
B-317	2,405,571.7	338,888.1	601.6 (1)
B-318	2,405,520.5	339,436.1	645.3 (1)
B-319	2,405,462.4	339,429.9	644.0 (1)
B-320	2,405,516.0	340,491.8	611.3 (1)
B-321	2,405,830.2	338,752.7	616.7 (1)
B-322	2,405,754.8	338,922.5	596.2 (1)
B-323	2,405,815.6	338,927.9	599.2 (1)
B-324	2,405,191.0	339,323.7	648.5 (1)
B-325	2,405,252.9	339,328.3	648.0 (1)
B-326	2,405,176.4	339,454.7	653.8 (1)
B-327	2,405,233.1	339,452.1	654.9 (1)
B-328	2,405,699.7	339,176.8	609.6 (1)
B-329	2,405,802.7	339,189.6	608.5 (1)
B-330	2,405,916.0	339,200.6	607.7 (1)
B-331	2,406,407.0	339,872.7	642.7 (1)
B-332	2,406,874.3	339,907.3	642.6 (1)
B-333	2,406,421.4	339,667.3	644.3 (1)
B-334	2,406,888.5	339,700.6	625.3 (1)
B-335	2,405,475.6	340,767.3	774.7 (1)
B-336	2,405,516.3	340,492.2	771.3 (1)
B-337	2,405,528.1	340,239.5	771.6 (1)
B-338	2,406,234.5	340,738.3	697.3 (1)

TABLE 3, Continued

Mahantango Shale Formation Elevation

Proposed BBNPP Facility

Berwick, Pennsylvania

Boring I.D.	**X (Easting)**	**Y (Northing)**	**Top of Shale Formation El. (ft) (NAVD88)**
B-339	2,406,149.5	340,480.0	692.7 (1)
B-340	2,406,433.9	340,298.2	689.9 (1)
B-341	2,406,458.8	339,825.7	631.3 (1)
B-342	2,406,467.5	339,721.5	640.2 (1)
B-343	2,406,467.5	339,772.5	632.8 (1)
B-344	2,406,301.5	339,762.0	641.1 (1)
B-345	2,406,203.7	339,746.4	663.0 (1)
G-301	2,405,430.7	339,151.8	626.8 (1)
G-302	2,405,219.0	339,297.6	647.3 (1)
G-303	2,405,865.5	338,699.0	616.7 (1)
B-401	2,405,131.3	340,123.5	728.2 (2)
B-402	2,405,121.8	340,214.8	746.8 (2)
B-403	2,405,041.4	340,114.6	717.7 (2)
B-404	2,405,137.0	340,031.7	726.7 (2)
B-405	2,405,221.4	340,131.9	746.6 (2)
B-406	2,405,115.5	340,287.0	760.9 (2)
B-407	2,404,971.7	340,169.2	708.7 (2)
B-408	2,404,983.0	340,045.1	703.4 (2)
B-409	2,405,035.7	339,978.7	717.6 (2)
B-410	2,405,145.9	339,957.9	727.4 (2)
B-411	2,405,273.9	339,979.5	743.7 (2)
B-412	2,405,294.5	340,068.4	741.4 (2)
B-413	2,405,283.0	340,208.1	763.2 (2)
B-414	2,404,983.5	339,887.7	713.8 (2)
B-415	2,405,086.0	339,896.9	724.1 (2)
B-416	2,404,992.4	339,788.7	703.9 (2)
B-417	2,405,095.5	339,799.3	714.7 (2)
B-418	2,405,203.4	339,853.4	724.0 (2)
B-419	2,405,278.2	339,860.4	724.4 (2)
B-420	2,405,150.7	340,403.2	763.3 (2)
B-421	2,405,228.9	340,410.5	768.5 (2)
B-422	2,405,472.5	339,721.4	712.6 (2)
B-423	2,405,533.8	339,727.1	711.0 (2)

TABLE 3, Concluded
Mahantango Shale Formation Elevation
Proposed BBNPP Facility
Berwick, Pennsylvania

Boring I.D.	X (Easting)	Y (Northing)	Top of Shale Formation El. (ft) (NAVD88)
B-424	2,405,458.5	339,869.0	722.0 (2)
B-425	2,405,519.4	339,874.3	721.0 (2)
B-426	2,404,892.7	340,296.2	725.2 (2)
B-427	2,404,955.0	340,301.4	740.3 (2)
B-428	2,404,878.1	340,444.2	732.4 (2)
B-429	2,404,939.6	340,449.2	736.2 (2)
B-430	2,405,389.9	340,147.0	753.6 (2)
B-431	2,405,519.4	340,160.6	762.9 (2)
B-432	2,405,665.6	340,173.5	761.5 (2)
B-433	2,405,380.5	340,485.7	766.3 (2)
B-434	2,404,822.9	339,642.0	693.2 (2)
B-435	2,406,056.3	339,687.6	675.2 (2)
B-436	2,406,180.7	339,698.7	649.0 (2)
B-437	2,406,305.3	339,709.7	647.4 (2)
B-438	2,406,429.6	339,721.4	640.0 (2)
B-439	2,406,541.8	339,757.1	634.6 (2)
B-440	2,406,546.2	339,706.7	625.0 (2)
B-441	2,407,095.2	339,619.8	611.1 (2)
B-442	2,406,579.0	339,570.7	633.0 (2)
B-443	2,405,090.2	341,137.5	693.2 (2)
B-444	2,405,751.3	341,108.6	758.2 (2)
MW301A	2,405,396.7	339,097.6	626.0
MW301A1	2,406,939.7	339,410.2	630.0
MW303A	2,405,505.3	341,505.7	706.1
MW304A	2,408,455.4	340,228.2	643.6
MW305A2	2,407,096.8	341,888.6	631.6
MW306A	2,404,351.7	338,899.6	624.5
MW307A	2,407,086.0	337,632.5	651.6
MW308A	2,405,979.8	338,355.5	627.9
MW309A	2,408,989.2	338,707.9	652.4
MW310A	2,405,156.3	339,453.8	653.5
MW-401	2,405,097.7	340,753.3	764.5 (2)
MW-402	2,405,855.9	340,870.7	755.2 (2)
MW-403	2,405,542.4	340,579.3	741.9 (2)
MW-404	2,404,985.3	340,170.5	702.0 (2)
MW-405	2,404,646.4	339,970.5	653.8 (2)
MW-406	2,404,789.8	339,710.4	692.5 (2)
MW-407	2,405,144.3	339,784.9	719.7 (2)
MW-408	2,405,819.9	340,342.3	756.0 (2)
MW-409	2,405,905.4	339,760.7	713.4 (2)

(1) - Reference 10, Table 2.5.4-17.
(2) - From Weaver Boos review of Reference 3.

TABLE 4
Summary of Baseline Flow Model Inputs
Proposed BBNPP Facility
Berwick, Pennsylvania

Description of Input Values	Value	Units
General Data		
Title:	BBNPPBaseline0.vmf	
Number of layers:	7	
Overburden is represented in model Layer 1		
Shallow bedrock is represented in model Layers 1 through 4		
Deep bedrock is represented in model Layers 2 through 7		
Approximate area of model domain:	1.8	mi^2
Boundary Conditions		
Top Boundary Type:	Recharge	
Default basin recharge rate (all cells unless otherwise specified):	14.2	in/year
Transition areas from lowland to upland:	21.8	in/year
Upland areas:	29.0	in/year
Horizontal Boundaries:		
Basin is encompassed by no-flow boundaries in all layers:	Inactive	n/a
Constant head north of Power Block in Layers 2 through 7 (bedrock inflow from high upland):	700 to 720	ft msl
Constant head south and east of ESWEMS Pond in Layers 4 through 7 (deep bedrock outflow):	621 to 695	ft msl
General Head to far south (outflow to Susquehanna River):	480	ft msl
Rivers (Walker Run and tributary) within model domain:		
Stage based on USGS map and field measurements:	variable	ft msl
Bottom depth below stage:	2	ft
Bottom thickness:	1	ft
Bottom conductivity:	3.53E-04	cm/s
Bottom Boundary at base of Layer 7 - No Flow:	Inactive	n/a
Formation Properties		
Overburden Aquifer Soil (Layer 1 only)		
Thickness (south of Power Block and beneath ESWEMS Pond, variable elsewhere):	~60	ft
Horizontal Conductivity:	5.9E-02	cm/s
Vertical Conductivity:	5.9E-03	cm/s
Specific Storage:	2.00E-06	1/ft
Specific Yield (0.322 used in manual calculations):	0.322	N/A
Effective Porosity:	0.322	
Total Porosity:	0.322	
Shallow Bedrock (Layers 1 through 4)		
Thickness (beneath Power Block and other uplands, variable elsewhere):	~200	ft
Horizontal Conductivity:	5.43E-04	cm/s
Vertical Conductivity:	5.43E-05	cm/s
Specific Storage:	2.00E-06	1/ft
Specific Yield:	0.01	
Effective Porosity:	0.1	
Total Porosity:	0.1	
Deep Bedrock (Layers 2 through 7)		
Thickness (beneath Power Block and elsewhere):	~600	ft
Horizontal Conductivity:	1.64E-04	cm/s
Vertical Conductivity:	1.64E-05	cm/s
Specific Storage:	2.00E-06	
Specific Yield:	0.01	
Effective Porosity:	0.1	
Total Porosity:	0.1	

TABLE 5

Steady State Groundwater Flow Model Mass Budgets

Proposed BBNPP Facility

Berwick, Pennsylvania

MODEL SCENARIO	Flows to Aquifer (ft^3/day)		Flows to Aquifer (ft^3/s)		Flows to Aquifer (gpm)	
BASELINE FLOW MODEL	**IN**	**OUT**	**IN**	**OUT**	**IN**	**OUT**
Constant Head Boundary	181,950	129,774	2.1	1.5	950	670
River Leakage	124,455	169,047	1.4	2.0	650	880
Head Dependent Boundary	0.0	217,971	0.0	2.5	0.0	1,130
Recharge	218,152	0.0	2.5	0.0	1,130	0.0
Totals	524,557	516,792	6.1	6.0	2,720	2,680

DEWATERING WITHOUT FLOW BARRIER AT ESWEMS	IN	OUT	IN	OUT	IN	OUT
Constant Head Boundary	232,523	75,290	2.7	0.87	1,210	390
Power Block Excavation Drains	0.0	9,749	0.0	0.11	0.0	50
Cooling Towers Excavation Drains	0.0	13,880	0.0	0.16	0.0	70
ESWEMS Excavation Drains	0.0	48,165	0.0	0.56	0.0	250
ESWEMS Excavation Dewatering Wells (28 wells)	0.0	129,140	0.0	1.5	0.0	670
Subtotal Dewatering Outflows						**1,040**
River Leakage	164,679	150,389	1.9	1.7	860	780
Head Dependent Boundary	0.0	217,503	0.0	2.5	0.0	1,130
Recharge	218,140	0.0	2.5	0.0	1,130	0.0
Totals(1)	615,341	644,117	7.1	7.5	3,200	3,350

DEWATERING WITH FLOW BARRIER AT ESWEMS	IN	OUT	IN	OUT	IN	OUT
Constant Head Boundary	199,158	121,964	2.3	1.4	1,030	630
Power Block Excavation Drains	0.0	9,749	0.0	0.11	0.0	50
Cooling Towers Excavation Drains	0.0	13,880	0.0	0.16	0.0	70
ESWEMS Excavation Drains	0.0	30,310	0.0	0.35	0.0	160
ESWEMS Excavation Dewatering Wells (14 wells)	0.0	13,462	0.0	0.16	0.0	70
Subtotal Dewatering Outflows						**350**
River Leakage	137,510	154,932	1.6	1.8	710	800
Head Dependent Boundary	0.0	217,753	0.0	2.5	0.0	1,130
Recharge	218,152	0.0	2.5	0.0	1,130	0.0
Totals(1)	554,821	562,051	6.4	6.5	2,880	2,920

(1) Totals vary slightly from overall budget because drain and dewatering outflows are reported from the zone-specific mass budgets used to separately estimate flows from the power block, cooling towers, and ESWEMS dewatering systems.

Notes:

Constant Head Boundary - Represents inflows to the model domain originating in the bedrock ridge to the north and deep outflows to southeast.

Drains - Represent trench drains in exposed bedrock surface, used in all excavations, and well point lines in overburden at the ESWEMS pond.

Dewatering wells are used only at the ESWEMS pond excavation to drain high-conductivity overburden aquifer.

River Leakage - Represents exchanges of water between Walker Run and the Aquifer.

Head Dependent Boundary - Represents outflow from the southern edge of the model domain to the Susquehanna River.

Recharge - Represents recharge to groundwater at 14.2 to 29 in. per year over the 1.8 mi^2 model domain.