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HYDROGEOLOGIC STUDY OF FMPC DISCHARGE TO THE GREAT MIAMI RIVER

REPORT

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EXECUTIVE SUMMARY

As required by Order 14B of the Director's Orders and Findings, a hydrogeologic study of the Feed Materials Production Center (FMPC) discharge to the Great Miami River was performed to determine if the discharge from the FMPC effluent pipeline is located within the zone of influence of the production well field operated by the Southwestern Ohio Water Company (SOWC) or any other major production field, and to qualitatively and quantitatively determine any associated adverse environmental impact.

The scope of work for this study centered on analytical and numerical modeling studies of the surface water and ground water environments. Models were used to establish the approximate boundaries of the zone of influence of the SOWC wells, and to quantify the effects of three mixing processes on the impacts caused by the FMPC discharge. These processes included the mixing of the effluent discharge with background water in the Great Miami River, the mixing of induced infiltration from reaches of the river upstream from the FMPC discharge with that from impacted reaches downstream from the discharge, and the mixing of the ground water flow component originating from the river with the regional aquifer flow and other sources of recharge prior to reaching the SOWC wells.

An extensive review of available data and a limited field program provided both the input data base for the models and the calibration data used to test model performance. A sensitivity analysis was also completed to test the sensitivity of the model results and study conclusions to the assumed site conditions and parameter values. The results of the data review and modeling studies allow the following three general conclusions to be made:

1. The discharge from the FMPC effluent pipeline likely occurs within a reach of the Great Miami River that contributes flow, via induced infiltration, to the SOWC collector wells. The relative contribution of flow to the SOWC wells from the river downstream from the discharge is, however, a small fraction of the flow contributed from upstream reaches of the river. The sensitivity testing of the model indicates that the FMPC discharge could actually be outside of the capture zone of the SOWC wells if the river infiltration rate is greater than assumed.

2. Even if the FMPC discharge is within the zone of influence of the SOWC wells, the incremental impact of the effluent on the water quality of the pumped water lies within the range of variability of previous observations and is below analytical detection limits under average conditions. Therefore, no observable improvements in water quality would result in the SOWC wells from eliminating the effluent effects (e.g., by relocating the pipeline).

3. A mass balance of the sources of uranium observed in the SOWC collector wells cannot be performed at this time.

Based on the results of this study and the conclusions drawn therefrom, no further studies specifically addressing the impact of the FMPC discharge on the SOWC collector wells are recommended.

The SOWC collector wells, the river, and the regional ground water flow system assessed in this preliminary study remain of utmost importance to the overall issues being addressed in the sitewide Remedial Investigation/Feasibility Study (RI/FS) and other related studies. The results of the preliminary modeling studies have been of value in expanding the current understanding of these sitewide and regional issues. Additional investigations that will address the remaining uncertainties will be performed as part of the sitewide RI/FS.

1.0 INTRODUCTION

1.1 OBJECTIVE OF STUDY

As required by Order 14B of the Director's Orders and Findings, the objectives of the Hydrogeologic Study of the Feed Materials Production Center (FMPC) Discharge to the Great Miami River are to determine if the discharge from the FMPC effluent pipeline is located within the "zone of influence" of the production well field operated by the Southwestern Ohio Water Company (SOWC) or any other major production field, and to qualitatively and quantitatively determine any associated adverse environmental impact.

The site features of principal interest to this study are identified in Figure 1.1-1. The limits of the study area extend beyond the area shown in this figure since the modeling studies require consideration of regional-scale conditions to establish appropriate boundary conditions and to fully account for the capture zone of the SOWC wells.

1.2 SCOPE OF WORK

The scope of work for the hydrogeologic study of the FMPC discharge was developed around a series of analytical and field tasks to provide both the theoretical mechanisms for predicting the extent of influence and impacts, and an extended data base for model input and calibration. The following five tasks were proposed for completion over an approximate 12-month period, with an indication as to which have been completed to date.

Task 1 - Geologic/Hydrogeologic Description and Historical Overview

A description of the local geology and hydrogeology in the vicinity of the outfall pipe and collector wells was completed. This description was derived from available project and historic data. Cross sections were constructed for the local area.

Task 2 - Sensitivity Study

Numerical and/or analytical modeling was utilized to study contaminant dilution and dispersion in both the surface water and ground water environments, including the hydraulic interconnection between the collector wells and the Great Miami River. This computer modeling included an evaluation of the

surface water and hydrogeologic regime as well as sensitivity analysis to characterize the level of confidence in the results in relation to the study objectives.

Task 3 - Evaluation of River Sediments

The study also entailed an evaluation of river bed sediment properties to determine the mechanisms for transport of contaminant particles through the sediments. Approximately six grab samples of river bed sediment were taken for grain size analysis to evaluate stream bed hydraulic properties.

Task 4 - Water Quality Studies

An evaluation of water quality within the study area will utilize empirical, statistical, and trend analyses of sample data and will require a sufficient time frame (6 to 12 months) to allow for seasonal fluctuations. Surface water and ground water samples will be taken monthly for a period of six months to a year.

Task 5 - Piezometric Studies

An aquifer water level elevation study was completed for the area surrounding the SOWC well field. Monthly water level measurements made over a period of 12 months provided the data base. The study incorporated available SOWC data and pertinent Miami Conservancy District (MCD) and Westinghouse Materials Company of Ohio (WMCO) data.

The results of this hydrogeologic study were based directly on the completion of Task 2. The data review completed in Task 1 provided the hydrologic and geologic settings that formed the basis of the input data base to the models, and also proved useful in model calibration. Historic data were also used to satisfy the intent of Task 5, and provided the primary model calibration data and illustrated the long-term stability of ground water flow patterns. Whereas the first round of field sampling for Tasks 3 and 4 was completed, the laboratory data were not available for use in this report. Field observations were used to confirm model assumptions, however. The continuation of Tasks 3 and 4 is not considered necessary to support the conclusions of this study however, a variation of the field program will be performed under the sitewide RI/FS and the results will be evaluated in terms of any potential effects on the conclusions of this study.

2.0 PROJECT SETTING: REVIEW OF EXISTING INFORMATION

2.1 GEOLOGIC AND HYDROGEOLOGIC SETTING

The geology of the area surrounding the oxbow meander of the Great Miami River near Ross (Venice), Ohio has been described by Fenneman (1916), Durrell (1961), and Spieker (1968a). The bedrock consists of predominantly flat-lying Ordovician shales with thin, interbedded layers of limestone. This bedrock unit, which is part of the Cincinnatian Series, reaches a total thickness of approximately 800 feet. Unconformably overlying the Cincinnatian Series are approximately 150 feet of Pleistocene glacial valley fill deposits. Ohio was covered by continental ice sheets four times during the Pleistocene epoch. During an interglacial period prior to the Illinoisan and Wisconsin glaciation events, the ancestral Great Miami River became entrenched in its present bedrock valley to depths of 200 feet. As indicated in the hydrogeologic cross sections (Figures 2.1-1 through 2.1-10), the buried valley is about one-and-one-half miles wide and is U-shaped, having a broad, relatively flat bottom and steep valley walls.

During the subsequent Illinoisan and Wisconsin glaciations, the valley of the Great Miami River was filled with glaciofluvial sand and gravel deposited by the meltwaters of the younger ice sheets. Interbedded glacial till deposits occur within the outwash deposits, but are in most cases of limited lateral extent. The till deposits are composed primarily of poorly sorted pebbles, cobbles, and boulders in a predominantly clay matrix. Within the study area till deposits of Illinoisan and Wisconsin Age overlie the bedrock uplands where they form the thick unconsolidated sediment layers beneath the soil zone. The western part of the FMPC is built on an abandoned trough of the ancestral Ohio River, also known as the New Haven Trough. In this trough, the sand and gravel aquifer is overlain by up to 90 feet of glacial till (Figures 2.1-2 through 2.1-4).

As a result of Pleistocene events, the relatively impermeable bedrock has been incised by ancestral rivers and filled with permeable, well-sorted sand and gravel. Interstratified till has produced confining layers of lower permeability in many areas. Watkins and Spieker (1971) performed extensive seismic refraction surveys to determine the thickness and extent of water-

bearing sand and gravel deposits filling the bedrock valley. Test drilling was used in conjunction with the refraction surveys to verify the accuracy of the seismic determinations of the depth to the valley floor. The top of bedrock map (Figure 2.1-11) was derived from the top of bedrock map produced by Watkins and Spieker (1971) and additional information provided by Leow (1985), and Vormelker (1985). Figures 2.1-2 through 2.1-10 depict geologic conditions throughout the model area including the top of bedrock. Figures 2.1-2, 2.1-9, and 2.1-10 more specifically categorize the local geologic conditions around the oxbow meander studied in this report.

Large ground water supplies occur in the outwash deposits and are recharged by three principal sources. Although the shales and limestones have a low permeability, small amounts of water do occur in erratically distributed joints and cracks producing seepage into the glacial deposits. The permeability of the bedrock has been estimated by Dove (1961) to be five gallons per day (gpd) per square foot. Recharge by precipitation amounts to approximately 570,000 gpd per square mile of catchment area (Dove 1961). Finally, recharge of the valley train deposits by infiltration of the Great Miami River occurs. Under natural conditions, the gradient of flow is from the aquifer to the river except during dry periods when the gradient is reversed. In recent years, increased aquifer usage has resulted in induced infiltration from the river. This will be more fully described in Section 2.2.

Regional hydrogeologic environments of the buried channel aquifer have been investigated and reported extensively by the U.S. Geological Survey (USGS). A hydrogeologic environment describes a portion of an aquifer possessing hydrologic and geologic properties that differ from the properties of aquifers in adjacent areas. Five major hydrogeologic environments in the Great Miami River Valley have been identified and mapped by Spieker (1968a) (Figure 2.1-12). Of the five hydrogeologic environments in the Great Miami River Valley, the Type I Hydrogeologic Environment most closely describes the hydrogeologic conditions in the vicinity of the oxbow meander.

The Type I Hydrogeologic Environment is found along the floodplain of the Great Miami River to the south and east of the FMPC facility. The lithology

of the aquifer consists principally of sand and gravel. Scattered lenses of clay or fine-grained material may exist anywhere in the environment; however, these lenses are not of sufficient thickness or areal extent to act as semiconfining layers or otherwise affect ground water movement. The potential for induced stream infiltration exists in these areas. Transmissivity generally ranges from 300,000 to 500,000 gpd per foot [40,000 to 67,000 square feet per day (feet^2/day)] and the Type I aquifer may be classified with a storage coefficient (S) of about 0.2. Individual wells can yield as much as 3,000 gallons per minute (gpm) (Spieker 1968a).

Other hydrogeologic environments shown in the Figure 2.1-12 that are taken into consideration in this modeling study include the following:

- Type II: Sand and Gravel Aquifer - This consists of Types II-A-1, II-A-2, II-B-1, and II-B-2 (Spieker 1968a); only Type II-A-2 has been determined to exist in the study area. This environment is less than 150 feet thick and recharge by induced stream infiltration is not available.

- Type III: Sand and Gravel Aquifer Overlain by Clay - The potential for induced stream infiltration does not exist. The transmissivity and storage properties are highly variable.

- Type V: Shale and Limestone Bedrock Overlain by Till - Relatively impermeable shale and limestone bedrock. Small water supplies are available in overlying till and bedrock fracture zones.

The Type II Hydrogeologic Environment is characterized by 150 to 200 or more feet of sands and gravels with no areally extensive interstratified clay layers present. Recharge by induced stream infiltration is not available. The coefficient of storage is about 0.2. Large ground water supplies are not generally available from the Type II aquifer due to its limited areal extent and proximity to bedrock valley walls.

The Type III Hydrogeologic Environment is characterized by 50 feet or more of clayey till overlying the main buried channel aquifer. In the region of the FMPC, the buried channel aquifer is divided into an upper and lower part by a semipervious clay layer approximately 10 to 20 feet thick occurring approximately 140 feet below land surface. Hence, the lower aquifer is

classed as a semiconfined or leaky confined aquifer. Spieker and Norris (1962) have estimated a coefficient of storage of 0.001 for the lower sand and gravel aquifer. Spieker (1968a) estimated a transmissivity range of 35,000 to 300,000 gpd per foot (4,700 to 40,000 feet^2/day).

The Type V Hydrogeologic Environment includes all of the area outside of the buried channel. These areas are uplands which consist of shale with interbedded limestone overlain by 50 feet or less of clay-rich till. Large quantities of ground water are not generally transported through this material. Well yields vary widely, generally ranging from zero to ten gpm. However, sand and gravel lenses are erratically distributed throughout this material and, in some cases, wells completed in these units may yield up to 50 gpm.

The Type IV Hydrogeologic Environment was not considered in this study. It is typified by valleys filled largely with clay. No examples of this environment are found within the study area.

Ground water contour maps were constructed from data supplied by WMCO and MCD. These maps, which are presented in Figures 2.1-13 through 2.1-25, represent ground water flow patterns for April 1986 and on a monthly basis from September 1986 until August 1987, inclusive. Figure 2.1-13 for April 1986 incorporates additional data from a larger area within the buried channel aquifer. This data was used as the basis for calibration of the steady state condition for the ground water zone of influence modeling study. Table 2.1.1 lists details on wells and the selection rationale for all the wells used to construct this map.

Several conclusions can be drawn from these figures. The ground water flows from the west, north, and east towards the center of the study area and ground water exits the study area by flowing southwest through the arm of the buried channel aquifer near New Baltimore. The SOWC pumping wells produce a pronounced and persistent cone of depression in the water table centered on the pumping wells. The pumping draws the water table down to approximately 10 to 15 feet below projected prepumping water levels. Bedrock geometry plays an important part in the shape of the cone of depression which extends more in the west-east direction than the north-south direction. At the FMPC site,

water flows to the east from north of the site and to the south in the southwestern portion of the site. The implications here are that water traveling through the northern part of the FMPC site would move towards the SOWC wells and water traveling through the southern part of the FMPC site would move south towards Fernald.

All of the other ground water contour maps (2.1-14 through 2.1-25) show the same general ground water flow pattern, where data is available. They exhibit only relatively minor fluctuations in ground water elevations, probably due to seasonal variations and a variable pumping schedule of the SOWC wells. The persistence of a relatively stable cone of depression was used to justify the use of steady state analysis in the ground water modeling described in Section 3.2 and Appendix A.

Hydrographs of selected wells were plotted against the river hydrograph at the Ross Bridge. These hydrographs are presented in Figures 2.1-26 through 2.1-29. Figure 2.1-30 shows the river stage at Ross compared with that at Hamilton. Table 2.1.2 shows the maximum, minimum, and mean ground water elevations for January 1, 1986 through September 1, 1987 for the wells shown in the hydrographs (data was supplied by MCD). The monitoring wells and collector wells show a broad cyclic trend on a yearly basis. High ground water elevations occur in the late fall and early winter months (November, December, and January). Low ground water elevations occur during the late summer months (August, September, and October).

Figure 2.1-26 shows hydrographs from the two active SOWC collector wells plotted against monthly river elevations measured at Ross Bridge. Neither collector well hydrograph shows regular seasonal variation or follows changes in the river stage. Influences on the ground water levels include infiltration, river stage, and SOWC wells pumping schedule. No information was available on the latter, but this factor is most likely to have the primary influence on ground water levels and thereby account for the hydrograph pattern.

Rivers which fully penetrate aquifers are generally considered to be sources of recharge to pumping wells whose cone of influence intersects the river

Rivers which fully penetrate aquifers are generally considered to be sources of recharge to pumping wells whose cone of influence intersects the river bank. The Great Miami River is a partially penetrating river and, therefore, while the river will contribute to flow at the pumping wells, a component of flow will also come from the aquifer on the opposite side of the river. This is evidenced by the contour maps and the hydrographs. The contour maps show cones of depression around the pumping wells extending past the river. This is more clearly shown by studying the hydrographs for Collector Well 2 (Figure 2.1-26) and Monitoring Wells S-2 and SW-1 (Figure 2.1-27). Collector Well 2 is located northwest of the river. Monitoring Well S-2 is on the same side of the river as Collector Well 2 but to the west of the oxbow. Monitoring Well SW-1 is located southeast of the river, on the opposite side of the river as Collector Well 2 (inside the oxbow) as seen in Figure 2.1-1. The fluctuations in water levels shown by these monitoring wells are synchronized with Collector Well 2 from October to December 1986.

Away from the influence of the SOWC pumping wells, the monitoring wells close to the river exhibit fluctuations in ground water level which are related to changes in the river level. An example is Monitoring Well BU-13, which is shown in Figure 2.1-29.

2.2 HYDROLOGY OF GREAT MIAMI RIVER

The Great Miami River is the receiving stream for the FMPC effluent discharge, and represents the main surface water feature in the vicinity of the FMPC. It flows to the southwest and has a drainage area of 3,630 square miles at the Hamilton gage, which is located approximately 10 miles upstream from the FMPC discharge pipe. The river in the area of interest exhibits meandering patterns that result in sharp directional changes over distances of less than 3,000 feet (Figure 2.2-1). Directly east of the FMPC, and within the immediate study area, the river passes through a 180-degree curve known as "Big Bend." A 90-degree bend in the river also occurs near New Baltimore, approximately two miles downstream from the FMPC point of discharge.

The average discharge of the river at Hamilton, based on 55 years of records, is 3,305 cubic feet per second (ft^3/sec). Using drainage area scaling, the corresponding average flow at the FMPC point of discharge has been estimated

to be 3,534 ft^3/sec. The maximum discharge ever recorded on the Great Miami River at Hamilton occurred on March 26, 1913, and was estimated to be 352,000 ft^3/sec. The maximum discharge since the construction of five retarding basins in 1922 was 108,000 ft^3/sec and occurred on January 21, 1959. The minimum daily discharge of 155 ft^3/sec was recorded on September 27, 1941. This value is approximately half of the 7-day, 10-year low flow value (Q_{7-10}) of 267 ft^3/sec, computed by the U.S. Geological Survey for the Hamilton Gage (MCD).

Figure 2.1-30 presents the continuous record of river stage at the Hamilton gage for 1985 and 1986. The stage is shown to fluctuate by only a few feet over most of the year periodically increasing up to approximately 12 feet above normal flow conditions. Also shown in Figure 2.1-30 are flow values recorded at the Ross Bridge during WMCO's routine monitoring program. The general pattern of variability is similar to that observed at the Hamilton gage.

The hydrologic characteristics of the Great Miami River throughout the area of interest were investigated using the HEC-2 computer model developed by the Hydrologic Engineering Center of the U.S. Army Corps of Engineers. The program was developed for the calculation of water surface profiles and related hydrologic parameters such as flow velocity for steady, gradually varied flow in natural or manmade channels. The effects of various obstructions such as bridges, culverts, weirs, and structures in the floodplain can be considered in the computations. The computational procedure is based on the solution of the one-dimensional energy equation, with energy loss due to friction evaluated using Manning's equation.

The input data for the HEC-2 program includes flow regimes, discharges, cross-sectional geometry, starting water surface elevation, reach lengths, and energy loss coefficients. All input data for the Great Miami River was obtained from the U.S. Army Corps of Engineers, Louisville District. Data was provided on 21 sections in a reach extending over five miles from a point downstream from the City of New Baltimore to a point just upstream of the Ross Bridge (Figure 2.2-1). Of particular interest are five sections spanning a river distance of approximately 1.5 miles near the FMPC discharge, with two

river Sections I and J located upstream from the discharge and three river Sections F, G, and H located downstream. A starting water surface elevation of 455.0 feet [mean sea level (MSL)] at the confluence of the Ohio River and the Great Miami River was used to initiate the program. This river surface elevation value was taken from the local USGS topographic map and represents average flow conditions in the Ohio River.

The hydrologic parameters used for purposes of this study were based on the HEC-2 model results for the average flow rate of 3,534 ft^3/sec. These results are summarized in Table 2.2.1. Representative values for water depth, velocity, channel width, and hydraulic radius were computed using a weighted average of the values for the aforementioned five river sections (F-J). The weighting was used to account for the differences in length of the respective sections being represented by the values. The following values were obtained for the average flow case:

Water Depth = 5.5 feet
Flow Velocity = 2.0 feet per second (ft/sec)
Channel Width = 346 feet
Hydraulic Radius = 5.1 feet

These particular parameters are the basis for subsequent modeling efforts, as described in Chapter 3.0.

As a check of the representativeness of these values, each was compared to the following average values for all 21 sections along the Great Miami River:

Water Depth = 5.7 feet
Flow Velocity = 2.3 ft/sec
Channel Width = 302 feet
Hydraulic Radius = 5.6 feet

This comparison of values indicates that the hydrologic characteristics of the river in the vicinity of the FMPC discharge point are similar to overall river conditions. Consequently, the use of the weighted average values for the five local stations is justified for subsequent modeling studies.

Sand and gravel deposits of the buried valley aquifer underlie much of the Great Miami River bed. The ground water within these deposits is hydraulically connected with the river (Dove 1961 and Spieker 1968a). The natural ground water gradient flows from the aquifer to the river, with ground water discharging into the river. In areas of excessive ground water pumping from the sand and gravel aquifers near the river, a reversal of the natural hydraulic gradient can occur. When the pumping of wells near the river is great enough for the cone of depression to intersect the river, the hydraulic gradient is reversed and infiltration from the river into the aquifer is induced. The sand and gravel aquifer has very little or no interstratified retarding clay layers in the area of the FMPC discharge. Pumping of the SOWC well field is sufficiently close to the river to permit induced recharge by stream infiltration.

The rate of such recharge by stream infiltration varies widely with respect to both place and time. Factors which influence the recharge rate include fluctuating river levels, different hydraulic gradients, and changing stream bed conditions caused by silting and dredging. Seasonal changes in water temperature can also affect the infiltration rate since the viscosity of the water varies with temperature.

Two studies of river bed infiltration rate were conducted during the summers of 1956 near Ross (Dove 1961) and 1962 in Fairfield Township (Spieker 1968a). Infiltration rates were calculated to be 240,000 and 492,000 gpd per acre of stream bed, respectively. Both tests were performed in similar terrains under low streamflow conditions at water temperatures of approximately 80 degrees Fahrenheit.

2.4 FMPC MAIN EFFLUENT LINE

The FMPC discharge pipeline to the Great Miami River represents a permitted discharge of wastewater from the FMPC to the surface waters of the State. The discharge is regulated by an National Pollutant Discharge Elimination System (NPDES) permit, with compliance monitoring performed at Manhole 175 before the effluent leaves the site boundary. The wastewater conveyed by the main effluent line comes from three principal sources:

- Treated effluent from the general sump via the

- Treated effluent from the general sump via the biodenitrification surge lagoon (previously via the clear well)
- Treated effluent from the sanitary wastewater treatment plant
- Storm water runoff, principally from the production area.

The effluent line, which is a 4,650-foot long, 16-inch diameter cast-iron pipe, was constructed in 1952. Seven concrete manholes are located along the line for access and maintenance purposes. The depth of burial of the pipeline ranges from approximately 4 to 16 feet, with a maximum and minimum slope of 12.9 and 0.1 percent, respectively. The invert of the concrete-encased submerged discharge is located near the bottom of the Great Miami River, approximately 15 inches below the lowest recorded water level at the discharge point. River flow in this reach, which is within the 180-degree "Big Bend," has been observed to be turbulent.

In 1986, the average rate of discharge from the pipeline was 0.5 million gallons per day (MGD) or 0.78 ft^3/sec. The maximum discharge rate observed in 1986 was 1.1 MGD (1.7 ft^3/sec), and the minimum flow rate was 0.2 MGD (0.31 ft^3/sec). The NPDES permit requires monitoring for flow rate, pH, suspended solids, ammonia, oil and grease, residual chlorine, and nitrate. A summary of NPDES data is presented in Table 2.4.1. The Department of Energy (DOE) also requires daily sampling for radionuclides, with the daily samples being composited on a weekly basis for laboratory analysis. Table 2.4.2 presents average annual concentrations of monitored radionuclide parameters at Manhole 175. The concentrations at the upstream and closest downstream locations are also shown.

The radiological constituent to be investigated in this study is total uranium. Based on the weekly composites, the average concentration of total uranium in the FMPC effluent discharge in 1986 was found to be 450 picocurries per liter (pCi/ℓ). This was less than the average value of 661 pCi/ℓ

measured in 1985 (Westinghouse Material Company of Ohio 1986 and 1987). The selection of total uranium as the parameter of concern to this study was based on the following three reasons:

- Total uranium has represented the key parameter of study in most previous investigations and monitoring programs at the FMPC site. Consequently, the available data base for total uranium far exceeds that for other radionuclides.

- With the exception of Technetium-99, the average concentration of uranium in the FMPC effluent greatly exceeds that of other radionuclides. The concentration of total uranium also represents the largest percentage (82 percent) of the allowable standard among the radionuclides, indicating its foremost importance when investigating potential impacts. For comparison, the average Technetium-99 concentration represents only 2.2 percent of the standard (Westinghouse Material Company of Ohio 1987).

- The solubility and attenuation properties of total uranium are representative of those of other radionuclides; thus, the selection of total uranium for transport and impact evaluation would not be deficient in terms of neglecting species with exceptionally higher mobility.

The potential need to consider other radionuclides or NPDES constituents in future impact studies is addressed in Chapter 4.0.

2.5 DESCRIPTION OF SOWC WELL FIELD AND OPERATION

In 1952, the SOWC installed a large-diameter radial collector well in the sand and gravel glacial outwash deposits east of the Great Miami River near Ross, Ohio. A generalized schematic of a collector well is shown in Figure 2.5-1. The collector well was pumped at an average rate of 10 MGD from 1952 to 1955. Its effective radius, as shown in Figure 2.5-2, is approximately 200 feet. In 1955, a second collector well was installed with an effective radius of 212 feet to establish an adequate water supply for 13 industries in the Mill Creek Valley area. Historical data from the 1950s indicate that the average pumping rate for the SOWC was about 14 to 15 MGD after completion of the second well. From 1980 through 1986, this pumping rate has increased to about 18.4 MGD (Miami Conservancy District, 1987). Spieker (1968b) and Dove (1961) concluded that from 60 to 76 percent of the total flow from the

collector wells comes from induced recharge from the river (values are representative for ranges of historical pumping rates).

Water which is pumped from Collector Wells 1 and 2 is piped about 24 miles through a 36-inch-diameter main to a reservoir in the Mill Creek Valley. When the water enters the valley, it flows by gravity to the industries served by the SOWC system. In 1986, a third collector well was installed for emergency use only. Due to the standby status of Collector Well 3, the total flow for 1987 from the three wells is not expected to exceed the current 18.4 MGD level. This level is expected to be maintained in the near future (MCD).

Additional significant pumping centers located within the model study area are the FMPC and the Albright and Wilson (AW) production wells. Their estimated average pumping rates are 0.5 and 0.14 MGD, respectively. The AW well is located about 2,000 feet south of FMPC along Paddy's Run Road.

3.0 MODELING STUDIES

3.1 GREAT MIAMI RIVER WATER QUALITY MODEL

The objective of the modeling studies was to provide qualitative and quantitative information on any environmental impacts of the FMPC discharge on the SOWC collector wells. These studies necessarily focus on subsurface flow and solute transport models to determine if contaminants can reach the wells, and if so, at what concentration. However, any effluent from the FMPC discharge pipeline potentially reaching the SOWC wells first passes through the Great Miami River system. The input data to the ground water solute transport analysis must, therefore, account for the intermediate mixing of the effluent with the background flow in the river. The Great Miami River water quality model was used to quantify the resultant dilution and the profile of uranium concentration in the river for use in the ground water modeling studies.

The zone of influence of the SOWC wells could potentially span that portion of the river receiving effluent discharge from the pipeline; therefore, it is necessary to evaluate the near-field concentration profile near the point of discharge. The preferential modeling approach would be to use a hydrodynamic dispersion model that accounts for the progressive lateral and longitudinal spread of the contaminant plume away from the source. On the other hand, the fact that the discharge occurs near the channel bottom in a zone of turbulent flow and meandering curves would promote the potential for nearly complete mixing of the effluent across the channel. These conditions could allow the use of a complete mix model and its lesser data requirements.

The trade-off between the simplicity of a complete mix model and the additional resolution provided by a more data-intensive hydrodynamic dispersion model will be assessed by analyzing each as an extreme case. The complete mix model will provide a lower bound on local concentrations due to the necessary "averaging" of the effluent effects across the channel. The hydrodynamic dispersion model, on the other hand, will tend to overestimate actual concentrations since the initial mixing caused by the momentum of the discharge is being neglected. The results of the two models will be evaluated within the context of the ground water modeling needs to establish appropriate parameter values.

3.1.1 Complete Mix Model

The underlying assumption of the complete mix model is that the total mass flux of uranium in the effluent mixes completely and instantaneously with the background mass flux of uranium in the river. Since the mass flux can be represented by the product of flow rate and uranium concentration, the concentration in the river after mixing can be calculated as:

$$C_{river} = (Q_p C_{eff} + Q_r C_{back})/(Q_p + Q_r) \qquad (1)$$

in which:

C_{river} = Concentration of uranium in the river after mixing

Q_p = Rate of effluent discharge from the pipe

Q_r = Flow rate in the Great Miami River

C_{eff} = Concentration of uranium in the effluent

C_{back} = Background concentration of uranium in the river upstream from the point of discharge.

Two effects of the use of the complete mix model are the following:

- The concentration of uranium is taken to be constant across a given cross section (i.e., no lateral or vertical gradients are considered)
- If Q_r remains constant (i.e., any gains or losses in river flow are neglected), the uranium concentration does not change in the longitudinal (downstream) direction under steady-state conditions.

Average conditions can be evaluated by assigning the following average values, as reported in Chapter 2.0, to the parameters:

- Q_r = 3,534 cubic feet per second (cfs)
- Q_p = 0.78 cfs
- C_{eff} = 450 pCi/ℓ
- C_{back} = 1.2 pCi/ℓ.

The background concentration of 1.2 pCi/ℓ uranium in the river is the average of 52 water samples collected on a weekly basis in 1986 at the Ross Bridge.

Because this sampling location is approximately two miles east and upstream from the FMPC, it is considered beyond the influence of surface drainage and discharges from the site. Using Eq. (1), the resulting concentration of uranium in the river below the discharge is 1.3 pCi/ℓ. The impact of the effluent discharge on the river is observed to be an approximate 8 percent increase in uranium concentration over background. This small increase results from the extremely small percentage of total flow, and thus total mass flux, represented by the higher concentration effluent stream.

In evaluating the extreme range of impacts that can be expected in the river, the assumption was made that extreme low flows in the discharge pipe would not occur during extreme high flows in the river, and vice versa. The storm water runoff contribution to the effluent justifies this assumption. Consequently, the extreme cases can be represented by the following combinations of parameters reported in Chapter 2.0:

- Minimum impact:

  - $Q_r = Q_{ave} = 3{,}534$ cfs
    $Q_p = Q_{min} = 0.31$ cfs
    $C_{eff} = 450$ pCi/ℓ
    $C_{back} = 1.2$ pCi/ℓ
    $C_{mix} = 1.24$ pCi/ℓ (3.3 percent increase)

  - $Q_r = Q_{max} = 108{,}000$ cfs
    $Q_p = Q_{ave} = 0.78$ cfs
    $C_{eff} = 450$ pCi/ℓ
    $C_{back} = 1.2$ pCi/ℓ
    $C_{mix} = 1.203$ pCi/ℓ (0.25 percent increase)

- Maximum impact:

  - $Q_r = Q_{ave} = 3{,}534$ cfs
    $Q_p = Q_{max} = 1.7$ cfs
    $C_{eff} = 450$ pCi/ℓ
    $C_{back} = 1.2$ pCi/ℓ
    $C_{mix} = 1.42$ pCi/ℓ (18 percent increase)

- $Q_r = Q_{min} = 267$ cfs (7-day, 10-year low flow)
  $Q_p = Q_{ave} = 0.78$ cfs
  $C_{eff} = 450$ pCi/ℓ
  $C_{back} = 1.2$ pCi/ℓ
  $C_{mix} = 2.5$ pCi/ℓ (108 percent increase).

The minimum impact is insignificant and likely nondetectable. The maximum impact would occur under low flow conditions in the river. The 108 percent increase in uranium concentration is based on a seven-day, ten-year low flow condition, and therefore is an extreme, short duration impact that would be considerably dampened prior to reaching the SOWC wells.

The assumption of constant effluent and background river concentrations is a limiting factor used to represent a wide range of possible conditions in a simplified manner. However, the concomitant use of extreme pipeline and river flows is considered to more than offset any errors introduced by the use of constant concentrations. In particular, the dominant mass flux term is the background mass flux in the river. The use of extreme river flows, which vary by almost three orders of magnitude from 267 cfs to 108,000 cfs, more than compensates for any temporal changes in the 1.2 pCi/ℓ background concentration when computing extreme mass flux values. The range in river background values in the 52 samples from 1986 was only 0.81 pCi/ℓ to 3.0 pCi/ℓ.

3.1.2 Surface Flow Hydrodynamic Dispersion Model

The use of a hydrodynamic dispersion model more adequately accounts for the contaminant distribution in the near-field vicinity of the discharge pipe. The discharge itself is considered as a point source of uranium release to the river. Convective and dispersive transport relationships are used to account for the lateral and longitudinal distribution of uranium concentration with the river water.

3.1.2.1 Governing Equations

For turbulent flow in a prismatic open channel, the differential equation for hydrodynamic dispersion in the three-dimensional case is (Fisher, 1973, p. 60, Eq. 4; Li, 1983, p. 549):

$$\frac{\partial \bar{C}}{\partial t} + \bar{u}\frac{\partial \bar{C}}{\partial x} + \bar{v}\frac{\partial \bar{C}}{\partial y} + \bar{w}\frac{\partial \bar{C}}{\partial z} = \frac{\partial}{\partial x}\left(D_x \frac{\partial \bar{C}}{\partial x}\right) + \frac{\partial}{\partial y}\left(D_y \frac{\partial \bar{C}}{\partial y}\right) + \frac{\partial}{\partial z}\left(D_z \frac{\partial \bar{C}}{\partial z}\right) \quad (2)$$

in which $\bar{C}$ is the concentration; x, y, and z are the coordinates in the longitudinal, vertical, and lateral directions, respectively; $\bar{u}$, $\bar{v}$, and $\bar{w}$ are the velocities in the x, y, and z directions, respectively; D_x, D_y, and D_z are the dispersion coefficients in the x, y, and z directions, respectively; and t is the time. The velocity components $\bar{v}$ and $\bar{w}$ are those of the secondary convective currents in the plane of the cross section, with $\bar{w}$ representing lateral velocity and $\bar{v}$ representing vertical velocity. Figure 3.1-1 presents this modeling framework and the associated parameters.

The two-dimensional form of Eq. (2), which is based on the assumption of complete mixing in the vertical direction with a velocity field that is primarily undirectional such that the secondary convective currents can be neglected, is the following (Shen, 1978, page 36, Eq. 3):

$$\frac{\partial \tilde{C}}{\partial t} + V\frac{\partial \tilde{C}}{\partial x} = \bar{D}_x \frac{\partial^2 \tilde{C}}{\partial x^2} + \bar{D}_z \frac{\partial^2 \tilde{C}}{\partial z^2} \quad (3)$$

where $\tilde{C}$ is the average concentration and V is the constant velocity in the longitudinal direction. Although the transverse and vertical velocity components ($\bar{v}$, $\bar{w}$) have been dropped from Eq. (3), the associated convective effects remain accounted for in the empirically-based values of the longitudinal and lateral dispersion coefficients ($\bar{D}_x$, $\bar{D}_z$).

Many empirical formulas have been suggested for $\bar{D}_x$ and $\bar{D}_z$ in natural streams. For example, Liu (1977, as presented by Li, 1983, p. 550, Eq. 13-76) proposed:

$$\bar{D}_x \simeq 0.18\,(U_*/V)^{1.5}\,\frac{Q^2}{U_* R^3} \quad (4)$$

where Q is the discharge, R is the hydraulic radius, and U_* is the shear velocity given by the following equation (Li, 1983, p. 549):

$$U_* = (\tau_o/\rho)^{0.5} = (gHS_o)^{0.5} \tag{5}$$

in which τ_o is the shear stress, ρ is the mass per unit volume, g is the gravitational acceleration, S_o is the channel slope, and H is the depth of flow.

For the transverse dispersion coefficient,

$$D_z = 0.23HU_* = 0.23H\ (gHS_o)^{0.5} \tag{6}$$

has been suggested for relatively straight channels. Larger values should be used for meandering streams (Li, 1983, p. 550).

For average flow conditions, the average flow depth was reported in Section 2.2 to be 5.5 feet. The channel slope for the five river sections of interest was found from the HEC-2 input to be 0.0008741 feet/feet. Using these values, the shear velocity can be calculated from Eq. (5) to be:

$$\begin{aligned} U_* &= (32.2 \times 5.5 \times 0.0008741)^{0.5} \\ &= 0.393 \text{ feet per second (ft/sec)} \end{aligned}$$

Using the average velocity of 2.0 ft/sec and average hydraulic radius of 5.1 feet, the average longitudinal dispersion coefficient can be calculated from Eq. (4) to be:

$$\begin{aligned} \overline{D}_x &= 0.18\ \left(\frac{0.393}{2.0}\right)^{1.5} \frac{(3{,}534)^2}{(0.393)(5.1)^3} \\ &= 3{,}756 \text{ square feet per second (ft}^2\text{/sec)} \end{aligned}$$

The transverse dispersion coefficient [Eq. (6)] is:

$$\begin{aligned} \overline{D}_z &= (0.23)(5.5)(0.393) \\ &= 0.5 \text{ ft}^2\text{/sec} \end{aligned}$$

Fisher (1973, p. 73, Table 2) lists the results of experiments on longitudinal dispersion in laboratory channels, natural streams, and canals. Using the

values in this table for natural streams, the range of the longitudinal dispersion coefficient is:

$$\frac{\bar{D}_x}{HU_*} = 74 \text{ to } 7{,}500 \text{ ft}^2/\text{sec} \qquad (7)$$

With H = 5.5 ft and U_* = 0.393 ft/sec,

$$\bar{D}_x = 160.0 \text{ to } 16{,}211 \text{ ft}^2/\text{sec}$$

As can be seen, the value of 3,756 ft^2/sec computed above for the Great Miami River is well within this range.

Fisher (1973, p. 68) also deduced, from secondary velocity profiles, a corresponding range for transverse dispersion coefficients in a uniform open channel as:

$$\frac{\bar{D}_z}{HU_*} = 0.51 \text{ to } 2.4 \text{ ft}^2/\text{sec} \qquad (8)$$

With H = 5.5 ft and U_* = 0.393 ft/sec,

$$\bar{D}_z = 1.1 \text{ to } 5.2 \text{ ft}^2/\text{sec}$$

The computed value of 0.5 ft^2/sec for the Great Miami River is slightly below this range. The lower value would tend to underestimate the rate of mixing and dilution in the lateral directions, which is conservative since the hydrodynamic dispersion model is being used to establish an upper bound on the concentration profile.

3.1.2.2 Model Assumptions

The previous section described a widely used river dispersion model for predicting the change in concentration in both the lateral and longitudinal directions from a point source of contamination. Values representative of conditions in the Great Miami River were shown to be generally consistent with published results of field and laboratory studies. Before applying this model to the case of the FMPC effluent discharge, however, it is appropriate to consider the following principal assumptions underlying the model. Reference

is made to Figure 3.1-1 for a schematic representation of the river, as used in the model.

- The cross sections of the river can be represented by an average width and average depth that will remain unchanged in the longitudinal direction.

- The longitudinal velocity of the water at any section is constant across the section, and does not change from section to section in the longitudinal direction. Any potential dispersion of contaminants caused by lateral velocity gradients is accounted for in the dispersion coefficients.

- The width of the river ($D_1 + 2B + D_2$) is constant.

- The effluent discharging into the river has a constant concentration (C_{eff}). The significance of this assumption has been discussed in Section 3.1.1.

- The discharge point of the pipeline has a width of 2B and can be located at any place along the cross section.

- The banks of the river are impervious; i.e., the contaminants cannot migrate beyond these boundaries. This assumption is conservative since the loss of contaminants as a result of induced infiltration would reduce the actual concentration in the longitudinal direction.

- Along the z axis at the point of discharge, the incremental concentration caused by the effluent is zero at all locations except for the discharge width 2B.

- The radionuclides are transported only by convection due to the velocity of the flowing water and dispersion effects; the gravitational effects on the radionuclide particles are neglected, since complete dissolution is being assumed. Any uranium attached to suspended solids would not migrate to the SOWC wells; thus, this assumption makes all uranium available for release.

- The source is assumed to be vertically continuous throughout the water column at the point of discharge, as discussed in the next section.

- The secondary currents are neglected, including any entrainment mixing that would occur as the result of the momentum of the discharge. This will conservatively overestimate the concentration field.

3.1.2.3 Surface Flow Hydrodynamic Dispersion Model Application

A two-dimensional analytical model, called STRIP1B (IT, 1987), was used for the solution of the dispersion model, in accordance with Eq. (3). STRIP1B was developed by the staff of IT Corporation (IT) and has been extensively verified with the use of IT's other code called GEOFLOW (IT, 1986).

Eq. (3) is the governing partial differential equation of STRIP1B. Although STRIP1B was developed for solute transport in porous media, it can be used for the solution of the partial differential equation for surface flow solute transport using the appropriate dispersion coefficients. STRIP1B calculates the concentrations and convective-dispersive flux components from a time-dependent solute "strip" source in a uniform flow field bounded by two impervious boundaries. Either unsteady or steady-state conditions can be simulated. As can be seen from Figure 3.1-1, the source can be located at any place between the impervious boundaries.

The results from the STRIP1B model are given in terms of a normalized concentration, C/Co, where C is the concentration of uranium at any point (x,z) along the river resulting from the introduction of uranium at concentration Co at the FMPC effluent line. The resultant concentration C must be added to the background concentration in the river to yield the total uranium concentration. This can be mathematically represented as:

$$C_{river} = Co\ [(C/Co)_{model}] + C_{back} \qquad (9)$$

where:

C_{river} = Concentration of uranium at a given point (x,z) in the river downstream from the FMPC discharge (pCi/ℓ)

Co = Concentration of uranium from the effluent discharge at the point of initial mixing = 23.3 pCi/ℓ (see below)

$(C/Co)_{model}$ = Normalized concentration predicted by the model for the point (x,z)

C_{back} = Background concentration of uranium in the river = 1.2 pCi/ℓ

After substituting,

$$C_{river} = 23.3\ (C/Co)_{model} + 1.2 \qquad (10)$$

The value of Co was established by assuming that the effluent from the FMPC pipeline ($Qp = 0.78\ ft^3/sec$; $C_{eff} = 450\ pCi/\ell$) mixes instantaneously with only that portion of the river flow within a 1.4-foot vertical section. The width of this section corresponds to the diameter of the effluent pipeline. The mixing over the full depth of the river is not only consistent with the two-dimensional nature of the model, but field observations of "bubbling" at the water surface also suggest that the entrainment velocity of the effluent entering at the bottom of the river penetrates the full depth of flow. When the effluent is mixed with this "strip" of river flow, the resultant initial concentration Co becomes 23.3 pCi/ℓ.

The input to the dispersion model for the average flow case was developed from the values previously described, and includes the following (refer to Figure 3.1-1):

D_1 = 0.0 feet (i.e., discharge is assumed to occur at the west bank of the river)

D_2 = 346 feet (channel width) - 1.4 feet (mixing zone) = 344.6 feet

2B = 1.4 feet

$V = \bar{u} = 2.0$ ft/sec

$\bar{D}_x = 3{,}756\ ft^2/sec$

$\bar{D}_z = 0.5\ ft^2/sec$

The model results, in terms of both the normalized concentrations (C/Co_{model}) and the corresponding values of C_{river} from Eq. (10), are given in tabular form for selected river locations in Table 3.1.1 The value of z is the lateral distance from the west bank of the river, while the value of x is the longitudinal distance downstream from the point of discharge from the FMPC effluent pipeline.

The results in Table 3.1.1 indicate a rapid decrease in concentration in both the lateral and longitudinal directions away from the source. The decrease is more dramatic in the lateral direction since the uranium is both dispersing and mixing with a relatively high volumetric rate of river flow.

The general result of the dispersive mixing model is a gradual "smoothing" of the lateral concentration profile in the downstream direction. As the longitudinal coordinate (x) goes to infinity, the results of the dispersion model will match those of the complete mix model since the uranium in the effluent will have spread uniformly across the channel. Concentrations exceeding the complete mix value of 1.3 pCi/ℓ, as shown in Table 3.1.1, occur only near the west bank until large longitudinal distances are traversed. Even at a distance of x = 5,000 feet, the complete mix value is exceeded for only 80 feet in the lateral direction. By this point, the maximum uranium concentration of 1.85 pCi/ℓ at the west bank is within 50 percent of the background value.

It was noted in an earlier section that the lateral dispersion coefficient ($\bar{D}_z$) used in the model falls below published ranges of typical values. Any increase in the dispersion coefficient would increase the rate of lateral migration of contaminants, which would more rapidly lower the resultant concentrations as the result of mixing with additional river water. To illustrate this point, the dispersion model was rerun using a value of $\bar{D}_z$ = 5.2 ft^2/sec, which is the upper limit of the typical range given by Eq. (8). A much more rapid decline in uranium concentration was observed to occur in both the longitudinal and lateral directions. For example, at a distance of x = 500 feet, the predicted uranium concentration at the west bank is 2.4 pCi/ℓ, which is less than 50 percent of the value reported in Table 3.1.1 for $\bar{D}_z$ = 0.5 ft^2/sec. This value corresponds to an approximate threefold increase in the amount of mixing.

The importance of near-field mixing in the vicinity of the source can also be ascertained from the rapid decrease in concentrations away from the source. The assumption that no entrainment mixing of the discharge plume occurs outside of the 14-inch "strip" would, therefore, lead to predicted uranium concentrations that could greatly exceed actual concentrations. It is also likely that pumping of the large SOWC collector well located within the river bend to the east would

preferentially draw water from the eastern portions of the river channel. This would create secondary currents with an eastern component across the river which, in turn, would promote lateral mixing and dilution. These abnormally high secondary currents have likewise been neglected in the lateral mixing calculations, and again the uranium concentrations predicted by the hydrodynamic dispersion model would be expected to be overestimated.

3.1.3 Application of Results to Ground Water Model

The application of both a complete mix model and a hydrodynamic dispersion model for the Great Miami River had as a principal objective the development of a representative uranium source term for the ground water solute transport model. The results of the two modeling efforts indicate that a complete mix model provides an acceptable approximation of actual river conditions at distances greater than one mile downstream from the FMPC discharge. The corresponding concentration of uranium at all points in the river would, therefore, be set equal to 1.3 pCi/ℓ. Any potential errors introduced by this complete mix approximation will have little effect on the overall study results since the river reaches at distances exceeding one mile will not contribute significant flow to the SOWC wells.

For distances less than one mile, the actual concentration profile will fall between the complete, uniformly mixed case and the results of the dispersion model given in Table 3.1.1. An evaluation of the appropriateness of these two models must be accomplished within the context of the data requirements of the ground water model. Individual grid cells of the ground water modeling grid, for which input data are required on a one-to-one basis, have minimum dimensions of approximately 250 feet by 250 feet in the vicinity of the FMPC discharge to the river. As a comparison, the lengths of the dispersive mixing zones in the lateral direction are only on the order of tens of feet (Table 3.1.1). The entire mixing zone would, therefore, lie within a single grid cell, and to average the concentration profile over the cell to obtain a representative value would be the same as utilizing a complete mix model.

In the aforementioned case, if the entire width of the river was contained within a single cell, the average uranium concentration to be used as model input would have to equal the complete mix value of 1.3 pCi/ℓ in order to

conserve mass. To assume any larger concentration value throughout the cell, in order to at least partially account for the lateral concentration distribution, would be erroneous since the resultant total mass of uranium in the section would exceed the mass of uranium actually available. A straightforward and conceptually acceptable solution would be to assign the complete mix value of 1.3 pCi/ℓ to all river cells downstream from the FMPC outfall.

Two potential improvements to this approach have been considered. The first was to reduce the size of the ground water modeling grid to better accommodate the lateral concentration profile of uranium in the river. This modification would only be of value if the resultant plume to the SOWC wells would remain identifiable and not well-mixed with other ground water prior to reaching the wells at depth. This is highly unlikely, however, given the small percentage of flow originating in any single, small river cell. Further, to create a scenario in which the concentration differences are meaningfully different would require cells with lengths on the order of tens of feet. The additional complexities and levels of effort of modeling this situation cannot be justified by the technical requirements of this study. In the end, any potential changes in uranium concentration in the SOWC wells would likely remain within the error bars of the overall modeling study and no recognizable improvement would have been achieved.

A second possible modification would account for the model result that lateral mixing occurs only within the westernmost half of the river channel within the first mile. In those cases where the ground water model grid intersects the river, the entire mass of uranium from the FMPC effluent could be allocated to the modeling cell to the west. The corresponding uranium concentration in the western cell would be 1.4 pCi/ℓ, which is consistent with the scenario that only half of the river flow is completely mixing with the effluent. The concentration of uranium in the corresponding cell to the east would remain at the background value of 1.2 pCi/ℓ. Whereas this modification would appear to produce higher concentrations of uranium at the SOWC wells than the complete mix case, such would not be true if the eastern cell preferentially contributes more water to the wells. That is, if more water is yielded by the eastern cell, the total mass of uranium reaching the SOWC wells could be higher by maintaining a 1.3 pCi/ℓ value in both cells rather than allocating all uranium to the western

cell. Because initial ground water modeling results indicate that relatively more flow is, in fact, contributed by the eastern cells, a final decision was made to retain the complete mix value of 1.3 pCi/ℓ for all river sections.

In summary, the results of the surface flow hydrodynamic dispersion model indicate significant concentration gradients only at a length scale of tens of feet. Since any release to the subsurface environment from such small areas would represent an extremely low percentage of flow to the SOWC wells, the identity of such a plume of elevated concentration would be lost prior to reaching the wells. Consequently, the concentration gradients are of little importance to a quantitative evaluation of the impacts of the FMPC discharge on the SOWC wells. The use of a complete mix model to establish an average uranium concentration of 1.3 pCi/ℓ in the river is considered appropriate to satisfy the study objectives.

3.2 GROUND WATER MODEL

3.2.1 Model Selection

The selection of the most appropriate computer code for a given project is generally driven by the technical requirements and objectives of the study, the types and amount of data available and the accessibility of the code and its compatibility with available equipment. The situation for the Fernald site is no exception. However, in the case of Fernald, certain site features and study requirements have required the selection of a state-of-the-art computer code with advanced capabilities. The satisfaction of these minimum requirements eliminated most computer codes from further consideration in the selection process.

For reasons discussed in the following sections, the SWIFT III computer code was selected for use in both this hydrogeologic study of the FMPC discharge and the overall, site-wide Remedial Investigation and Feasibility Study (RI/FS). Use was also made of IT's GEOFLOW model to fully satisfy the initial requirements of the FMPC discharge study reported herein. The reasons for this dual modeling effort are explained in Section 3.2.1.3.

3.2.1.1 Selection Criteria

The ground water modeling component of the hydrogeologic study of the FMPC discharge could have been performed independently of other current and future efforts of the site-wide RI/FS. However, the need to avoid any redundancy of effort and to eventually achieve a consistency of results with the overall RI/FS dictated that the subject modeling study be performed in anticipation of future RI/FS modeling requirements. This programmatic approach is fully consistent with the RI/FS Work Plan, which calls for the performance of a preliminary modeling study to refine the planned field investigations. As such, the technical and operational criteria for model selection were developed in terms of the full set of current and future study requirements.

For purposes of both this study and the overall RI/FS, four minimum requirements of the modeling code were established. These included the following:

- Codes were only considered that had three-dimensional or quasi-three-dimensional modeling capabilities. This is necessary to account for vertical flow through varying types of geologic strata, and to simulate the effects of vertical hydraulic gradients caused by regional pumping at depth.

- The capacity to quantitatively predict contaminant concentration at receptor locations is considered necessary to fully satisfy the requirements of the RI/FS. Therefore, only codes with options to model solute transport and associated attenuation/retardation processes were considered.

- Only models that have been adequately verified and previously applied under similar project settings were considered. This criterion was necessary due to the sensitive nature of the Fernald work and the magnitude of future decisions that could be based on model predictions.

- The immediate availability of the modeling code and accompanying documentation within the public domain was necessary to satisfy near-term deliverables.

Available modeling codes were evaluated against these four criteria to establish a "short list" of codes for additional evaluation. Four codes were found to satisfy these criteria, consisting of SWIFT II/III, GEOFLOW, SWENT, and the Princeton Transport Code.

Numerous other selection criteria were also established to refine the selection process in terms of the specific site conditions and study requirements. These included:

- The capacity to model unconfined flow regimes is a preferred option in case unsaturated flow beneath the river or waste storage units is eventually found to be a critical process.

- Although the radionuclides of most concern do not require the consideration of daughter products, the capacity to model decay chains would become a consideration if other radionuclides are found to be important.

- The mathematical representation of attenuation/retardation processes (e.g., adsorption) and decay processes would provide flexibility in the range of constituents that can eventually be modeled.

- The capacity to handle a wide variety of boundary conditions is preferred so as not to limit the available options for best representing actual site conditions.

- Although not considered necessary, the option to consider density variations and temperature or concentration effects on fluid viscosity could be beneficial to best simulate certain critical processes (e.g., leakage through the river bed).

- The convenience of model application was considered based on features such as pre- and post-processing capabilities, user documentation, mesh generation, solution method, restart capability, applicability to available computer systems, and user familiarity.

3.2.1.2 Selection of the SWIFT III Code

Table 3.2.1 has been prepared to summarize these selection criteria for the four models that remained after the initial screening process. Following a critical evaluation of the relative benefits and deficiencies of the respective codes, a decision was made to select SWIFT III as the primary code for use in both the hydrogeologic investigation of the FMPC discharge area of influence and the site-wide RI/FS. SWIFT III most comprehensively satisfied the full set of selection criteria, and no significant limitations were identified.

A noteworthy benefit of the SWIFT III code is that the SWIFT series of models has been successfully applied on similar projects. Most notably, these applications have included two studies in or near the Fernald site area. These were an evaluation of ground water corrective action at the Chem-Dyne hazardous waste site at Hamilton, Ohio (Ward, D. S. et al., 1987a for the Ohio Environmental Protection Agency), and a preliminary characterization study of the ground water flow system near the FMPC (GeoTrans, 1985 for the Ohio Environmental Protection Agency), which incorporated the area of the FMPC discharge and SOWC wells.

One concern with SWIFT III is that the capabilities of the model, and thus the complexities to the user, may exceed what is necessary for the successful execution of the Fernald project. This concern is allayed by the long-term role of the model in the Fernald RI/FS; that is, the additional complexities will not be a burden to the long-term schedule. In addition, the same features that appear to be superfluous at this point in time may be found to be necessary as the project develops.

3.2.1.3 Dual Use of the GEOFLOW Model

The aforementioned operational complexities of the SWIFT III model, in addition to a lack of direct user experience with the SWIFT III code, created an uncertainty in relation to the near-term deliverables required as part of the FMPC discharge study. For this reason, a decision was made to perform a concurrent, parallel modeling study using IT's GEOFLOW model. The GEOFLOW code satisfies the four principal criteria for model selection, although the overall attributes of the model fall short of those of the SWIFT III code.

The immediate availability of the GEOFLOW code on IT's computer system, along with a high level of user familiarity due to IT's past extensive usage on similar projects, allowed the conceptualization, calibration, and sensitivity testing to proceed on an accelerated schedule as the work on the SWIFT III progressed at a more uncertain pace. Whereas both codes eventually produced a calibrated ground water flow model, as reported in later sections and the appendix, progress on the GEOFLOW model continued at the accelerated pace. Consequently, many of the interim findings reported herein represent GEOFLOW model results. The intent, however, is to proceed with the SWIFT III model

for the long-term application to the Fernald RI/FS due to its additional features and preferred status.

The parallel modeling efforts provide several additional advantages. The first is that the independently produced results from the two models can serve as a direct quality assurance check of model performance. The GEOFLOW code has been thoroughly tested against more routinely used and widely-accepted ground water flow models (e.g., the USGS' McDonald and Harbaugh model [McDonald and Harbaugh, 1984]). Consequently, a close matching of results from the GEOFLOW and SWIFT III models indirectly validates the SWIFT III code against standard flow models that did not satisfy the minimum criteria for use on the Fernald project. It should also be noted that the accelerated efforts on the GEOFLOW model greatly enhanced the calibration of the SWIFT III model and thus reduced the time and effort that would have been required if SWIFT III had been used alone.

3.2.2 Model Description

3.2.2.1 SWIFT III

The SWIFT model has been developed, maintained, and applied by Sandia National Laboratories. The U.S. Nuclear Regulatory Commission (NRC) sponsored this work under its high-level nuclear-waste program. Between 1982 and 1985, the capability of SWIFT was expanded to include fractured media, a free water surface, and extended boundary conditions (SWIFT II). Since 1985, GeoTrans has modified the code at their own expense. The newest code is designated as SWIFT III. The three models, SWIFT, SWIFT II, and SWIFT III, are fully transient (with steady state options), three-dimensional, finite-difference codes which solve the coupled equations for flow and transport in geologic media.

Listed below are the significant benchmark versions of the code used in the development of SWIFT III.

CODE	DEVELOPER	SOURCE OF FUNDING	REFERENCES
SWIP	Intercomp	USGS	Intercomp, 1976
SWIPR	Intera	USGS	Intera, 1979
SWIFT	Intera	NRC	Dillon et al., 1978 Cranwell and Reeves 1981 Finley and Reeves, 1982 Ward et al., 1984
SWIFT II	GeoTrans	NRC	Reeves et al., 1986a Reeves et al., 1986b Reeves et al., 1986c
SWIFT III	GeoTrans	GeoTrans	Ward, 1987

The following paragraphs summarize the evolution of each version.

SWIP (Survey Waste Injection Program) - 1976
Under funding by the USGS, Intercomp developed a three-dimensional model for assessing the effects of deep well injection of wastes into saline aquifers. The code is a hybrid of hydrologic and petroleum technology. Capabilities include the coupled solution of equations for ground water flow, heat, and brine transport. Additional features include variable density, viscosity, well bore friction, and heat loss. The documentation report contains several verification and validation problems.

SWIPR (Survey Waste Injection Program Revised) - 1979
Under contract to the USGS, Intera, a former division of Intercomp, extended the SWIP model to include a free water surface, adsorption, and decay for contaminant transport. Both SWIP and SWIPR are inactive; that is, updates have not been issued. Both codes are designed for Control Data mainframes with Fortran 4.

SWIFT (Sandia Waste-Isolation Flow and Transport) - 1981
Under contract to the NRC, Intera extended the code to simulate transport of chains of radionuclides. In contrast to the brine equation, nuclides are assumed to be of trace quantities; that is, the concentration does not affect the fluid density. A steady-state flow option was included. The code is

intended for performance assessment of high-level nuclear waste repositories. In support of the code, two documents were prepared. The SWIFT self-teaching curriculum (Finley and Reeves, 1982) includes 11 application problems. The verification and field comparison document (Ward et al., 1984) details 11 more problems, demonstrating that the code compares favorably with analytical and field data. Data input instructions are given in Reeves and Cranwell (1981).

SWIFT II (Sandia Waste-Isolation Flow and Transport for Fractured Media) - 1986
Under contract to the NRC, GeoTrans, a former subsidiary of Intera, extended the SWIFT code to include fractured media, an enhanced free water surface routine, and extended boundary conditions. To provide the user with complete information, a document on the theory and implementation of the model was prepared (Reeves et al., 1986a). The mathematical development is fully detailed in this 200-page report. Data input instructions are given in Reeves et al., (1986b). In a supplemental document, eight problems and data sets were prepared for self-instruction (Reeves et al., 1986c).

SWIFT III (Sandia Waste-Isolation Flow and Transport for Fractured Media) - 1987
GeoTrans modified the SWIFT II code to use the FORTRAN 77 language and to create a more computer general model. The SWIFT III model permits local one-dimensional subsystems to be attached, as desired, to the grid blocks comprising the global system. The local units may be used either to characterize the secondary porosity of a fractured media or to extend the boundaries of the system in a relatively inexpensive manner. Data input instructions are given in Ward (1987a).

The SWIFT III program consists of a main routine and about 70 supporting subroutines. The basic organization is focused upon the three global integration modules ITER, ITERS, and ITERC. Subroutine ITER solves the coupled partial differential equations for fluid flow, heat transport, and brine transport under transient conditions; ITERS integrates the flow and brine transport under transient conditions; and ITERC solves the coupled partial differential equations for transport of a radionuclide chain. All other routines provide support functions for the integration.

3.2.2.2 GEOFLOW

The computer program GEOFLOW is a finite element program capable of numerically simulating fluid flow and solute mass transport in a two-dimensional or quasi-three-dimensional ground water system. GEOFLOW was developed in 1976 and has been continually improved since then. The aquifer can be confined, semiconfined (leaky), or unconfined. Both transient and steady-state models of fluid flow and solute mass transport can be solved. In the fluid flow simulations, the aquifer can be nonhomogeneous, anisotropic, and of nonuniform thickness. Multiple wells with time-dependent flow rates can also be specified in the model. In the solute mass transport simulations, geochemical reactions such as adsorption, acid neutralization, and radioactive decay can be incorporated by specifying proper characteristic coefficients.

The main routine of GEOFLOW contains two mutually dependent finite element subprograms; one is the flow model which solves the ground water flow equations, and the other is the solute mass transport model which solves the hydrodynamic dispersion equation. Results reported at user-specified times in the simulation period include piezometric heads, velocity and flow (discharge) vectors, concentrations, saturated thicknesses, and retardation factors for acid-front neutralization. To supplement the numerical results produced by GEOFLOW, a graphical post-processing program permits the plotting of potentiometric contours, velocity vectors, and isopachs.

The GEOFLOW program has been verified extensively with the use of analytical solutions taken from the literature and developed by IT's staff, and by the successful reproduction of predicted results with those of other established models such as the USGS' McDonald and Harbaugh model. Additionally, the program has been used in numerous projects to predict the flow and solute transport rates. A complete documentation of the GEOFLOW program and verification testing is provided in the GEOFLOW User's Manual (IT, 1986).

3.2.3 Conceptualization of the Model

IT utilized previously published work as well as a knowledge of the site to develop a conceptual design for the ground water model. The main objective of the model is to determine the relationships between the Great Miami River, the aquifers in the region, and the SOWC pumping wells. For this reason, the

model area was chosen to cover the entire area of possible ground water influence by the SOWC pumping wells (Figure 3.2-1). With reference to Figure 3.2-1, the grid north was oriented 30 degrees west of true north to orient the bedrock trough approximately west to east across the grid. This accommodated the establishment of boundary conditions parallel to the grid lines.

In the grid east-west direction, the grid extends from two miles west of the FMPC to approximately one-half mile east of Ross. In the grid north-south direction, it extends from three-quarters of a mile north of Shandon to one-half mile south of New Baltimore.

At the extremities of the model area, where little detail is required, the finite difference grid size is 2,000 by 2,000 feet. The element size becomes gradually smaller inward and reaches a minimal size of 250 by 250 feet in the area covering the SOWC pumping wells and the meander loop on the Great Miami River. More details on the grid system, including figures showing the model grid for both GEOFLOW and SWIFT III, are provided in Appendix A.

For purposes of this study of the impacts of the FMPC discharge, the ground water flow model was developed for steady state conditions in two dimensions. As the overall RI/FS study progresses, the model will be expanded into three dimensions and the geologic/hydrogeologic complexities in the vertical direction will be incorporated into discrete layers. Figure 3.2-2 presents a simplified cross section west to east across the model area. The hydrogeologic environments are the same as those described by Spieker, 1968a, and in Section 2.1 of this report. Hydrogeologic Environment I represents the eastern portion of the study area and incorporates the SOWC well field and the Great Miami River. Hydrogeologic Environment III represents the western portion of study area, overlain by till. The clay layer shown in the figure is a discontinuous layer, but acts to reduce bulk vertical hydraulic conductivity in much of this portion of the study area. The three-dimensional model will eventually take into account the various geologic and hydrogeologic units shown in Figure 3.2-2, and is expected to consist of 3 to 6 vertical layers. The two-dimensional modeling efforts described in Appendix A did not account for these vertical inhomogeneities, but considered conditions and flow

properties to be uniform throughout the entire thickness of the aquifer. Appendix A provides a discussion of the GEOFLOW and SWIFT III ground water flow model development and the subsequent aquifer analysis.

3.2.4 Model Calibration

As described in Appendix A and Table 3.2.2, the initial input for the ground water flow models was developed from a careful interpretation and extrapolation of data from numerous sources for a compilation of hydraulic values for the site. The initial model results reproduced the general ground water flow patterns throughout the study area; nevertheless, a refinement was achieved through a series of calibration runs using revised parameter values and/or modified boundary conditions (Figure 3.2-3). The model results are compatible with actual values recorded in April 1986 (Figure 2.1-13).

The performance criterion for model calibration focused on how well the predicted water surface elevations at specific grid locations matched field observations of water levels in wells at these same grid locations. As shown in Tables 3.2.3 and 3.2.4, the calibrated version of both the SWIFT III and GEOFLOW models achieved an excellent fit with field observations from April 1986. Note that only one calibration run was required for SWIFT III since the input data had already been refined via GEOFLOW runs. The successful calibration of both models using essentially the same input data base serves as a direct quality assurance check of model performance. Details of the respective calibration runs are given in Appendix A.

The total lack of anomalous predictions in Tables 3.2.3 and 3.2.4 indicates that the models are satisfactorily reproducing important flow processes and boundary conditions throughout the model area. The values assigned to geometric and hydraulic parameters in the final calibration run are all within the range of expected values. This not only increases the confidence in the predicted results, but would tend to minimize the chance that newly collected field data will force major revisions to the preliminary models.

3.2.5 Sensitivity Testing

The purpose of sensitivity testing is twofold. First, sensitivity testing is used to determine the possible errors introduced into the predictions by

uncertainties in the critical input parameter values. Second, sensitivity testing provides a means for evaluating the range of results that would be expected due to seasonal or other predictable changes in important parameters. In either case, the outcome depends on how sensitive the model results are to changes in the parameter of interest.

To perform the sensitivity testing on the calibrated models, numerous runs were made. Each run corresponded to a change in one parameter value within the expected range of values for that parameter, while holding all other parameters constant. A summary of the range of values for sensitivity analysis are presented in Table 3.2.5. Due to the compressed schedule for this work, only the GEOFLOW model was available for extensive sensitivity testing (Appendix A). Similar results would be expected for the SWIFT III model.

The principal results of the sensitivity testing are summarized in Table 3.2.6. The parameters shown to most highly influence the model results are the rate of vertical leakage through the river bottom and the hydraulic conductivity of the aquifer. Since the aquifer hydraulic conductivity is relatively well understood, a decision was made to hold this parameter constant while varying the river bed leakage during subsequent sensitivity testing. Little quantitative information is currently available on the rate of vertical movement of water through the river bottom. This same parameter is also highly affected by seasonal flow and temperature fluctuations, localized changes in sediment properties, and even external activities such as dredging. Any future field or analytical studies for the purposes of refining the model near the SOWC wells must prioritize a more detailed evaluation of the dynamics of flow through the river bottom.

3.2.6 Model Results

3.2.6.1 Zone of Influence (Capture Zone)

The principal objective of the ground water flow model was to establish whether the FMPC discharge to the Great Miami River is within the zone of influence of the SOWC collector wells. The term "zone of influence" as used in relation to this study is best interpreted as a "capture zone" or the zone

from which any water originating from a point within that zone would eventually reach the SOWC wells. The limits of the capture zone were developed from a plot of velocity vectors generated by GEOFLOW, as shown in Figure 3.2-3 for the calibrated model case. By following consecutive velocity vectors, the final disposition of a water particle, and thus whether or not a given location is within the capture zone can be determined. As indicated by the "best fit" line in Figure 3.2-4, the zone of influence developed from the calibrated model would in fact encompass the FMPC discharge. However, the discharge can be observed in Figure 3.2-4 to lie near the southern, downgradient boundary of the zone of influence. It therefore becomes necessary to test the sensitivity of this result to the model parameters. Since the calibrated model was found to be most sensitive to the rate of river leakage, the predicted bounds on the zone of influence were evaluated by assigning values equal to 0.5 and 10 times the calibrated value to the river leakage factor (river leakage factors outside this range create unacceptable model calibration for collector well discharges of 18 MGD).

The resultant "range of model uncertainty" in terms of the zone of influence is also illustrated in Figure 3.2-4. The upper bound on the zone of influence corresponds to the lower leakage factor, since water would have to be drawn to the collector wells from a greater area within the aquifer to make up for the lower river recharge. The FMPC discharge point does, in fact, fall within the range of model uncertainty; therefore, it could lie outside the capture zone under conditions of high river recharge to the aquifer. A similar uncertainty is associated with much of the FMPC property, including the waste storage area. Both of these areas lie within the predicted zone of model uncertainty.

3.2.6.2 Impacts on the SOWC Collector Wells

The finding that the FMPC discharge could lie within the zone of influence of the SOWC collector wells dictates that the potential impacts of that hydraulic connection be evaluated. The long-term intent of the RI/FS is to apply a calibrated solute transport model to help resolve such issues. To proceed with such a model at this point in time would be premature, however, due to the high degree of uncertainty regarding other potentially important influences. These include, for example, the potential contribution of radionuclides from the FMPC to the west via the ground water system, and the

related need to establish and account for meaningful background concentrations in ground water.

Even without the support of a calibrated solute transport model, the results of this study can be used to quantify potential impacts on the SOWC wells at a level of accuracy that will permit the development of general conclusions. Figure 3.2-3 depicts each flow component of the model that contributes to the flow being pumped from the SOWC collector wells. Table 3.2.7 has been prepared from the GEOFLOW results to summarize the relative contribution to the 18 MGD pumping rate of the SOWC wells that originates from the river (i.e., Q_R in Figure 3.2-5). The calibrated model predicted that 76 percent of the 18 MGD flow being pumped from the SOWC wells, or 13.7 MGD, is due to induced infiltration from the river along its full length within the zone of influence (Figure 3.2-4). The contribution from the river ranges from 72 percent for the low leakage case to 82 percent for the high leakage case.

If this flow, at an average uranium concentration of 1.2 pCi/ℓ (background concentration in the river), directly mixes with "clean" ground water from the sand and gravel aquifer, the resultant complete mix value in the SOWC wells would be 0.912 pCi/ℓ for the calibrated model case. This value is close to the observed values in SOWC Collector Well No. 1. The SOWC Collector Well No. 2 has exhibited lower concentrations, but even this would be expected since the lower pumping rate on the outside of the river bend would tend to draw a lesser flow contribution from the river.

The incremental contribution from the FMPC effluent discharge can be worked into this simplified model by considering that only a small percentage of the river segment contributing to the collector wells lies downstream from the FMPC discharge (Figure 3.2-4). The reason for this situation is that the strong regional gradient south of the collector wells is sufficient to maintain a net southern velocity component. The collector wells can more easily draw water from upgradient locations. The results of the calibrated model show that only six percent of the total river contribution comes from the river segments downstream from the discharge point. Under the same assumptions made above for the calibrated model case, and assuming a uranium concentration of 1.3 pCi/ℓ in the river below the point of discharge, the

simplified model gives a uranium concentration of 0.917 pCi/ℓ in the SOWC wells. The incremental contribution from the FMPC discharge is thus 0.005 pCi/ℓ, or only a 0.5 percent increase over the value that would result if only background river water had contributed to the SOWC wells.

Even though this simplified view of potential impacts is not fully representative of actual field conditions, it can be considered as an upper bound on the effects of the river on the SOWC wells. Any retardation or attenuation of uranium in the river sediments and subsurface environment will directly reduce the predicted impact. In addition, any contribution to the uranium concentrations observed in the SOWC wells that is coming from natural, background conditions or other sources such as the FMPC would require a reduction in the river contribution in order to maintain a mass balance.

The background concentration, even at low levels, would likely have a much greater impact than the FMPC effluent discharge when evaluating a mass balance at the SOWC wells. A statistical analysis of the March 1986 ground water sampling results was performed to estimate the "background" uranium concentration. Wells 1NH, State 8, State 10, State 16, H-113, 12-3, and 2-CW were selected as being beyond the potential influence of the site and possibly representative of background uranium concentrations.

The frequency distribution of uranium concentrations observed was distinctly bimodal. Wells State 8, State 10, and H-113 form one cluster, while State 16, 1NH, and 12-3 form a second. This bimodality is probably attributable to two different geologic material source areas within the hydrogeologic regime or geologic matrix. One source area is from the northeast beyond Ross and the other is from the northwest beyond Shandon. The first group of wells, State 8, State 10, and H-113, produce water containing very low concentrations of uranium (0.04 to 0.12 pCi/ℓ) which is in the range of the analytical detection limits. The second group of wells, 1NH, 12-3, and State 16, produce water containing nearly three times as much uranium (0.17 to 0.35 pCi/ℓ) as the first group.

The Student's t-test was used to evaluate the difference in average uranium concentration found in the two groups. The northwest quadrant wells (State 9,

State 10, and H-113) contain significantly less uranium than the southwest quadrant wells (northwest group mean = 0.07 pCi/ℓ; southwest group mean = 0.233 pCi/ℓ. The corresponding t-statistics was -4.84, indicating that the probability that this difference is due to chance is less than 2 percent at a 95 percent confidence level.

In light of these findings, the average uranium concentration for the southwest quadrant could serve as an estimate of the background concentration of uranium. This value of 0.233 pCi/ℓ is only a point estimate of the background uranium concentration. Actual background uranium concentration is subject to seasonal sampling and analytical variability. Nevertheless, the magnitude of this value far exceeds the 0.005 pCi/ℓ incremental impact caused by the FMPC discharge.

4.0 CONCLUSIONS AND PLANS FOR FURTHER STUDY

As stated in Chapter 1.0, the purpose of this interim report is to present the preliminary findings of a hydrogeologic study of the FMPC discharge and its relationship to the SOWC collector wells, and to recommend a future course of action based on the preliminary results. The results of the data review and modeling studies reported in Chapters 2.0 and 3.0 allow the following three general conclusions to be made:

1. The discharge from the FMPC effluent pipeline likely occurs within a portion of the Great Miami River that contributes flow, via induced infiltration, to the SOWC collector wells. The relative contribution of flow to the SOWC wells from the river downstream from the discharge is, however, a small fraction of the flow contributed from upstream reaches of the river. The reasons are that the SOWC wells are located upstream from the discharge, and a strong regional gradient of ground water flow to the south (i.e., downstream) exists as the width of the buried valley aquifer narrows in this area. The sensitivity testing of the model indicates that the FMPC discharge could actually be outside of the capture zone of the SOWC wells if the river infiltration rate is greater than assumed.

2. Even if the FMPC discharge is within the zone of influence of the SOWC wells, the incremental impact of the effluent on the water quality of the pumped water lies within the range of variability of previous observations and is below analytical detection limits under average conditions. Therefore, no observable improvements in water quality would result in the SOWC wells from eliminating the effluent effects (e.g., by relocating the pipeline). The principal reasons for this nondetectable incremental impact are the small percentage of total uranium mass flux contributed by the effluent discharge to the river, and the aforementioned small percentage of the total induced river flow that originates downstream from the discharge pipeline.

3. A mass balance of the sources of uranium observed in the SOWC collector wells was not performed at this time. The model results indicate a potential eastward component of flow from the FMPC Production Area and waste storage units to the SOWC wells. This finding is consistent with previously reported interpretations of regional data on ground water elevations and gradients. However, both the Production Area and the waste

storage units are within the area of modeling uncertainty in relation to the capture zone of the SOWC wells, and will be studied further under the FMPC RI/FS. The background concentration of uranium in ground water, even at low levels, also remains an unresolved issue that may be significant when quantifying a mass balance at the SOWC wells.

Based on the results of this study and the conclusions that could be drawn, no further studies specifically addressing the impact of the FMPC discharge on the SOWC collector wells are considered necessary. These would include, for example, an extension of the analysis to include parameters other than uranium, or the conduct of tracer studies to evaluate the local mixing patterns near the effluent discharge to the Great Miami River. The types of criteria to be evaluated in deciding on whether other constituents require consideration include: the mass or concentration in the effluent, river, or well of concern; the migration potential of the constituent (i.e., solubility, retardation factor, etc.); and the associated toxicological or radiological significance. As discussed in Section 2.4, the choice of total uranium as the only parameter of interest for this study appeared justified based on previously collected data. The decision not to extend the impact study to other radionuclides remains appropriate in light of the results of this study.

Any attempt to quantify the impacts of inorganic parameters monitored under the NPDES program could introduce unresolvable technical complexities. For example, the modeling of nitrates will force more consideration of other sources and background conditions (in the river and wells) due to the extensive agricultural activity and other possible industrial discharges. The chemical transformations of nitrogen compounds would also have to be considered. Residual chlorine would require consideration of the gaseous phase and transfer dynamics at the air-water interface. These complexities, in addition to the fact that the discharge is currently regulated for these parameters, provide little justification to include any NPDES parameters in the impact analysis at this time.

The SOWC collector wells, the river, and the regional ground water flow system assessed in this preliminary study remain of utmost importance to the overall issues being addressed in the sitewide RI/FS and other related studies. The

results of the preliminary modeling studies have been of value in expanding the current understanding of these sitewide and regional issues. In the following paragraphs, additional investigations are described that will address the remaining uncertainties as part of the sitewide RI/FS.

Although the conclusions discussed above can be justified within the context of the modeling results obtained to date, two extensions of the SWIFT III model will be completed for the full resolution of the mass balance at the SOWC collector wells and other issues important to the RI/FS. It is the intent of the overall RI/FS to proceed with SWIFT III modeling using a two-phased approach specifically structured to provide the best reflection of the existing hydrogeologic environment and at the same time maintain the highest level of quality assurance. The first phase emphasizes three-dimensional ground water flow, whereas the second phase couples solute transport to flow. The three-dimensional model will permit a refinement of the local river effects when pumping from depth, and could influence the predicted extent of the capture zone since the direction of small, vertically integrated velocity vectors near the currently predicted boundaries of the capture zone could be altered once a third dimension is considered. The solute transport model will address contaminant dispersion as well as contaminant retardation. The latter is particularly important due to its influence on the rate of plume migration, which typically lags the velocity of ground water flow.

The parallel calibration of both the GEOFLOW and SWIFT III models has sufficiently established the reliability of the results for the model components applied in this study. However, the verification completed to date on the SWIFT III model is not of a complexity and scope consistent with the projected level of modeling to be performed for the RI/FS. The future use of additional capabilities of SWIFT III will be verified, as necessary, during the course of the RI/FS. A continued parallel use of GEOFLOW represents one intended approach. A comparison of SWIFT III results with the three-dimensional flow component of the USGS' McDonald and Harbaugh model will also be used for verification purposes due to the widespread use and acceptability of the latter model.

The regional issues to be addressed in the RI/FS will require the establishment of representative background concentrations for uranium in ground water. The available data base indicates that natural variations in uranium concentration occur in the vicinity of the site, possibly the result of different geologic settings. The regional influences of major pumping systems on ground water flow patterns, and thus contaminant transport patterns, add complexity to the issue of background concentration. The resolution of this issue is a proposed objective of the sitewide remedial investigation.

The model sensitivity runs performed to date indicate a high degree of sensitivity to the rate of induced infiltration from the river. A representative value for the leakage factor has not yet been directly measured in the field; however, the value of river infiltration predicted by the calibrated model falls within the range of previously published values of this parameter within the general vicinity of the site. Further, the results of the model have established a range of possible values of leakage factors beyond which the predicted results lose credibility in relation to field observations. The overall conclusions of this preliminary study do not change for any assumed leakage factor within this range. It is proposed, however, that a direct field determination of the leakage factor be completed as part of the sitewide RI/FS due to the possible sensitivities of the overall extent of the capture zone on this parameter. If the field measurements fall outside of the currently assumed range, the results and conclusions of this interim study will be reevaluated.

The sediment sampling and grain-size analysis program described in Chapter 5.0 will be of limited value in quantifying the leakage factor in the river since only the top several inches of the active sediment layer are being collected and tested. It is recommended that a more extensive program be conducted that would involve the installation of piezometers in the river bed with subsequent slug testing. The program would have to be repeated at multiple locations to develop representative values and during different seasons to establish a relationship with water depth and water temperature. This scope of work has not yet been approved under the current sitewide RI/FS.

The calibration of the comprehensive ground water model for areas to the east of the FMPC and in the vicinity of the SOWC wells will be enhanced by the proposed installation of additional monitoring wells in these areas. The principal purpose in installing these wells will be to provide additional ground water elevation data to the west and south of the SOWC wells since these directions are the most important to any investigation of the FMPC and the effluent line. The wells can also serve as additional ground water quality monitoring points at off-site locations.

5.0 FIELD PROGRAM

5.1 INTRODUCTION

Sampling of the Great Miami River water and bottom sediments is being performed to accomplish the following objectives:

- To establish background (upstream) levels of dissolved uranium in the river
- To assess the rate of dilution downstream from the FMPC outfall of dissolved uranium discharged to the river
- To provide qualitative estimates of the hydraulic properties of the stream bed as they relate to induced infiltration of the river water (hydraulic conductivity) and channel roughness (Manning coefficient, n).

To accomplish the above-stated objectives, the following tasks are being performed, each of which is described further in the following sections:

- River water sampling for uranium concentration downstream from the Ross bridge to about one mile below the FMPC outfall, including background stations
- Flow velocity measurement at river sampling locations
- River stage measurement at the "Ross Bridge" at Route 126, 1.3 miles upstream, and at the bridge at New Baltimore, three miles downstream from the FMPC outfall
- Channel depth profiling at the Ross Bridge and approximately 100 yards downstream from the FMPC outfall, and channel observations at other selected river locations
- Sampling of the Proctor & Gamble outfall, located on the east bank of the river, approximately 300 yards downstream from the Ross Bridge, pending approval and coordinated through WMCO (Conditional)
- River bottom sediment sampling for grain-size analysis and visual descriptions of river-reach hydraulic characteristics.
- Stream channel observations to assess the applicability of using historical data and reports.

The surface water and sediment sampling locations referred to in this section are shown in Figure 5.1-1.

5.2 RIVER BED SEDIMENT PROPERTIES AND GROUND WATER LEVELS

In order to provide refined estimates of the hydraulic properties of the stream bed as they relate to the interaction between the river and the aquifer, river bottom sediment sampling will be performed for grain-size analysis. Samples will be obtained at three upstream and three downstream locations. The information will be used to estimate vertical hydraulic conductivities.

The river bottom sediment sampling will be performed at three locations upstream from the FMPC outfall and at three locations downstream. Samples will be obtained using a Wildco-Eckman bottom dredge. Three samples will be obtained at each location and composited by the laboratory. Samples will be analyzed for grain-size distribution using sieve analysis down to a No. 200 U.S. Standard sieve, followed by hydrometer analysis (or approved equivalent).

Existing water level data obtained from WMCO and the MCD are presently being compiled and evaluated. Seasonal data available over the past year will be plotted on base maps and contoured to evaluate ground water flow directions and gradients surrounding the SOWC well field and the Great Miami River. Additionally, the information will be used to evaluate seasonal trends in ground water levels.

Monthly data will be collected over the next ten months to expand the ground water level data base so that predictions of future trends can be made. Data will be obtained from the Miami Conservancy District and WMCO, and augmented by additional data collection as necessary.

5.3 WATER QUALITY STUDIES

5.3.1 Monthly Sampling Program

The following tasks will be performed monthly at the initial sampling round and during subsequent months for a total period of ten months. Background

river water sampling will be performed at the Ross Bridge. Samples will be obtained at the centers of five cross-sectional subdivisions having approximately equal discharge rates. The sampling subdivision will be defined by first determining the total river discharge in increments across the channel in accordance with Buchanan, T.J., and W.P. Somers, 1969, "Discharge Measurements at Gaging Stations." During periods of low flow, discrete samples will be obtained from each sampling subdivision at six-tenths the channel depth. During periods of high flow (for which the channel depth is greater than ten feet) samples will be obtained at two-tenths and eight-tenths the channel depth. Three sampling traverses of the channel will be performed. Samples obtained during each traverse will be mixed to generate three replicate composite samples in the field for laboratory analysis during each sampling round.

Flow velocity measurements will be obtained at the time of sampling by using a direct-reading Marsh-McBirney velocity meter. Additionally, river stage measurements will be obtained during sampling by measuring the difference in elevation between surveyed reference points on the bridges at Ross and New Baltimore and the water surface in the center of the main portion of the channel.

If approved, samples of the Proctor & Gamble outfall will be obtained at three locations across the width of the outfall and mixed to form one composite sample. Two replicate samples will be obtained for laboratory analysis during each sampling round.

5.3.2 One-Time Sampling Program

River water samples will be collected on one occasion at four locations over the 1.3-mile distance between the Ross Bridge and the FMPC outfall. Samples will be obtained at single locations from the middle of the main portion of the channel at six-tenths the depth. Two replicate samples will be obtained at each location, resulting in eight samples for laboratory analysis.

River water sampling will also be completed approximately 150 yards downstream from the FMPC outfall. The program will consist of discrete sampling at six-tenths the channel depth at five locations at equal distances across the

channel width. Two sampling traverses of the channel width will be performed, resulting in ten discrete samples for laboratory analysis.

River water samples will be collected at five additional locations over the one-mile distance downstream from the FMPC outfall. Samples will be obtained at single locations from the middle of the main portion of the channel at six-tenths the depth. Two replicate samples will be obtained at each location, resulting in ten samples for laboratory analysis.

Channel depth profiling at the Ross Bridge and 100 yards downstream from the FMPC outfall will be performed using a weighted tape. Channel observations at these and other selected river locations will be performed to assess the general stability of the channel and the applicability of using historical data and reports.

Flow velocity measurements will be obtained at the location and depth of each river water sampling point using a direct-reading Marsh-McBirney velocity meter.

5.3.3 Field and Analytical Procedures

Sampling of river bottom sediments and surface and ground water will be in accordance with applicable sections of the Fernald RI/FS Quality Assurance Project Plan (QAPP). Sample documentation, chain-of-custody, decontamination, and health and safety procedures established for the RI/FS will be followed. Laboratory procedures outlined in the IT/RSL laboratory QA manual will be followed for sample analysis and reporting.

River water samples will be obtained using a discrete-depth sampler in accordance with the sample handling guidelines presented in "Methods for Determination of Radioactive Substances in Water and Fluvial Sediments" (Thatcher et al., 1977), and "Methods for Collection and Analysis of Water Samples for Dissolved Minerals and Gases" (Brown et. al, 1970). River water sampling will proceed in a downstream-to-upstream direction.

River water samples will be filtered in the field through a 0.45 micron filter to remove suspended solids, and acidified to prevent adsorption of radionuclides on the walls of the sampling containers in accordance with "National Handbook of Recommended Methods for Water-Data Acquisition," (U.S. Department of the Interior, 1977).

5.4 PRELIMINARY FINDINGS OF FIELD PROGRAM

The initial portion of the field program for data collection on the Great Miami River was implemented on September 11 through 14, 1987. The tasks performed at this time included both the monthly sampling tasks and the one-time sampling tasks described in Section 5.3 of this report. The primary tasks performed included:

- Background river water sampling for dissolved uranium in channel traverses at the Ross Bridge
- River water sampling for dissolved uranium in channel traverses downstream from the FMPC outfall
- River water sampling for dissolved uranium at discrete locations upstream and downstream from the FMPC outfall
- River bottom sediment sampling for qualitative assessment of channel bed hydraulic characteristics.

The river water and bottom sediment sampling locations are shown in Figure 5.1-1. The river sampling locations are numbered sequentially in an upstream direction. Further descriptions of the sampling locations and the types and numbers of samples obtained are listed in Table 5.4.1.

In the performance of these tasks, flow velocity and river stage measurements were obtained at the Ross Bridge and observations of the channel characteristics for discharge calculations were made.

The order of execution of the field program was as follows:

- Water quality sampling was performed upstream from the FMPC outfall at Locations 7 through 10, proceeding in a downstream-to-upstream direction.
- Background water quality sampling traverses were performed at the Ross Bridge

- Water quality sampling was performed downstream from the FMPC outfall at Locations 1 through 5, proceeding in a downstream-to-upstream direction

- Water quality sampling traverses were performed 200 feet downstream from the FMPC outfall.

- River bottom sediment sampling was performed, proceeding in a downstream-to-upstream direction at Locations 2, 3, 5, 7, 8, and 9.

5.4.1 River Water Sampling

River water quality samples were obtained using a Kemmerer discrete depth sampler which consists of an open cylinder with spring-loaded end caps. To obtain a sample, the sampler is lowered to the desired sampling depth. The cylinder ends are closed by releasing a weight which falls along the rope from which the sampler is suspended. The weight triggers a spring-loaded mechanism closing the ends of the sampler. The sampler is then raised and a water sample is removed through a port on the bottom of the sampler.

River current velocity measurements were obtained using a Marsh-McBirney velocity meter. The instrument determines velocity in feet per second at a discrete depth. A probe, with an open orifice on the downstream side, is lowered to the desired depth. The current velocity is determined from the pressure drop created at the orifice.

At river sampling Locations 1 through 5 and 7 through 10, water samples were obtained from the portion of the channel in which the current was observed to be strongest. All samples were obtained from six-tenths the channel depth at the chosen locations. Two samples were obtained at each location along with current velocity measurements at the same depth.

Background water quality sampling at the Ross Bridge was performed at five sampling subdivisions across the channel. The spacing width of each sampling subdivision was weighted by discharge so that each subsample represented approximately 20 percent of the flow. The spacing of each sampling location was determined in the following manner:

- The channel width was divided into 21 subsections, each approximately 12-1/2 feet in width. The vertical guard

rail supports on the bridge overhead were used for reference.

- At the center of each subsection, the channel depth was measured using a graduated rod.
- At the center of each subsection, the current velocity was measured at six-tenths the channel depth.
- For each subsection, the discharge through the section was determined from the product of the section width, depth, and current velocity.
- The total discharge of the river at the Ross Bridge was determined by summing the discharges through the individual subsections.
- Five water sampling subdivisions (A through E) were selected by combining subsections used for discharge measurements such that each sampling subdivision represented approximately 20 percent of the total river discharge.
- The center of each of the five sampling subdivision was determined for use as the sampling location.

The channel cross section at the Ross Bridge, with sampling locations and water depths, is shown in Figure 5.4-1. Water samples were obtained from each of the five sampling subdivisions in three sampling traverses. Samples obtained from each traverse were mixed in equal portions to generate three composite samples representing each sample traverse for laboratory analysis.

River water sampling 200 feet downstream from the FMPC outfall at Location 6 was performed at five sampling locations approximately equally-spaced across the channel. The spacing of each sampling location was determined by measuring the total channel width, dividing the width into sections of equal width, and determining the centers of the sections for use as the sampling locations. The channel cross section downstream from the outfall, with sampling locations and depths, is shown in Figure 5.4-2.

5.4.2 Bottom Sediment Sampling

River bottom sediment samples were obtained using a Wildco-Eckman bottom dredge. The dredge consists of an open-ended metal box, one cubic foot in capacity, with spring-operated jaws which close over the bottom. A messenger

weight, dropped along the rope from which the dredge is suspended, is used to trigger the jaws.

Three samples were obtained from each bottom sediment sampling location, to be composited by the laboratory for grain-size analysis. Samples were obtained from the same general vicinities as river water samples having corresponding location numbers (Figure 5.1-1 and Table 5.4-1). Individual sampling locations were spread out to obtain representative coverage of the bottom sediments at each sampling site. Samples were obtained from relatively calm portions of the channel which could be waded. This approach was dictated by the difficulties encountered in operating the dredge in strong current and the need to be sure finer-grained sediment was not washed from the sample. The loss of fine-grained material would bias the grain size analysis and result in higher hydraulic conductivities than are actually present. Bottom sediment particles greater than two inches in diameter were excluded from samples sent for grain size analysis.

5.4.3 Laboratory Testing

Laboratory testing of the river water and bottom sediment samples is currently being performed. The results of the laboratory testing and the analyses of the test results will be reported as they become available.

River water samples are being tested for gross alpha and gross beta activity, cesium 37, ruthenium-106, total dissolved uranium, uranium isotopes, thorium and thorium isotopes, and strontium-90. The dissolved portion is defined as that portion which passes through a filter having a pore size of 0.45 microns. Field preparation of the water samples obtained at the Ross Bridge included coarse prefiltering through a 0.7-micron filter. This was done prior to the generation of composite samples to remove the majority of suspended solids. Laboratory preparation of all water samples collected includes filtering through a 0.45 micron filter and the addition of nitric acid to prevent subsequent precipitation and adsorption of dissolved constituents.

River bottom sediment samples are being analyzed for grain-size distribution down to a No. 200 U.S. standard sieve and hydrometer analysis for silt and clay size particles.

Water quality laboratory testing is being performed by the IT analytical laboratory in Oak Ridge, Tennessee. Grain-size analyses on sediment samples are being performed by the IT soils and materials testing laboratory in Export, Pennsylvania.

5.4.4 Observed River Channel Characteristics

During the sampling program, channel depths and current velocities were measured, and qualitative observations were made to assess the stability of the channel and the bottom sediment properties.

Measured current velocities and channel depths are listed in Table 5.4.2. Observed velocities ranged from 0.50 to 2.10 feet per second in main portions of the channel. The highest current velocity observed was at Location 2, in a constricted channel immediately downstream from a set of rapids.

Three sets of rapids were observed downstream from the FMPC outfall: immediately upstream from Location 2, immediately upstream from Location 3, and at Location 5. One rapid was also encountered between the outfall and the Ross bridge, approximately 500 feet downstream from Location 9. At each section of rapids, current velocities were high, and the bottom material was rocky and relatively free of fine-grained sediments. Channel depths in rapid sections ranged from inches to several feet. The drop in water surface levels through sections of rapids was estimated to range from one to two feet. Directly opposite the FMPC outfall, the current was observed to be swift and turbulent, but relatively deep with no formation of rapids.

Low flow conditions were in existence at the time the field program was implemented. The discharge of the river, calculated prior to sampling at the Ross Bridge, was approximately 400 cubic feet per second (cfs). This value is considered realistic given the range of flows typically recorded during this portion of the year at the U.S. Geologic Survey gaging station at Hamilton, eight miles upstream. The probable error in discharge measurement is estimated to be plus or minus 15 percent. A channel obstruction in the form of a tree stump hung on a gravel bar near the left bank and caused drifting of the boat. Some channel subsections used in measuring the discharge represented greater than 5 percent of the total discharge, as specified by

Buchanan and Somers (1969). This degree of error is considered to be acceptable considering that the magnitude of recorded discharge rates for the Great Miami River ranges over three orders of magnitude.

Bottom sediment descriptions for sediment sampling locations are listed in Table 5.4.3. Locations at which fine-grained material are present correspond to relatively long, still reaches of the river. In general, the sediments are coarser and contain smaller amounts of fine-grained material downstream from the FMPC outfall than do upstream sediments.

A qualitative assessment of channel stability is given in Table 5.4.4. Major deviations from previously documented channel configurations are:

- There is presently a prominent gravel bar in the channel center beginning approximately 250 feet downstream from the FMPC outfall and extending to approximately 200 feet upstream from Location 3
- Channel depths are presently greater in some upstream reaches then previously recorded.

Greater depths in upstream reaches are probably due, to a large extent, to dredging operations conducted by the U.S. Army Corps of Engineers in recent years. In general, however, the position of the main channel has remained relatively stable. Figures 5.4-3 and 5.4-4 compare recently measured cross-sectional channel configurations at the Ross Bridge and near the FMPC outfall with configurations developed in 1961 (Dove, 1961).

5.4.6 Implications for Impact of Effluent from FMPC Outfall

Based on field observations, the effluent from the FMPC outfall may not mix extensively with river water immediately in the vicinity of the source. The effluent would then be expected to remain relatively close to the west bank of the river towards the outside of the oxbow bend. The probability of this occurring is increased by the presence of an eddy pool immediately downstream from the outfall, which turns water caught within the eddy toward the shore.

A gravel bar, presently exists downstream from the FMPC, and splits the channel into two distinct channels during periods of low flow. It appears that the majority of the river flow, and thus the effluent, stays in the

western side of the bar. During periods of high flow, it is anticipated that this effect would be lessened due to overtopping of the gravel bar by higher water levels and an increased amount of turbulence.

These field observations are generally consistent with conditions assumed in the two surface water modeling studies. Assumptions and results of the dispersion model are still considered to be conservative by underestimating the amount of initial mixing. In addition, the presence of the flow divide does not affect the results since lateral mixing near the source predicted by the model never extended even to the middle of the channel. If most flow stays within the western channel as observed, then the complete mix scenario could also remain as an acceptable approximation. Recalling that the induced infiltration from the river preferentially occurs along the eastern side, the local splitting of flow could, in fact, yield a smaller net contribution of uranium to the SOWC wells.

It is also of interest to compare the measured river conditions with the results predicted by the HEC-2 model. To perform this comparison, the HEC-2 model was run with a flow rate of 400 ft^3/sec as measured in the field. The weighted average values of flow velocity and water depth predicted by the model for sections bounding the FMPC outfall were 1.1 ft/sec and 5.3 feet, respectively. The same averaging process was completed on field measurements from four nearby stations used in the field program, with the corresponding values being 0.9 ft/sec and 5.8 feet. The values compare very favorably, thereby providing a degree of confidence in using the HEC-2 results for the Great Miami River water quality model in Section 3.1.

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TABLES

TABLE 2.1.1

SUMMARY OF SELECTED MONITORING WELL DATA

WELL I.D.	STATE PLANE COORDINATES(a) E	N	GROUND SURFACE ELEVATION (feet MSL)	SCREEN ELEVATION TOP-BOTTOM (feet MSL)	MONITORED ZONE	SOURCE OF DATA(b)	RATIONALE FOR SELECTION
River	1393780	481730	555.84	-	-	MCD	-
Coll-1	1388610	479530	555.48	NA	Sand & Gravel	MCD	Pumping well - not used to construct ground water contour maps
Coll-2	1388520	481400	535.14	NA	Sand & Gravel	MCD	Pumping well - not used to construct ground water contour maps
SW-4A	1387625	479375	536.38	NA	Sand & Gravel	MCD	Geographical location indicates well is completed in sand and gravel aquifer
SW-3A	1388150	479300	535.98	NA	Sand & Gravel	MCD	Geographical location indicates well is completed in sand and gravel aquifer
R-7	1388650	480250	535.10	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
SW-1	1390860	478810	540.61	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
K-2N	1388670	481202	538.75	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
K-1	1388830	480830	540.34	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
K-4	1389860	481090	542.01	NA	Sand & Gravel	MCD	Geographical location indicates well is completed in sand & gravel aquifer
EL-1	1390170	482100	536.01	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
K-3	1388380	481680	541.01	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
B-1	1387034	480633	540.80	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
B-2	1386136	481180	538.30	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
B-3	1385018	481514	535.50	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
B-4	1388045	480443	539.30	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
IT-4	1384984	481528	536.64	502-492	Upper Sand & Gravel	IT	Monitors sand & gravel aquifer
H-127	1385433	479115	540.80	478-476	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
O-2W	1384500	477800	534.85	404	Sand & Gravel	MCD	Monitors sand & gravel aquifer
O-2E	1384520	477800	534.09	492	Sand & Gravel	MCD	Monitors sand & gravel aquifer
BLK	1383957	476918	574.20	494-489	Sand & Gravel	ODNR/IT	Monitors sand & gravel aquifer
BHMHP-D	1384467	475625	575.50	475-465	Sand & Gravel	ODNR/IT	Monitors sand & gravel aquifer
O-3	1388090	485250	554.10	NA	Sand & Gravel	MCD	Geographical location indicates well is completed in sand & gravel aquifer
WW-1	1391730	485970	551.23	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
LB-1	1392490	483630	554.01	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
WK-1	1393890	482660	543.05	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
ER-1	1392220	480970	535.75	NA	Sand & Gravel	MCD	Monitors sand & gravel aquifer
IT-6	1380706	476416	574.07	498-488	Upper Sand & Gravel	IT	Monitors sand & gravel aquifer
STATE 16	1372807	479621	610.40	490-485	Sand & Gravel	ODNR/IT	Monitors sand & gravel aquifer
12-5	1373903	474575	611.70	507-505	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
12-7A	1373785	473520	604.10	NA	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
16-1-S	1385401	467800	521.70	NA	Sand & Gravel	IT/GS	Elevation of base of well (490 ft. msl) is in sand & gravel aquifer
H-129	1383418	469993	539.10	496-489	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
H-124	1381284	469780	519.20	463-459	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
H-128	1379929	469765	516.00	NA	Sand & Gravel	IT/GS	Elevation of base of well (497 ft. msl) is in sand & gravel aquifer
H-126	1382261	471596	531.70	NA	Sand & Gravel	IT/GS	Elevation of base of well (509 ft. msl) is in sand & gravel aquifer
DE	1382241	473619	600.10	NA	Sand & Gravel	IT/GS	Elevation of base of well (502 ft. msl) is in sand & gravel aquifer
RE	1380243	472495	525.69	NA	Sand & Gravel	IT/GS	Geographical location indicates well is completed in sand & gravel aquifer
H-115	1379932	472521	528.50	448.5-432.5	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
H-122	1378810	474290	540.00	NA	Sand & Gravel	IT/GS	Elevation of base of well (456 ft. msl) is in sand & gravel aquifer
7-8A	1379607	474344	529.30	NA	Upper Sand & Gravel	IT/GS	Elevation of base of well (482 ft. msl) is in upper sand & gravel aquifer
D-S	1379666	474775	546.70	NA	Sand & Gravel	IT/GS	Monitors sand & gravel aquifer
HK-D	1380047	475864	578.90	403-393	Lower Sand & Gravel	ODNR/IT	Monitors sand & gravel aquifer
FMPC-15-S	1380201	476149	577.70	498-493	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
BPH	1377655	477575	576.60	491-487	Upper Sand & Gravel	ODNR/IT	Monitors upper sand & gravel aquifer
H-105	1378612	476121	570.80	NA	Upper Sand & Gravel	IT/GS	Monitors upper sand & gravel aquifer
FMPC-17-D	1379163	476265	534.20	470-465	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
FMPC-18-S	1378660	479300	571.31	517-512	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
FMPC-9	1378050	479570	555.40	457-452	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
IT-5	1382029	476752	594.22	499-489	Upper Sand & Gravel	IT	Monitors upper sand & gravel aquifer
FMPC-14-D	1379579	476805	533.90	491-486	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
FMPC-16-S	1378900	477700	540.50	510-503	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
IT-1	1381891	478683	577.93	500-490	Upper Sand & Gravel	IT	Monitors upper sand & gravel aquifer
FMPC-F3	1379901	479820	579.34	390-369	Lower Sand & Gravel	FMPC	Pumping well - not used to construct ground water contour maps
IT-2	1382997	479761	593.22	501-491	Upper Sand & Gravel	IT	Monitors upper sand & gravel aquifer
IT-3	1383292	481657	609.70	503-493	Upper Sand & Gravel	IT	Monitors upper sand & gravel aquifer
FMPC-13-S	1382100	481700	588.73	515-510	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
FMPC-10	1379840	481730	586.56	480-460	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
FMPC-3	1379000	481870	558.80	474-454	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
FMPC-11	1377958	482271	583.58	510-455	Upper Sand & Gravel	FMPC	Monitors upper sand & gravel aquifer
RB	1376759	485834	579.90	NA	Sand & Gravel	IT	Monitors sand & gravel (IT data)
PALLET CO	1376516	482407	579.20	509-505	Upper Sand & Gravel	ODNR/IT	Monitors upper sand & gravel aquifer

(a)The coordinates are shown in Figure 2.1-1.

(b)FMPC = Feed Materials Production Center

MCD = Miami Conservancy District

GS = U.S. Geological Survey

IT = IT Corporation

ODNR = Ohio Department of Natural Resources

TABLE 2.1.2

SUMMARY OF GROUND WATER LEVEL FLUCTUATIONS IN SELECTED WELLS

WELL NO.(a)	MAXIMUM(b)	(DATE)	MINIMUM	(DATE)	MEAN	RANGE	NO. OF OBSERVATIONS(c
Coll. 1	506.27	(01/31/86)	475.27	(8/11/86)	492.90	31.00	19
Coll. 2	512.37	(04/29/87)	497.54	(9/26/86)	504.91	14.83	19
SW-1S	515.39	(10/02/87)	505.25	(9/26/86)	510.68	10.14	19
SW-2S	507.05	(10/02/87)	482.85	(8/11/86)	497.34	24.20	19
ER-1	525.89	(03/27/86)	519.76	(3/06/87)	522.38	6.13	19
O-2E	522.44	(03/27/86)	518.26	(9/26/86)	519.93	4.18	19
BU-13	528.71	(03/15/86)	521.18	(9/11/86)	524.14	7.53	385

(a)Refer to Figure 2.1-1 for well locations.

(b)All elevations are in feet above mean sea level.

(c)The data base was compiled by Miami Conservancy District and covers the period from January 1, 1986 through September 1, 1987.

TABLE 2.2.1

HYDROLOGIC PARAMETERS: GREAT MIAMI RIVER

CROSS SECTION (a)	APPROXIMATE RIVER MILE(b)	CROSS SECTIONAL AREA ($feet^2$)	AVERAGE DEPTH (feet)	SURFACE WIDTH (feet)	HYDRAULIC RADIUS (feet)	AVERAGE VELOCITY (feet/sec)	WATER SURFACE ELEVATION (feet MSL)
A1	20.49	844	3.12	270	3.2	4.2	501.6
A2	20.77	1,170	3.86	303	4.3	3.0	505.8
A3	20.83	1,420	5.42	262	5.4	2.5	506.3
N. Balt Bridge	20.84	1,420	5.42	262	5.4	2.5	506.3
A4	20.91	1,499	5.68	264	5.5	2.4	506.8
A5	21.57	1,327	4.26	311	4.3	2.7	509.0
B	21.83	1,855	5.79	320	6.0	1.9	510.7
C	22.15	1,063	3.87	274	4.2	3.3	512.4
D	22.45	1,824	5.22	349	5.1	1.9	514.7
E	22.81	1,662	5.61	296	5.7	2.1	516.2
F	22.13	2,002	6.55	305	6.5	1.8	517.3
G	23.70	1,638	3.90	420	3.8	2.2	520.6
H	23.93	1,148	3.56	322	3.6	3.1	522.8
I	24.18	1,613	4.43	364	4.4	2.2	524.3
J	24.49	2,541	8.19	310	8.1	1.4	524.5
K	24.83	2,422	8.20	295	8.5	1.5	524.6
L	25.09	2,022	8.57	236	8.4	1.8	524.7
M	24.45	2,069	8.33	248	8.0	1.7	524.9
N	25.57	2,042	6.47	316	5.9	1.7	525.0
Ross Bridge	25.58	2,042	6.47	316	5.9	1.7	525.0
0	25.66	2,017	6.91	292	6.5	1.8	525.2

(a)Refer to Figure 2.2-1 for cross section locations.

(b)River miles scaled from USGS topographic map (Figure 2.2-1). Values reported in the Hamilton County Flood Insurance Study are approximately one mile greater at each cross section.

TABLE 2.4.1

SUMMARY OF 1986 NPDES DATA AT MANHOLE 175

PARAMETER	UNITS	NUMBER OF SAMPLES	DAILY MINIMUM	DAILY MAXIMUM	ANNUAL AVERAGE	NPDES PERMIT LIMITS DAILY MAXIMUM	NPDES PERMIT LIMITS MONTHLY AVERAGE	PERCENT COMPLIANCE
pH	pH Units	Daily Grab	6.9	9.7	NA(a)	Range = 6.5 to 10.0		100
Nitrate	kg/day as N	53	6	849	148	3,180	1,590	100
Ammonia	kg/day as N	53	0.1	4	0.8	43	28	100
Residual Chlorine	mg/l	26	<0.02	0.09	<0.03	0.1	NA	100
Oil & Grease	mg/l	53	<5	9	<5	15	NA	100
Suspended Solids	mg/l	53	<2	31	<5.5	60	20	100
Flow Rate	MGD	Continuous	0.199	1.092	0.501	NA	NA	NA

(a)NA = not applicable

Reference: Feed Materials Production Center
Environmental Monitoring Annual Report for 1986
Table 28

TABLE 2.4.2

SUMMARY OF AVERAGE ANNUAL CONCENTRATIONS
OF RADIONUCLIDES FROM MANHOLE 175 AND GREAT MIAMI RIVER

PARAMETER	SAMPLING LOCATION(a)		
	W1 (pCi/l)	MANHOLE 175 (pCi/l)	W3 (pCi/l)
Gross Alpha	3	NA(b)	3
Gross Beta	5	NA	5.8
Cs-137	<8	<1.5	<7
Np-237	NA	<0.015	NA
Pu-238	NA	<0.015	NA
Pu-239/240	NA	<0.015	NA
Ra-226	<0.5	<6.6	<0.5
Ra-228	<0.5	<6	<0.6
Ru-106	NA	<15	NA
Sr-90	<1.3	1.3	1.7
Tc-99	3	2200	6
Th-232	NA	0.78	NA
U-234	0.74	160	0.83
U-235	0.030	8.5	0.040
U-236	0.0060	29	0.0090
U-238	0.75	250	0.90
Uranium	1.2	450	1.4

(a)Refer to Figure 1.1-1 for sampling locations.

(b)NA = Not analyzed.

Reference: Feed Materials Production Center
Environmental Monitoring Annual Report for 1986
Tables 12 and 13, April 1987.

TABLE 3.1.1

RESULTS OF HYDRODYNAMIC DISPERSION MODEL AT SELECTED RIVER LOCATIONS

X-AXIS	DISTANCE ALONG Z-AXIS								
	0 FEET	10 FEET	20 FEET	30 FEET	50 FEET	80 FEET	120 FEET	170 FEET	346 FEET
0 ft									
C/Co(a)	1.000	0	0	0	0	0	0	0	0
Cr(b)	24.5	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
100 ft									
C/Co	0.5752	0.0100	0.0022	0.00087	0.00023	0.00005	0	0	0
Cr	14.6	1.43	1.25	1.22	1.21	1.2	1.2	1.2	1.2
200 ft									
C/Co	0.3641	0.0198	0.0045	0.0018	0.00047	0.00011	0.00002	0	0
Cr	9.7	1.66	1.30	1.24	1.21	1.2	1.2	1.2	1.2
300 ft									
C/Co	0.2623	0.0285	0.0069	0.0027	0.00072	0.00016	0.00003	0	0
Cr	7.3	1.86	1.36	1.26	1.22	1.2	1.2	1.2	1.2
500 ft									
C/Co	0.1690	0.0414	0.0114	0.0046	0.0013	0.00029	0.00006	0.00001	0
Cr	5.1	2.16	1.47	1.31	1.23	1.21	1.2	1.2	1.2
800 ft									
C/Co	0.1127	0.0503	0.0174	0.0075	0.0021	0.00049	0.00010	0.00002	0
Cr	3.83	2.37	1.61	1.37	1.25	1.21	1.2	1.2	1.2
1,200 ft									
C/Co	0.0802	0.0511	0.0230	0.0110	0.0033	0.00080	0.00017	0.00003	0
Cr	3.07	2.39	1.74	1.46	1.28	1.22	1.2	1.2	1.2
2,000 ft									
Cr/Co	0.0534	0.0438	0.0276	0.0161	0.0057	0.0015	0.00033	0.00006	0
Co	2.44	2.22	1.84	1.58	1.33	1.23	1.21	1.2	1.2

TABLE 3.1.1
(Continued)

X-AXIS	DISTANCE ALONG Z-AXIS								
	0 FEET	10 FEET	20 FEET	30 FEET	50 FEET	80 FEET	120 FEET	170 FEET	346 FEET
3,000 ft									
C/Co	0.0395	0.0357	0.0273	0.0189	0.0083	0.0025	0.00059	0.00011	0
Cr	2.12	2.03	1.84	1.64	1.39	1.26	1.21	1.2	1.2
4,000 ft									
Cr/Co	0.0322	0.0302	0.0253	0.0195	0.0102	0.0035	0.00090	0.00018	0
Cr	1.95	1.90	1.79	1.65	1.44	1.28	1.22	1.2	1.2
5,000 ft									
C/Co	0.0277	0.0265	0.0233	0.0192	0.0113	0.0045	0.00124	0.00027	0
Cr	1.85	1.82	1.74	1.65	1.46	1.30	1.23	1.21	1.2

(a)C/Co normalized input concentration.

(b)Cr = Concentration in river (pCi/l).

TABLE 3.2.1

EVALUATION OF THREE-DIMENSIONAL TRANSPORT CODES FOR MODELING OF THE FERNALD SITE

			SPECIAL FLUID FLOW OPTIONS			SPECIAL TRANSPORT OPTIONS				BOUNDARY CONDITIONS						
PROGRAM NAME	AVAILABILITY	COMPUTERS/ (LANGUAGE)	UNCONFINED/ CONFINED FLOW	DENSITY VARIATIONS	TEMPERATURE/ CONCENTRATION DEPENDENT VISCOSITY	ADSORPTION	PARTICLE TRACKING(a)	RADIONUCLIDE DECAY CHAINS	DECAY OF SINGLE SPECIES	PRESCRIBED FLUXES, HEADS & CONCENTRATIONS	CARTER-TRACEY INFINITE RESERVOIR QRS RESERVOIR INFLUENCE FUNCTION	WASTELEACH MODEL	SOLUTION METHOD (NUMERICAL MODEL,) EQUATION SOLVER)	PROBLEM SIZE RESTRICTIONS	PRE/POST PROCESSING (PLOTTING GRID GENERATION) RESTART, ETC.) (PLOTTERS SUPPORTED UNKNOWN)	VERIFICATION
SWIFT II/ SWIFT III	Swift II will be offered to public in future. Date is unknown. GEOTRANS will provide SWIFT III	Prime 550 VAX CDC and others (Fortran 77)	Yes	Yes	Yes	Nonlinear	Yes (in post processor)	Yes	Yes	Yes	Yes	Yes	Finite Difference Iterative (two-line successive over-relaxation) (Direct for small problems)	Unknown Large problems would require grid refinement	Contour Plotting Time Series Plotting Restart Facility	25 test problems
GEOFLOW	2-D source code in-house; 3-D version flow portion of code is in-house; 3-D solute transport under development. Could be available within six weeks if needed.	Prime 750 IBM-PC (Fortran 77)	Yes	No	No	Linear	No	No	Yes	Yes	No	No	Finite element for each horizontal layer; finite difference for vertical direction direct solution	Depends on number of layers and band width of horizontal model; Typically greater than 2,000 elements by 30 layers are possible on PRIME	2-D contour plotting available; 3-D version in preparation; restart in preparation for 3-D version	2-D large set of test problems; 3-D could be prepared within 6 to 8 weeks
SWENT	Source code is in-house	CRAY, CYBER/ (Adheres closely to Fortan 77)	No	Yes	Yes	Nonlinear	No	Yes	Yes	Yes	Yes	Yes	Finite difference iterative (direct for small problems)	1,000 grid blocks on CDC	Printer plotting	Six simple test problems
PRINCETON TRANSPORT CODE	Source code in-house	PRIME 750 (Fortran 77)	Yes	No	No	Nonlinear	No	No	Yes	Yes	No	No	Same as 3-D version of GEOFLOW	1,000 to 2,000 elements by 30 layers	Limited plotting	Small set of test problems
McDONALD/ HARBAUGH CODE	Source code in-house	PRIME 750 (Fortran 77)	Yes	No	No	No	No	No	No	Yes	No	No	Finite difference iterative - strongly implicit	3,000+ cells	Limited plotting	Large set of test problems

(a)Particle tracking algorithms could be written for any flow code. However, only SWIFT III has this algorithm as of this date.

TABLE 3.2.2

HYDROGEOLOGIC PARAMETERS
COMPILED FROM VARIOUS SOURCES(a)

SOURCE OF DATA	TRANSMISSIVITY T (gpd/ft)(b)	STORAGE COEFFICIENT S	HORIZONTAL HYDRAULIC CONDUCTIVITY K_h (gpd/ft^2)	VERTICAL HYDRAULIC CONDUCTIVITY K_v (ft/day)	THICKNESS-AQUIFER (ft)	THICKNESS-TILL (ft)	THICKNESS-CLAY (ft)	VERTICAL HYDRAULIC CONDUCTIVITY K_v (CLAY)	RECHARGE(c) (FROM PRECIPITATION) (in/yr)	RECHARGE (FROM RIVER) (gpd/acre)	RECHARGE (FROM INFLOW)
Dove 1961 (SOWC well field; Ross, Ohio)	370,000	0.2	2,466		150				12	240,000	
Smith 1962(d)/Klaer 1968(d) (Cincinnati Bolton well field)	158,000 258,000 400,000	0.23	2,460 900						21		200,000 gpd/mile of valley wall(k)
Kazmann 1950(d) (SOWC well field; Ross, Ohio)	285,600 331,200		2,380 2,760		120-125						
Schaefer & Walton 1956 (SOWC well field; Ross, Ohio)	292,800 333,600		2,440 2,780		120				8.5		
Klaer & Kazmann 1943 (Hamilton south well field)	350,000 450,000		2,340-2,420		>150 >202						
Spieker & Norris 1962 (unpub.) (FMPC site; Fernald, Ohio)	140,000(e) LA(f) 300,000 (combined) (III)	0.01(e)(g) 0.001 (III)	2,000 (III)		70 LA (III)		12 (III)	2.5 gpd/ft^2 (III) 3.2 gpd/ft^2 (III)			
Lewis 1968 (SOWC well field; Ross, Ohio)			2,500 3,020								
Smith 1960(d) (New Miami-Mouth of Four-Mile Creek)	521,000	0.0003(e)	5,790		90						
Smith 1962(d) (Ross Township Section 16 - West Bank, Great Miami River)	676,000		4,000		178						
Spieker 1968a (Hamilton south well field)	300,000-500,000 35,000(e)-300,000 (III)	0.2 0.002-0.01 (III)	2,000-3,340(h)		150-200 (UA = 70-120, LA = 70-90)(i)(j)	>50 (III)	10 (III)				
GeoTrans 1985 (Compilation from various sources)	150,000 LA (III)		350 ft/day 250 ft/day (III)	35 2.5 (III)			10-20 (III)	0.025 ft/day (III)	15 6 (III)		
Spieker 1968b (Hamilton south well field)	400,000 300,000 (III)	0.2 0.1 (III)			150-250 200 (III)				15.2(k) 6-21(k)	24,000-500,000	38 gpd/linear foot of valley wall (k)

Notes:

(a)Parameters refer to Hydrogeologic Environment I unless otherwise specified with (III) which refers to Hydrogeologic Environment III (refer to Figure 2.1-12).

(b)Conversion from gallons per day (gpd) to feet^3/day approximately x 7.481.

(c)Recharge generally occurs during the months of October, November, March, and April (Spieker 1968a).

(d)Taken from GeoTrans, 1985.

(e)Values were not used in calculations because they are either suspect or are not directly relevant to this problem.

(f)LA = Lower aquifer, Area III.

(g)GeoTrans, 1985 gives S = 0.01 from Spieker & Norris, 1962. The reference itself gives S = 0.001 for the lower aquifer in Hydrogeologic Unit Area III.

(h)Calculated from transmissivity and aquifer thickness values.

(i)UA = Upper aquifer, Area III.

(j)Thicknesses from cross sections.

(k)Values are given for the entire area, including Areas I and III.

TABLE 3.2.3
COMPARISON OF APRIL 1986 OBSERVED GROUND WATER LEVELS AND SWIFT III COMPUTED VALUES

WELL IDENTIFICATION	SWIFT III(a) COLUMN I	ROW J	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
12-7A	3	32	527.4	529.8	-2.4	2.4
STATE 16	4	41	526.3	529.9	-3.6	3.6
H-124	5	8	505.8	506.6	-0.8	0.8
H-122	5	26	521.4	524.3	-2.9	2.9
RE	5	17	515.7	517.6	-1.9	1.9
H-115	5	18	518.7	518.5	0.2	0.2
7-8A	6	25	520.7	523.1	-2.4	2.4
H-105	6	32	523.4	525.8	-2.4	2.4
BPH	6	35	525.0	526.8	-1.8	1.8
FMPC-17	7	31	523.9	525.0	-1.1	1.1
H-129	9	6	507.7	504.9	2.8	2.8
H-126	8	10	512.6	511.4	1.2	1.2
HK-	9	29	521.5	523.8	-2.3	2.3
FMPC-16-S	9	34	524.8	525.6	-0.8	0.8
FMPC-9	9	38	526.4	526.9	-0.5	0.5
DE	10	17	519.9	519.2	0.7	0.7
FMPC-18-S	10	37	525.3	526.2	-0.9	0.9
PALLET CO	9	42	525.4	530.1	-4.7	4.7
16-1-S	10	3	504.3	504.5	-0.2	0.2
FMPC-3	11	40	524.9	527.9	-3.0	3.0
FMPC-14-D	12	39	524.1	526.7	-2.6	2.6
IT-5	13	28	522.7	523.1	-0.4	0.4
IT-1	14	33	523.2	523.9	-0.7	0.7
FMPC-10	14	39	523.6	526.1	-2.5	2.5
BLK or IT-3	16	25	522.5	522.3	0.2	0.2
IT-2	18	33	522.9	522.8	0.1	0.1
FMPC-13-5	18	38	522.9	523.8	-0.9	0.9
02-E	18	26	522.2	521.9	0.3	0.3
IT-3	20	36	522.5	522.7	-0.2	0.2
H-127	21	29	522.3	521.3	1.0	1.0
IT-4	24	34	521.9	521.4	0.5	0.5
B-3	24	34	520.2	521.4	-1.2	1.2
B-2	28	32	520.6	520.2	0.4	0.4
SW-4A	29	25	519.1	518.0	1.1	1.1
B-1	29	31	521.3	519.6	1.7	1.7
SW-3A	31	24	518.3	515.0	3.3	3.3
R-7	34	26	520.7	514.1	6.6	6.6
B-4	31	28	520.3	518.2	2.1	2.1
K-4	40	27	520.6	520.5	0.1	0.1
EL-1	42	30	522.9	520.7	2.2	2.2
03	42	38	526.5	522.3	4.2	4.2
ER-1	45	22	525.9	523.3	2.6	2.6
LB-1	48	30	522.3	523.9	-1.6	1.6

TABLE 3.2.3
(Continued)

WELL IDENTIFICATION	SWIFT III COLUMN I	SWIFT III ROW J	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
WK1	49	24	525.5	523.8	1.7	1.7
WW-1	48	35	528.2	524.4	3.8	3.8

SUM OF DIFFERENCES/NUMBER OF OBSERVATIONS	0.11 feet
SUM OF ABSOLUTE DEVIATIONS/NUMBER OF OBSERVATIONS	1.74 feet
STANDARD DEVIATION OF DIFFERENCES	2.23 feet

(a)Refer to Figure A-2 for grid cell locations.

TABLE 3.2.4
COMPARISON OF APRIL 1986 OBSERVED GROUND WATER LEVELS AND GEOFLOW COMPUTED VALUES

WELL IDENTIFICATION	GEOFLOW ELEMENT NUMBER (a)(b)	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
12-7A	50	527.4	531.0	-3.6	3.6
STATE 16	86	526.3	531.0	-4.7	4.7
H-124	91	505.8	504.5	1.3	1.3
H-122	109	521.4	523.8	-2.4	2.4
RE	140	515.7	516.0	-0.3	0.3
H-115	101	518.7	516.5	2.2	2.2
7-8A	189	520.7	522.0	-1.3	1.3
H-105	196	523.4	525.0	-1.6	1.6
BPH	199	525.0	526.5	-1.5	1.5
FMPC-17	238	523.9	525.0	-1.1	1.1
H-129	256	507.7	505.0	2.7	2.7
H-126	260	512.6	511.0	1.6	1.6
HK-	278	521.5	523.5	-2.0	2.0
FMPC-16-S	283	524.8	525.5	-0.7	0.7
FMPC-9	288	526.4	527.0	-0.6	0.6
DE	309	519.9	517.0	2.9	2.9
FMPC-18-S	364	525.3	526.0	-0.7	0.7
PALLET CO	334	525.4	529.0	-3.6	3.6
16-1-S	296	504.3	504.0	0.3	0.3
FMPC-3	368	524.9	528.0	-3.1	3.1
FMPC-14-D	399	524.1	526.0	-1.9	1.9
IT-5	421	522.7	523.0	-0.3	0.3
IT-1	458	523.2	524.0	-0.8	0.8
FMPC-10	464	523.6	526.5	-2.9	2.9
BLK or IT-3	547	522.5	522.0	0.5	0.5
IT-2	589	522.9	523.0	-0.1	0.1
FMPC-13-5	593	522.9	524.0	-1.1	1.1
O2-E	615	522.2	522.0	0.2	0.2
IT-3	660	522.5	523.0	-0.5	0.5
H-127	727	522.3	522.0	0.3	0.3
IT-4	806	521.9	522.0	-0.1	0.1
B-3	841	520.2	522.0	-1.8	1.8
B-2	948	520.6	521.0	-0.4	0.4
SW-4A	978	519.1	521.0	-1.9	1.9
B-1	1015	521.3	521.0	0.3	0.3
SW-3A	1040	518.3	517.0	1.3	1.3
R-7	1158	520.7	519.0	1.7	1.7
B-4	1187	520.3	520.0	0.3	0.3
K-4	1277	520.6	522.0	-1.4	1.4
EL-1	1343	522.9	522.0	0.9	0.9
O3	1351	526.5	524.0	2.5	2.5
ER-1	1375	525.9	524.0	1.9	1.9
LB-1	1449	522.3	524.0	-1.7	1.7

TABLE 3.2.4
(Continued)

WELL IDENTIFICATION	GEOFLOW ELEMENT NUMBER (a)(b)	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
WK1	1463	525.5	524.0	1.5	1.5
WW-1	1473	528.2	525.5	2.7	2.7

SUM OF DIFFERENCES/NUMBER OF OBSERVATIONS	-0.39 feet
SUM OF ABSOLUTE DEVIATIONS/NUMBER OF OBSERVATIONS	1.50 feet
STANDARD DEVIATION OF DIFFERENCES	1.80 feet

(a)Wells are located in upper left corner of element.

(b)Refer to Figure A-2 for grid element locations.

TABLE 3.2.5

SUMMARY OF MODEL INPUT PARAMETERS - INITIAL AND RANGE OF VALUES FOR SENSITIVITY ANALYSIS

	ZONE I		ZONE III		
	RANGE OF VALUES FOR SENSITIVITY ANALYSIS	INITIAL VALUES	RANGE OF VALUES FOR SENSITIVITY ANALYSIS	INITIAL VALUES	NOTES AND REFERENCES
Aquifer Values					
Porosity	0.2 - 0.3	0.25	0.2 - 0.3	0.25	Estimated from geologic logs and site experience
S (storage)	N/A(a)	0.2	N/A	0.2/.05(b)	Mean of various sources
K_h (horizontal hydraulic conductivity) (feet/day)	100 - 800	400	100 - 800	400	Statistically derived from various sources
K_v (vertical hydraulic conductivity) (feet/day)	$K_h/10 - K_h$	$K_h/3$	$K_h/10 - K_h$	$K_h/3$	Estimated from site experience
b (aquifer thickness) (feet)	N/A	Variable	N/A	Variable	From contour plots of bedrock and top of aquifer
Recharge (from precipitation) (inches/year)	8 - 20	14	3 - 12	6	
Daily pumping ($feet^3$/day):					
- FMPC wells	N/A	N/A	N/A	64,000	
- Southwestern Ohio Water Company wells					
(Collector 1)	N/A	1,600,000	N/A	N/A	Estimated
(Collector 2)	N/A	800,000	N/A	N/A	Estimated
- Albright and Wilson well	N/A	N/A	N/A	19,000	Information obtained by IT for 1986

TABLE 3.2.5
(Continued)

	ZONE I		ZONE III		
	RANGE OF VALUES FOR SENSITIVITY ANALYSIS	INITIAL VALUES	RANGE OF VALUES FOR SENSITIVITY ANALYSIS	INITIAL VALUES	NOTES AND REFERENCES
River stage (feet MSL)	520.5 - 528.6	524 at Ross Bridge	N/A	N/A	
River water temperature (°F)	N/A	60	N/A	N/A	
Ground water temperature (°F)	N/A	54	N/A	54	
Interbed Values					
K_v clay (vertical hydraulic conductivity of clay) (feet/day)	N/A	N/A	3×10^{-5} - 3×10^{-2}	3×10^{-3}	Estimated
b clay (thickness of interbed) (feet)	N/A	N/A	10 - 25	20	
Porosity of clay	N/A	N/A	0.45 - 0.75	0.6	Assumed (Freeze & Cherry, 1979, pg. 37)
Bedrock Values					
K_hB (horizontal hydraulic conductivity of bedrock) (feet/day)	3×10^{-5} - 3×10^{-1}	3×10^{-3}	3×10^{-5} - 3×10^{-1}	3×10^{-3}	Estimated
Porosity of bedrock	0 - 0.1	0.1	0 - 0.1	0.1	Assumed (Freeze & Cherry, 1979, pg. 37)

(a)N/A = Not applicable.
(b)0.2/0.05 - The first value refers to the upper aquifer and the second value refers to the lower aquifer.

TABLE 3.2.6

SUMMARY OF GEOFLOW SENSITIVITY RUNS

	CALIBRATION RUN	SENSITIVITY RUN NUMBERS 1	2	3	4	5	6	7
Input Parameter								
Aquifer hydraulic conductivity (feet/day)								
Zone 1	400	400	400	400	400	400	400	400
Zone 2	-	-	-	-	-	-	-	-
Zone 3	300	300	300	300	300	300	300	300
Zone 4	150	150	150	150	150	150	150	150
Recharge from precipitation (inches/year)								
Zone 1	14	20	10	14	14	14	14	14
Zone 2	-	-	-	-	-	-	-	-
Zone 3	6	8	4	6	6	6	6	6
Zone 4	6	8	4	6	6	6	6	6
River bed leakage (factor) (day^{-1})	0.35(1.0)	0.35(1.0)	0.35(1.0)	0.035(0.1)	0.175(0.5)	0.70(2.0)	3.5(10.0)	35.0(100.0)
Comparison of observed versus computed results (Model Output) (feet)								
All 45 Wells:								
Mean difference(a)	-0.39	-0.89	+0.07	+3.52	+0.29	-0.76	-1.10	-1.17
Mean absolute difference(b)	1.50	1.81	1.42	3.87	1.66	1.73	2.03	2.10
Standard deviation of algebraic difference(c)	1.80	1.92	1.81	3.50	2.03	1.91	2.00	2.02

TABLE 3.2.6
(Continued)

	CALIBRATION RUN	SENSITIVITY RUN NUMBERS 1	2	3	4	5	6	7
Wells 12-7A, State 16, PALLET CO, FMPC-3, and WW-1 excluded:								
Mean difference(a)	-0.14	-0.63	+0.28	+4.01	+0.63	-0.54	-0.89	-0.97
Mean absolute difference(b)	1.24	1.55	1.24	4.03	1.41	1.52	1.83	1.91
Standard deviation of algebraic difference(c)	1.47	1.64	1.60	3.23	1.63	1.70	1.82	1.85

(a)Sum of observed minus computed values divided by number of wells.

(b)Sum of absolute value of observed minus computed value divided by number of wells.

(c)Standard deviation of observed minus computed values.

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TABLE 3.2.7

WATER BALANCE FROM MODEL OUTPUT FOR COLLECTOR WELL DISCHARGE

Water Balance

$$Q_D = Q_P + Q_{BR} + Q_R + Q_A$$

where:

Q_D = Pumping discharge from collector wells

Q_P = Component from precipitation recharge

Q_{BR} = Component from bedrock aquifer

Q_R = Component from entire river

Q_A = Component from sand and gravel aquifer

Model Sensitivity Cases

Case 1 = River leakage factor for model calibration run = 0.35 day^{-1}
Case 2 = River leakage factor 0.5 times calibration factor = 0.175 day^{-1}
Case 3 - River leakage factor ten times calibration factor = 3.5 day^{-1}

Results

$$Q_D = Q_R + (Q_{BR} + Q_P + Q_A)$$

Case 1 Q_D = 76 percent + 24 percent(a)

Case 2 Q_D = 72 percent + 28 percent(a)

Case 3 Q_D = 82 percent + 18 percent(a)

(a)Q_P = Six inches/year in Model Area III, 14 inches/year Model Area I, based on model calibration

Q_{BR} = Assumed zero for model, however, has been estimated to be 38 gpd per linear foot of valley wall (Spieker, 1986b)

Q_A = Will be determined in future modeling studies

TABLE 5.4.1

RIVER SAMPLING LOCATIONS

LOCATION NO.	LOCATION DESCRIPTION(a)	NUMBERS AND TYPES OF SAMPLES COLLECTED
1	4,400 feet downstream of FMPC outfall in channel center	2 discrete water samples
2	3,600 feet downstream of FMPC outfall, right center of channel	2 discrete water samples 1 composite sediment sample
3	2,300 feet downstream of FMPC outfall, right center of channel, 200 feet downstream of gravel bar in channel center	2 discrete water samples 1 composite sediment sample
4	1,600 feet downstream of FMPC outfall, center of channel to right of gravel bar, 100 feet upstream of gas pipeline crossing	2 discrete water samples
5	900 feet downstream of FMPC outfall, channel on right side of gravel bar	2 discrete water samples 1 composite sediment sample
6	200 feet downstream of FMPC outfall, at five equally-spaced channel subdivisions	10 discrete water samples from 2 sampling traverses 2 composite water samples from 2 sampling traverses
7	600 feet upstream of FMPC outfall, right center of channel	2 discrete water samples 1 composite sediment sample
8	3,200 feet upstream of FMPC outfall, right center of channel	2 discrete water samples 1 composite sediment sample
9	5,600 feet upstream of FMPC outfall, left center of channel	2 discrete water samples 1 composite sediment sample
10	7,000 feet upstream of FMPC outfall, channel center	2 discrete water samples 1 composite sediment sample
11	8,000 feet upstream of FMPC outfall at Ross Bridge, at five channel subdivisions, spacing weighted by discharge	3 composite water samples from 3 sampling traverses

(a)Orientation of view for sampling location descriptions--looking downstream.

TABLE 5.4.2

MEASURED CURRENT VELOCITIES
IN MAIN PORTIONS OF CHANNEL
GREAT MIAMI RIVER, SEPTEMBER 12-13, 1987

LOCATION NO.(a)	MEASURED CURRENT VELOCITY (feet/sec)(b)	MEASURED CHANNEL DEPTH (feet)	DEPTH OF VELOCITY MEASUREMENT (feet)
1	0.50	9.1	5.5
2	2.10	2.9	1.7
3	1.50	5.3	3.2
4	1.35	2.5	1.5
5	-(c)	3.4	-
6	-(c)	1.3 to 4.1(d)	-
7	0.55	5.3	3.2
8	0.45	7.0	4.2
9	0.50	7.3	4.4
10	0.50	5.4	3.2
11	0.10 to 0.75(e)	1.0 to 6.6(e)	0.6 to 4.0

(a)Refer to Figure 5.1-1 for sampling locations.

(b)All velocity measurements obtained at sixth-tenths the channel depth.

(c)Water in meter - Not functioning properly.

(d)Five measurements approximately 25 feet apart across channel width; mean = 2.8 feet.

(e)Twenty-one measurements approximately 12-1/2 feet apart across channel width; mean velocity - 0.40 feet/sec; mean depth - 3.2 feet; total discharge at section = approximately 400 $feet^3$/sec.

TABLE 5.4.3

DESCRIPTIONS OF BOTTOM SEDIMENTS
AT RIVER SEDIMENT SAMPLING LOCATIONS

LOCATION NO.(a)	MAJOR SEDIMENT CHARACTERISTICS
2	Clean medium sand, with little (up to 20%) coarse sand, decreasing amounts of coarse sand to medium gravel towards main channel; increasing amounts of fine sand and traces of silt towards eddy pool
3	Clean, well-graded gravel (rounded pebbles up to 1/2-inch diameter) with no fine sand or silt
5	Primarily rounded cobbles and flat stones (up to five inches in size) with sediment between consisting of well-graded silty sand
7	Clean, well-graded medium to coarse sand with some fine gravel, trace to no silt
8	Well-graded medium to coarse sand with some coarse gravel (up to 1.5 inches in diameter), trace fine sand
9	Very soft silty organic muck, with trace to little medium sand

(a)Refer to Figure 5.1-1 for sampling locations.

TABLE 5.4.4

QUALITATIVE ASSESSMENT OF CHANNEL STABILITY

LOCATION NO.(a)	MAJOR CHANNEL CONFIGURATION	COMPARISON WITH PREVIOUSLY DOCUMENTED CONFIGURATION (DOVE, 1961)
1	90-degree turn to south, gravel bar to inside, depth greater than six feet over majority of width, deeper to outside of bend.	No cross section available.
2	Main portion of channel center-right (looking downstream), edd, pool to left, depths approximately three feet.	Present channel shallower, position of main channel appears stable.
3	Main portion of channel center-right (looking downstream), depths less than three feet.	Similar.
4	Main portion of channel to right (looking downstream), major gravel bar divides channel upstream of location.	Gravel bar does not appear on previously documented sections.
5	Main channel to right (looking downstream) of prominent gravel bar.	Gravel bar not documented in past, but main channel consistently to right.
6	Main channel to right, depths up to four feet.	Similar.
7	Main channel to right-center.	Similar.
8	Main channel in approximate center, depths up to seven feet.	Shallower in previously documented sections.
9	Main channel to left-center (outside of bend), depths up to seven feet or greater.	Similar.

TABLE 5.4.4
(Continued)

LOCATION NO.	MAJOR CHANNEL CONFIGURATION	COMPARISON WITH PREVIOUSLY DOCUMENTED CONFIGURATION (DOVE, 1961)
10	Main channel in approximate center, depths five to six feet.	Similar.
11	Main channel to left-center (looking downstream), depths up to six and one-half feet.	Similar position for main portion of channel, but previous depths are shallower.

(a)Refer to Figure 5.1-1 for sampling locations.

FIGURES

DRAWING NUMBER 303317-B10

DRAWN BY	J. LOGRECO	9-13-87
CHECKED BY		
APPROVED BY		

100% BLUE 206 50% BLK 198 198

LEGEND

- FMPC EFFLUENT LINE DISCHARGE POINT
- BACKGROUND SURFACE WATER SAMPLING POINT (ROSS BRIDGE)
- RIVER FLOW DIRECTION
- MANHOLE

REFERENCES:

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO - IND., SHANDON, OHIO GREENHILLS, OHIO, CINCINNATI WEST, OHIO, ADDYSTON, OHIO - KY, AND HOOVEN OHIO. DATED: 1955 (PRI1981), 1965 (PRI1981), 1965 (PRI1981), 1961 (PRI1981), 1982 AND 1981 SCALE: 1" = 2000'.

SCALE

0 2000 4000 FEET

FIGURE I. I-1

FMPC RI/FS AREA OF INFLUENCE STUDY AREA

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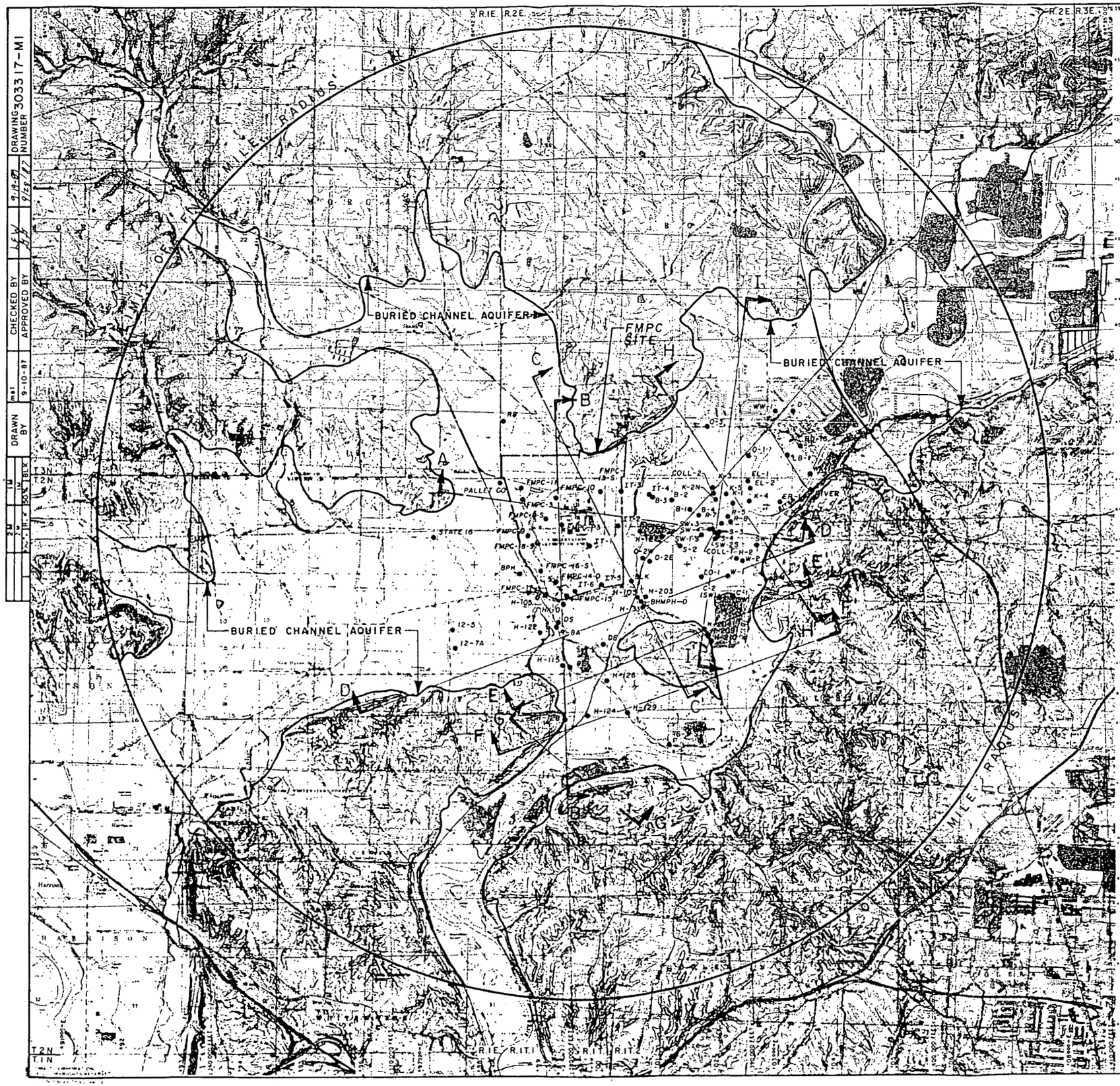

DRAWN BY	mel	9-10-87	CHECKED BY	JFW	9-19-87	DRAWING NUMBER 303317-M1
			APPROVED BY		9/29/87	

LEGEND:

- ● EL-2 WELL NAME AND LOCATION
- ▲ RIVER RIVER ELEVATION LOCATION (ROSS BRIDGE)

NOTES:

1. WELL BLK ALSO KNOWN AS M-106 WELL BHMPH-D ALSO KNOWN AS M-106.

REFERENCES

BASED ON MEASUREMENTS CONDUCTED BY IT CORPORATION IN APRIL, 1985 AND BY THE SOUTHWESTERN OHIO WATER COMPANY ON MARCH 27, 1986

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO-IND., SHANDON, OHIO, GREENHILLS, OHIO; CINCINNATI WEST, OHIO, ADDYSTON, OHIO-KY., AND HOOVEN, OHIO DATED 1955 (PR1981), 1965 (PR1981), 1965 (PR1981), 1961 (PR1981), 1982, AND 1981
SCALE 1:24000

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO 303063-M5, PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-1

SELECTED WELL AND CROSS SECTION LOCATIONS

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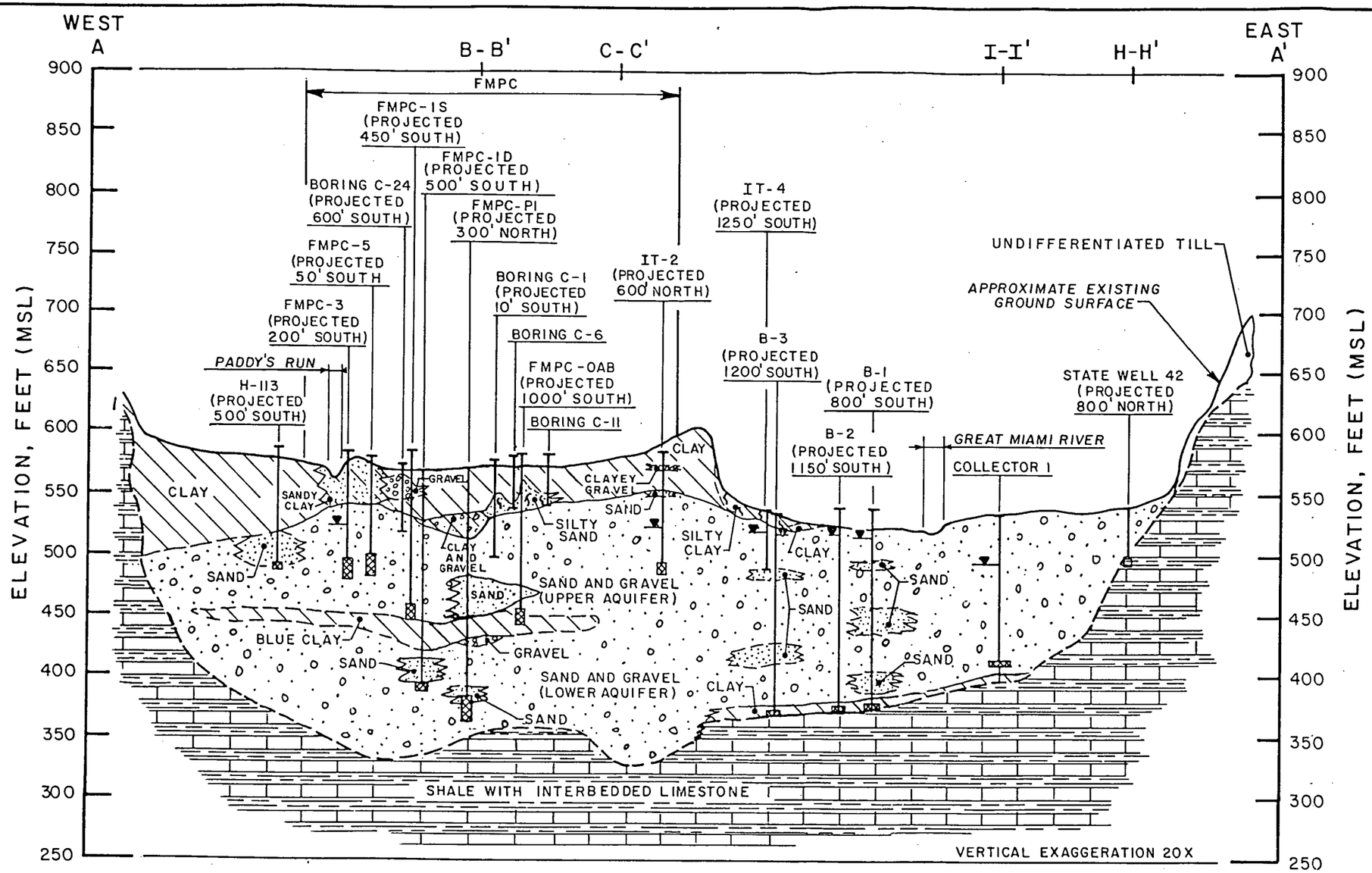

LEGEND:

OPEN INTERVAL

WATER LEVELS (MEASURED 3/27/86 THRU 4/11/86.)

REFERENCES:

MODIFIED FROM U.S.G.S. PROFESSIONAL PAPER No. 605-A USING AVALIABLE OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASKS A AND C REPORTS AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-B16. PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986.

CROSS SECTION A-A'

(LOOKING NORTH)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION A-A' SEE FIGURE 2.1-1.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

FIGURE 2.1-2

HYDROGEOLOGIC CROSS SECTION A-A'

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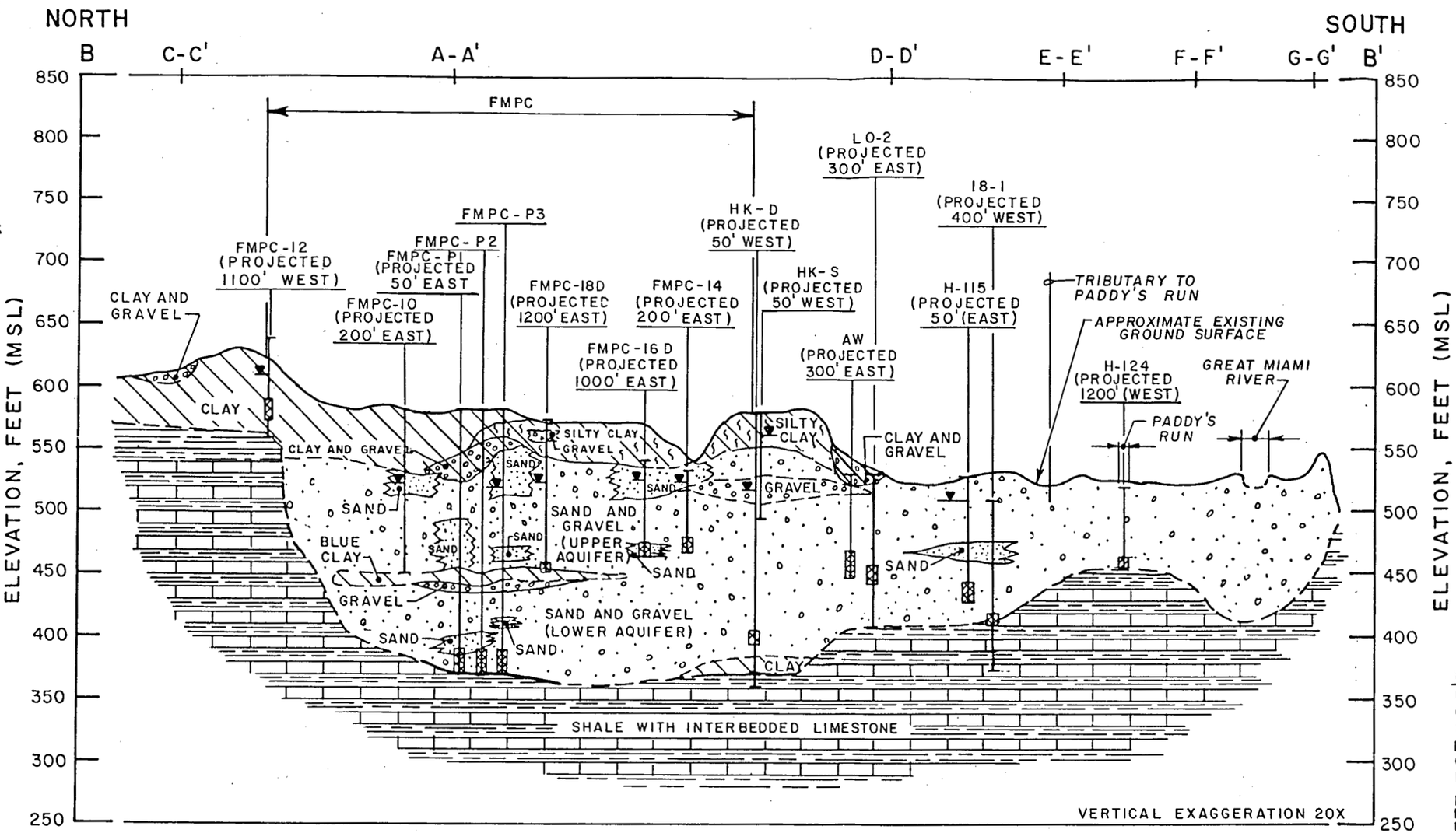

LEGEND:

OPEN INTERVAL

WATER LEVELS (MEASURED 3/27/86 TO 4/11/86.)

REFERENCES:

U.S.G.S PROFESSIONAL PAPER No. 605-A, OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASK C REPORT AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-B17. PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986.

CROSS SECTION B-B'

(LOOKING EAST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION B-B' SEE FIGURE 2.1-1.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

FIGURE 2.1-3

HYDROGEOLOGIC CROSS SECTION B-B'

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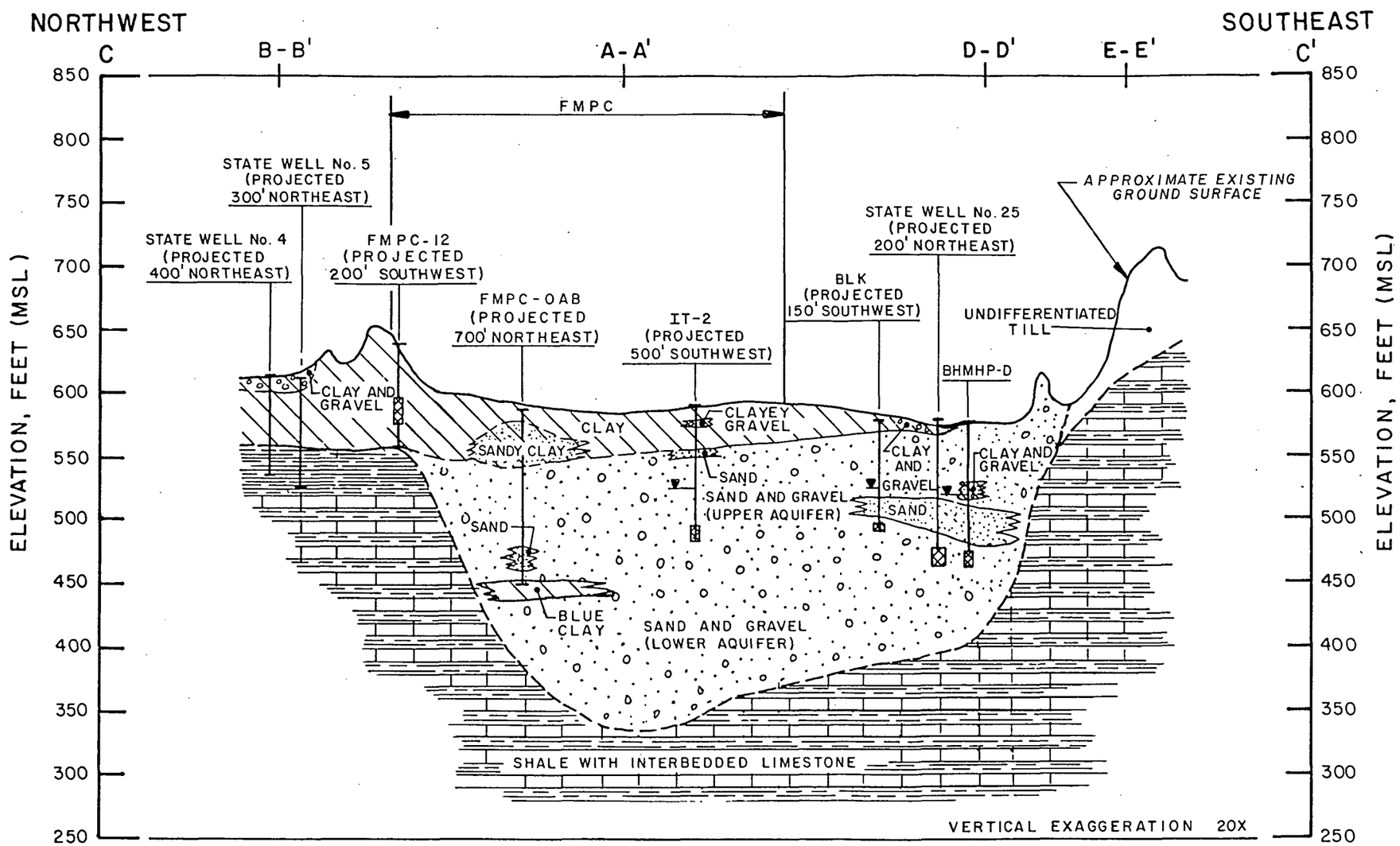

LEGEND:

OPEN INTERVAL

WATER LEVELS (MEASURED 3/27/86 THRU 4/11/86.)

REFERENCES:

U.S.G.S. PROFESSIONAL PAPER No. 605-A, OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASK C REPORT TASK C REPORT, AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-B18. PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986.

CROSS SECTION C-C'

(LOOKING NORTHEAST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION C-C' SEE FIGURE 2.1-1.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

FIGURE 2.1-4

HYDROGEOLOGIC CROSS SECTION C-C'

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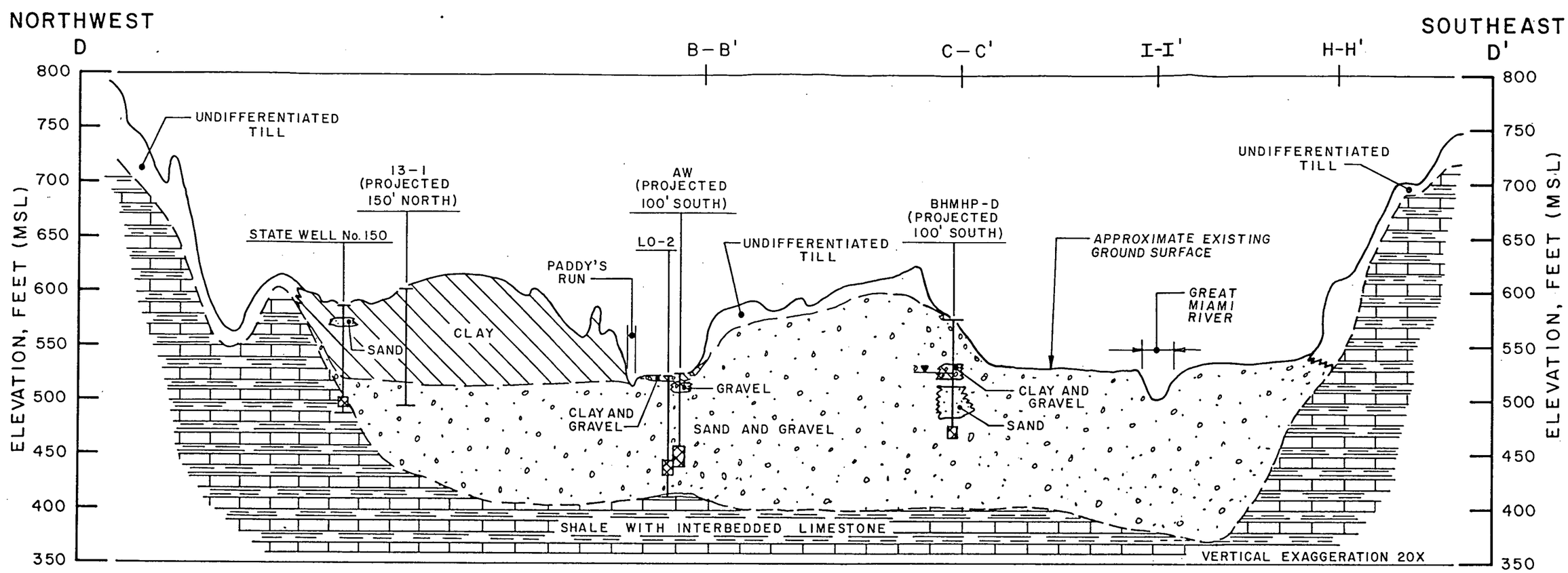

CROSS SECTION D-D'

(LOOKING NORTHWEST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

REFERENCES:

U.S.G.S. PROFESSIONAL PAPER No. 605-A, OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASK C REPORT AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-B19. PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

NOTE:

FOR LOCATION OF CROSS SECTION D-D' SEE FIGURE 2.1-1.

LEGEND

OPEN INTERVAL

WATER LEVELS (MEASURED 3/27/86 THRU 4/11/86.)

FIGURE 2.1-5

HYDROGEOLOGIC CROSS SECTION D-D'

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APPROVED BY		9/16/87

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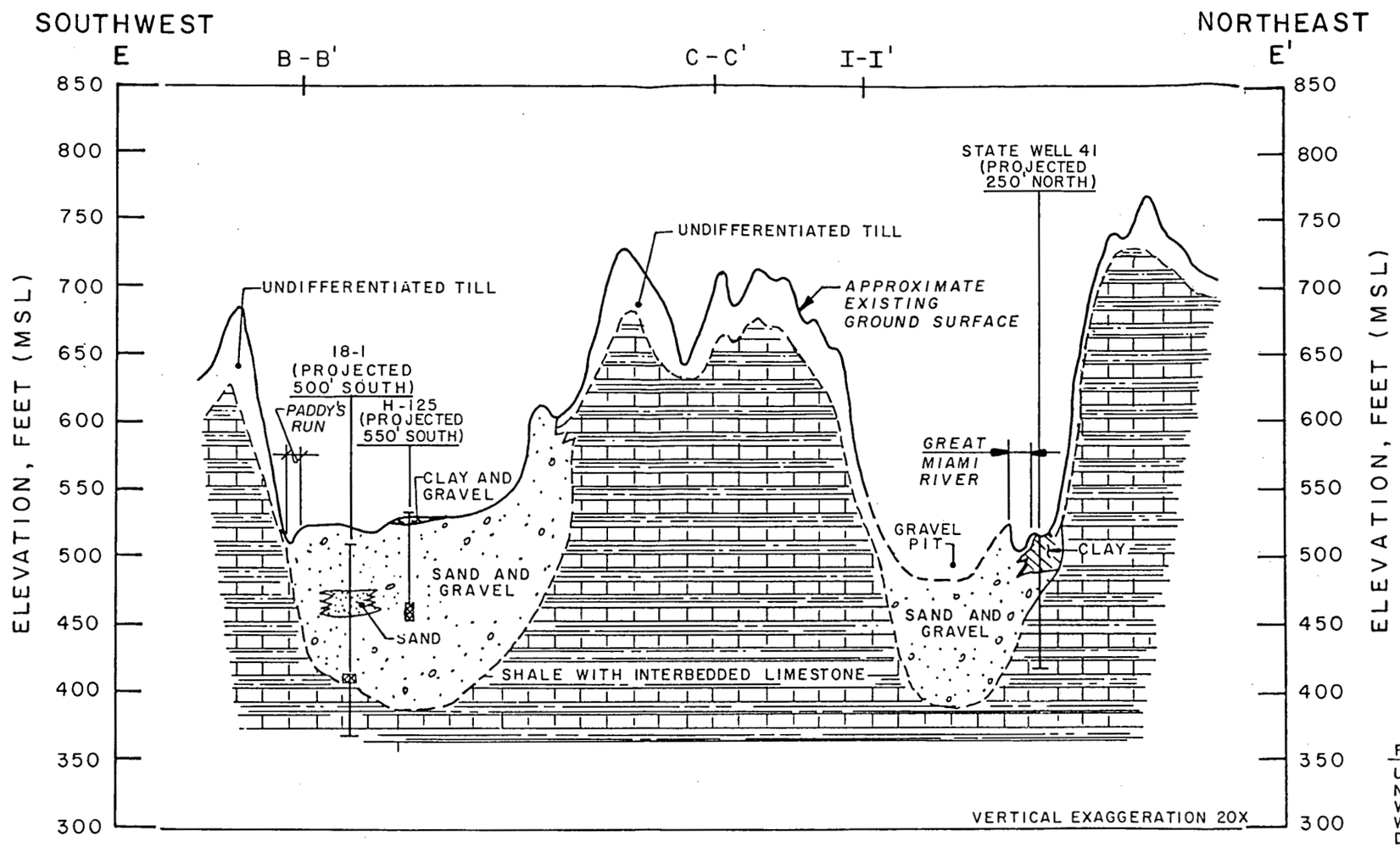

LEGEND

OPEN INTERVAL

WATER LEVELS (MEASURED 3/27/86 THRU 4/11/86.)

CROSS SECTION E-E'

(LOOKING NORTHWEST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION E-E' SEE FIGURE 2.1-1.

REFERENCES:

U.S.G.S. PROFFESSIONAL PAPER No.605-A, OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASK C REPORT AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-B20 PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION NOVEMBER .1986.

FIGURE 2.1-6

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

HYDROGEOLOGIC CROSS SECTION E-E'

PREPARED FOR

FERNALD RI/FS
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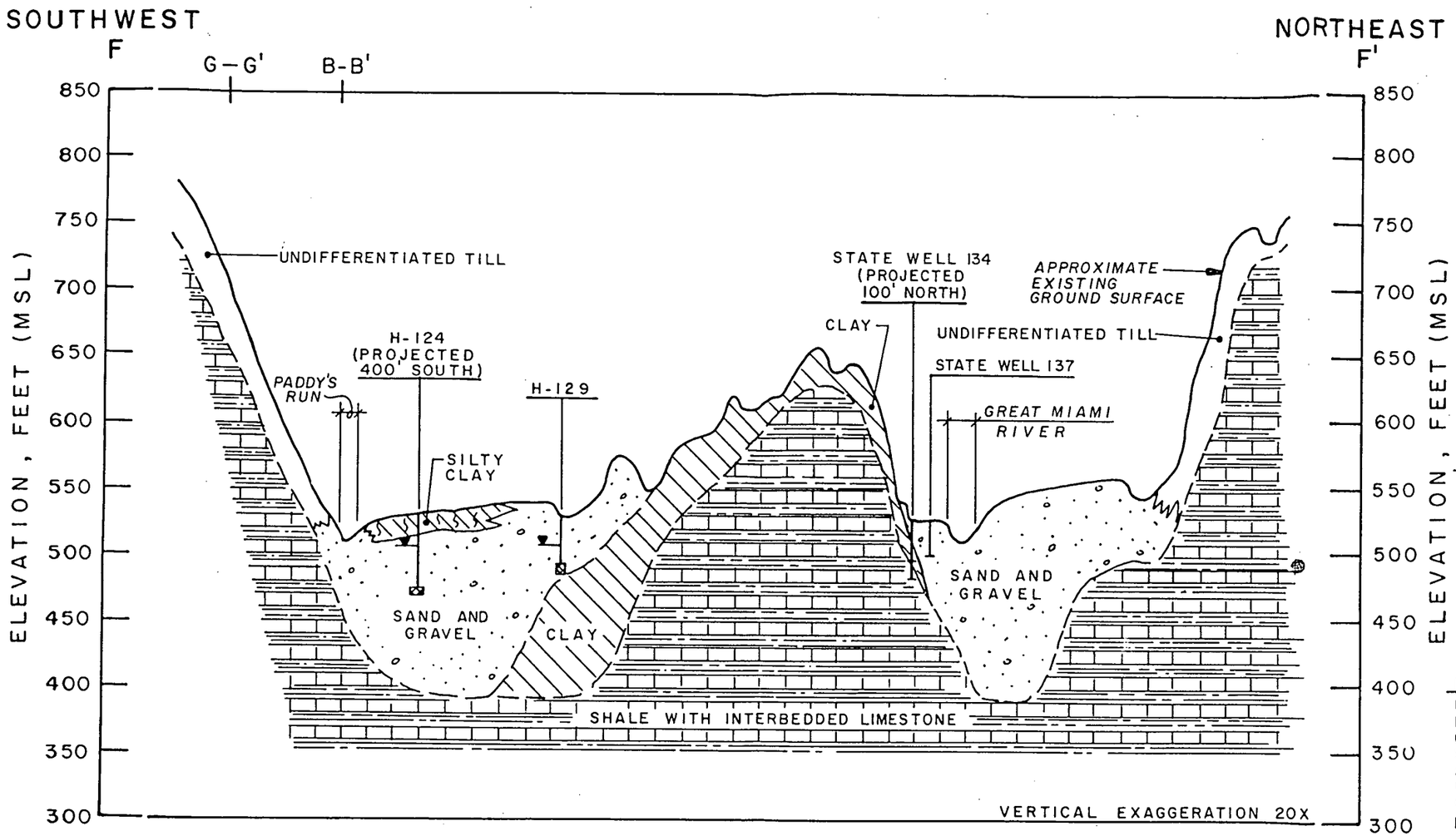
		APPROVED BY	9/29/87		



LEGEND

OPEN INTERVAL

WATER LEVEL (MEASURED 3/27/86 THRU 4/11/86.)

REFERENCES:

U.S.G.S. PROFESSIONAL PAPER No. 605-A USING AVAILABLE OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASK C REPORT, AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-B21. PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986.

CROSS SECTION F-F'

(LOOKING NORTHWEST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION F-F' SEE FIGURE 2.1-1.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

FIGURE 2.1-7

HYDROGEOLOGIC CROSS SECTION F-F'

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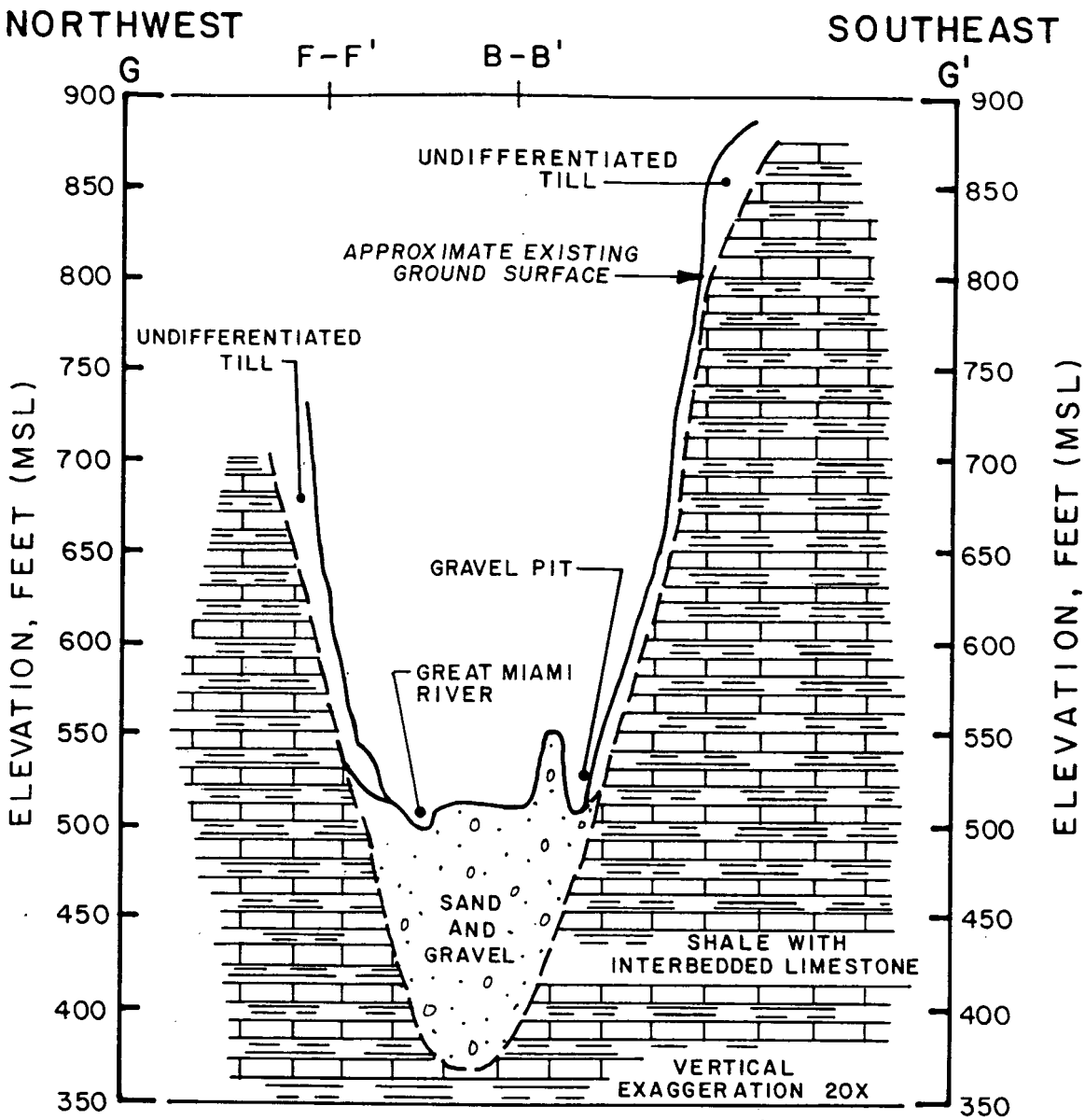

CROSS SECTION G-G'

(LOOKING NORTHEAST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:
FOR LOCATION OF CROSS SECTION G-G' SEE FIGURE 2.1-1.

REFERENCES:
U.S.G.S. PROFESSIONAL PAPER No.605-A, OHIO STATE WATER WELL RECORDS, SOWC WATER WELL DATA, DAMES AND MOORE DOE, FEED MATERIALS PRODUCTION CENTER GROUND WATER STUDY TASK C REPORT AND IT BORING LOGS.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO. 303063-A17. PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986.

FIGURE 2.1-8

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

HYDROGEOLOGIC CROSS SECTION G-G'

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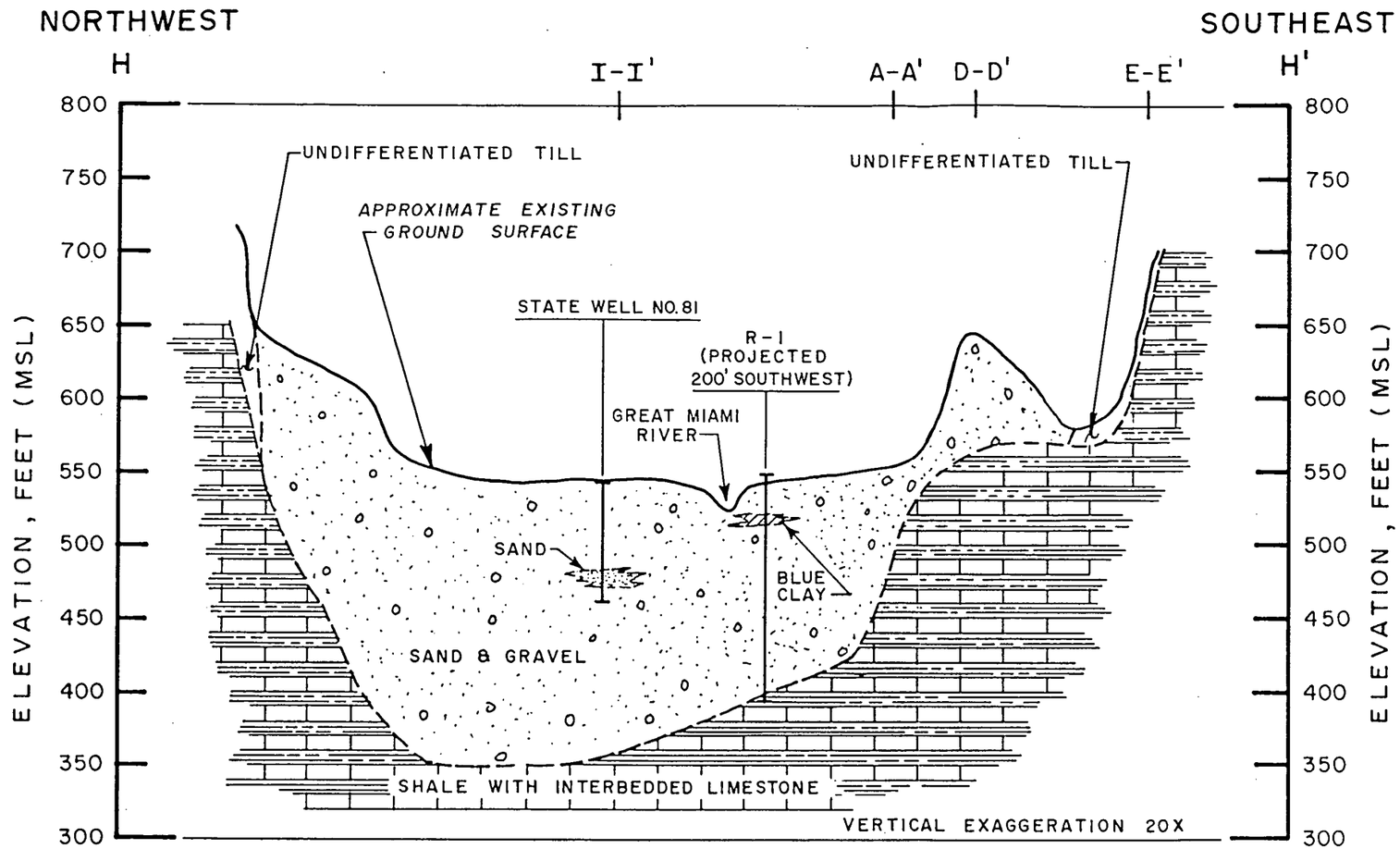

CROSS SECTION H-H'

(LOOKING NORTHEAST)

LEGEND

OPEN INTERVAL

REFERENCES:

U.S.G.S. PROFESSIONAL PAPER NO. 605-A, OHIO STATE WATER WELL RECORDS.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS.

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED.

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION H-H' SEE FIGURE 2.1-1.

FIGURE 2.1-9

HYDROGEOLOGIC CROSS SECTION H-H'

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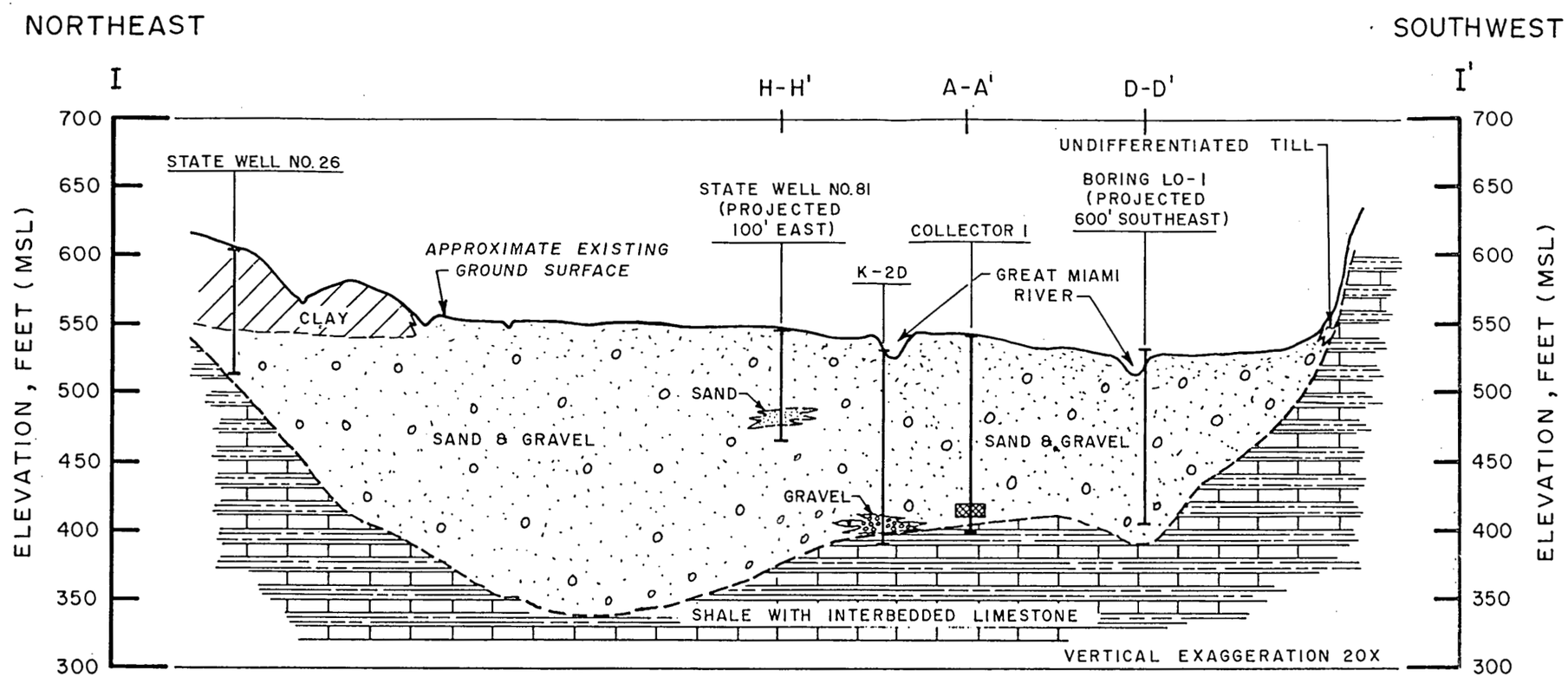

LEGEND:

OPEN INTERVAL

CROSS SECTION I-I'

(LOOKING EAST-SOUTHEAST)

VERTICAL SCALE

0 100 200 FEET

HORIZONTAL SCALE

0 2000 4000 FEET

NOTE:

FOR LOCATION OF CROSS SECTION I-I' SEE FIGURE 2.1-1.

REFERENCES:

OHIO STATE WATER WELL RECORDS, STATE OF OHIO DEPARTMENT OF NATURAL RESOURES DIVISION OF WATER TECHNICAL REPORT NO. 4, AND REPORT PREPARED FOR OHIO ENVIRONMENTAL PROTECTION AGENCY BY GEO-TRANS, INC.

THE BORING LOGS AND RELATED INFORMATION DEPICT SUBSURFACE CONDITIONS ONLY AT THE SPECIFIC LOCATIONS AND DATES INDICATED. SOIL CONDITIONS AND WATER LEVELS AT OTHER LOCATIONS MAY DIFFER FROM CONDITIONS OCCURRING AT THESE BORING LOCATIONS. ALSO THE PASSAGE OF TIME MAY RESULT IN A CHANGE IN THE CONDITIONS AT THESE BORING LOCATIONS

THE DEPTH AND THICKNESS OF THE SUBSURFACE STRATA INDICATED ON THE SECTIONS WERE GENERALIZED FROM AND INTERPOLATED BETWEEN THE TEST BORINGS. INFORMATION ON ACTUAL SUBSURFACE CONDITIONS EXISTS ONLY AT THE LOCATION OF THE TEST BORINGS AND IT IS POSSIBLE THAT SUBSURFACE CONDITIONS BETWEEN THE TEST BORINGS MAY VARY FROM THOSE INDICATED

FIGURE 2.1-10

HYDROGEOLOGIC CROSS SECTION I-I'

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DRAWING NUMBER 303317-M4

9-19-87

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LEGEND

700 — BEDROCK SURFACE CONTOURS CONTOUR INTERVAL AS SHOWN (FEET MSL)

REFERENCES

DWG TITLED: TOP OF ROCK MAP OF BUTLER, COUNTY OHIO, OPEN FILE MAP 224 STATE OF OHIO, DNR DIVISION OF GEOLOGICAL SURVEY BY JACK A LEON DATE JULY 1985, SCALE 1" = 4000' CONTOUR INTERVAL 50 FEET

DWG TITLED: TOP OF ROCK MAP OF HAMILTON COUNTY OHIO, OPEN FILE MAP 229 STATE OF OHIO, DNR DIVISION OF GEOLOGICAL SURVEY BY VORMELKER, JOEL DATE JULY 1985, SCALE 1" = 4000' CONTOUR INTERVAL 100 FEET

DWG TITLED: SEISMIC REFRACTION SURVEY OF PLIESTOCENE DRAINAGE CHANNELS IN THE LOWER GREAT MIAMI RIVER VALLEY, OHIO, U.S. GEOLOGICAL SURVEY PROFESSIONAL PAPER 605-B, 1971, BY J.S. WATKINS AND A.M. SPIEKER. CONTOUR INTERVAL 50 FEET.

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO—IND.; SHANDON, OHIO; GREENHILLS, OHIO; CINCINNATI WEST, OHIO; ADDYSTON, OHIO—KY.; AND HOOVEN, OHIO, DATED 1955 (PR 1981), 1965 (PR 1981), 1965 (PR 1981), 1961 (PR 1981), 1982 AND 1981. SCALE 1" = 2000'.

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO 303063-M19 PREPARED FOR U.S. DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986

SCALE

0 2000 4000 6000 FEET

FIGURE 2.1-11

TOP OF BEDROCK SURFACE CONTOUR MAP

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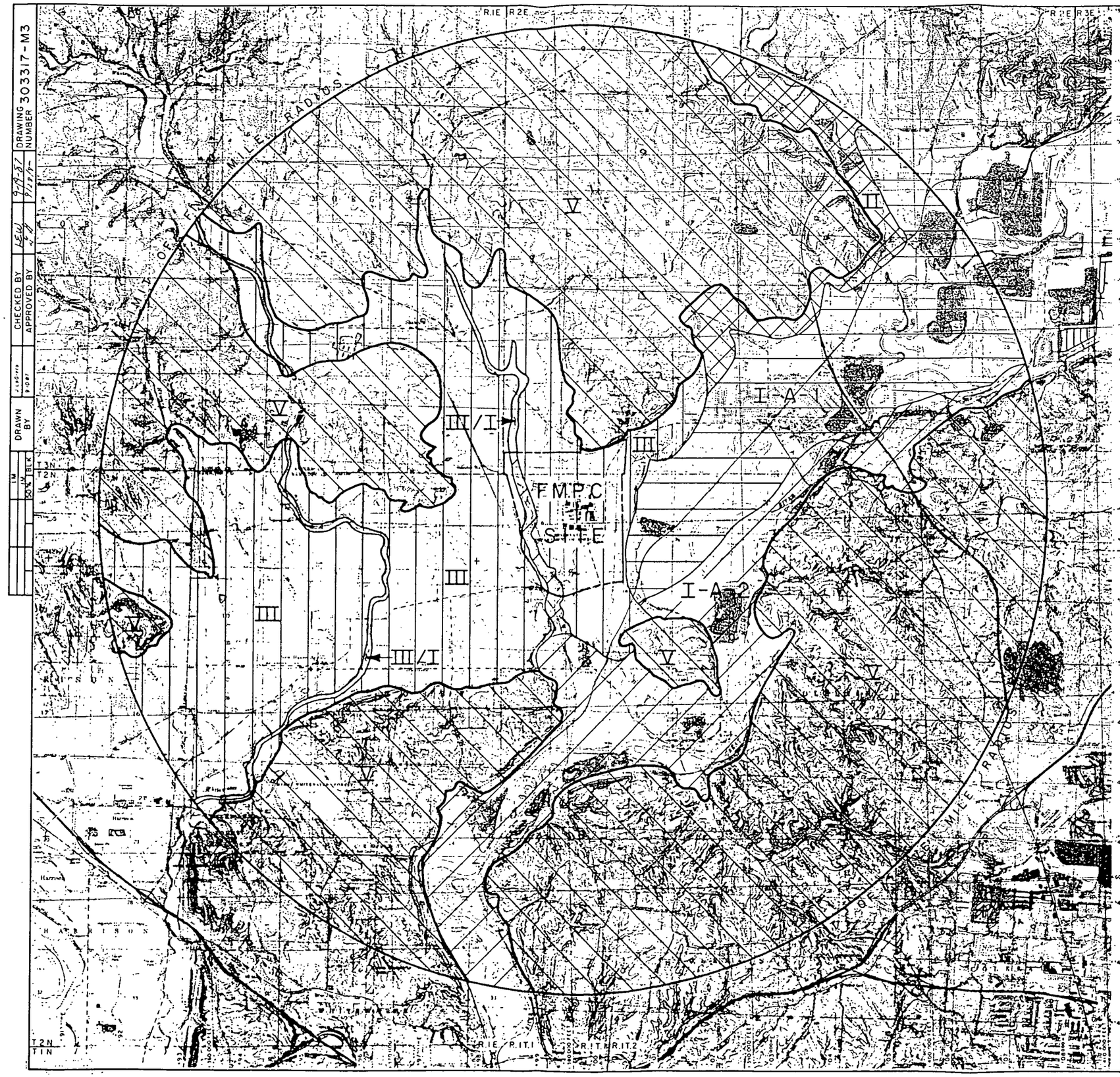

LEGEND

- I-A-1 — SAND AND GRAVEL AQUIFER, 150-200 FEET OR MORE THICK; NO AERIALLY EXTENSIVE CLAY LAYERS PRESENT; RECHARGE BY INDUCED STREAM INFILTRATION AVALIABLE
- I-A-2 — SAND AND GRAVEL AQUIFER, LESS THAN 150 FEET THICK, NO AERIALLY EXTENSIVE CLAY LAYERS PRESENT; RECHARGE BY STREAM INFILTRATION AVALIABLE
- II — SAND AND GRAVEL AQUIFER, LESS THAN 150 FEET THICK; NO AERIALLY EXTENSIVE CLAY LAYERS PRESENT; RECHARGE BY STREAM INFILTRATION NOT AVALIABLE.
- III — SAND AND GRAVEL AQUIFER, OVERLAIN BY CLAY; STREAM RECHARGE GENERALLY NOT AVALIABLE
- V — SHALE BEDROCK OVERLAIN BY 50 FEET OR LESS OF RELATIVELY IMPERMEABLE TILL
- III/I — STREAM VALLEYS POTENTIALLY [illegible] THROUGH TILL TO SAND AND GRAVEL AQUIFER
- FMPC SITE BOUNDARY

REFERENCES

MODIFIED FROM GROUND WATER HYDROLOGY AND GEOLOGY OF THE LOWER GREAT MIAMI RIVER VALLEY OHIO, U.S.G.S. REPORT NO. 605-A USING SOIL SURVEYS OF BUTLER COUNTY (1980) AND HAMILTON COUNTY (1982) AND IT FIELD OBSERVATIONS

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO-IND., SHANDON, OHIO, GREENHILLS, OHIO; CINCINNATI WEST, OHIO, ADDYSTON, OHIO-KY., AND HOOVEN, OHIO DATED 1959 (PR1981), 1965 (PR1981), 1965 (PR1981), 1961 (PR1981), 1982, AND 1981 SCALE 1" = 2000'

FIGURE ADAPTED FROM FERNALD LITIGATION DRAWING NO 303063-M18 PREPARED FOR U.S DEPARTMENT OF ENERGY BY IT CORPORATION, NOVEMBER, 1986

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-12

HYDROGEOLOGIC ENVIRONMENTS

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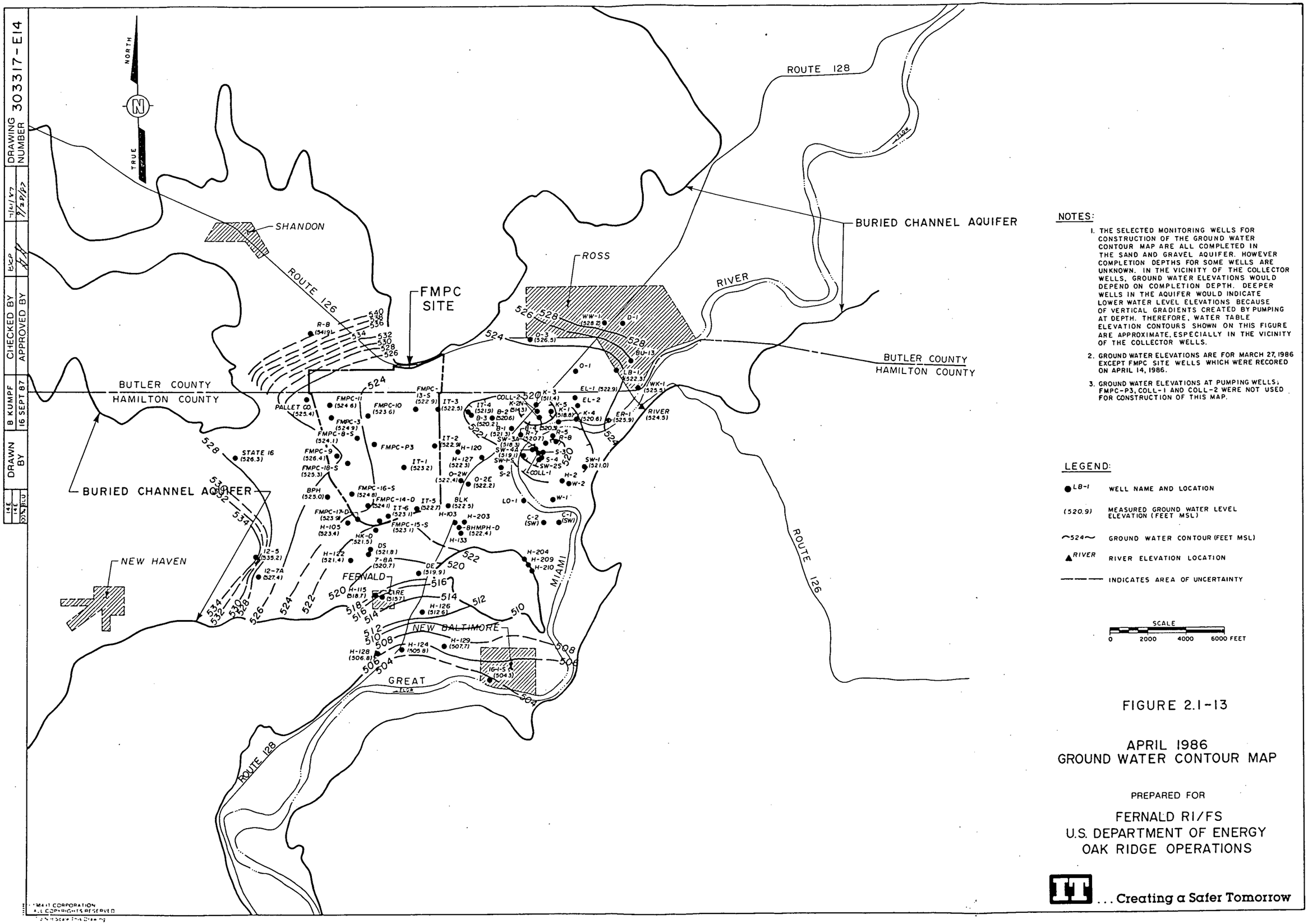

FIGURE 2.1-13

APRIL 1986
GROUND WATER CONTOUR MAP

PREPARED FOR

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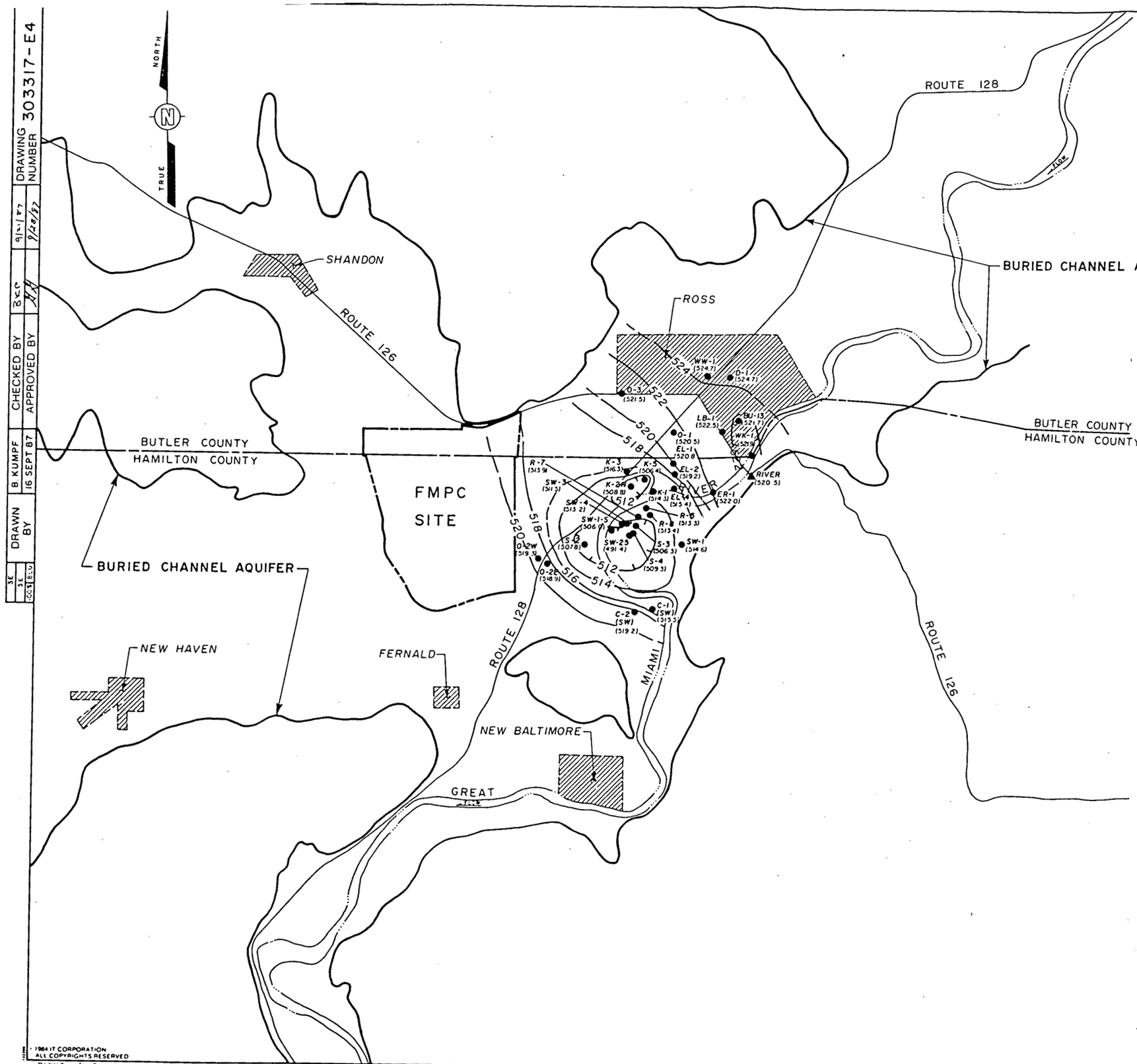

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.

2. GROUND WATER ELEVATIONS FOR WELLS WERE MEASURED ON AUGUST 28, 1986.

LEGEND:

- D-1 — WELL NAME AND LOCATION
- (524.7) — MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
- — 524 — GROUND WATER CONTOUR (FEET MSL)
- ▲ RIVER — RIVER ELEVATION LOCATION
- — — — INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-14

SEPTEMBER 1986
GROUND WATER CONTOUR MAP

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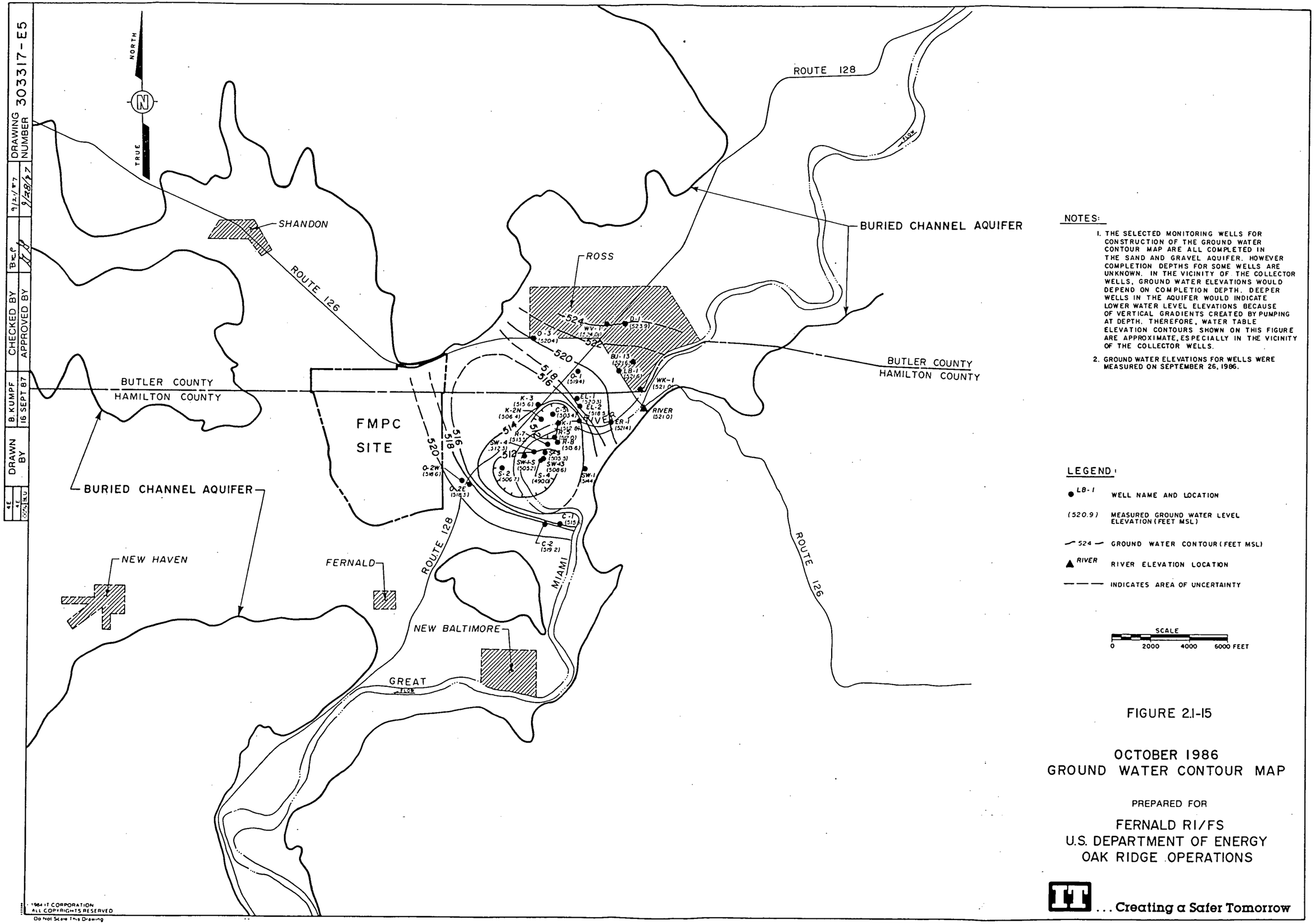
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9/21/87
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B. KUMPF
16 SEPT 87
DRAWN BY
NORTH
TRUE
SHANDON
ROUTE 126
ROUTE 128
BURIED CHANNEL AQUIFER
ROSS
BUTLER COUNTY
HAMILTON COUNTY
FMPC SITE
NEW HAVEN
FERNALD
NEW BALTIMORE
GREAT MIAMI RIVER
FLOW
RIVER
524
522
520
518
516
514
512
WV-1 (524.01)
D-1 (523.9)
O-3 (520.4)
BU-13 (521.6)
LB-1 (521.6)
WK-1 (521.0)
O-1 (519.4)
EL-1 (520.3)
EL-2 (518.5)
ER-1 (521.4)
K-3 (515.6)
K-2N (506.4)
G-3 (503.4)
K-1 (512.8)
TH-5 (512.0)
R-8 (510.6)
R-7 (513.2)
SW-4 (512.3)
SW-4S (503.2)
SW-4S (503.3)
SW-4S (508.6)
S-2 (506.7)
S-4 (490.0)
SW-1 (514.4)
O-2W (516.6)
O-2E (516.3)
C-1 (515.)
C-2 (519.2)
RIVER (521.0)
NOTES:
1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.
2. GROUND WATER ELEVATIONS FOR WELLS WERE MEASURED ON SEPTEMBER 26, 1986.
LEGEND:
LB-1 WELL NAME AND LOCATION
(520.9) MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
524 GROUND WATER CONTOUR (FEET MSL)
RIVER RIVER ELEVATION LOCATION
INDICATES AREA OF UNCERTAINTY
SCALE
0 2000 4000 6000 FEET
FIGURE 2.1-15
OCTOBER 1986
GROUND WATER CONTOUR MAP
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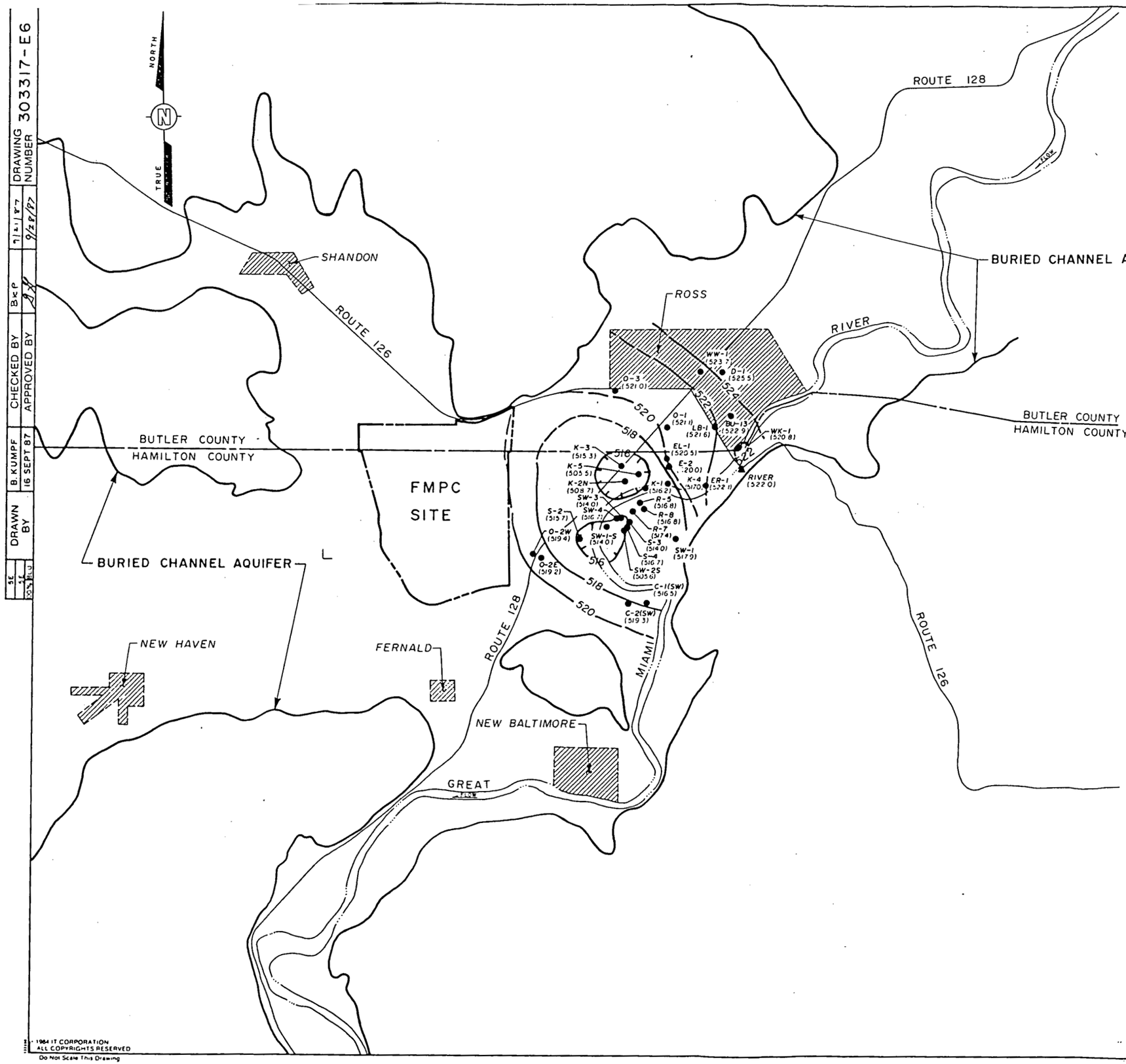

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.
2. GROUND WATER ELEVATIONS FOR WELLS WERE MEASURED ON OCTOBER 30, 1986.

LEGEND:

- ● LB-1 — WELL NAME AND LOCATION
- (520.9) — MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
- 524 — GROUND WATER CONTOUR (FEET MSL)
- ▲ RIVER — RIVER ELEVATION LOCATION
- — — — INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-16

NOVEMBER 1986
GROUND WATER CONTOUR MAP

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	9/28/87
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APPROVED BY	
B. KUMPF	16 SEPT 87
DRAWN BY	

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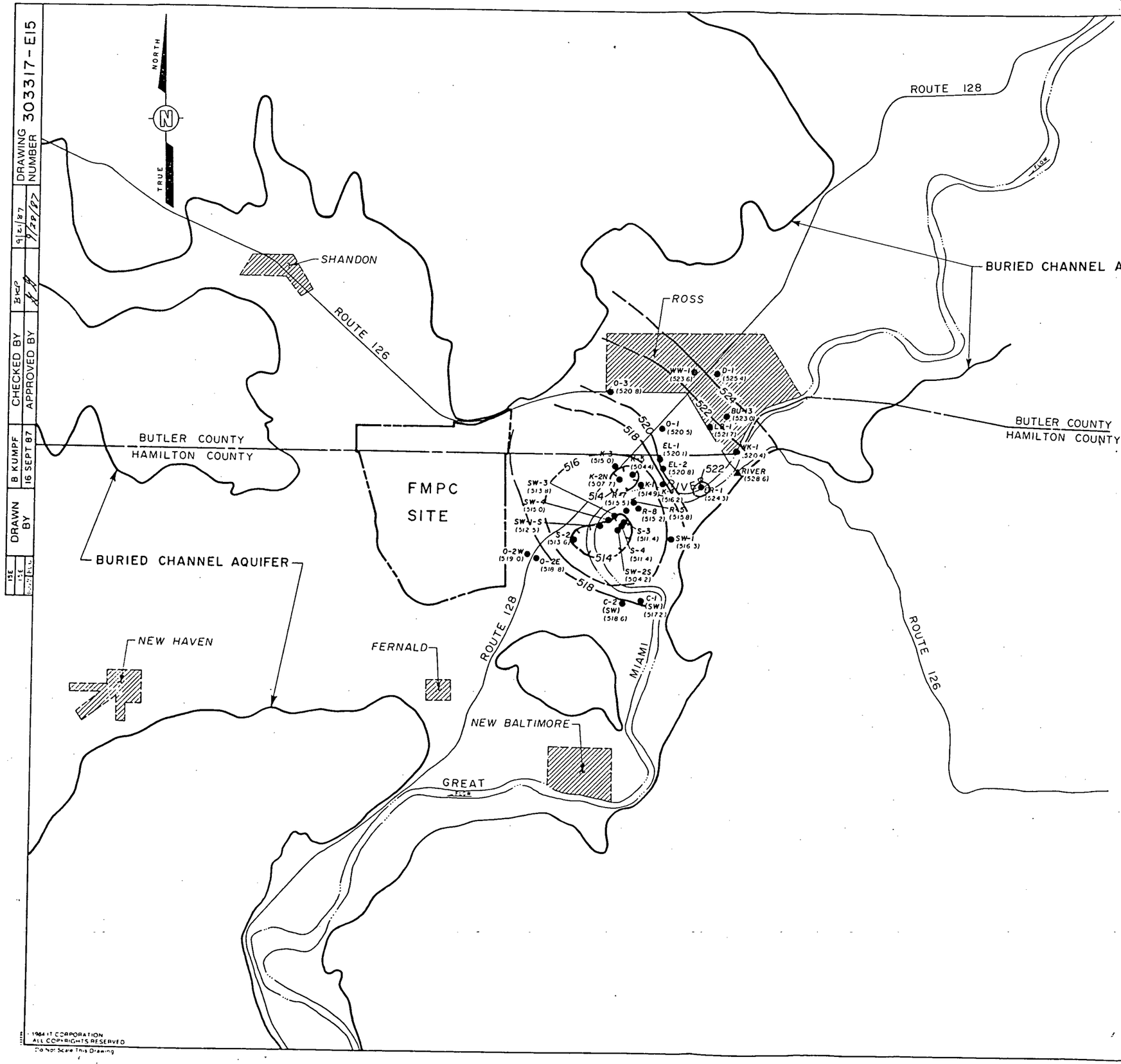

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.

2. GROUND WATER ELEVATIONS FOR WELLS WERE MEASURED ON NOVEMBER 26, 1986.

LEGEND:

- ● LB-1 — WELL NAME AND LOCATION
- (521.7) — MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
- — 524 — GROUND WATER CONTOUR (FEET MSL)
- ▲ RIVER — RIVER ELEVATION LOCATION
- — — — INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-17

DECEMBER 1986
GROUND WATER CONTOUR MAP

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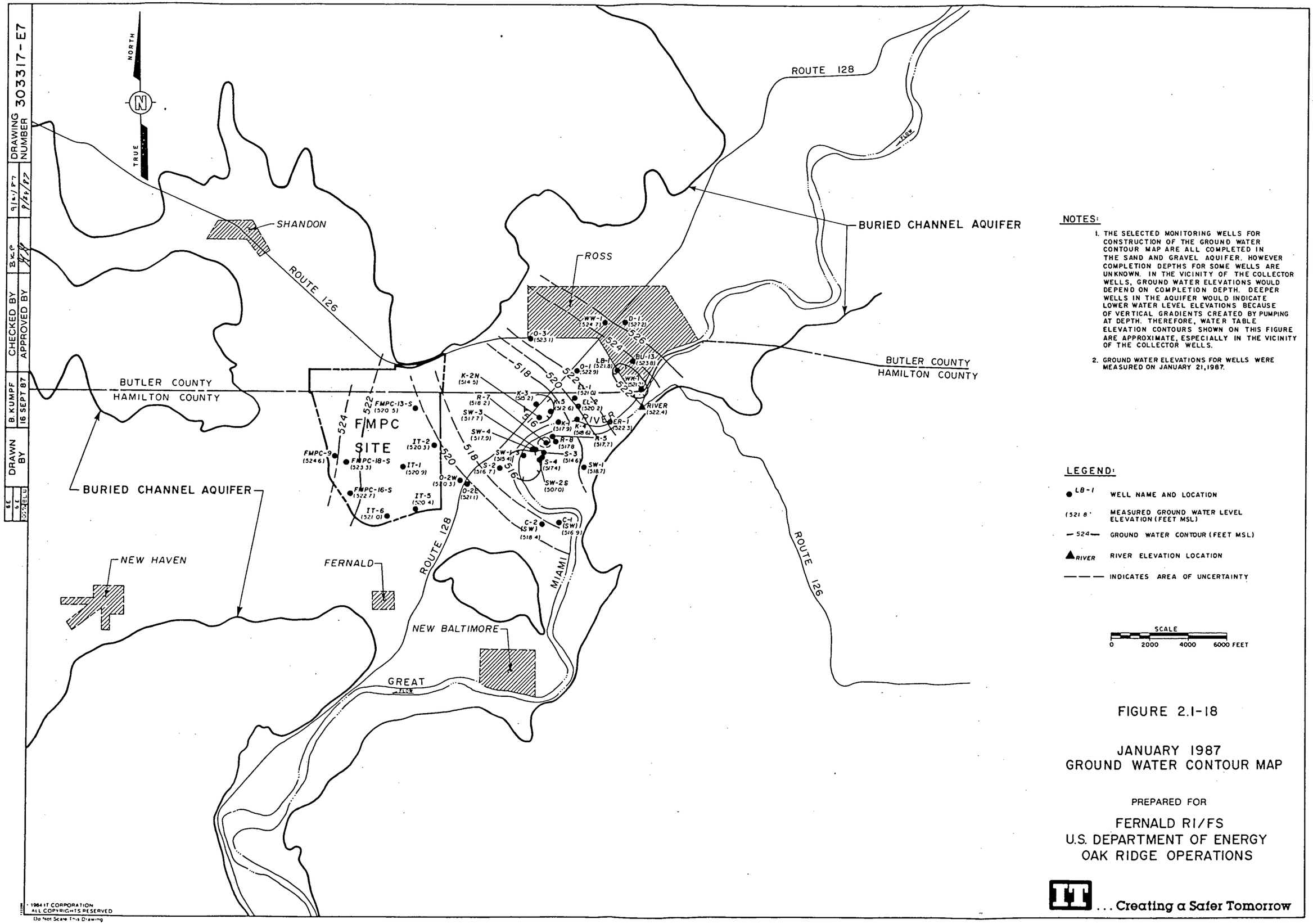

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.

2. GROUND WATER ELEVATIONS FOR WELLS WERE MEASURED ON JANUARY 21, 1987.

LEGEND:

- ● LB-1 — WELL NAME AND LOCATION
- (521.8) — MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
- — 524 — GROUND WATER CONTOUR (FEET MSL)
- ▲ RIVER — RIVER ELEVATION LOCATION
- — — — INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-18

JANUARY 1987
GROUND WATER CONTOUR MAP

PREPARED FOR

FERNALD RI/FS
U.S. DEPARTMENT OF ENERGY
OAK RIDGE OPERATIONS

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DRAWING NUMBER 303317-E7
9/24/87
B.K.P.
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B. KUMPF 16 SEPT 87
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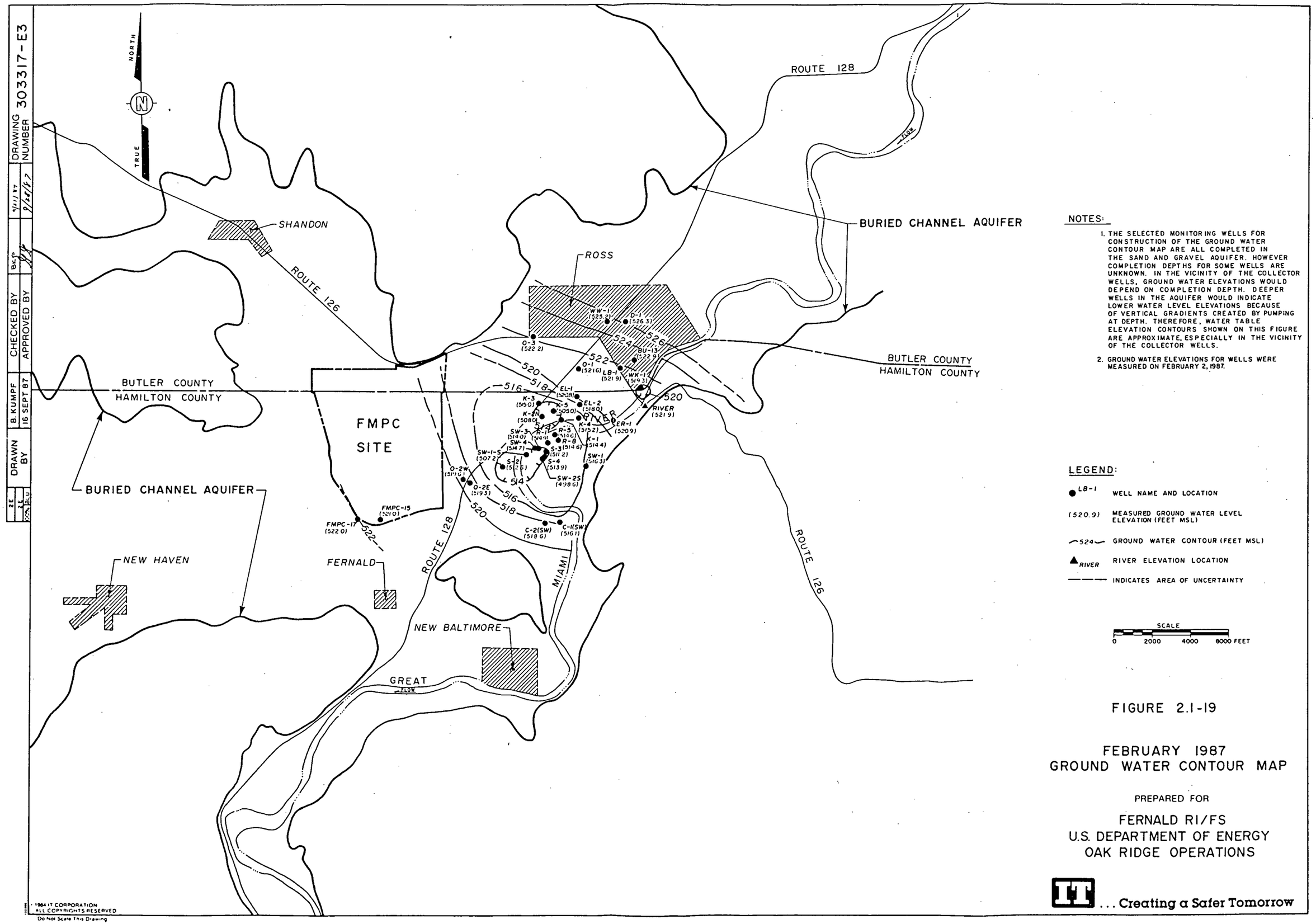
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NORTH
TRUE
SHANDON
ROUTE 126
ROUTE 128
ROSS
BURIED CHANNEL AQUIFER
BUTLER COUNTY
HAMILTON COUNTY
FMPC SITE
NEW HAVEN
FERNALD
NEW BALTIMORE
GREAT MIAMI RIVER
FLOW
WW-1 (525.2)
D-1 (526.3)
O-3 (522.2)
O-1 (521.6)
LB-1 (521.9)
BU-13 (522.9)
WK-1 (519.3)
EL-1 (520.8)
EL-2 (518.0)
K-3 (550)
K-5 (505.0)
K-2N (508.0)
K-4 (515.2)
ER-1 (520.9)
SW-3 (514.0)
R-1 (514.9)
R-5 (513.0)
R-6 (514.6)
K-1 (514.4)
SW-4 (514.7)
S-3 (511.2)
SW-1 (516.3)
SW-1-S (507.2)
S-2 (512.6)
S-4 (513.9)
SW-2S (499.6)
O-2W (511.6)
O-2E (519.3)
FMPC-15 (521.0)
FMPC-17 (522.0)
C-2(SW) (518.6)
C-1(SW) (516.1)
RIVER (521.9)
522
524
526
520
518
516
514
NOTES:
1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.
2. GROUND WATER ELEVATIONS FOR WELLS WERE MEASURED ON FEBRUARY 2, 1987.
LEGEND:
LB-1 WELL NAME AND LOCATION
(520.9) MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
524 GROUND WATER CONTOUR (FEET MSL)
RIVER RIVER ELEVATION LOCATION
INDICATES AREA OF UNCERTAINTY
SCALE
0 2000 4000 8000 FEET
FIGURE 2.1-19
FEBRUARY 1987
GROUND WATER CONTOUR MAP
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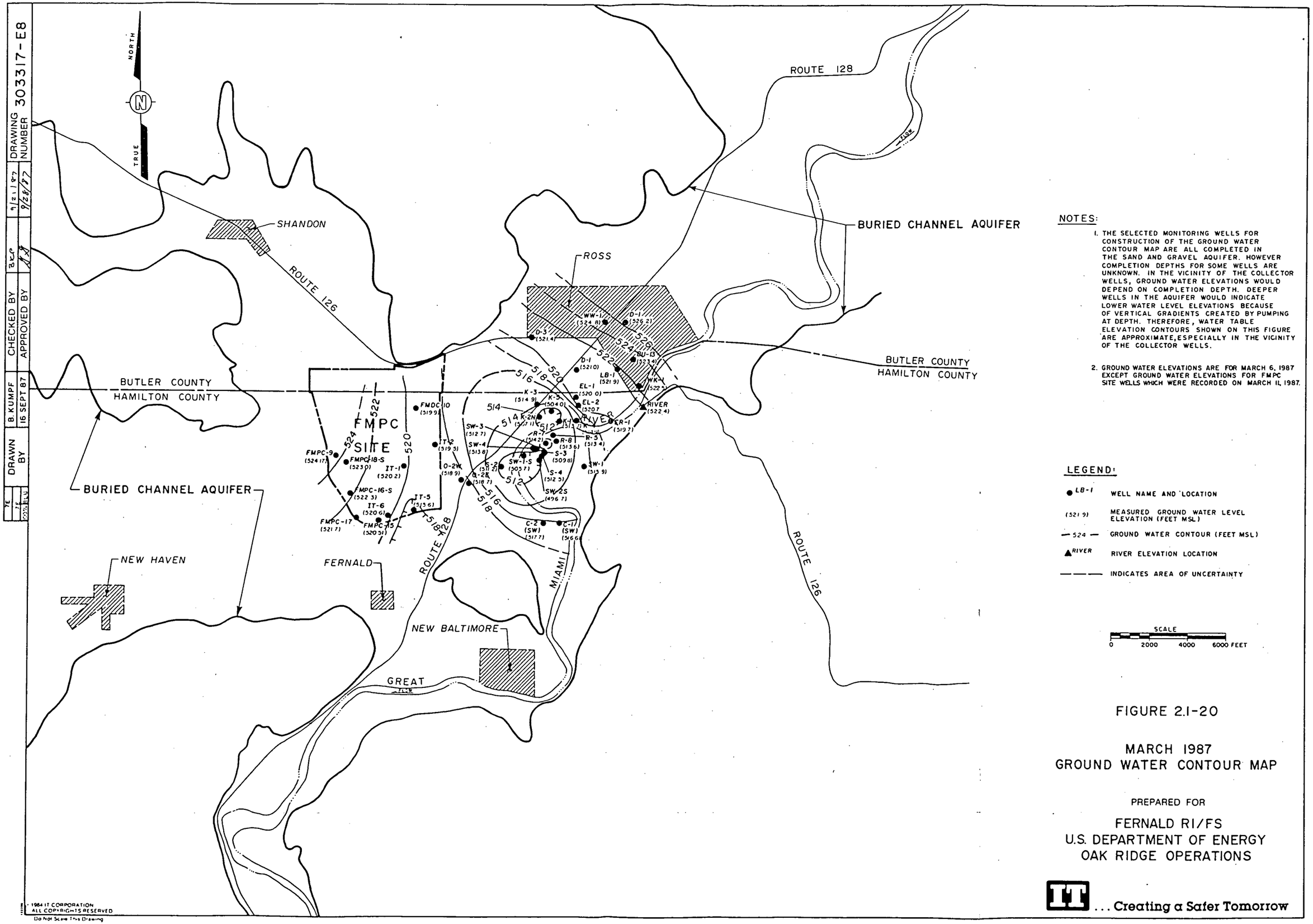

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.

2. GROUND WATER ELEVATIONS ARE FOR MARCH 6, 1987 EXCEPT GROUND WATER ELEVATIONS FOR FMPC SITE WELLS WHICH WERE RECORDED ON MARCH 11, 1987.

LEGEND:

● LB-1	WELL NAME AND LOCATION
(521.9)	MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
— 524 —	GROUND WATER CONTOUR (FEET MSL)
▲ RIVER	RIVER ELEVATION LOCATION
— — —	INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-20

MARCH 1987
GROUND WATER CONTOUR MAP

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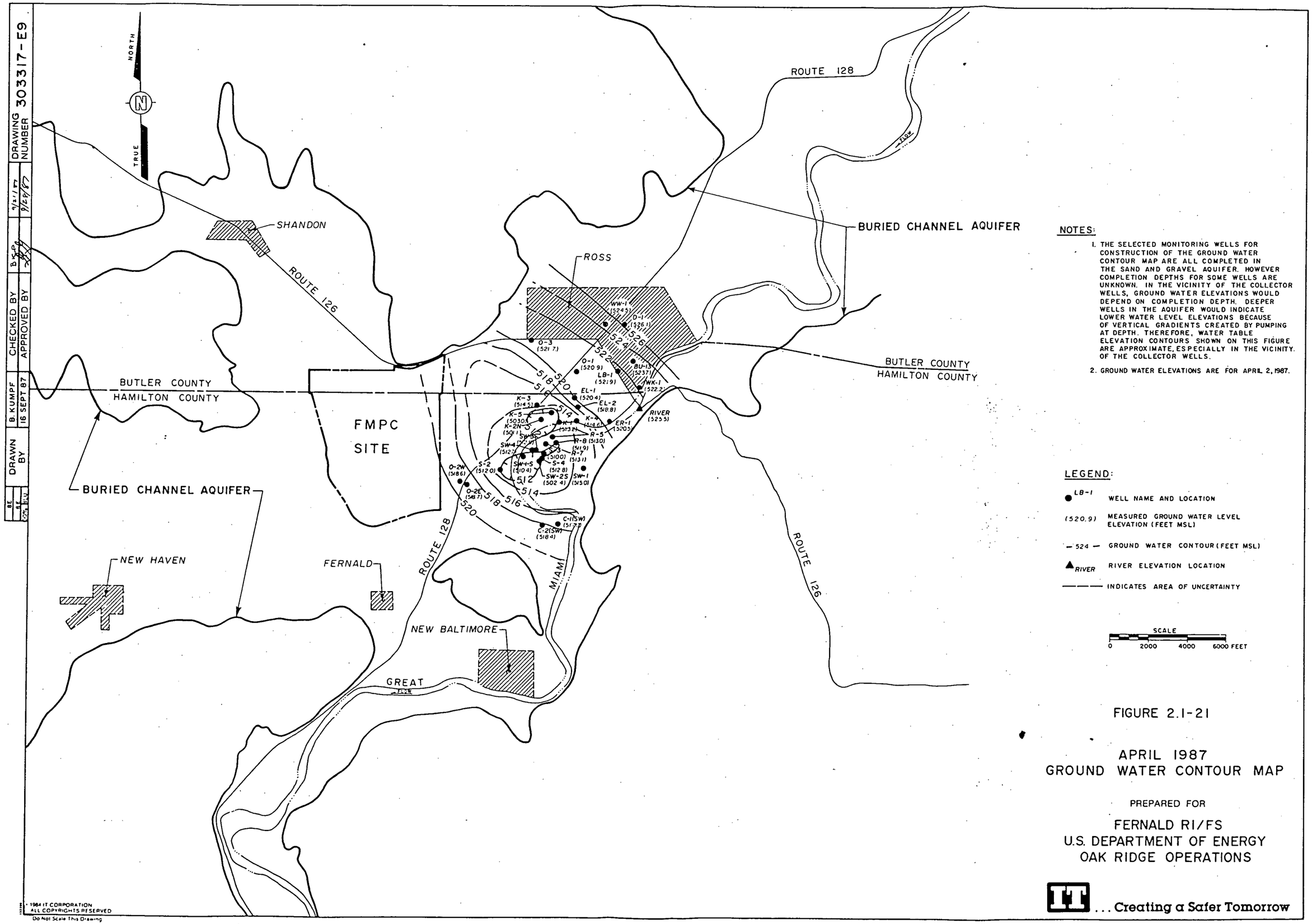

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.
2. GROUND WATER ELEVATIONS ARE FOR APRIL 2, 1987.

LEGEND:

- ● LB-1 — WELL NAME AND LOCATION
- (520.9) — MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
- — 524 — GROUND WATER CONTOUR (FEET MSL)
- ▲ RIVER — RIVER ELEVATION LOCATION
- ——— INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-21

APRIL 1987
GROUND WATER CONTOUR MAP

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DRAWN BY	B. KUMPF
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CPS	BKU

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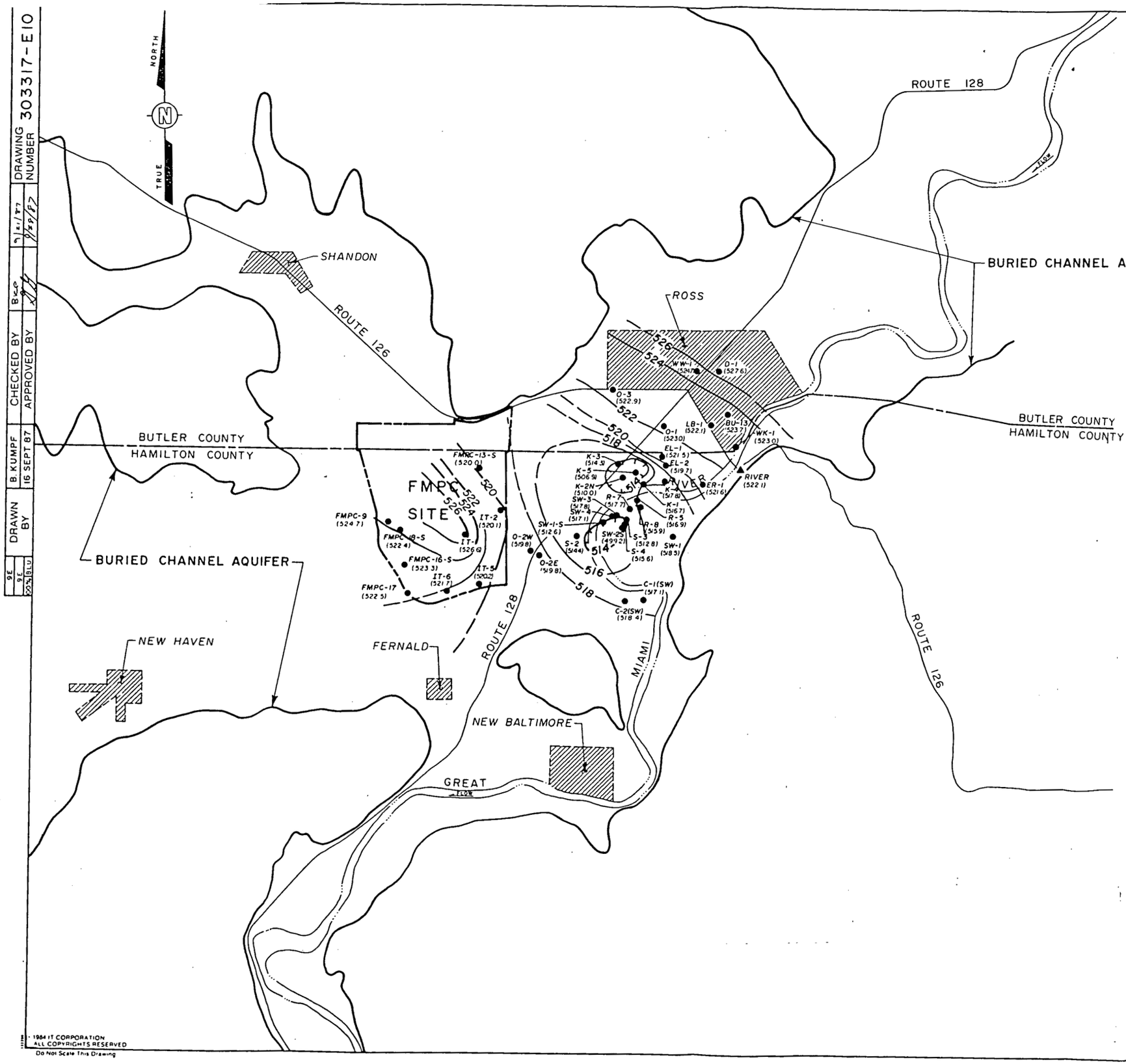

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DRAWN BY	B. KUMPF
	16 SEPT 87

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.
2. GROUND WATER ELEVATIONS ARE FOR APRIL 29, 1987 EXCEPT FMPC SITE WELLS WHICH WERE RECORDED ON APRIL 30, 1987 AND BU-13 WHICH WAS RECORDED ON MAY 3, 1987.

LEGEND:

- ● LB-1 — WELL NAME AND LOCATION
- (520.9) — MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
- — 524 — GROUND WATER CONTOUR (FEET MSL)
- ▲ RIVER — RIVER ELEVATION LOCATION
- ——— INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-22

MAY 1987
GROUND WATER CONTOUR MAP

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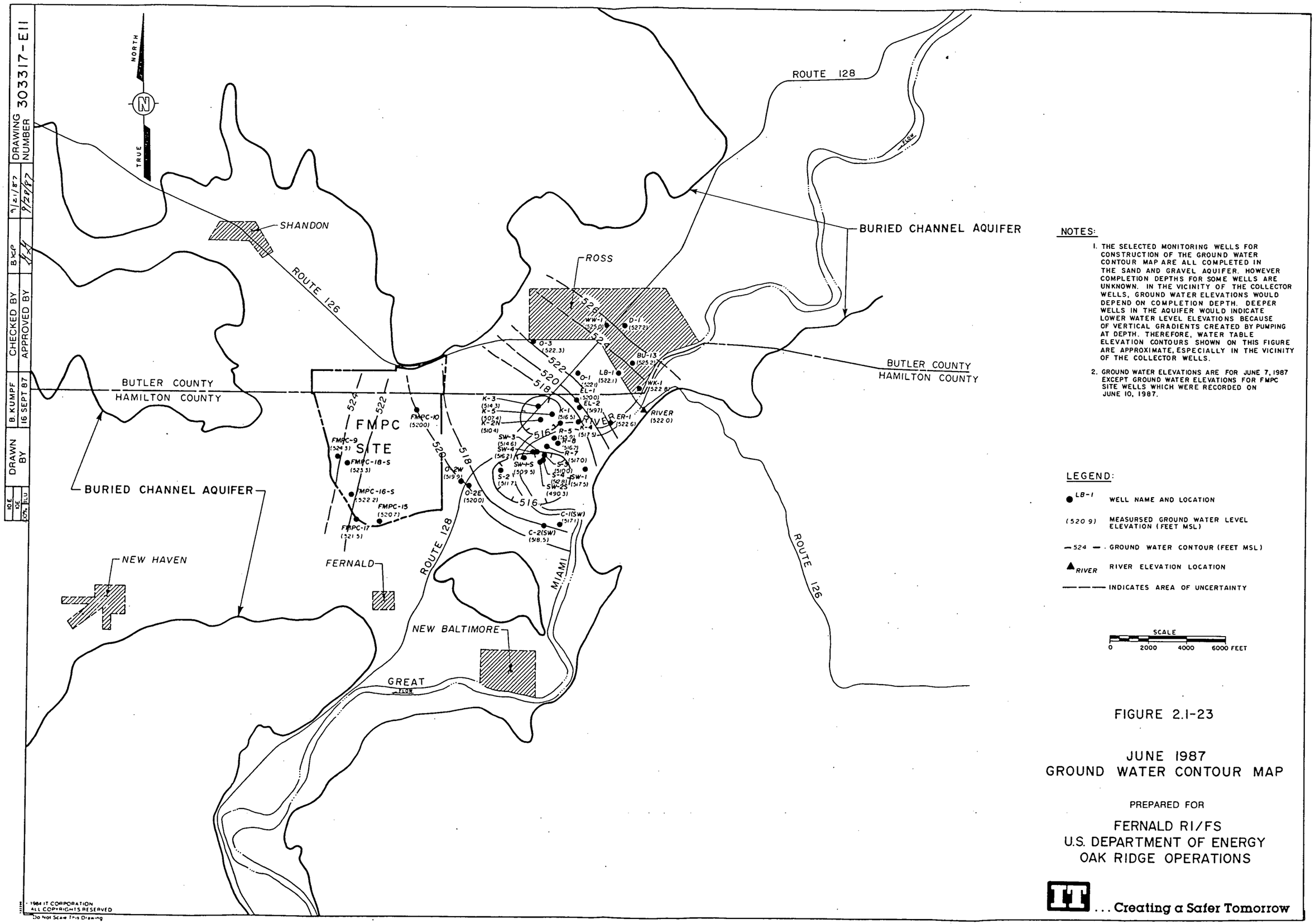

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.

2. GROUND WATER ELEVATIONS ARE FOR JUNE 7, 1987 EXCEPT GROUND WATER ELEVATIONS FOR FMPC SITE WELLS WHICH WERE RECORDED ON JUNE 10, 1987.

LEGEND:

- LB-1 — WELL NAME AND LOCATION
- (520.9) — MEASURSED GROUND WATER LEVEL ELEVATION (FEET MSL)
- 524 — GROUND WATER CONTOUR (FEET MSL)
- RIVER — RIVER ELEVATION LOCATION
- INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-23

JUNE 1987
GROUND WATER CONTOUR MAP

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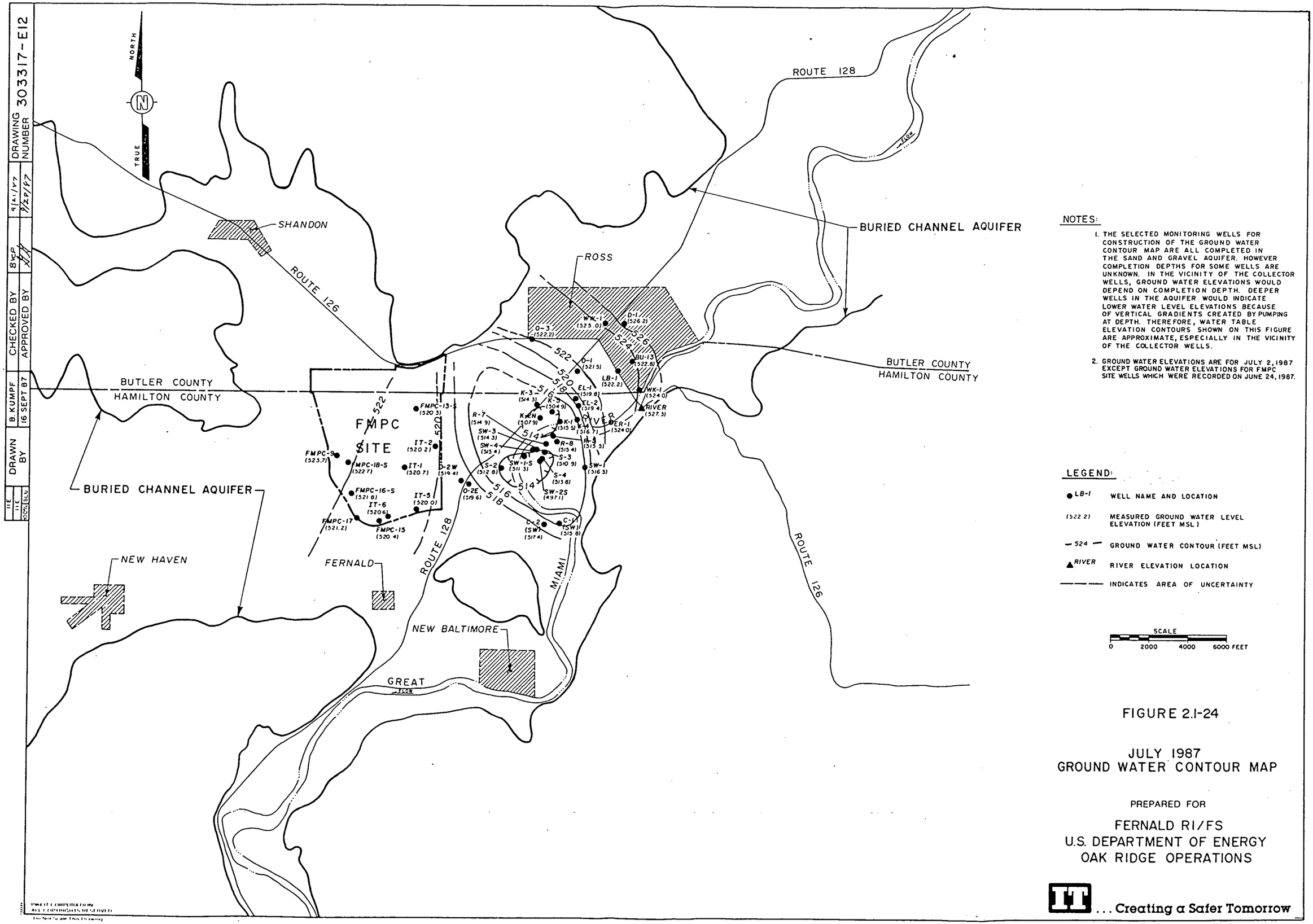

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.

2. GROUND WATER ELEVATIONS ARE FOR JULY 2, 1987 EXCEPT GROUND WATER ELEVATIONS FOR FMPC SITE WELLS WHICH WERE RECORDED ON JUNE 24, 1987.

LEGEND:

● LB-1	WELL NAME AND LOCATION
(522.2)	MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
— 524 —	GROUND WATER CONTOUR (FEET MSL)
▲ RIVER	RIVER ELEVATION LOCATION
— — —	INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-24

JULY 1987
GROUND WATER CONTOUR MAP

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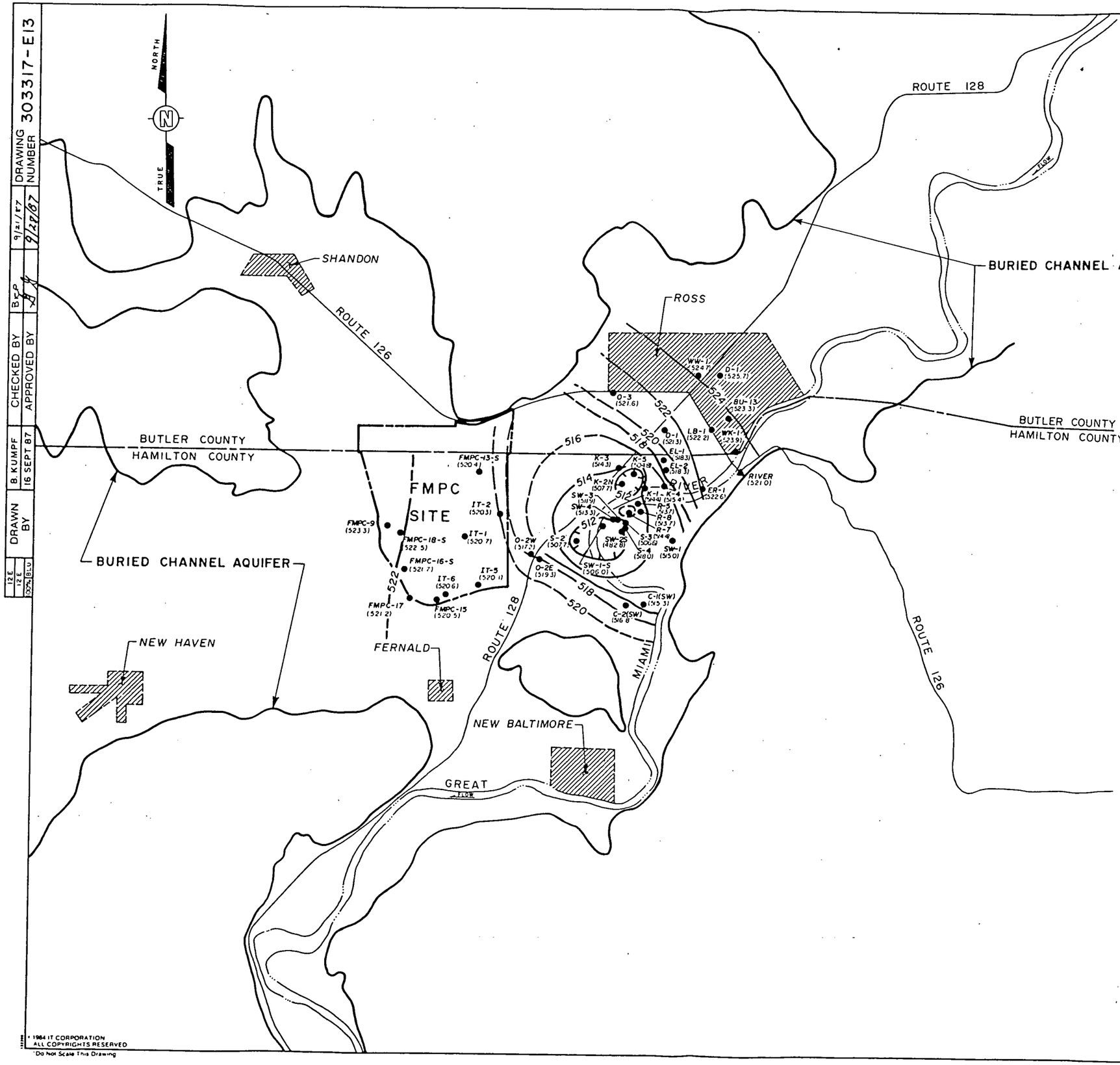

NOTES:

1. THE SELECTED MONITORING WELLS FOR CONSTRUCTION OF THE GROUND WATER CONTOUR MAP ARE ALL COMPLETED IN THE SAND AND GRAVEL AQUIFER. HOWEVER COMPLETION DEPTHS FOR SOME WELLS ARE UNKNOWN. IN THE VICINITY OF THE COLLECTOR WELLS, GROUND WATER ELEVATIONS WOULD DEPEND ON COMPLETION DEPTH. DEEPER WELLS IN THE AQUIFER WOULD INDICATE LOWER WATER LEVEL ELEVATIONS BECAUSE OF VERTICAL GRADIENTS CREATED BY PUMPING AT DEPTH. THEREFORE, WATER TABLE ELEVATION CONTOURS SHOWN ON THIS FIGURE ARE APPROXIMATE, ESPECIALLY IN THE VICINITY OF THE COLLECTOR WELLS.
2. GROUND WATER ELEVATIONS ARE FOR 8 AUGUST 1987 EXCEPT GROUND WATER ELEVATIONS FOR FMPC SITE WELLS WHICH WERE RECORDED ON 24 JULY 87.

LEGEND:

● LB-1	WELL NAME AND LOCATION
(520.9)	MEASURED GROUND WATER LEVEL ELEVATION (FEET MSL)
—524—	GROUND WATER ELEVATION (FEET MSL)
▲ RIVER	RIVER ELEVATION LOCATION
— — —	INDICATES AREA OF UNCERTAINTY

SCALE
0 2000 4000 6000 FEET

FIGURE 2.1-25

AUGUST 1987
GROUND WATER CONTOUR MAP

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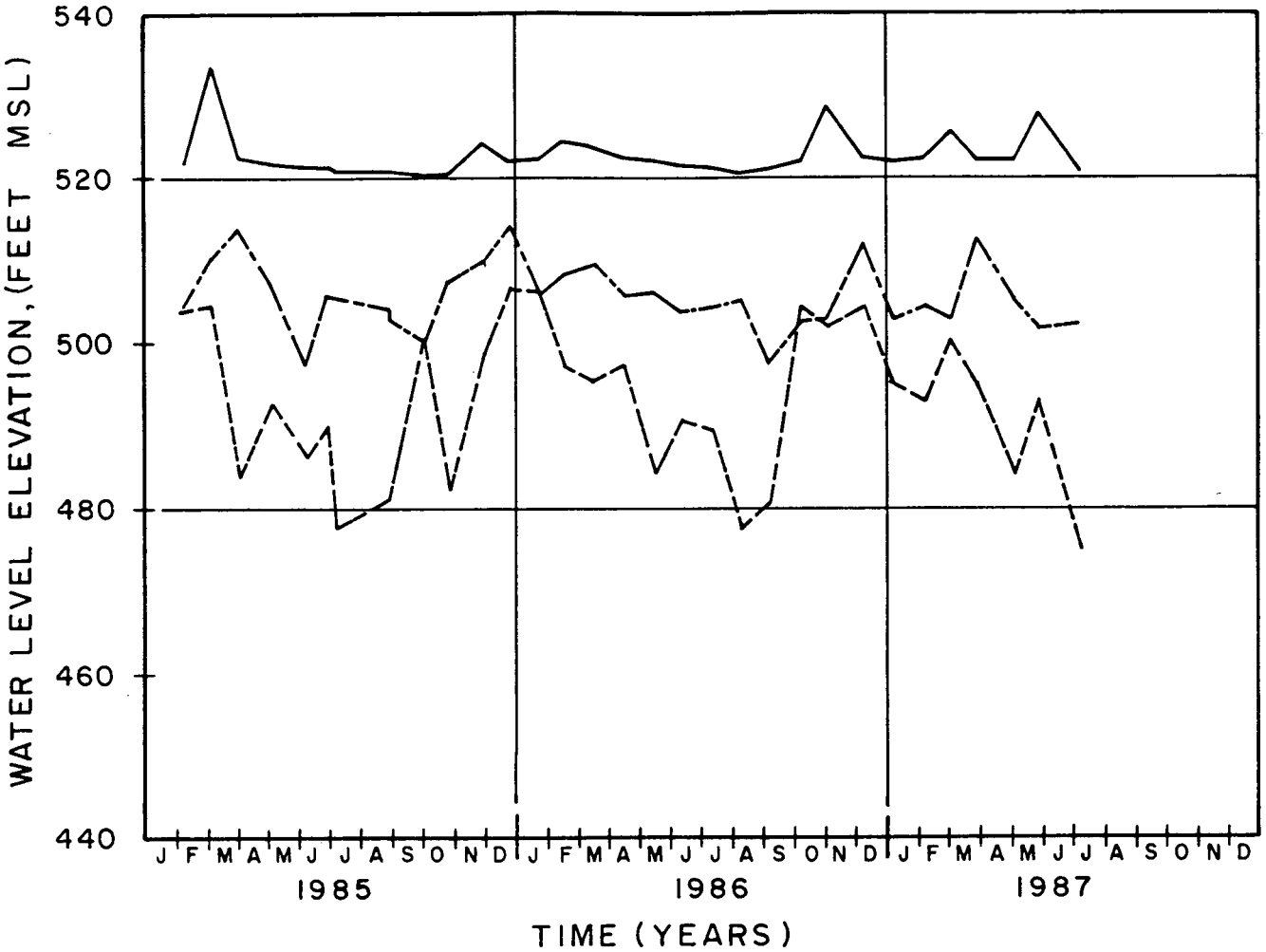

LEGEND:

——— GREAT MIAMI RIVER AT ROSS BRIDGE

—·—·— COLLECTOR WELL 1

– – – – COLLECTOR WELL 2

NOTE:

SEE FIGURE 2.1-1 FOR SITE LOCATIONS.

FIGURE 2.1-26

COLLECTOR WELLS AND RIVER HYDROGRAPHS:

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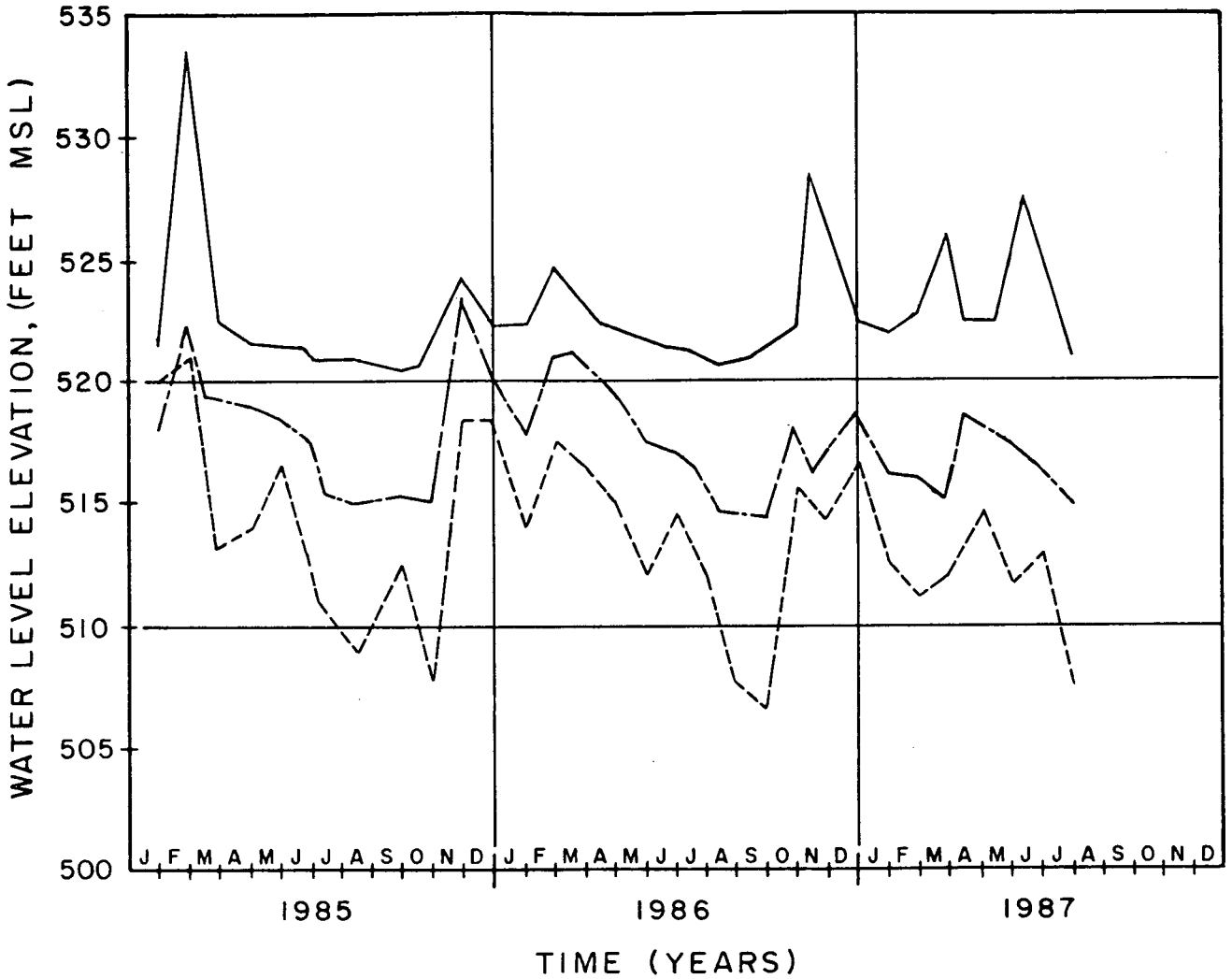

LEGEND:

——— GREAT MIAMI RIVER AT ROSS BRIDGE

– – – – S-2

– – – – SW-1

NOTE:

SEE FIGURE 2.1-1 FOR SITE LOCATIONS.

FIGURE 2.1-27

MONITORING WELL AND RIVER HYDROGRAPHS: WELLS SW-1 AND S-2

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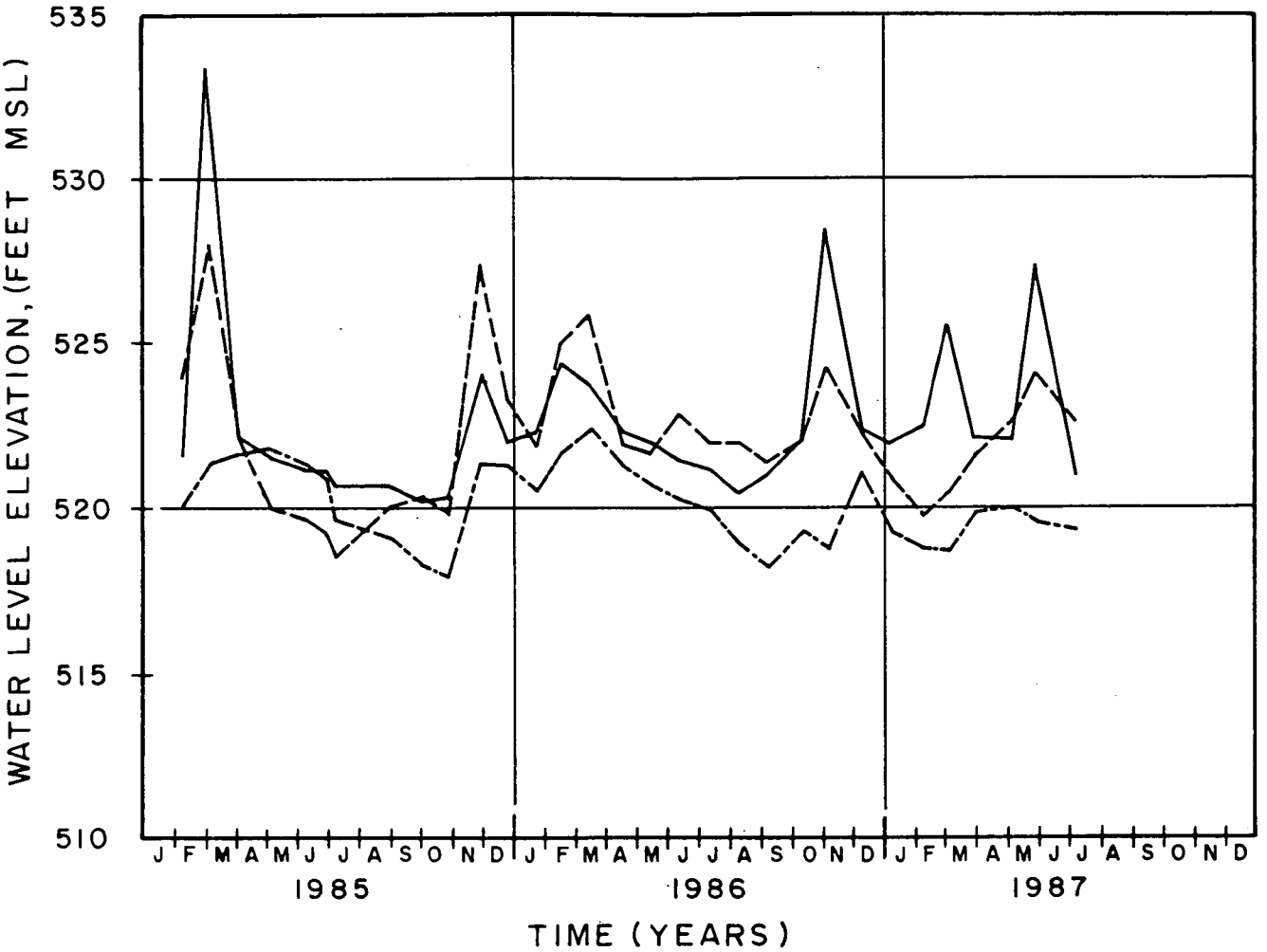

LEGEND:

——— GREAT MIAMI RIVER AT ROSS BRIDGE

– – – – ER-1

—·—·— O-2E

NOTE:

SEE FIGURE 2.1-1 FOR SITE LOCATIONS.

FIGURE 2.1-28

MONITORING WELL AND RIVER HYDROGRAPHS: WELLS ER-1 AND O-2E

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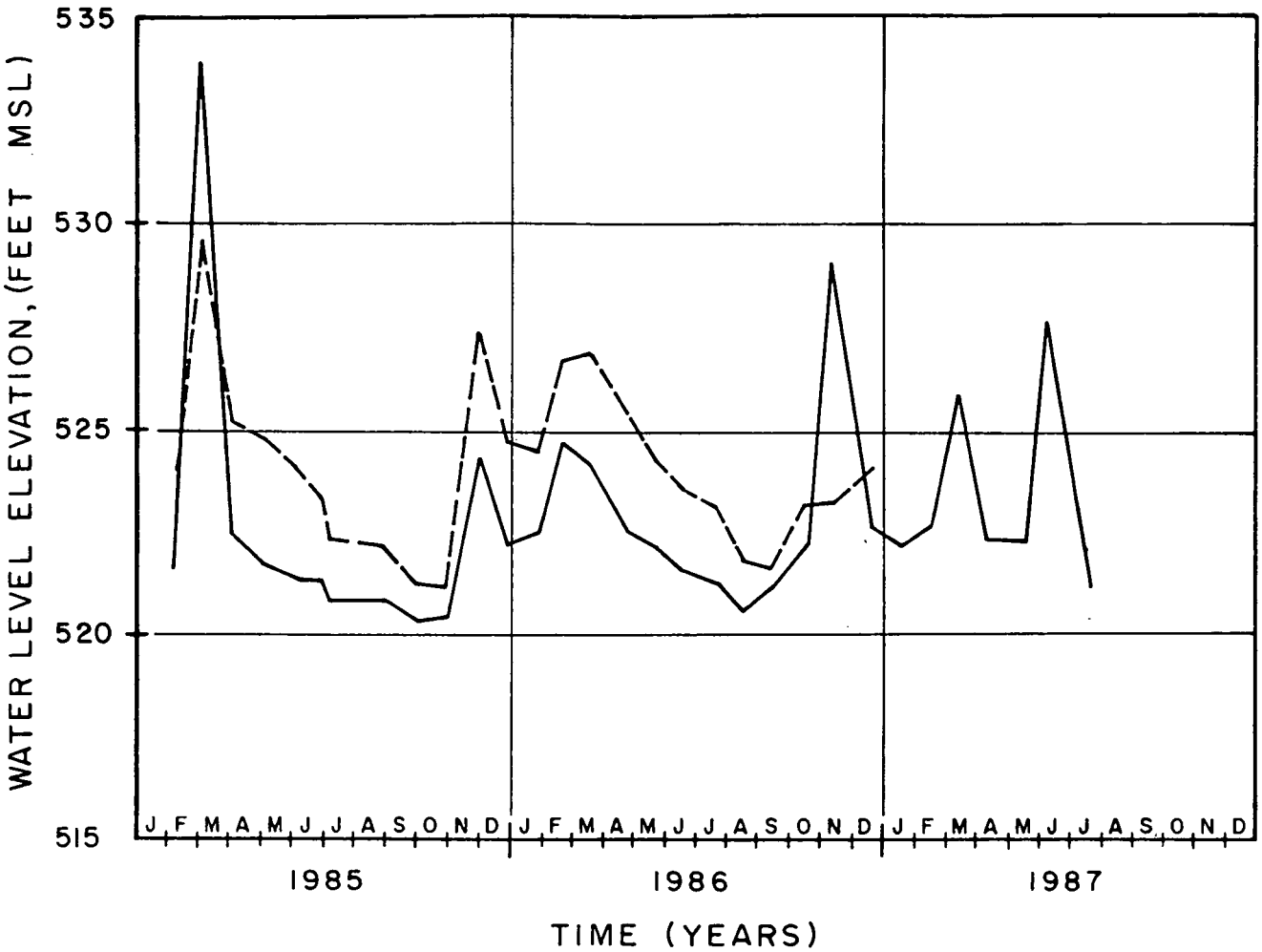

LEGEND:

——— GREAT MIAMI RIVER AT ROSS BRIDGE

– – – – BU-13

NOTE:

SEE FIGURE 2.1-1 FOR SITE LOCATIONS.

FIGURE 2.1-29

MONITORING WELL AND RIVER HYDROGRAPHS: WELL BU-13

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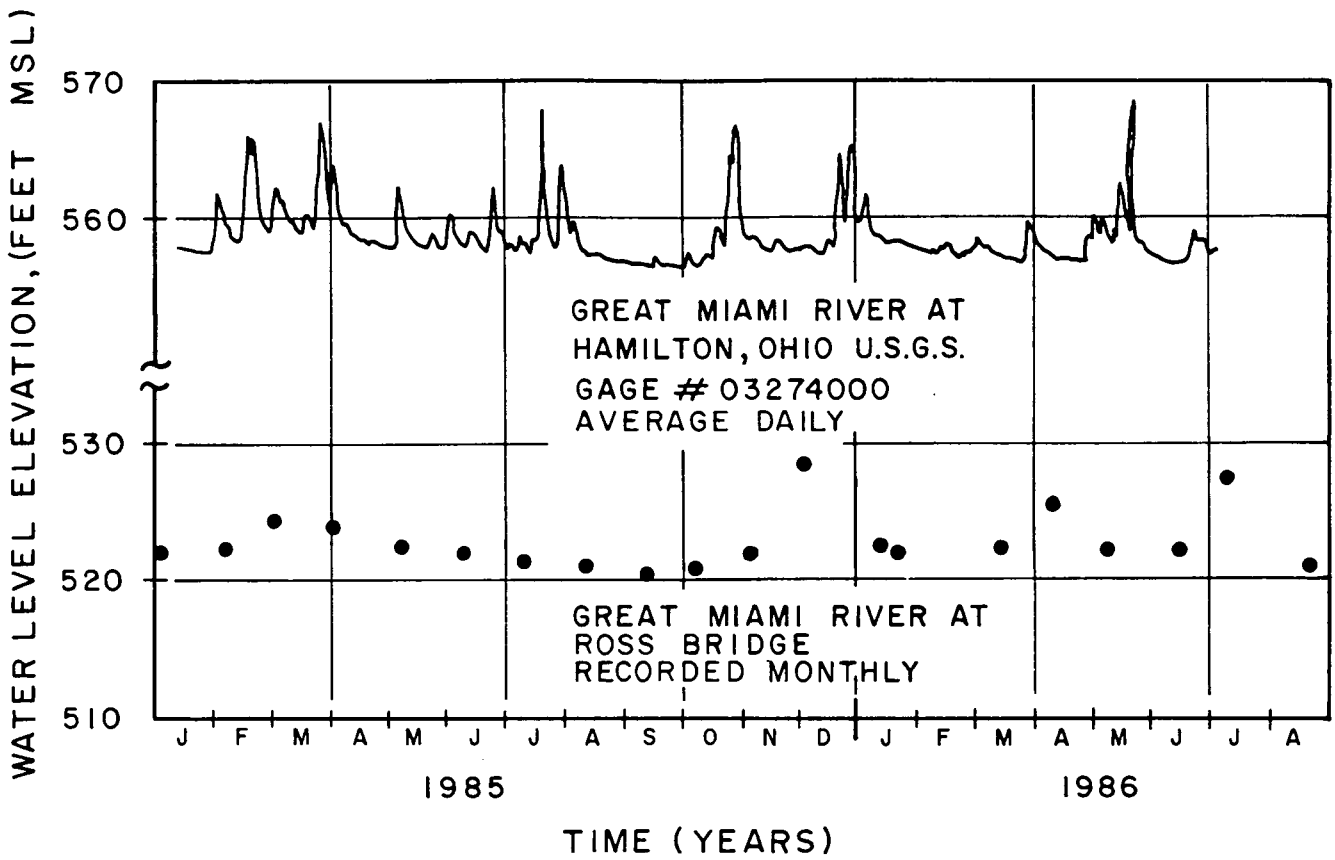

FIGURE 2.1-30

RIVER STAGE AT
HAMILTON AND ROSS
OHIO, 1985 AND 1986

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NOTE:

SEE FIGURE 2.1-1 FOR ROSS BRIDGE LOCATION.

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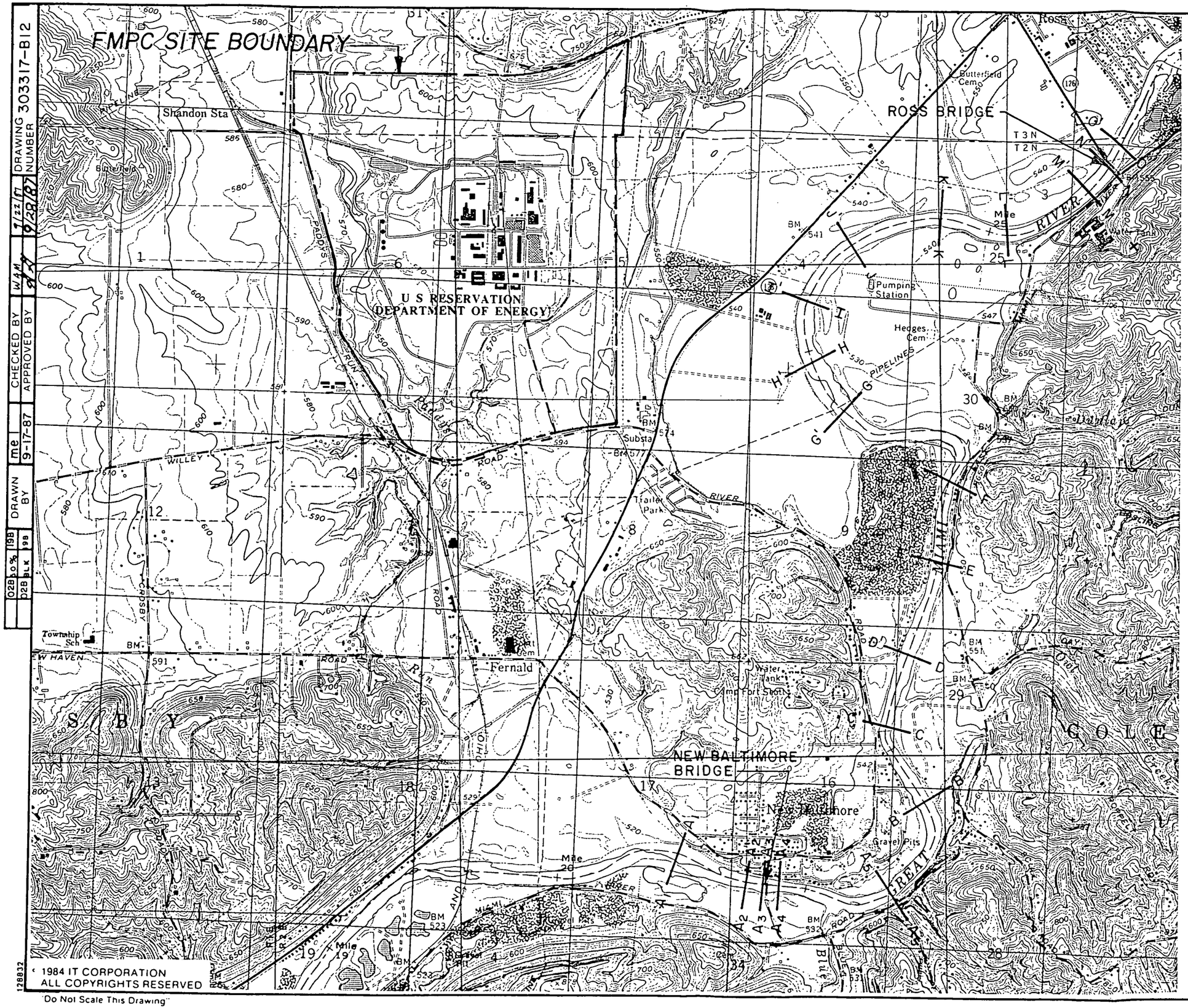

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02B	10%
02B	BLK

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LEGEND

B'——B RIVER CROSS SECTION LOCATIONS HEC-2 MODELING

REFERENCES:

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO-IND., SHANDON, OHIO GREENHILLS, OHIO, CINCINNATI WEST, OHIO, ADDYSTON, OHIO-KY, AND HOOVEN OHIO. DATED: 1955 (PR1981), 1965 (PR1981), 1965 (PR1981), 1961, PR1981), 1982 AND 1981 SCALE: 1" = 2000'

HEC-2 WATER SURFACE PROFILES US ARMY CORPS OF ENGINEERS COMPUTER PROGRAM VERSION OF NOVEMBER, 1976 UPDATED MARCH, 1982.

SCALE

0 2000 4000 FEET

FIGURE 2.2-1

LOCATION OF RIVER CROSS SECTIONS USED TO OBTAIN HYDRAULIC CHARACTERISTICS

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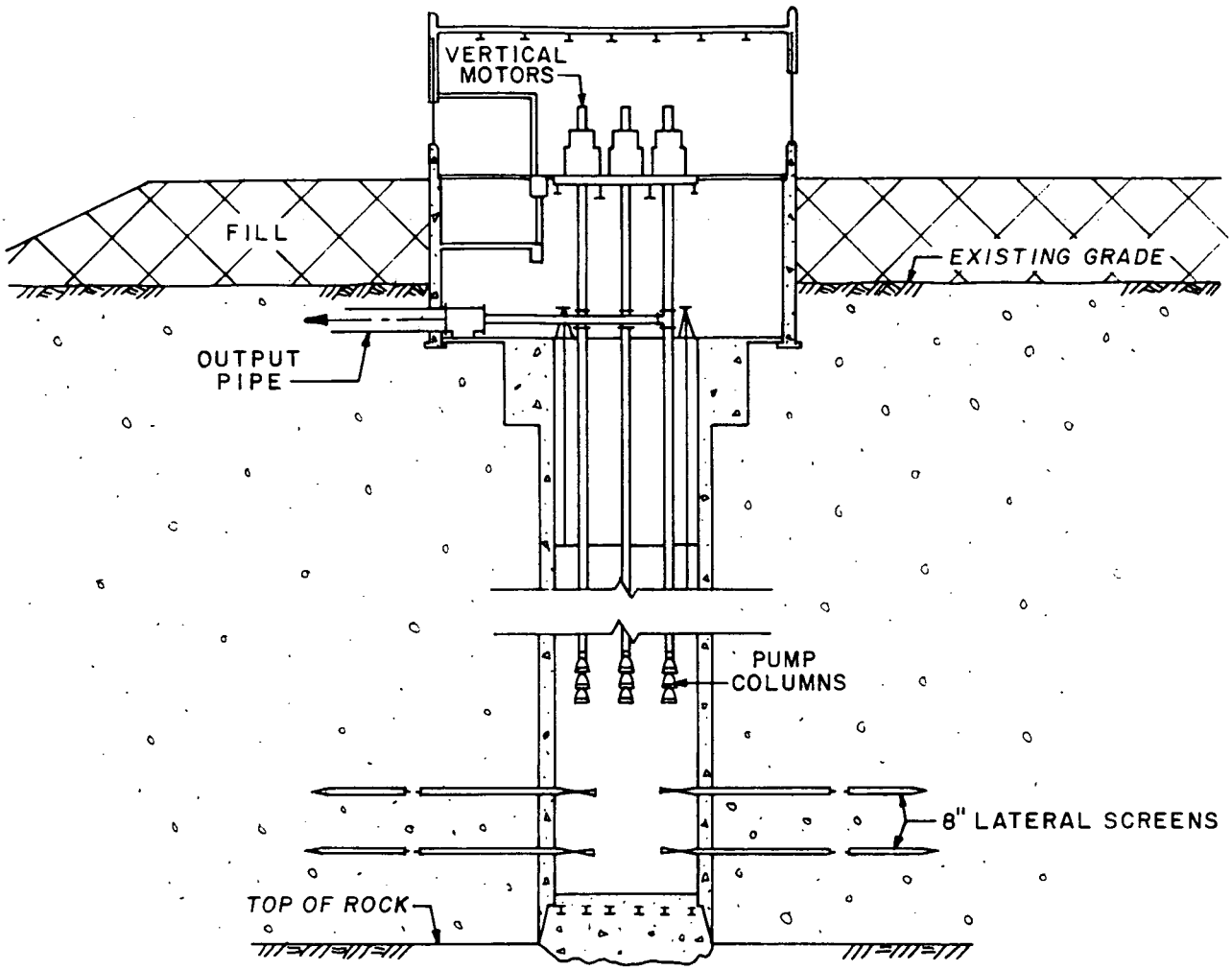

FIGURE 2.5-1

GENERALIZED SECTION OF SOUTHWESTERN OHIO WATER CO. COLLECTOR WELL

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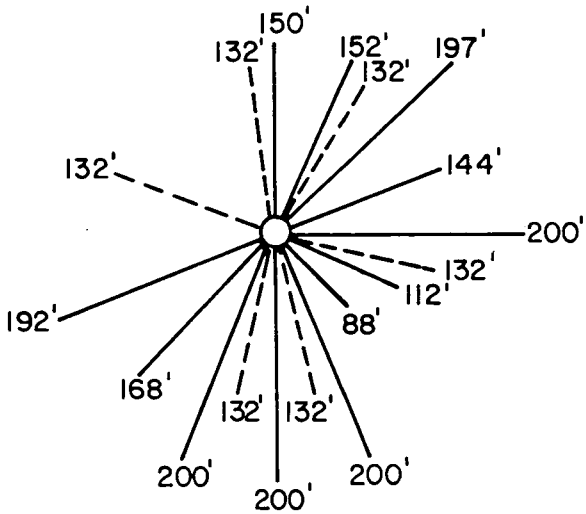
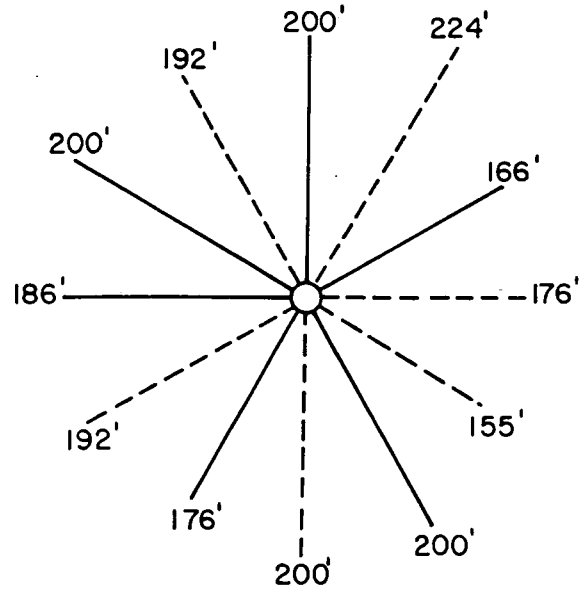


COLLECTOR WELL 1

ELEVATION OF LOWER TIER 409.7 FT.
ELEVATION OF UPPER TIER 433.0 FT.



COLLECTOR WELL 2

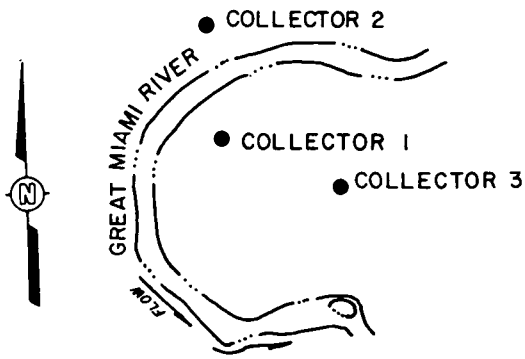
ELEVATION OF LOWER TIER 413.0 FT.
ELEVATION OF UPPER TIER 417.0 FT.

LEGEND

--- LOWER TIER LATERAL SCREENS
—— UPPER TIER LATERAL SCREENS

NOTE:

INFORMATION NOT AVAILABLE ON LATERAL ORIENTATION FOR COLLECTOR 3.



PLAN

FIGURE 2.5-2

DIAGRAM SHOWING POSITION AND LENGTH OF HORIZONTAL SCREENS IN SOUTHWESTERN OHIO WATER COMPANY COLLECTOR 1 AND COLLECTOR 2

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65360

83

130

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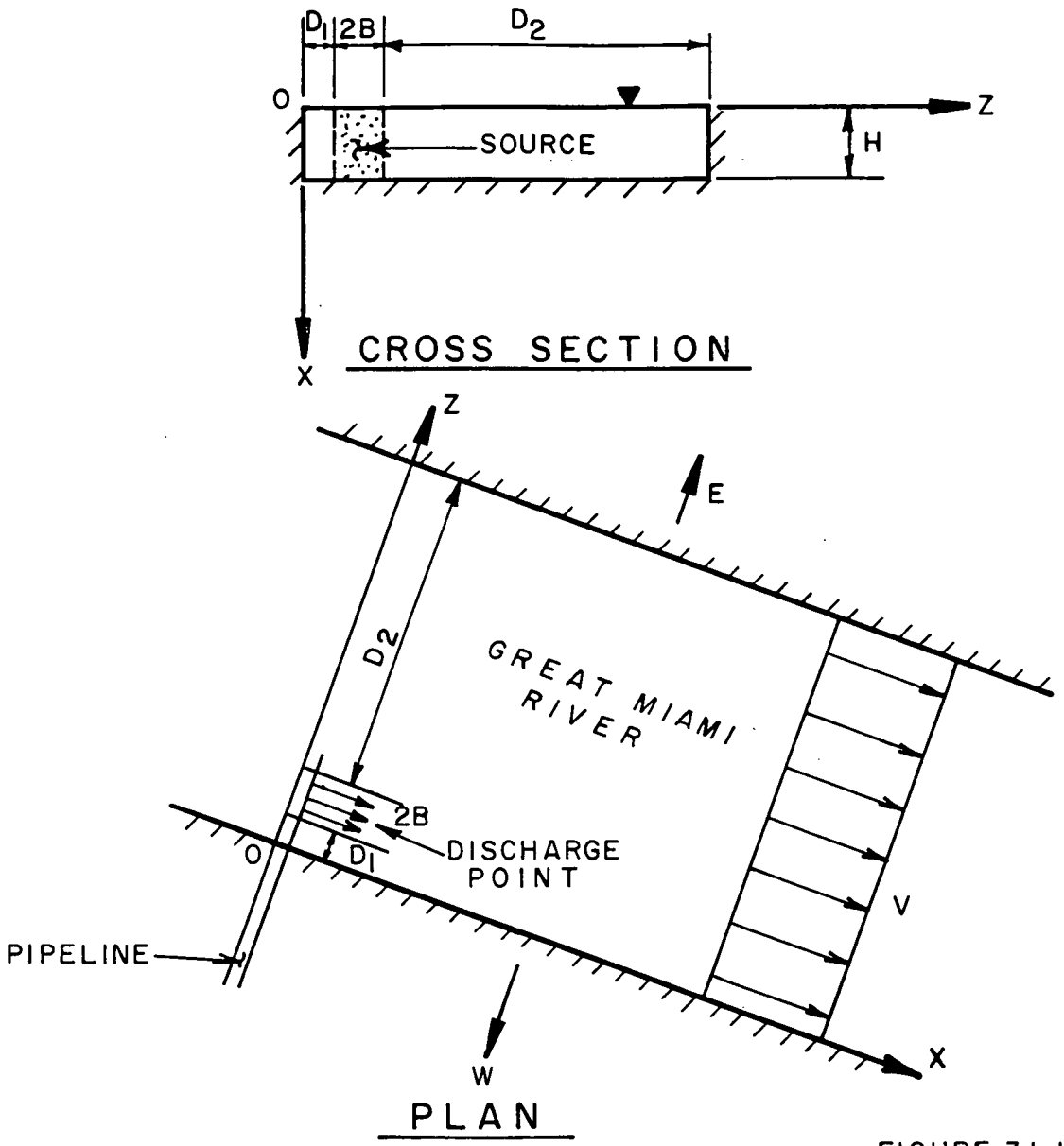

FIGURE 3.1-1

IDEALIZED SCHEMATIC REPRESENTATION OF THE HYDRODYNAMIC DISPERSION MODEL

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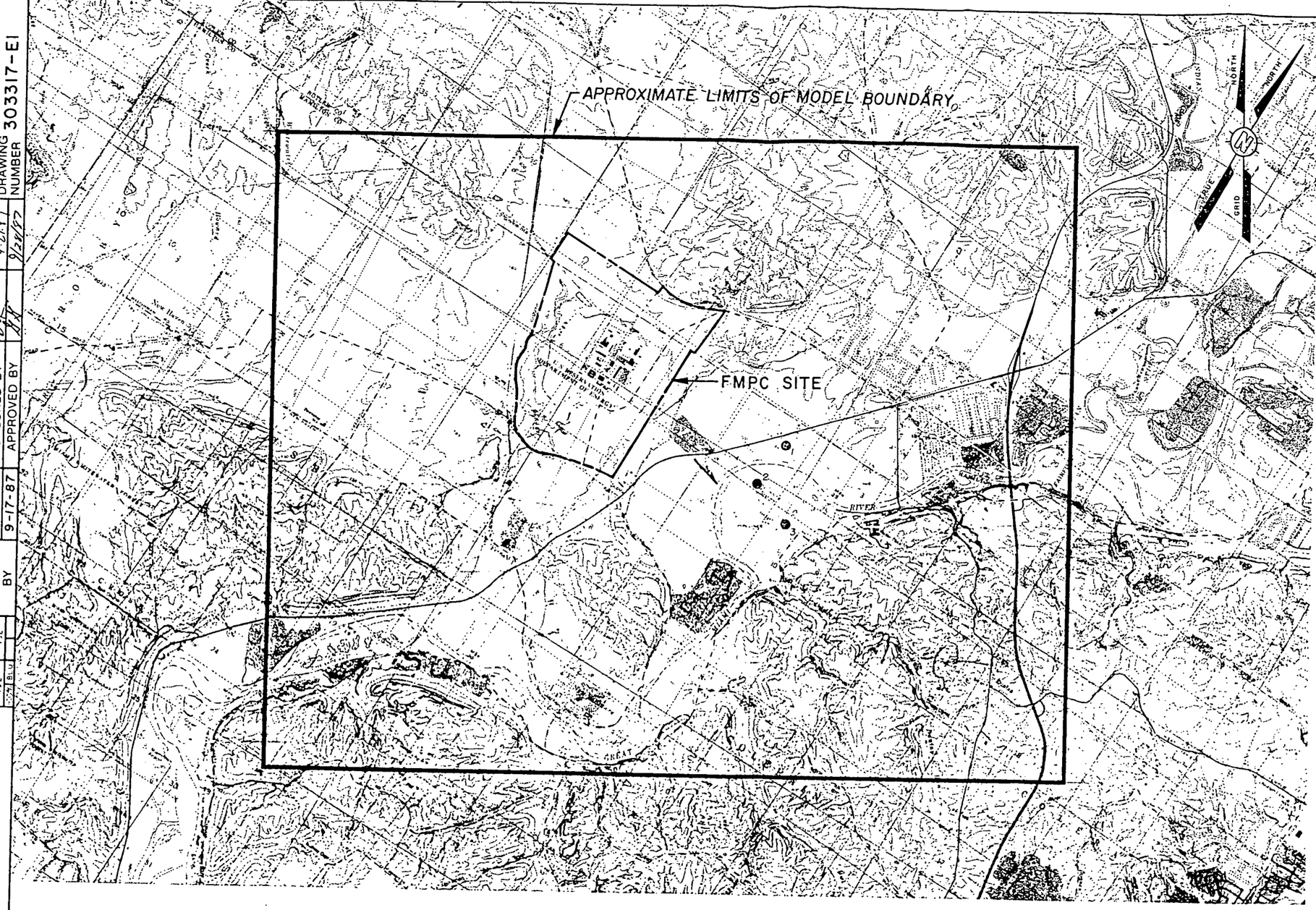

LEGEND:

FMPC EFFLUENT LINE DISCHARGE POINT

$\bullet_3$ SOWC COLLECTOR WELL

SCALE
0 2000 4000 6000 FEET

FIGURE 3.2-1

MODEL AREA LOCATION MAP

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REFERENCES

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO-IND.; SHANDON, OHIO, GREENHILLS, OHIO; ADDYSTON, OHIO-KY DATED 1955 (PR 1961), 1965 (PR 1981), 1965 (PR 1981), AND 1982 SCALE- 1" = 2000'.

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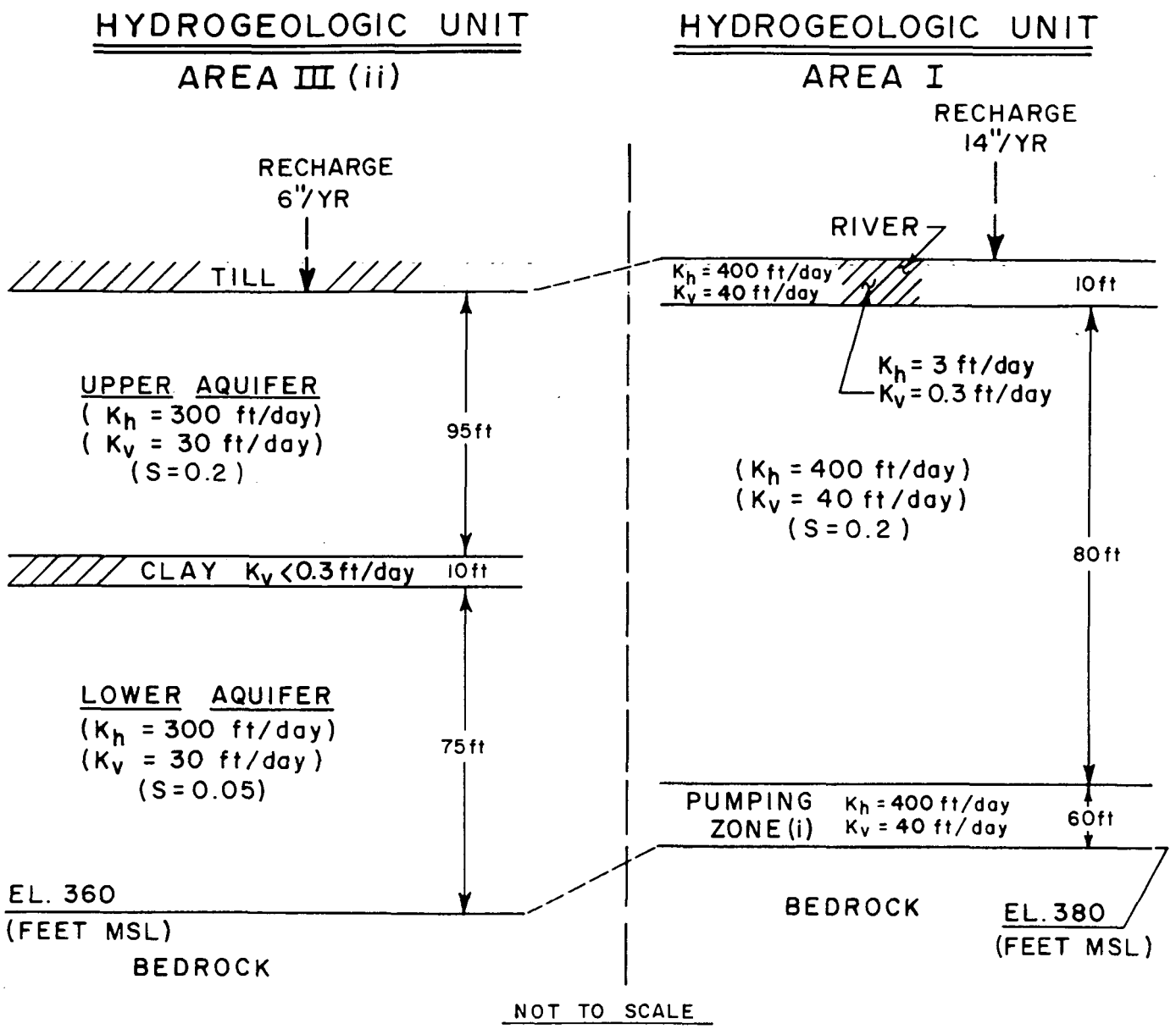

1. THIS DESIGN REPRESENTS A FIRST APPROXIMATION FOR TYPICAL MODEL INPUT PARAMETERS.
2. (i) COLLECTOR WELLS 1 & 2 PUMP WATER BETWEEN ELEV'S 409 & 433 FT.
3. (ii) HYDROGEOLOGIC UNITS ARE DEFINED IN TEXT.
4. K_h = HORIZONTAL HYDRAULIC CONDUCTIVITY
   K_v = VERTICAL HYDRAULIC CONDUCTIVITY

FIGURE 3.2-2

CONCEPTUAL DESIGN FOR "ZONE OF INFLUENCE" GROUNDWATER MODEL

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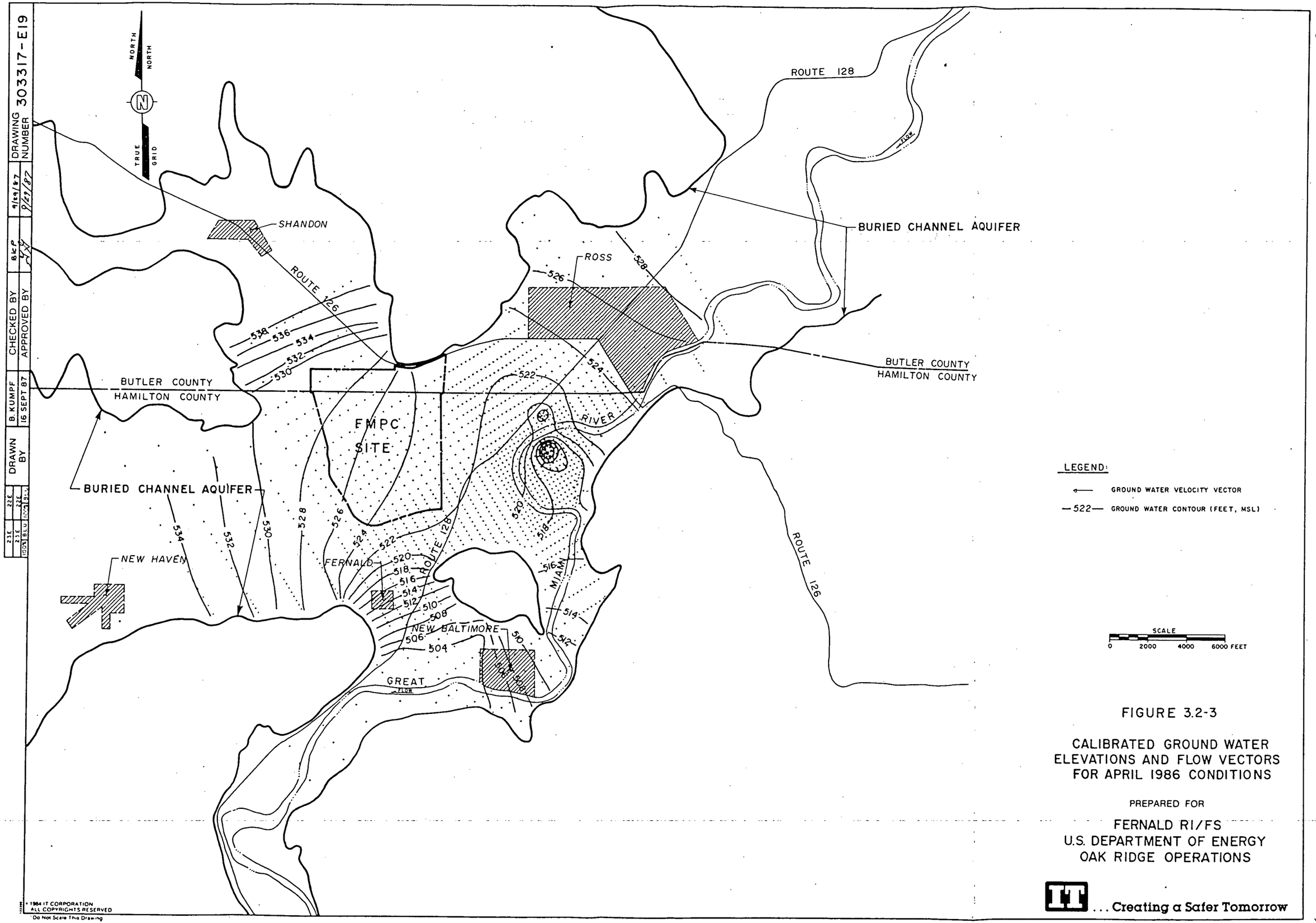

FIGURE 3.2-3

CALIBRATED GROUND WATER ELEVATIONS AND FLOW VECTORS FOR APRIL 1986 CONDITIONS

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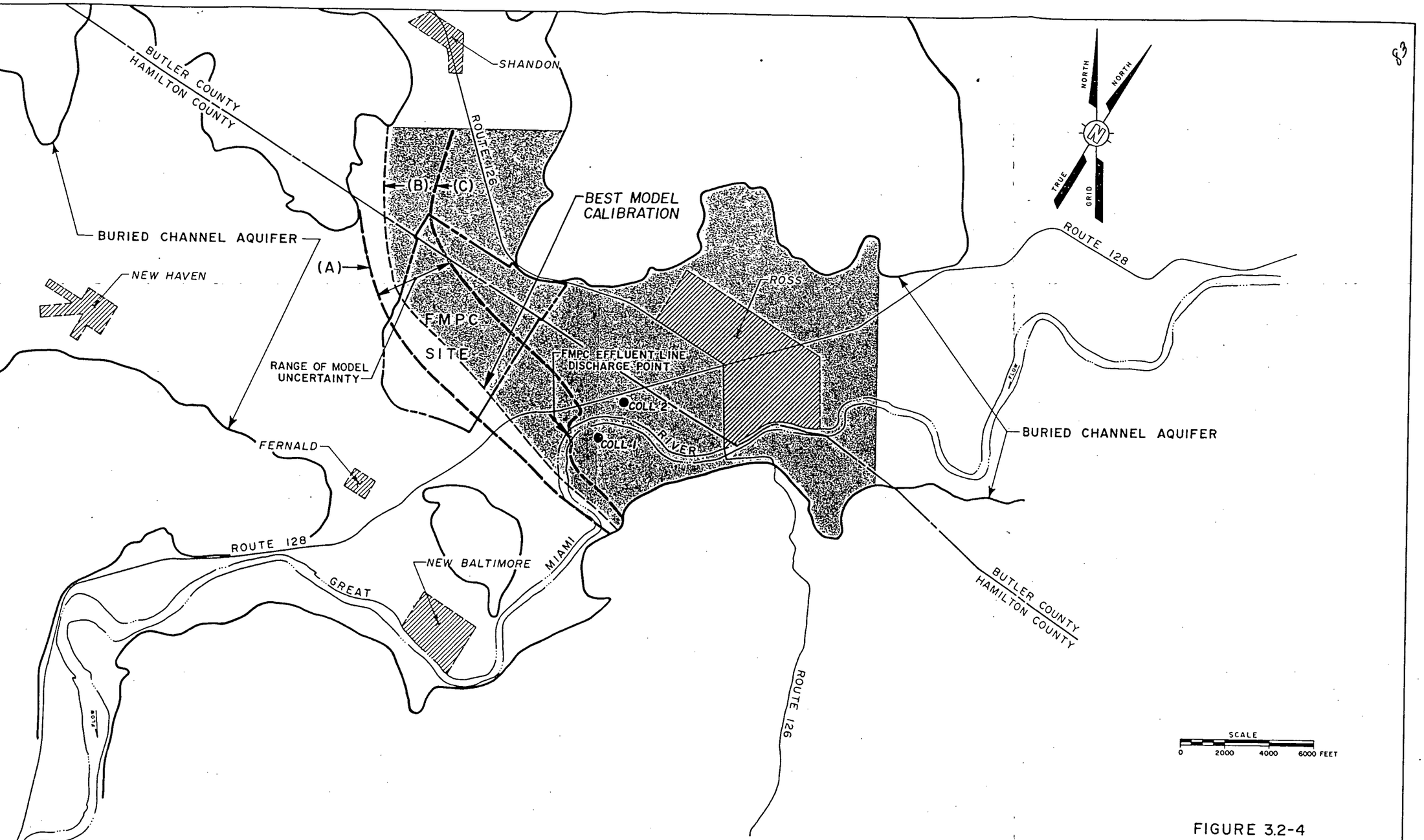

LEGEND

POTENTIAL "ZONE OF INFLUENCE" PREDICTED FROM GROUND WATER MODELING

(A) RIVER LEAKAGE FACTOR 0.17 DAY^{-1}

(B) RIVER LEAKAGE FACTOR 0.35 DAY^{-1}

(C) RIVER LEAKAGE FACTOR 3.5 DAY^{-1}

FIGURE 3.2-4

LIMITS OF COLLECTOR WELL PUMPING "ZONE OF INFLUENCE"

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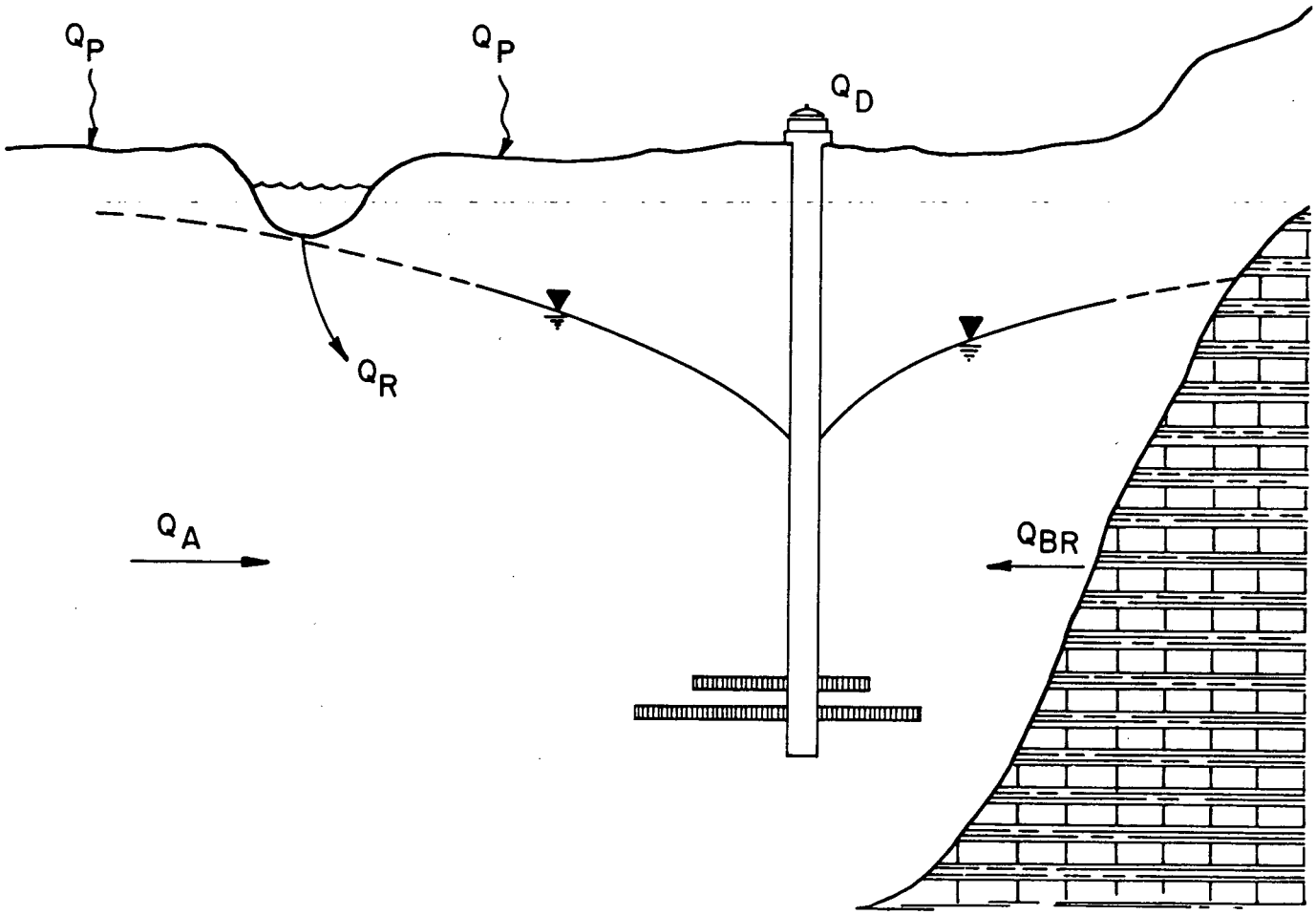

Q_D = PUMPING DISCHARGE FROM COLLECTOR WELLS

Q_P = COMPONENT FROM PRECIPITATION

Q_{BR} = COMPONENT FROM BEDROCK AQUIFER

Q_R = COMPONENT FROM RIVER

Q_A = COMPONENT FROM SAND AND GRAVEL AQUIFER

$Q_D = Q_P + Q_{BR} + Q_R + Q_A$

FIGURE 3.2-5

WATER BALANCE FOR COLLECTOR WELL DISCHARGE

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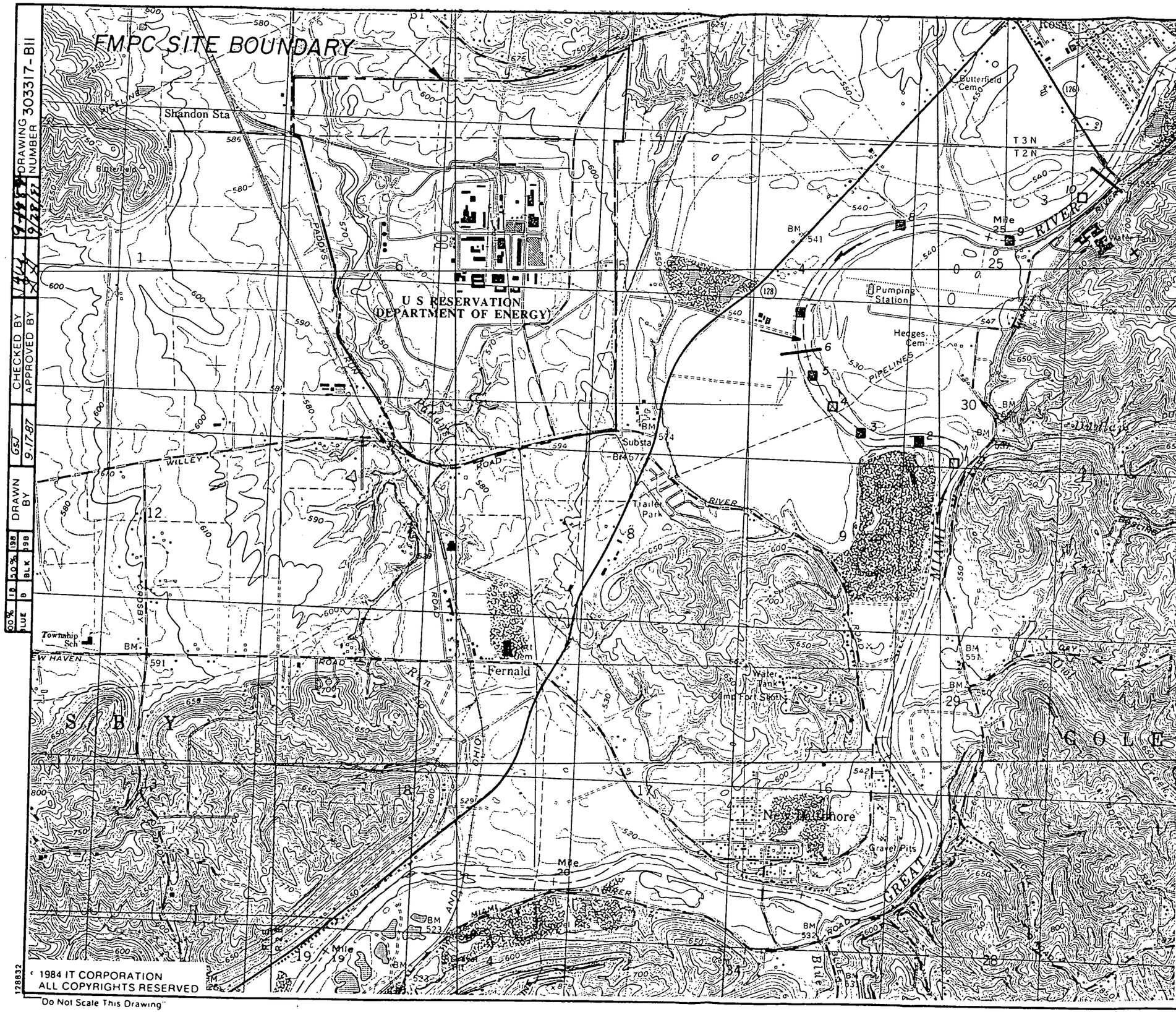

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90%	10	50%	198
BLUE	B	BLK	198

128832

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- □ 10 POINT SURFACE WATER SAMPLE LOCATION
- ■ 7 POINT SURFACE WATER AND SEDIMENT SAMPLE LOCATION
- —— 6 MULTIPLE SAMPLE LOCATION
- → FMPC EFFLUENT LINE DISCHARGE POINT
- → RIVER FLOW DIRECTION

REFERENCES:

7.5 MINUTE USGS TOPOGRAPHIC MAPS OF HARRISON, OHIO - IND., SHANDON, OHIO GREENHILLS, OHIO, CINCINNATI WEST OHIO, ADDYSTON, OHIO - KY, AND HOOVEN OHIO. DATED: 1955 (PRI 1981), 1965 (PRI 1981), 1965 (PRI 1981), 1961 (PRI 1981), 1982 AND 1981 SCALE: 1" = 2000

SCALE

0 2000 4000 FEET

FIGURE 5.1-1

SURFACE WATER AND SEDIMENT SAMPLING POINTS

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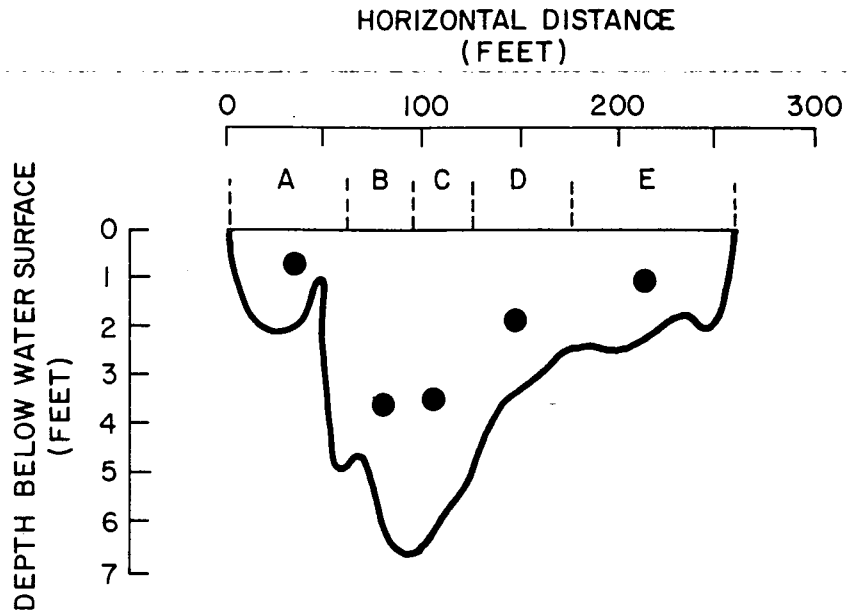

LEGEND

● WATER SAMPLING LOCATIONS

A-E CROSS-SECTIONAL SUBDIVISION DESIGNATION (SPACING WEIGHTED BY DISCHARGE)

NOTES:

1. CROSS-SECTIONAL ORIENTATION LOOKING DOWNSTREAM.
2. VERTICAL EXAGGERATION 25:1.
3. SAMPLES TAKEN FROM 6/10 DEPTH.
4. COMPOSITES FROM THREE SAMPLING TRAVERSES OBTAINED.
5. SECTION LOCATION IS SHOWN ON FIGURE 5.1-1.

FIGURE 5.4-1

CHANNEL CROSS-SECTION WITH SAMPLING LOCATIONS AND DEPTHS ROSS BRIDGE (LOCATION 11)

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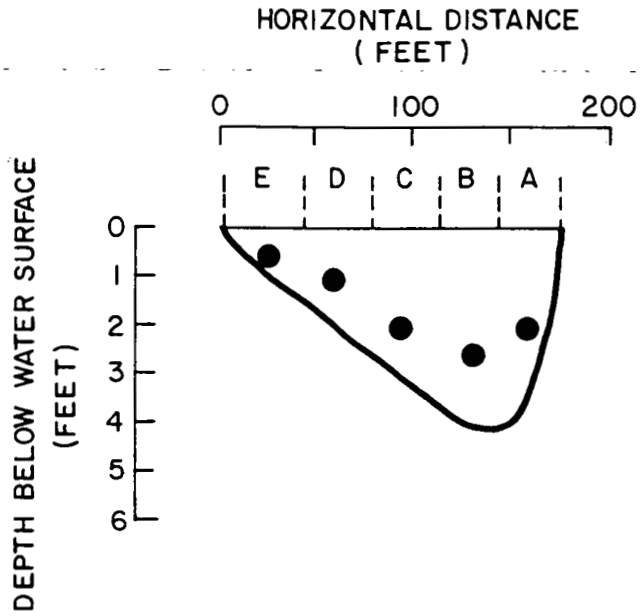

LEGEND

● WATER SAMPLING LOCATION

A-E CROSS-SECTIONAL SUBDIVISION DESIGNATION (SPACING APPROXIMATELY EQUAL)

NOTES:

1. CROSS-SECTIONAL ORIENTATION LOOKING DOWNSTREAM.
2. VERTICAL EXAGGERATION 25:1.
3. SAMPLES TAKEN FROM 6/10 DEPTH.
4. COMPOSITES FROM TWO SAMPLING TRAVERSES OBTAINED 9/12/87; DISCRETE SAMPLES FROM TWO TRAVERSES OBTAINED 9/14/87.
5. SECTION LOCATION IS SHOWN ON FIGURE 5.1-1.

FIGURE 5.4-2

CHANNEL CROSS-SECTION WITH SAMPLING LOCATIONS AND DEPTHS NEAR OUTFALL (LOCATION 6)

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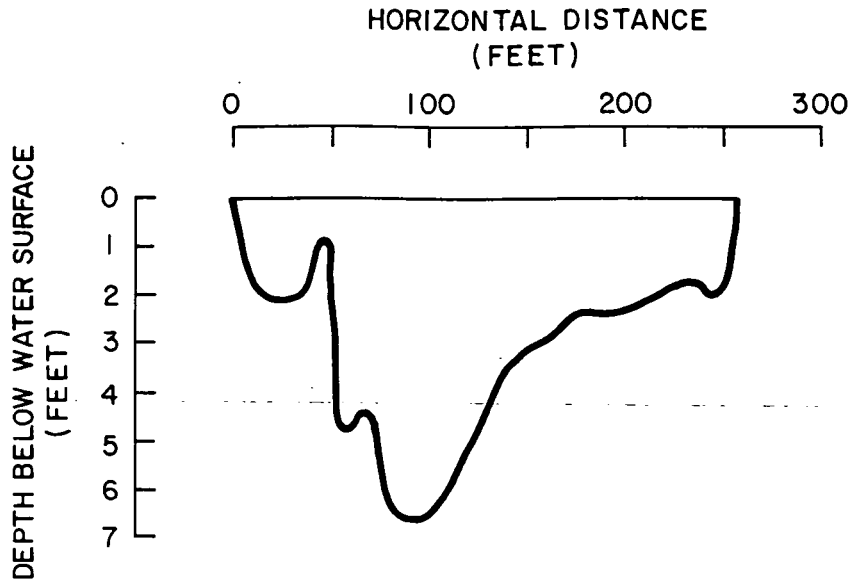
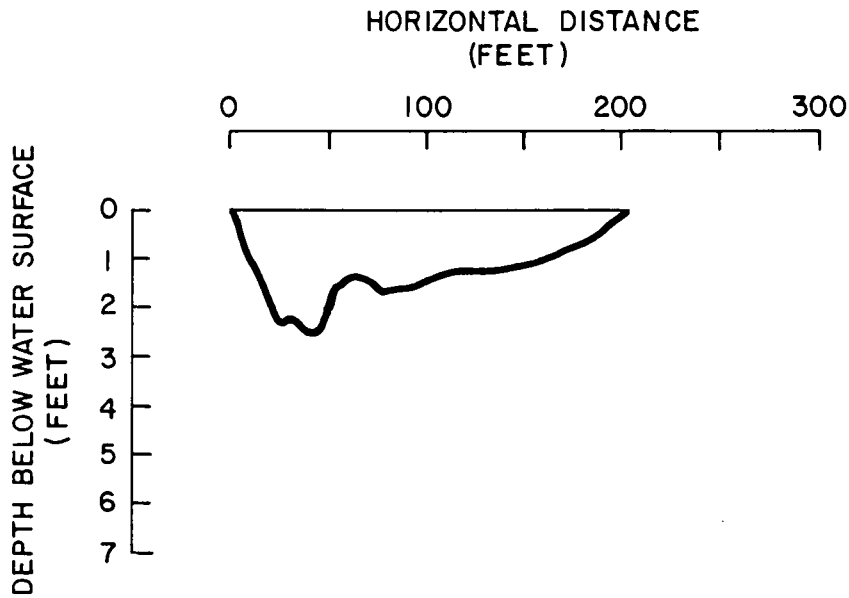


A. MEASURED RIVER BOTTOM CONFIGURATION, SEPT. 11, 1987



B. HISTORICAL RIVER BOTTOM CONFIGURATION, AFTER DOVE (1961)

NOTES:

1. CROSS-SECTIONAL ORIENTATION LOOKING DOWNSTREAM.
2. VERTICAL EXAGGERATION 25:1.
3. SECTION LOCATION IS SHOWN ON FIGURE 5.1-1.

FIGURE 5.4-3

PRESENT VERSUS HISTORICAL BOTTOM CONFIGURATION, ROSS BRIDGE (LOCATION II)

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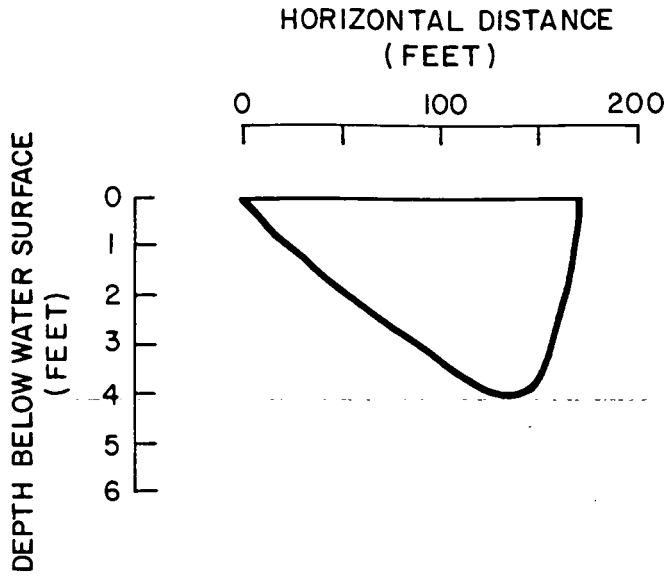
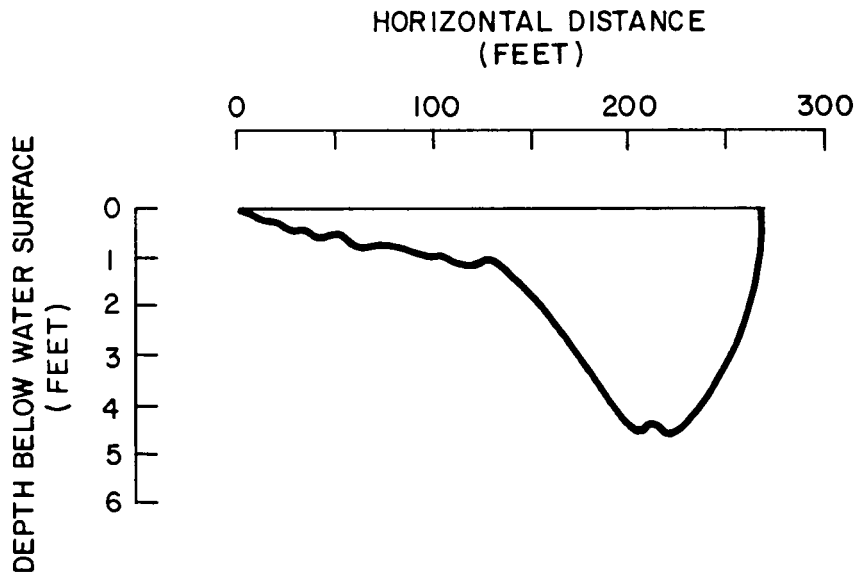


A. MEASURED RIVER BOTTOM CONFIGURATION, SEPT. 12, 1987



B. HISTORICAL RIVER BOTTOM CONFIGURATION, AFTER DOVE (1961)

NOTES:

1. CROSS-SECTIONAL ORIENTATION LOOKING DOWNSTREAM.
2. VERTICAL EXAGGERATION 25:1.
3. SECTION LOCATION IS SHOWN ON FIGURE 5.1-1.

FIGURE 5.4-4

PRESENT VERSUS HISTORICAL BOTTOM CONFIGURATION, NEAR OUTFALL (LOCATION 6)

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APPENDIX A

APPENDIX A

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APPENDIX A

A.1.0 INTRODUCTION

The selection of the most appropriate modeling code for this study is based on technical requirements and objectives of the study, the types and amount of data available, the accessibility of the code and its compatibility with available equipment. The SWIFT III computer code was selected for use in both this hydrogeologic study of the FMPC discharge and the overall, site-wide Remedial Investigation and Feasibility Study (RI/FS). Use was also made of IT's GEOFLOW model to fully satisfy the initial requirements of the FMPC discharge study.

Due to the complexities associated with data input to the SWIFT III code and calibration of the resulting model, IT opted to undertake a parallel effort to prepare and calibrate a finite element model of the hydrogeologic study area using GEOFLOW. This effort has led to the successful development of a 2-D numerical model which adequately reproduces ground water data obtained in the field. Input parameters used in this calibrated model have been transferred to the SWIFT III model, and a calibrated SWIFT III Model was produced. Since the GEOFLOW model progressed at an accelerated rate, the sensitivities of several model input parameters were tested using GEOFLOW with the results presented in this appendix.

The following sections of this appendix describe the development, calibration, and testing of the GEOFLOW model and preparation and testing of the SWIFT III model.

A.2.0 GEOFLOW MODELING

Calibration of the two-dimensional SWIFT III model was achieved by simultaneously using GEOFLOW to model the region of interest. GEOFLOW, developed by IT, is a two-dimensional (2-D) finite element, ground water flow and solute mass transport computer program that can be applied to confined, semiconfined, or unconfined aquifers. The program consists of two independent subprograms. By providing hydrodynamic properties such as hydraulic conductivity, storage coefficient, pumping rate, etc., the hydrodynamic subprogram computes ground water elevations and, consequently, the ground

water velocity vectors. The resulting velocity vectors are incorporated into the solute transport subprogram to yield the concentration distribution of chemical constituents in the flow domain. Transient and steady-state solutions for both the flow and mass transport equations can be computed by the program. The governing equations, features, and assumptions of the GEOFLOW program are described further in the user's manual (IT, 1986).

The advantages provided by GEOFLOW include extensive documentation, user-oriented routines for data input, and a graphical postprocessing program to plot ground water surface contours, velocity vectors, and isopachs from the computer output.

A.2.1 Model Setup

A conceptual design for the model was developed based on historical data (Section 3.0) as well as current information gained from work at the site. The principal objective of the modeling effort is to determine the relationship between the Great Miami River, the aquifers in the region, and the SOWC pumping wells. For this reason, the model grid was chosen to cover the entire area of possible ground water influence by the SOWC pumping wells. The grid north was oriented 30 degrees west of the true north to orient the bedrock trough approximately west-east across the grid. In the grid west-east direction, the grid extends from two miles west of the FMPC to approximately one-half mile east of Ross. In the grid north-south direction, it extends from three-quarters of a mile north of Shandon to one-half mile south of New Baltimore. At the extremes of the model area, where less detail is required, elements are 2,000 by 2,000 feet. The element size becomes gradually smaller inward and reaches a minimal size of 250 by 250 feet in the area surrounding the SOWC pumping wells and the meander loop on the Great Miami River. In all, the rectangular grid system consists of 44 rows and 51 columns. The length along the x axis is 32,000 feet and the length along the y axis is 25,000 feet.

GEOFLOW is a finite element program and SWIFT III is a cell-centered finite difference program. The same grid was developed for the two models. The model grid for SWIFT III is shown in Figure A-1 and the model grid for GEOFLOW is shown in Figure A-2. The nodes on the finite element (GEOFLOW) grid

correspond to the corners of the cell-centered finite difference (SWIFT III) grid and likewise the element centers of the finite element grid correspond to the centers of the finite different cells. This permits a one-to-one correlation between cells and elements, and facilitates transfer of information between the two models.

Subsequent to preparation of the initial grid, elements within the study area representing regions of relatively impermeable bedrock were deleted from the GEOFLOW model to improve model efficiency and to conserve computer storage space. These elements are shown as the shaded areas in Figure A-2. A similar option is not available in the SWIFT III model, so the same effect was achieved by assigning a minimum aquifer thickness to blocks composed of bedrock.

A.2.2. Model Input Data and Assumptions

The hydrodynamic subprogram of GEOFLOW requires input data on the area's geologic units and their characteristics; the type of aquifer (confined, semiconfined, or unconfined); the hydrogeologic characteristics of the aquifer, including hydraulic conductivity and thickness; recharge and discharge zones; and ground water usage. The user must also enter either the ground water flow rate or the hydraulic heads (water level elevations) along the boundaries of the study area.

Because of the persistance of a relatively stable cone of depression in the area around the SOWC collector wells (Section 2.1), a steady-state approach to ground water modeling was taken.

The initial data entered into the model were based on previous study reports, data obtained from field and laboratory measurements, and literature reviews. These data are described below.

Aquifer Type, Thickness, and Hydraulic Conductivity

The geology of the site has been discussed in Section 2.1. The aquifer is composed of highly permeable, well-sorted sand and gravel. For the initial 2-D model the study area was modeled as an unconfined aquifer. The study area

encompasses several distinct hydrogeologic environments as defined by Spieker 1968a and described in Section 2.1 of this report. The following table correlates the model zones used in this study with the Hydrogeologic Environments described by Spieker.

MODEL ZONE	SPIEKER 1968a HYDROGEOLOGIC ENVIRONMENT	BRIEF DESCRIPTION
1	I	Sand & gravel aquifer with no overlying till
2	V	Bedrock
3	III	Sand and gravel aquifer with overlying till

The river is treated separately and a fourth zone was added during calibration as described later in this section. The Paddy's Run area was also treated as a special case, it is in the Type III Hydrogeologic Environment (Spieker, 1968a), but has the recharge characteristics of the Type I Hydrogeologic Environemnt.

Bedrock elevations range from 350 to over 700 feet MSL. These elevations were used to represent the base of the aquifer and were incorporated into the model by entering the corresponding elevation for each element of the grid system. Since observed ground water elevations in the study area fall below 550 feet MSL, elements with bedrock base elevations of greater than 575 feet MSL were deleted from the model. The borders that these elements share with elements retained in the grid were treated as no-flow boundaries.

Hydraulic conductivities have been extensively researched in the study area. The assumed initial hydraulic conductivities and ranges expected in Zones 1 and 3 are given in Table 3.2.5.

Recharge

Estimates of recharge from precipitation in the study area range from 8 to 20 inches in Zone 1 and in the Paddy's Run area. Recharge from precipitation ranges from 3 to 12 inches in Zone 2 (Table 3.2.5). The aquifer has the potential to receive recharge from the Great Miami River. Therefore, the river was modeled by assuming that the river bed is equivalent to an overlying

semiconfined aquifer. The leakage factor assigned for the confining layer was based upon a vertical permeability of two to three orders of magnitude lower than the horizontal permeability of the aquifer. The stage in the river was assumed to vary from 525 feet MSL at the upstream end to 494 feet at the downstream end of the study area. These values were obtained from a HEC-2 model of the river, using a river stage of 524 feet MSL at the Ross Bridge (Described in Section 3.1).

Pumping Rates

Four pumping wells were included in the model. These wells and their assigned pumping rates are as follows (Table 3.2.5):

WELL	ASSUMED PUMPING RATE (feet^3/day)
SOWC Collector 1	1,644,000 (12.3 MGD)
SOWC Collector 2	822,000 (6.1 MGD)
FMPC P-3	64,000 (.5 MGD)
Albright and Wilson	19,000 (.14 MGD)

Ground Water Elevations

The ground water contour map for April 1986 (Figure 2.1-13) was used to calibrate the model. The ground water contours were extrapolated to the model boundaries to obtain ground water elevation at these locations. Ground water levels along the perimeter of the grid within the Hydrogeologic Environments I and III were held constant during the modeling runs by assigning the extrapolated ground water elevations to each of the boundary nodes.

A.2.3 Model Calibration

The objective of model calibration is to use realistic model input to produce an output consistent with actual values (water levels) observed in the field. An iterative procedure was performed to calibrate the model by comparing computed ground water levels with observed data for April 1986. During this process, the differences between actual and computed ground water elevations were examined and hypotheses developed to explain the observed deviations. If the calibration was not satisfactory, the parameters were modified, within limits consistent with the data presented in Table 3.2.5, and a new simulation was performed. This process of examining the results, explaining the variances, and modifying the necessary input parameters was repeated until results that closely matched the observed data were obtained.

River bed leakage and, to a lesser extent, aquifer hydraulic conductivity were determined to be the critical factors in the flow model calibration. Calibrated values for these and other input parameters are presented in Table A.1. For the final calibration run, it was necessary to add a third hydraulic conductivity zone in the northernmost two rows of elements grid north of the FMPC site (Zone 4). The aquifer in this area has been found to contain more clay; thus, the addition of a zone of lower conductivity in this area is consistent with field observations.

Table A.2 presents a comparison of observed versus computed ground water levels for the calibrated model for 45 wells in the study area. Differences between observed and computed values range from -4.7 feet to +2.7 feet with 38 percent (N=17) of the values falling with ±1 feet, 75 percent (N=34) falling within ±2 feet, 91 percent (N=41) falling within ±3 feet, and 98 percent (N=44) falling within ±4 feet. The average difference between the observed and computed elevations is -0.39 feet. This average, which is obtained by dividing the sum of the differences, by the the number of observations (wells), shows that there is a net 4.7-inch bias in the computed values. A second statistical parameter was calculated by dividing the sum of the absolute values of the differences by the number of observations. The calculated value of this parameter is 1.50, indicating the average absolute difference between the actual and computed values is approximately 1.5 feet. A third statistical parameter is the standard deviation of the differences. This parameter had a value of 1.80. The values calculated for these three parameters are considered acceptable for a ground water flow model encompassing an area of nearly 36 square miles.

The comparison of actual versus computed values is even more satisfactory if Wells 12-7A, State-16, FMPC-3, Pallet Co, and WW-1 are excluded from the analyses (Table A.3). These five wells are located near the grid boundaries where limited information is available concerning ground water elevations. Ground water levels heads at these boundaries have been extrapolated from the closest wells and could easily be in error by several feet or more. Computed ground water elevations at nodes near the grid boundaries are more adversely impacted by errors in the assumed boundary conditions than nodes in the interior of the grid. Excluding the above five wells gives a average

difference between the observed and calculated values of -0.14 feet, an average absolute difference of 1.24 feet, and a standard deviation of 1.47 feet. This is a highly satisfactory comparison, particularly since a majority of the remaining 40 wells are located in the primary areas of interest (near the FMPC plant and the collector wells).

It should be noted that the set of calibrated values presented in Table A.1 may not be the only set that gives a satisfactory calibration. Other combinations of aquifer hydraulic conductivity, river bed leakage, and recharge rates may give similar results. Determining what these possible combinations are requires extensive testing. Preliminary results for the calibration runs suggest that acceptable calibration can be obtained with hydraulic conductivities as much as 1.5 times higher than the values presented in Table A.1. The sensitivity of the model to modification of river bed leakage and recharge to the aquifer are discussed in the following section.

A.2.4 Sensitivity Analysis

The sensitivity of the model to changes in the values of the input parameters was limited to two variables, river bed leakage and recharge. Of these, river bed leakage is of the most interest since it is the variable that is least well known and of greatest concern relative to the impact of the Great Miami River on the collector wells.

A summary of the seven sensitivity runs that were performed is presented in Table A.3. In two runs, recharge was varied while aquifer hydraulic conductivity and river bed leakage were held constant. In the other five runs, aquifer hydraulic conductivity and recharge were held constant while the river bed leakage was multiplied by a factor of 0.1, 0.5, 2, 10, and 100 in relation to the calibrated run.

The results, presented in Table A.3, show that changes in recharge produce proportional changes in the average difference between the observed and computed values. Increasing the recharge rate relative to the calibrated run results in a higher net negative difference between the observed and computed values while lowering recharge gives a slightly positive net difference. In fact, the lower recharge rates give a slightly better calibrated model

relative to two of the three statistical parameters provided all 45 wells are included in the analysis. However, the differences are minor and what is perhaps the key parameter, the average difference between observed and computed values divided by the number of observations for the subset of 40 wells, is closer to zero for the calibrated run than for Sensitivity Run No. 2.

Decreasing the river bed leakage by a factor of 10 (Sensitivity Run No. 3) resulted in a poorly calibrated model. Computed ground water elevations were lower than observed levels at most wells, with differences exceeding 10 feet near the SOWC Collector Wells. Using this river bed leakage and changing other input parameters within their ranges of acceptable values did not produce a calibrated model, which suggests that actual river bed leakage is more than 0.035 day^{-1}.

Mean differences between actual and computed ground water levels were less than ±1 foot when river bed leakage was reduced or increased by a factor of 2. Differences in this range may be compensated for by manipulation of other input parameters. This indicates that actual river bed leakage may fall in the range of 0.175 and 0.7 day^{-1}.

Further increasing river bed leakage by factors of 10 and 100 produced increasingly poorer results, with computed elevations exceeding observed elevations by an average of one foot or more. By changing the river bed leakage from 0.35 day^{-1} to 3.5 day^{-1} the mean difference between the observed and computed elevations declined from -0.39 to -1.10 feet. However, an additional order of magnitude change in river bed leakage (to 35 day^{-1}) caused the mean difference between the observed and computed values to decrease only from -1.10 to -1.17 feet.

In order to develop a realistic zone of influence for the SOWC pumping wells, plots of flow vectors were produced from the GEOFLOW model output. The flow vectors indicated that the "zone of influence" extended to the model boundaries to the east of Ross and to the northeast of Shandon. The calibration run was used to produce a "best-fit" bound to the zone of influence. This is shown in Figure 3.2-4. River bed leakage was varied in

several sensitivity runs of the GEOFLOW model. The model proved to be somewhat sensitive to the river bed leakage parameter and the area of uncertainty shown in Figure 3.2-4 was developed by using factors of 0.5 times and 10 times the calibrated river bed leakage as the lower and upper bounds of the parameter, respectively.

A.3.0 SWIFT III MODELING

A.3.1 Model Input Data and Assumptions

The physical layout of the SWIFT III grid and model conceptualization are given in Subsection 3.2.2.1 and the grid configuration is shown in Figure A-1. SWIFT III accepts two levels of discretization, the larger more general level labeled "global system" and a finer subset called "local subsystems." The local subsystem is usually used in the representation of fractured media. For the water table aquifer case only the global system may be utilized. The global aquifer system was refined for the 2-D case by differentiating between the buried valley aquifer sands and gravels and outcrops of lower permeability bedrock.

Several boundary conditions were established for the model. It was assumed that no significant vertical recharge occurred through the base of the sand and gravel aquifer. Thicknesses for each cell were determined by interpretation of geologic logs and published literature. Bedrock outcrops that define the aquifer channels were also assumed to be no flow boundaries. The amount of contributing bedrock ground water flow may be considered insignificant in light of the large transmissivity of the aquifer. Finally, all areas along the grid model boundaries that lie within the buried valley aquifer were given constant potentiometric heads (Dirichlet condition) based on the best available data. Specific values for grid boundary conditions are listed in Table A.4.

Recharge and discharge potential along the Great Miami River was accommodated by source/sinks located at each river cell and by designating a constant hydraulic head to each cell. The source/sinks are specified within the SWIFT III data input as boundary conditions equivalent to those used for wells. This equivalent well submodel for the river requires setting a water

level within the well bore equal to the river stage level and providing a well index based upon the river bed leakage. The well submodel defines the recharge/discharge to be equal to the product of the well index with the difference between head in the cell and the head in the well bore. In this way, the well submodel is equivalent to the semiconfined river bed model used in GEOFLOW as discussed in Section A.2.2. Leakage factors due to variances in effective river bed leakage, aquifer hydraulic conductivities, and the potential for either river recharge or discharge were accounted for by varying well index.

Other variables that must be assigned values in the model include hydraulic conductivity in the vertical and horizontal directions (aquifer and bedrock), porosity (aquifer and bedrock), aquifer thickness, river recharge, and well pumping rates. Horizontal hydraulic conductivities in the aquifer were chosen as 400 feet per day (ft/day). A horizontal hydraulic conductivity of 0.003 ft/day was assigned to represent bedrock. A porosity of 0.1 was assumed for the aquifer and a porosity of 0.05 was assumed for bedrock. Thicknesses of the aquifer material for each cell were estimated to the nearest 10 feet. The river bed leakage was assigned to be equal to that obtained from the GEOFLOW calibration.

A.3.2 MODEL CALIBRATION

To accomplish model calibration, boundary values and hydrogeologic parameters are varied across a range of known conditions until a high correlation between point source field measurements and model output is achieved. In this case, GEOFLOW was used to generate the optimum set of boundary and hydrogeologic parameters and this data set was transferred to SWIFT III. Input parameters in SWIFT III vary from GEOFLOW in one respect; GEOFLOW assigned three different horizontal hydraulic conductivity zones within the buried valley aquifer. For the SWIFT III model one zone was used. It should be noted that the selection of three discrete zones for the GEOFLOW model was based on interpretation of field data and the differences between the zones were minor. Consequently, it is generally accepted that initially restricting SWIFT III to one general hydraulic conductivity value for the buried valley aquifer does not detract to any degree from its validity. Two calibration runs were performed using the SWIFT III model.

Table A.4 summarizes the input data and Table A.5 the current state of calibration for the SWIFT III model. Ground water elevations computed by the model are compared with actual water levels measured in 45 selected wells during April 1986. Ground water elevations generated by the model ranged from 4.7 feet above to 6.6 feet below the observed levels. The mean difference between the observed and calculated ground water levels is 0.11 feet, and the standard deviation of the mean is 2.23 feet. The average absolute difference of the differences is 1.74 feet. It is believed that an even better calibration could be achieved with minor input adjustments to SWIFT III. Reviewing output differences between GEOFLOW and SWIFT III (Table A.6) shows a significant correlation (0.90) with a standard deviation of 1.27 feet. Differential values ranged from -2.2 feet to +4.9 feet between the two models.

A.3.3 Sensitivity Analysis

The sensitivity of a model to variable inputs is an important part of model development. Two separate tests were made with the 2-D version of SWIFT III. The initial test utilized the given input calibration parameters established by GEOFLOW. A second run decreased the river bed leakage of the Great Miami River by one order of magnitude. The change resulted in a significant lowering of the aquifer ground water table for several thousand feet radially around the two SOWC collector wells. This type of analysis will be expanded during the site-wide RI/FS to include all inputs for which ranges of values have been published, i.e., hydraulic conductivity, porosity, surface recharge/discharge, and pumping rates. Part of this effort will be the identification of key sensitive input parameters. Sensitivity evaluation will continue as the model is expanded to three dimensions. New hydrogeologic conditions such as the inclusion of the "blue clay" layer may affect the rates and direction of ground water flow and solute transport. Continued refinement of SWIFT III will quantify the rough estimations given previously in this report for SOWC well field influence on a regional scale (Figure 3.2-4).

TABLE A.1

SUMMARY OF INPUT DATA FOR THE CALIBRATED GEOFLOW MODEL

I. Finite Element Grid System(a)

Length: (x-direction)	32,000 feet
Width: (y-direction)	25,000 feet
Number of elements:	1,553
Number of nodes:	1,688

II. Hydrogeologic Parameters

Base of aquifer:	From 350 to 550 feet MSL
Precipitation recharge:	
Zone 1 (river flood plane):	14 inches/year
Zone 2 (bedrock elements):	Not used
Zone 3 (aquifer covered by till):	6 inches/year
Zone 4 (top two rows of elements):	6 inches/year
Note: River elements lack precipitation recharge	
Hydraulic conductivities (K):	
Zone 1:	400 feet/day
Zone 2:	Not used
Zone 3:	300 feet/day
Zone 4:	150 feet/day
Note: Hydraulic conductivity is assumed isotropic within each zone	
Type of aquifer:	Unconfined
Flow regime:	Steady state

TABLE A.1
(Continued)

II. Hydrogeologic Parameters (cont'd)

Boundary conditions:

Grid lower west boundary:	492 feet MSL
Grid upper west boundary:	535 feet MSL
Grid north boundary:	540 feet MSL
Grid south boundary:	504 feet MSL
Grid east boundary:	530 feet MSL

Extraction wells production schedule:

Well Name	Pumping Rate
Collector 1 (SOWC)	1,644,000 $feet^3$/day
Collector 2 (SOWC)	822,000 $feet^3$/day
FMPC-P3	64,000 $feet^3$/day
Albright and Wilson	19,000 $feet^3$/day

River bed leakage factor: 0.35 day^{-1}

(a)Refer to Figure A-2 for grid element locations.

TABLE A.2
COMPARISON OF APRIL 1986 OBSERVED GROUND WATER LEVELS AND GEOFLOW COMPUTED VALUES

WELL IDENTIFICATION	GEOFLOW ELEMENT NUMBER (a)(b)	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
12-7A	50	527.4	531.0	-3.6	3.6
STATE 16	86	526.3	531.0	-4.7	4.7
H-124	91	505.8	504.5	1.3	1.3
H-122	109	521.4	523.8	-2.4	2.4
RE	140	515.7	516.0	-0.3	0.3
H-115	101	518.7	516.5	2.2	2.2
7-8A	189	520.7	522.0	-1.3	1.3
H-105	196	523.4	525.0	-1.6	1.6
BPH	199	525.0	526.5	-1.5	1.5
FMPC-17	238	523.9	525.0	-1.1	1.1
H-129	256	507.7	505.0	2.7	2.7
H-126	260	512.6	511.0	1.6	1.6
HK-	278	521.5	523.5	-2.0	2.0
FMPC-16-S	283	524.8	525.5	-0.7	0.7
FMPC-9	288	526.4	527.0	-0.6	0.6
DE	309	519.9	517.0	2.9	2.9
FMPC-18-S	364	525.3	526.0	-0.7	0.7
PALLET CO	334	525.4	529.0	-3.6	3.6
16-1-S	296	504.3	504.0	0.3	0.3
FMPC-3	368	524.9	528.0	-3.1	3.1
FMPC-14-D	399	524.1	526.0	-1.9	1.9
IT-5	421	522.7	523.0	-0.3	0.3
IT-1	458	523.2	524.0	-0.8	0.8
FMPC-10	464	523.6	526.5	-2.9	2.9
BLK or IT-3	547	522.5	522.0	0.5	0.5
IT-2	589	522.9	523.0	-0.1	0.1
FMPC-13-5	593	522.9	524.0	-1.1	1.1
02-E	615	522.2	522.0	0.2	0.2
IT-3	660	522.5	523.0	-0.5	0.5
H-127	727	522.3	522.0	0.3	0.3
IT-4	806	521.9	522.0	-0.1	0.1
B-3	841	520.2	522.0	-1.8	1.8
B-2	948	520.6	521.0	-0.4	0.4
SW-4A	978	519.1	521.0	-1.9	1.9
B-1	1015	521.3	521.0	0.3	0.3
SW-3A	1040	518.3	517.0	1.3	1.3
R-7	1158	520.7	519.0	1.7	1.7
B-4	1187	520.3	520.0	0.3	0.3
K-4	1277	520.6	522.0	-1.4	1.4
EL-1	1343	522.9	522.0	0.9	0.9
03	1351	526.5	524.0	2.5	2.5
ER-1	1375	525.9	524.0	1.9	1.9
LB-1	1449	522.3	524.0	-1.7	1.7

TABLE A.2
(Continued)

WELL IDENTIFICATION	GEOFLOW ELEMENT NUMBER (a)(b)	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
WK1	1463	525.5	524.0	1.5	1.5
WW-1	1473	528.2	525.5	2.7	2.7

SUM OF DIFFERENCES/NUMBER OF OBSERVATIONS	-0.39 feet
SUM OF ABSOLUTE DEVIATIONS/NUMBER OF OBSERVATIONS	1.50 feet
STANDARD DEVIATION OF DIFFERENCES	1.80 feet

(a)Wells are located in upper left corner of element.
(b)Refer to Figure A-2 for grid element locations.

TABLE A.3

SUMMARY OF GEOFLOW SENSITIVITY RUNS

		SENSITIVITY RUN NUMBERS						
	CALIBRATION RUN	1	2	3	4	5	6	7
Input Parameter								
Aquifer hydraulic conductivity (feet/day)								
Zone 1	400	400	400	400	400	400	400	400
Zone 2	–	–	–	–	–	–	–	–
Zone 3	300	300	300	300	300	300	300	300
Zone 4	150	150	150	150	150	150	150	150
Recharge from precipitation (inches/year)								
Zone 1	14	20	10	14	14	14	14	14
Zone 2	–	–	–	–	–	–	–	–
Zone 3	6	8	4	6	6	6	6	6
Zone 4	6	8	4	6	6	6	6	6
River bed leakage (factor) (day^{-1})	0.35(1.0)	0.35(1.0)	0.35(1.0)	0.035(0.1)	0.175(0.5)	0.70(2.0)	3.5(10.0)	35.0(100.0)
Comparison of observed versus computed results (Model Output) (feet)								
All 45 Wells:								
Mean difference(a)	-0.39	-0.89	+0.07	+3.52	+0.29	-0.76	-1.10	-1.17
Mean absolute difference(b)	1.50	1.81	1.42	3.87	1.66	1.73	2.03	2.10
Standard deviation of algebraic difference(c)	1.80	1.92	1.81	3.50	2.03	1.91	2.00	2.02

TABLE A.3
(Continued)

	CALIBRATION RUN	SENSITIVITY RUN NUMBERS 1	2	3	4	5	6	7
Wells 12-7A, State 16, PALLET CO, FMPC-3, and WW-1 excluded:								
Mean difference	-0.14	-0.63	+0.28	+4.01	+0.63	-0.54	-0.89	-0.97
Mean absolute difference(b)	1.24	1.55	1.24	4.03	1.41	1.52	1.83	1.91
Standard deviation of algebraic difference(c)	1.47	1.64	1.60	3.23	1.63	1.70	1.82	1.85

(a)Sum of observed minus computed values divided by number of wells.

(b)Sum of absolute value of observed minus computed value divided by number of wells.

(c)Standard deviation of observed minus computed values.

TABLE A.4

SUMMARY OF INPUT DATA FOR THE CALIBRATED SWIFT III MODEL

I. Finite Difference Grid System(a)

Length: (i-direction)	32,000 feet
Width: (j-direction)	25,000 feet
Number of cells:	2,244
Number of layers:	1

II. Hydrogeologic Parameters

Base of aquifer:	From 350 to 600 feet MSL
Precipitation recharge:	
Zone 1 (river flood plane):	14 inches/year
Zone 2 (bedrock high):	0 inches/year
Zone 3 (aquifer covered by till):	6 inches/year
Horizontal hydraulic conductivities (K):	
Zone 1 and Zone 3:	400 feet/day
Zone 2:	0.003 feet/day
Porosities:	
Zone 1 and Zone 3:	0.25
Zone 2:	0.1
Type of aquifer:	Unconfined
Flow regime:	Steady state

TABLE A.4
(Continued)

II. Hydrogeologic Parameters (cont'd)

Boundary conditions:

Grid lower west boundary:	492 feet MSL
Grid upper west boundary:	535 feet MSL
Grid north boundary:	540 feet MSL
Grid south boundary:	504 feet MSL
Grid east boundary:	530 feet MSL

Extraction wells production schedule:

Cell I, J(a)	Pumping Rate
34, 24	1.65×10^6 feet^3/day
37, 31	8.14×10^5 feet^3/day
12, 37	6.44×10^4 feet^3/day
7, 25	1.86×10^4 feet^3/day

River bed leakage factor: 0.35 day^{-1}

(a)Refer to Figure A-1 for grid cell locations.

TABLE A.5
COMPARISON OF APRIL 1986 OBSERVED GROUND WATER LEVELS AND SWIFT III COMPUTED VALUES

WELL IDENTIFICATION	SWIFT III(a) COLUMN I	SWIFT III(a) ROW J	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
12-7A	3	32	527.4	529.8	-2.4	2.4
STATE 16	4	41	526.3	529.9	-3.6	3.6
H-124	5	8	505.8	506.6	-0.8	0.8
H-122	5	26	521.4	524.3	-2.9	2.9
RE	5	17	515.7	517.6	-1.9	1.9
H-115	5	18	518.7	518.5	0.2	0.2
7-8A	6	25	520.7	523.1	-2.4	2.4
H-105	6	32	523.4	525.8	-2.4	2.4
BPH	6	35	525.0	526.8	-1.8	1.8
FMPC-17	7	31	523.9	525.0	-1.1	1.1
H-129	9	6	507.7	504.9	2.8	2.8
H-126	8	10	512.6	511.4	1.2	1.2
HK-	9	29	521.5	523.8	-2.3	2.3
FMPC-16-S	9	34	524.8	525.6	-0.8	0.8
FMPC-9	9	38	526.4	526.9	-0.5	0.5
DE	10	17	519.9	519.2	0.7	0.7
FMPC-18-S	10	37	525.3	526.2	-0.9	0.9
PALLET CO	9	42	525.4	530.1	-4.7	4.7
16-1-S	10	3	504.3	504.5	-0.2	0.2
FMPC-3	11	40	524.9	527.9	-3.0	3.0
FMPC-14-D	12	39	524.1	526.7	-2.6	2.6
IT-5	13	28	522.7	523.1	-0.4	0.4
IT-1	14	33	523.2	523.9	-0.7	0.7
FMPC-10	14	39	523.6	526.1	-2.5	2.5
BLK or IT-3	16	25	522.5	522.3	0.2	0.2
IT-2	18	33	522.9	522.8	0.1	0.1
FMPC-13-5	18	38	522.9	523.8	-0.9	0.9
O2-E	18	26	522.2	521.9	0.3	0.3
IT-3	20	36	522.5	522.7	-0.2	0.2
H-127	21	29	522.3	521.3	1.0	1.0
IT-4	24	34	521.9	521.4	0.5	0.5
B-3	24	34	520.2	521.4	-1.2	1.2
B-2	28	32	520.6	520.2	0.4	0.4
SW-4A	29	25	519.1	518.0	1.1	1.1
B-1	29	31	521.3	519.6	1.7	1.7
SW-3A	31	24	518.3	515.0	3.3	3.3
R-7	34	26	520.7	514.1	6.6	6.6
B-4	31	28	520.3	518.2	2.1	2.1
K-4	40	27	520.6	520.5	0.1	0.1
EL-1	42	30	522.9	520.7	2.2	2.2
O3	42	38	526.5	522.3	4.2	4.2
ER-1	45	22	525.9	523.3	2.6	2.6
LB-1	48	30	522.3	523.9	-1.6	1.6

TABLE A.5
(Continued)

WELL IDENTIFICATION	SWIFT III COLUMN I	ROW J	OBSERVED GROUND WATER LEVEL (feet MSL)	COMPUTED GROUND WATER LEVEL (feet MSL)	DIFFERENCE OBS.-COMP. GWL (feet)	ABSOLUTE OF DIFF. (feet)
WK1	49	24	525.5	523.8	1.7	1.7
WW-1	48	35	528.2	524.4	3.8	3.8

SUM OF DIFFERENCES/NUMBER OF OBSERVATIONS	0.11 feet
SUM OF ABSOLUTE DEVIATIONS/NUMBER OF OBSERVATIONS	1.74 feet
STANDARD DEVIATION OF DIFFERENCES	2.23 feet

(a)Refer to Figure A-1 for grid cell locations.

TABLE A.6
COMPARISON OF GEOFLOW AND SWIFT III RESULTS

GEOFLOW ELEMENT NUMBER (a)(b)	SWIFT III(c) COLUMN I	ROW J	GEOFLOW GROUND WATER LEVEL (feet MSL)	SWIFT III PRESSURE (psi)	EQUIVALENT GWL (feet MSL)	DIFFERENCE GEOFLOW -SWIFT (feet)	ABSOLUTE DIFFERENCE
50	3	32	531	229.56	529.8	1.2	1.2
86	4	41	531	229.62	529.9	1.1	1.1
91	5	8	504.5	219.53	506.6	-2.1	2.1
109	5	26	523.8	227.18	524.3	-0.5	0.5
140	5	17	516.0	224.30	517.6	-1.6	1.6
101	5	18	516.5	224.70	518.5	-2.0	2.0
189	6	25	522.0	226.68	523.1	-1.1	1.1
196	6	32	525.0	227.84	525.8	-0.8	0.8
199	6	35	526.5	228.30	526.8	-0.3	0.3
238	7	31	525.0	227.49	525.0	0.0	0.0
256	9	6	505.0	218.80	504.9	0.1	0.1
260	8	10	511.0	221.61	511.4	-0.4	0.4
278	9	29	523.5	226.99	523.8	-0.3	0.3
283	9	34	525.5	227.74	525.6	-0.1	0.1
288	9	38	527.0	228.32	526.9	0.1	0.1
309	10	17	517.0	224.98	519.2	-2.2	2.2
364	10	37	526.0	228.03	526.2	-0.2	0.2
334	9	42	529.0	229.69	530.1	-1.1	1.1
296	10	3	504.0	218.63	504.5	-0.5	0.5
368	11	40	528.0	228.75	527.9	0.1	0.1
399	12	39	526.0	228.23	526.7	-0.7	0.7
421	13	28	523.0	226.68	523.1	-0.1	0.1
458	14	33	524.0	227.04	523.9	0.1	0.1
464	14	39	526.5	227.96	526.1	0.4	0.4
547	16	25	522.0	226.33	522.3	-0.3	0.3
589	18	33	523.0	226.56	522.8	0.2	0.2
593	18	38	524.0	226.97	523.8	0.2	0.2
615	18	26	522.0	226.17	521.9	0.1	0.1
660	20	36	523.0	226.51	522.7	0.3	0.3
727	21	29	522.0	225.91	521.3	0.7	0.7
806	24	34	522.0	225.96	521.4	0.6	0.6
841	24	34	522.0	225.96	521.4	0.6	0.6
948	28	32	521.0	225.42	520.2	0.8	0.8
978	29	25	521.0	224.48	518.0	3.0	3.0
1015	29	31	521.0	225.17	519.6	1.4	1.4
1040	31	24	517.0	223.16	515.0	2.0	2.0
1158	34	26	519.0	222.78	514.1	4.9	4.9
1187	31	28	520.0	224.56	518.2	1.8	1.8
1277	40	27	522.0	225.54	520.5	1.5	1.5
1343	42	30	522.0	225.63	520.7	1.3	1.3
1351	42	38	524.0	226.32	522.3	1.7	1.7
1375	45	22	524.0	226.77	523.3	0.7	0.7

TABLE A.6
(Continued)

GEOFLOW ELEMENT NUMBER (a)(b)	SWIFT III(c) COLUMN I	ROW J	GEOFLOW GROUND WATER LEVEL (feet MSL)	SWIFT III PRESSURE (psi)	EQUIVALENT GWL (feet MSL)	DIFFERENCE GEOFLOW -SWIFT (feet)	ABSOLUTE DIFFERENCE
1449	48	30	524.0	227.01	523.9	0.1	0.1
1463	49	24	524.0	227.00	523.8	0.2	0.2
1473	48	35	525.5	227.24	524.4	1.1	1.1

SUM OF DIFFERENCES/NUMBER OF OBSERVATIONS	0.26 feet
SUM OF ABSOLUTE DIFFERENCES/NUMBER OF OBSERVATIONS	0.90 feet
STANDARD DEVIATION OF DIFFERENCES	1.27 feet

(a)Wells are located in the upper left corner of the element.

(b)Refer to Figure A-2 for grid element locations.

(c)Refer to Figure A-1 for grid cell locations.

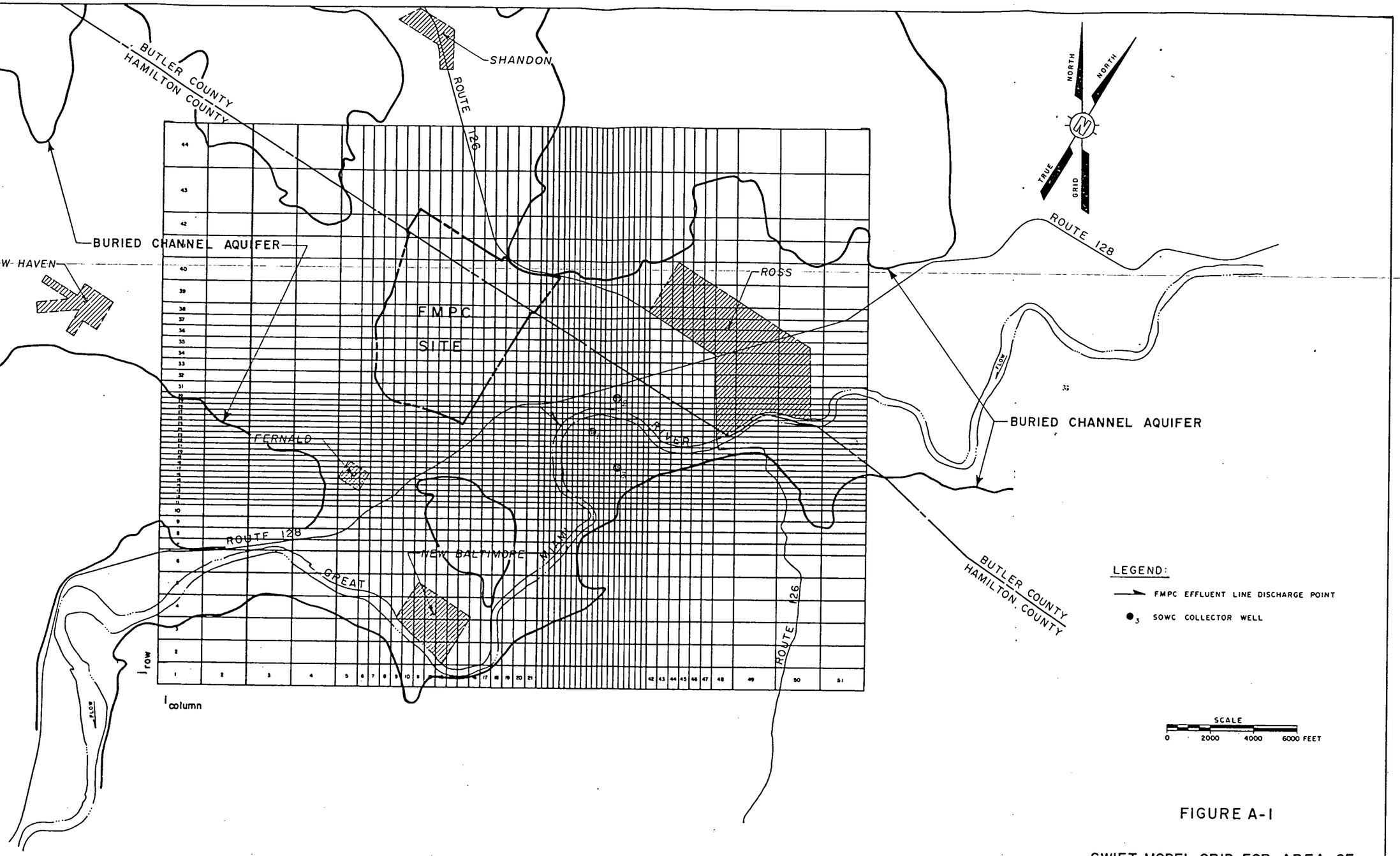

FIGURE A-1

SWIFT MODEL GRID FOR AREA OF INFLUENCE STUDY

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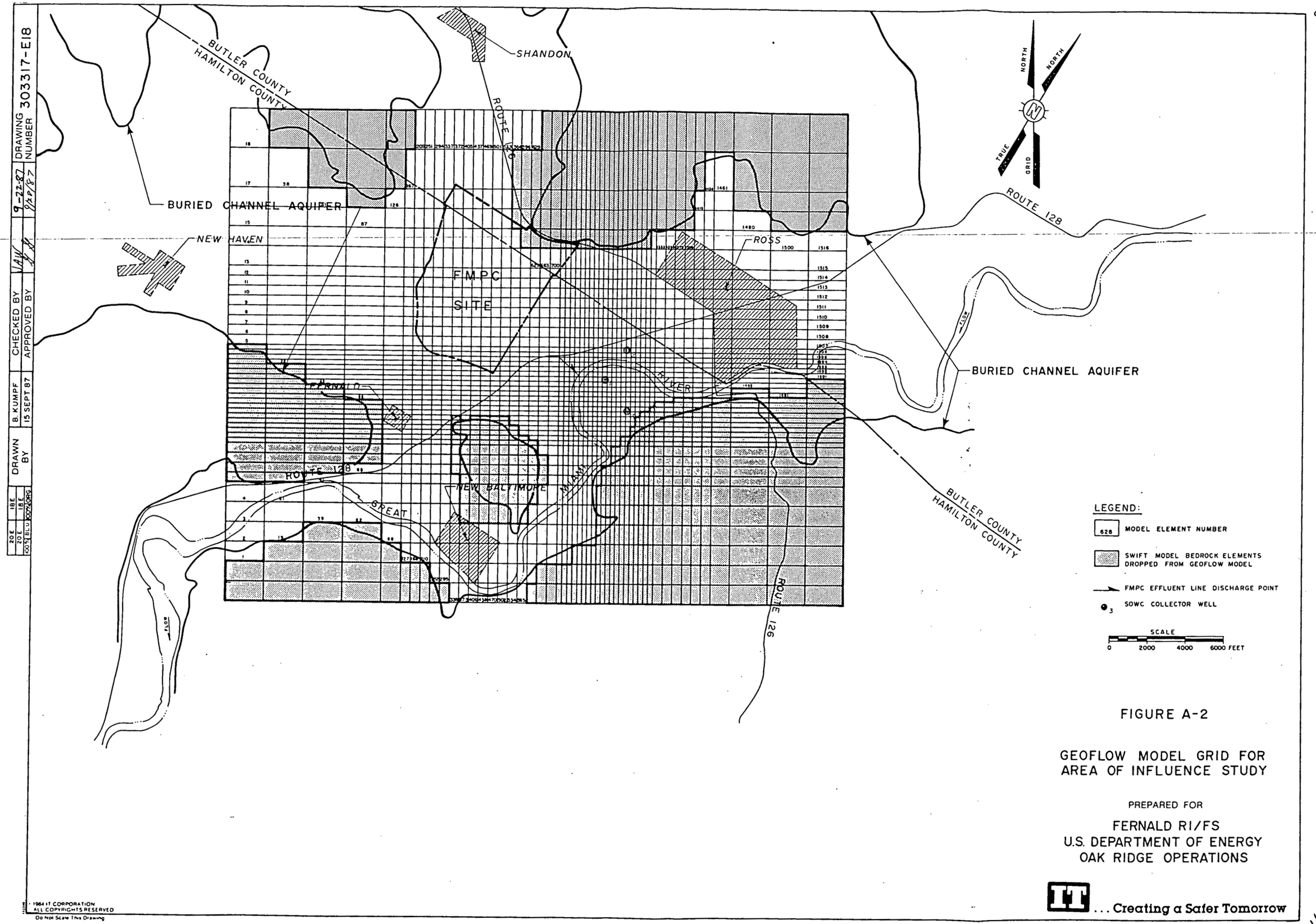

FIGURE A-2

GEOFLOW MODEL GRID FOR AREA OF INFLUENCE STUDY

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