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VOL 2

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FINAL DESIGN ANALYSIS
LIQUID WASTE DISPOSAL FACILITY
NORTH BOUNDARY EXPANSION
ROCKY MOUNTAIN ARSENAL

Commerce City, Colorado

FY 80

Project No. 34

Prepared by
BLACK & VEATCH
CONSULTING ENGINEERS
Kansas City, Missouri

For
U.S. ARMY ENGINEER DISTRICT, OMAHA
CORPS OF ENGINEERS
Omaha, Nebraska

April 1980

8/266 R25
Treatment Design
North Boundary

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CHAPTER I

INTRODUCTION

A. AUTHORITY AND SCOPE.

1. Authority. The Design Documents for the Liquid Waste Disposal Facility, North Boundary Expansion were authorized by Directive No. 14, Design 80-MCA-Omaha District, dated 16 August 1979.

2. Scope. This work consists of the design and preparation of Final Design Documents, with onboard review, for the construction of facilities to eliminate the migration of chemical contaminants through the North Boundary Aquifer Channel.

B. APPLICABLE CRITERIA.

1. General.

Appendix C with Supplement, Instructions for Contract No. DACA45-79-C-0019

2. Publications.

Department of Labor, Occupational Safety and Health Act Standards Manual

Department of the Army Technical Manual, TM 5-809-10, Seismic Design for Buildings

Department of Defense, DOD 4270.1-M, Construction Criteria Manual

Department of the Army Technical Manual, TM 5-822-2, General Provisions and Geometric Design for Roads, Streets, Walks, and Open Storage Areas

Department of the Army Technical Manual, TM 5-820-3, Drainage and Erosion-Control Structures for Airfields and Heliports

Department of the Army Technical Manual, TM 5-820-4, Drainage for Areas other than Airfields

Department of the Army Technical Manual, TM 5-810-5, Plumbing

National Electrical Code NFPA No. 70

Life Safety Code NFPA No. 101

National Electrical Safety Code

C. PURPOSE AND FUNCTION. The primary purpose and function of this project is to reduce contaminant levels leaving Rocky Mountain Arsenal to within approved standards. These contaminants are leaking from storage basins, entering the subsurface soil and water table, and in some cases are being transported across the Arsenal boundaries by ground water.

D. GENERAL DESCRIPTION OF WORK.

1. The northern boundary containment and treatment facility (Building 808) will be expanded as follows:

a. Extend the slurry trench containment barrier 3,840 feet to the east and 1,400 feet to the west.

b. Cap the trench with a cover of impermeable clay.

c. Install twenty-nine dewatering wells on the upstream side of the new containment barrier.

d. Install twenty-six recharge wells on the downstream side of the new containment barrier.

e. Install nineteen dewater wells on the upstream side of the existing containment barrier.

f. Connect all new wells to the existing treatment facility.

g. Expand existing Building 808 twenty-five feet to the east.

h. Provide influent and effluent wet wells at Treatment Building 808.

i. Construct a 12-foot wide all-weather perimeter and access road around the new barrier and well system.

j. Provide electrical power service to the entire area.

k. Provide an earth berm crossing at the "D" Street crossing of the containment barrier.

l. Provide a barrier, First Creek crossing, and low water perimeter road creek crossings.

m. Provide a ground water monitoring well system.

n. Provide a 6,000 gallon liquid propane storage tank at Building 808.

CHAPTER II

ARCHITECTURAL

A. GENERAL.

The existing insulated metal building will be enlarged to provide additional interior space. This will be accomplished by adding a 25 by 40 foot extension to the east end of the existing 40 foot wide building. The added area will be the same height as the existing building.

All new components used in the addition, except insulation, will match existing components in material type, gage, profile, color, etc. and will provide proper fit with existing components when installed.

Insulation for the metal building addition will be mineral fiber semi-rigid board with a vinyl vapor barrier face sheet. This material will replace the existing painted foam board because of the fire hazard of exposed foam materials. These materials, including urethane, extruded polystyrene, and expanded polystyrene will not be used because of their ability to ignite easily and to produce toxic smoke.

A sliding sash window, a personnel door, and an overhead-type door will be removed from the existing end wall and reinstalled at the same relative locations in the new end wall. The openings from which the doors are removed will be retained for passage between the new and existing building areas. The opening from which the window is removed will be closed.

Existing building components reused in the new construction will be touchup field painted as required to provide a like new finish. If existing components are damaged beyond reasonable repair during the construction process or otherwise unusable, they will be replaced with matching new components.

CHAPTER III

STRUCTURAL

A. SCOPE OF WORK. A listing of references applicable to this section is found in the introduction to this Design Analysis. Recommended structures to be provided by this project include the following:

1. Foundation slab and footing for expansion of existing treatment plant building.

2. Reinforced concrete influent and effluent wet well.

The design information listed in this section is applicable to all structures.

B. FOUNDATION DESIGN DATA.

1. Depth. A minimum depth of 3.5 feet below final grade was used for all foundations to protect against frost damage.

2. Bearing pressures. Footings were sized for a maximum allowable soil bearing pressure of 1,400 pounds per square foot.

3. Earth pressures. For design of walls below final grade, a fully saturated earth pressure was used.

C. DESIGN LOADS.

1. Roof live load, 30 psf.

2. Floor live load

   a. Slab on grade, 150 psf

   b. Suspended slab, 100 psf

3. Wind load.

   a. American National Standards A58.1-72 and as amended

b. High loss potential facility

c. 100 year mean recurrence interval

d. Exposure "C"

e. Basic wind speed = 85 mph

4. Seismic, Zone 1, Z = 0.25 designed in accordance with TM 5-809-10.

D. FLOOR SLABS.

1. Slab on grade over 6-inch layer of capillary water barrier.

2. Structural floor, concrete slab.

E. MATERIALS.

1. Concrete.

a. Class AA, 4000 psi compressive strength at 28 days for concrete wet wells.

b. Class A, 3000 psi compressive strength at 28 days for all concrete not otherwise noted.

c. Reinforcement, ASTM A 615 or ASTM A 617. Ties and stirrups, Grade 40; all others Grade 60.

F. VIBRATION. The only mechanical equipment which will be installed on the structures are pumps and motors. Vibrations produced by this equipment will be readily absorbed by the concrete structure without any adverse effects. Isolation of the equipment from the structure is not required.

G. ALTERNATIVES. There are no structural systems competitive with reinforced concrete for the facilities included in this project.

CHAPTER IV

MECHANICAL

A. CRITERIA LISTING.

1. Publications.

Department of Defense Manual, DOD 4270.1-M, Construction Criteria Manual

Department of the Army, TM 5-810-1, Mechanical Design-Heating, Ventilating, and Air Conditioning

Department of the Army, TM 5-810-5, Plumbing

Department of the Army, TM 5-810-6, Mechanical Design - Gas Fitting

Department of the Army, TM 5-785, Engineering Weather Data

Project Development Brochure, Rocky Mountain Arsenal, Liquid Waste Disposal Facility, North Boundary expansion, Revised 31 July 1979

U.S. Army Engineer Waterways Experiment Station, Engineering and Construction Materials Compatibility Study

Minutes of review meetings of February 14 and 15, 1980, and February 28 and 29, 1980

B. DEWATERING AND RECHARGE SYSTEM.

1. Design Conditions.

a. Contaminated ground water flows northward to be intercepted by an impervious barrier. A total of 35 dewatering wells will remove contaminated ground water from the alluvial aquifer. Nineteen dewatering wells will remove contaminated water from the Denver Sands for treatment and reintroduction through 38 recharge wells. These recharge wells are located on the north side of the barrier. The dewater wells will be divided into three treatment areas corresponding to the areas in which three different combinations of contaminants are found. Dewatering flow rates range from 3.5 gpm to 23 gpm. Recharge rates range from 6 gpm to 29 gpm.

b. Six dewatering wells and twelve recharge wells presently exist and will be incorporated into the new system. The existing dewatering wells contain submersible pumps of 20 gpm capacity at 90 feet of head. Pump motors are 460 volt, 3-phase, 3/4 horsepower each. All existing wells will be modified to meet the design of the new wells.

2. <u>System Description</u>.

a. Dewater system.

(1) Each dewater wellhead will extend aboveground through a concrete slab. The concrete slab will be located atop an earth mound sized to place the wellhead above the flood plain.

(2) Dewater well pumps will be submersible, centrifugal type of materials shown to be suitable according to the U.S. Army Engineer Waterways Experiment Station, Engineering and Construction Materials Compatability Study. Motors will be 240 volt, single-phase. Pumps will be controlled by level sensing electrodes suspended in the well. The sensors will be set to turn the pumps on and off at water levels determined from the geotechnical analysis. The pump will be suspended in the well by a 1-1/4-inch diameter corrosion resistant steel pipe.

(3) Each pump will be equipped with a turbine type flow meter of corrosion resistant steel with a local readout of gpm flow, as well as a totalizer. Appropriate balancing valves and check valves made of polyvinyl chloride (PVC) will be placed in line with the meter and connected by 1-1/2 inch Schedule 80 PVC pipe. The meter and valves will be located above the concrete slab and, after the last valve, the PVC pipe will turn down through the slab to a distance of 5 feet below ground level (below maximum frost penetration). The valves, meter, and piping above the concrete slab will be protected from freezing by a thermostatically controlled, self limiting, pipe trace heating system. The valves, meter, pipe, and tracing will be covered by insulation tape.

(4) The concrete slab will be covered by a metal bulkhead door set on a concrete curb around the perimeter of the slab.

(5) The dewatering wells will be connected to the treatment influent wet wells by means of an underground collection header of Schedule 80 PVC plastic pipe. The three treatment sections will be isolated by gate valves placed in the header in such a way that the boundary of any section may be extended by addition of wells from an adjacent section. The PVC manifold pipe connecting a given treatment section header with the wet wells will lie in the same trench as the section dewater header. At the point where all three manifold lines meet, they will be run in the same trench to the wet wells.

(6) Well placement is determined from geotechnical analysis using data derived from pump tests and computerized modeling of the alluvial aquifer system. Pipe sizes are selected by balancing minimum friction loss against minimum pipe diameter (minimum cost).

The total head loss in the system is determined by use of the Water Distribution Analysis computer program. Input to the program includes pipe length, diameter, and roughness, input flow rates, and desired outflow pressure. Output from the program includes head loss and function pressures. The output from the program is included in the mechanical calculations. The head loss output from the computer program is used to select pump horsepower from manufacturers' catalogs.

b. The recharge system will consist of an underground Schedule 80 PVC plastic pipe connecting the organic treatment effluent wet well to the recharge header to which each well is likewise connected.

Each recharge wellhead will be housed in the same manner as the dewater wells.

Each well will be equipped with a totalizing flow meter, appropriate balancing valves, a pressure relief valve (regulator), and a shutoff valve, which is activated by an electrode probe in the well. Should the ground surrounding the well become incapable of accepting recharge water, the shutoff valve will divert excess flow to other wells. In the event that all of the wells are unable to accept recharge water, a pressure relief valve attached to the recharge header is activated and the recharge water is discharged into the ditch along the perimeter road. The pressure relief valve is enclosed in an underground vault, accessible by a locking manhole cover. The overflow valve pit also contains appropriate gate valves and a cumulative watermeter.

CHAPTER V

ELECTRICAL

A. GENERAL. This design is based on, but not limited to, the applicable publications, codes, and specification listed in the introduction to this narrative.

B. SCOPE. This design will generally consist of the following details:

1. Interior.
   - a. Lighting and receptacles
   - b. Service entrance
   - c. Motor control center
   - d. Dewater well control
   - e. Recharge well service
2. Exterior.
   - a. Primary service
   - b. Transformers
   - c. Overhead distribution
   - d. Well control cable

C. INTERIOR.

1. Lighting, 175-watt mercury vapor fixtures, will be provided and will match all existing fixtures. Switches shall be installed at the doors. Voltage will be 120 volts.

2. Receptacles will be provided in two locations and will be 20 amperes, 120 volt, duplex type.

3. Conduit system will be rigid aluminum or zinc-coated steel.

4. Conductors will be copper or aluminum with insulation conforming to the NEC. Conductors will be installed in dry locations, damp locations, underground and submersible locations. Conductors for the well controls will be underground telephone type.

5. Service entrance to the building will be relocated in the new building extension. The service will be underground into the new motor control center. The existing MCC will be served from the new MCC-1. Service will be 480 volt, 3-phase. Service entrance to each dewater well will be underground 240/120 volt, single-phase. A breaker will be provided at each well.

6. Motor control center will be located in the new building extension. The MCC will serve the existing MCC, five new pumps, and the recharge wells. Future provisions will be made for adding one additional section to serve five pumps and additional loads. The MCC will contain starters for the new pumps. A load of 50 kVA was used for the existing MCC according to instructions received from Rocky Mountain Arsenal.

7. Existing panels, 240/120 volt, will be used to serve the new 120 volt loads.

8. Motors at the building will be vertical type, totally enclosed, located outside at the wet wells. Motors at the wells will be submersible.

9. Reduced voltage starters will be provided on the two 50 horsepower motors at the building. Type of starters will be at the option of the Contractor.

10. Well control panel will be located in the building for remote control of the dewater wells. The panel will have an OFF-ON switch and a

red light for each well. The panel will turn off the wells when a high water level is reached in the appropriate wet well. The panel will have a local bell and light for visual and audio notification. A remote alarm system will transmit a signal over telephone lines (provided by others) to the fire station. The fire station will have an alarm box which will sound the local alarm.

11. Dewater well control will be at each well. The pump will be controlled by a local H-O-A switch. In the AUTO position, a level probe contact and the remote OFF-ON switch contact will cycle the pump. The remote OFF-ON switch and red light will be operated by induction type well control relays which use telephone type wire. One pair of telephone wires will be routed underground from each well to the well control panel in the building. A heat tape will be provided in each well for freeze protection.

12. Recharge well will be provided with 480 volt, single-phase underground service. A 480/120 volt transformer will serve a level controlled solenoid and heat tape at each well. A breaker will serve as a disconnect and protection device.

13. Existing dewater wells will be revised to raise the wellhead above grade. Electrical service will be changed from 480 volt, 3-phase to 240 volt, single-phase in order to standardize all pumps. All electrical equipment will be removed and reinstalled as needed.

D. EXTERIOR

1. Primary service to the existing building is 13.2 kV, 3-phase, 4-wire. The transformers are three 25 kVA, single-phase, 13.2 kV-480 volt. The utility company is presently providing secondary power service at

480/277 volt. The utility company will remove their equipment and provide primary power with metering at 13.2 kV. The new service to the wells will be 13.2 kV, single-phase, line-to-line. Construction will be suitable for future expansion to 3-phase. A new service will be provided for the new building at 13.2 kV, 3-phase to the new transformers.

2. Transformers will be provided to serve the new building. Three 50 kVA pole-mounted single-phase transformers will provide 13.2 kV-480/277 volt, Delta-Wye service. Transformers for the service to the well will be single-phase, 13.2 kV-240/120 volt pole-mounted. Sizes vary from 15 to 37-1/2 kVA. All service to the building and wells will be underground.

3. Aerial conductors for the primary line will be based on ASCR, size No. 2. Secondary conductors will be No. 2 aluminum, messenger supported duplex and No. 2 copper underground.

4. Fused cutouts and lighting arrestors will be provided at each transformer.

5. Telephone type cable will be routed underground to each of the dewater wells for control. The cable will be routed in the trench with dewater piping.

6. Existing 480 volt overhead lines will be modified to conform to the new distribution.

7. Existing light poles at the building will be relocated because of a new wet well.

CHAPTER VI

GROUND WATER CONTAINMENT ANALYSIS

A. INTRODUCTION

1. General Ground Water Conditions.

a. The Rocky Mountain Arsenal has received a Cease and Desist Order from the State of Colorado to prevent further migration of contaminated water across its North Boundary. Contaminants related to liquid wastes resulting from the past manufacture of chemical warfare agents and recent manufacture of herbicides, pesticides, and fungicides have reached the saturated zone and are being transported by ground water. Ground water flow is generally to the north and northwest toward the South Fork Platte River.

b. Ground water is generally transmitted by porous media under both unconfined and confined conditions with intermediate degrees of confinement. Aquifers consist of shallow unconsolidated sands and gravels overlying relatively dense sands of the Denver Formation that are interbedded with shales and siltstones. The Denver Sands consist of irregular lenticular shaped beds confined by more extensive siltstones and shales. The shallow unconsolidated aquifer consists of channelized alluvial deposits that are absent or unsaturated in some areas. Ground water is generally unconfined in the alluvial aquifer, but locally clayey or silty saturated soils result in semiconfined conditions. The alluvium is generally much more permeable than the underlying Denver Sands.

c. At the North Boundary, the alluvial aquifer is at least 100 times (2 orders of magnitude) more permeable than the Denver Sands within the upper 100 feet or more. Consequently the alluvium transmits the vast majority of the flow across the North Boundary.

d. Flow paths in the alluvial aquifer converge at the North Boundary. The western flow path passes beneath the eastern part of Basin F and trends northeasterly to the North Boundary. The eastern flow path has a northerly trend along First Creek. After these flow paths cross the boundary, they split into north and northwesterly trends.

e. Contaminants are concentrated in irregular plumes primarily in the western flow path. Most of the contaminants are concentrated in the alluvial aquifer with lesser contaminant levels detected in the Denver Sands. Leakage from Basin F is one source of contaminants, however, other basins, pipelines, and the sewage lagoon are additional suspected sources.

2. <u>Containment Design Concept</u>.

a. A pilot containment facility consisting of a slurry trench cutoff wall, dewatering wells, treatment plant, and recharge wells has been in successful operation since the summer of 1978. This facility will be expanded by extending the cutoff wall 3,840 feet east and 1,400 feet to the west. Additional dewatering wells will be provided to intercept all of the flow in the alluvial aquifer and suspected or possible contaminated flows in the upper Denver Sands. Treatment capacity will be expanded and additional recharge wells provided to re-inject treated water to essentially restore the natural flow system.

b. The original concept for dewatering the alluvial aquifer was to optimize (minimize) the number of dewatering wells required to intercept flows and to manifold all of this water together for treatment (together with existing wells). With this concept, the new alluvial dewatering wells would have been concentrated in areas near First Creek where the saturated thickness and the well capacity is the greatest. It is estimated that 8 to 10 wells in addition to the existing b would have been required. The exact number, location, and spacing of wells for this concept were not determined because in February 1980 redirection was received from Corps of Engineers (COE) to proceed with the dewatering concept described below. This redirection required a different approach with more rigorous analyses to define the hydraulics of the flow system and to estimate contaminant fluxes through each flow segment. The concept adapted by RMA and COE requires detailed quantification of flow and contaminant fluxes for each segment of the alluvial aquifer, so that three zones of flow can be intercepted and manifolded to separate treatment modules. Dewatering wells have to be distributed across the entire flow system to minimize dispersion of contaminants by gradient changes. This redirected concept required development of a digital finite-difference flow model to define the distribution of flows through the system with the degree of accuracy required to make this concept technically feasible.

c. Alluvial aquifer dewatering wells up gradient from the cutoff wall are to selectively intercept 3 zones of contamination by manifolding groups of wells across the barrier thus permitting separate treatment of these waters. The dewatering rate will be as close to the natural flow rate as possible. The dewatering rate will have to slightly exceed the natural flow rate, at least during initial years of operation, to prevent excessive rise in water levels and flooding over the cutoff wall in low lying areas.

d. The cutoff wall extensions will be constructed by excavating bentonite slurry trenches which will be backfilled with select material mixed with bentonite clay to form a hydraulic barrier through the alluvium and into the Denver Formation. The cutoff wall extensions will penetrate shallow Denver Sands that have or are close to having hydraulic connection with the alluvial aquifer at the barrier. Additionally, the cutoff wall will penetrate fractured shales to provide protection against fracture flow through the underlying shales. Field testing and analysis indicates fracture dominated flow is not significant and the cutoff will be deeper than is probably necessary.

e. The existing slurry cutoff wall will be left undisturbed because analyses indicate that the existing barrier is quite adequate. There is a shallow and rather extensive Denver Sand layer beneath the existing barrier that contains low levels of contaminants. Flow through this sand layer will be intercepted by Denver Sand dewatering wells, although the flow through this sand layer is only about 0.75 gpm under existing gradients and available analyses indicate this water meets standards for DIMP, DCPD, DBCP and Fluorides. Concern has been expressed about flow through fractures in shales between the base of the existing barrier and the underlying Denver Sand. Computations indicate this flow, if not intercepted by the Denver Sands dewatering wells, would amount to only 0.06 gpm which is insignificant. Even if Denver Sand dewatering wells are not constructed, flow beneath the existing cutoff wall would be and is insignificant, totaling only 0.81 gpm under natural gradients. Therefore, it is ESA's recommendation that the existing pilot cutoff wall be left undisturbed (it should not be deepened) and that Denver Sand dewatering wells be used to monitor the quality of flow and dewater the shallow Denver Sand on an as-needed basis.

f. Denver Sands dewatering wells will be constructed to intercept suspected or possible contaminated flows beneath the cutoff wall in the Denver Sands to depths of up to 105 feet. A pumping depression will be developed to contain and collect these flows. Contaminant levels are expected to generally meet water quality standards, and the wells will be monitored closely and pumped on an as-needed basis.

g. Recharge wells constructed downgradient from the cutoff wall will re-inject the treated water. Recharge will be distributed across the flow system so that natural flows are maintained within the constraints of barrier operation. It is estimated that about 110 percent of the natural alluvial flow will be recharged because of the overpumping requirement for operation of dewatering wells, at least during the initial years of operation. Pumpage from the Denver Sands will also be recharged into the alluvial aquifer. This amount is expected to be insignificant in comparison to alluvial flows.

3. Methodology

a. Existing data were collected and analyzed. Data stored on magnetic tapes were screened and coded for retrieval in a usable form. Preliminary geologic sections were constructed, water levels and chemical data were contoured, and time concentration graphs were constructed. Existing pump test data were re-interpreted for hydraulic parameters.

b. A field exploration program was planned and performed to provide more detailed geologic, geohydrologic, and chemical data. Along the cutoff wall alinement, 30 test holes were drilled of which 5 were converted to monitoring wells by installing casings and screens. The alluvium was sampled with a split-spoon and the Denver Formation was cored. Laboratory tests were run on soil samples and cores. Four test wells, each with two to three observation wells, were constructed in the alluvial aquifer (2) and the Denver Sands (2). Pump tests were performed and the data were interpreted to determine aquifer characteristics.

c. A finite difference model was developed of the North Boundary area alluvial flow system to simulate flow conditions and to support the design of dewatering and recharge wells. This effort was not planned in the original scope of work. However, direction was received to evaluate selective interception of contaminant zones and to proceed with design on that basis. This required a more rigorous analysis of flows across the boundary that could best be simulated and analyzed with finite difference techniques. This model enabled us to distribute flows across the boundary within the limits of precision of the hydraulic conductivity data and the water level contours used for calibration of the model. The model was then used to distribute dewatering and recharge rates for wells and simulate the hydraulic effects on the alluvial flow system.

d. Contaminant fluxes were estimated for each dewatering well based on hydraulic effects simulated by the model and evaluation of contaminant plumes. Also upper limit fluxes were estimated for each dewatering well based on the highest concentrations upgradient from the barrier system. Dispersion and sorptive effects were ignored in these estimates, resulting in conservative values, especially for upper limit estimates.

e. Pump tests of two test wells in Denver Sands were used to design dewatering wells. These wells will develop a pumping trough to intercept suspected contaminants. Considerable judgment along with distance drawdown calculations had to be used to design well spacing and pumping rates because of the irregular configuration and location of sand lenses.

f. The slurry cutoff wall extensions were designed as geologic and soils data became available, independent of geohydrologic and chemical studies. Specifications were prepared based on existing data, and backfill requirements were evaluated after gradation tests of soils were completed. Excavation requirements were incorporated into design drawings as they became available.

g. Specifications and design drawings were prepared for alluvial aquifer dewatering and recharge wells, Denver Sands dewatering wells, and for monitoring wells.

h. Monitoring wells for the alluvial aquifer and the Denver Sands were located. Existing wells were incorporated as much as possible into the monitoring system. In many cases only general designs can be provided because of lack of subsurface information.

B. HYDRAULIC ANALYSIS - ALLUVIAL AQUIFER.

1. Criteria

a. Alluvial aquifer system flows across the North Boundary must be determined to estimate dewatering and recharge rates and to estimate contaminant fluxes. The distribution of system flows must be estimated with reasonable accuracy so that dewatering and recharge rates can be distributed with a minimum amount of disturbance to the natural flow system.

b. There are no criteria for selecting the number of dewatering wells. An upper practical limit might be 440 wells each designed to pump 1.1 gpm. This would stress the alluvial flow system the least, resulting in less dispersion of contaminants. A lower limit might be 16 wells, which would result in larger local stresses and mixing of contaminant plumes. A total of 35 dewatering wells were selected (including the 6 existing wells) as a compromise based largely on judgment and experience with the pilot facility.

c. Dewatering rates and distribution of pumping rates must be sufficient to prevent flooding over the top of the cutoff wall.

d. Recharge wells must be spaced so that recharge rates re-establish the alluvial flow system off post. Also, each well must be capable of receiving the distributed rate for each location without surface flooding.

e. Design of the North Boundary barrier system requires a detailed knowledge of flow through the alluvial aquifer in the project vicinity to meet criteria for interception of selected flow components. A digital simulation model of the geohydrologic system provides the best tool for defining total system flows as well as flows through any selected zone. It also provides a means of evaluating aquifer hydraulic responses to pumping and recharge wells as well as breakdown scenarios.

2. Hydraulic Model Development.

a. Simulation of the geohydrologic system in the vicinity of the North Boundary was accomplished by construction of a digital model as proposed by Trescott, Pinder and Larson (USGS, 1976). This finite difference model simulates the aquifer's response to stresses in two dimensions and enables accurate representation of complex boundary conditions and system heterogeneities by approximating the partial differential equation governing ground water flow with finite differences for the derivatives at numerous distinct nodes representing the aquifer. The resulting system of algebraic equations (one for each node in the system) is solved using a highly efficient technique known as the "strongly implicit procedure". For the North Boundary model, the finite-difference grid contains 2,958 cells (29 rows by 102 columns) as shown on Figure VI-1. Each cell has a node at its center. The cells are 100 feet by 100 feet near the slurry cutoff wall and are up to 100 feet by 500 feet to the north and to the south. Given a distinct system geometry, aquifer characteristics, boundary conditions, and initial water levels, the model solves for the average hydraulic heads at each node.

FINITE DIFFERENCE GRID

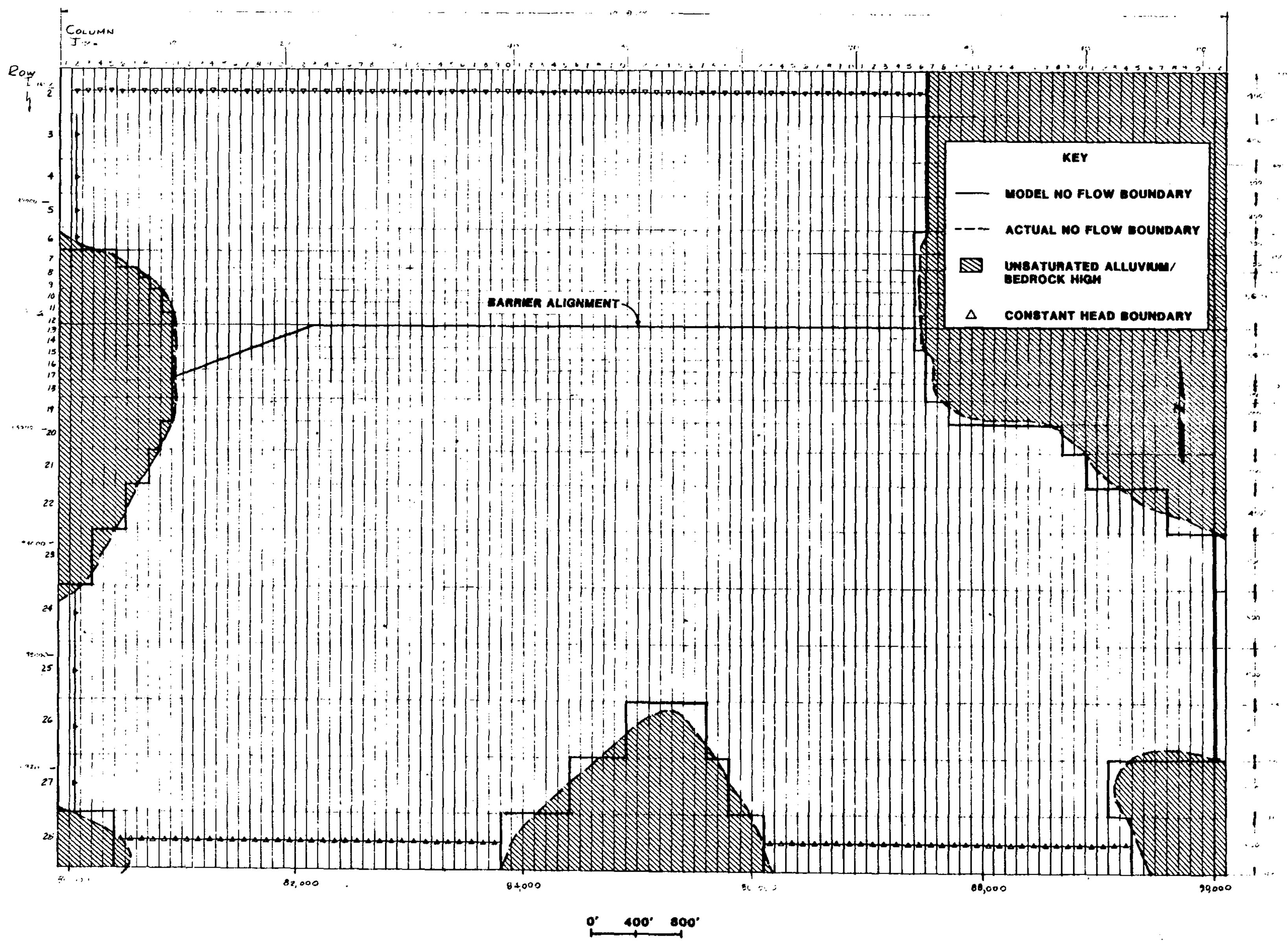


Figure VI-1

b. Boundary conditions modeled consist of no-flow boundaries and constant head boundaries. No-flow boundaries are represented by specifying a permeability of zero at the nodes outside the boundary. The harmonic mean of the permeability at the cell boundary is zero, and as a result there is no flow across the boundary. A boundary condition of this type was used where alluvium is absent or unsaturated and along the small basin to the southeast. The bedrock high areas are believed to be much less permeable than the alluvial aquifer, and their treatment as no-flow areas is therefore justified. Constant head boundaries were assumed where no physical boundaries existed. Along these boundaries, heads were fixed at "steady state" values which were based upon best available water level data. These fixed head boundaries will not influence model results when hydraulic stresses are located far from these boundaries and the simulation period is short.

c. The finite-difference model assumes the aquifer may be represented as a two dimensional, isotropic, heterogeneous unconfined system with a nonleaky underlying layer. Within the model area, recharge from precipitation is negligible and evapotranspiration is assumed to be negligible. There are evapotranspiration losses in the bog area mainly down-gradient from the barrier, but the losses are probably less than 10 percent of the alluvial aquifer flow.

d. Computation of in-well hydraulic heads at the pumping and recharge wells was accomplished by employing a form of the Thiem equation. This was necessary for extrapolating from the average hydraulic head for each cell to the head at the effective well radius (8 inches for pumping wells and 1 foot for recharge wells). This approximation is based on the following assumptions: (1) flow takes place within a square well block (grid cell in three dimensions) and can be described by a steady state equation with no external sources; (2) the aquifer is isotropic and homogeneous within the well block; (3) only one well is in the well block and it is fully penetrating; (4) flow is laminar; and (5) well loss is negligible. For design purposes, model produced drawdowns were increased by 10 percent to account for well friction losses.

e. Calibration of the finite-difference model consists of distributing permeabilities throughout the nodal system so that model simulated water levels match observed water levels that are reasonably near a steady state. This is necessary because inflows and outflows to the system are unknown but are assumed to be equal because recharge and evapotranspiration within the modeled area are negligible. The finite-difference model requires that an average hydraulic conductivity, specific yield, bedrock elevation, and water level be specified at each node. The saturated thickness of the alluvial aquifer was determined by the elevation difference between water level contours shown on Figure VI-2 and bedrock contours shown on Figure VI-3. Saturated thickness is shown on Figure VI-4. Water level contours used are based on spring 1979 water level measurements. These water levels were compared with other historic water level measurements and were judged to be a reasonably good representation of steady state conditions. Hydraulic conductivities and specific yields are based on eight pump tests performed by WES and two new pump tests performed by ESA in 1980. A summary of these latter two pump tests is

GROUNDWATER ELEVATIONS

SPRING 1979

NORTH BOUNDARY AREA - RMA

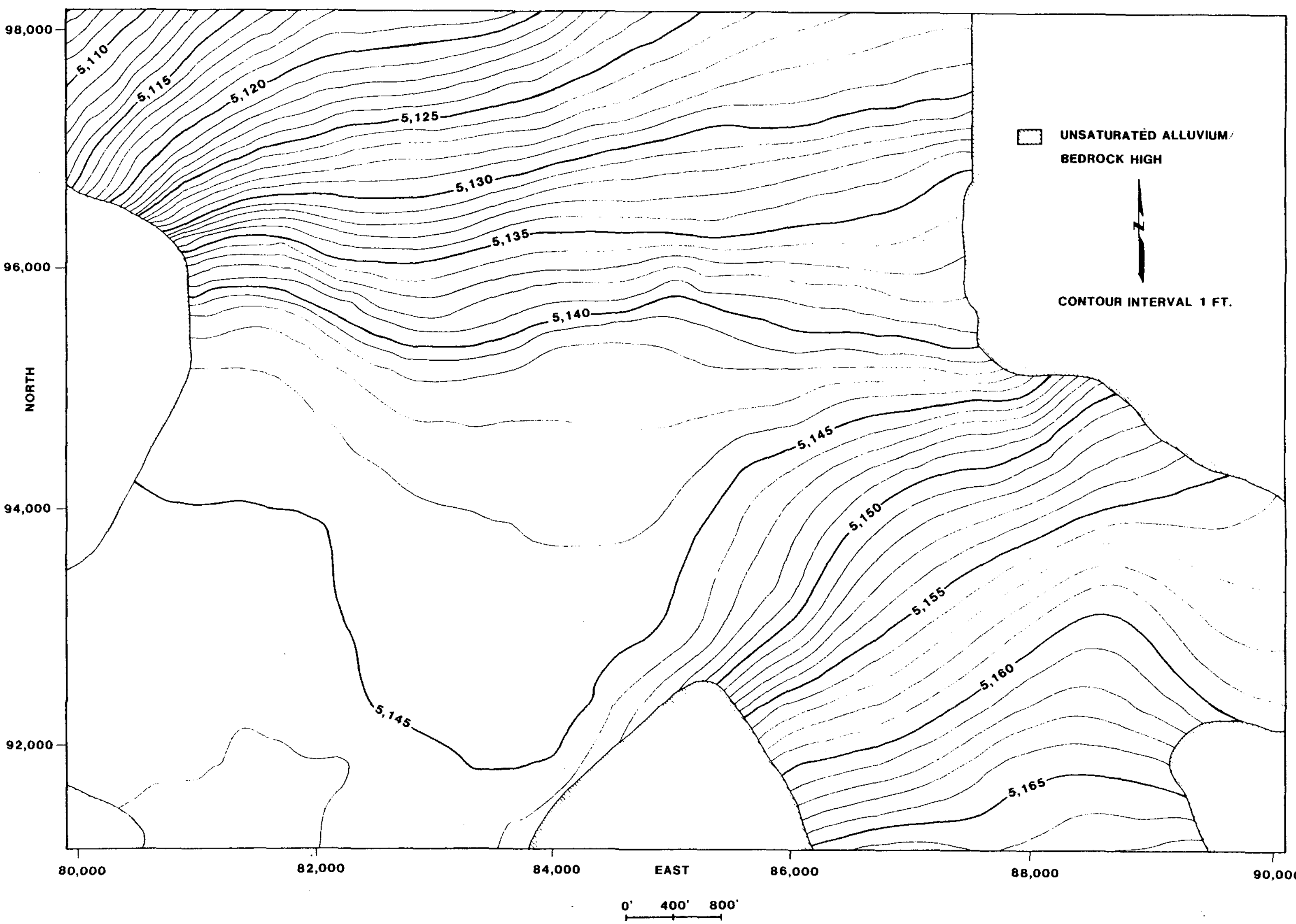


Figure VI—2

BEDROCK CONTOUR MAP

NORTH BOUNDARY AREA - RMA

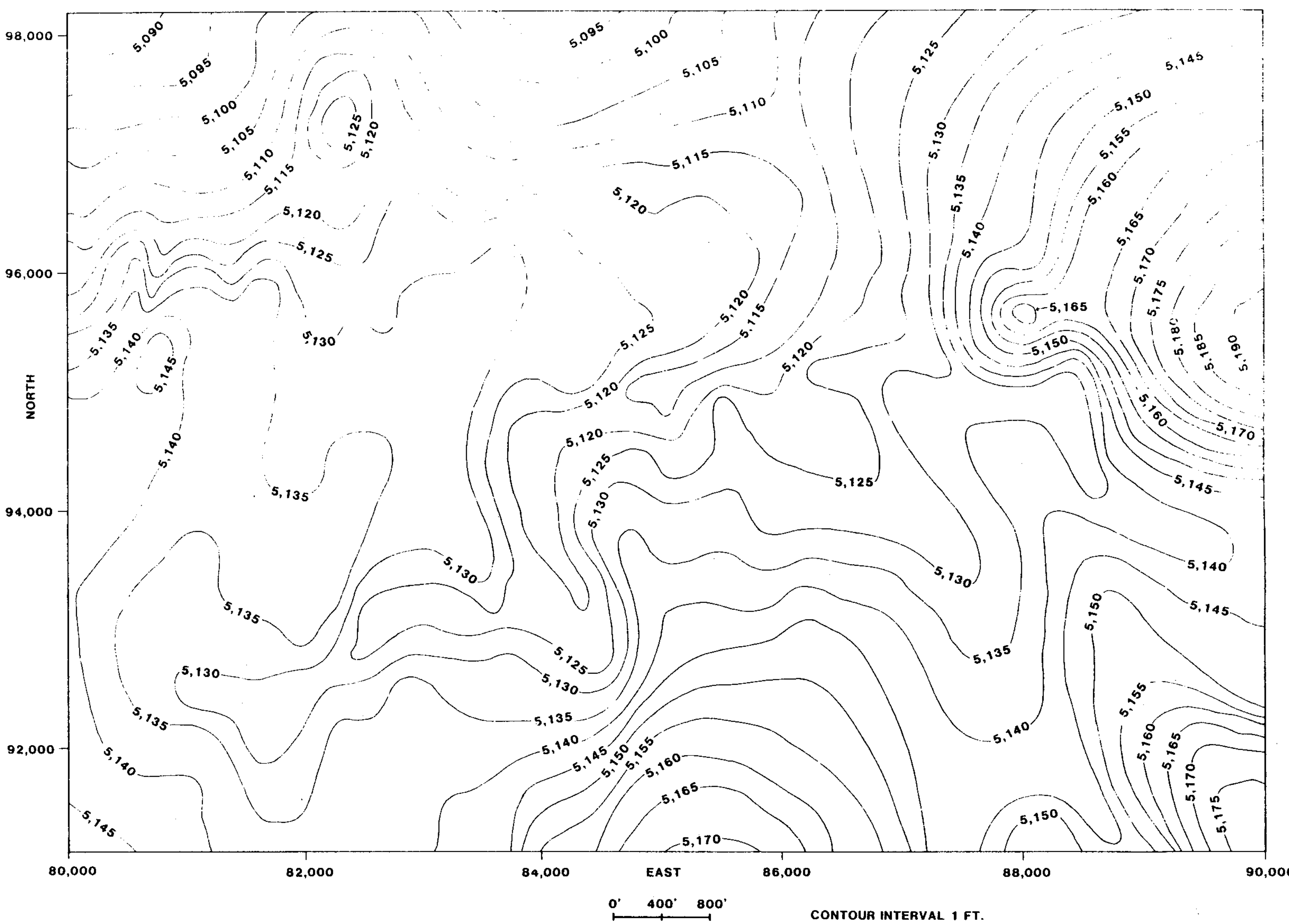


Figure VI-3

SATURATED THICKNESS OF ALLUVIAL AQUIFER

NORTH BOUNDARY AREA - RMA

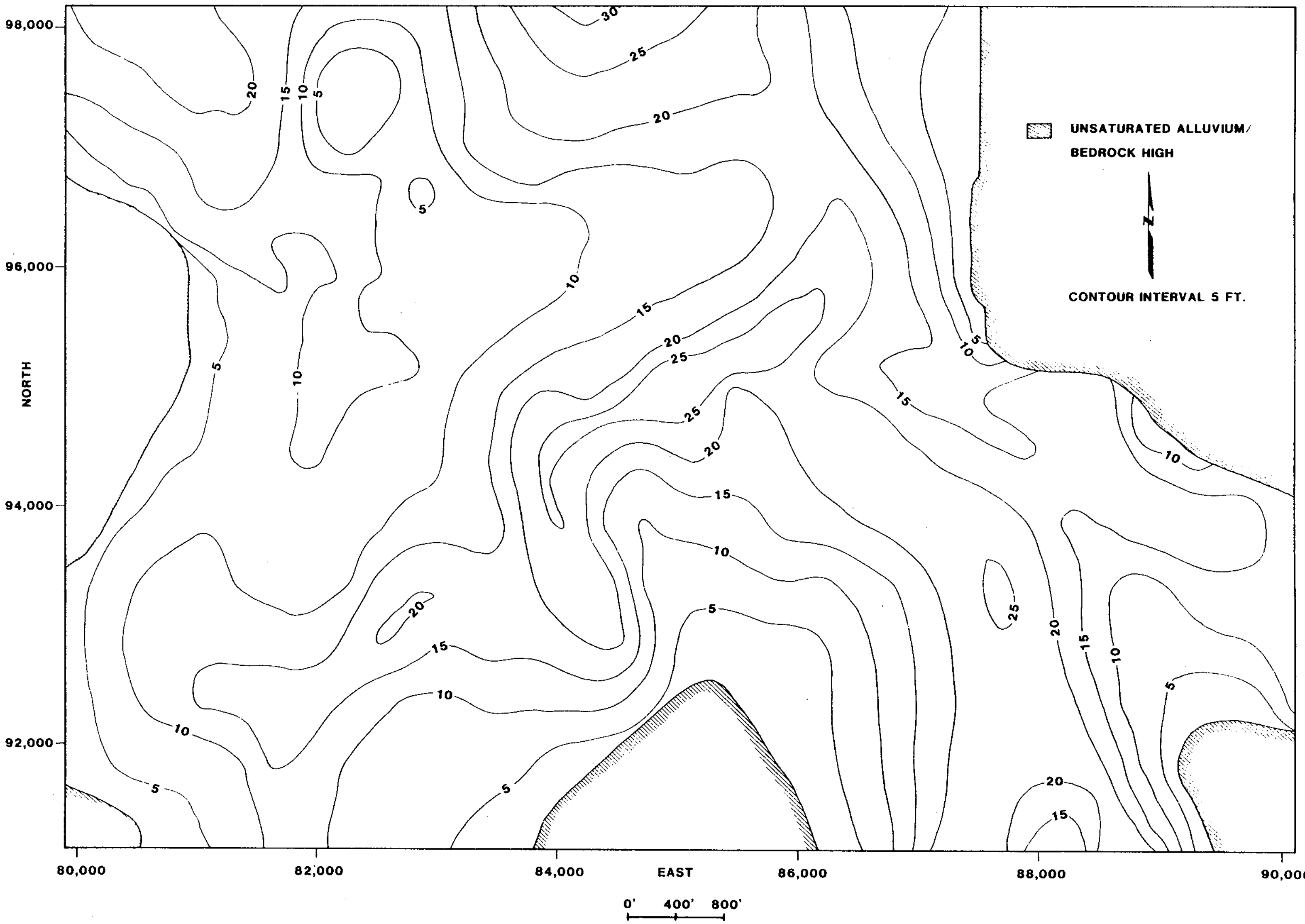


Figure VI-4

shown on Table VI-1 and calculation sheets and field data plots are appended. Calculated specific yields ranged from 0.35 to 0.01 and a vertically averaged value of 0.1 was used to best represent conditions near the dewatering and recharge wells. Specific yield is not an important factor in calibration of the model because it is not a function of steady state head distribution. As a result, calibration of the model is dependent on the hydraulic conductivities assigned each node and the accuracy of the modeled flows is, therefore, dependent on the validity of hydraulic conductivities determined from pump test data. The modeling technique forces fluxes throughout the system to balance so that hydraulic conductivities are correct relative to cells where pump test data were obtained when calibrated to observed steady state water levels.

f. The finite-difference model was calibrated using the inverse method. This method included the following steps:

(1) Development of steady state water levels which was accomplished by contouring the best available water level data for 202 observation wells distributed throughout most of the system. Resulting water level contours are shown on Figure VI-2.

(2) Trial values of hydraulic conductivity were estimated based on pump tests between Basin F and the North Boundary. All available pump test data were analyzed using the unconfined type curves of Neuman (1975) which are appended with calculation sheets. Initial hydraulic conductivities for the model area were established using a model calibration procedure proposed by Hunt and Wilson (1974), and Day and Hunt (1977).

TABLE VI-1

SUMMARY

of

PUMP TEST RESULTS

in

ALLUVIUM, ROCKY MOUNTAIN ARSENAL

Test No.	Obs. Well	T (gpd/ft)	ft^2/d	S_y
1032-1 (24150)	1031 (24149)	19,864	2,656	0.003
	1030 (24148)	23,837	3,187	0.14
1032-2 (24150)	1030 (24148)	20,342	2,719	0.02
	1031 (24149)	20,342	2,719	0.0027
Approx. Average for	1032 (24150)	21,096	2,820	0.0414
1036-2 (24153)	1033 (24151)	18,794	2,513	0.01

NOTES:

Average k in vicinity of well 1032 = $\frac{21,096}{17}$ = 1,241 gpd/ft^2 = 60,558 ft/yr

Average k in vicinity of well 1036 = $\frac{18,794}{11}$ = 1,709 gpd/ft^2 = 83,377 ft/yr

T = transmissibility

S_y = specific yield

k = horizontal hydraulic conductivity of alluvium

(3) Using the above data, the analysis was then carried out until a steady state condition was reached.

(4) A comparison of the observed and calculated water levels was then made and the hydraulic conductivities adjusted until a suitable match was obtained between the observed steady state water levels established in (1) and those obtained from the model using adjusted hydraulic conductivities. Resulting model hydraulic conductivities are represented by transmissivity contours shown on Figure VI-5 (transmissivity = hydraulic conductivity times the saturated thickness).

g. After calibration, 98.4 percent of the active nodes were within 2 feet of observed water levels, 96.7 percent were within 1.5 feet and 93.4 percent were within 1 foot. Considering the local seasonal variation and the scarcity of data in some areas, this calibration was judged to be adequate for design purposes. The simulated steady state ground water levels are shown on Figure VI-6.

3. Simulation and Analysis.

a. The natural flow through the system was computed by the model to be 440 gpm. Once the barrier is in place this flow must be captured by the dewatering wells. More water will have to be pumped and recharged than 440 gpm because of several factors.

(1) In the long term, pumping will lower ground water levels in the proximity of the dewatering wells and will induce more flow through the system because of the steeper gradients induced by well drawdowns.

(2) At the initiation of pumping, the influence of each well is small and flow will bypass the pumping wells causing a rise in ground water

TRANSMISSIVITY CONTOURS

NORTH BOUNDARY AREA - RMA

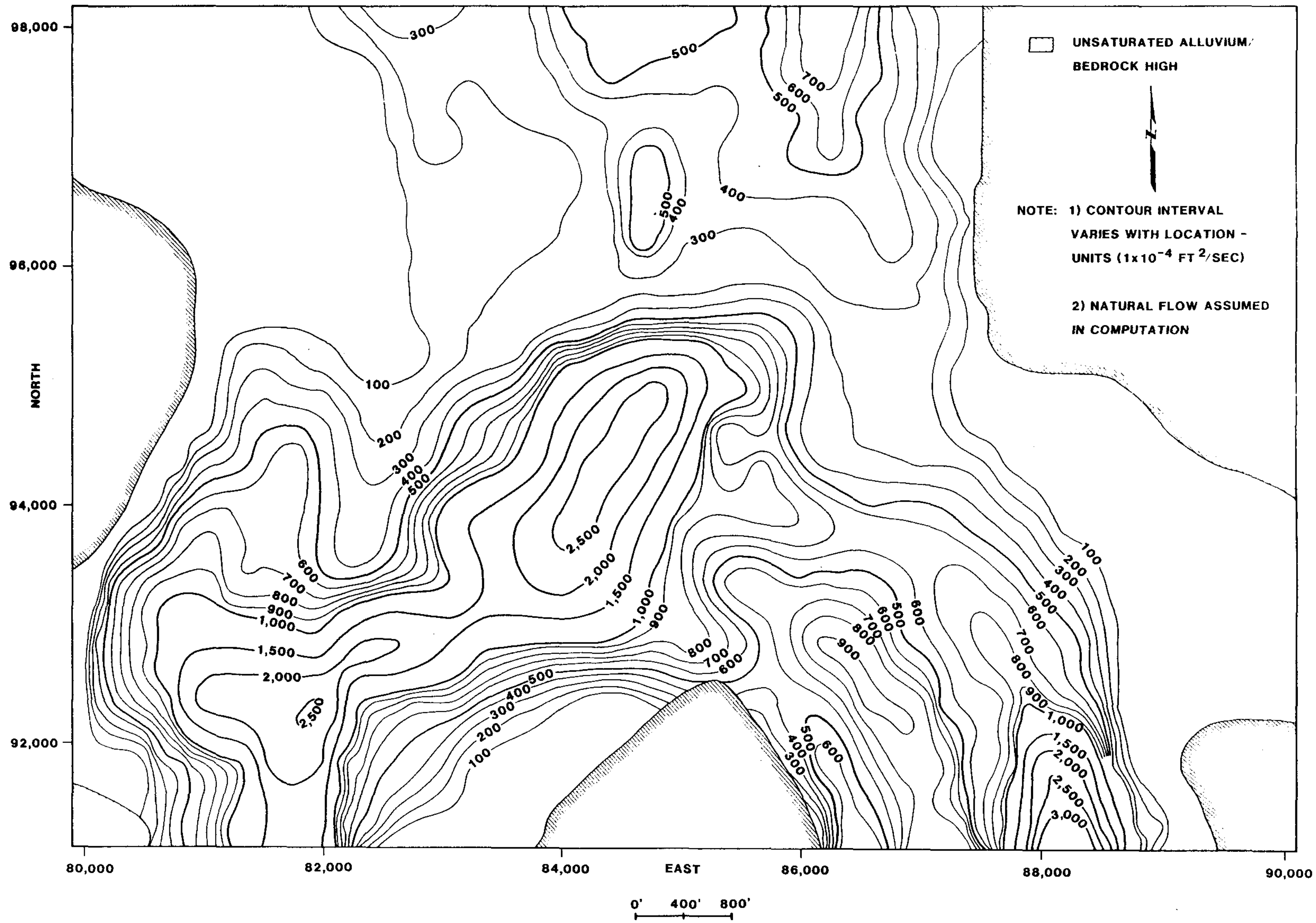


Figure VI-5

GROUNDWATER ELEVATIONS

SIMULATED STEADY STATE

NORTH BOUNDARY AREA - RMA

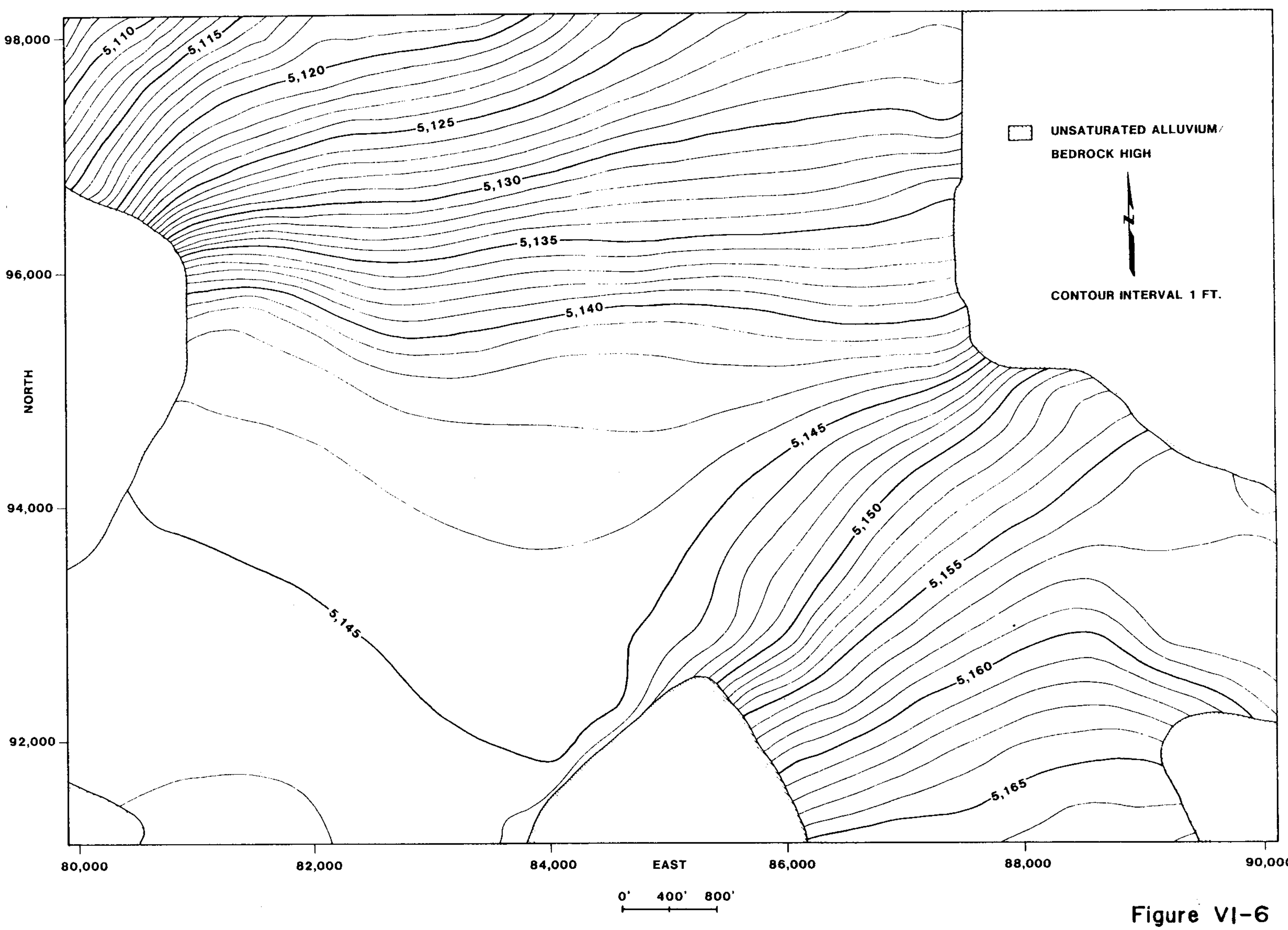


Figure VI-6

levels near the barrier. If the wells are extracting flow equal to the natural flow rate, some water would come from storage upstream of the dewatering wells. On the downstream side of the dewatering wells (near the cutoff wall), flowing water would accumulate. To prevent this rise in ground water levels during early time, pumping must capture the natural flow plus water taken from storage.

(3) It is desirable to lower water levels in the aquifer between the pump wells and the cutoff wall because in the event of failure of dewatering wells, the dewatered zone serves as a storage buffer against flooding. To create this ground water storage buffer pumping must exceed natural flow rates.

(4) While system flows were computed as precisely as possible, both the total system flow rate and local flow rates can be in error. As a precaution against flooding, a safety factor is included in design pumping rates.

d. Design dewatering and recharges rates were based on natural flows plus 10 percent. Natural flows were calculated for 100 foot segments (each cell) along the barrier. The water was distributed to each dewatering and recharge well based on its likely zone of influence. The 10 percent additional pumping was found to be sufficient to prevent significant flooding based on the simulation model. It should be noted that the design pumping rates are a best estimate based on interpretation of pump test data. Since these values may have to be adjusted during operation, each pumping well is designed to have a pumping range of ± 50 percent of its design value. This design flexibility also will allow for an increase in individual pumping rates to compensate for individual well shutdowns for maintenance or failure.

c. Figure VI-7 shows the simulated steady state ground water surface resulting from the pumping and recharge system. Steady state conditions should be reached in approximately 4-1/2 years assuming flow into the modeled area remains reasonably constant. 2

d. Figure VI-8 shows ground water profiles at various simulated times, located 50 feet south of the cutoff wall. This figure shows the sequence of water level changes that will occur during the first few years of operation. These stages are summarized below.

(1) Natural flow conditions are assumed to exist when the cutoff wall is first constructed. Instantaneous construction is assumed for modelling purposes.

(2) Pumping begins and water levels rise near the cutoff wall because of the limited influence of the pumping wells. Water is removed from storage upstream of the pumping wells and some of the system flow avoids capture by flowing between the wells. This stage should be carefully monitored. Water levels are expected to rise, but not high enough to cause flooding. If water levels rise higher or more rapidly than indicated in the profiles then pumping must be increased or flooding will occur. There is a lag time between initiation of pumping, significant water level changes, and water level control near the barrier. Therefore, the monitoring of water levels at the cutoff wall will be a key indicator of the need for pump changes.

STEADY STATE GROUNDWATER ELEVATIONS

SIMULATION OF DEWATERING AND RECHARGE WELL OPERATION

NORTH BOUNDARY AREA - RMA

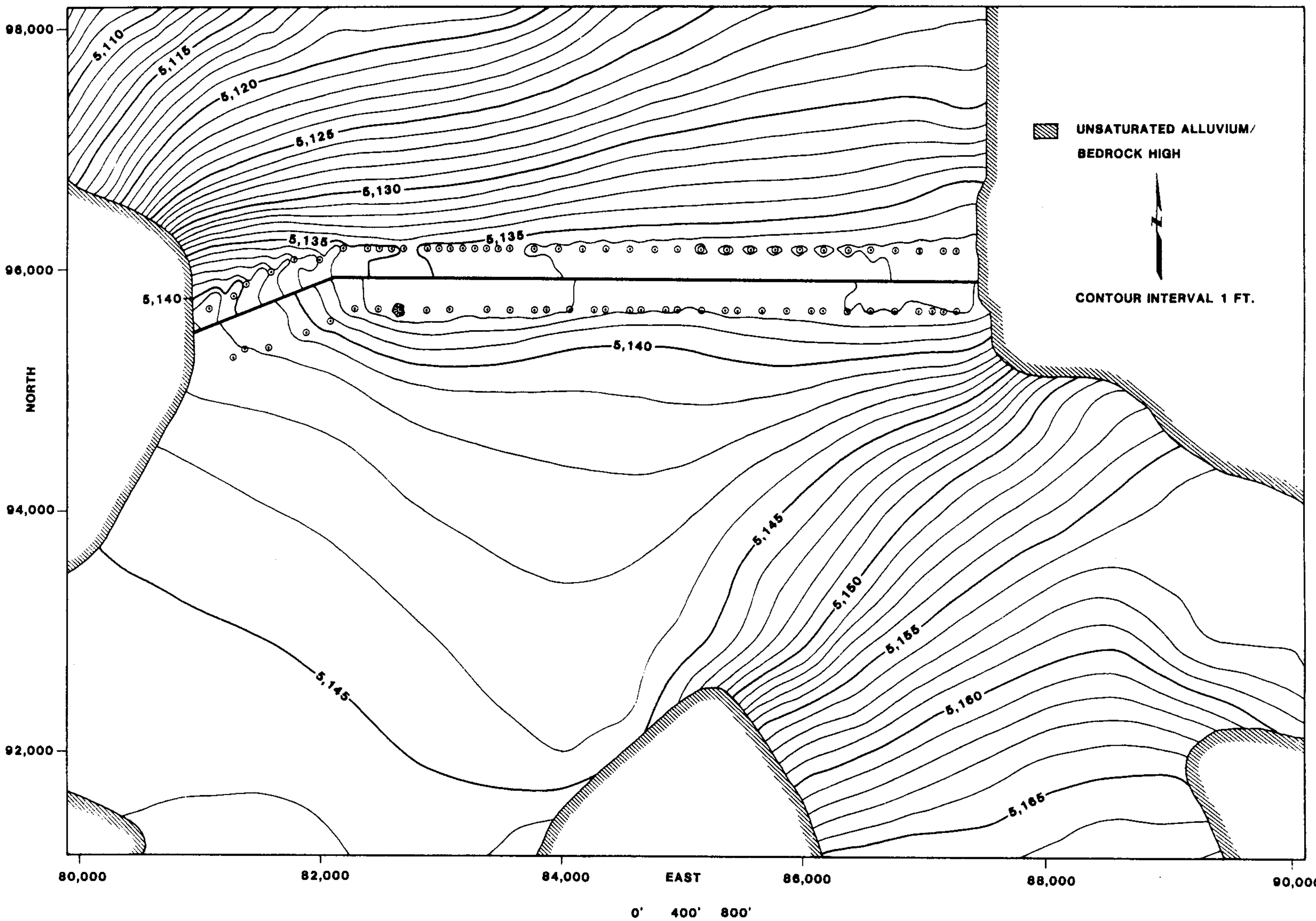
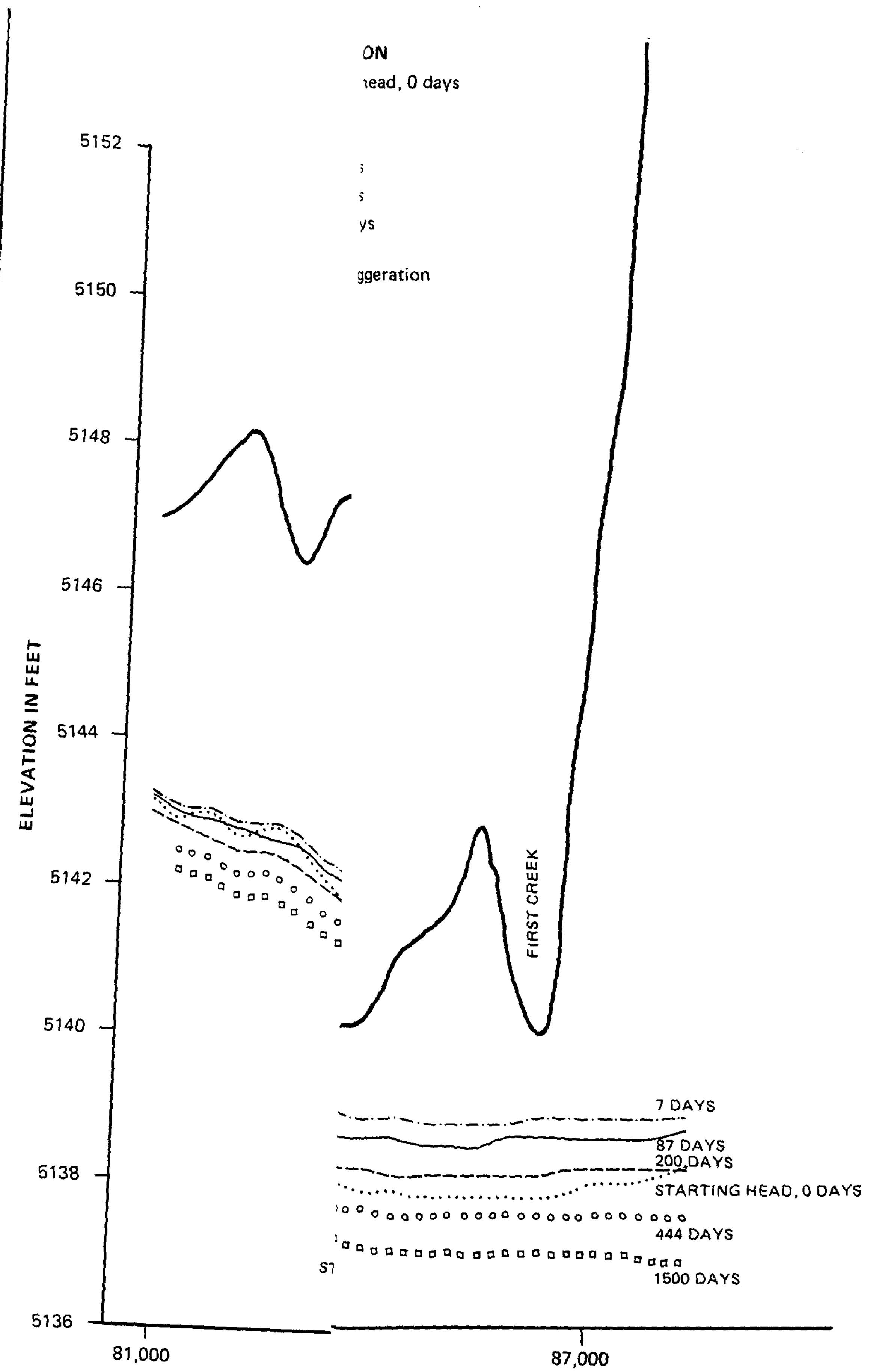


Figure VI-7



ON
ıead, 0 days
ys
ggeration
ELEVATION IN FEET
5152
5150
5148
5146
5144
5142
5140
5138
5136
FIRST CREEK
7 DAYS
87 DAYS
200 DAYS
STARTING HEAD, 0 DAYS
444 DAYS
1500 DAYS
81,000
87,000

Figure VI–8

(3) Water levels have peaked and now decline. Water coming from storage upstream of the pumping wells has diminished. The zone between the wells and barrier begins to dewater creating a storage buffer. Water levels (50 feet south of the cutoff wall) fall below presystem levels after about one year.

(4) Water levels have stabilized and minimal adjustments to the system are required.

e. The recharge system is coupled to the discharge system in that total flow rates must be the same. Design recharge rates have been established. If these rates are altered during operation, care should be taken not to over inject in the flood susceptable zone toward the east, particularly near the existing bog and First Creek.

f. Simulations studies were conducted to view the impact of a total pump system failure once the cutoff wall is in place. Three scenarios were considered.

(1) Total pumping system failure once barrier is in place before pumping ever begins.

(2) Total pumping system failure after 200 days of operation.

(3) Total pumping system failure after 1,500 days of operation.

For each case, flooding occurred after a relatively short period. Figure VI-9 shows the zones where flooding is likely to occur for the scenarios listed above. Some of the zones of flooding shown near the south margin of the map area along First Creek are due to differences between observed and calibrated water levels. Unfortunately, this is where the least accurate water level data exist, and the simulated flood zone is only

MODEL SIMULATION RESULT
FLOODING DUE TO PUMP SYSTEM FAILURE

(ASSUMING NO DRAINAGE ALONG FIRST CREEK)

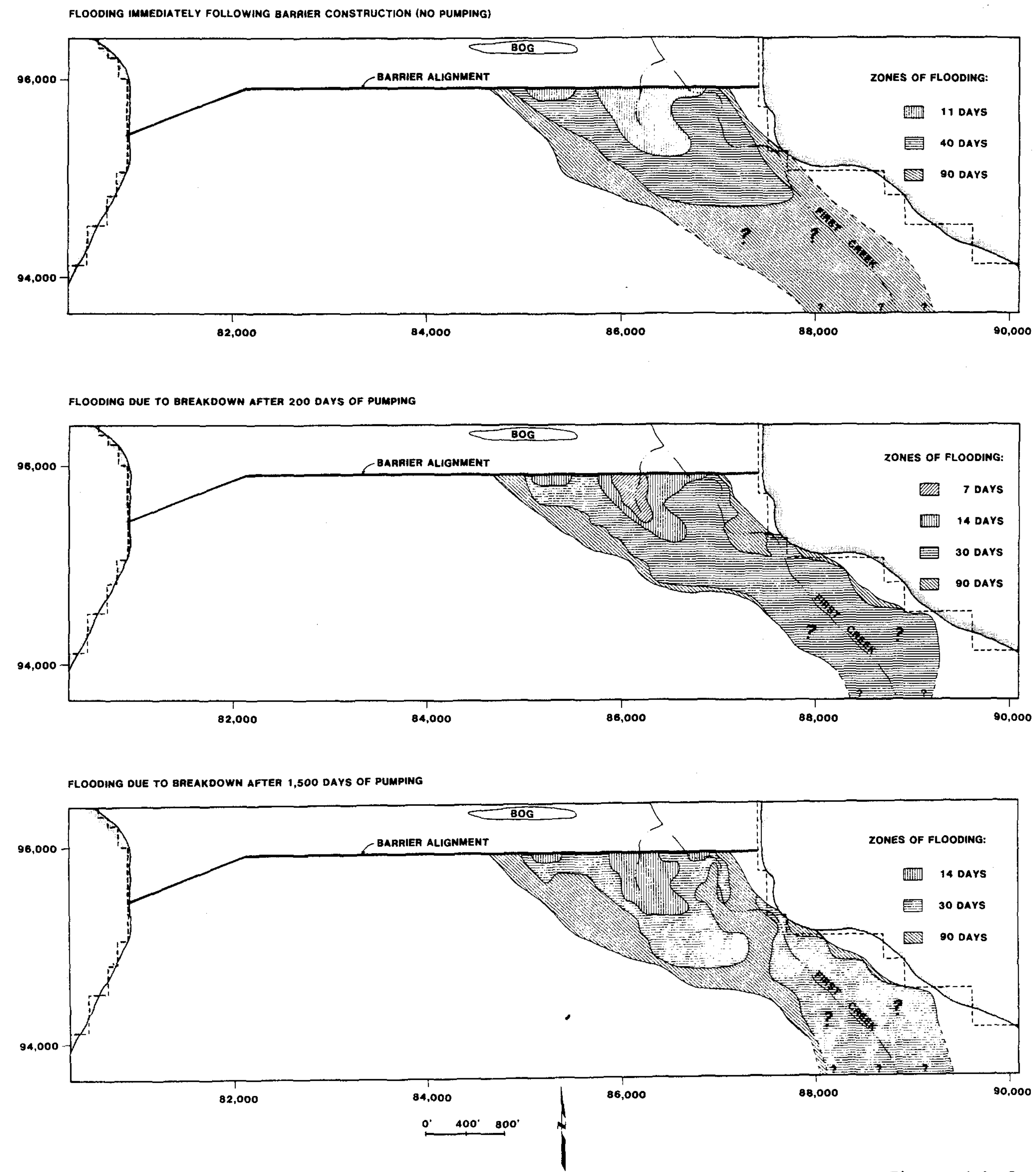


Figure VI-9

approximate. However, the flooding simulated near the barrier is relatively accurate.

g. The resulting impacts of flooding are summarized below.

(1) If total pump well failure occurs just after the barrier is in place, flooding after 11 days will occupy over 4,500 square feet and will first occur approximately 1,600 feet from the eastern end of the cutoff wall. The flooded zone will continue to develop with time.

(2) If a total pump well failure occurs after 200 days of operation, flooding will occur almost immediately. A significant flooded zone appears in less than one week.

(3) A failure after 1,500 days of operation will result in significant flooding after two weeks.

h. The need to intercept 110 percent of the natural flow rate will have an impact on the flow system to the south. This is especially true in the vicinity of the northern end of Basin F where ground water gradients are very gentle. However, the eventual water level responses to North Boundary operations at Basin F are unknown. It is recommended that a regional model of the entire arsenal should be developed so that impacts can be predicted and containment facilities can be designed to be compatible with each other.

i. The North Boundary finite-difference model should be used as an operational tool. As the barrier system is operated, the model should be recalibrated to monitoring data which will refine the model's predictive capabilities and assist operations.

j. The influence of dewatering wells on First Creek appears to be minimal. A pump test performed near First Creek from the alluvial aquifer did not indicate a recharge boundary effect after five days of pumping. The soils beneath the creek channel are clayey and must have a relatively low vertical permeability.

k. Design of alluvial dewatering wells is shown in design drawings and described in the specifications. Model predicted well capacities will probably not be achieved in some cases because of irregular clayey zones and cemented sands. As a result, pumping rates will probably have to be adjusted to some degree depending on actual performance in the field. This is not expected to adversely affect operations, but it will require careful adjustment of the system.

l. Twenty-nine new alluvial dewatering wells are proposed for the North Boundary containment system, for a total of 35 when the six existing operating wells are included. Pumping rates were assigned to each well in such a way as to minimize distortion of the ground water flows while also intercepting the flow across the North Boundary. Projected pumping rates range from 1.0 to 26.2 gpm with drawdowns of 1.42 to 4.84 feet at steady state.

m. Alluvial dewatering wells will be constructed of steel with a stainless steel screen of 0.060 inch slot size. Type 316L steel was selected for the screens because of its high corrosion resistance characteristics. Boreholes for construction of the alluvial dewatering wells will be 16 inches in diameter, which will allow a minimum 4-1/2-inch gravel pack around the 6-inch (i.d.) well screen. Screen lengths and placement were determined by constructing geologic profiles along the dewatering well alinement and interpreting bedrock contacts and location of sand lenses from adjacent boreholes. Wherever possible, screens were placed so that pumping water levels would be at or above the top of the screen.

n. Twenty-six new recharge wells are proposed for the North Boundary containment system for a total of 38 when the 12 existing wells are included. Flows from the treatment plant were apportioned among the recharge wells by use of the finite-difference model with the objective of restoring natural flow conditions without flooding. Recharge rates range from 0.4 gpm to 37.0 with increases in head of approximately 0.53 foot to 2.26 feet in the wells. Flows from the Denver Sand dewatering wells will add an additional 31 gpm to the recharge well system. The distribution of this small increase in flow was not assigned to specific wells and will be determined operationally. In an emergency, the bog could be used for recharge and it could probably accommodate a major part of the recharge water. The bog is well located for this purpose, and if it were used for recharge, the natural flow system would probably be restored a short distance downstream.

o. All recharge wells will be constructed in the alluvial aquifer, using stainless steel screen of 0.060-inch slot size. Wells will be of the gravel envelope type with a 24-inch well bore, for a large effective radius. Screen and casing will be 16 inches in diameter.

C. GROUND WATER CONTAMINATION - ALLUVIAL AQUIFER.

1. Criteria.

a. The containment facility design objective at the North Boundary is to intercept three cones of contamination so that the variable contaminant levels in these zones can be intercepted and treated separately. Water quality studies were performed to determine the extent of contaminant plumes and estimate the mass fluxes of contaminants for each dewatering well.

b. Chemical contamination of the alluvial aquifer ground water in an area between Basin F and the North Boundary was investigated using ground water chemical analysis data bases provided by RMA. Four chemical constituents were investigated in detail. These constituents and the number of wells for which ground water chemical analyses were performed are listed in Table VI-2. These chemical analysis data were first carefully screened to eliminate those data which were obtained from wells placed in the Denver Formation. For purposes of developing contours of present 1979 contaminant concentration levels, average maximum concentrations for wells with 1979 data were used. In addition to the raw data from chemical analyses described above, interpretations of similar data developed by other investigators (WES, 1979; D'Appolonia, 1979; Konikow, 1977) were studied.

c. Major emphasis was placed on developing contours representative of essentially present 1979 contamination levels. Data were not averaged over long periods of time. This is an important consideration. If a contaminant's concentration at a specific location is increasing (or

Table VI-2

CONTAMINANTS INVESTIGATED IN BASIN F TO

NORTH BOUNDARY STUDY AREA

(Wells in Alluvium)

Contaminant	Number of Wells Sampled and Tested	Number of Groundwater Samples Tested
Fluoride	184	1434
DIMP	189	1750
DCPD	184	959
DBCP	188	1750

decreasing) with time due to contaminant plume migration and dispersion, then averaging data over a long time period would generally yield an artificially low (or high) concentration level.

d. Chemical contamination contours of three of the four constituents investigated were developed and are shown on Figures VI-10 through VI-12. Contamination contours for DBCP could not be developed since sufficient data for these constituents do not presently exist. It should be noted that the contours represent "present day" (i.e. 1979) contamination levels only and are not representative of future contamination levels. Estimation of future contamination levels is complicated by the following factors:

(1) Contaminant plume convection and dispersion phenomena are complicated.

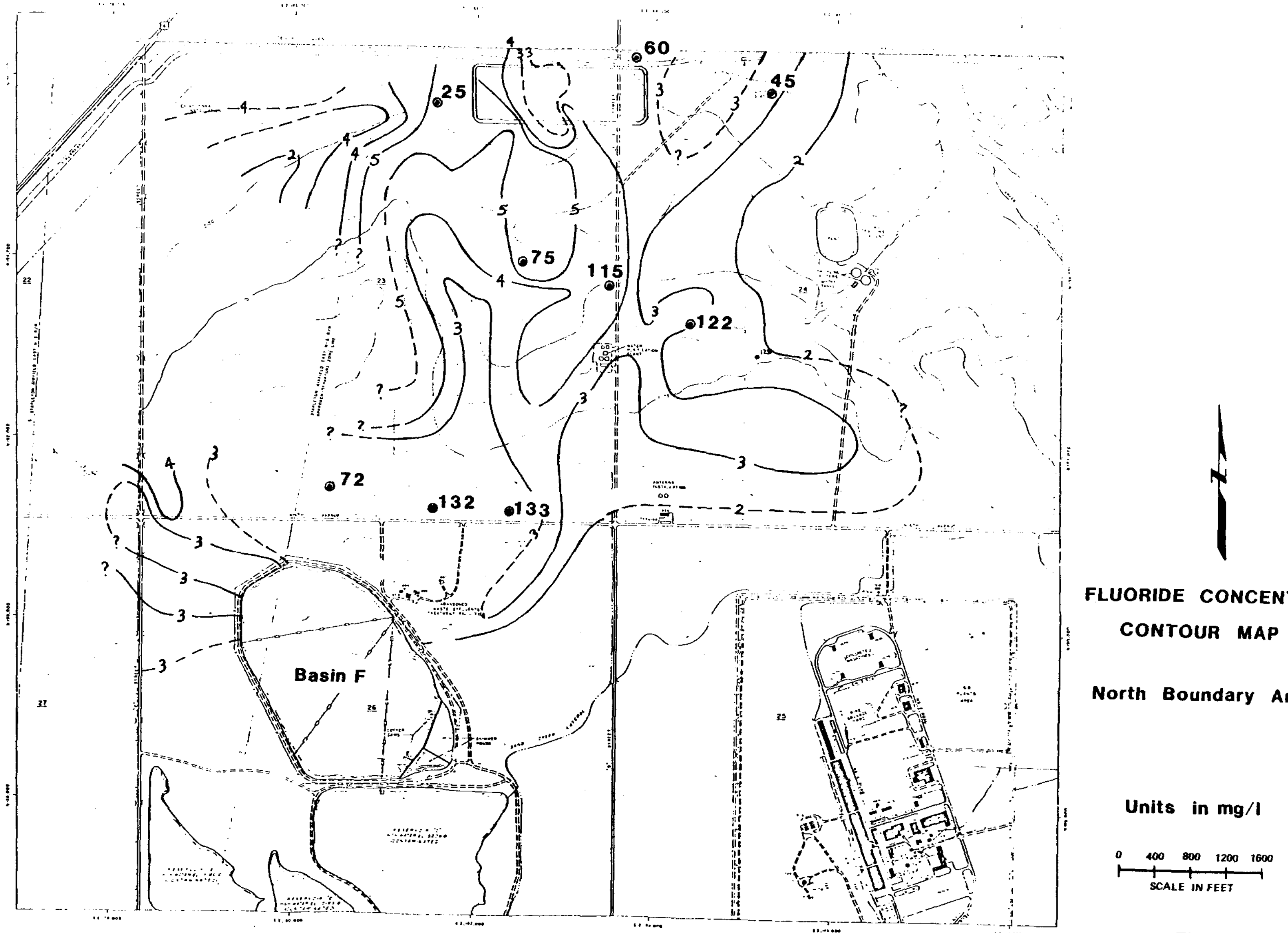
(2) Multiple unknown contaminant sources are probably present or have existed within the ground water system at RMA.

(3) Complex chemical reactions may be occurring between the various chemical contaminants, natural ground water, and soluble constituents in the formation.

(4) Sorption of contaminants by formation material is probably occurring to some degree.

e. Despite the limitations of the chemical contamination contours noted above, the contours on Figures VI-10 through VI-12 indicate the following general trends:

(1) Contaminants are contained in isolated plumes or pulses.

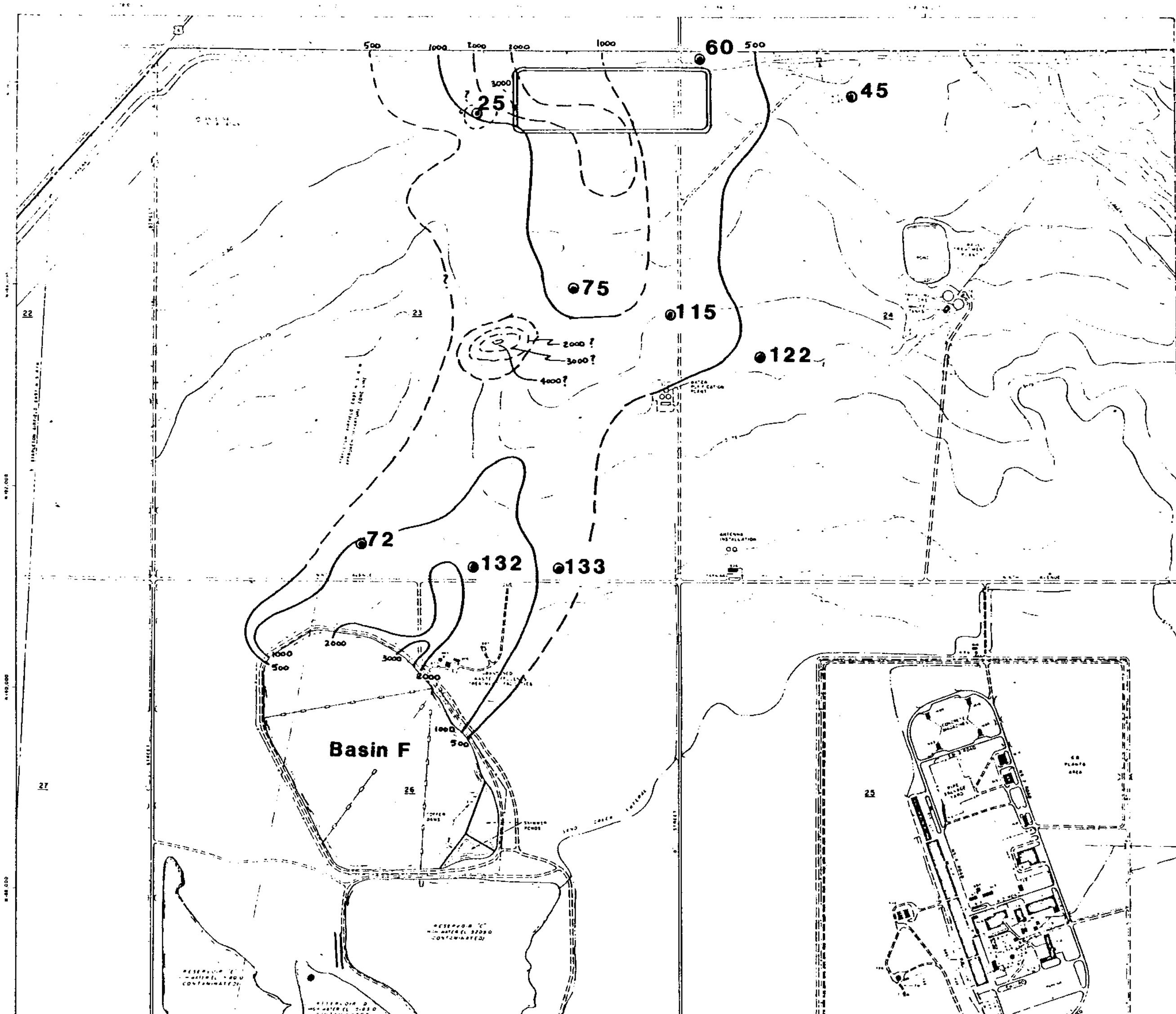


FLUORIDE CONCENTRATION CONTOUR MAP

North Boundary Area

Units in mg/l

Figure VI-10



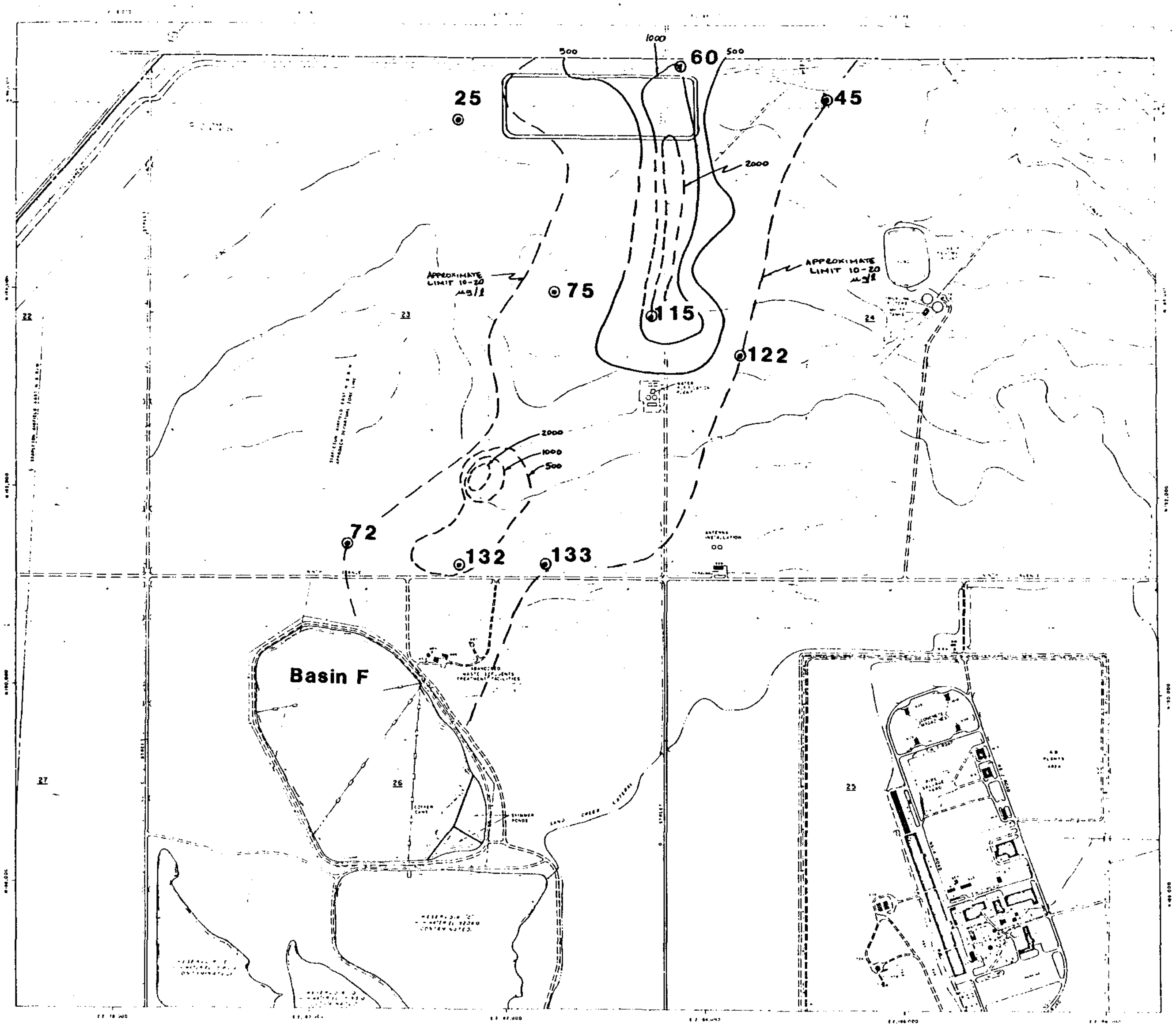
DIMP CONCENTRATION CONTOUR MAP

North Boundary Area

Units in µg/l

0 400 800 1200 1600
SCALE IN FEET

Figure VI-11



DCPD CONCENTRATION CONTOUR MAP

North Boundary Area

Units in µg/l

0 400 800 1200 1600
SCALE IN FEET

Figure VI-12

(2) Some contaminant plumes appear to be migrating from Basin F to the North Boundary, while some isolated plumes probably have other sources.

(3) High contaminant concentrations are generally confined to the western portion of the North Boundary.

f. Chemical contaminant breakthrough curves for nine selected wells were developed and studied. The breakthrough curves shown on Figures VI-13 through VI-15 represent the contaminant time histories at the given locations. The nine wells selected for analysis were chosen from a total of 19 wells within the North Boundary area having at least four years of chemical analysis data. The locations of the 19 wells having four years of data are shown on Figure VI-16. The selection of the nine wells chosen from the 19 for analyses was based on their relative position with respect to plume locations and their spatial coverage of the North Boundary area.

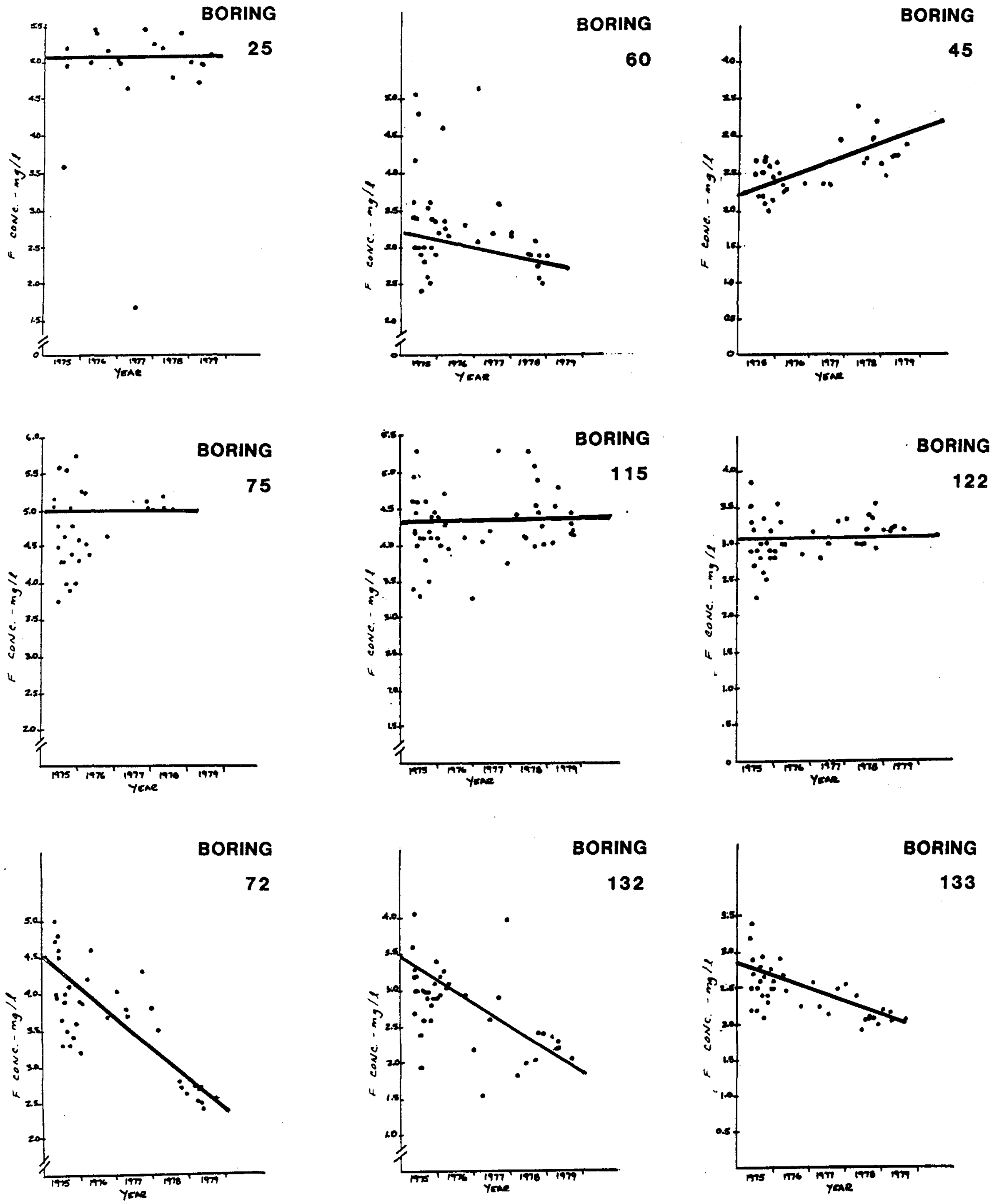
g. Breakthrough curves were developed by drawing straight lines through the rather scattered chemical concentration versus time data shown on Figures VI-13 through VI-15. These lines were used to illustrate general trends in the data and do not represent detailed contamination trends.

2. Analysis.

a. The following observations were made from the chemical analysis data shown in the breakthrough curves and contour plots generated from present 1979 data:

(1) Fluoride

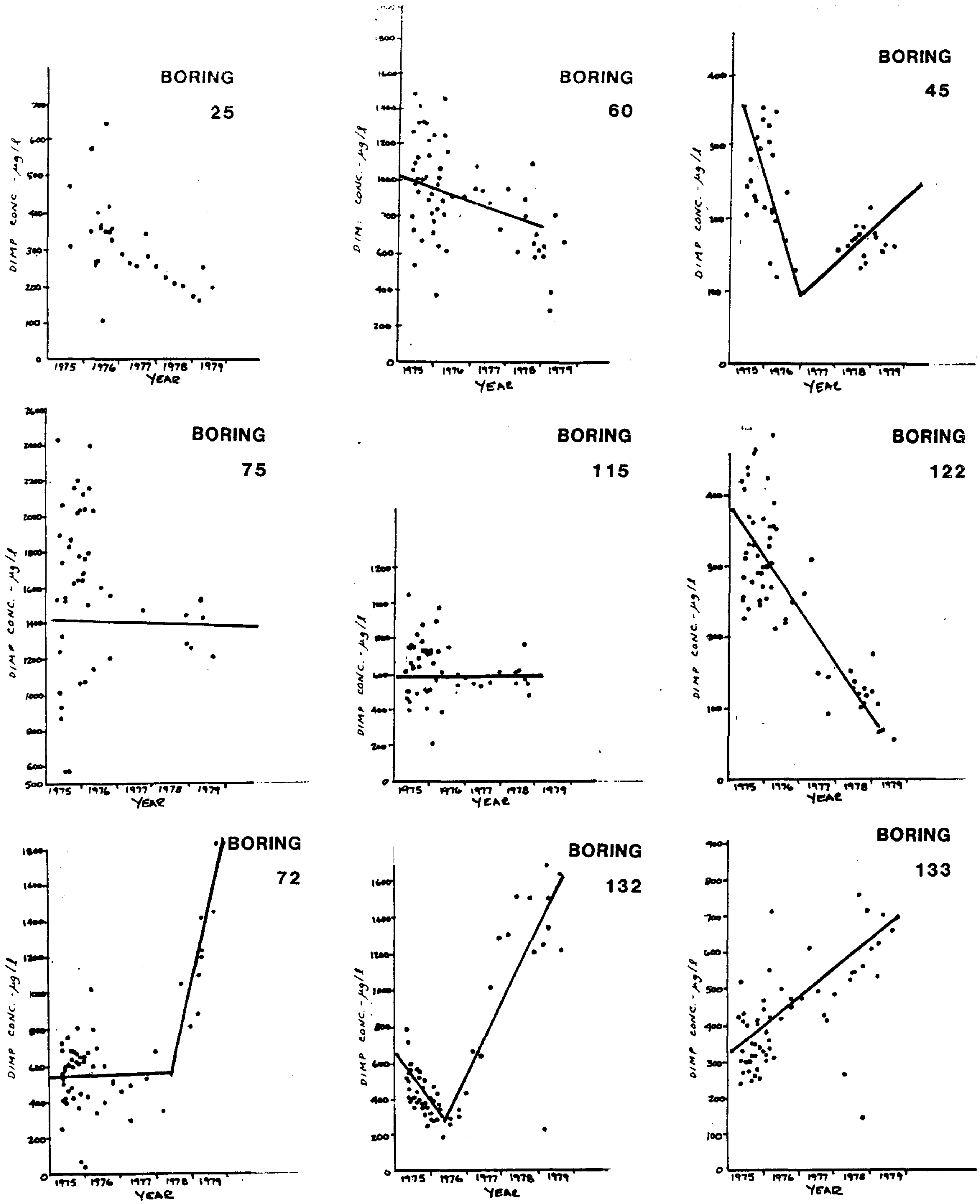
(a) Wells close to Basin F show a general downward trend in concentrations indicating that the plume peak has passed these locations.



BREAKTHROUGH CURVES

FLUORIDE CONCENTRATION - mg/l

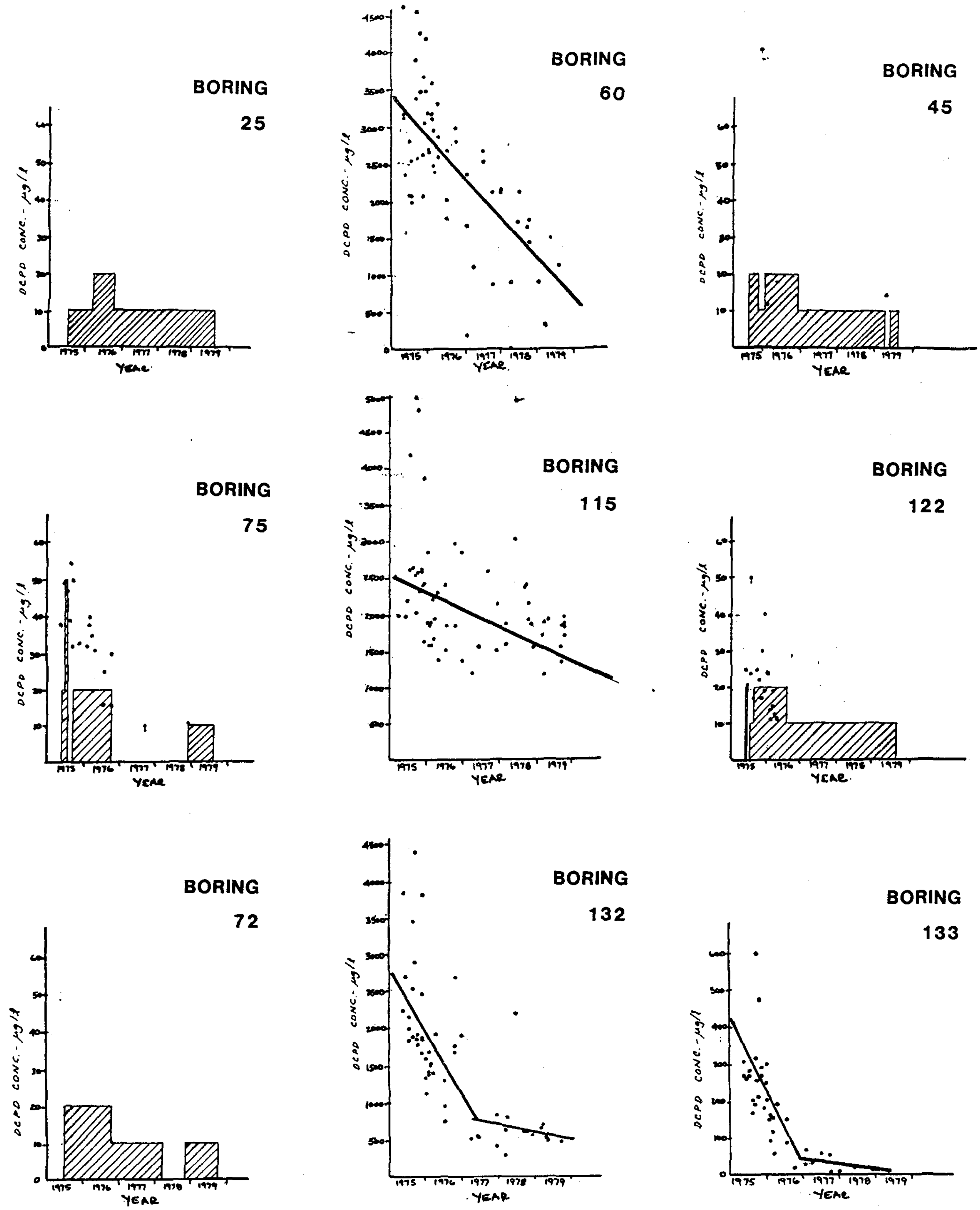
Figure VI-13



BREAKTHROUGH CURVES

DIMP CONCENTRATION - µg/l

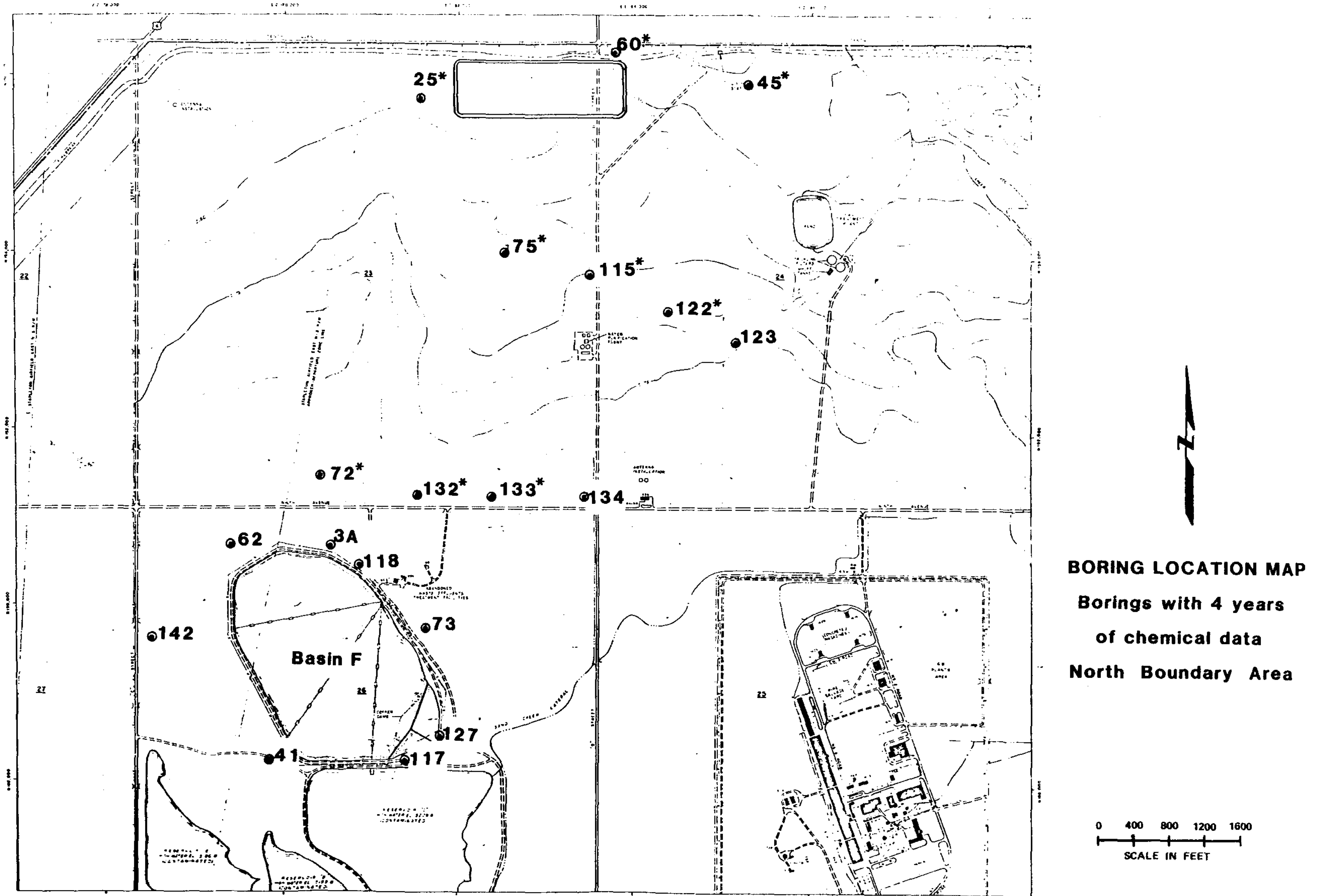
Figure VI-14



BREAKTHROUGH CURVES

DCPD CONCENTRATION - µg/l

Figure VI-15



BORING LOCATION MAP
Borings with 4 years
of chemical data
North Boundary Area

FIGURE VI-16

*Indicates that boring was used to develop Breakthrough Curves

(b) Concentration of contaminants in wells midway between Basin F and North Boundary has remained relatively constant within the time period considered. This indicates that the center of plume has been passing these points for some time and concentrations should start to decrease.

(c) Wells within the western portion of North Boundary area indicate constant or dropping concentrations which may be related to the influence of the Pilot Facility.

(d) A well within the western portion of North Boundary shows increasing concentrations due to migration of the dispersed plume front.

(2) DIMP

(a) Wells close to Basin F show increasing concentrations and indicate a new plume migration from Basin F.

(b) Wells midway between Basin F and North Boundary show constant or decreasing concentrations.

(c) Wells near the North Boundary show decreasing concentrations to the west and adjacent to the Pilot Plant but increasing concentrations to the east.

(3) DCPD

(a) All wells show a general trend in data. However, contamination should start to increase along North Boundary as a new plume moves northward as shown on the contour map of this constituent.

(4) DBCP

(a) This contaminant could not be contoured and analyzed because of limitations of data.

b. The following conclusions can be made based on the observations stated above and other observations of chemical analysis data.

(1) The breakthrough curves illustrate the variable patterns of migration and dispersion of contaminant plumes.

(2) The contamination levels should not be expected to remain constant with time.

(3) Unexplained trends in the breakthrough curves, contaminant plume migration patterns and isolated contaminant pulses (or slugs) illustrate the complexity of the natural flow system.

(4) Multiple sources have probably contributed to the complex plume pattern. Basin F is one obvious source, but other sources have not been defined in detail.

These conclusions again suggest that estimation of future contamination levels along the North Boundary cannot be accomplished without a detailed solute-transport model. Only present day 1979 contaminant concentrations can be estimated with confidence. At best, only maximum future concentrations along the North Boundary can be estimated using an understanding of the present ground water flow system and present maximum contaminant concentration existing in the area between Basin F and the North Boundary. An upper limit for maximum future contaminant concentrations of the North Boundary pumping system can be estimated by assuming that: (1) the ground water system will not be greatly affected by the proposed North Boundary slurry trench barrier and associated treatment facilities; and (2) dispersion of contaminants is neglected.

c. Mass fluxes for the four previously mentioned contaminants were calculated for two situations listed below:

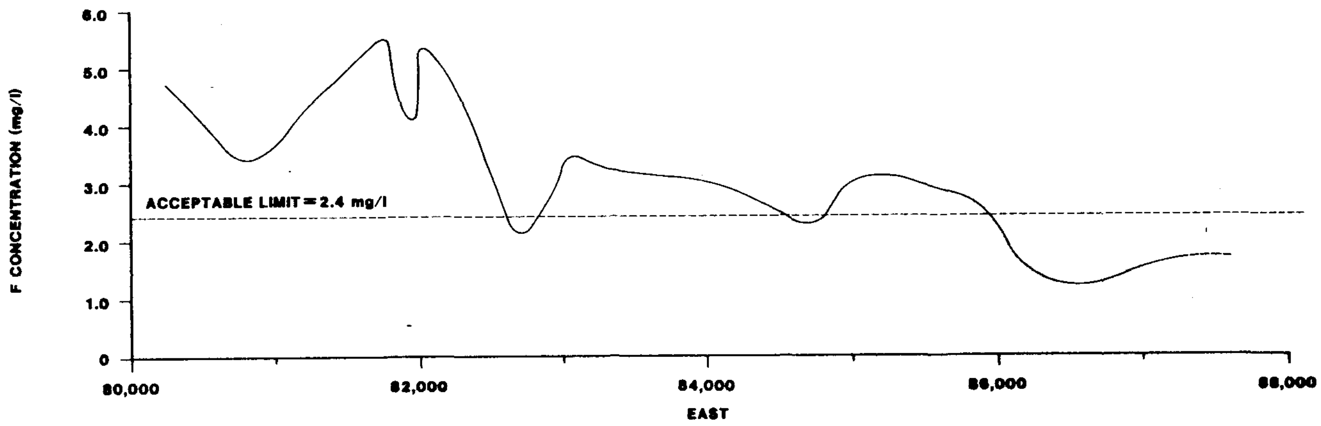
(1) Present 1979 contaminant concentrations along the cutoff wall alinement.

(2) Upper limit to probable future contamination concentrations along the cutoff wall alinement.

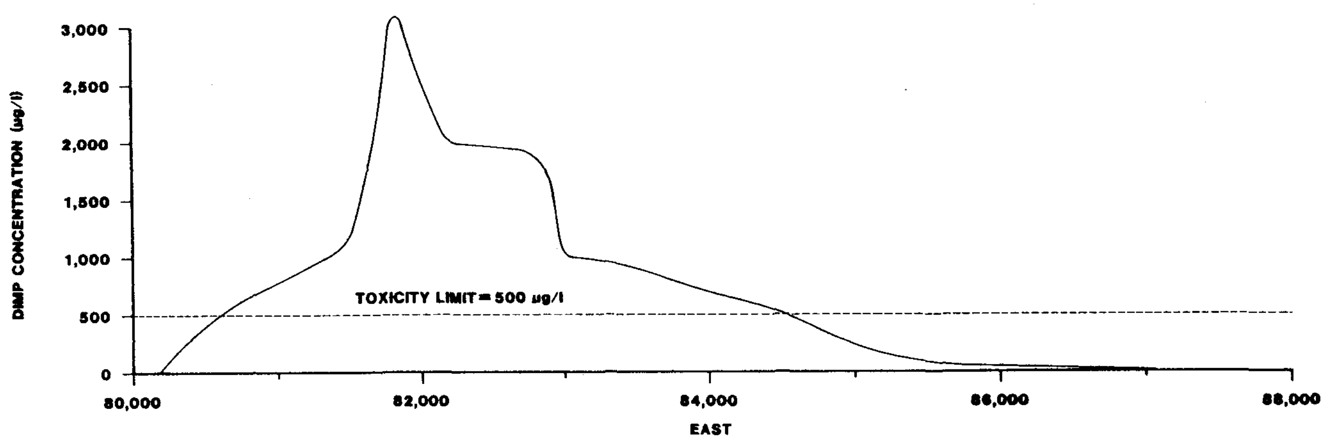
d. Flux values were calculated by multiplying the pumping rate of each of the 35 dewatering wells by the concentration of the contaminant in the area influenced by the well. The pumping rates and total system discharge were obtained from the finite difference modeling results. The concentrations of the contaminants were determined differently for the above two cases.

e. Profiles showing contaminant concentrations along the cutoff alinement were developed from the contaminant contour maps previously described. These profiles are shown on Figures VI-17 and VI-18. A statistical average concentration for the area influenced by each discharge well was calculated from the values on the profiles. To be conservative, this average was weighted toward the value at the dewatering well if this value was higher. The results of these calculations are shown in Table VI-3 for the four chemical contaminants fluoride, DIMP, DBCP, and DCPD. The sums for the entire system are also shown.

f. Upper limit of probable future contaminant concentrations was estimated by using detailed knowledge of the ground water flow system to project the highest concentrations from the contaminant contour maps to the cutoff wall alinement. This approach is equivalent to assuming that

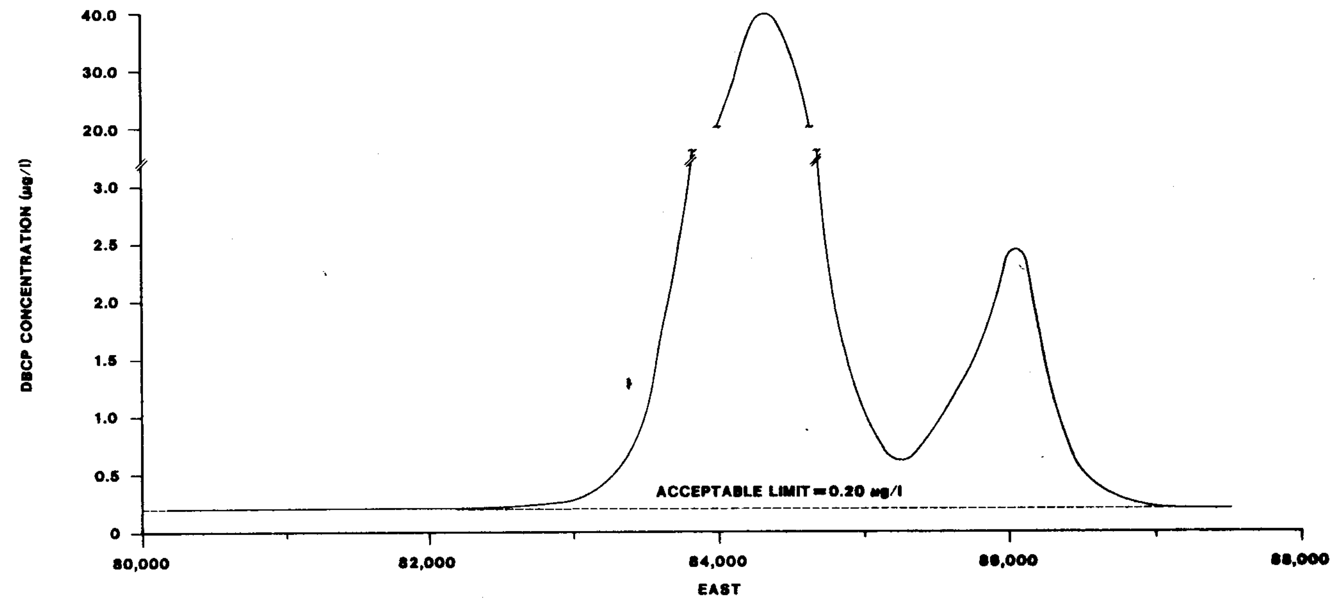


FLUORIDE CONCENTRATION ALONG 95,930 N
YEAR 1979

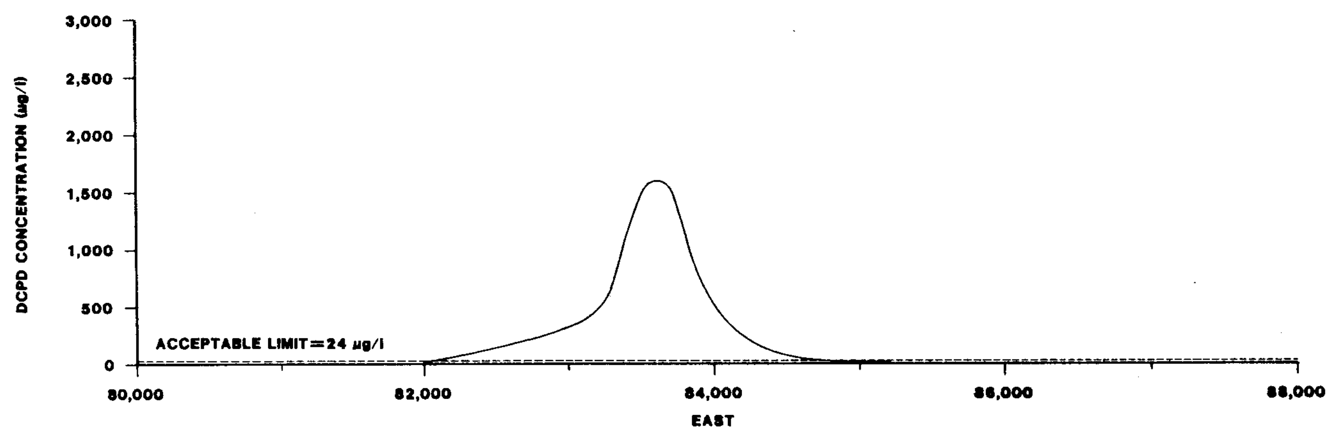


DIMP CONCENTRATION ALONG 95,930 N
YEAR 1979

Figure VI-17



DBCP CONCENTRATION ALONG 95,930 N
YEAR 1979



DCPD CONCENTRATION ALONG 95,930 N
YEAR 1979

Figure VI-18

Table VI-3

PRESENT (1979) CONTAMINANT MASS FLUX
(Using Weighted Average Values from Wells)

Well Number	Pumping Rate Q GPM	Pumping Rate Q Liters/ Day	Fluoride Average Conc.-gm/ℓ	Fluoride Mass Flux gm/day	Dimp Avg. Conc. gm/ℓ	Dimp Mass Flux gm/day	DCPD Avg. Conc. gm/ℓ	DCPD Mass Flux gm/day	DBCD Avg. Conc. gm/ℓ	DBCD Mass Flux gm/day
DW-1	3.95	21464	.00342	73.41	.00195	41.85	.00015	3.22	.00000022	.0047
DW-2	5.72	31082	.00240	74.60	.00191	59.37	.00020	6.22	.00000023	.0072
DW-3	8.92	48471	.00275	133.30	.00157	76.10	.00028	13.57	.00000025	.0121
DW-4	11.25	61132	.00334	204.18	.00099	60.52	.00039	23.84	.00000032	.0196
DW-5	12.76	69337	.00327	226.73	.00094	65.18	.00079	54.78	.00000054	.0374
DW-6	14.87	80803	.00315	254.53	.00087	70.30	.00147	118.78	.00000122	.0986
DW-7	16.21	88085	.00310	273.06	.00080	70.47	.00143	125.96	.00000229	.2017
DW-8	16.45	89389	.00305	272.64	.00073	65.25	.00085	75.98	.00001000	.8939
DW-9	17.94	97485	.00296	288.56	.00066	64.34	.00039	38.02	.00002569	2.5044
DW-10	14.78	80314	.00281	225.68	.00061	48.99	.00021	16.87	.00003830	3.0760
DW-11	14.78	80314	.00262	210.42	.00056	44.98	.00012	9.64	.00003903	3.1347
DW-12	20.00	108679	.00241	261.92	.00049	53.25	.00007	7.61	.00003735	4.0592
DW-13	22.84	124112	.00232	287.94	.00040	49.64	.00004	4.96	.00001779	2.2080
DW-14	21.94	119221	.00254	302.82	.00031	36.96	.00003	3.58	.00000200	.2384
DW-15	20.78	112918	.00292	329.72	.00024	27.10	.00003	3.39	.00000117	.1321
DW-16	20.13	109386	.00315	344.57	.00018	19.69	.00003	3.28	.00000078	.0744
DW-17	25.95	141011	.00315	444.18	.00013	18.33	.00003	4.23	.00000068	.0959
DW-18	26.18	142261	.00295	419.67	.00009	12.80		---	.00000093	.1323
DW-19	25.66	139435	.00279	389.02	.00006	8.37		---	.00000131	.1827
DW-20	23.85	129600	.00255	330.48	.00004	5.18		---	.00000193	.2501
DW-21	15.29	83085	.00203	168.66	.00003	2.49		---	.00000234	.1944
DW-22	13.78	74880	.00148	110.82	.00003	2.25		---	.00000185	.1385
DW-23	18.62	101180	.00128	129.51	.00003	3.04		---	.00000092	.0931
DW-24	18.94	102919	.00122	125.56	.00002	2.06	<.00003	---	.00000043	.0288
DW-25	19.13	103952	.00132	137.22	.00002	2.08		---	.00000028	.0291
DW-26	10.60	57600	.00146	84.10	.00002	1.15		---	.00000023	.0133
DW-27	10.00	54340	.00157	85.31	<.00001	0.54		---	.00000021	.0114
DW-28	9.00	48906	.00164	80.21	<.00001	---		---	.00000020	.0098
DW-29	6.00	32604	.00168	54.78	.00001	---		---	.00000020	.0065
DW-30	1.03	5597	.00401	22.44	.00085	4.76		---	.00000020	.0011
DW-31	1.95	10596	.00473	50.12	.00105	11.13	<.00003	---	.00000020	.0021
DW-32	2.98	16193	.00519	84.04	.00164	26.56		---	.00000020	.0032
DW-33	5.51	29941	.00535	160.18	.00270	81.14		---	.00000020	.0060
DW-34	3.97	21573	.00520	112.18	.00236	50.91	.00005	1.08	.00000020	.0043
DW-35	3.91	21247	.00456	96.89	.00198	42.07	.00010	2.12	.00000022	.0047
	ΣQ=485.67 gpm	2639111 ℓ/day		Σflux= 6851.4 gm/day		Σflux= 1128.85 gm/day		Σflux= 517.13 gm/day		Σflux= 17.910 gm/day

the plumes propagate according to the principles assumed for the finite difference flow model and ignores the presence of convective or dispersive flow phenomena. This simple approach is presumed to be conservative and the maximum values, as presented in Table VI-4 are considered as an upper limit of concentrations to be expected.

g. The values given in Table VI-3 are based on 1979 data and are assumed to be representative of the contamination levels one would presently observe along the trench alinement. The upper limit values given in Table VI-4 are indicative of maximum expected levels but no inference as to when these levels may be expected can be made.

h. It is observed that future levels of DCPD can be expected to increase up to 300 percent with respect to 1979 levels. All other contaminants can be expected to increase only by 10 to 15 percent. This is probably due to the various source and plume propagation conditions of the different contaminants.

D. DEWATERING WELLS - DENVER SANDS.

1. Criteria.

a. Contaminants including fluorides, DIMP, DBCP and DCPD have been detected in Denver Sands at the North Boundary. Contaminant levels are generally low and are erratically distributed in sand lenses ranging from depths 20 to 105 feet below ground surface. In general, more contamination is present to the west of the study area beneath the more concentrated contaminant plumes in the overlying alluvial aquifer.

Table VI-4

PROBABLE UPPER LIMIT OF CONTAMINANT MASS FLUX
(Based on 1979 Chemical Distribution data)

Well Number	Pumping Rate Q GPM	Pumping Rate Q Liters/Day	Fluoride Average Conc.-gm/ℓ	Fluoride Mass Flux gm/day	Dimp Avg. Conc. gm/ℓ	Dimp Mass Flux gm/day	DCPD Avg. Conc. gm/ℓ	DCPD Mass Flux gm/day	DBCD Avg. Conc. gm/ℓ	DBCD Mass Flux gm/day
DW-1	3.95	21464	.00575	123.42	.003750	80.49	.00200	42.93		
DW-2	5.72	31082	.00575	178.72	.004250	132.10	.002500	77.71		
DW-3	8.92	48471	.00575	278.71	.004250	206.00	.003000	145.41		
DW-4	11.25	61132	.00550	336.23	.002250	137.55	.002500	152.83		
DW-5	12.76	69337	.00450	312.02	.001500	104.01	<.003000	<208.01		
DW-6	14.87	80803	.00430	347.45	.000850	68.68	<.003000	<242.41		
DW-7	16.21	88085	.00375	330.32	.000800	70.47	.002000	176.17		
DW-8	16.45	89389	.00350	312.86	.000750	67.04	.001250	111.74		
DW-9	17.94	97485	.00340	331.45	.000700	68.24	.001000	97.49	Insufficient Data Provided For Analysis	
DW-10	14.78	80314	.00330	265.04	.000600	48.19	.000750	60.24		
DW-11	14.78	80314	.00325	261.02	.000550	44.17	.000500	40.16		
DW-12	20.00	108679	.00315	342.34		---	.000400	43.47		
DW-13	22.84	124112	.00310	384.75		---	.000300	37.23		
DW-14	21.94	119221	.00310	369.59		---	.000250	29.81		
DW-15	20.78	112918	.00310	350.05		---	.000200	22.58		
DW-16	20.13	109386	.00310	339.10		---	.000100	10.94		
DW-17	25.95	141011	.00310	437.13		---	.000020	2.82		
DW-18	26.18	142261	.00300	426.78		---		<1.42		
DW-19	25.66	139435	.00270	376.47		---		---		
DW-20	23.85	129600	.00230	298.08	<.00050	---		---		
DW-21	15.29	83085	.00210	174.48		---		---		
DW-22	13.78	74880		---		---		---		
DW-23	18.62	101180		---		---		---		
DW-24	18.94	102919		---		---		---		
DW-25	19.13	103952	<.00200	---		---	<.000010	---		
DW-26	10.60	57600		---		---		---		
DW-27	10.00	54340		---		---		---		
DW-28	9.00	48906		---		---		---		
DW-29	6.00	32604		---		---		<.33		
DW-30	1.03	5597	.00575	32.18	.002750	15.39	.000550	3.08		
DW-31	1.95	10596	.00575	60.93	.003000	31.79	.000575	6.09		
DW-32	2.98	16193	.00575	93.11	.003750	60.72	.000600	9.72		
DW-33	5.51	29941	.00600	179.65	.003000	89.82	.000625	18.71	Insufficient Data Provided For Analysis	
DW-34	3.97	21573	.00600	129.44	.003000	64.72	.000650	14.02		
DW-35	3.91	21247	.00575	122.17	.002750	58.43	.001000	21.25		
	Σ Q=485.67 gpm	2639111 ℓ/day		Σ flux = 7193.49 gm/day		Σ flux = 1347.81 gm/day		Σ flux = 1576.57 gm/day		

b. Nemagon (DBCP) is the most prevalent contaminant in the area in the Denver Sands but it usually occurs in very low concentrations near detection limits of 0.2 μg/l. Since this is the water quality standard for interception and treatment, it is very difficult to judge if true contamination exists due to possible errors in sampling and chemical analyses. This constituent, therefore, controls the extent of suspected contamination in the Denver Sands.

c. Fluorides, DIMP, and DCPD have only been detected at levels above water quality standards in isolated cases. Fluorides above 2.4 mg/l have only been reported from Wells 991M and 984S. DIMP concentrations above 500 μg/l has only been reported from Well 991M and DCPD concentrations above 24 μg/l has only been reported in Well 981S. Analyses of samples collected from Wells 1041 and 1045 during pump tests did not reveal concentrations of any of the four constituents above water quality standards. Well 1045 was pumped from a shallow sand that is beneath the existing barrier where contamination levels are high in the overlying alluvium. Interception of Denver Sands may prove that waters generally meet water quality standards. Nevertheless, higher contamination levels may exist upgradient from the barrier and it is prudent to develop facilities for interception combined with monitoring. The Denver Sands constitute very low permeability aquifers, and contaminant plumes will move at extremely slow rates (probably no more than about 40 feet per year).

d. The extent and geometry of Denver Sands was determined from borehole data and shown on Figure VI-19. Sands that are suspected to be contaminated were determined from chemical data.

EXPLANATION FOR GEOLOGIC CROSS SECTION ALONG CENTER LINE OF PROPOSED BARRIER

UNITS

SP	Sand, poorly sorted
SM	Silty Sand
SC	Clayey Sand
GP	Gravel, poorly sorted
GC	Clayey Gravel
ML	Silt, low plastic
CL	Clay, low plastic
CH	Clay, high plastic

SYMBOLS

Top of Denver Formation; contact between Alluvium and Denver Formation.

Approximate depth of existing trench.

Depth of proposed trench.

Depth of weathering

Sand lenses in Denver Formation.

Denver sand lenses intercepted by dewatering wells.

Approximate contact separating alluvial clays and silts from alluvial sands and gravels.

Spring, 1979, water levels.

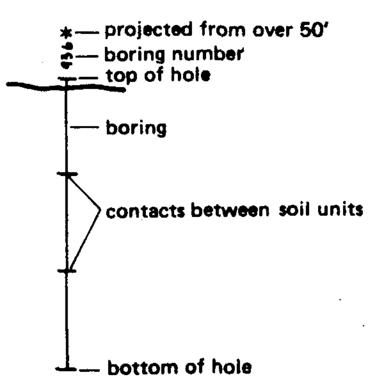
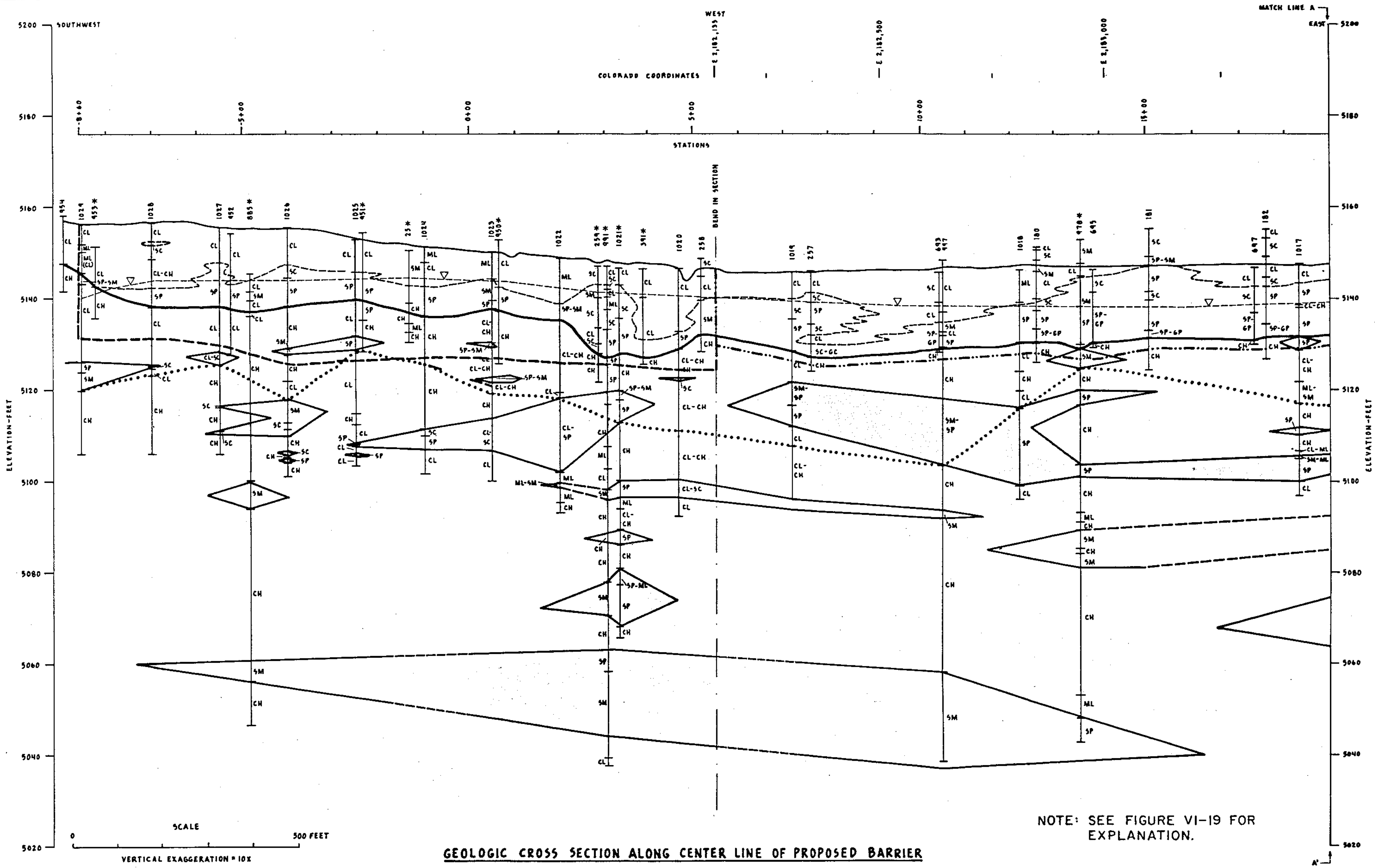


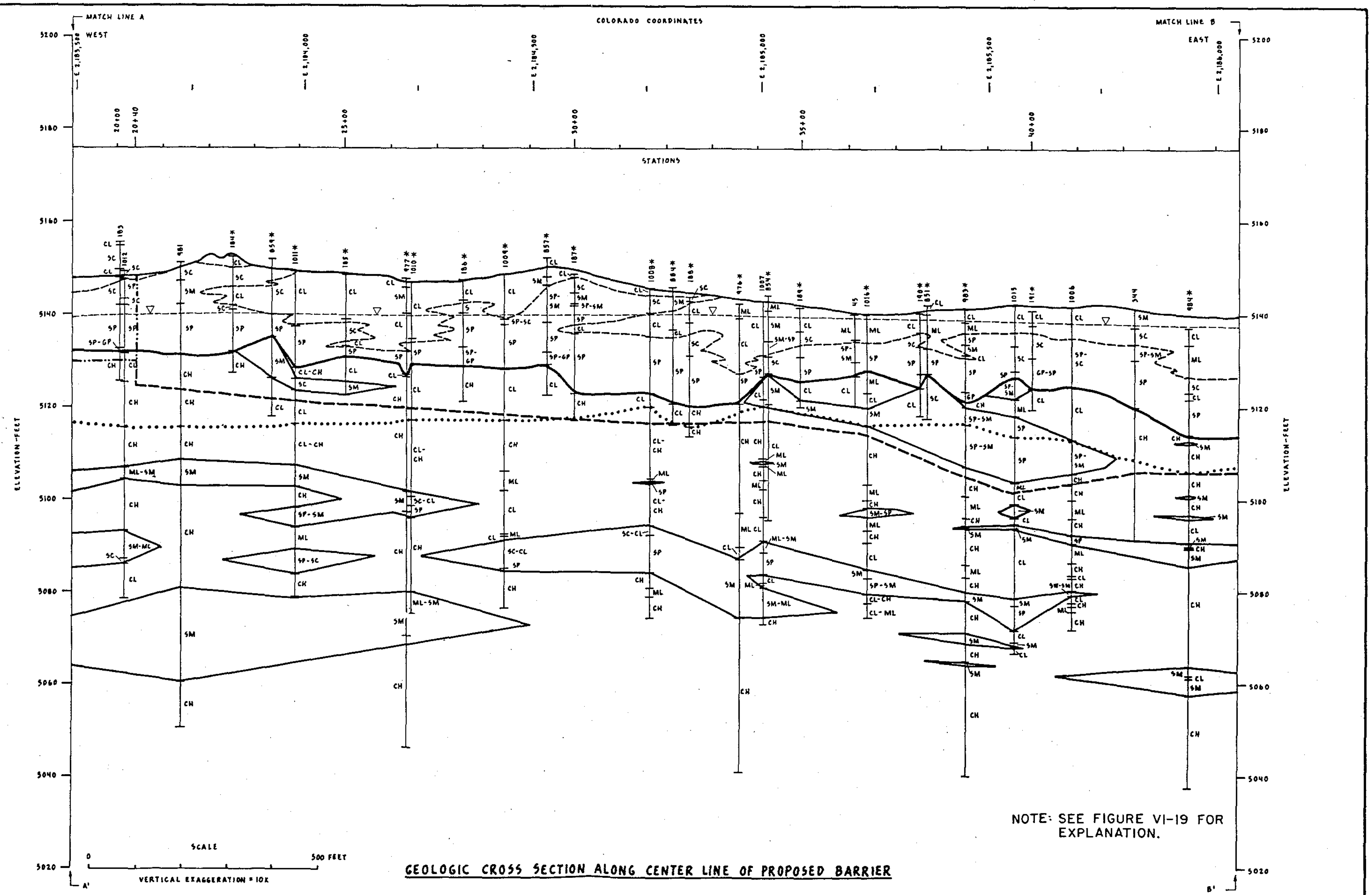
Figure VI-19



NOTE: SEE FIGURE VI-19 FOR EXPLANATION.

GEOLOGIC CROSS SECTION ALONG CENTER LINE OF PROPOSED BARRIER

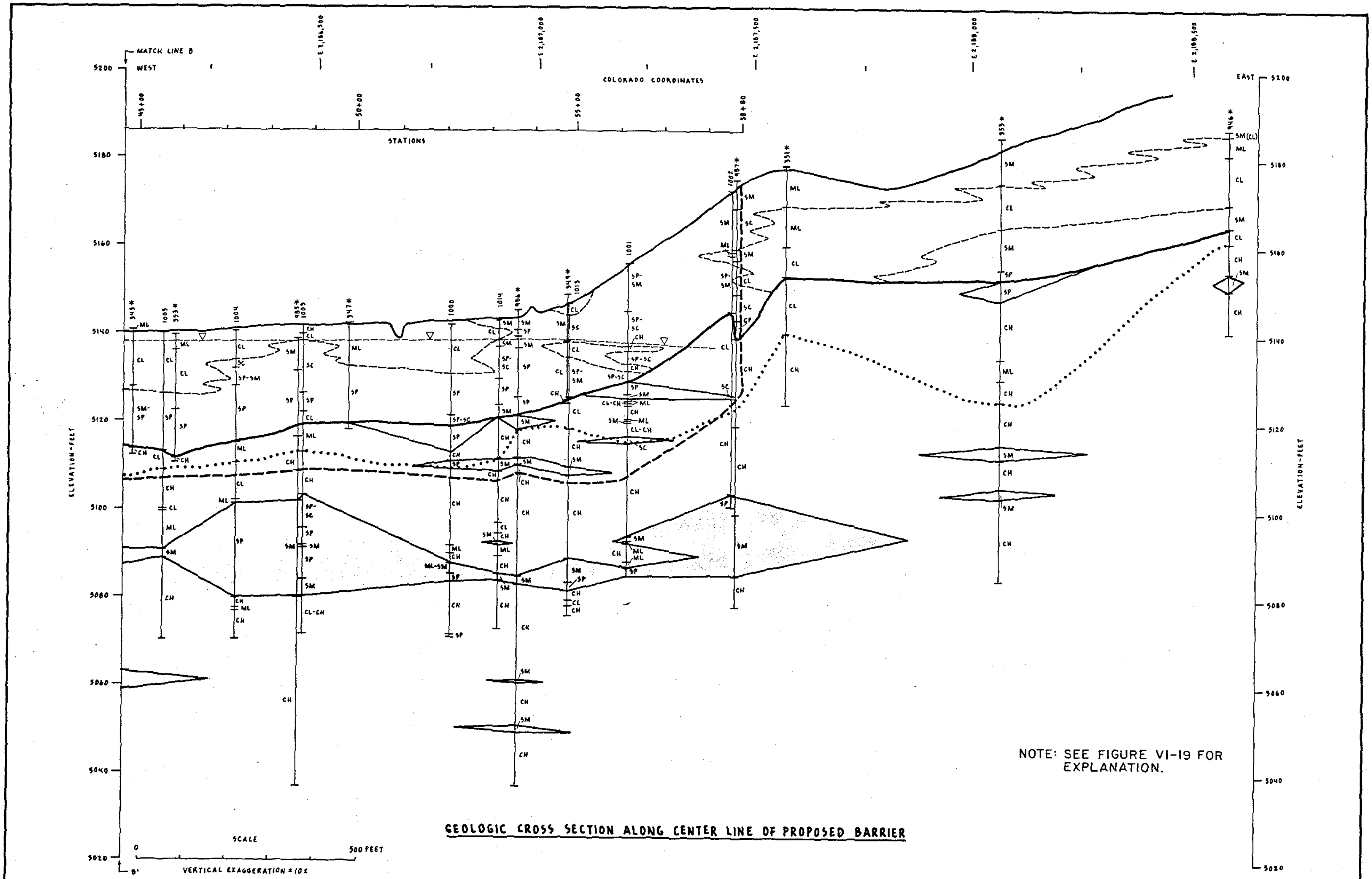
Figure VI-19A



NOTE: SEE FIGURE VI-19 FOR EXPLANATION.

GEOLOGIC CROSS SECTION ALONG CENTER LINE OF PROPOSED BARRIER

Figure VI-19B



GEOLOGIC CROSS SECTION ALONG CENTER LINE OF PROPOSED BARRIER

Figure VI-19C

e. A dewatering well system is required to develop a hydraulic sink (pumping trough) that will intercept contaminated flows. The dewatering well system is designed to operate on an as-needed basis and also serve to monitor contaminants.

f. Hydraulic design criteria were provided by two test wells each with observation wells. Pump tests were run to determine aquifer characteristics. These tests were supplemented by slug tests run by WES in observation wells. The two test wells were located at the pilot barrier and near First Creek.

2. Analysis.

a. Pump test results are summarized in Table VI-5 and field data, data plots, and calculations along with logs of test wells and observation wells are appended. The hydraulic conductivity ranged from 8.3 to 11.8 gpd per square foot in the two sand lenses tested and the average hydraulic conductivity is about 10 gpd per square foot (488 feet per year). Both tests indicated a high degree of confinement with little leakance. A slight amount of leakance was detected from testing Well 1041 near First Creek which permitted calculation of the vertical hydraulic conductivity of the confining shales. The average value calculated from two observation wells for vertical hydraulic conductivity was approximately 0.1 foot per year. WES ran a slug test nearby in shale and obtained a horizontal hydraulic conductivity of about 20 feet per year. Both of these values suggest fracture-dominated flow for this type of rock.

b. Distance drawdown calculations were used to estimate dewatering well spacings and design pumping rates. An average hydraulic

Table VI-5
SUMMARY
of
PUMP TEST RESULTS
in
DENVER SANDS, ROCKY MOUNTAIN ARSENAL

Test No.	Obs. Well	T (gpd/ft)	S	k'/m'	m'	k' (gpd/ft^2)	k' (ft/yr)
1041-1 (24154)	1042 (24155)	176	0.0004	0.000176	15	0.00264	0.129
"	985 24144 (24144)	196	0.00015	0.000082	15	0.00123	0.060
"	1043 24156 (24156)	243	0.000026	N/A		N/A	N/A
"	1041 (24154)	148	N/A	(recovery test in pumped well)			
	Approx. Average	200	0.0001				
1045-1* (23176)	1018* (23127)	754*	0.0042	(obscured by boundary effects)			
"	1046* (23177)	682*	0.0051	(obscured by boundary effects)			
	*(Test aborted because constant pumping rate not maintained, results unreliable.)						
1045-2 (23176)	1018 (23127)	234	0.0027	(obscured by boundary effects)			
"	1046 (23177)	184	0.0044	(obscured by boundary effects)			
"	1045 (23176)	202	N/A	(recovery test in pumped well)			
	Approx. Average	200	0.0036				

NOTES:

Average k in vicinity of well 1041 $= \frac{200}{24} = 8.3$ gpd/ft^2 = 405 ft/yr

Average k in vicinity of well 1045 $= \frac{200}{17} = 11.8$ gpd/ft^2 = 576 ft/yr

Average $k_{(horiz)}$ Denver Sands = 10 gpd/ft^2 = 488 ft/yr

Average k' Denver Shale = 0.019 gpd/ft^2 = 0.094 ft/yr

T = transmissibility
S = storage coefficient
m' = saturated thickness of confining layer (Denver shale)
k' = vertical hydraulic conductivity of confining layer (Denver Shale)
$k_{(horiz)}$ = hydraulic conductivity of aquifer (Denver Sands)

conductivity of 10 gpm per square foot was used in these calculations for all sands although slug tests indicate that permeabilities may be lower in many of the sand lenses. Well spacings of about 200 feet were generally selected with pumping rates of 1 to 5 gpm based on sand thickness. Some variation in spacing was used depending on the location and geometry of the sands. The calculated natural flow through the sands that are suspected to be contaminated is estimated at only about 3 gpm.

c. A total of 19 dewatering wells were designed to pump a total of 31 gpm of about 10 times the natural flow rate. The large number of wells are necessary to develop a deep sink to intercept contaminants during relatively short pumping periods. Because of severe boundaries, interference and drawdown calculations are very approximate and much of the design is based on judgment. It is estimated that wells will pump only 10 to 100 days before maximum design drawdowns in the wells are reached. When that point is reached the wells will have to be cycled on and off. This cycling or intermittent pumping will reduce the average total pumping rate, estimated to be about 15 gpm or less.

d. Because dewatering wells are designed for low pumping rates, slotted 4-inch diameter PVC casing will be used for screened intervals in a 10-inch well bore. The 3-inch annular space will be gravel packed to prevent caving of clay shales into the screened intervals. A 10-inch PVC conductor casing placed in a 16-inch well bore and grouted in place with cement by the mud displacement method will be used to seal the well from the overlying alluvial aquifer. Details are shown on design drawings and described in the specifications.

e. Dewatering wells should be monitored closely for contaminant concentrations. If contamination is not present, they should be shut down and used only for periodic monitoring. Pumping will stress these sands and could induce more rapid movement of contaminants into the piezometric sink from subcrop areas.

E. SLURRY TRENCH CUTOFF WALL.

1. Criteria.

a. Develop a 3,800 foot eastern extension of the existing slurry cutoff wall to provide a near-impermeable barrier which will prevent ground water migration through the alluvial aquifer.

b. Develop a 1,400 foot southwest extension of the existing slurry cutoff wall to prevent ground water flow around the west end of the pilot barrier.

c. The trench for the extended cutoff walls will be excavated through the alluvial aquifer and into the Denver Formation (bedrock) to a depth where foundation materials are considered relatively impermeable.

d. The alluvial soils within the trench alinement consist of a mixture of clays, silts, sands, and gravels and vary in thickness from 13 feet to 20 feet in the southwest extension and from 12 feet to 28 feet in the eastern extension. Within the trench alinement the water table is generally located 2 to 10 feet below the existing ground surface.

e. In order to evaluate various techniques of the slurry trench cutoff barrier concept and to develop general information on slurry trench specifications, the following tasks were carried out:

- A review of published literature.
- Discussions with various contractors and engineers knowledgeable in the field.
- Collection, review and evaluation of a number of specifications on the slurry trench method.

f. A field exploration program including laboratory testing was performed to provide supplementary design data. This program included 30 boreholes along the trench alinement, sieve analyses of soils, and unconfined compression tests of rock cores. A geologic section along the alinement is shown on Figure VI-19 and pertinent design data are shown on design drawings.

g. For this project a soil-bentonite slurry trench was selected to contain ground water flow under the anticipated gradients imposed by operation of the discharge and recharge wells. This procedure is probably the most common method which has been used in the past to cut off or slow down flow of water or other liquids through the ground. Calculations (appended) indicate that flow through the cutoff wall will total less than 0.1 gpm to the north. Model simulations indicate that the hydraulic gradient through the cutoff wall will slope to the north.

h. There are several types of bentonites available for use in slurry trenches. Basically, however, all of them have the property that in the presence of water they swell substantially by absorption of water molecules into the face of the montmorillonite clay platelets which largely make up the bentonite. By properly mixing the bentonite with water, a viscous slurry is formed which is mobile and has a very low permeability. With the slurry placed in a trench, it exerts a lateral fluid pressure on the sides of the trench and serves to stablize the opening which might otherwise collapse. The basic characteristics desired of the slurry are:

- It must have enough density and viscosity to support the trench walls without excessive slurry loss into the soil.
- It should be mobile enough to be displaced when necessary, by a slurry-backfill mixture and to fill voids in the trench walls.
- It should have a very low permeability.
- It should be stable and durable and not flocculate out of solution.

The specifications for the slurry trench cutoff barrier were written with the objective of economically and safely achieving these characteristics.

i. The soil that is mixed into the backfill is most often specified as to grain size and allowable soil types. The addition of the soil has the purpose of reducing the amount of bentonite used, adds body to the slurry and reduces the trench compressibility. It is generally agreed that to achieve these objectives, soil types with either excessive silt or clay content or organic content should not be used. Also, to maintain impermeability, high percentages of gravels or larger particles should not be allowed.

j. Detailed limits as to gradation of the sand fraction were not specified since it is obviously economically desirable to use the soil excavated from the trench as backfill. However, the maximum allowable particle size was set as 1.5 inches and the percent passing the No. 200 sieve was set at a maximum of 25 percent to insure that the permeability and compressibility of the trench were satisfactory.

k. The depth of the cutoff wall is probably greater than necessary. Denver Formation cores indicated considerable fracturing in the upper weathered portion. However, calculations indicate that even with a fracture permeability of 20 feet per year, the upper 20 feet of Denver Shales would only transmit about 0.3 gpm under existing gradients. Therefore, the cutoff wall design is very conservative.

F. <u>NORTH BOUNDARY MONITORING SYSTEM</u>.

1. <u>Criteria</u>.

a. Monitoring of water quality and water levels is required to prove the effectiveness of the North Boundary containment system and to provide data for dewatering, treatment, and recharge operations. Monitoring is required both on post in the North Boundary area and off post to the north in Sections 13 and 14.

b. Periodic water quality data are needed in both the alluvial aquifer and in the Denver Sands to determine the extent of contamination and concentration levels flowing to the containment system from upgradient. Similarly, these data are needed downgradient to establish time-concentration trends with barrier system operation. Detailed water quality data is needed at the alluvial dewatering wells to divert intercepted flow to the appropriate treatment nodules.

c. Periodic water level data are needed to demonstrate the degree to which the natural flow regime is disturbed by barrier operation both upstream and downstream of the barrier. Detailed water level monitoring is needed near the cutoff wall to control pumping rates and drawdowns of the alluvial dewatering wells, to prevent flooding over the cutoff wall, and to balance pumping with natural flows within design limitations. Dewatering wells must pump an estimated 110 percent of natural flows until near steady state is reached (up to 4-1/2 years). Gradually pumping rates can then be reduced to close to natural flows. However, the First Creek bog area is very sensitive to flooding because of the high water table and there is little margin for error. One positive factor, however, is that the flood prone area has the best quality water, and flooding in this area would not be as serious as in other zones of the flow system.

d. Water quality in the Denver Sands needs to be monitored to better define the depth, extent and levels of contamination with time. There is a downward component of flow from the suspected contaminant sources on the arsenal into the Denver Formation. Ground water velocities are generally much slower in the Denver Sands because of lower hydraulic conductivities. Therefore, it is possible that high contaminant concentrations have not reached the North Boundary. Also, the full depth of contaminant flow or potential flow is unknown at the barrier location. Sampling and testing has only been performed to depths of about 100 feet, and it is unknown if contaminants exist at greater depths. Full understanding of Denver Sand contamination is a regional problem and a monitoring system should be developed that incorporates the whole arsenal flow system which is beyond the scope of this study.

2. Analysis.

a. The North Boundary monitoring system should not be treated as part of the barrier system design except for the parts of the monitoring system needed for operation. The monitoring wells needed for system operation are located in close proximity to the alluvial dewatering wells, the cutoff wall, and the recharge wells. Distant monitoring wells should not be included in the barrier design because: (1) information is not available for definitive well design; (2) well construction will have to be combined with exploration; and (3) location of off post wells will have to be coordinated with land access (perhaps dictated by access). Also, uncertainties exist as to how many existing wells can be used and how many will be destroyed by construction.

b. Typical designs were developed for shallow and deep monitoring wells. The shallow wells are designed to monitor the alluvial aquifer or the first Denver Sand encountered if overlying alluvium is unsaturated or absent. The deep monitoring wells are designed to monitor sand layers overlain by another aquifer (either Denver Sands and/or the alluvial aquifer) and sealed against cross contamination from an overlying aquifer(s).

c. Shallow monitoring wells will be constructed using a 12-inch diameter well bore and 4-inch PVC casing with a gravel envelope. The 4-inch casing will be perforated with milled slots. The upper 5 feet of the annular space will be filled with cement to form a seal against infiltration of surface water. This type of well is shown on the design drawing and described in the specifications.

d. Deep monitoring wells will be constructed using a 12-inch diameter well bore close to the top of the sand zone to be monitored. A 6-inch diameter conductor casing will be installed and grouted in place with cement using a mud displacement method. A nominal 6-inch diameter hole will be air-rotary drilled through the sand zone and 4-inch PVC casing with milled slots will be installed. A gravel envelope will not be used in most cases because the Denver Sands are dense enough to prevent sanding in general. If loose sand is encountered a coarse sand envelope can be installed. This typical deep well design is shown on design drawings and described on the specifications.

G. FIELD EXPLORATION SUMMARY. Field work for the project commenced January 3, 1980 and was completed March 23, 1980. A total of 48 holes were drilled (Numbers 1000 through 1047) to depths ranging from 20.5 feet to 80.0 feet. Thirty holes were located along or adjacent to the proposed barrier alinement; 18 holes were located in the vicinity of the recharge and discharge well alinement. A total of 19 of the holes were completed as wells. Sieve analyses were run on 62 samples within the alluvium. Between one and four drill rigs were operating on the site, 5 to 7 days per week. Drilling companies used for the project were Custom Auger Drilling and Virginia Drilling, both of Denver, Colorado.

Terminology used and qualitative as well as quantitative sample assessment methods are described in ESA's Field Exploration Manual. It is important to note that certain terms used by ESA field personnel may differ slightly from the U.S. Army Corps of Engineers usage of the same term.

The 30 exploration holes drilled along or adjacent to the proposed barrier alinement included Numbers 1000 through 1029. East of the existing pilot barrier, depths ranged from 65.5 feet to 75.2 and 49.9 feet to 80.8 feet along and to the west of the existing pilot barrier. These holes were drilled utilizing the following procedure. A 6-inch flight auger was used to auger through the alluvium and standard split-spoon samples were driven approximately every 5 feet. Five and one-half inch, temporary steel casing was then placed within the alluvium and partially into the weathered Denver Formation. The Denver Formation was cored continuously with PQ-3 wireline coring equipment. Diamond bits and three different types of carbide bits were used. The holes were geophysically logged by Colorado Well Logging of Golden, Colorado. Spontaneous Potential, resistivity, gamma, gamma-gamma, neutron, and caliper logs were run on each hole. Twenty-five holes were backfilled with a 50-50 slurry mixture of bentonite and cement. Five holes (Nos. 1024, 1021, 1019, 1018, and 1017) were completed with isolated well screens utilizing a bentonite seal at the bottom, a filter pack of pea gravel around the screened interval, a bentonite seal above the screened interval, and a 50-50 slurry mixture of bentonite and cement to the surface. The temporary steel casing was removed from all of the holes.

The 18 holes located in the vicinity of the recharge/discharge well alinement (Nos. 1030 through 1047) were drilled using 6-inch flight or hollow stem augers, or 5-inch, 8-inch, or 11-3/4-inch tricone bits. Depths of the holes ranged from 20.5 feet to 67.0 feet. Standard split-spoon samples were driven approximately every 5 feet in Holes 1030, 1031, 1033, 1034, 1035, 1037, 1038, 1039, and 1040. Four holes, 1032, 1036 (alluvium), 1041, and 1045 (Denver Sand), were completed as wells. Steel casing and screen 6-inches in diameter was installed in Wells 1032 and 1036, and a gravel envelope was used around the screen. Wells 1041 and 1045 in the Denver Sand were completed using 4-inch slotted PVC and a thin gravel envelope. A conductor casing was cemented into the alluvium above the screened zone. Pump tests were run on these holes for up to 5 days. Nine holes, 1030, 1031, 1033, 1034, 1042, 1043, 1044, 1046, and 1047, were completed as observation wells using 2-inch slotted PVC pipe. The 5 holes not completed as wells, 1035, 1037, 1038, 1039, and 1040, were backfilled with a 50-50 slurry mix of bentonite and cement.

CHAPTER VII

ROADS, DRIVE, PARKING AREA, AND DRAINAGE

A. PERIMETER AND ACCESS ROADS AND BUILDING 808 ACCESS DRIVE.

Perimeter, access roads, and Building 808 access drive were designed for one-way traffic with 12-foot wide aggregate surface pavement and 4-foot wide aggregate paved shoulders. The pavement design was based on design class F, Category I, and the total compacted aggregate thickness is 6 inches. A subgrade CBR of 5 was used.

B. D STREET EXTENSION.

D Street extension was designed for one-way traffic with 16-foot wide aggregate surface pavement and 4-foot wide aggregate paved shoulders. The pavement design was based on design class F, Category I, and the total compacted aggregate thickness is 6 inches. A subgrade CBR of 5 was used.

C. PARKING AREA.

The parking area (expanded Building 808 access drive) was designed with aggregate surface pavement based on design class F, Category I, with a total compacted aggregate thickness of 6 inches. A subgrade CBR of 5 was used.

D. DRAINAGE.

Drainage shall be accomplished with stream channels, ditches, and culverts located appropriately. The levee containing First Creek flood flows was designed for a 10 year return interval event with 2 to 2-1/2 feet of freeboard. The levee will protect against the 100 year flood event with less than 1 foot of freeboard. The design storm for drainage culverts was the 10 year return interval event.

CHAPTER VIII

DIVISION 1 - GENERAL REQUIREMENTS

1A Special Provisions

1B Warranty of Construction (to be provided by COE)

1C Environment Protection

1D Special Safety Requirements

DIVISION 2 - SITE WORK

2A Removal and Disposition of Materials and Equipment

2B Excavation, Filling, and Backfilling for Buildings

2C Excavation, Trenching, and Backfilling for Utilities Systems

2D Excavation and Backfilling Working Surface, Slurry Trench

2E Clearing and Grubbing for Roads and Structures

2F Grading

2G Gravel Surfacing

2H Seeding

2J Storm-Drainage System

DIVISION 3 - CONCRETE

3A Concrete (For Building Construction)

DIVISION 4 - NOT USED

DIVISION 5 - METALS, STRUCTURAL AND MISCELLANEOUS

5A Miscellaneous Metal

DIVISION 6 - NOT USED

DIVISION 7 - NOT USED

DIVISION 8 - NOT USED

DIVISION 9 - FINISHES

9A Painting, General

9B Decorating Schedule (Interior Design Schedule)

DIVISION 10 - SPECIALTIES

10A Slurry Trench Ground Water Barrier

DIVISION 11 - NOT USED

DIVISION 12 - NOT USED

DIVISION 13 - SPECIAL CONSTRUCTION

13A Metal Buildings

13B Monitoring Wells - Alluvium

13C Denver Sand Monitoring Wells (DW 36 to DW 54)

13D Denver Sand Dewatering Wells

13E Dewater Wells

13F Recharge Wells

DIVISION 14 - NOT USED

DIVISION 15 - MECHANICAL

15A Gas Fitting

15B Pumps, Water, Centrifugal

15C Pumps, Water, Vertical Turbine

DIVISION 15 - MECHANICAL (Continued)

15D Waterlines

15E Heating Systems, Direct Gas-Fired Units

15F Pressure Vessels for Storage of Compressed Gases

15G Identification of Piping

DIVISION 16 - ELECTRICAL

16A Electrical Work, Interior

16B Electrical Work, Exterior

APPENDIX A

DEPARTMENT OF THE ARMY
OMAHA DISTRICT, CORPS OF ENGINEERS
6014 U.S. POST OFFICE AND COURTHOUSE
OMAHA, NEBRASKA 68102

MROED-MF DACA 45-79-C-0019 26 February 1980
(Rocky Mountain Arsenal, CO)

Black & Veatch
P.O. Box 8405
Kansas City, MO 64114

Gentlemen:

Reference your Contract No. DACA 45-79-C-0019 for the Design of Liquid Waste Disposal Facility, North Boundary Expansion, Project No. 34 at Rocky Mountain Arsenal, Colorado.

Inclosed is the foundation report for the Fluoride Building Site. This report is to be included in the final design analysis. The boring plan and logs are to be shown on the design documents.

If there are any questions concerning the above, please contact this office.

Sincerely,

O. H. Asleson

O. H. ASLESON
Chief, Military Branch
Authorized Representative of
the Contracting Officer

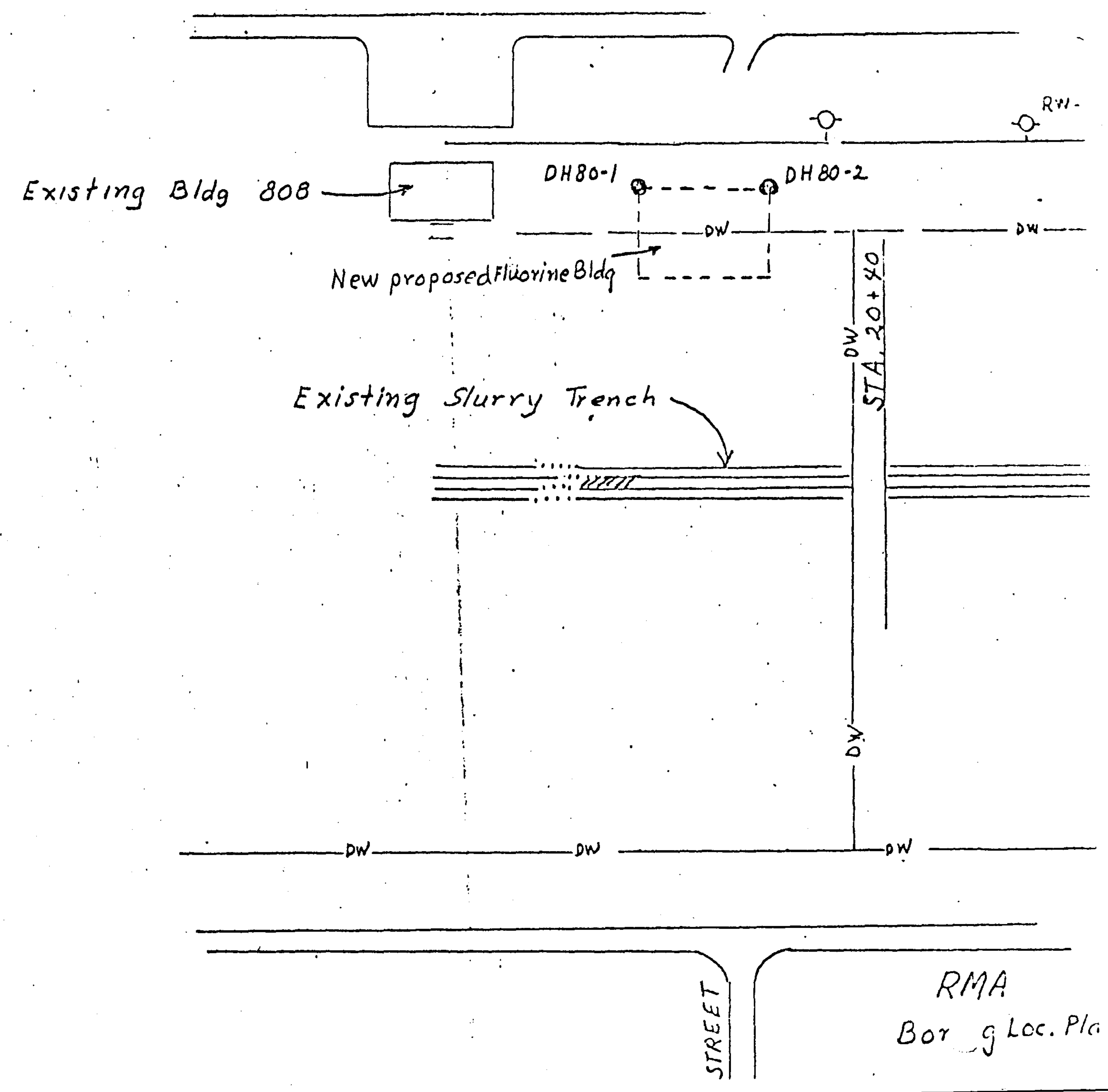
1 Incl
As stated

FN'D. REP'T. FOR THE PROPOSED FLUORIDE BLDG. SITE AT RMA, COLO.

Subsurface Investigation. On 14 January 1980, by phone request from the design A-E, this office was requested to make two borings for the proposed Fluoride Bldg. and two borings for a proposed bridge slab crossing over the slurry trench. Our geologist, Mr. Zeltinger, was at RMA the 15th of January. and arranged for a drill crew to make two borings DH 80-1 and 80-2 for the proposed Fluoride building site. The other two hole locations requested for the slurry trench crossing on the extended centerline of "D" street, because of the open ditch at this area, was too muddy to get drilling equipment on to the locations requested. It was later decided that a compacted fill across this ditch area and over the slurry trench would eliminate the need of borings for the slab bridge and would provide an adequate road bed. The two borings for the building were drilled to depths of 16.5 feet. Field standard penetration tests were made in both borings to determine approximate in-place density of the foundation soils. Representative disturbed samples of the types of materials encountered were obtained from the split spoon samples of the penetration tests.

Laboratory and Field Data. The disturbed samples taken from the borings were visually classified in the laboratory and natural moisture content determinations were made. The upper 6 feet of both borings show the foundation soils to be sandy clay (CL) described as being stiff, moist brown and slightly calcareous. The upper 2 feet of boring 80-1 could possibly be fill material. Below the surface clay the material becomes clayey sand (SC) then changing to fine sand (SP) near the bottom of the borings. The clayey sands are described as being medium dense, moist and brown. Near the bottom of both borings the sand becomes very moist to wet and ground water level is estimated to be about 16 feet below the ground surface. The natural moisture contents of the upper clay ranged from 7.9 to 13.5% and in the clayey sands and sands from 10 to 21%. The standard penetration blow counts in the clays range from 4 to 19 with the low blow counts being near the bottom of the clay layer and just below proposed footing depths.

Adopted Design Data. It is understood the proposed Fluoride Building is to be metal frame, pre-engineered flexible type structure having a concrete floor slab on grade. It will be rectangular shaped about 60 by 90 feet in plan dimension. Footings for the structure will be spread and continuous type bearing a minimum of 3.5 feet below grade for protection from frost. Based on the low standard penetration blow counts encountered below footing depths, it is recommended an allowable excess soil bearing pressure of 1400 psf be used for design. This would minimize any possible settlements. However, if a higher allowable bearing is required, as an alternate, over excavation for 3 or 4 feet below footings to remove any softer clays and replacement with a recompacted fill could be required and a value of 2500 to 3000 psf could be used for design.


RW-
Existing Bldg 808
DH80-1
DH80-2
DW
New proposed Fluorine Bldg
STA. 20+40
Existing Slurry Trench
DW
STREET
RMA
Bor g Loc. Pla

RMA

Fluorine Bldg. Borings

DH 80-1

El. 5152.5±

15 Jan 80

	N	M		
5155				
				4" Gravel
	19	13.5	CL	SANDY CLAY (Fill) moist, Brown Slightly Calcareous
5150	10		CL	SANDY CLAY, Med Stiff moist, Brown Sl. Calc.
	10			
5145	8	13.9	SC	Very CLAYEY SAND Med Dense, moist, Brown Calcareous
	3			Sandier with depth
5140	10			
	20	11.7 ∇ ?	SP	SAND, Brown Med Dense, Very moist to Wet Poorly graded
5135				

Hole Stayed Open to 16'

DH 80-2

El. 5152.5±

15 Jan 80

N	M		
18	7.9	CL	SANDY CLAY, Stiff moist, Brown slightly Calcareous
9			
4			
6	10.1	SC	CLAYEY SAND Med Dense moist Brown, Sl. Calc.
11			
15			
18	21.0 ∇ ?	SP	SAND, Med Dense Moist, Brown Poorly graded Wet @ 16.5'

LEGEND

El. Elev. at top of hole, taken from Topo.
M Natural moisture content in per cent
N Standard penetration, blows per foot required to drive a 2 inch split Spoon sampier by a 140 lb. wt. Dropping 30 inches.
∇ : Estimated Water Level, Sand becomes wet.

APPENDIX B

REFERENCES FOR CHAPTER VI

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