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CALCULATIONS

Objectives:

In order to reduce the rate of groundwater flowing into the excavations during construction of the Levy County Nuclear Power Plant (LNP) Units 1 and 2, two different kinds of flow barriers will be constructed. A diaphragm wall will be installed around the perimeter of each nuclear island (NI) (Figures 1 and 2, located in Attachment B). In addition, the limestone beneath each nuclear island (NI) will be drilled and pressure-grouted prior to excavation. The objectives of this document are to:

1) Determine the rate of groundwater seepage that will enter the excavation following the installation of the two flow barriers. In addition, prepare a preliminary design for a dewatering system.

2) Evaluate potential for piping failure, boiling problems, and general rock failure due to heave/uplift that could occur as a result of upward seepage through the grouted limestone bedrock.

3) Evaluate the amount of drawdown that will occur around the NIs after construction as a result of pumping by the production wells, located approximately 3,700 feet (1,139 m) northeast of LNP Unit 2. Assess whether the calculated drawdown and increased hydraulic gradients can potentially cause erosion of sediments from voids or decrease strength of the rock mass beneath the NIs.

Background:

Groundwater levels in the surficial aquifer in the LNP area have been measured between El. 41.3 (MW-11S) and 42.54 feet (MW-7S) above mean sea level (ft. NAVD) in March 2007 (Figure 3). These levels are very close to the ground surface (approximately El. 43 ft. NAVD). Groundwater elevations in the Upper Floridan aquifer (monitored via deep wells) were slightly lower, but were also very close to the ground surface in March 2007 (El. 40.73 (MW-12D) to 42.21 (MW-8D) ft. NAVD (Figure 4). During the excavation for construction of the LNP Nuclear Islands, the excavations will proceed down through the entire thickness of the undifferentiated Quaternary silty sand and sand deposits. The silty sand overburden deposit varies in thickness, but is approximately 67 feet thick in the vicinity of LNP Units 1 and 2 (Ref. 1).

Once both low-permeability barriers are constructed, dewatering wells will be drilled and installed within the perimeter of the diaphragm wall (Figure 5) and will be screened through the entire thickness of the surficial sand unit and 30 feet down into the grouted rock section. The sand and the upper portion of the grouted rock will be dewatered gradually as the excavation proceeds downward. Six proposed wells located along the inside perimeter of the diaphragm wall will be preserved as the surficial sand is excavated from the NI area (Figure 5).

Sump pumps will also be placed at various locations within the excavation as deemed necessary during excavation to handle (1) upward groundwater seepage, (2) flow through "windows" in the diaphragm wall and (3) surface water and rainfall that enters the excavation.

Once the excavation has reached the upper surface of the bedrock (El. -24 ft. NAVD), the weathered and/or fractured bedrock will be removed and leaking fractures, if discovered, will be drilled and pressure-grouted during a second round of grouting. If the diaphragm wall is found to be cracked or leaking anywhere, additional drilling and grouting outside of the diaphragm wall will be performed to help minimize leakage into the excavation. Thus, extra care will be given to sealing off each NI excavation pit and reduce groundwater seepage rate before upward construction of the NI begins.

A 35-foot thick layer of roller-compacted concrete (RCC) will be placed over the entire surface of the floor of the NI excavation after the excavation bottom has been prepared with dental concrete and shallow grouting, if necessary. As the RCC is installed, the dewatering wells located along the inside perimeter of the diaphragm wall will be preserved or reconstructed, and dewatering will continue during the entire period of NI construction.

The calculations in Part One were performed to determine the rate of pumping that will be necessary to dewater the surficial sand and the upper portion of the grouted bedrock during excavation and construction of each NI. The dewatering will be maintained until the NI is constructed to final grade (i.e., El. +51 ft. NAVD). At this time, pumping can be discontinued and the wells will be converted to monitoring wells, if deemed useful. Otherwise, they will be pressure-grouted and eliminated in accordance with Florida State Regulations.

The numerical modeling performed in Part One involves only pumping wells and sumps that are positioned inside the hydraulic barriers. The dewatering wells and sumps will be used to dewater and keep the excavation dry. However, these wells and sumps will not significantly affect hydraulic heads outside of the flow barriers and therefore will not cause any significant reduction in external hydraulic pressures. The calculations included in Part Two were performed to determine if the resulting hydraulic gradients and groundwater seepage could cause piping, heave, or uplift problems/failure in the grouted limestone bedrock.

Four large-diameter, groundwater production wells are to be installed northeast of Unit 2. Each well will be screened in the Avon Park Formation (i.e., the Upper Floridan aquifer) and each will be capable of pumping 1,000 gallons per minute (gpm). In Part Three calculations, a second groundwater model was constructed and employed to simulate steady-state drawdown resulting from the groundwater production wells. The groundwater modeling was performed to evaluate whether the production wells might cause significant reductions in the hydraulic heads near the NIs, or cause potential for sediment removal from voids and weakening of the rock structure near the NIs.

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Design Inputs:

The design inputs for geologic conditions at the site are:

- The unconsolidated, undifferentiated Quaternary overburden deposits are approximately 67 feet thick at both LNP 1 and LNP 2, and consist primarily of fine sand and silty sand (Ref. 1).
- The uppermost bedrock, extending from 67 to approximately 400 ft. bgs, consists of limestone belonging to the Avon Park Limestone Formation (middle Eocene). This limestone is moderately to highly permeable and constitutes the Upper Floridan aquifer (UFA) in the area (Ref. 2).
- According to FSAR Section 2.4.12 (Ref. 2), the lower portion of the Avon Park Formation is composed of lower-permeability limestone that has quartz and gypsum that fills fractures and vugs. This zone represents the Middle Confining Unit (MCU) of the Floridan aquifer. Deep borings drilled to characterize the two LNP sites went as deep as 500 ft. bgs, but "did not encounter the MCU or the underlying Oldsmar or Cedar Keys formations of the Lower Floridan aquifer (LFA). However, traces of evaporate deposits and quartz infilled porosity typical of the MCU were observed sporadically" below 400 ft. bgs, indicating that these borings may have approached the less permeable MCU (Ref. 2). For modeling purposes, the UFA is considered to be 426 feet thick and extends from 67 ft. to 493 ft. bgs.
- According to FSAR Section 2.4.12 (Ref. 2), the MCU (lower portion of the Avon Park Formation) is relatively impermeable, forms the MCU of the Floridan aquifer, and is approximately 400 feet thick in the vicinity of the LNP site. The MODFLOW numerical model was constructed to include the surficial aquifer (Layer 1) and the entire UFA (Layers 2-6 of the model, see Table 1). The base of the model was conservatively defined to be El. -450 ft. NAVD (493 ft. bgs), which represents the base of the UFA and the top of the MCU. The base of the model (i.e., the top of the MCU) is defined as a no-flow boundary.

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The following design inputs regarding the hydraulic characteristics of the geologic materials include the following:

- According to Ref. 3, the hydraulic conductivity (K) of the surficial sand aquifer (determined from slug tests) ranges from 0.9 to 28.6 ft./day, and averages approximately 9.2 ft./day. The average K value (9.2 ft./day) was used in the groundwater flow model for Layer 1. Based on one multi-well pumping test (Ref. 4), and a saturated thickness of the surficial aquifer of 67 feet (Ref. 1), the hydraulic conductivity was found to be somewhat higher (21 to 35 ft./day). In model Run No. 6 (Table 2), a higher value of K (35 ft./day) for the surficial aquifer was used to evaluate if this condition would significantly affect flow into the excavation.
- According to Ref. 3, the hydraulic conductivity (K) of the Upper Floridan aquifer (determined from slug tests) ranges from 2.4 to 54.4 ft./day, and averages approximately 13.9 ft./day. The average K value (13.9 ft./day) was used in the groundwater flow model to represent the uppermost and lowermost sections of the UFA (model layers 2 and 6).
- According to FSAR Section 2.4.12 (Ref. 2), "the most productive interval of the Upper Floridan aquifer appears to be at depths of approximately 30 to 60 meters (100 to 300 feet) bgs." In the groundwater flow model, Layers 3-5 (representing 97 to 293 ft. bgs) were defined as having a higher K value (twice the average value, or 27.8 ft./day), thereby simulating the "more productive zone" in the UFA. In model Run No. 5, the hydraulic conductivity of the limestone in Layers 3 through 5 was increased 179 ft./day to determine how sensitive the model is to changes in this parameter, and how the predicted seepage rate will change because of the uncertainty in this model input parameter.

The following design inputs have been made regarding the hydraulic characteristics of the hydraulic barriers:

- The average hydraulic conductivity of the diaphragm wall is expected to be approximately 10^{-6} cm/s (0.002835 ft./day) (Ref. 5).
- The diaphragm wall will be approximately 3.5 ft. thick, 97 ft. in height (El. +43 to -54 ft. NAVD, Fig. 2), and have a total perimeter length of 817 ft.
- The entire surface area (in plan) that lies within the diaphragm wall is 38,121 ft^2 (Fig. 1).
- The grouted limestone zone beneath the NI is considered to be 75 feet thick and it extends laterally beneath the entire NI and 10 feet outward beyond the diaphragm wall (Fig. 2).
- The diaphragm wall extends 30 feet down into grouted limestone (i.e., it is keyed 30 feet into the grouted limestone zone) (Fig. 2).

In model Run No. 7, the model was used to simulate a scenario where panels in the diaphragm wall separate at three different locations, thus allowing groundwater to flow through the vertical "windows" into the excavation. For this scenario, each panel separation was defined to be 3.5 feet wide and ran vertically through model Layers 1 and 2. Locations of these three "windows" are shown on Figure 5. At each separation, Layer 1 silty sand was used to fill the gaps between wall panels and has a K value of 9.2 ft./day (same as Layer 1).

Although two AP1000 plants are to be constructed at this site, it is assumed that the excavation and dewatering operations would be performed for one plant at a time. Thus, the calculations contained herein are for dewatering of one location only. Model inputs and output results are documented and have been archived for each model run, and are summarized herein.

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Methodology:

These calculations consist of three parts.

Part One Calculations and Numerical Modeling, Calculation of Dewatering Rates:

Part One includes calculations and numerical modeling to determine the rate of pumping that would need to be employed to remove groundwater from the surficial aquifer (i.e., silty sand) directly below the Nuclear Island (NI), if two different kinds of flow barriers are installed prior to pumping and excavation. These two barriers include a diaphragm wall (0-97 feet below ground surface [bgs]) around the perimeter of the NI and a 75-foot thick zone of pressure-grouted limestone bedrock beneath the NI and 10 feet around the outer edge of the diaphragm wall (Figures 1 and 2).

If the barriers have a very low permeability, then only groundwater contained within the barriers needs to be removed. This calculation was performed in order to estimate the minimum pumping rate that would need to be employed for dewatering purposes. In this calculation, the hydraulic barriers are considered to have very low permeability (i.e., no significant quantity of groundwater seeps into the excavation). The only water that is removed from the excavation is groundwater contained in the pore spaces within the surficial aquifer, and precipitation that falls directly into the excavated area. The area within the diaphragm wall to be excavated is 38,121 ft^2 (Figure 1).

Based on a pumping test performed in the surficial aquifer, CH2M Hill calculated the specific yield of the surficial sands ranges from 0.012 to 0.17 (Table 2.4.12-207 in Ref. 4). These specific yield values are relatively low. When considering saturated soils, the total porosity may be calculated, using data from Ref. 6, as follows:

Porosity Calculations:
$e*S = w_{av} G_{s\,ave}$
$n=e/(1+e)$

e: Void ratio
n: Porosity
S= Percent saturation = 100 %
w_{ave}= Average Moisture Content (Soil layers; S1,S2,S3)
w_{ave}= 24.9 % (Ref. 6)
Gs_{ave}= Average Specific Gravity (Soil layers; S1,S2,S3)
Gs_{ave}= 2.77 (Ref. 6)

e x 1= 0.249 x 2.77 e=0.69 n=0.41

Based on the data from Ref. 6, a more conservative value of 0.41 was calculated for total porosity, as shown above. Although drainable porosity (i.e., specific yield) of an aquifer is always less than total porosity, the value of 0.41 was used for dewatering calculations, because it is a more conservative (i.e., it will yield higher values of pumping rates to keep the excavation dry).

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Using a porosity (n) value of 0.41 and a water table elevation of 42 ft. NAVD (Figure 3), the following calculations were made to estimate the total amount of groundwater contained within the surficial aquifer beneath one of the NIs:

Area of Saturated Sand (A) =	38,121 ft^2
Thickness (T) = (42 ft. + 24 ft.) =	66 ft
Volume of Saturated Sand (V_s) =	2,516,000 ft^3
Volume of water (V_w) = V_s* 0.41 =	1,032,000 ft^3
Volume of water in Gallons (V_g) = V_w * 7.48 =	7,720,000 gallons

If this amount of water was pumped from the excavation over a four-month period (122 days), then the rate of pumping of groundwater (P_{gw}) will be:

7,720,000 gallons / (122 days * (24 hrs/day)*(60 min/hr))
= 44 gpm (4-month period)

If this amount of water was pumped from the excavation over a two-month period (61 days) or a 6-month period (183 days), then the rate of pumping of groundwater (P_{gw}) will be:

7,720,000 gallons / (61 days * (24 hrs/day)*(60 min/hr))
= 88 gpm (2-month period)

7,720,000 gallons / (183 days * (24 hrs/day)*(60 min/hr))
= 29 gpm (6-month period)

During excavation, surface water runoff will be diverted away from the excavation. However, the pumps and sumps will need to contend with rainfall that falls directly into the excavation. There are three meteorological stations that are located within 35 miles of the LNP site (Fig. 6). Long-term rainfall data for these three stations (Ref. 7) are summarized in Table 3. Based on data from these three stations, the average annual rate of rainfall at the LNP site is approximately 55 inches. The highest average monthly rainfall rates occur in June through September. The average rainfall over these four months is 30.24 inches (2.52 ft.). However, to be more conservative, the long-term average rainfall measured at the Usher Tower meteorological station is 60.24 inches (5.02 ft.) annually and 32.56 inches (2.71 ft.) for the 4-month period of June through September (Table 3). Based on the Usher Tower-average rainfall rates during the four wettest months, the total amount of rainwater that will enter the excavation (V_{ppt}) will be:

Area of Excavation (A) =	38,121 ft^2
Height of Precipitation (h) =	2.71 ft
Volume of Precipitation =	103,000 ft^3
Volume of Precipitation in Gallons = V_{ppt} * 7.48 =	770,000 gallons

If this amount of water was pumped from the excavation over a four-month period (122 days), then the rate of pumping for rainwater (P_{ppt}) will be:

770,000 gallons / (122 days * (24 hrs/day)*(60 min/hr))
= 4.4 gpm

Thus, without groundwater leaking into the excavation from beyond the flow barriers, approximately 48 gpm ($P_{gw} + P_{ppt}$) would need to be pumped from the excavation during a four-month period.

The same calculations were performed, assuming however that the excavation will be dewatered over a 2-month (61 days) and a 6-month (183 days) period assuming the same rainfall rates as the wettest months. For 2 months, the average pumping rate will need to be approximately 92± gpm. For a 6-month period, the average pumping rate would only need to be 33± gpm.

This calculation describes an unlikely scenario where the flow barriers are completely impermeable and no water leaks into the excavation. The pumping rates of 33 to 92 gpm are the minimum pumping rates that would need to be performed in order to remove pore water and rainfall from the excavation over a 2- to 6-month period. This calculation presents a lower bound to the magnitude of pumping that might actually be required. These calculations do not consider scenarios involving hurricane or extremely large rainstorm events during the excavation and construction phases of the NI. The 2-, 4-, and 6-month pumping rates necessary to dewater the excavation are presented in Figure 7.

It is possible that some groundwater will seep through the hydraulic barriers into the excavation and will be removed by the dewatering system. In order to calculate how much groundwater will seep into and be pumped from the excavation, a three-dimensional transient numerical groundwater flow model was developed. This model includes six different layers to represent the diaphragm wall, grouted rock layer, pumping wells, sumps, and three different geologic materials. Design hydraulic properties reported by CH2M HILL (Refs. 2-4) were used to construct and operate the model.

The three-dimensional, finite difference groundwater flow model was constructed to perform the calculations using spatially-variable boundary conditions, multiple geologic layers, spatially-variable hydraulic conductivity properties, and non-uniform hydraulic barrier conditions. The modeling was performed using the U.S. Geological Survey's MODFLOW 2000 software, which is contained in a graphical user interface (GUI) software package called Visual MODFOW Premium Version 4.2 (Ref. 8). The Commercial Grade Dedication and the Validation and Verification for this software have been completed (Ref. 9 and 10).

A three-dimensional groundwater flow model was constructed to calculate a steady-state pumping rate that will be necessary to keep the excavation dry when it achieves its greatest depth (El. -24 ft. NAVD). The finite-difference grid is square in shape and represents an area 3,850 feet on a side (Figure 8). The total model area has been discretized (i.e., divided) into 400 rows and 400 columns, which creates 960,000 cells in the model domain.

The NI is positioned in the center of the model grid (Figures 5 and 8). The model area is discretized finer in the center of the model. The inner portion of the model area (400 ft. x 400 ft.) is divided into 3.5 ft. x 3.5 ft. grid cells. This area totally encompasses the NI. The outermost portions of the model area include cells that are 14 ft. x 14 ft. and 7 ft. x 14 ft. (Figure 8).

The model domain extends vertically from El. +43 ft. NAVD down to El. -450 ft. NAVD; the total thickness of the model domain is 493 feet (Table 1). The domain is divided into six layers. Layer 1 represents the surficial aquifer. Layers 2-6 represent the upper permeable portion of the Avon Park Formation (i.e., the UFA). The base of the model is considered to be a no-flow boundary and represents the top of the MCU.

Constant-head boundaries were assigned to the outer edges of the model, based on the interpolated hydraulic heads shown on Figures 3 and 4 for Layer 1 and Layers 2-6, respectively. The potentiometric surfaces for March 2007 are slightly above average, so the estimated boundary conditions should be conservative for calculating groundwater flow into the excavation. The constant-head values assigned to each corner of the model for each layer are listed in Table 4. Visual MODFLOW software performed a linear interpolation and assigned constant heads to each cell along all four sides of the model area, based on the linear interpolations and the hydraulic head values listed in Table 4 as the basis for interpolation.

The hydraulic conductivities assigned to each geologic material and the flow barriers for each model run are shown on Table 2. Model Run No. 1 uses the design hydraulic conductivities for each model component. Therefore, Model Run No. 1 is considered to be the baseline case for the numerical groundwater model. Figure 9 represents a cross section through the model that parallels the long axis of the NI. In Figure 9, the Layer 1 white cells represent the silty sand (surficial aquifer). The two green vertical lines of cells in Layers 1 and 2 represent the diaphragm wall. The blue zone in Layers 2 and 3 beneath the NI represent the 75-foot thick zone of grouted limestone. The deep red zones of Model Layers 2 and 6 represent the moderately permeable Avon Park Formation, and the purple zone in Layers 3-5 represent a more productive zone of the UFA.

The MODFLOW model was run as a transient model. Six dewatering wells were placed along the inside perimeter of the diaphragm wall, as shown in Figure 5. The wells extend downward all the way through the surficial sands (Layer 1) and 30 feet into the upper grouted limestone (Layer 2). MODFLOW drain cells were used to represent the wells. The drain cells at the six locations in Layer 1 were assigned a constant elevation of -23.8 ft. NAVD, which is slightly above the deepest level of excavation. A drain cell was also assigned to all six locations, but placed in Layer 2. These six drain cells were assigned constant drain elevations of -53 ft. NAVD. Drain cells were used to represent dewatering wells because constant drain elevations can be assigned to these cells and the model calculates drainage rates to these cells to maintain the assigned elevations. In other words, the model calculates the amount of "pumping" necessary to maintain the assigned groundwater elevation.

In addition to the 12 drain cells described above (two cells for each dewatering well), drain cells were also placed over the entire floor of the excavation in Layer 1. Changing elevations were assigned to these cells, which represent the water levels that are maintained by sumps as the floor of the excavation gets deeper. The transient model was set up so that the first four stress periods are each 20 days in duration and the excavation floor drops 15 feet between each stress period. For the first 20 days of each model run (stress period 1), the drain cells were assigned an elevation of +28 ft. NAVD. For day 21-40 of a simulation (stress period 2), the drain elevations were dropped to +13 ft. NAVD. For the next 20 days (day 41-60, stress period 3), the drain elevations were dropped further (-2 ft. NAVD). From day 61 to 80 (stress period 4), the drain elevations were set to -17 ft. NAVD. Finally, during the last stress period (stress period 5) of the transient run (80-730 days), the Layer 1 drain elevations were set equal to -23.8 ft. NAVD. Thus, each model run was set up to simulate the excavation occurring as five steps, getting progressively deeper over an 81-day period. Because

the maximum drawdown will occur when the excavation is deepest (-24 ft. NAVD) and the RCC has not been emplaced, the maximum seepage rates, maximum pumping rates, and maximum hydraulic gradients will also occur when the excavation has reached its deepest point and placement of the RCC has not occurred (stress period 5). The dewatering rates listed in Table 2 are the highest rates that are anticipated for each scenario that was simulated.

The model calculates a distribution of hydraulic heads for the entire model domain, such that water mass is conserved (i.e., net water flow into and out of the model domain approaches zero) and the change in head for any individual model cell for successive mathematic iterations approaches zero (closure requirement set to maximum change of 0.01 foot).

For Model Run No. 1, the model calculated that 67 gpm will leak through the wall and through the floor of the excavation into the drain cells. This includes flow into the six wells that penetrate the grouted limestone of Layer 2. The potentiometric surface predicted for Layer 1 at 81 days shows the pit floor is dry (Figures 10 and 11). The potentiometric surface predicted for Layer 2 is shown in Figure 12. The potentiometric surface in Layer 2 is depressed in the grouted limestone around the six wells (Fig. 12). The potentiometric surface in Layer 2 is relatively flat across the center of the NI (Fig. 12). The hydraulic potential in Layer 2 near the center of the Nuclear Island (i.e., at observation well Obs-3) is predicted to be El. -9.27 ft. NAVD after 730 days of dewatering (Table 2).

In the model, six hypothetical observation wells were placed in the model. Three of these (Obs-1, Obs-2, and Obs-5) are screened in Layer 1 and three (Obs-3, Obs-4, and Obs-6) are screened in Layer 2 (Figure 12). Observation wells Obs-1 and Obs-3 are located close to the center of the excavation. Observation wells Obs-2 and Obs-4 are located in the lower left corner of the excavation. Observation wells Obs-5 and Obs-6 are located outside and to the west of the excavation. These hypothetical wells were used as reference points to plot changes in groundwater levels during each transient simulation.

Figure 11 shows a cross section of the excavation area and the phreatic surface that result after 81 days of dewatering. Overall, a pumping rate of approximately 67 gpm is needed to keep the excavation dry and to keep the grouted limestone zone beneath the excavation partially depressurized. Figure 13 shows hydraulic heads in each observation well over simulated time. The groundwater elevations in the two Layer 1 wells (Obs-1 and Obs-2) dropped the furthest (down to El. -23.8 ft. NAVD). The two observation wells (Obs-3 and Obs-4) screened in the grouted limestone (Layer 2) were depressed significantly, but not as much as the predicted drop in the Layer 1 wells. The decrease in groundwater elevations in the two observation wells located outside of the NI area (Obs-5 and Obs-6) were only depressed by about 2.5 feet.

Based on this modeling, a continuous dewatering rate of 67 gpm will be necessary to dewater the excavation and keep it dewatered during construction.

In subsequent model runs (Run No. 2-7), the hydraulic properties of various portions of the groundwater model were adjusted upward (i.e., hydraulic conductivities were increased) to determine how the seepage rates would vary if the hydraulic conductivities of the materials have been underestimated in Run No. 1. This sensitivity analysis was performed in order to estimate worst case scenarios of pumping rates that might need to be employed during the construction of the LNP Units 1 and 2. The changes to the Base Run model (i.e., Run No. 1) are specified in Table 2. Model Runs 2 through 7 were performed to test the sensitivity of the model to changes in hydraulic conductivity values assigned to the diaphragm wall (Runs 2, 3, and 7), the

grouted limestone zone (Run 4), the native limestone of Layers 3-5 (Run 5), and the surficial sand aquifer (Run 6).

The results of these model runs are listed in Table 2. Increasing the K value of the diaphragm wall by a factor of ten (Run No. 2) increased the seepage rate into the excavation to 147 gpm. Decreasing the K value of the diaphragm wall by a factor of ten (Run No. 3) decreased the dewatering rate to 56 gpm (Run 3). Increasing the K of the grouted limestone by a factor of 10 (Run No. 4) caused the overall dewatering rates to increase to 452 gpm. Increasing the K value for the native limestone of Layers 3-5 (Run No. 5) and the surficial aquifer (Run No. 6) had virtually no effect on inflow seepage rates (Table 2).

In Run No. 7, three 3.5-ft. wide vertical "windows" were simulated in the diaphragm wall, which represent offsets between adjacent concrete panels. These locations are shown on Figure 14. These "windows" can result if a panel shifts either during or after construction. The overall leakage into the excavation during Run No. 7 reached 94 gpm. As shown in Figure 14, drawdown occurred in the surficial aquifer (Model Layer 1) outside of the diaphragm wall in the vicinity of the three windows.

The dewatering rates determined for model Runs 3, 5, and 6 were nearly the same as the base run of 67 gpm. The dewatering rates calculated for Run Nos. 2, 4, and 7 (94 - 452 gpm) were greater. The highest seepage rate (452 gpm) was calculated for Run No. 4. In this model run, the K value for the grouted limestone zone was set to a relatively high number (2.835 ft./day).

Part Two Calculations, Stability of the Dewatered Base:

The stability of the dewatered base is checked for local failure by piping or boiling and the general failure of the rock below excavation level by heave or uplift. The local reduction of effective stress due to upward seepage is checked at the exit point where the minimum seepage path length exists.

Consider a 15 feet wide block (1/2 of diaphragm wall penetration depth (D) of 30 feet) of the heave zone adjoining to diaphragm wall as shown in Figure 15. The hydraulic heads at the bottom and the top of the block are 10 ft. NAVD and -24 ft. NAVD, respectively (Figure 15). The average value of head loss in the block is:

$$10 - (-24) = 34 \text{ feet}$$

and the average hydraulic gradient (I_{av}) is:

$$I_{av} = 34/30 = 1.13$$

The factor of safety (FS) against heaving in the rock is modified from the expression given in Ref. 11:

$$FS = F/U$$

where: F = submerged unit weight of the grouted limestone block + shear force acting on the block
U = uplift force caused by seepage on the same volume of rock.

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For per unit length of the excavation:

F = $((\gamma_{sat} - \gamma_w) \times D \times D/2) + (C \times D)$ (neglecting shear force due to frictional component of the rock - conservative)
U = Volume of block x i_{av} x γ_w,

Where: γ_{sat} = saturated unit weight of rock (pcf),
γ_w, = unit weight of water (pcf),
C = cohesion component of the rock (psi), and
D = penetration depth of diaphragm wall (ft.)

The bedrock properties of the existing conditions are given in Reference 12 (Pages 8 and 9 of 25). For the top bedrock layer of LNP 2; the unit weight (γ_{sat}) = 134 pcf, cohesion (C) = 24 psi, and friction angle (Ø) = 24°. For the top bedrock layer of the LNP 1; the unit weight (γ_{sat}) = 138 pcf, cohesion (C) = 37psi, and friction angle (Ø) = 24° (Ref. 12). The calculation shown below was performed for LNP 2, since the rock density and cohesion values are less for this site and the calculation is therefore more conservative.

Neglecting improvement in bedrock properties due to grouting, cohesion and frictional force along the diaphragm wall and prism interface and frictional component of the shear strength along the rock and prism interface, the following conservative values are considered for checking the stability of the bedrock due to dewatering:

LNP 2

Unit Weight (γ_{sat}) = 134 pcf
Cohesion (C) = 24 psi = 24 x 144 psf = 3456 psf
F = (134-62.4) x30x15 + 3456x30 = 135,900 lbs/ft.
U = 30x15x1.13x62.4 = 31730 lb/ft.
FS = 135,900/31730 = 4.3 > 1.5

Based on this calculation, the stability of the excavated bedrock is safe against heaving or uplift (i.e., FS > 1.5, which is the generally accepted safety factor) at both LNP 1 and 2.

Part Three Calculations, Drawdown and Instability Due to Post-Construction Production Wells:

Four production wells are to be installed at the LNP site, spaced 1,000 ft. apart as shown on Figure 16. The nearest well (i.e., the southernmost well) is located approximately 4,225 ft. northeast of LNP 1, and approximately 3,700 ft. northeast of LNP 2. The Visual MODFLOW software package was used to estimate the impact to the groundwater regime beneath LNP 2 due to operation of these production wells. For 1.56 MGD production rate (estimated average rate), the resulting drawdown at LNP 2 is less than 0.3 m (less than 1 ft.). For 3.0 MGD production rate (estimated the seven-day peak rate), the resulting drawdown at LNP 2 is less than 0.6 m (2 ft.).

As discussed in FSAR Subsection 2.4.12.1.2 (Ref. 2), the water table ranges in depth at LNP 1 and LNP 2 areas from less than 0.3 m (1 ft.) below ground surface during rainy periods to approximately 1.5 m (5 ft.) bgs during drier periods. Since the operation of the production wells results in drawdown at LNP 2 that is

less than or in the range of this seasonal fluctuation of over 1.2 m (4 ft.), the groundwater regime is judged to not be adversely impacted by the wells, placed at least 1,128 m (3,700 ft.) away from the LNP units, and the potential for increased solution activity or sediment entrainment from voids due to changes in the subsurface flow velocity at LNP 1 and LNP 2 is negligible.

Conclusion:

Model inputs and output results are documented and have been archived for each model run, and are summarized herein.

Based on the Part One calculations, the excavation can be dewatered with relative ease; approximately 67 gpm needs to be pumped to keep the excavation dry during excavation and construction of the Nuclear Island. This could be accomplished with a combination of pumping wells around the perimeter of the excavation and sumps located at various points within the excavation. The wells can be maintained and pumped as the Nuclear Island is being built from the bottom upward. The six wells located inside the flow barriers can be four inches in diameter and have 100 gpm submersible pumps installed in each of the six wells. This will allow extra pumping capacity if unanticipated hydrogeological conditions are encountered, or separation in the diaphragm wall occurs. All of the pumped water will be discharged at least 600 feet away from the excavation in order to avoid mounding and short-circuiting of flow near the excavation.

If " holes," "windows," or "leaks" in the diaphragm wall are discovered during excavation, then additional drilling and grouting can be selectively performed in specific target zones outside the diaphragm wall to reduce seepage rates during excavation.

When the excavation reaches its greatest depth (i.e., competent bedrock surface; approximately El. -24 ft NAVD), then the bedrock surface should be thoroughly cleaned and inspected. The dewatering wells should be shut down for several days and the surface of the excavation should be inspected again. If distinct or noticeable locations are observed where groundwater is seeping upward through the floor of the excavation, then a quick program of selective drilling and pressure grouting should be performed to minimize the leakage.

Because of uncertainties in the hydraulic conductivities of the diaphragm wall, grouted limestone slab, the natural limestone formation, and the overburden sand deposit, the seepage into the excavations may deviate from the calculated value of 67 gpm. A sensitivity analysis using the model showed that seepage rates may vary between 67 and 457 gpm.

The Part Two calculations showed that uplift, heave, or piping will not be a problem in regard to the grouted limestone slab.

The Part Three modeling related to the four production wells showed that even when the wells are producing at 3.0 MGD, the drawdown in the Unit 2 area will be less than 2.0 feet, which is in the same general range as the fluctuations that occur naturally in the UFA during each year.

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ATTACHMENT A
Tables

TABLE 1 GEOLOGIC UNITS, FLOW BARRIERS, DEWATERING WELLS, AND DRAINS (SUMPS) INCORPORATED INTO GROUNDWATER MODEL

Model Layer	Elevation (ft amsl)		Thickness (feet)	Geologic Units Included in Layer	Flow Barriers Included in Layer	Dewatering Wells and/or Drains Included in Layer
	Top	Bottom				
1	+43	-24	67	Silty sand surficial deposits	diaphragm wall	six dewatering wells, surface drains, and sumps
2	-24	-54	30	Limestone	diaphragm wall; grouted limestone beneath the NI	six dewatering wells (same as above) screened in the grouted limestone
3	-54	-99	45	Limestone with greater hydraulic conductivity [1]	grouted limestone beneath the NI	
4	-99	-150	51	Limestone with greater hydraulic conductivity [1]		
5	-150	-250	100	Limestone with greater hydraulic conductivity [1]		
6	-250	-450	200	Limestone		

[1] CH2M Hill stated in FSAR 2.4.12.1.2 that "Based on limited downhole geophysical testing and monitoring of drilling fluid losses at the LNP site, the most productive interval of the Upper Floridan aquifer appears to be at depths of approximately 30 to 60 meters (100 to 300 feet) bgs." (Ref. 2)

ft amsl = elevation, in feet above mean sea level
NI = nuclear island

TABLE 2 HYDRAULIC CONDUCTIVITIES USED DURING EACH MODEL RUN AND CALCULATED DEWATERING RATES

Model Simulation Run No.	Hydraulic Conductivity [1]										Dewatering Rate [2]		Hydraulic Head in Grouted Limestone (Layer 2) Near Center of NI
	Layer 1 Sand		Layers 2 & 6 Limestone		Layers 3-5 Limestone		Grouted Limestone		Diaphragm Wall				
	(ft/day)	(cm/s)	(ft/day)	(cm/s)	(ft/day)	(cm/s)	(ft/day)	(cm/s)	(ft/day)	(cm/s)	(gpm)	(ft^3/day)	
1 (Base Run)	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.002835	1.0E-06	66	12800	-9.27
2	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.02835	1.0E-05	148	28400	-8.85
3	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.0002835	1.0E-07	56	10800	-9.37
4	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	2.835	1.0E-03	0.002835	1.0E-06	452	87000	-5.77
5	9.2	3.2E-03	13.9	4.9E-03	179.0	6.3E-02	0.2835	1.0E-04	0.002835	1.0E-06	67	12900	-9.16
6	35.0	1.2E-02	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.002835	1.0E-06	68	13000	-9.79
7	9.2	3.2E-03	13.9	4.9E-03	27.8	9.8E-03	0.2835	1.0E-04	0.002835 [3]	1.0E-06 [3]	94	18100	-9.28

[1] 1.0 cm/s = 2,835 ft/day

[2] All dewatering accomplished by wells and sumps installed within the flow barriers.

[3] In Run No. 7, the diaphragm wall had a typicl hydraulic conductivity of 1.0E-06 cm/s, except for three vertical windows in the wall where the diaphragm panels separate and a 3.5-foot gap occurs. This simulation was run to determine how seepage rate would be affectedd by multiple defects in the wall. The cells representing each "gap" were assigned hydraulic conductivites of the surrounding sand (3.2E-03 cm/s).

cm/s = centimeters per second
gpm = gallons per minute
ft/day = feet per day
ft^3/day = cubic feet per day
NI = Nuclear Island

TABLE 3. PRECIPITATION DATA FOR THREE NEARBY METEOROLOGICAL STATIONS

Meteorological Station	Period of Record	Mean Precipitation (inches per month)												Mean Annual Precipitation (inches per year)
		Jan	Feb	Mar	April	May	June	July	Aug	Sept	Oct	Nov	Dec	
Usher Tower, Levy County [1]	1961-1990	4.21	4.21	4.38	3.22	3.54	6.66	9.33	9.71	6.86	1.95	2.59	3.58	60.24
Inverness 3 SE, Citrus County [2]	1948-2007	3.01	3.19	4.11	2.39	3.36	7.49	7.98	8.45	6.30	2.87	1.87	2.55	53.57
Ocala, Marion County [3]	1948-2007	2.93	3.37	3.94	2.78	3.53	7.26	7.63	6.79	6.24	3.03	2.10	2.72	52.33
Average of Three Stations		3.38	3.59	4.14	2.80	3.48	7.14	8.31	8.32	6.47	2.62	2.19	2.95	55.38

Source of Data: Southeast Regional Climate Center, 2008.

1) http://www.sercc.com/cgi-bin/sercc/cliNORMNCDC.pl?fl9120
2) http://www.sercc.com/cgi-bin/sercc/cliMAIN.pl?fl4289
3) http://www.sercc.com/cgi-bin/sercc/cliMAIN.pl?fl6414

TABLE 4 HYDRAULIC HEADS ESTIMATED FROM FSAR FIGURES AND USED TO ASSIGN CONSTANT HEADS TO MODEL BOUNDARIES

Model Layer	Model Coordinates (model x,y in ft), Corners of Model Area				Estimated Head, March 2007				FSAR Figure No.
	North	East	South	West	North	East	South	West	
1 Surficial Aquifer	3850, 3850	3850, 0	0,0	0,3850	42.75	44.5	41.7	41.2	2.4.12-203 Rev. C
2- 6 Avon Park Lms.	3850, 3850	3850, 0	0,0	0,3850	42.0	43.0	40.7	40.4	2.4.12-204 Rev. C

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ATTACHMENT B
Figures

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APPROVED BY	
JMJ	3/28/08
DRAWN BY	
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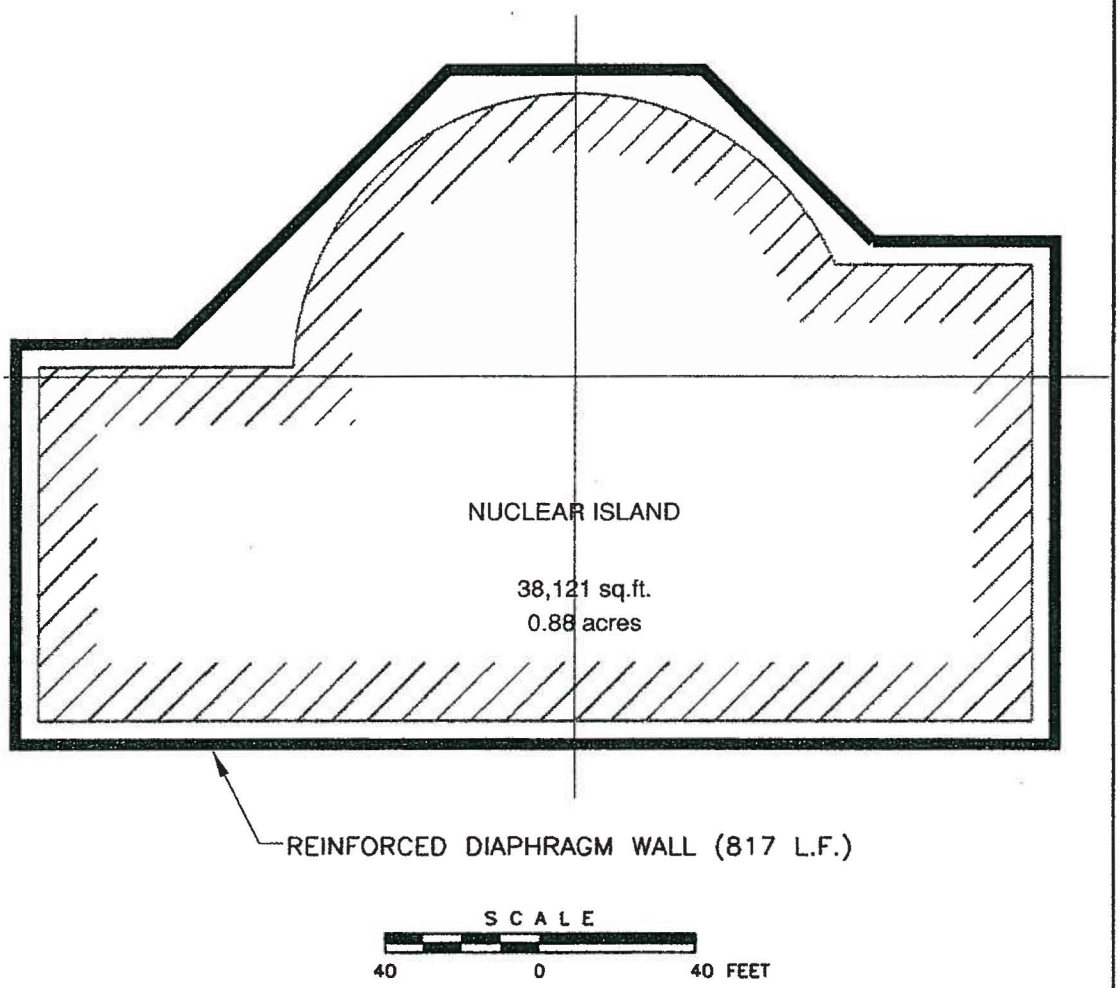

FIGURE 1

PLAN VIEW OF NUCLEAR ISLAND AND DIAPHRAGM WALL

LEVY COUNTY NUCLEAR PROJECT
INGLIS, FLORIDA

PREPARED FOR

SARGENT & LUNDY
CHICAGO, ILLINOIS

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ENGINEERS & CONSULTANTS

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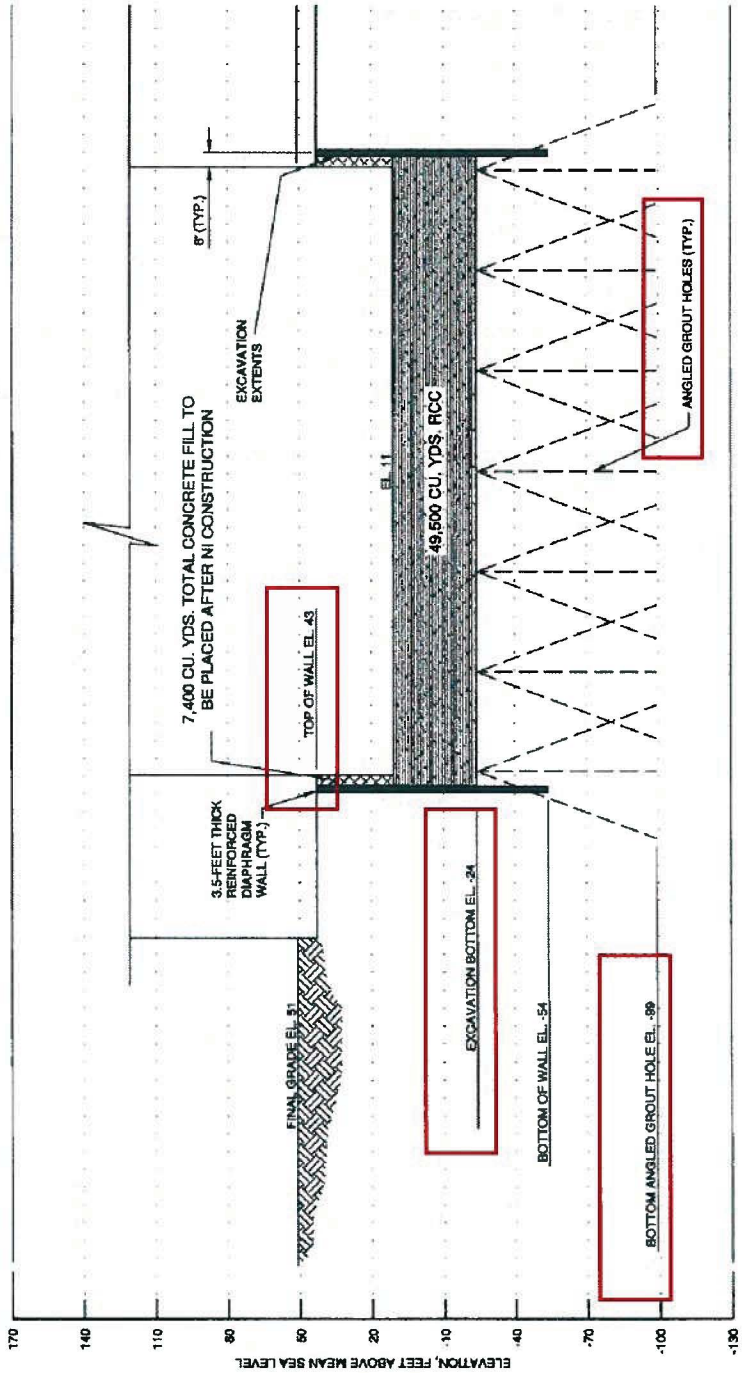

FIGURE 2. SCHEMATIC CROSS SECTION OF NUCLEAR ISLAND, DIAPHRAGM WALL, AND GROUTED BEDROCK

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LNP COL 2.4-4

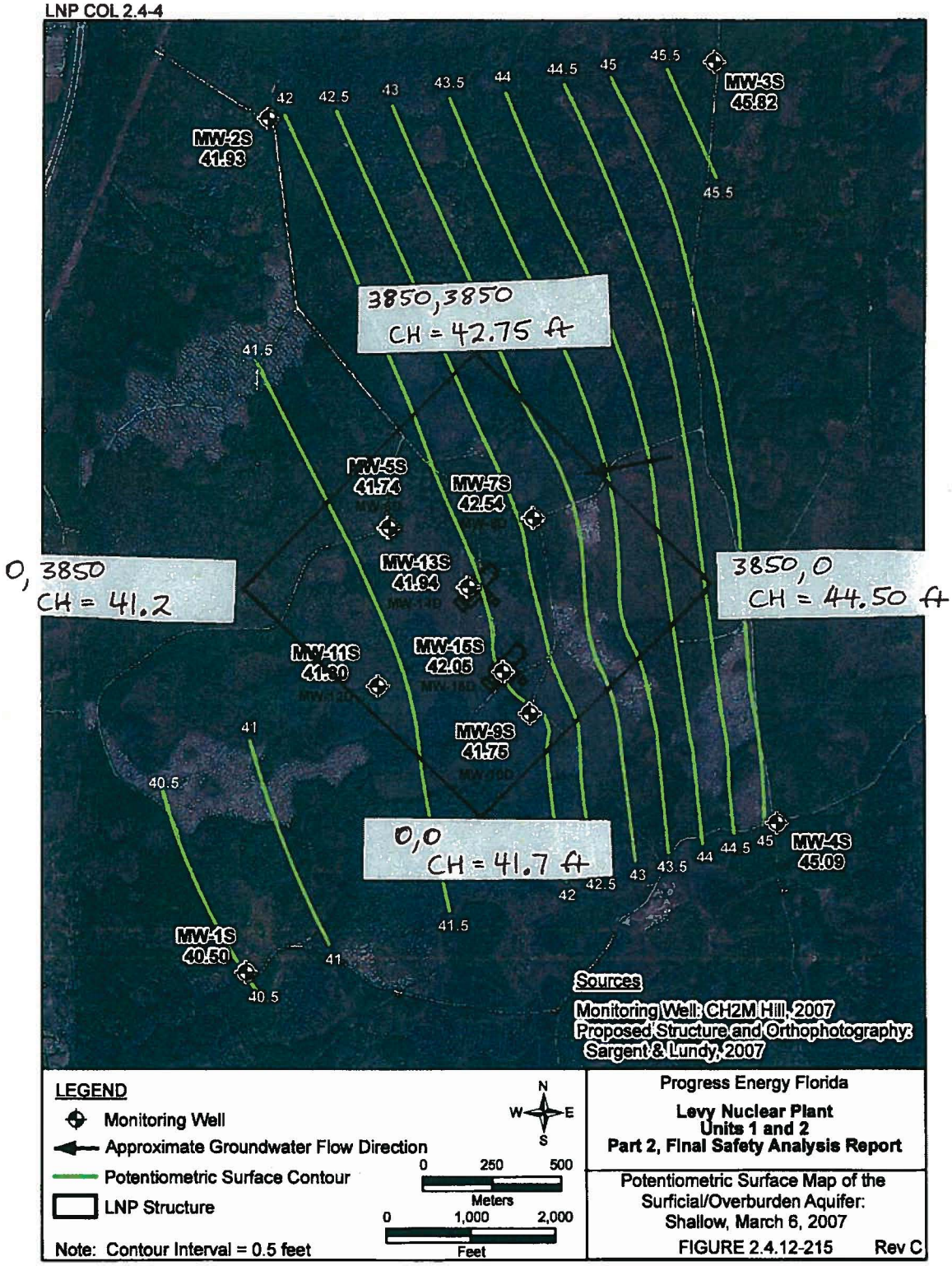

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FIGURE 3. CONSTANT HEADS (CH) ASSIGNED TO THE CORNERS OF MODEL LAYER 1 (Surficial Aquifer) (Ref. 2)

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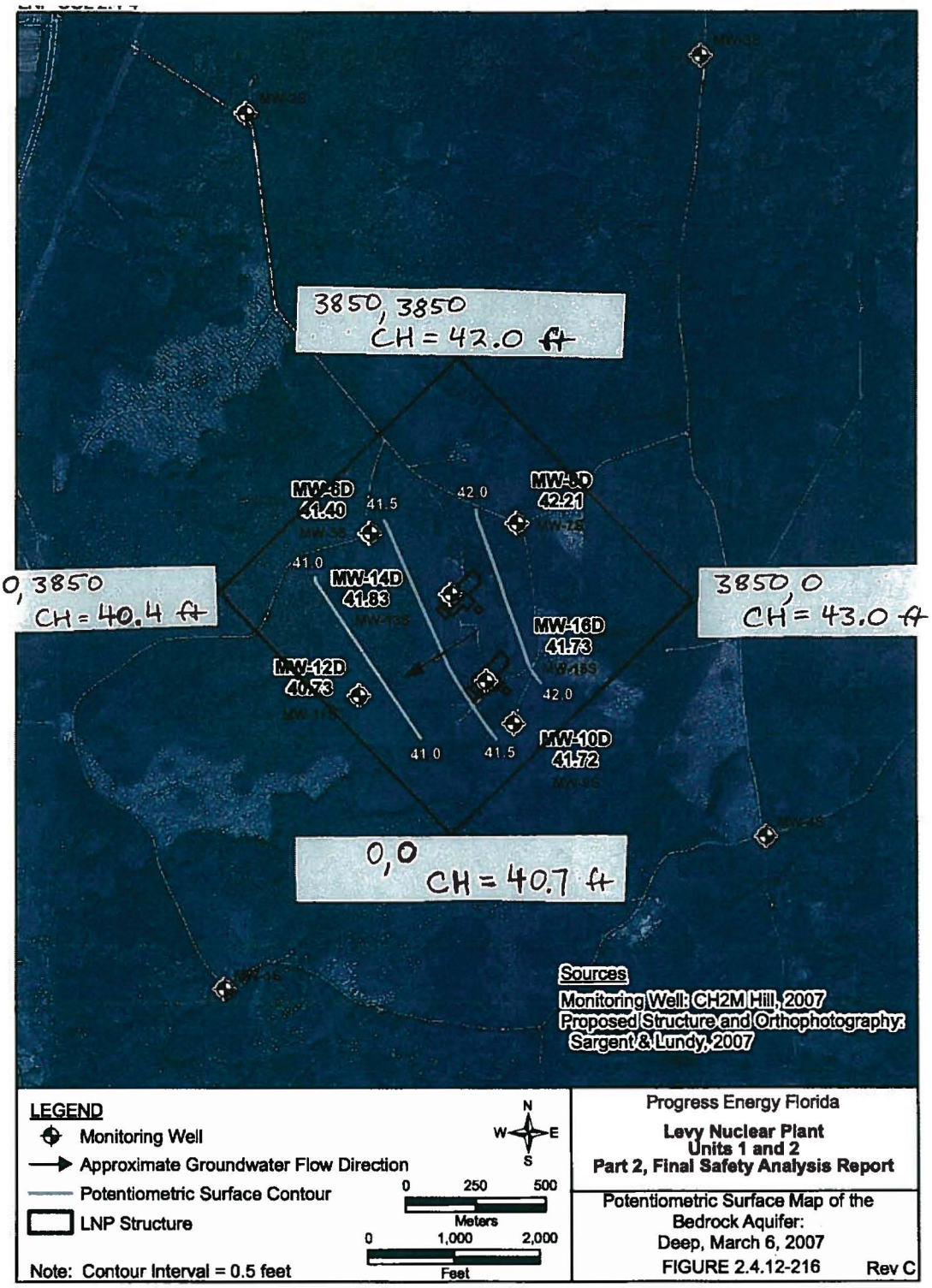

FIGURE 4. CONSTANT HEADS (CH) ASSIGNED TO THE CORNERS OF MODEL LAYERS 2-6 (Avon Park Formation) (Ref. 2)

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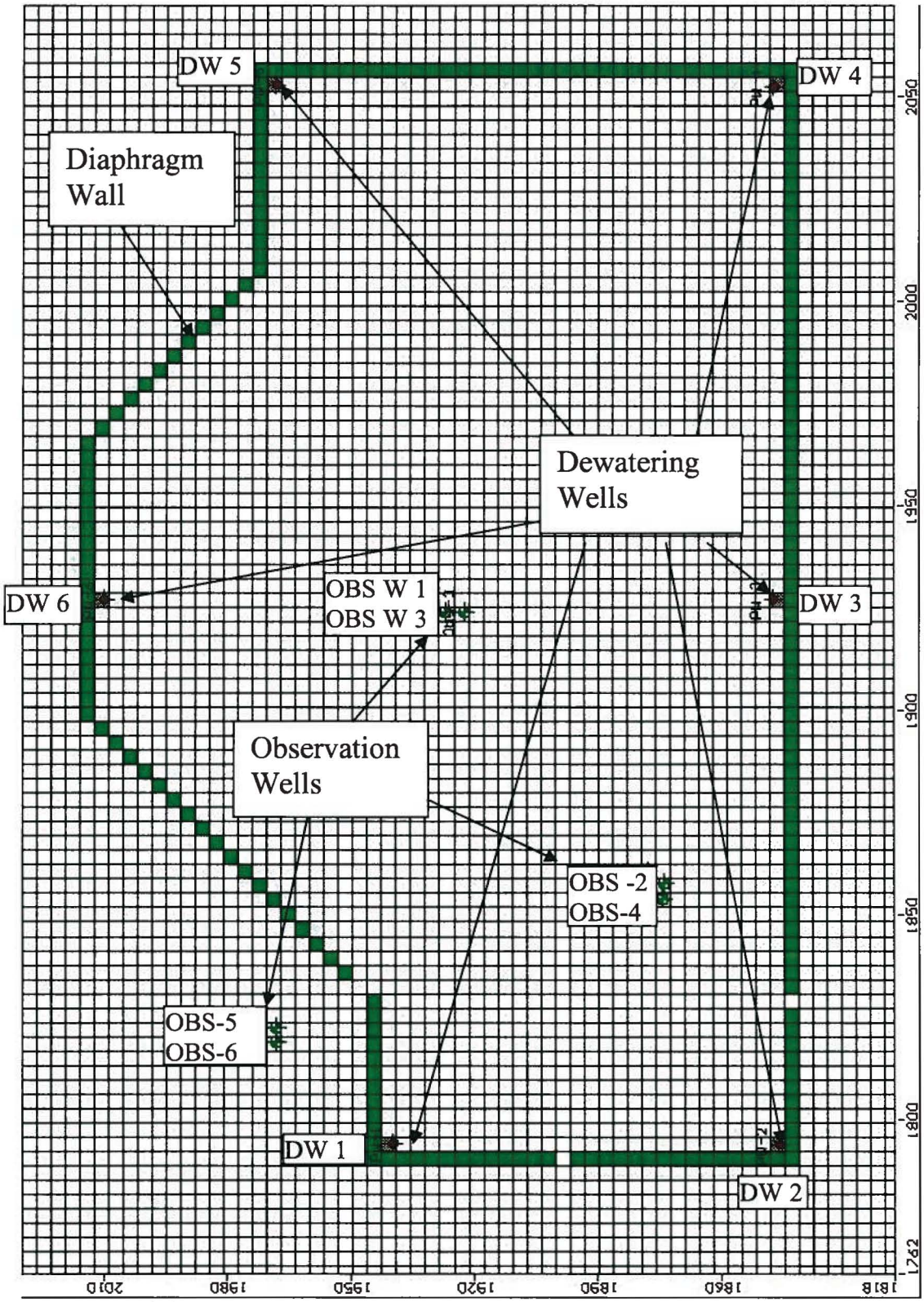

FIGURE 5. LOCATIONS OF DIAPHRAGM WALL, DEWATERING WELLS, AND OBSERVATION WELLS (OBS = Observation Well, DW =Dewatering Well)

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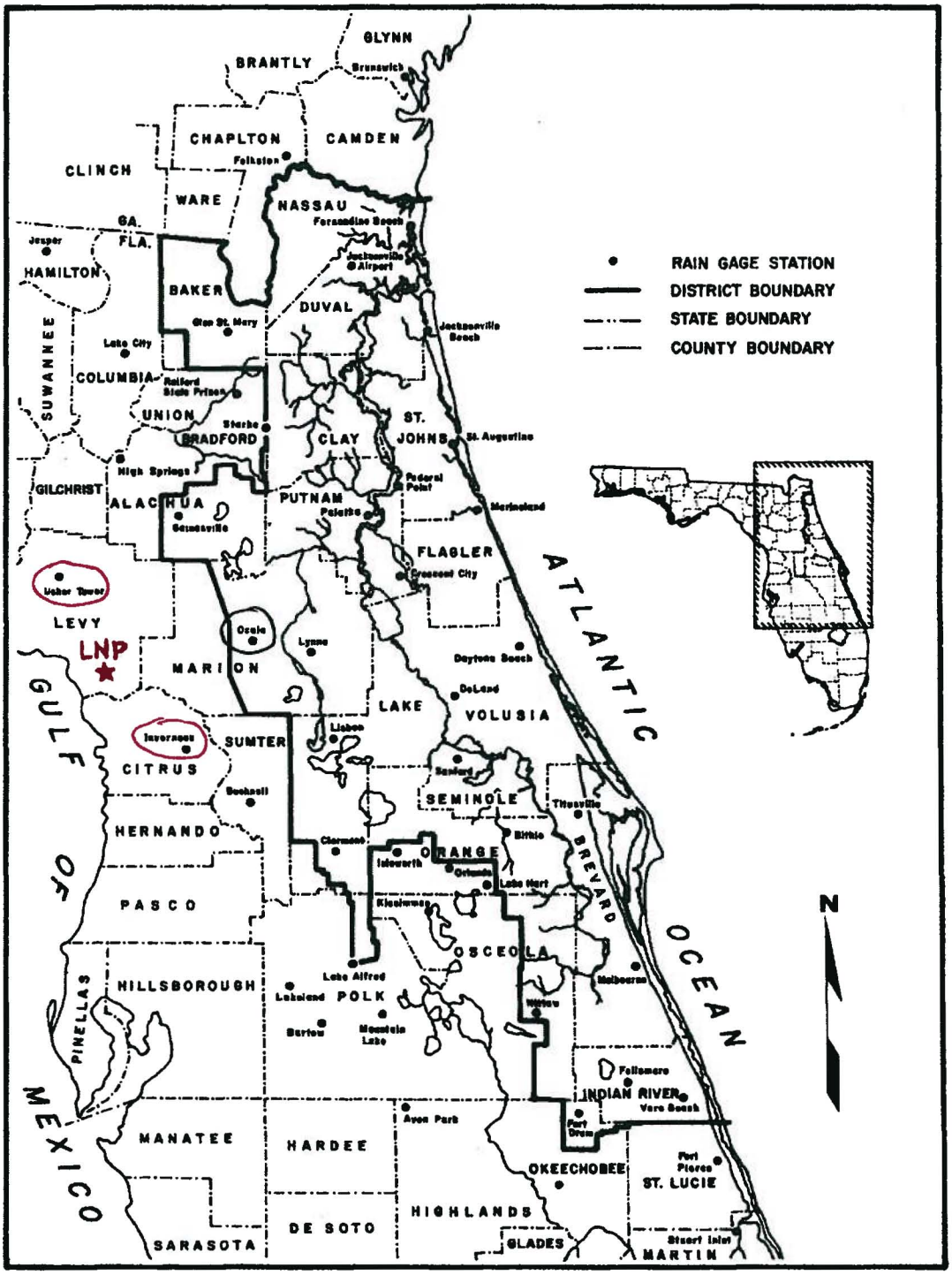

FIGURE 6. METEOROLOGIC STATIONS LOCATED CLOSEST TO LNP (modified from Rao, 1988 [Ref. 13])

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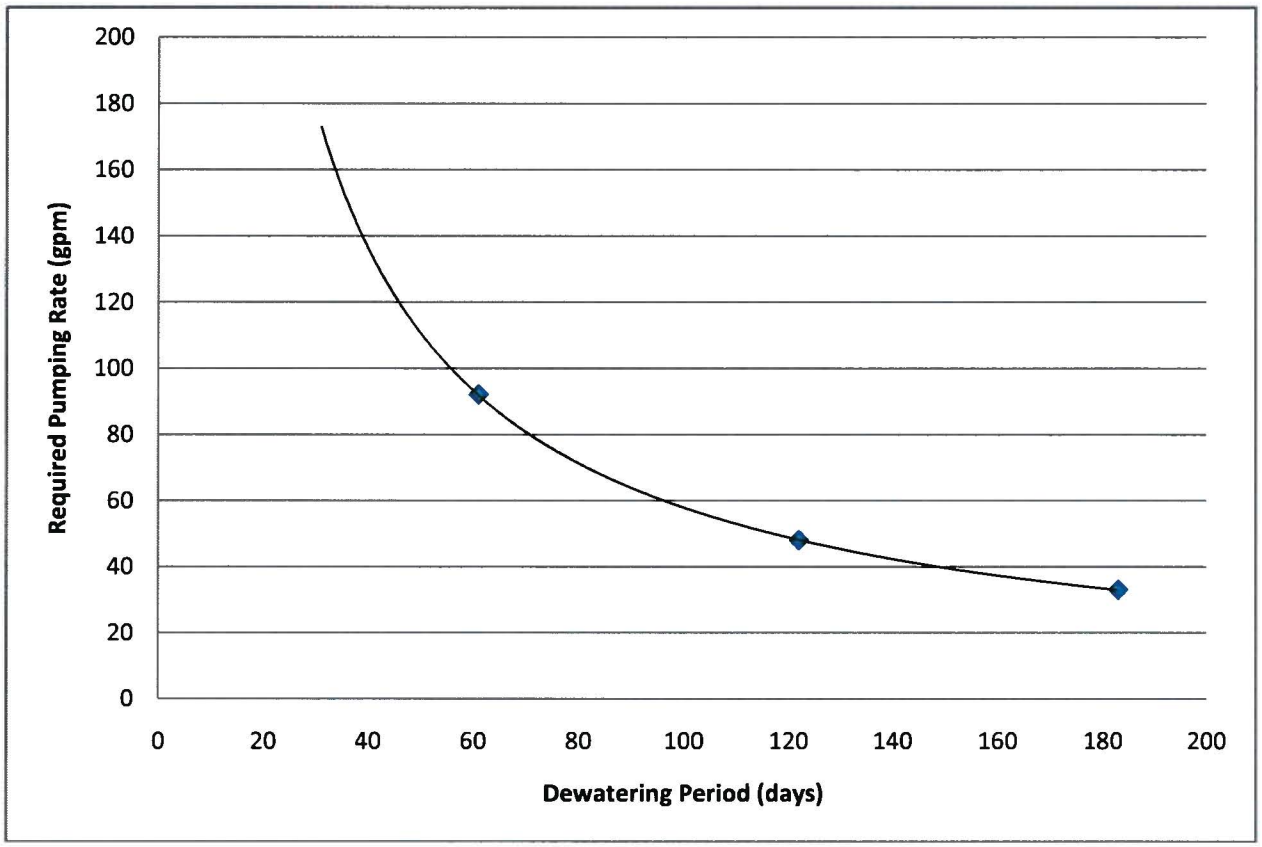

FIGURE 7. CALCULATED PUMPING RATES VERSUS DEWATERING PERIOD

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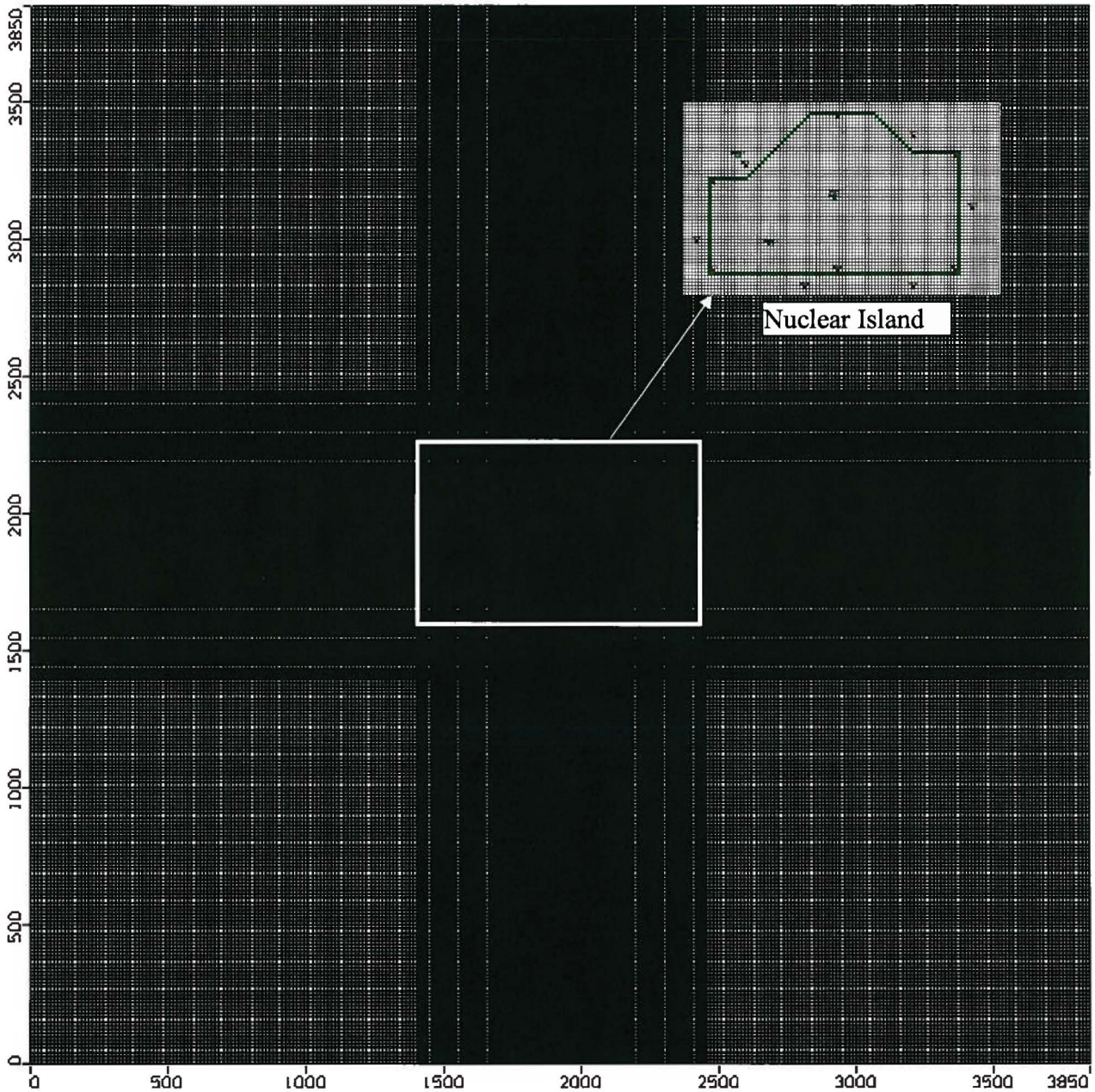

FIGURE 8. MODEL GRID (400 ROWS BY 400 COLUMNS) AND LOCATION NUCLEAR ISLAND IN THE GRID CENTER

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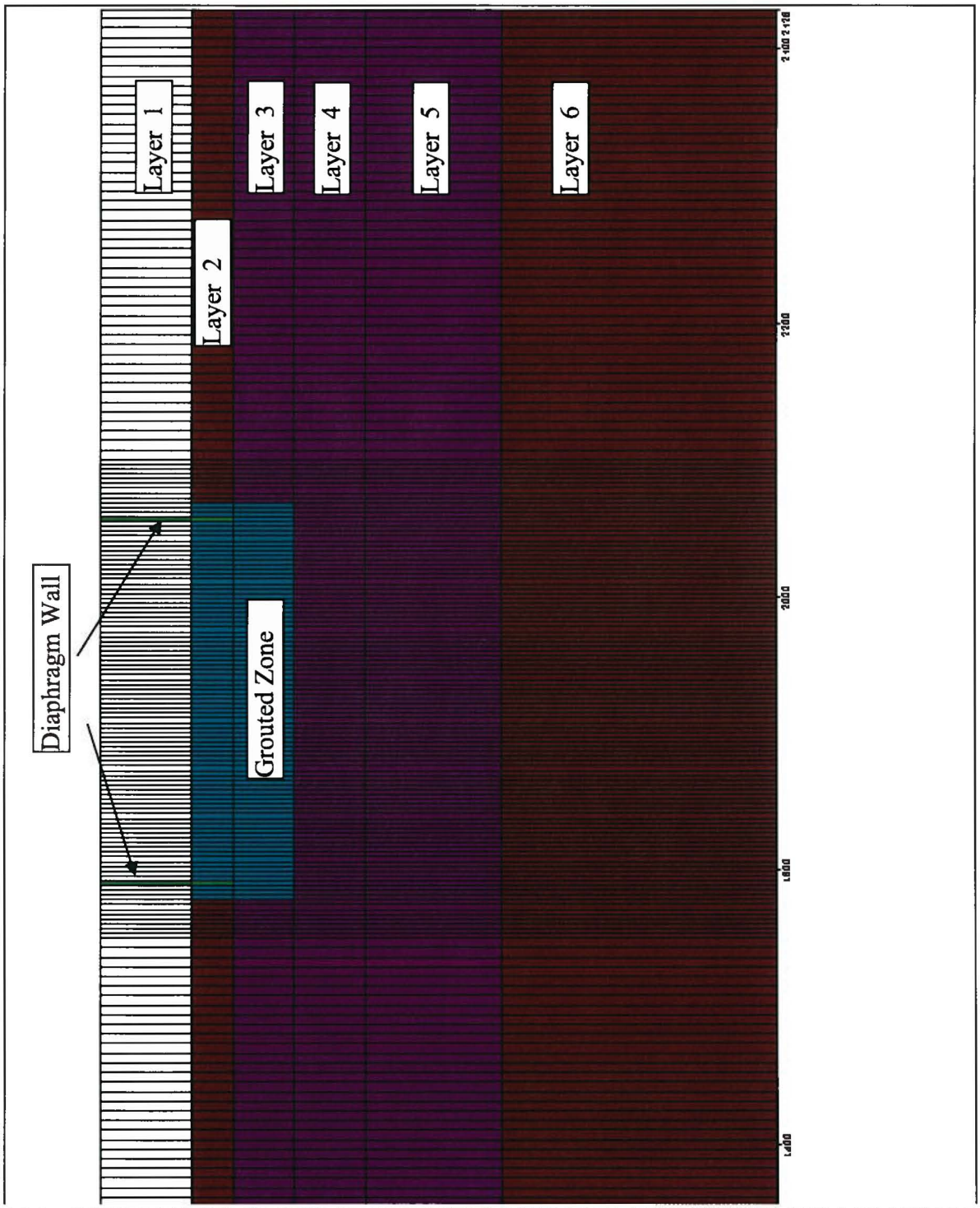

FIGURE 9. SCHEMATIC CROSS SECTION OF MODEL LAYERS, DIAPHRAGM WALL, AND GROUTED LIMESTONE SLAB

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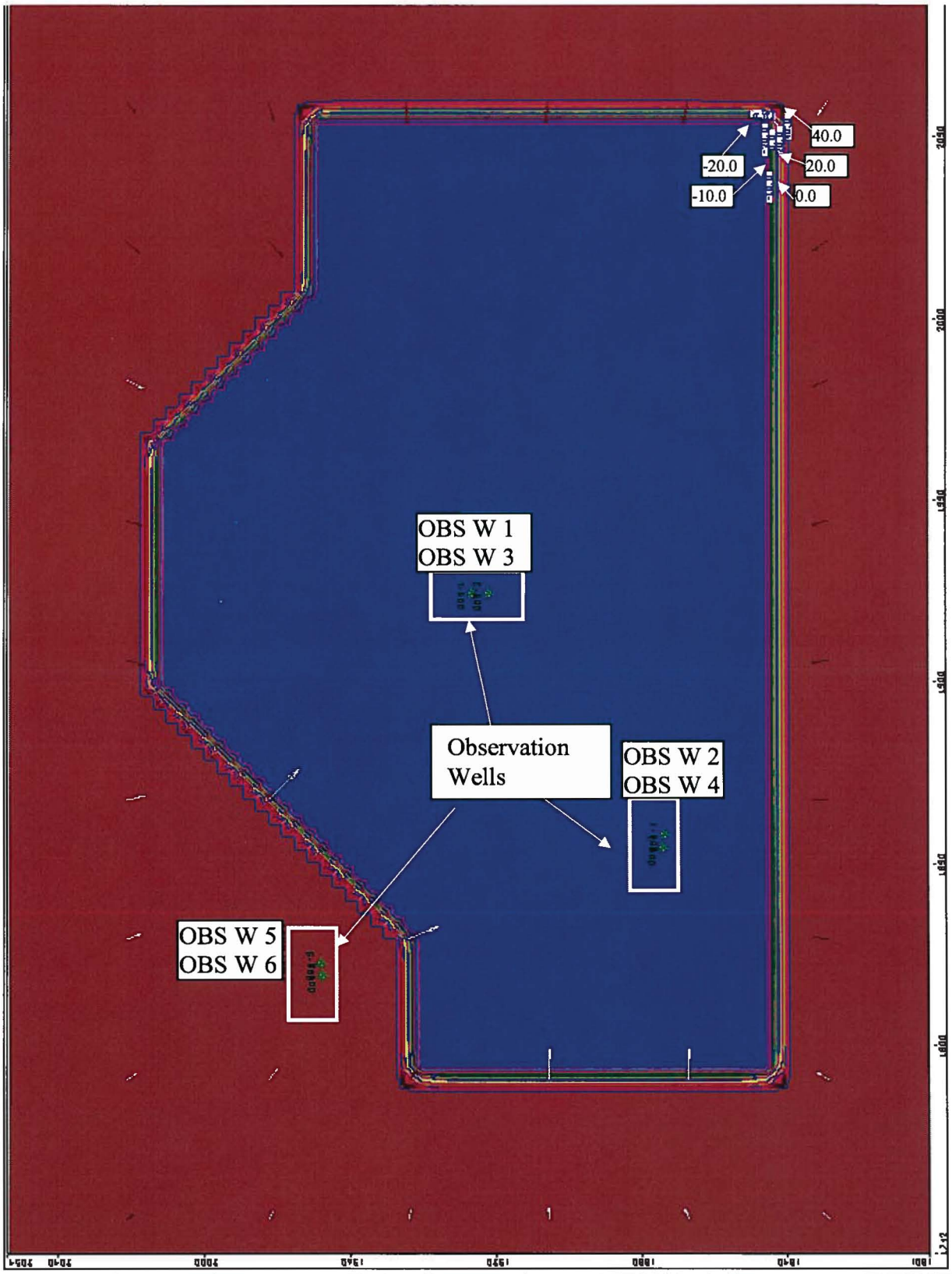

FIGURE 10. POTENTIOMETRIC SURFACE IN MODEL RUN NO. 1, LAYER 1, AT TIME = 81 DAYS

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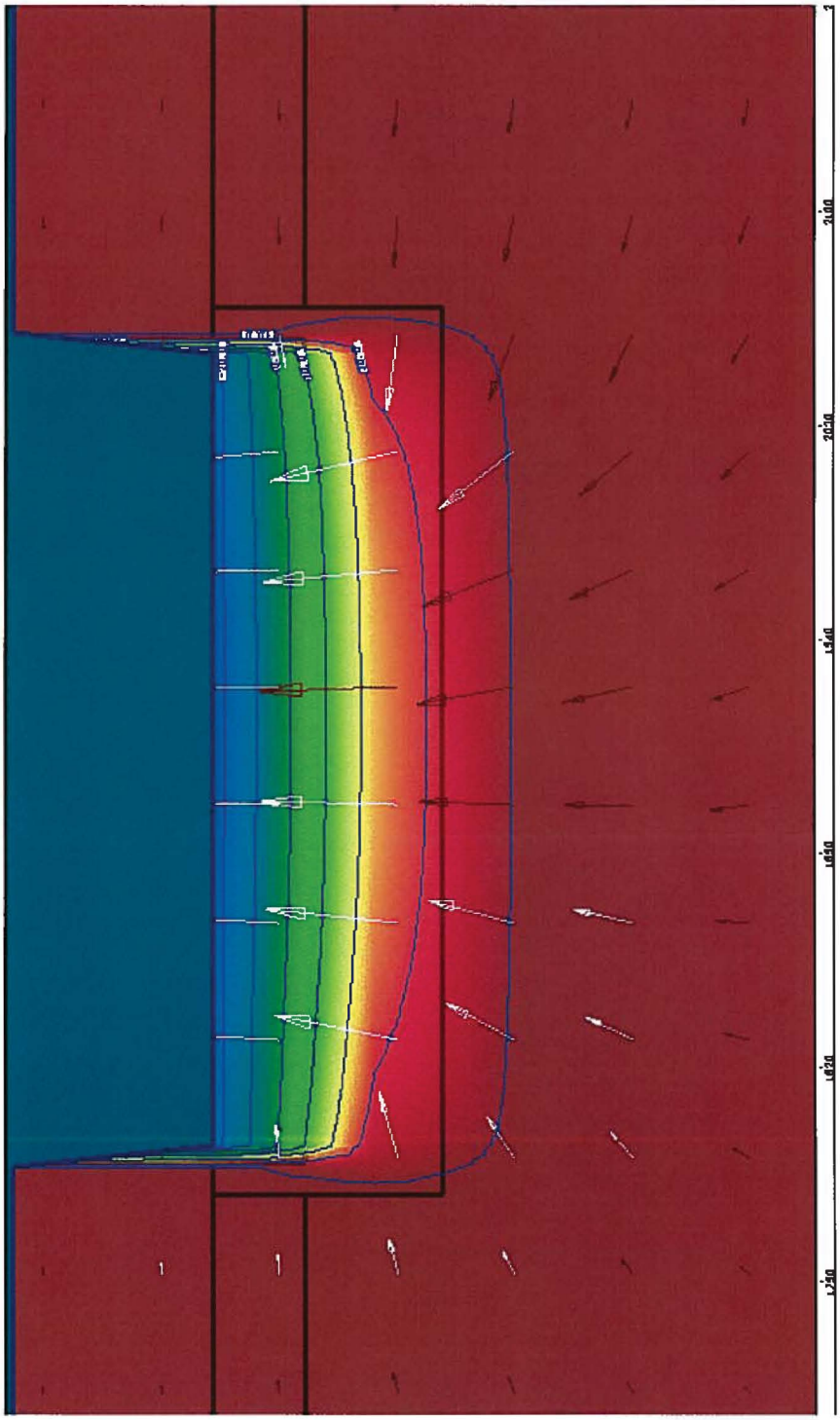

FIGURE 11. CROSS SECTION OF EXCAVATION PIT, SHOWING POTENTIOMETRIC CONTOURS AND FLOW VECTORS, MODEL RUN NO. 1 (Time= 81 Days)

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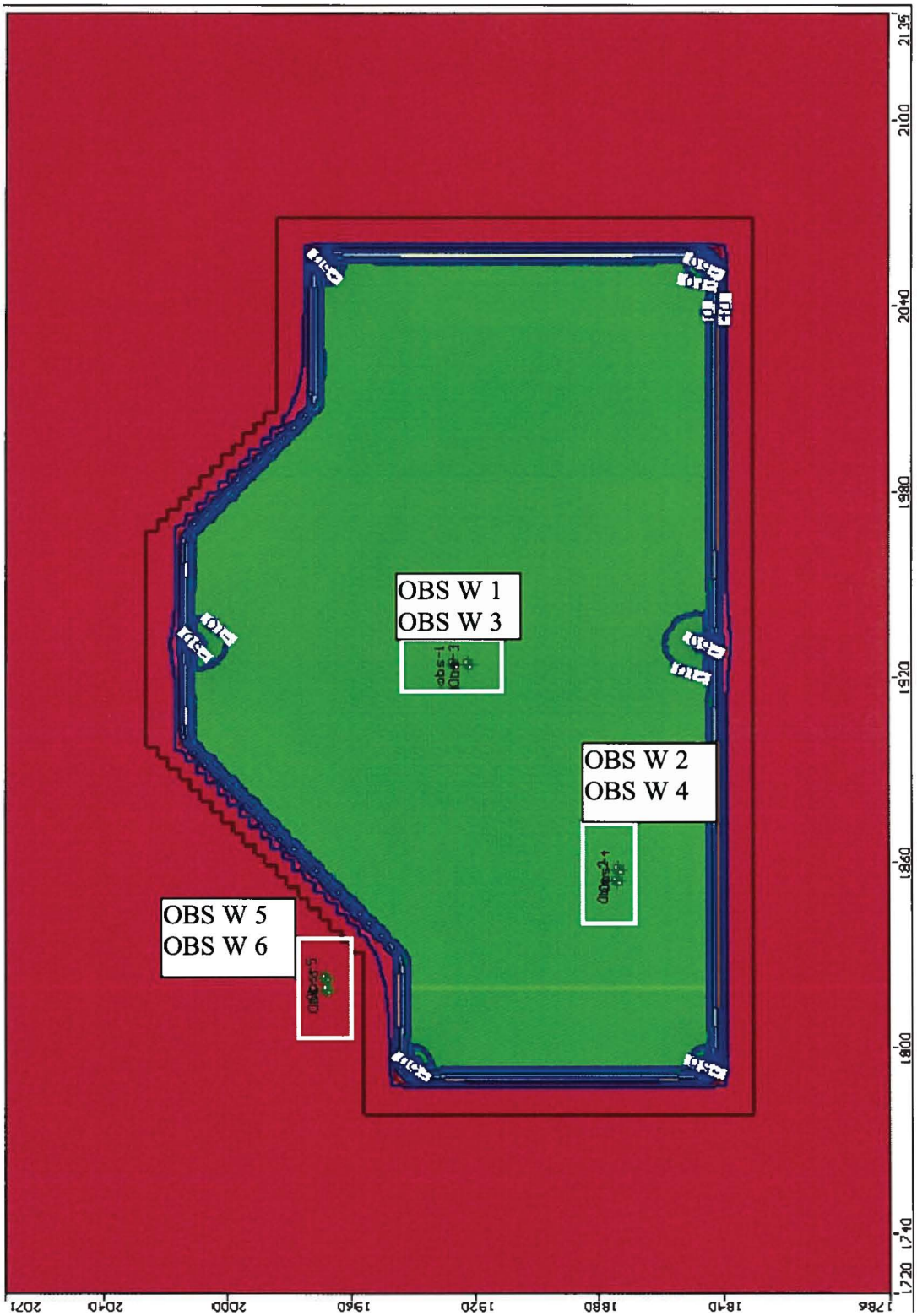

FIGURE 12. POTENTIOMETRIC SURFACE IN MODEL RUN NO. 1, LAYER 2, AT TIME = 81 DAYS

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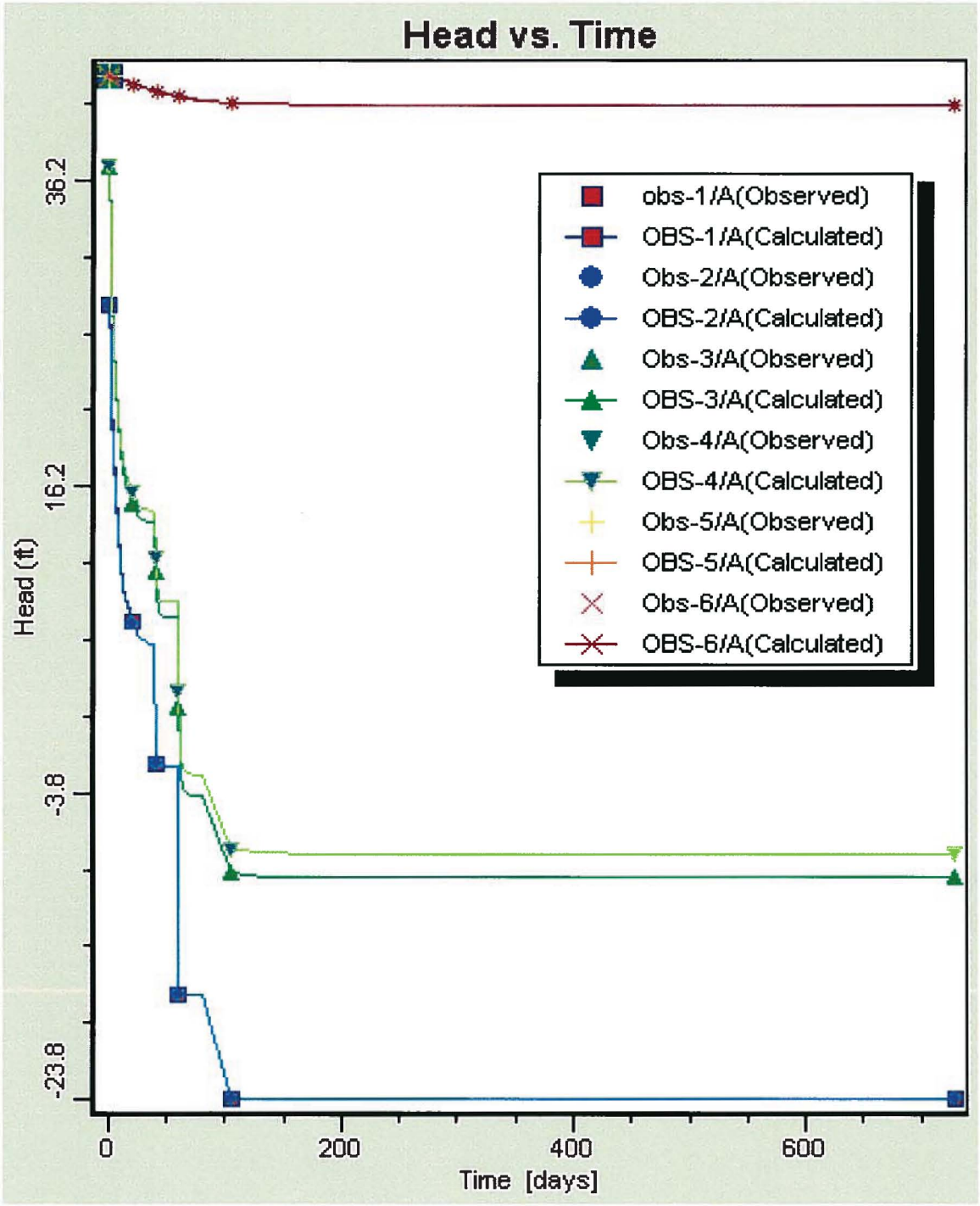

FIGURE 13. HYDRAULIC HEAD VERSUS TIME AT SIX LOCATIONS
(locations of observation wells shown on Figure 5)

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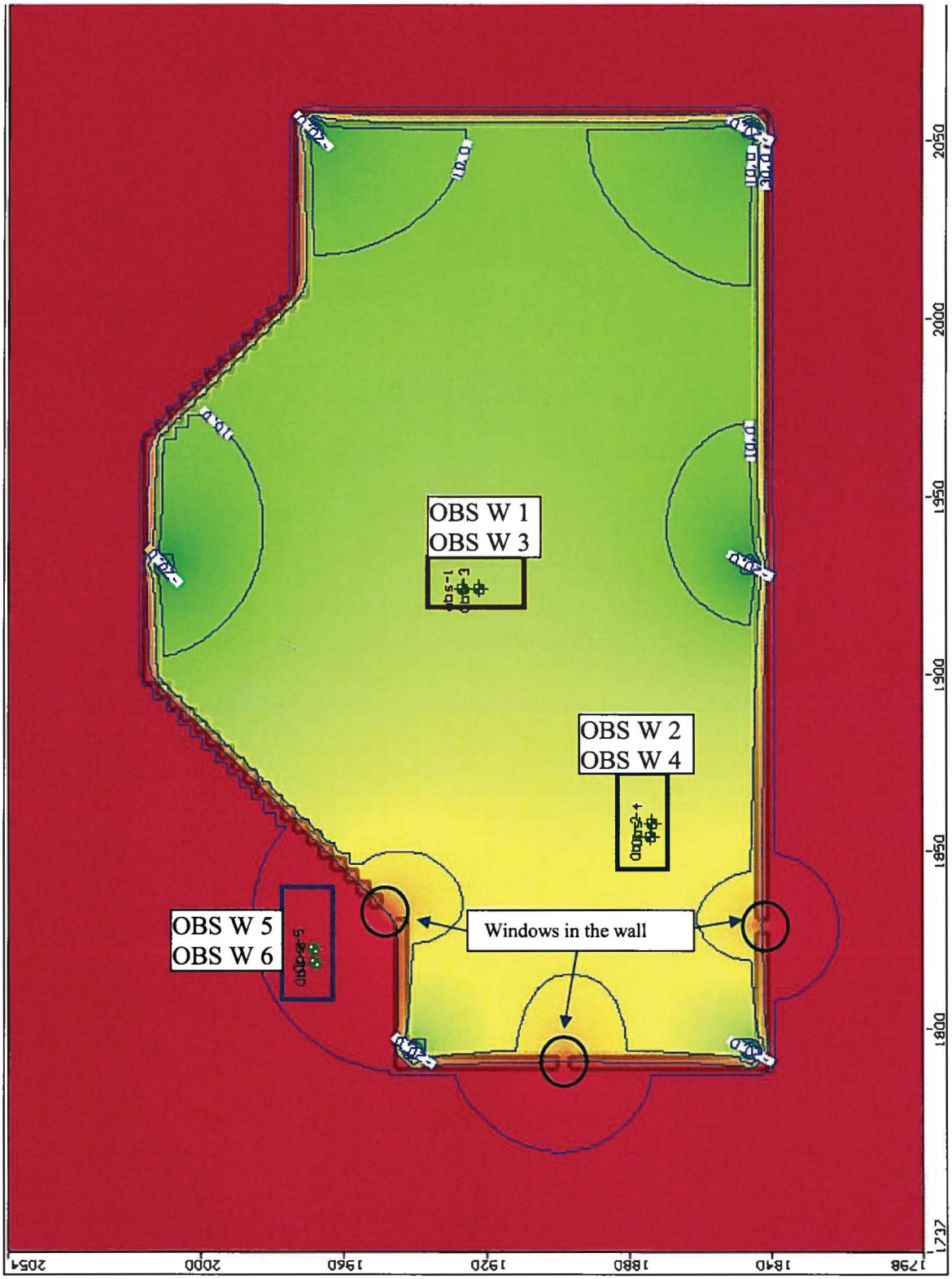

FIGURE 14. POTENTIOMETRIC SURFACE IN MODEL RUN NO. 7, LAYER 1, AT TIME = 81 DAYS

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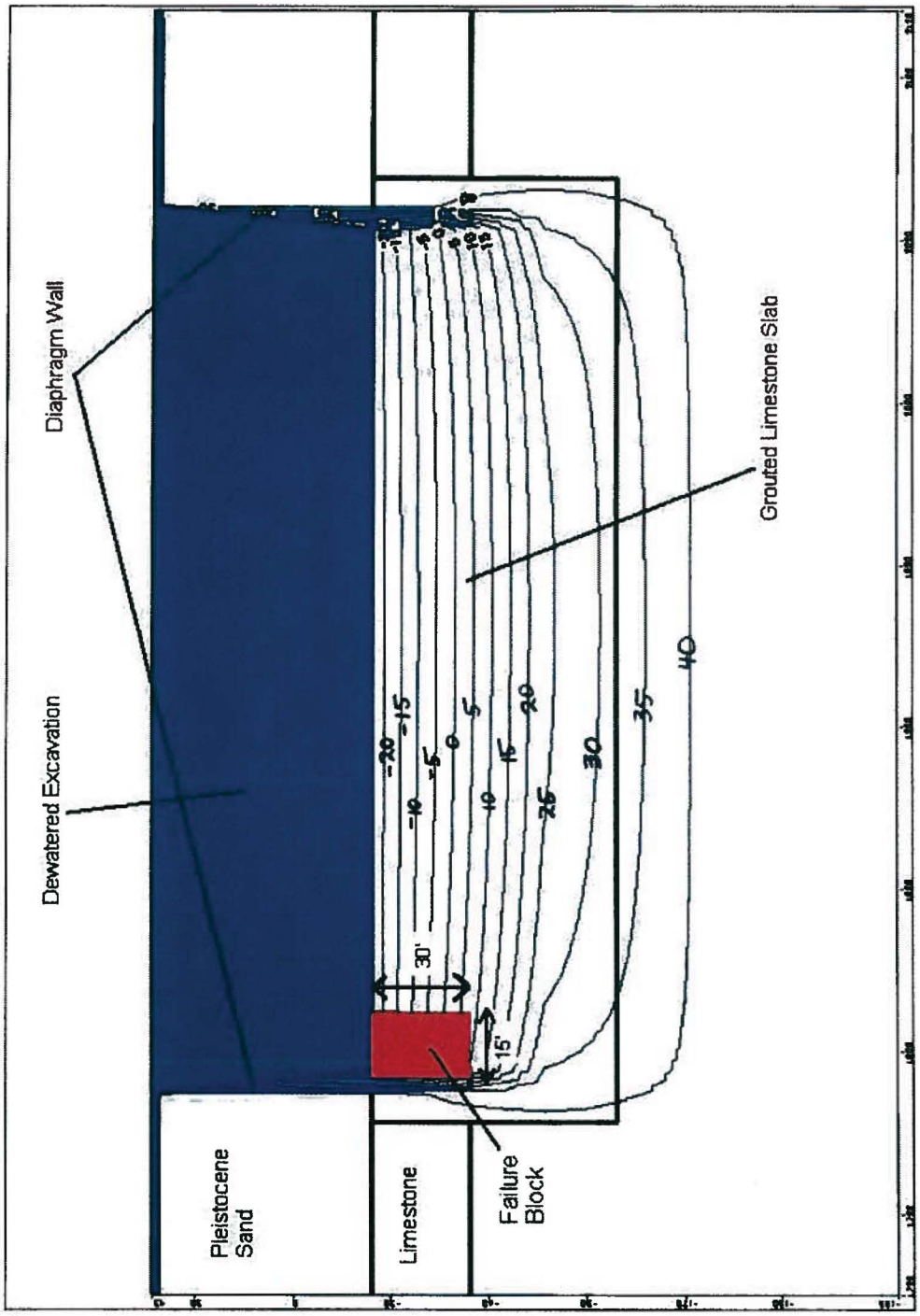

FIGURE 15. INPUT DATA FOR UPLIFT CALCULATIONS

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By JPS Date 5/23/08 Subject Design of Excavation Sheet No. __ of ____
Chkd. by MD Date 5/23/08 Dewatering System (Revision 1) Proj. No. 07-3935

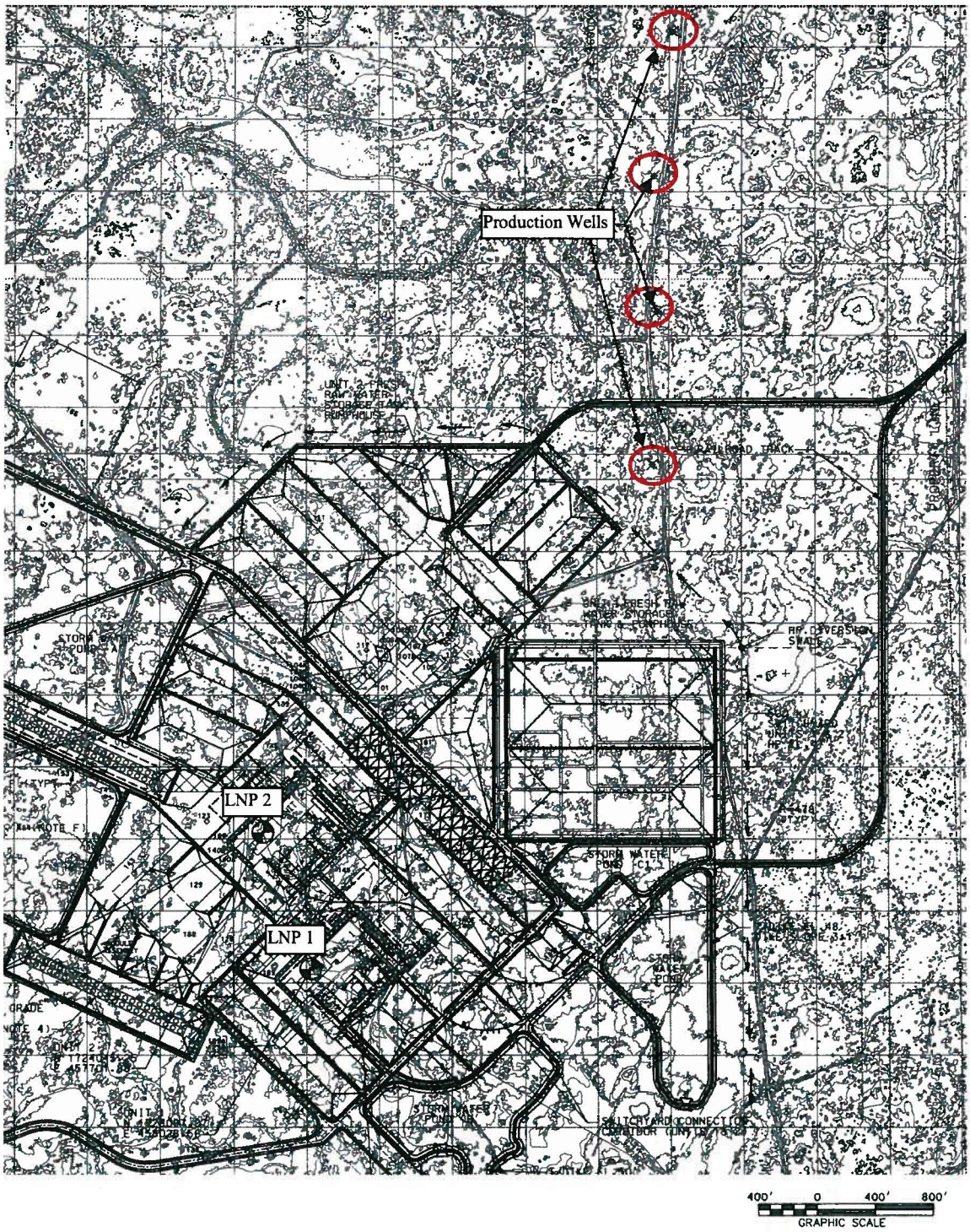

FIGURE 16. LOCATION OF FOUR WATER-SUPPLY WELLS NORTHEAST OF LNP (modified from Drawing LNG-0000-XG-001 Prelim 05-06-2008)

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ATTACHMENT C
References

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CONSTRUCTION DEWATERING AND GROUNDWATER CONTROL

New Methods and Applications
Third Edition

PATRICK POWERS, P.E.
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used to backfill the trench. With a soil-based backfill, slurry trench construction is usually performed by excavating a continuous trench using a backhoe (Fig. 21.18). The width of the trench can be variable, often dictated by the type of excavation equipment used, but generally is in the range of 2 to 5 ft (0.6 to 1.5 m), with a 3-ft (1-m) width most common. Trench widths narrower than 2 ft (0.6 m) can inhibit the backfilling of the trench and cause bridging of the backfill and entrapment of slurry. Thicker trenches may be necessary to prevent hydraulic fracture or piping of the backfill into the surrounding soil where the completed trench is exposed to large differential heads such as may exist beneath a dam. For instance, the Corps of Engineers [21-10] recommend that the width of an S-B trench be at least 0.1 ft (0.03 m) wide for every 1 ft (0.3 m) of differential head.

Stability of the trench is maintained during excavation by filling the trench with a viscous slurry, whose level in the trench is maintained near ground surface and several feet above the level of the prevailing groundwater table at all times. The slurry is typically a mixture of 4 to 6% bentonite (by weight) and water with an initial specific gravity between 1.03 and 1.07. Where groundwater levels are at or near ground surface elevations, construction of temporary earthen berms or work platforms is required to achieve the necessary differential head for trench stability.

As shown in Fig. 21.19, during excavation, a thin cake of bentonite forms on each side of the trench as clay particles are filtered from the slurry as it escapes out into the surrounding soil. Formation of the filter cake stops the loss of slurry and causes a differential head to develop between the slurry and groundwater. The hydrostatic pressure of the slurry opposes the active earth pressures and acts to stabilize the trench walls. When initially prepared, the slurry is only slightly heavier than water because it contains only a small amount of solids. As the excavation progresses, clay, silt, and sand particles become suspended in the slurry. The suspended sediments increase the weight of the slurry and thereby enhance trench stability. Trench stability therefore depends on the properties of both the slurry and surrounding soils. Xanthakos [21-11] and Filz, Adams, and Davidson [21-12] provide procedures for evaluation of trench stability. The final depth of the trench is dictated by the depth to the cutoff stratum. Usually the slurry trench is keyed at least 3 ft (1 m) into the cutoff stratum. For a hanging slurry trench (i.e., for cutoff of certain contaminants such as LNAPLs), the depth of the seasonally lowest water table usually determines trench depth.

In an S-B trench, the excavated soil, if suitable, or imported fill is mixed at the surface with small amounts of bentonite slurry from the trench to form the trench backfill. The slurry addition gives the backfill a cohesion that makes it behave like a high-slump concrete and flow as a viscous mass when pushed into the trench. Mixing of the backfill is usually done on one side of the trench using a bulldozer (Fig. 21.20) but, where space is limited or greater control

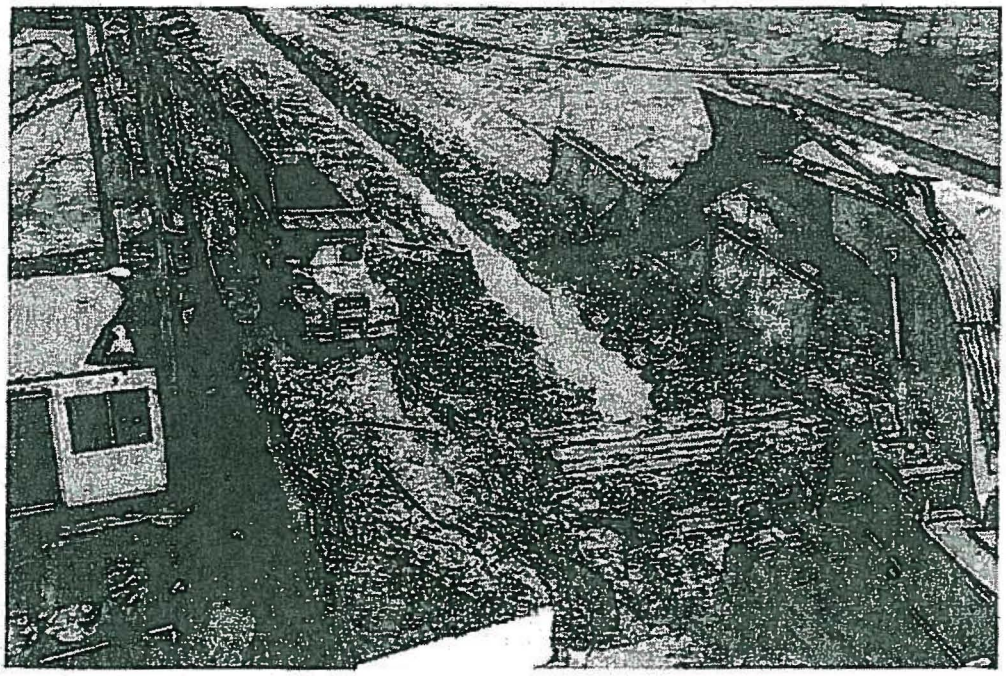

Figure 21.18 Slurry trench excavation with backhoe. Courtesy Mueser Rutledge Consulting Engineers.

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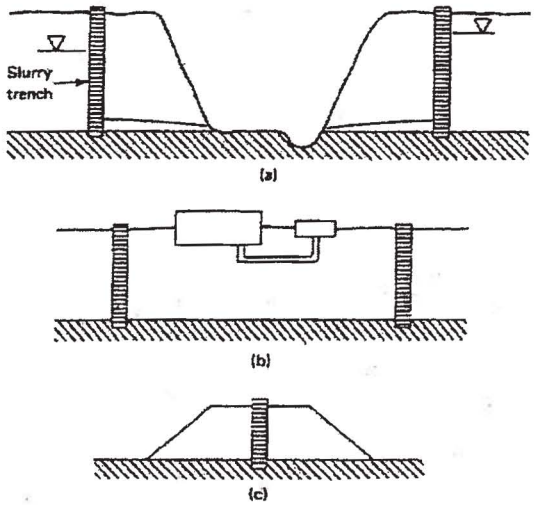

Figure 21.16 Slurry trench applications. (a) Construction dewatering. (b) Containment of groundwater pollution. (c) Sealing of dikes and dams.

pends principally on the proper selection and design of the backfill and the quality control used in its construction.

The U.S. Army Corps of Engineers [21-10] indicate that slurry trenches using S-B backfill were first employed for groundwater cutoff in the United States as early as 1945 and advanced further with improvements in excavating equipment; however, it is only in the last 25 years that the method has experienced rapid growth and innovation due primarily to widespread acceptance in the field of permanent environmental containment. Slurry trenches have been employed for dewatering, for permanent service to control seepage under dams and levees, and to contain groundwater pollution from sanitary landfills or industrial spillage (Fig. 21.16). Where steel sheet piling has been inserted in the trench during backfill placement, the method has also been used to provide both groundwater cutoff and temporary excavation support. Plastic sheetpiling has also been used to provide excavation support in shallow trenches.

Slurry Trench Construction

The general sequence of slurry trench construction is illustrated in Fig. 21.17. Construction variations are due primarily to the depth of the trench and the types of materials

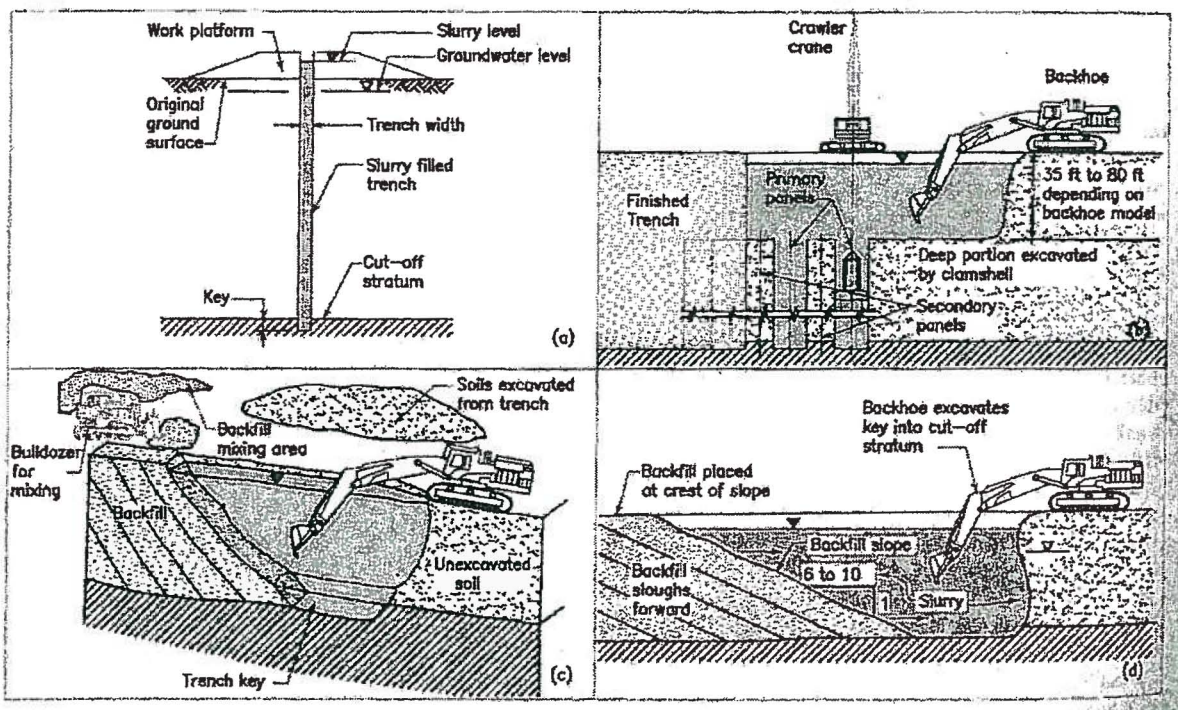

Figure 21.17 Slurry trench construction. (a) Definitions and terms. (b) Trench excavation proceeds to a suitable cutoff stratum with bentonite slurry used to maintain trench stability. Excavation to depths of up to 80 ft (24.4 m) is possible with a backhoe. Crane mounted clamshell buckets can be used where greater depths are required. (c) Mixing of the backfill is usually done on one side of the trench using a bulldozer. (d) The mixed backfill is pushed in place by a dozer displacing the bentonite slurry to form the completed cutoff trench. Excavation, mixing, and backfill placement proceed in a more or less continuous process with a minimum length of trench remaining open under the slurry and new slurry added to replace slurry used to mix the backfill and keep the trench full.

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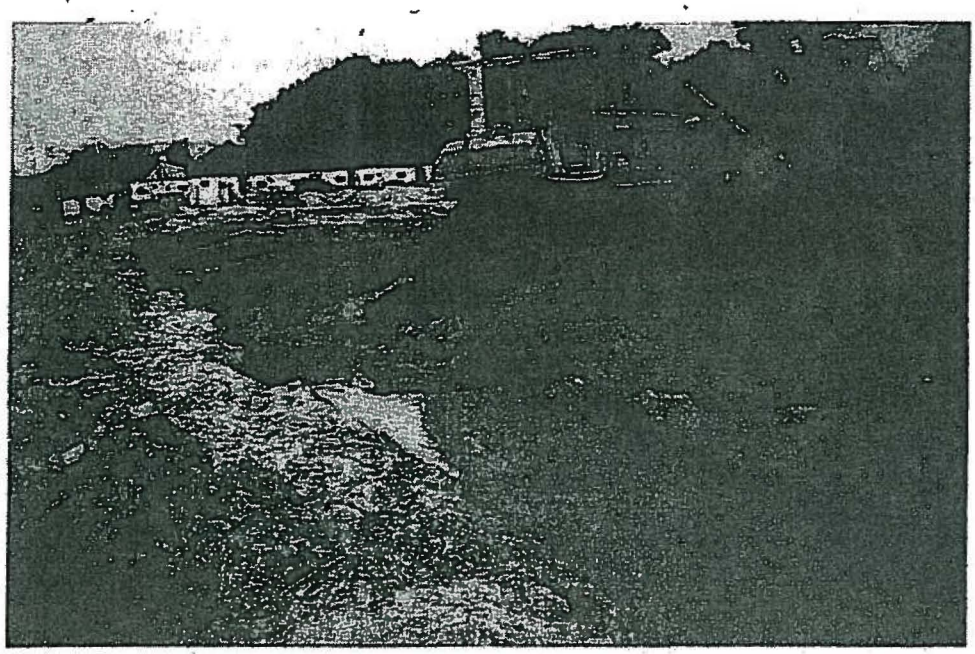

Figure 21.20 Backfill mixing with bulldozer adjacent to trench. Courtesy Mueser Rutledge Consulting Engineers.

relied upon in design since its integrity following excavation and backfilling is not assured.

A S-C-B trench is constructed similar to an S-B trench, except cement grout is also added to the backfill. The cement gives the backfill considerable strength, which can be important when working adjacent to structures or for trench construction beneath dams and other impoundments where backfill compressibility and resistance to piping under high reservoir heads become important. The backfill slope of S-C-B is steeper, with slopes in the range of 1V:3H to 1V:6H common. The steeper slope reduces the length of open trench and enhances trench stability relative to an S-B trench. The addition of cement, however, complicates the construction as time now becomes a factor in the mixing and placement of the backfill. Ryan and Day [21-13] indicate that S-C-B backfill must be placed within a few hours of batching or risk affecting trench continuity and the desired low hydraulic conductivity. Hydraulic conductivity of S-C-B backfill generally ranges from 0.02 to 0.01 gpd/ft^2 (1×10^{-8} to 5×10^{-9} m/sec).

Trench Construction with Self-hardening Slurries

In contrast to the two-step process of excavation and backfill that is required with a soil-based backfill, slurry trench construction with self-hardening slurry is performed in a single step, with the slurry left in place following excavation to harden and form the permanent backfill. Introduction of cement results in important differences in trench construction and long-term properties of the cutoff compared to an S-B slurry trench. The calcium in the cement inhibits hydration and causes flocculation of the bentonite. This results in more viscous slurry and a more permeable filter cake. As a consequence, hydraulic conductivity of C-B backfill is usually in the range of 2×10^{-2} gpd/ft^2 (1×10^{-8} m/sec), or an order of magnitude or more than that of S-B backfill. Time also becomes a factor in trench construction as excavation must proceed to the design cutoff depth prior to initial set of the cement or the C-B mix adjusted with retarders to delay set. C-B trenches can be excavated as a continuous trench or as a series of alternating and overlapping panels (Fig. 21.17b). In a C-B trench, 10 to 20% cement (by weight) is typically added to the bentonite slurry, raising its specific gravity to between 1.15 and 1.3. With the higher density C-B slurry, trench stability becomes less of a concern. With panel excavation, alternate "primary" panels are excavated under C-B slurry and allowed to set. Once set, excavation of the intervening "secondary" panels proceeds also under slurry. The secondary panels overlap and excavate the ends of the primary panels to provide a continuous trench. Due to its higher strength and resulting ability to resist internal erosion or piping and without the constraints imposed by backfill placement, narrower trench widths of between 2 and 2.5 ft (0.6 to 0.75 m) are viable and generally used to offset the higher material costs of cement.

Equipment and Plant

The excavating method is chosen based on the width and depth of the trench, the type of soil, accessibility to the trench at the ground surface and other factors. For depths less than 50 ft (15.2 m), use of a backhoe is preferred since

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USHER TOWER, FLORIDA

NCDC 1961-1990 Monthly Normals

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Mean Max. Temperature (F)	68.0	70.5	77.0	82.6	88.3	91.5	91.8	91.7	89.9	84.1	76.3	70.3	81.8
Highest Mean Max. Temperature (F)	80.7	77.2	81.8	86.4	92.5	95.6	94.6	94.3	92.5	87.8	81.6	77.6	83.9
Year Highest Occurred	1974	1975	1976	1989	1962	1981	1983	1980	1972	1985	1973	1984	1990
Lowest Mean Max. Temperature (F)	59.6	61.5	69.6	77.3	83.7	87.9	87.4	88.3	86.7	78.6	69.6	63.0	79.0
Year Lowest Occurred	1977	1978	1969	1961	1966	1965	1967	1967	1966	1964	1976	1963	1966
Mean Temperature (F)	55.1	57.2	63.4	68.4	74.4	79.3	80.6	80.8	78.8	71.1	62.9	57.1	69.1
Highest Mean Temperature (F)	68.4	64.5	68.0	72.2	77.1	82.8	83.0	83.4	81.2	76.5	69.6	65.2	71.7
Year Highest Occurred	1974	1990	1973	1970	1975	1981	1987	1980	1970	1985	1985	1971	1990
Lowest Mean Temperature (F)	47.2	49.3	55.9	63.4	72.5	76.9	76.6	78.2	75.8	67.0	56.6	48.0	67.2
Year Lowest Occurred	1977	1968	1969	1961	1988	1965	1967	1967	1967	1987	1976	1963	1968
Mean Min. Temperature (F)	42.2	43.8	49.7	54.1	60.6	67.0	69.4	70.0	67.8	58.0	49.5	43.8	56.3
Highest Mean Min. Temperature (F)	56.0	53.1	56.2	60.7	63.7	70.0	71.7	72.6	70.1	65.2	58.2	53.1	59.3
Year Highest Occurred	1974	1990	1973	1970	1990	1981	1987	1987	1977	1985	1985	1971	1990
Lowest Mean Min. Temperature (F)	31.3	35.1	42.1	48.4	57.2	64.5	65.7	65.5	63.3	52.0	42.0	33.0	53.5
Year Lowest Occurred	1981	1968	1969	1987	1967	1984	1967	1969	1969	1987	1970	1963	1969
Mean Precipitation (in.)	4.21	4.21	4.38	3.22	3.54	6.66	9.33	9.71	6.86	1.95	2.59	3.58	60.24
Highest Precipitation (in.)	14.50	8.73	9.74	14.61	13.14	13.11	19.40	20.44	20.12	7.12	7.52	9.84	99.47
Year Highest Occurred	1964	1970	1987	1982	1976	1974	1964	1971	1988	1971	1972	1964	1964
Lowest Precipitation (in.)	0.25	0.12	0.71	0.00	0.26	1.27	3.38	3.17	0.86	0.14	0.00	0.40	44.37
Year Lowest Occurred	1974	1962	1976	1967	1965	1977	1972	1980	1972	1973	1978	1984	1968

INVERNESS 3 SE, FLORIDA (084289)

Period of Record Monthly Climate Summary

Period of Record : 7/ 1/1948 to 6/30/2007

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	70.3	72.4	77.5	82.7	88.3	90.9	91.6	91.3	89.5	83.8	77.5	71.9	82.3
Average Min. Temperature (F)	44.9	46.7	51.6	56.8	63.5	69.8	71.7	71.7	70.1	62.0	52.9	46.8	59.0
Average Total Precipitation (in.)	3.01	3.19	4.11	2.39	3.36	7.49	7.98	8.45	6.30	2.87	1.87	2.55	53.57
Average Total SnowFall (in.)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Percent of possible observations for period of record.
Max. Temp.: 97% Min. Temp.: 97% Precipitation: 97.1% Snowfall: 97.2% Snow Depth: 97.1%
Check Station Metadata or Metadata graphics for more detail about data completeness.

Southeast Regional Climate Center, sercc@climate.ncsu.edu

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OCALA, FLORIDA (086414)

Period of Record Monthly Climate Summary

Period of Record : 7/ 1/1948 to 6/30/2007

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Average Max. Temperature (F)	70.6	73.2	78.4	83.5	89.0	91.4	92.2	91.9	89.7	84.0	77.1	71.6	82.7
Average Min. Temperature (F)	46.1	47.9	52.5	57.2	63.8	69.7	71.5	71.5	69.5	61.9	53.4	47.5	59.4
Average Total Precipitation (in.)	2.93	3.37	3.94	2.78	3.53	7.26	7.63	6.79	6.24	3.03	2.10	2.72	52.33
Average Total SnowFall (in.)	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Average Snow Depth (in.)	0	0	0	0	0	0	0	0	0	0	0	0	0

Percent of possible observations for period of record.
Max. Temp.: 98.2% Min. Temp.: 97.8% Precipitation: 98.6% Snowfall: 98.6% Snow Depth: 98.6%
Check Station Metadata or Metadata graphics for more detail about data completeness.

Southeast Regional Climate Center, sercc@climate.ncsu.edu

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Principles of Geotechnical Engineering

FIFTH EDITION

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California State University, Sacramento

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8.4 Seepage Force

The preceding section showed that the effect of seepage is to increase or decrease the effective stress at a point in a layer of soil. Often, expressing the seepage force per unit volume of soil is convenient.

In Figure 8.2, it was shown that, with no seepage, the effective stress at a depth z measured from the surface of the soil layer in the tank is equal to $z\gamma'$. Thus, the effective force on an area A is

$$P_1' = z\gamma' A$$

(The direction of the force P_1' is shown in Figure 8.7a.)

Again, if there is an upward seepage of water in the vertical direction through the same soil layer (Figure 8.4), the effective force on an area A at a depth z can be given by

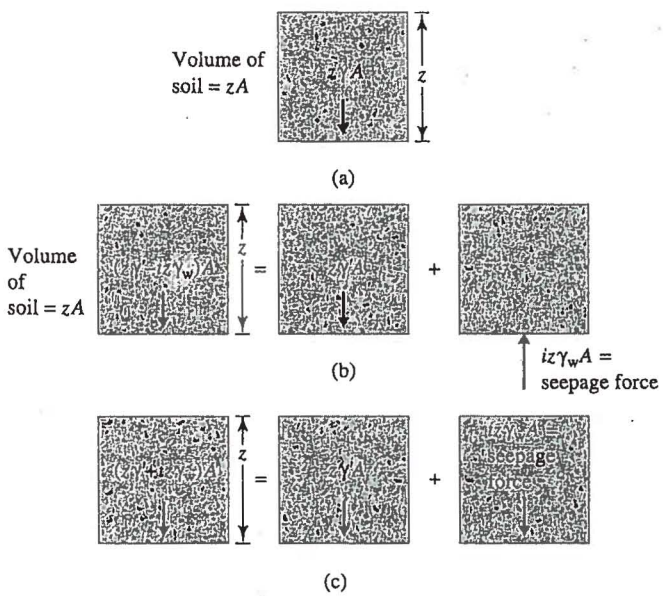
$$P_2' = (z\gamma' - iz\gamma_w)A$$

Hence, the decrease in the total force because of seepage is

$$P_1' - P_2' = iz\gamma_w A \tag{8.10}$$

The volume of the soil contributing to the effective force equals zA, so the seepage force per unit volume of soil is

$$\frac{P_1' - P_2'}{(\text{Volume of soil})} = \frac{iz\gamma_w A}{zA} = i\gamma_w \tag{8.11}$$



The force per unit volume, $i\gamma_w$, f
is, in the direction of flow. This upwa
larly, for downward seepage, it can be
direction per unit volume of soil is $i\gamma_w$

From the preceding discussions
unit volume of soil is equal to $i\gamma_w$, anc
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nets can be used to find the hydraulic
force per unit volume of soil.

This concept of seepage force c
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model tests, Terzaghi (1922) conclud
tance of $D/2$ from the sheet piles (wh
piles into the permeable layer). There
in a zone measuring D by $D/2$ in cross

The factor of safety against heav

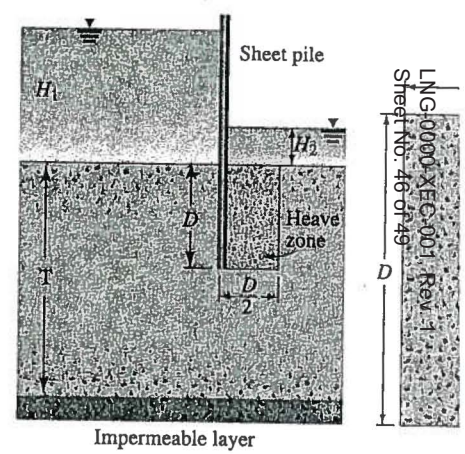
F

where FS = factor of safety
W' = submerged weight of soil
sheet pile = $D(D/2)(\gamma_{sat}$
U = uplifting force caused by

From Eq. (8.11),

U = (Soil volum

where i_{av} = average hydraulic gradien



(a)

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is

$$P_1' = z\gamma' A$$

P_1' is shown in Figure 8.7a.)
ıpward seepage of water in the vertical direction through
8.4), the effective force on an area A at a depth z can be

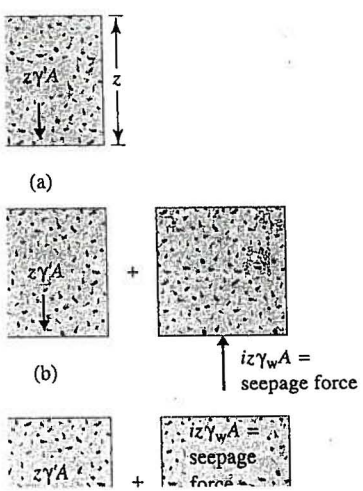
$$P_2' = (z\gamma' - iz\gamma_w)A$$

total force because of seepage is

$$P_1' - P_2' = iz\gamma_w A \tag{8.10}$$

oil contributing to the effective force equals zA, so the
ıme of soil is

$$\frac{P_1' - P_2'}{\text{Volume of soil)}} = \frac{iz\gamma_w A}{zA} = i\gamma_w \tag{8.11}$$



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The force per unit volume, $i\gamma_w$, for this case acts in the upward direction — that is, in the direction of flow. This upward force is demonstrated in Figure 8.7b. Similarly, for downward seepage, it can be shown that the seepage force in the downward direction per unit volume of soil is $i\gamma_w$ (Figure 8.7c).

From the preceding discussions, we can conclude that the seepage force per unit volume of soil is equal to $i\gamma_w$, and in isotropic soils the force acts in the same direction as the direction of flow. This statement is true for flow in any direction. Flow nets can be used to find the hydraulic gradient at any point and, thus, the seepage force per unit volume of soil.

This concept of seepage force can be effectively used to obtain the factor of safety against heave on the downstream side of a hydraulic structure. To see this, consider the case of flow around a sheet pile (Figure 8.8a). After conducting several model tests, Terzaghi (1922) concluded that heaving generally occurs within a distance of $D/2$ from the sheet piles (when D equals the depth of embedment of sheet piles into the permeable layer). Therefore, we need to investigate the stability of soil in a zone measuring D by $D/2$ in cross section, as shown in Figure 8.8a.

The factor of safety against heaving can be given by

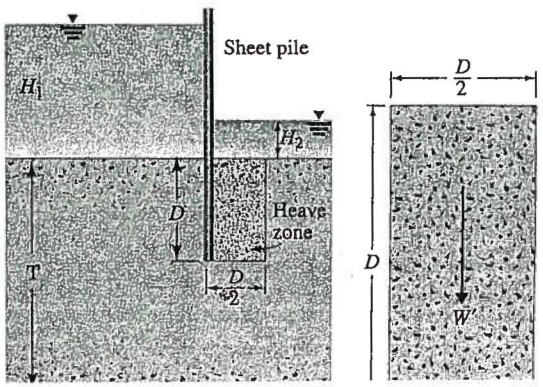
$$FS = \frac{W'}{U} \tag{8.12}$$

where FS = factor of safety
W' = submerged weight of soil in the heave zone per unit length of sheet pile $= D(D/2)(\gamma_{sat} - \gamma_w) = (\frac{1}{2})D^2\gamma'$
U = uplifting force caused by seepage on the same volume of soil

From Eq. (8.11),

$$U = (\text{Soil volume}) \times (i_{av}\gamma_w) = \tfrac{1}{2}D^2 i_{av}\gamma_w$$

where i_{av} = average hydraulic gradient at the bottom of the block of soil.



Substituting the values of W' and U in Eq. (8.12), we can write

$$FS = \frac{\gamma'}{i_{av}\gamma_w} \tag{8.13}$$

For the case of *flow around a sheet pile* in a *homogeneous soil,* as shown in Figure 8.8, it can be demonstrated that

$$\frac{U}{0.5\gamma_w D(H_1 - H_2)} = C_o$$

where C_o is a function of D/T (see Table 8.1). Hence, from Eq. (8.12),

$$FS = \frac{W'}{U} = \frac{0.5D^2\gamma'}{0.5C_o\gamma_w D(H_1 - H_2)} = \frac{D\gamma'}{C_o\gamma_w D(H_1 - H_2)} \tag{8.13a}$$

Table 8.1 Variation of C_o with D/T

D/T	C_o
0.1	0.385
0.2	0.365
0.3	0.359
0.4	0.353
0.5	0.347
0.6	0.339
0.7	0.327
0.8	0.309
0.9	0.274

Example 8.3

Consider the upward flow of water through a layer of sand in a tank as shown in Figure 8.9. For the sand, the following are given: void ratio (e) = 0.52 and specific gravity of solids = 2.67.

a. Calculate the total stress, pore water pressure, and effective stress at points A and B.

b. What is the upward seepage force per unit volume of soil?

Solution

a. The saturated unit weight of sand is calculated as follows:

$$\gamma_{sat} = \frac{(G_s + e)\gamma_w}{1 + e} = \frac{(2.67 + 0.52)9.81}{1 + 0.52} = 20.59 \text{ kN/m}^3$$

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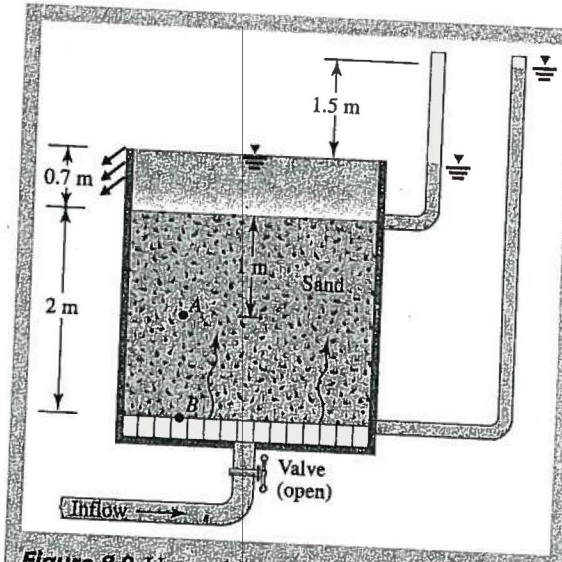

Figure 8.9 Upward flow of water through a layer of sand in a

Now, the following table can be prepared:

Point	Total stress, σ (kN/m²)	Pore water pre (kN/m²
A	$0.7\gamma_w + 1\gamma_{sat} = (0.7)(9.81) + (1)(20.59) = \mathbf{27.46}$	$(1 + 0.7) + \left(\frac{1}{}\right.$ $= (2.45)(9.81) =$
B	$0.7\gamma_w + 2\gamma_{sat} = (0.7)(9.81) + (2)(20.59) = \mathbf{48.05}$	$(2 + 0.7 + 1.5)\gamma$ $= (4.2)(9.81) =$

b. Hydraulic gradient (i) = 1.5/2 = 0.75. Thus, the se volume can be calculated as

$$i\gamma_w = (0.75)(9.81) = \mathbf{7.36\ kN/m^3}$$

Example 8.4

Figure 8.10 shows the flow net for seepage of water arou piles driven into a permeable layer. Calculate the factor stream heave, given that γ_{sat} for the permeable layer = 17 ness of permeable layer T = 18 m)

Solution

From the dimensions given in Figure 8.10, the soil pris

TECHNICAL PUBLICATION SJ 88-3

RAINFALL ANALYSIS FOR NORTHEAST FLORIDA

PART VI: 24-HOUR TO 96-HOUR MAXIMUM RAINFALL FOR RETURN PERIODS 10 YEARS, 25 YEARS, AND 100 YEARS

By

Donthamsetti V. Rao

Division of Engineering
Department of Water Resources
St. Johns River Water Management District
Palatka, Florida

May 1988

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