Shallow Ground Water in the Whitney Area, Southeastern Las Vegas Valley, Clark County, Nevada

Part II. Assessment of a Proposed Strategy to Reduce the Contribution of Salts to Las Vegas Wash

By Thomas J. Burbey

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BRUCE BABBITT, *Secretary*

U.S. GEOLOGICAL SURVEY

Dallas L. Peck, *Director*

For additional information write to:

District Chief
U.S. Geological Survey
333 W. Nye Lane, Room 203
Carson City, NV 89706-0866

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CONVERSION FACTORS AND VERTICAL DATUM

Multiply	*By*	*To obtain*
acre-foot (acre-ft)	0.001233	cubic hectometer
cubic foot per second (ft^3/s)	0.02832	cubic meter per second
cubic foot per day (ft^3/d)	0.02832	cubic meter per day
foot (ft)	0.3048	meter
foot per day (ft/d)	0.3048	meter per day
inch (in.)	25.40	millimeter
mile (mi)	1.609	kilometer
square foot (ft^2)	0.09290	square meter
square mile (mi^2)	2.590	square kilometer
ton per year (ton/yr)	0.9072	metric ton per year

SEA LEVEL

In this report, "sea level" refers to the National Geodetic Vertical Datum of 1929 (NGVD of 1929, formerly called "Sea-Level Datum of 1929"), which is derived from a general adjustment of the first-order leveling networks of both the United States and Canada.

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ABSTRACT

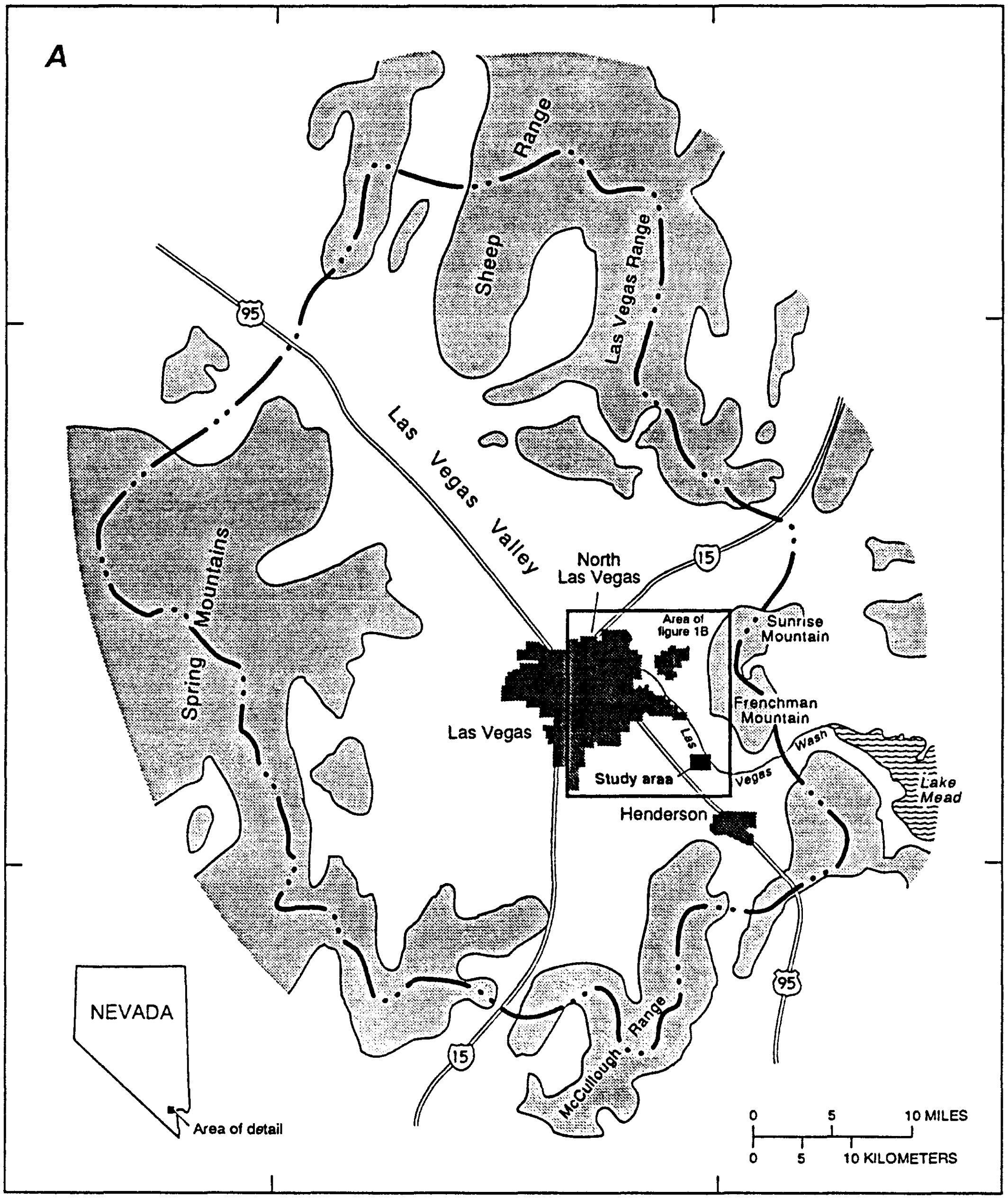
A continuing increase in the contribution of dissolved solids to Las Vegas Wash from various sources, including ground-water seepage, has resulted in the proposal of several salt-reduction strategies. One such strategy is the construction of an impoundment, or detention basin, consisting of a dike structure above ground and a slurry wall beneath the dike that would extend to the base of the shallow alluvial aquifer. The dike-and-wall system was expected to function by damming deeper, more saline flow in the shallow aquifer, while allowing only shallow, less saline flow to leave the detention basin. Theoretically, the shallow flow would become increasingly fresh over time, because it would no longer come in contact with the Muddy Creek Formation at the base of the shallow aquifer, which heretofore has been considered the principal source of salts in the aquifer.

This report describes the results of a study made to determine whether the proposed slurry wall could be effective in inhibiting saline ground water from seeping to the wash under various possible geohydrologic conditions. Field data were collected and analyzed and fluid- and solute-transport models were developed to test the effectiveness of the proposed strategy. X-ray analyses indicate that salt-bearing minerals, particularly gypsum, are relatively abundant in the alluvial deposits that make up the aquifer, and are largely responsible for the high dissolved-solids concentrations.

Three conceptual and mathematical models were developed to analyze the effects of the proposed slurry wall. Simulations of ground-water flow and solute transport were made using a finite-element model. Simulation indicates that most ground-water flow is through the center part of the 30-foot-thick aquifer (between depths of 10 and 25 feet) and therefore does not come in contact with the underlying Muddy Creek Formation; yet, the ground water is high in dissolved-solids concentration. In areas dominated by phreatophytes, simulations have shown that evapotranspiration is the major process influencing the distribution of dissolved solids. In areas dominated by hydrophytes and affected by effluent from upgradient sewage-treatment plants, mixing of the more dilute effluent with the more saline shallow ground water is the major influencing process. Simulation results also indicate that the slurry wall would not induce upward flow in the shallow aquifer farther upgradient than under present conditions. Furthermore, the quantity of flow at the base of the aquifer behind the slurry wall would not be reduced; rather, the flow would be deflected upward immediately adjacent to the wall. Results indicate that the aquifer is too thin for the proposed slurry wall to function as planned. Simulation indicates that much better results would be obtained with the slurry wall if the aquifer was one order of magnitude thicker. Thus, the study indicates that the proposed slurry wall would not decrease the dissolved-solids load entering Las Vegas Wash from the existing shallow aquifer.

INTRODUCTION

The study area, or Whitney area as it is referred to herein, is situated in the southeast part of Las Vegas Valley in Clark County, southern Nevada (fig. 1A). The Whitney area received its name from the U.S. Bureau of Reclamation's Whitney Verification Program, which in turn is named after the small community of Whitney on the outskirts of southeastern Las Vegas. The 1-mi^2 study area (fig. 1B) represents only a small part of the overall valley, which encompasses about 1,540 mi^2 (fig. 1A).



EXPLANATION

MOUNTAINOUS AREA

URBAN AREA

HYDROGRAPHIC-BASIN BOUNDARY

FIGURE 1.--Location of study area, sewage-treatment facilities, major washes, and other pertinent geographic features.

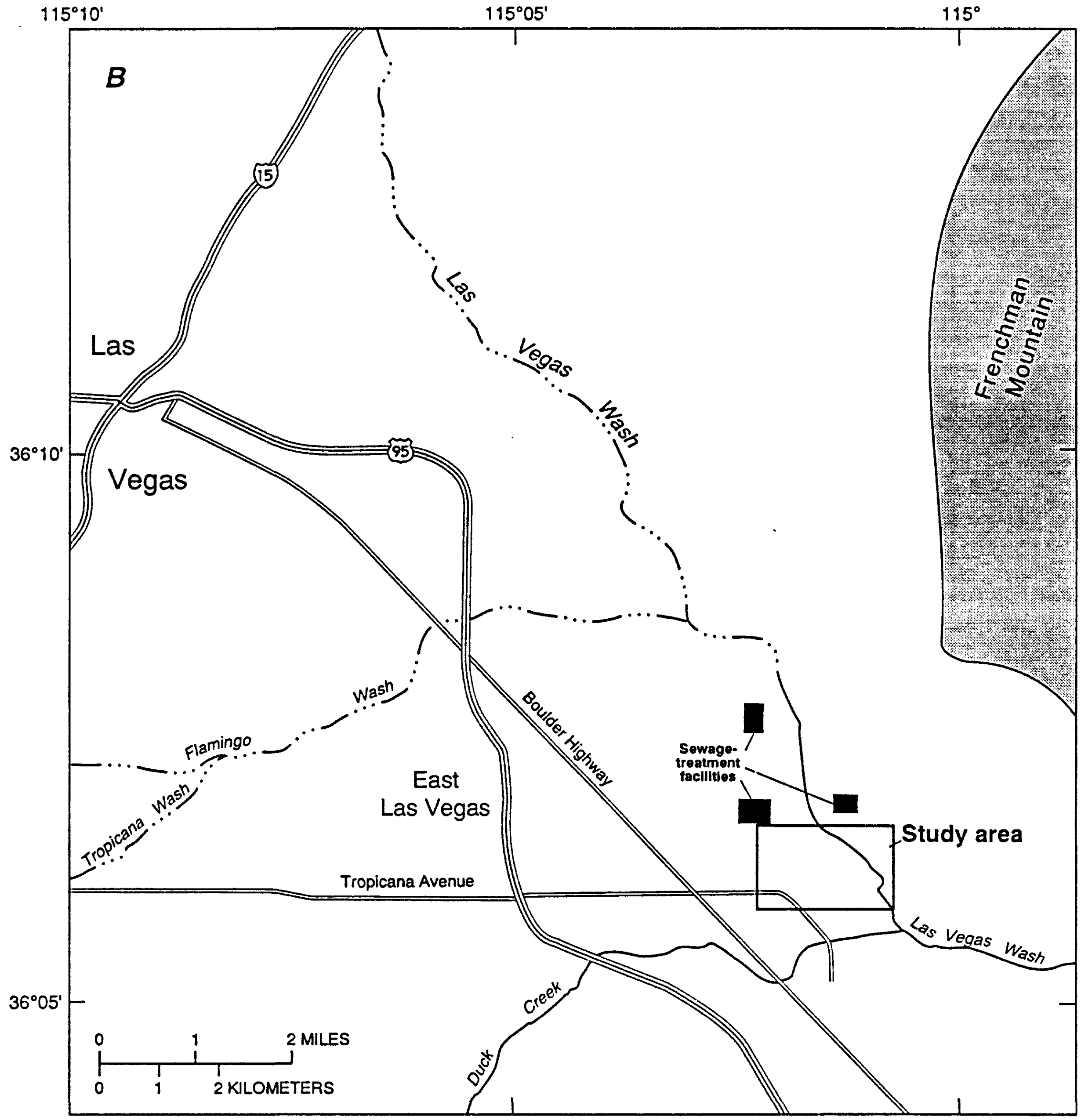


FIGURE 1.--Continued.

Background

Both natural and human activities have had an effect on ground-water flow and evapotranspiration in the vicinity of the Whitney area. Naturally occurring salts in the shallow aquifer system, coupled with rapid population growth and the resulting increases in water use, have caused the need for salt-reduction strategies. These strategies are aimed at decreasing the dissolved-solids content of ground water entering Las Vegas Wash (fig. 1B), which flows in turn to Lake Mead. The demand for water in Las Vegas Valley has increased from that obtained from a few domestic wells in the early part of the century to an annual use of more than 200,000 acre-ft (about two-thirds of which is imported from Lake Mead). Pumping of ground water has led to significant water-level declines in the central part of the valley (Terry Katzer, Las Vegas Valley Water District, oral commun., 1986).

In contrast to the continued ground-water level declines in the central part of the valley, levels have been rising slightly in the southeast part (Harrill, 1976, p. 23; Morgan and Dettinger, in press). Increased flow through the salt-laden shallow alluvial deposits, coupled with industrial discharge and large volumes of treated effluent, is responsible for the increased dissolved-solids load entering Las Vegas Wash. Intense irrigation of lawns and golf courses, urban runoff, and discharge from sewage-treatment facilities (Harrill, 1976, p. 26) have turned Las Vegas Wash from an ephemeral stream to a perennial stream with discharge averaging 110 ft^3/s (Frisbie and others, 1985, p. 58). During storms, flows in the wash can exceed 1,000 ft^3/s; a peak discharge of about 6,500 ft^3/s has been estimated just east of the Whitney area (Patrick A. Glancy, U.S. Geological Survey, oral commun., 1987). Figure 2 shows the relation between population growth in Las Vegas Valley and the total outflow to Las Vegas Wash. In this report, Las Vegas Wash refers specifically to the wasteway channel excavated in 1983 that constrained the effluent discharge from sewage-treatment facilities. Prior to 1983, treated effluent flowed in small channels throughout the densely vegetated flood plain that historically has also been referred to as Las Vegas Wash.

Erosion and headcutting due to increased discharges to the wash, as well as to runoff from major storms, have lowered the level of the channel by about 15 ft in the eastern part of the Whitney area. Ground-water levels have declined near the wash where the channel has been lowered. Vegetation types have changed in response to the declining water levels. Hydrophytic vegetation (swamp and marsh types, such as reeds and cattails) dominates areas where the water levels are within 2 ft of land surface (fig. 3), whereas phreatophytes (mainly saltgrass and salt cedar) generally dominate elsewhere. As headcutting continues to lower water levels in the vicinity of the wash, hydrophyte-dominated vegetation tends to die off and re-establish itself in areas farther upgradient where water levels remain shallow.

The increased dissolved-solids load entering Las Vegas Wash as a result of increased discharge has led to the development, by the U.S. Bureau of Reclamation, of several alternative strategies for reducing the dissolved-solids load that seeps into the wash by way of the shallow ground-water inflow. The salinity-control effort is part of the project authorized by the Colorado River Basin Salinity Control Act of 1974 (Public Law 93-320). Under Title II of the Act, a provision for a program to control the salinity of the Colorado River upstream from Imperial Dam was implemented in response to the Federal Water Pollution Control Act (Public Law 92-500). One such strategy involves constructing a series of detention basins adjacent to Las Vegas Wash. These basins would consist of a surface impoundment, or dike, overlying a vertical slurry wall that would penetrate the entire thickness of the aquifer. The detention basins were intended to reduce the dissolved-solids content of ground water seeping into the wash by impounding deeper, more saline water. As ground water from upgradient areas flowed into the detention basin, theoretically, it would become increasingly fresher because it would no longer be in contact with the Muddy Creek Formation at the base of the aquifer. The Muddy Creek Formation is known to contain significant amounts of gypsum and other soluble salts in some areas (Bohannon, 1984, p. 56). Thus, over time, the saline, more dense ground water near the bottom of the aquifer theoretically would become a stagnant pool, while fresher, less dense water would flow in the upper part of the aquifer above the more saline water. The fresher water would be allowed to leave the detention-basin area through a lined channel near the top of the slurry wall into an adjacent, down-gradient detention basin, where this process would be repeated.

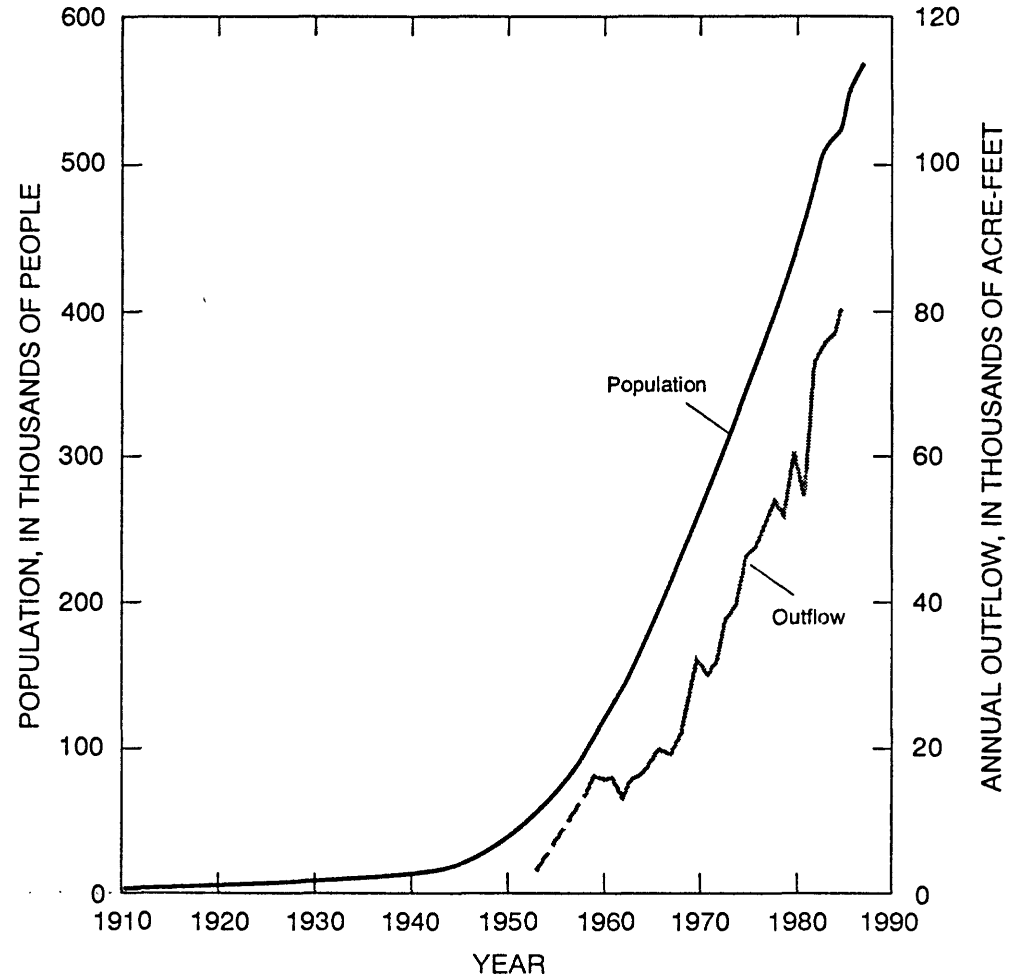


FIGURE 2.--Relation between population growth in Las Vegas Valley and surface-water outflow to Las Vegas Wash (from Patrick A. Glancy, U.S. Geological Survey, written commun., 1987). Dashed where approximate.

The cost of constructing the series of dikes and slurry walls would be considerable; hence, the U.S. Bureau of Reclamation selected the D-14 detention basin (fig. 3) as a test site to determine whether the strategy would be effective in reducing the dissolved-solids input to Las Vegas Wash. The site was chosen because as much as 60 percent of the shallow ground-water flow seeping into the wash was thought to be entering through the Whitney area (D. Art Tuma, U.S. Bureau of Reclamation, oral commun., 1986). Selection of an area with the greatest quantity of flow would provide an accurate test of the effectiveness of the detention basin as a salt-reduction strategy.

In the fall of 1986, the D-14 dike was constructed. It is composed of compacted alluvial-fan deposits. The level top of the dike reaches a maximum height of about 6 ft above land surface near the southeast corner. The top of the dike is level, so the elevation difference between the top of the dike and land surface becomes less along the western legs as the land surface rises to the west. An inlet structure on the north side of the dike (fig. 3) allows surface water (from the Monson Road floodway) and treated sewage effluent to enter the detention basin. The Monson Road floodway is an unlined channel carrying urban runoff and shallow ground-water seepage from the upgradient flood plain of the wash to the study area. At the south

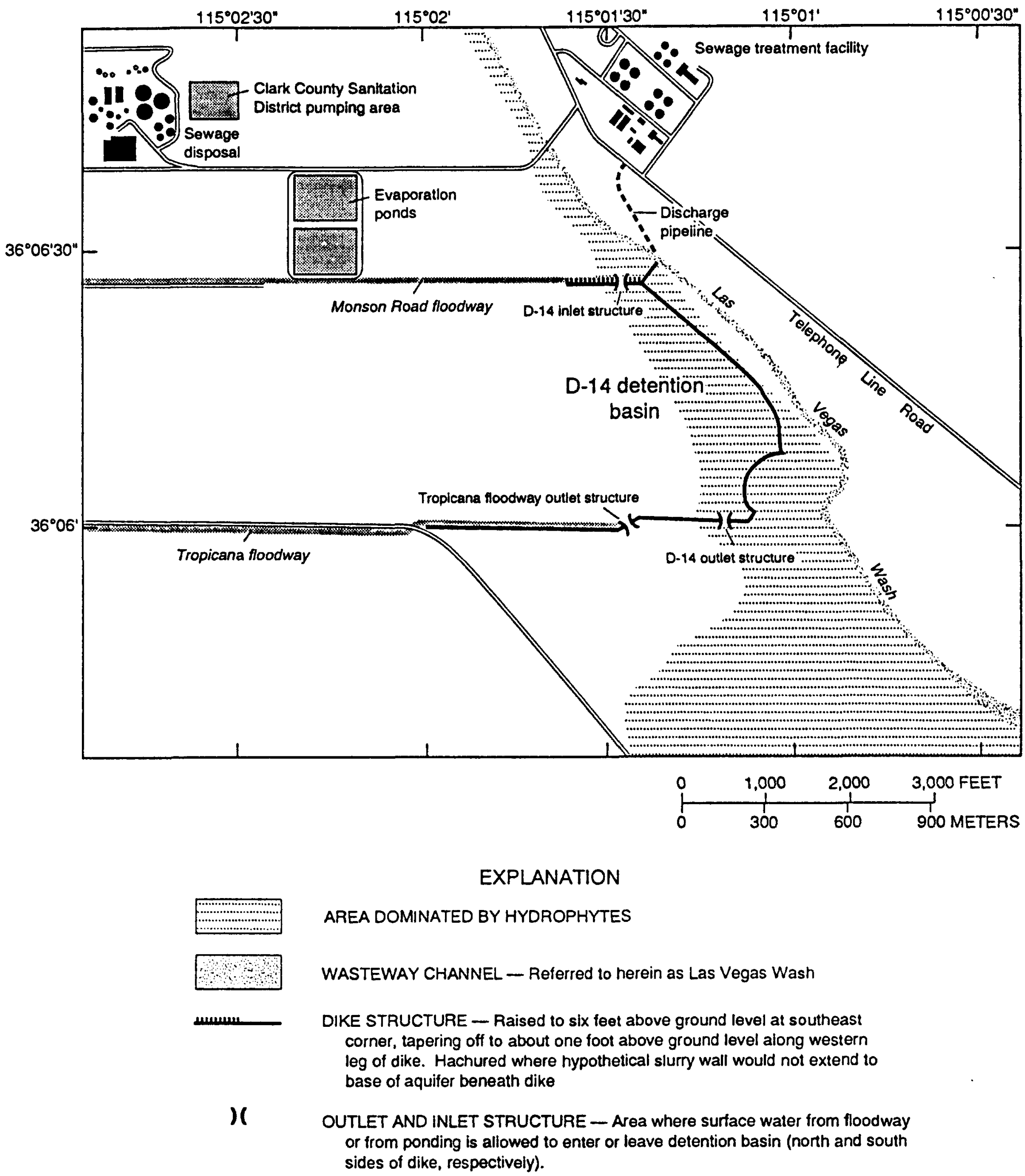


FIGURE 3.--Location of D-14 detention basin with respect to Las Vegas Wash, floodways, areas dominated by hydrophytes, and other geographic features.

end of the dike, two outlet structures allow surface water to leave the detention basin. The Tropicana floodway also enters the detention basin, but that inflow leaves the basin through an outlet structure farther to the east (fig. 3). The planned slurry wall would not be constructed unless simulation results suggest that the detention-basin strategy would work as proposed. If constructed, the slurry wall would extend vertically beneath the dike to the base of the aquifer except along the leg adjacent to the Monson Road floodway (shown with hachures on fig. 3). The detention basin would then act as a catchment for ground water that flows in a southeastward direction (the ground-water level nearly parallels the land surface).

The first phase of the study is described by Emme and Prudic (1991). Its purpose was to evaluate the areal, vertical, and seasonal variations in ground-water quality, and to describe the processes and factors controlling this quality. The effort involved (1) drilling numerous test holes and installing observation wells, which was done principally by the U.S. Bureau of Reclamation, (2) collecting lithologic samples for mineral-composition analysis, (3) making water-level measurements at the test wells, and (4) collecting water-quality samples for chemical analysis.

Purpose and Scope

This study, done by the U.S. Geological Survey in cooperation with the U.S. Bureau of Reclamation, was made to evaluate the effectiveness of the proposed detention-basin strategy in reducing the tonnage of salts entering Las Vegas Wash from the shallow ground-water system in the Whitney area. The study, which took place from spring 1986 to summer 1987, was divided into two phases--a chemical-analysis and synthesis phase and a solute-transport modeling phase.

The second phase of the study is described in this report. Its purpose is to develop both planimetric and cross-sectional solute-transport models to examine the effectiveness of the proposed detention basin. These models use both hypothetical and estimated aquifer and solute properties and boundary conditions--determined during both phases of this study--to evaluate whether the dike and proposed slurry wall could reduce solute loads entering Las Vegas Wash. Various aquifer properties were tested to determine the optimum conditions under which the proposed strategy would be most effective. The scope of this phase of the study involved (1) making aquifer tests at selected test wells to determine the hydraulic conductivity of the shallow alluvial aquifer, (2) collecting soil cores for analysis of total porosity and particle size, (3) making measurements of flow in floodways and from nearby pumping wells to estimate ground-water and surface-water inflow to the area, and (4) mapping phreatophyte and hydrophyte areas to delineate the areal distribution of evapotranspiration.

Acknowledgments

Appreciation is expressed to D. Art Tuma of the U.S. Bureau of Reclamation, who provided maps, coordinated U.S. Bureau of Reclamation activities, and provided considerable insight regarding the detention-basin strategy. The author also would like to thank Clifford I. Voss of the U.S. Geological Survey for his insightful comments, suggestions, and efforts that have led to realistic and accurate conceptual simulations of the study area.

PHYSIOGRAPHY AND GEOLOGY

Las Vegas Valley is bounded by the Spring Mountains to the west, the Sheep and Las Vegas Ranges to the north, the Frenchman and Sunrise Mountains to the east, and the McCullough Range to the south (fig. 1). Piedmont surfaces, representing coalescing alluvial aprons, and pediments form the transition between the steep mountain blocks and the lowlands that make up Las Vegas Valley (Dinger, 1977; Bell, 1981).

The valley slopes gently to the east and southeast, toward the Whitney area, and is drained by Las Vegas Wash, Flamingo Wash, Tropicana Wash, and farther south, by Duck Creek (fig. 1B). Las Vegas Wash, which empties into Lake Mead, receives the flow from the other three drainages.

Las Vegas Valley is underlain by as much as 5,000 ft of clastic sediments of Miocene to Holocene age. These sediments fill a structural basin that may have resulted from slippage along the Las Vegas Shear Zone to the north (Bell, 1981, p. 13; Plume, 1984, p. 21) and along normal faults that bound the east and possibly the west sides of the valley (Morgan and Dettinger, in press). The Muddy Creek Formation, of Miocene age, probably includes most of the clastic deposits that fill the basin. The thickness of this formation is inferred from well logs to range from more than 3,000 ft near the Whitney area to less than 500 ft in the northeast part of the valley (Malmberg, 1965, p. 21). The Muddy Creek Formation is composed of generally coarse sediments toward the west side of the valley and becomes finer grained toward the east and southeast (Morgan and Dettinger, in press). On the west side of Las Vegas Valley, the formation is dominated by sand, with gravel and thick clay lenses. On the east and southeast sides of the valley, the formation grades into a silty sand with clay lenses and evaporite deposits (Longwell and others, 1965, p. 48; Plume, 1984, pl. 2). These sediments may have been deposited in a lacustrine environment (Bohannon, 1984, p. 56). Younger alluvium overlies the Muddy Creek Formation in most of the valley. These deposits, which are of Pleistocene and Holocene age, represent poorly sorted alluvial gravel and sand (Malmberg, 1965, p. 22), that may reach thicknesses of 1,000 ft in the central and western parts of the valley. In the eastern and southeastern parts of the valley, however, these deposits are thin or non-existent where the Muddy Creek Formation is exposed along flanks of Frenchman Mountain (Domenico and others, 1964, p. 10). These younger alluvial deposits make up the principal aquifers within the valley (Plume, 1984, pl. 2).

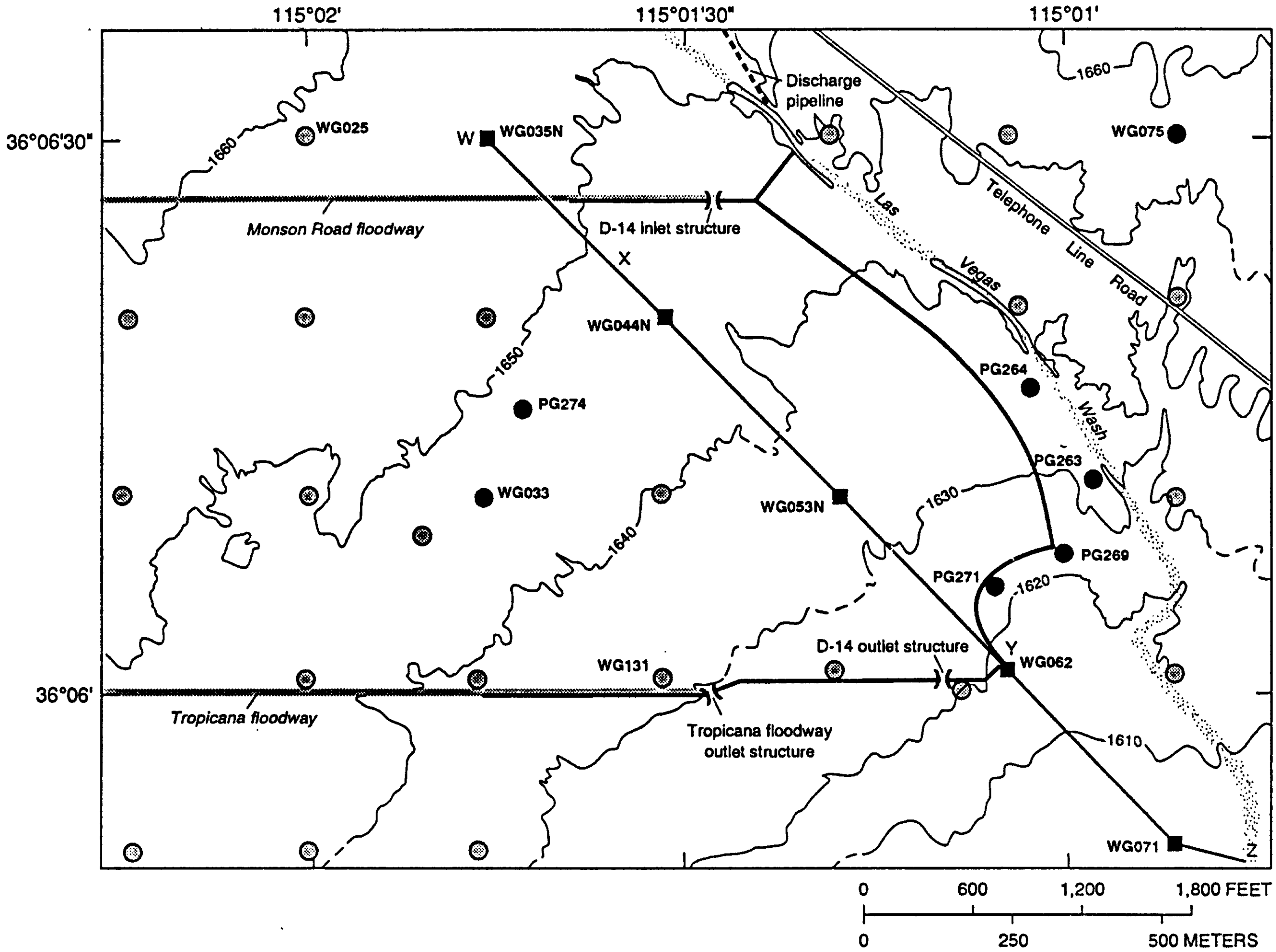
In the Whitney Area, the Muddy Creek Formation is mostly fine-grained and is overlain by an approximately 30-ft section of alluvial-apron and flood-plain deposits (see further discussion in subsequent section titled "Thickness"); the finer deposits are derived primarily from Las Vegas Valley to the west, whereas the coarser deposits (mainly gravel) probably originate in nearby Frenchman Mountain to the north. Drillers' logs for the Whitney area indicate that the Muddy Creek Formation is dominated by clay and silt and is relatively impermeable; however, the logs provide no evidence of evaporite deposits.

HYDROGEOLOGY

Water-level and water-quality data were collected from observation wells at 34 sites in the Whitney area (fig. 4). Several of the sites have two wells, and the cluster-well sites have as many as five wells, screened at discrete intervals vertically through the aquifer. The design, installation, and development of all observation wells in the Whitney area are discussed in detail in a companion report by Emme and Prudic (1991).

Properties of the Alluvial Deposits

An understanding of the hydrogeologic setting of the Whitney area and the physical and chemical processes that influence the nature and behavior of ground water flowing through the area are important to the understanding of how the dike and slurry wall might affect flow and transport in the shallow aquifer. Thus, key hydraulic and physical properties of both the alluvial sediments and the ground water were estimated.



EXPLANATION

DIKE STRUCTURE

1660 — — LAND-SURFACE CONTOUR — Dashed where approximately located. Contour interval 10 feet. Datum is sea level

W———Z LINE OF SECTION USED IN SUBSEQUENT FIGURES — Intermediate points X and Y are shown on map

WG131, PG271 OBSERVATION-WELL SITE — Shows location of wells used to collect water-level and water-quality data. Solid circles indicate well sites where hydrographs are provided in subsequent illustrations. Well sites labeled by name are referenced in text

WG053N CLUSTER-WELL SITE AND NAME — Shows location where each well in cluster is screened at different interval. Wells are used to collect water-level, water-quality, and slug-test data

FIGURE 4.--Location of observation wells.

Mineralogy and Particle Size

X-ray diffraction and particle-size analyses were made on sediment-core samples collected during drilling of cluster-well sites to provide information about the mineral assemblages present in the alluvial deposits and permeability of the shallow aquifer. The information was used to understand the possible origin of the minerals, and to relate the chemical character of the ground water to the minerals. Particle-size analyses were made to determine the spatial heterogeneities in the sediments, both vertically and areally. Particle-size data were used to estimate variations in permeability and, combined with the mineralogy, to determine the depositional history of the alluvial sediments.

X-ray diffraction data indicate that salt content, particularly gypsum, varies markedly both with depth and from site to site (Emme and Prudic, 1991). In general, near-surface samples have a considerably greater percentage of gypsum than do samples collected from greater depths. The greater percentage probably results from precipitation of salts in the shallow capillary fringe during summer months, due to evapotranspiration. A comprehensive description of the mineralogy at selected cluster-well sites is presented by Emme and Prudic (1991).

Analysis of particle-size data collected at cluster-well sites (table 1), using the Wentworth Classification System, indicates a wide range of sizes in most of the samples collected. Sizes range from medium gravel (coarse pebbles) to clay, with medium to fine sand and silt dominating. This size distribution is typical for alluvial-apron and flood-plain deposits (Morris and Johnson, 1967). Because of the diverse grain-size distribution, even qualitative relations between grain size and permeability are difficult to determine. However, certain trends in sediment grain size have been identified, both with depth and from well site to well site. For example, the coarser sediments are in the middle to deeper parts of the aquifer, whereas finer particles dominate the top and deepest parts of the aquifer (table 1).

Thickness

An abrupt change in lithology occurs at a depth of approximately 30 ft in the Whitney area. Drillers' logs indicate a sharp increase in drilling difficulty beneath the shallow alluvial deposits (at about 30 ft) at well sites WG035, WG044, WG131, and WG062 (fig. 4). Sediments at this depth were more elastic and difficult to penetrate with a hollow-stem auger. A sediment core from the depth interval of 32-33 ft at cluster-well site WG062 contained a large percentage of clay (table 1) and probably represented the top of the Muddy Creek Formation, a relatively thick, low-permeability unit in the lower part of the Valley (Bohannon, 1984; Smith, 1985; D. Art Tuma, U.S. Bureau of Reclamation, oral commun., 1986). Locally, the Muddy Creek Formation contains sand and gravel lenses, and may, as a result, be difficult to visually discern from the overlying alluvial deposits (Bohannon, 1984; Patrick A. Glancy, U.S. Geological Survey, oral commun., 1987). In the Whitney area, however, evidence suggests that the Muddy Creek Formation is dominated by fine-grained sediments and probably is relatively impermeable compared to the overlying alluvial deposits. On the basis of available information, the top of the formation probably ranges from 25 to 35 ft below land surface and thus an average thickness of 30 ft is indicated for the alluvial deposits.

Hydraulic Conductivity

Probably the most important property that influences the behavior of ground-water flow is hydraulic conductivity. If a porous medium is isotropic and the fluid is homogeneous, the hydraulic conductivity of the medium is the volume of water at the existing kinematic viscosity that will move in unit time under a unit hydraulic gradient through a unit area measured at right angles to the direction of flow (Lohman, 1972, p. 4). Estimates of hydraulic conductivity can vary widely depending on the technique used. Four general methods are commonly used to estimate hydraulic conductivity: (1) direct laboratory measurements including falling-head and constant-head permeameter tests on sediment core samples, (2) indirect laboratory methods based on particle-size or pore-structure distribution and consolidation tests on sediment cores, (3) slug tests at wells, and (4) aquifer tests at wells. The slug-test method, which provides an estimate of hydraulic conductivity near the well screen, was used in the Whitney study area.

TABLE 1.--*Particle-size data for sediment cores from cluster-well sites*

[Categories containing at least 20 percent of total sample are in bold print]

Site and depth interval (feet below land surface)	Percentage in each Wentworth particle-size category[1]							Median particle size (millimeters)	Overall classification
			Sand			Gravel			
	Clay	Silt	Fine	Medium	Coarse	Fine	Medium		
WG035N									
3-6	6	**83**	10	1	0	0	0	0.015	silt
6-9	12	**54**	**33**	1	0	0	0	.053	silt
12-23	**23**	**37**	6	5	13	9	7	.036	silt
26-30	13	**38**	**41**	4	2	2	0	.065	fine sand
WG053N									
5-8	4	**76**	17	3	0	0	0	0.034	silt
11-14	15	**27**	**40**	4	6	7	1	.10	fine sand
14-17	18	**44**	14	9	12	2	1	.028	silt
17-20	10	**25**	**26**	4	9	10	16	.12	fine sand
23-26	16	**26**	**39**	4	7	8	0	.10	fine sand
WG062N									
3-4	14	**61**	**20**	3	1	1	0	0.025	silt
5-6	4	**42**	**48**	3	3	0	0	.10	fine sand
11-12	4	**53**	17	6	11	8	1	.055	silt
15-16	3	**26**	**20**	7	12	**27**	5	.26	medium sand
22-23	6	13	**64**	5	6	6	0	.13	fine sand
32-33	**30**	**57**	10	2	1	0	0	.01	silt
WG071N									
8-9	17	**21**	5	14	17	19	7	0.38	medium sand

[1] Wentworth Classification System: clay, less than 0.004 millimeter; silt, 0.004 to 0.062 millimeter; fine sand, 0.062 to 0.25 millimeter; medium sand, 0.25 to 0.5 millimeter; coarse sand, 0.5 to 2 millimeters; fine gravel, 4 to 8 millimeters; and medium gravel, 8 to 16 millimeters.

Three slug-test procedures were used to estimate hydraulic conductivity at cluster wells in the study area. These techniques were applied to measurements of water level made after an instantaneous slug of water was injected into selected wells. The method described by Hvorslev (1951) assumes isotropic flow to a piezometer open only to a small interval of the aquifer. The aquifer is assumed to be homogeneous and infinite in extent in both the horizontal and vertical directions. The alluvial aquifer and water are assumed to be incompressible, and fluctuations in the water table due to the slug of water are assumed to be negligible also. The method of Bouwer and Rice (1976) was developed for unconfined aquifers and partially penetrating wells (well screened through only a part of the aquifer). Calculations are based on the Thiem equation for steady flow to a well, and assume a homogeneous and isotropic aquifer. The aquifer and water are assumed also to be incompressible. Finally, Cooper and others (1967) developed a slug-test method for fully penetrating wells in confined aquifers. Their method assumes radial flow to the well from a homogeneous, isotropic aquifer in which the aquifer and water are compressible.

Results of the slug-test analyses are presented in table 2. Differences in results are due to the differing assumptions inherent in each technique and the differing equations based on well design. Several assumptions had to be made to use the three techniques:

1. The method of Hvorslev (1951) was assumed to be applicable for screened intervals that are small in relation to aquifer thickness. This is a major assumption, as it decreases the effective radius used in the solution, jeopardizing the assumption of horizontal steady-state flow, which in turn may make estimated hydraulic conductivities too low.

2. The screened interval was used as the aquifer thickness in the method of Cooper and others (1967). This may introduce errors because vertical flow almost certainly occurs to some degree when a slug of water is introduced, probably resulting in higher estimated hydraulic conductivities than actually exist. The radius of the screened interval was considered to be equal to the radius of the well. This is acceptable if the sand pack is assumed to have a hydraulic conductivity on the same order as that of the aquifer.

3. No changes in storage were assumed to occur due to water-table fluctuations in any of the methods.

TABLE 2.--*Estimated horizontal and vertical hydraulic conductivities at cluster-well sites, from slug-test data*

Site and cluster-well designation	Depth interval (feet below land surface)	Estimated hydraulic conductivity (feet per day), and method of estimation used			
		Horizontal			Vertical
		Bouwer and Rice (1976)	Hvorslev (1951)	Cooper and others (1967)	$K_v=K^2/K_h$
WG035AN	4-6	(*a*)	(*a*)	(*a*)	(*a*)
BN	8-10	6	12	47	3
CN	13-15	13	24	88	6
DN	16-18	2	4	8	1
EN	28-30	1	2	6	.4
WG044AN	6-8	5	10	20	4
BN	10-12	5	9	26	3
CN	20-22	1	2	8	.5
DN	31-33	.4	.2	.6	.04
WG053AN	6-8	.3	(*b*)	(*c*)	(*c*)
BN	10-12	23	43	150	12
CN	16-18	8	12	42	3
DN	18-20	8	12	52	3
EN	23-25	2	3	8	.5
WG062A	4-6	.9	2	(*c*)	(*c*)
B	10-12	.3	.8	2	.3
C	15-17	.4	1	1	.7
D	20-22	4	6	22	2
E	26-28	.01	.04	.15	.01

[a] Water table below screened interval.
[b] Method not applicable for well screens extending above the water table.
[c] Method not applicable for shallow unconfined conditions.

The method of Bouwer and Rice (1976) produced conductivity values generally a factor or two lower than estimates produced using the method of Hvorslev (1951). The higher values estimated from Hvorslev's technique probably are due to the assumption by Hvorslev of infinite vertical (upward) extent of the flow system, which is not met when the well screen is immediately below the water table (Bouwer and Rice, p. 427). Furthermore, the small screened interval assumed in the Hvorslev technique decreases the effective radius to the extent that vertical flow cannot be ignored. The estimates based on the method of Cooper and others (1967) probably represent the upper limit of the range of acceptable horizontal hydraulic conductivities. The values are larger than the estimates from the other methods because Cooper and others assume, in their technique, that only horizontal flow occurs to the well screen. In other words, the length of the screened interval represents the aquifer thickness. In reality, however, vertical flow could be significant and probably accounts for the larger estimated hydraulic conductivities produced using the method of Cooper and others (1967). The method of Bouwer and Rice (1976) probably provides reasonable values; however, owing to the prevalence of clay-size particles (table 1) and the use of bentonite to seal the wells, perfect hydraulic connection between the bore hole and aquifer cannot be expected, even after well development, as clays may clog, or partly clog, some of the screened openings in the casing. Clogging may result in lower estimates of hydraulic conductivity.

Estimated horizontal hydraulic conductivities at cluster-well sites from the three methods described above are listed in table 2. Even with the different assumptions inherent in the three methods, the estimated hydraulic conductivities varied by a factor of only about six. Hydraulic conductivities estimated from specific capacities (discharge rate of a well divided by measured drawdown of water level within the well) in areas dominated by fine to coarse sand and gravel are more comparable to those calculated from the method of Cooper and others (1967); in areas dominated by silt, they are more comparable to hydraulic conductivities calculated from the method of Bouwer and Rice (1976).

Under optimal conditions in which the assumptions inherent in the development and application of the methods are true, a vertical hydraulic conductivity can be estimated by comparing the method of Hvorslev (1951), which assumes spherical, isotropic flow to a point (or small well screen relative to aquifer thickness), to the method of Cooper and others (1967), which assumes only horizontal flow to the well screen. This can be done using the relation given by Freeze and Cherry (1979, p. 177):

$$K_v = K^2/K_h, \quad (1)$$

where K_v = vertical hydraulic conductivity being estimated, in feet per day;

K = isotropic hydraulic conductivity calculated using the Hvorslev technique, in feet per day; and

K_h = horizontal hydraulic conductivity calculated using the method of Cooper and others (1967), in feet per day.

Vertical hydraulic conductivities estimated using eqn. 1 result in ratios of horizontal to vertical conductivity (anisotropy) of about 10:1 (table 2). This ratio may reflect the depositional history of the Whitney area, which is dominated by flood-plain deposits. This estimate, however, may be in error by as much as an order of magnitude, because vertical differences in conductivity are difficult to quantify without aquifer tests involving multiple wells screened at different depths.

Estimated conductivities (table 2) were used to produce a vertical profile of horizontal hydraulic conductivity through the entire aquifer and along a line connecting four cluster-well sites (fig. 5). This was accomplished by first averaging horizontal and vertical hydraulic conductivities over 5-ft depth intervals for the entire 30-ft saturated thickness of the aquifer at each well cluster (table 3), and then interpolating the averaged hydraulic conductivities between cluster-well sites. The average horizontal hydraulic conductivities were used to estimate the quantity of flow at each cluster-well site. Because the cluster-well sites are aligned nearly parallel to the flow path, the quantity of flow at each site could be readily estimated for a unit area (thickness times unit width of aquifer) using Darcy's law. The hydraulic gradient was estimated from measured water levels at wells upgradient from, downgradient from, and at the cluster well being assessed.

TABLE 3.--*Estimated horizontal and vertical hydraulic conductivities, hydraulic gradient, and ground-water flow rate for entire aquifer thickness of unit width at each cluster-well site*

Site	Hydraulic conductivity (feet per day)		Hydraulic gradient (feet per foot)	Flow (cubic feet per day)
	Horizontal	Vertical		
WG035N	30	3	0.0045	4.1
WG044N	17	2	.0077	3.9
WG053N	36	3	.0083	9.0
WG062	8	.7	.0167	4.0

Specific Yield

As the water table declines in response to evapotranspiration, the quantity of water removed from the aquifer is related to the specific yield of the sediments. Specific yield of a sediment is defined as the ratio of the volume of water that the sediment, after being saturated, will yield by gravity to the volume of sediment (Lohman, 1972, p. 6).

The quantity of water removed by evapotranspiration can affect simulation results. Thus, estimates of specific yield were determined from single core samples collected at cluster-well sites WG044N, WG053N, WG062, and WG071. Cores of 1.5-in. diameter were collected from the zone of water-table fluctuation, usually 1-2 ft below land surface at these cluster-well sites. The cores were sealed and taken to the laboratory, where they were cut into 2-in. lengths. The 2-in. samples were saturated in deionized water for 48 hours and then weighed. Once saturated, the samples were subjected to incremental pressures over a period of 1 week. Each pressure increment was held constant for a period of 24 hours, after which the sample was reweighed. An estimate of drainable water was determined by subtracting the weight of the sample prior to the increase in pressure, from the weight after a 24-hour period.

Figure 6 shows the results of the laboratory tests. Data for cluster-well sites WG044N and WG053N and for sites WG062 and WG071 are grouped together, because the sample pairs had similar drainage curves. The curves were used to estimate an approximate specific yield by defining the point on the curve where the slope becomes noticeably flatter. The specific yield ranged from 8 to 12 percent for all cores.

This technique works well for sandy soils, where the cumulative quantity of drainable water ceases to increase with increasing pressure. It is less satisfactory for silt and clay, which are common in the upper part of the aquifer in the Whitney area, because the cumulative quantity of drainable water from silt and clay generally does not cease with increasing pressure. This phenomenon is known in soil mechanics as creep.

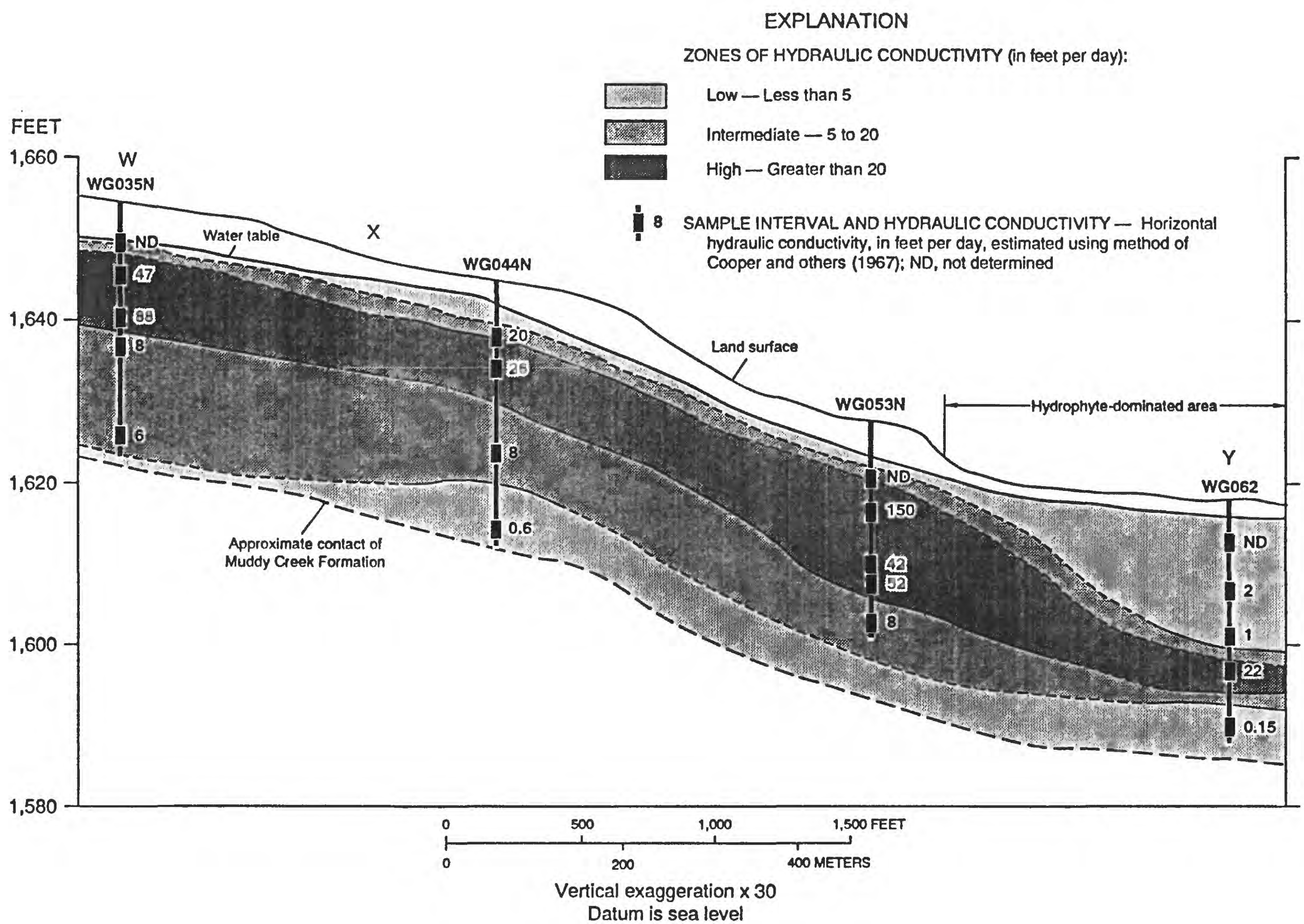


FIGURE 5.--Hydrogeologic section along line W-Y (figure 4), showing generalized distribution of horizontal hydraulic conductivity on basis of slug-test data for cluster wells WG035N, WG044N, WG053N, and WG062. Zone boundaries dashed where approximately located.

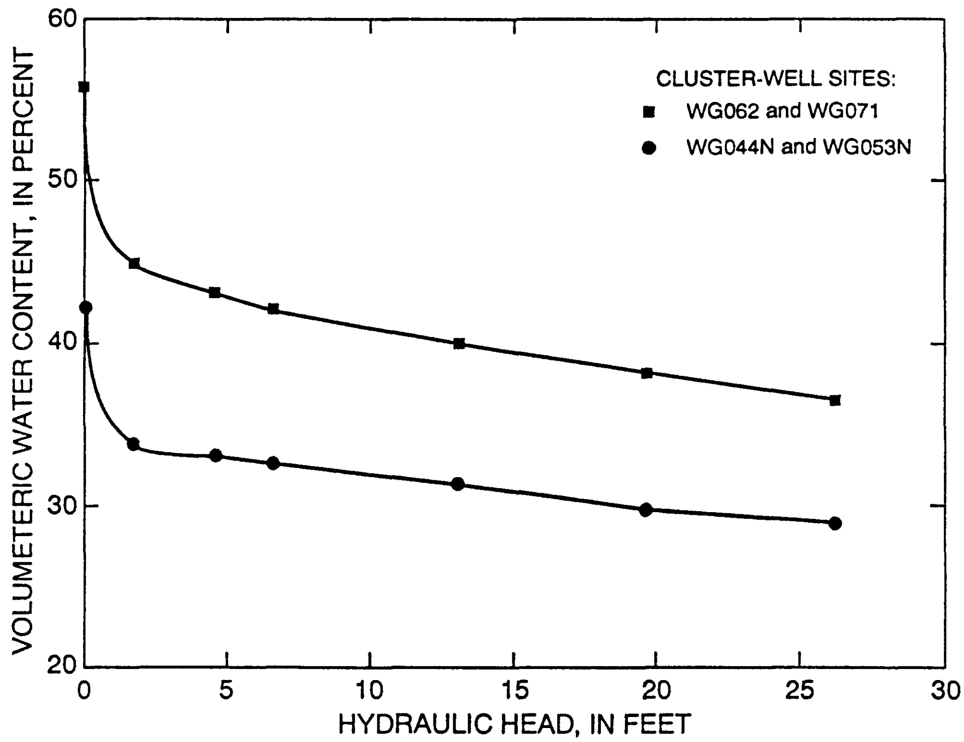


FIGURE 6.--Relation between volumetric water content and hydraulic head for cores taken from zone of water-table fluctuation at selected cluster-well sites. Shows averaged values for cluster-well sites WG062 and WG071 and for sites WG044N and WG053N.

Porosity

Porosity is the ratio of the total volume of voids within a rock or soil sample to the total volume of the sample, and is reported as a decimal or percentage. The total porosity should not be confused with the effective porosity, which refers to the volume of interconnected void spaces through which a fluid (usually water) can be transmitted. The total porosity can be considerably larger than the effective porosity, especially for fine-grained deposits.

Total porosities were estimated on the basis of laboratory measurements made on the same cores used to estimate specific yield. The estimates were made using the following equation (Piper, 1933):

$$n = \frac{(W_s/V_s)-(W_s/V)}{(W_s/V_s)}, \qquad (2)$$

where n = total porosity (dimensionless);
W_s = weight of oven-dried particles, in grams;
V_s = volume of solid particles, in cubic centimeters; and
V = total volume, in cubic centimeters.

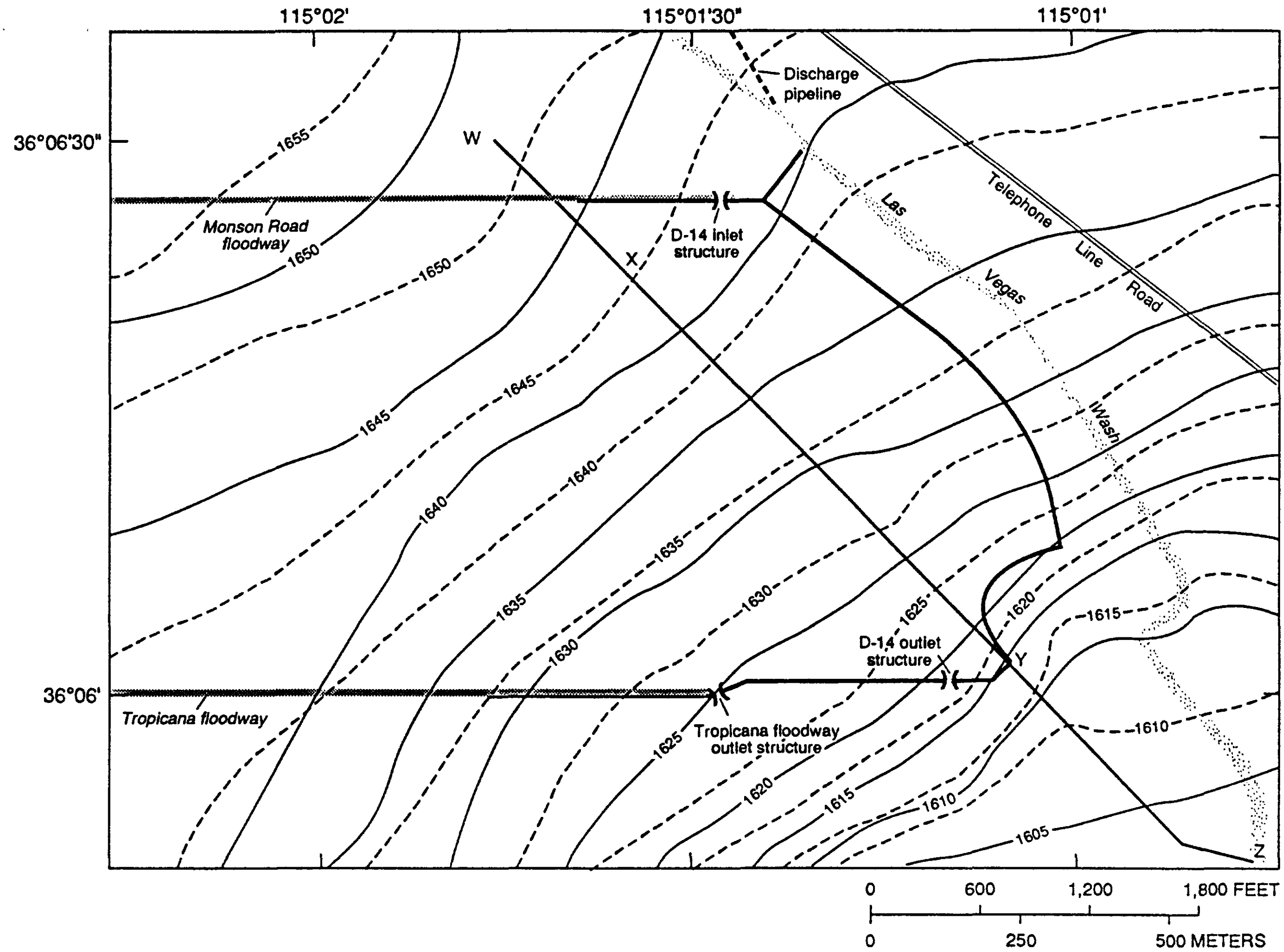
The estimated total porosities calculated from eqn. 2 range from 0.37 to 0.62; these high values probably reflect the large fraction of clay and, in the upper few feet of alluvium, peat. Had cores been obtained in the more highly permeable zones of the aquifer, estimated total porosities would probably be much lower. However, the coring device used to collect the samples was designed for shallow sampling; hence, deeper cores were not available for estimating porosity. These estimated values were used for the shallowest part of the saturated aquifer, but the porosities of deeper parts had to be estimated on the basis of grain-size analyses (table 1; Freeze and Cherry, 1979, p. 37).

Recharge to the Detention Basin

Estimating the quantity of recharge entering the detention basin is an important step in developing a conceptual model that accurately describes the hydrologic conditions of the study area. The magnitude of recharge was estimated on the basis of the hydrogeologic properties discussed in the previous sections. Recharge enters the detention basin as (1) ground water from upgradient areas, (2) surface water primarily from Monson Road floodway and upgradient sewage effluent, and (3) precipitation from storms. Because storms are infrequent, their effects are difficult to quantify; therefore, only the first two sources are estimated and considered in this study.

Ground water enters the detention basin through the shallow alluvial aquifer. This water originates primarily from treated sewage effluent in the flood plain adjacent to Las Vegas Wash (wasteway channel). Some additional flow may originate from secondary recharge as a result of lawn and golf-course watering and urban runoff in East Las Vegas (fig. 1B). The quantity of ground water entering the detention basin was estimated from Darcy's law using averaged horizontal hydraulic conductivities (table 3) along cross section W-Z (parallel to the flow path) shown in figure 4 and hydraulic gradients determined from the water table map in figure 7. An average volumetric inflow of 4.0 ft^2/d (0.0000463 ft^2/s) was estimated for a 30-ft aquifer thickness with a unit width of 1 ft. If ground-water inflow is assumed to be constant over the area of the detention basin, the estimate of inflow to the basin through a strip of aquifer 4,200-ft wide is 16,000 ft^3/d (0.2 ft^3/s). The 4,200-ft wide strip was determined by measuring the distance between the arms of the proposed slurry wall along a water-level contour (the estimated capture zone of the detention basin).

Although the estimate of ground-water inflow is based on available information, its accuracy is uncertain because of uncertainties in the aquifer thickness at cluster-well site WG035N (where volumetric flux was estimated), errors in estimating hydraulic conductivity, and the assumption that flow is areally constant over the thickness of the aquifer. Evidence that the estimated inflow rate may be low is provided by pumping and water-level data from the Clark County Sanitation District (CCSD) for an area approximately 3,000 ft northwest of well WG025 (figs. 3 and 4). In this area, the shallow aquifer was dewatered to allow for the construction of below-ground clarifiers. Pumping for this project began in mid-1986 and ended in the spring of 1987. Monitoring wells installed by CCSD to depths of 10 to 20 ft in the vicinity of the pumping wells went dry after 2-4 months of continuous pumping at an estimated constant rate of 2.5 ft^3/s. The resulting cone of depression grew during the period of pumping to encompass an area larger than the area of influence of the detention basin. Water-level declines in CCSD monitoring wells more than 3,000 ft from the pumping wells were between 1 and 3 ft. Consequently, the overall gradient toward the detention basin was lowered, resulting in a lower volume of water entering the basin (fig. 7). An estimate of ground-water inflow to the detention basin was made for conditions without the influence of pumping from the CCSD facility. This was done by extending the capture zone of the detention basin upgradient to the pumping zone and estimating the percentage of overlap between the zone of influence of the pumping wells and the capture zone of the detention basin. This percentage was then multiplied by the average pumping rate to determine the potential additional inflow to the detention basin. Using this approach, the additional estimated inflow was 0.8 ft^3/s and probably represents a maximum. Thus, estimates of ground-water inflow to the detention basin prior to CCSD pumping probably ranged from 0.2 to 1.0 ft^3/s. Seasonal variations in inflow are likely to be small relative to the total volumetric inflow rate, but probably are greatest during summer months when water use and urban runoff are at their peak.



EXPLANATION

—— 1650 —— - - - 1650 - - -	WATER-TABLE CONTOUR — Shows approximate altitude of water table. Solid lines show lowest measured water levels during year (in September 1986, after pumping began); dashed lines show highest measured water levels during year (in March 1986, prior to pumping). Interval 5 feet. Datum is sea level
————	DIKE STRUCTURE
W———Z	LINE OF SECTION USED IN OTHER FIGURES — Intermediate points X and Y are shown on map

FIGURE 7.--Water-table contours for March and September 1986, which depict the effects of pumping and evapotranspiration.

The second source of recharge is surface water that enters the detention basin at the north side of the dike from the Monson Road floodway, and treated effluent in the flood plain adjacent to the wash (fig. 3). The quantity of surface-water inflow to the study area probably is similar in volume to ground-water inflow on a yearly average, but varies more widely from season to season than ground-water inflow. Floodway flow and discharge of treated effluent are significantly greater during summer months when water use is at its peak. Estimated inflow to the detention basin ranges from 0.2 to 0.8 ft^3/s, and has a yearly average of about 0.5 ft^3/s (inflows can be much higher during storm events [D. Art Tuma, U.S. Bureau of Reclamation, oral commun., 1987]). Surface-water inflow provides water for vigorous hydrophyte growth in the eastern part of the detention basin (fig. 3).

Additional inflow to the detention basin along Tropicana floodway is substantial, but because the floodway is on the downgradient side of the detention basin and all the inflow leaves through an outflow structure farther to the east (fig. 3), its significance as a source of recharge to the study area is minimal. However, the Tropicana floodway does act as a sink for shallow ground water (less than 5 ft below land surface) within the detention basin.

Discharge from the Detention Basin

To estimate discharge from the detention basin, time-dependent physical processes affecting discharge need to be identified. These processes are important in developing an accurate conceptual model that will be used to simulate potential hydrogeologic effects of the proposed slurry wall. The quantity of discharge from the detention basin varies greatly from season to season, primarily owing to evapotranspiration--the main process by which water leaves the study area. Other sources of discharge include ground-water outflow beneath the dike structure and surface-water flow through outlet structures in the south side of the dike. Additional drainage of the aquifer occurs along the Tropicana floodway (fig. 3) when ground-water levels are above the excavated level of the channel. The total quantity of ground-water outflow is the most important source of discharge to estimate, because ground water is responsible for transporting appreciable dissolved-solids loads to the wash.

Evapotranspiration greatly influences the quantity of ground water that leaves the detention basin. Water levels generally are from 2 to 6 ft below land surface in the western part of the area, and range from 0 to 4 ft in the eastern part of the area. These shallow water levels are ideal for phreatophyte and hydrophyte communities to thrive. Hydrophytes occupy the eastern part of the area (fig. 3) where inflow from the Monson Road floodway and flood plain provides enough water to inundate the area with several inches of water during periods of low evapotranspiration; surface water often flows through the detention basin in small stream channels occupying topographically low areas. Upgradient from the inlet structure, in a topographically higher area, phreatophytes dominate. Evapotranspiration is greatest during summer months, when plants are thriving and temperatures are high (fig. 7). The lowest recorded water levels at most wells coincide with this period of high evapotranspiration. From September to early March, when much of the vegetation is dormant, water levels rise, reaching their peak between January and March. Figures 8 and 9 show the seasonal water-level fluctuations at selected well sites in areas dominated by hydrophytes and phreatophytes. Maximum yearly changes in water levels due to evapotranspiration range from 3.5 ft in phreatophyte-dominated areas to 2.0 ft in hydrophyte-dominated areas. The smaller net change in the hydrophyte areas probably is due to the influx of surface water that buffers the water-level response caused by evapotranspiration.

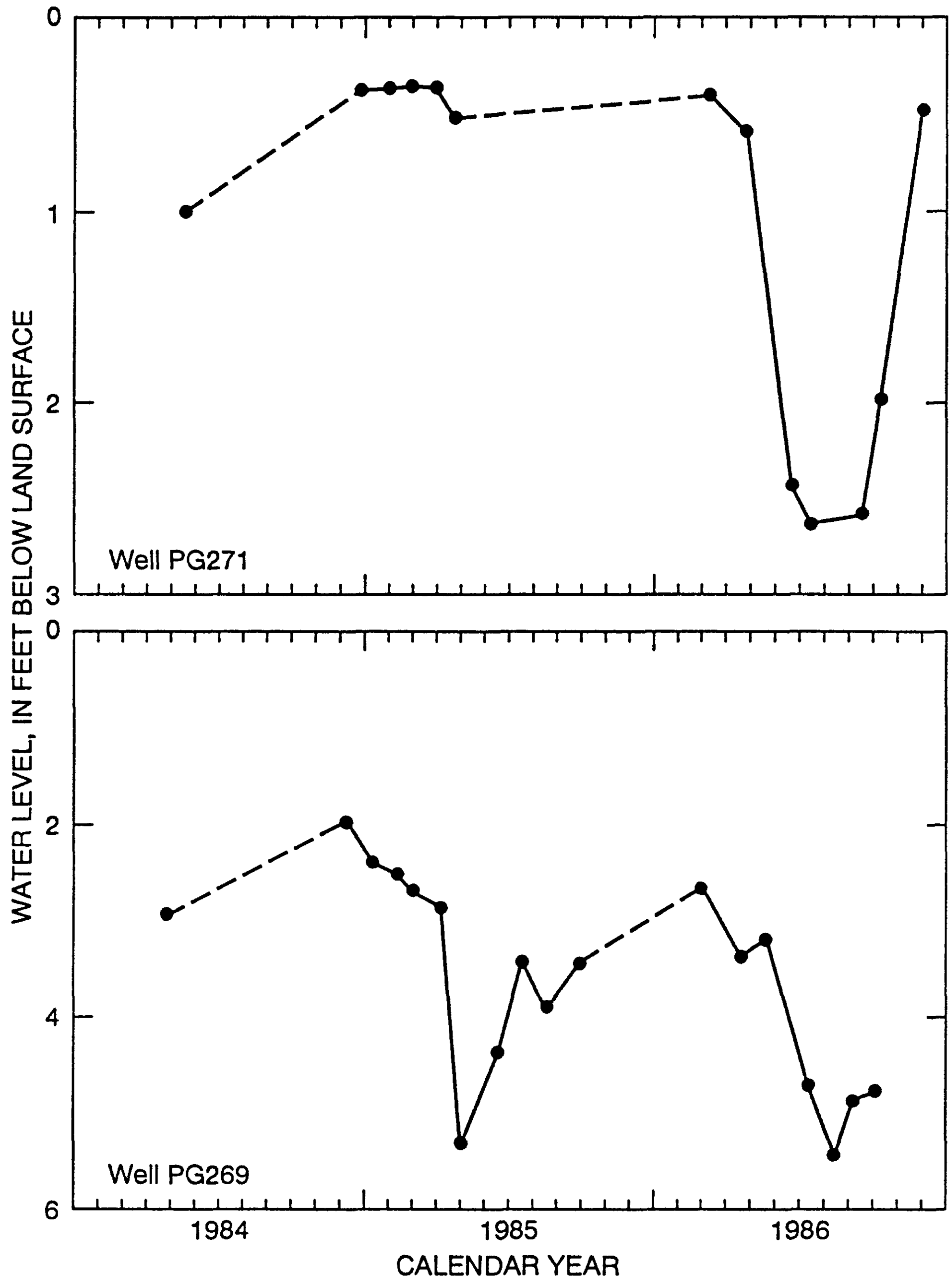


FIGURE 8.--Seasonal water-level fluctuations in areas of hydrophytic vegetation (wells PG271 and PG269). Dashed lines connect measurements more than 6 months apart.

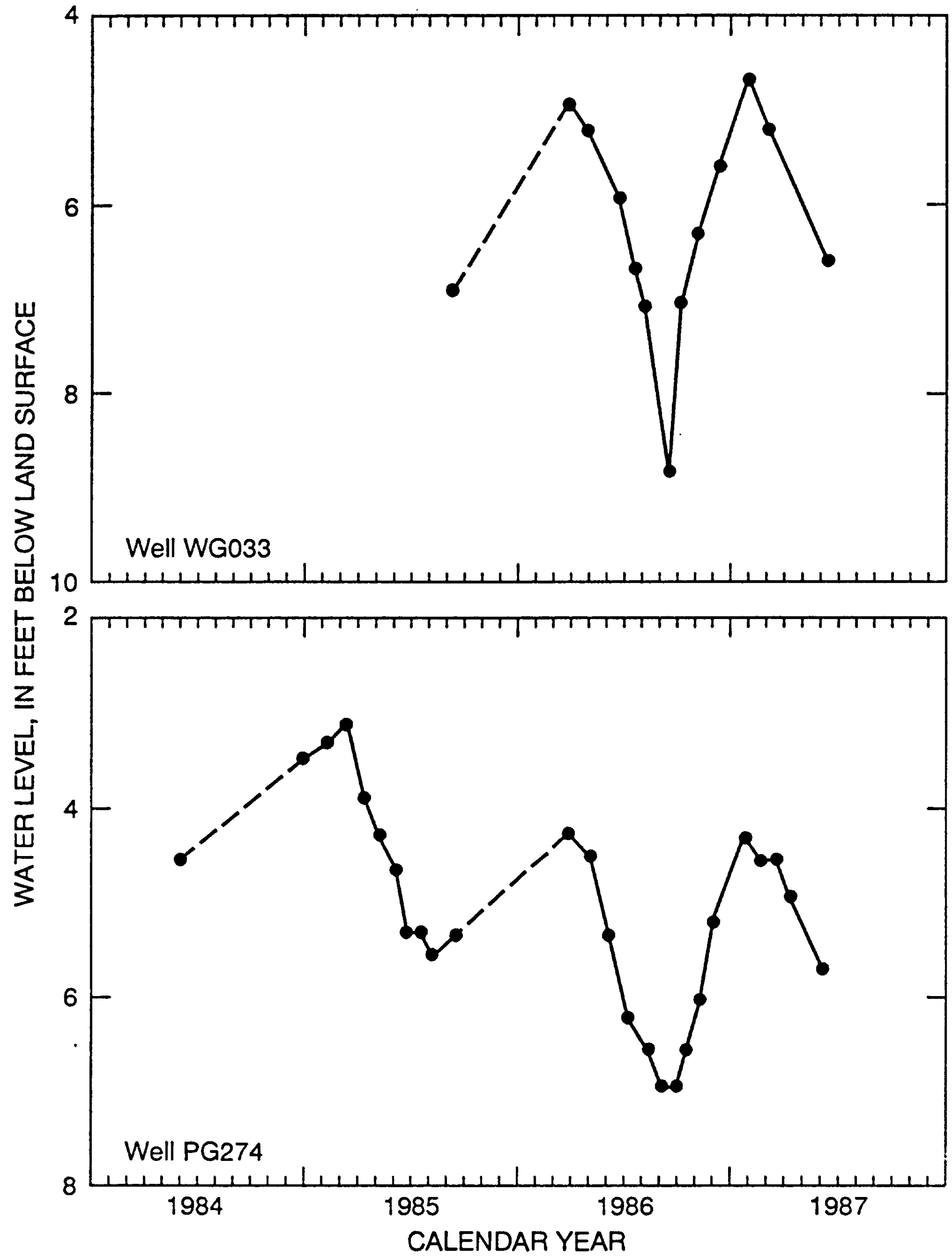


FIGURE 9.--Seasonal water-level fluctuations in areas of phreatophytic vegetation (wells WG033 and PG274). Dashed lines connect measurements more than 6 months apart.

Evidence suggests that discharge by evapotranspiration during summer months may be nearly equivalent to the combination of ground-water and surface-water recharge to the detention basin. Nitrate concentrations at well site PG264 (fig. 4) increase to twice their yearly average during late summer, reflecting the high concentrations in the wash 50 ft away. In other words, the wash acts as a sink during most of the year, but it can act as a source of water for the shallow aquifer during late summer. However, the quantity of water supplied by the wash during this time probably is small and localized. Furthermore, streamflow rates along a section of the wash in the Whitney area have been reported to decrease during late summer (Schmidt and Hess, 1980, p. 20), indicating that the wash may be a source of water to the shallow aquifer during late summer. Ground water may discharge from the detention basin in areas farther from the wash (south of the line of section W-Z shown in figure 4), but the quantity is probably small.

The total discharge by evapotranspiration is difficult to quantify, owing to the large seasonal fluctuations resulting from variations in temperature and water consumption by plants. A rough conjectural estimate of the annual discharge by evapotranspiration within the detention basin is 0.5 ft^3/s, on the basis of an evapotranspiration rate of 30 in/yr. During the summer, evapotranspiration and surface-water flow through the outlet structure probably are the most significant sources of discharge from the detention basin; ground-water discharge is probably minor. Inflow of surface water is at its maximum during summer months and is estimated to be as much as 0.8 ft^3/s. However, even during peak evapotranspiration, some surface flow--probably no more than 0.3 ft^3/s--passes through the outlet structures. Hence, the maximum net surface-water recharge that is consumed by evapotranspiration during the summer is about 0.5 ft^3/s. If evapotranspiration consumes all the estimated ground-water inflow (0.2-1.0 ft^3/s) during summer months, the total evapotranspiration can range from 0.7 to 1.5 ft^3/s, or 40-300 percent greater than the conservative annual rate estimated above for the entire detention-basin area.

During winter months, surface-water inflow nearly equals surface-water outflow. Measurements at the inlet structure indicate that inflow is at its yearly minimum of about 0.2 ft^3/s during the winter. Measured outflow at the structure along the south side of the dike was 0.3 ft^3/s during the same period. Hence, evapotranspiration during winter months approximately equals the quantity of ground water recharging the detention basin, minus the amount of ground water discharging from the detention basin. The rate of evapotranspiration during the winter probably is about 0.1 ft^3/s, when temperatures are much cooler and hydrophytes are dormant. Thus, ground-water outflow would range from about 0.1 to 0.9 ft^3/s. Until further work can be done to better quantify evapotranspiration, these estimates represent preliminary approximations.

Chemical Quality of Ground Water in the Study Area

Water-quality samples were collected at all available well sites in the Whitney area in April and September 1986, and January and June 1987. The water-quality data and the interpretation of the geochemical analyses are presented by Emme and Prudic (1991). This report deals with the dissolved-solids concentrations along section W-Z shown in figure 4. The dissolved-solids concentrations along this line of section are used with the estimated aquifer properties and recharge and discharge estimates to simulate the effect the proposed slurry wall would have on the flow and chemical quality of the aquifer system.

Dissolved-Solids Concentrations

The average dissolved-solids concentration (based on samples from wells at all sites) of ground water in the shallow alluvial aquifer beneath the detention basin, for the four sampling periods during 1986-87, was 6,600 milligrams per liter (mg/L)--much higher than in areas farther to the west in central Las Vegas Valley. The high concentrations of dissolved solids can be attributed to (1) evaporite minerals, especially gypsum, which occur naturally in the alluvial deposits in the study area and vicinity and are readily dissolved, (2) ground-water recharge high in dissolved solids, and (3) evapotranspiration that tends to increase the concentration of dissolved solids near the water table.

Gypsum, an evaporite mineral responsible for the high dissolved-solids concentration in the shallow aquifer, is relatively abundant in the near surface soils of the Whitney area. Soil samples indicate abundances as great as 15 percent of the total sample (Emme and Prudic, 1991). Much smaller percentages of gypsum, generally 1-4 percent, were found at greater depths. The one available sample taken from a reddish clay layer thought to be part of the Muddy Creek Formation showed no detectable gypsum or other evaporite minerals. However, gypsum is a common mineral within the Muddy Creek Formation (Bohannon, 1984, p. 56), and may be present at other places in the Whitney area.

The average dissolved-solids concentration of ground water entering the detention basin from upgradient areas is about 6,000 mg/L, slightly less than the average within the basin. The increase in dissolved-solids concentration within the basin is due to evapotranspiration, which tends to concentrate dissolved solids near the water table (Emme and Prudic, 1991). Emme and Prudic also report that the ground water beneath the detention basin is saturated with respect to gypsum. This suggests that further dissolution of evaporite minerals in the alluvial deposits is unlikely.

Surface-water inflow tends to lower the dissolved-solids concentration in areas downgradient from the inlet structure. Inflow from the Monson Road floodway generally is similar in quality to the upgradient ground water (that is, dissolved solids concentration of about 5,000 mg/L), but treated effluent entering the area has dissolved-solids concentrations of about 1,400 mg/L. The average dissolved-solids concentration of floodway inflow and treated effluent is about 3,000 mg/L. This combined inflow apparently mixes with the ground water in areas downgradient from the inlet structure (areas dominated by hydrophytes and corresponding to well site WG053N in section W-Z, figs. 3 and 4), resulting in a uniform distribution of dissolved solids throughout the full thickness of the aquifer. Figure 10 shows the distribution of dissolved-solids concentration along section W-Y in figure 4. The effects of evapotranspiration and surface water inflow are readily apparent. During storms, much of the buildup of evaporite salts near land surface is dissolved and transported to the wash; consequently, continual buildup of salts over many years is unlikely.

Dissolved-Solids Load to Las Vegas Wash

The total dissolved-solids load contributed to Las Vegas Wash from the detention basin was estimated, for the purpose of projecting how the proposed slurry wall would help to improve the quality of that inflow if the proposed D-14 detention basin were to work as planned when finally completed. Because of the uncertainty of both inflow and evapotranspiration rates (as discussed earlier), the estimated dissolved-solids load can be considered as highly approximate.

The average annual inflow to the detention basin (neglecting storm runoff) was assumed to be 1.2 ft^3/s. Of this amount, 40 percent (0.5 ft^3/s) was assumed to originate as surface-water inflow with an average dissolved-solids content of 3,000 mg/L. The remaining 60 percent (0.7 ft^3/s) was assumed to be ground-water inflow with a dissolved-solids concentration of 6,600 mg/L (a slightly higher dissolved-solids concentration is used than actually measured, to take into account increases in dissolved-solids concentration due to evapotranspiration in the detention basin). The estimate of 0.7 ft^3/s is a yearly average. The maximum estimated inflow of 1.0 ft^3/s was determined during summer months when shallow ground-water flows are at their peak. Combining these incoming loads and assuming no additional dissolution of salt, the overall average dissolved-solids concentration of water leaving the detention basin and seeping to the wash would be about 5,500 mg/L. The annual average outflow of ground water from the detention basin was estimated to be about 0.5 ft^3/s (using a value of 0.2 ft^3/s as surface-water outflow that is not included as part of the shallow ground-water flow system discharging to the wash or wasteway channel, and 0.5 ft^3/s as the annual evapotranspiration rate). The total dissolved-solids load to the wash on the basis of these estimates would be approximately 2,700 ton/yr. Schmidt and Hess (1980, p. 20) estimated that the total salt load in the wash was as high as 140,000 ton/yr in the 1970's; more recently, the total was estimated to be as high as 240,000 ton/yr (D. Art Tuma, U.S. Bureau of Reclamation, oral commun., 1987). Thus, the contribution from the Whitney area may represent as little as 1 percent of the total load. Even if the ground-water flow were as great as 1 ft^3/s, the dissolved-solids load to the wash would be only a small part of the total load of salts in the wash.

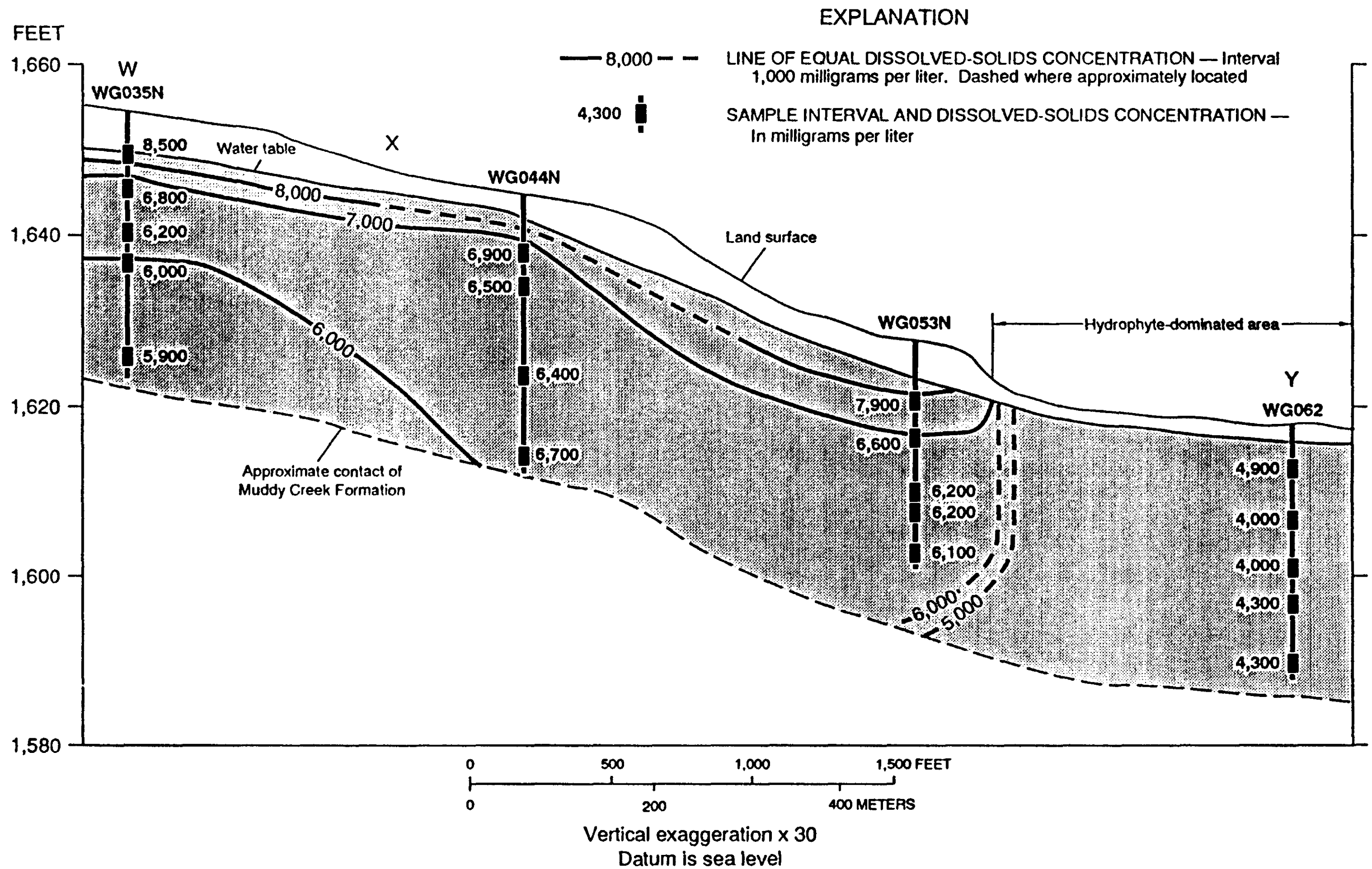


FIGURE 10.--Hydrogeologic section along line W-Y (figure 4), showing average water-table altitude and distribution of average dissolved-solids concentrations in 1986-87. Concentrations beneath hydrophyte-dominated area upgradient from cluster-well site WG062 are based on U.S. Geological Survey data from U.S. Bureau of Reclamation wells (D. Art Tuma, U.S. Bureau of Reclamation, written commun., 1987).

SIMULATING EFFECTS OF PROPOSED SLURRY WALL ON GROUND-WATER FLOW AND SOLUTE TRANSPORT

Preceding sections of this report describe the physical setting and hydrogeologic properties of the alluvial deposits, ground-water flow rates, and ground-water chemistry in the Whitney area. This section deals with the development of conceptual and mathematical models, using the information described in previous sections, to describe the effect the proposed slurry wall could have on ground-water flow and solute transport in, and surrounding, the detention basin.

Three conceptual models were developed and tested using mathematical (numerical) models, to analyze effects of implementing the proposed slurry wall (which would extend through the entire thickness of the aquifer beneath the dike structure). The first model is a planimetric ground-water flow model (solute transport is not included), which is designed to analyze flow characteristics in the vicinity of the detention basin. In other words, the model is intended to determine (1) whether ground water will continue to recharge the basin or whether, after some period of time, flow will be diverted around the basin as water levels rise behind the dike and slurry wall; and (2) how the distribution of hydraulic conductivity influences ground-water flow in the detention basin. These questions are addressed in the discussion of the planimetric model.

The second model is a cross-sectional ground-water and solute-transport model (along segment X-Y of section W-Z, fig. 4). It is designed to analyze the sensitivity of several aquifer and transport properties, to (1) determine which properties greatly affect the flow and dissolved-solids distribution in the detention basin, and (2) describe the conditions under which the proposed slurry wall would work most effectively.

The third model is an expansion of the second model to include downgradient areas from the slurry wall to Las Vegas Wash, with modifications to the program to incorporate seasonal variations in evapotranspiration at the water table. Field-estimated properties were included in a simulation of flow and solutes to replicate conditions prior to construction of the slurry wall and to project how a proposed slurry wall might affect flow and transport in the study area.

Figure 11 shows both a vertically exaggerated (*A*) and a true-scale section (no vertical exaggeration; *B*) through the Whitney area that is used in the latter two models, with the proposed slurry wall included. The vertical scale is exaggerated to show the flow and dissolved-solids distributions produced by the simulations.

Before the development of each conceptual model is discussed, the numerical technique used to express the conceptual model in mathematical terms is presented. In addition, some definitions that describe transport phenomena are given.

Mathematical Modeling Technique Used

The computer program used to simulate flow and transport of solutes in the conceptual models is known as SUTRA (Saturated-Unsaturated TRAnsport), a U.S. Geological Survey numerical model developed by Voss (1984). Analytical models, although exact, are greatly limited in their application primarily because of their inherent simplistic boundary conditions and restriction to linear problems. SUTRA employs standard finite-element approximations for the governing equations describing fluid mass, solute mass, and energy. This program was selected over others for the analysis of ground-water flow and dissolved-solids transport in the Whitney area for several reasons:

1. The program allows for geometric flexibility in application of boundary conditions and variable grid spacing.

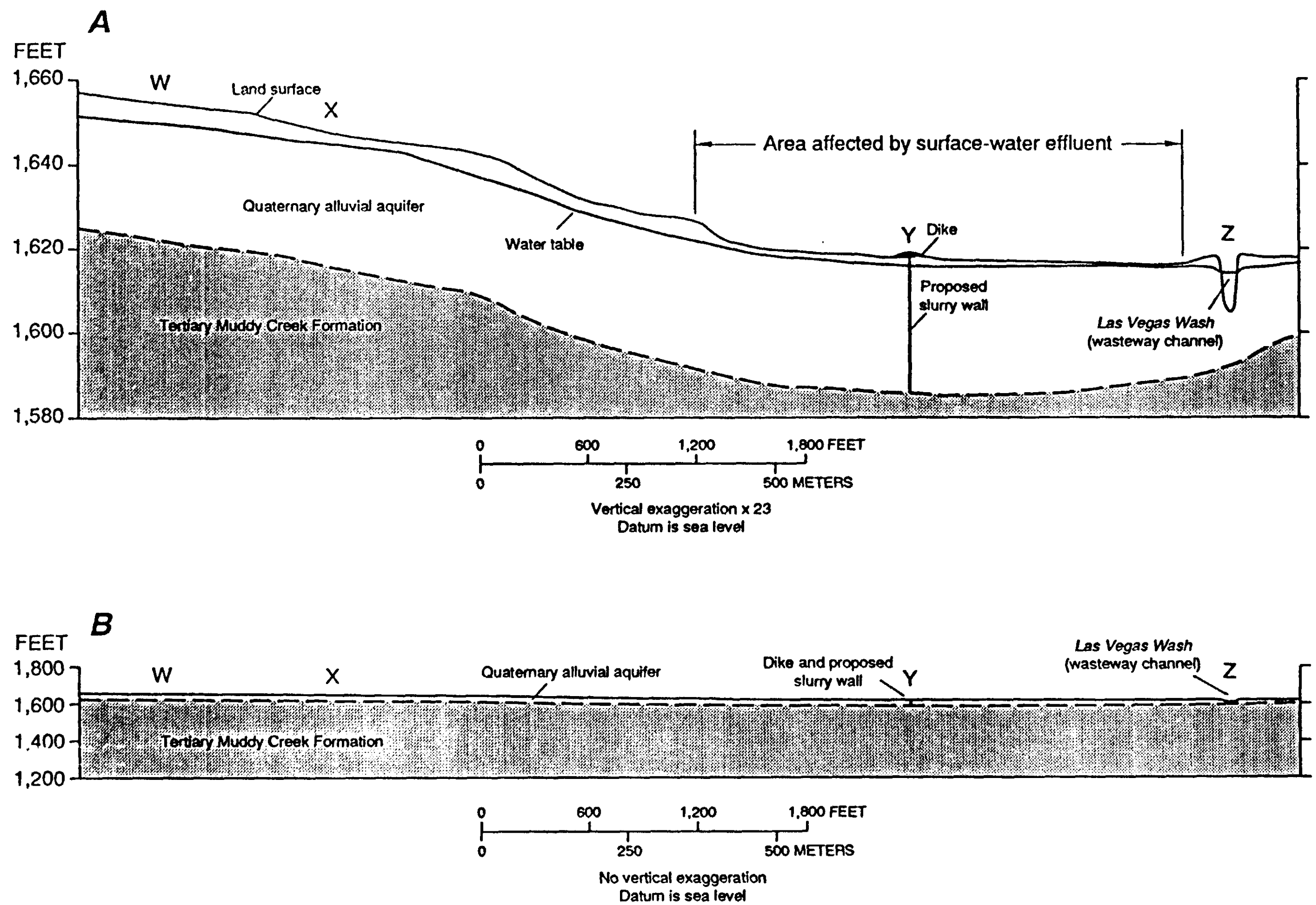


FIGURE 11.--Hydrogeologic sections along line W-Z (figure 4), showing contrast between (*A*) vertically exaggerated and (*B*) unexaggerated projections.

2. The program can perform simulations using planimetric or vertical sections.

3. The program incorporates single-species, density-dependent solute transport.

4. The program employs numerical techniques accurate enough to greatly diminish numerical error when large concentration gradients and time steps are used.

5. The program can be used to simulate both steady-state and transient ground-water flow and transport.

6. The program is logically organized, easy to understand, and well documented, allowing for easy modifications to accommodate specialized problems.

These advantages are directly related to the needs defined by the hydrogeologic and geochemical framework of the Whitney area. For example, incorporation of the dike and proposed slurry wall requires variable spacing of the model grid because of the size of the wall relative to the section of aquifer being simulated. Incorporation of density-dependent transport resulting from concentration gradients, over a period of time (that is, transient flow and transport), was necessary to test the effectiveness of the detention basin and its ability to reduce dissolved-solids loads to Las Vegas Wash. The ability of the program to handle time-dependent boundaries allowed for the examination of flow and dissolved-solids distributions immediately after implementation of the slurry wall during a single simulation. The ability to easily modify time-dependent boundaries to incorporate evapotranspiration was necessary to reproduce field-measured dissolved-solids distributions. SUTRA incorporates these attributes in a numerical model that solves the mass-balance equation describing flow and transport phenomena through the aquifer system.

The equation for fluid and solute-mass balance presented here is a simplified form of the equation used by Voss (1984, p. 61). The reasons for the simplification are fourfold. First, only saturated flow conditions were simulated in the analysis of the hydrogeologic conditions in the Whitney area. Second, energy transport was not used; a constant temperature was assumed, both temporally and spatially. Third, the adsorption of solutes on the aquifer sediments was not considered. Fourth, neither production nor decay of the solute mass was a component of this study. The omission of these processes has reduced the governing equation to the following:

$$\varepsilon\rho\frac{\partial C}{\partial t}+\varepsilon v\cdot\nabla C-\nabla\cdot[\varepsilon\rho(D_m I+D)]\cdot\nabla C = Q_p(C^*-C), \qquad (3)$$

where
$\rho(x,y,t)$ = fluid density, in grams per cubic meter;
$\varepsilon(x,y,t)$ = porosity (dimensionless);
$v(x,y,t)$ = average fluid velocity, in meters per second;
∇ = gradient operator;
$\nabla\cdot$ = divergence operator;
C (x,y,t) = fluid solute mass fraction, in grams of solute per gram of fluid;
D_m = apparent molecular diffusivity, including tortuosity effects, in meters squared per second;
I = identity tensor, 1;
D = dispersion tensor, in meters squared per second;
$Q_p(x,y,t)$ = fluid mass source, in grams of fluid per cubic meter per second; and
$C^*(x,y,t)$ = solute concentration of fluid sources, in grams of solute per gram of fluid.

Unlike the movement of ground water, which can be defined entirely by advective processes, the transport of solutes at a macroscopic scale is governed by mechanisms that approximate the effects of mixing of waters with differing solute concentrations, which are moving both faster and slower than the average (advective) velocity. These mechanisms are referred to as hydrodynamic dispersion and molecular diffusion. Hydrodynamic dispersion (D) represents the mixing of fluids that deviate from the average advective or solute flux (Voss, 1984, p. 38). If flow could be defined at a microscopic scale, an averaging of the velocities would not be necessary and the mechanical component of hydrodynamic dispersion would not be needed. Because fluid travels at rates and in directions that differ from the average advective flux, the incorporation of hydrodynamic dispersion is required to account for uncertainties in the true flow field. Generally, the better defined the heterogeneities within a given flow system, the smaller the component of dispersion necessary to describe the solute front. Molecular diffusion, sometimes referred to as ionic diffusion, is a mixing process caused by a concentration gradient within the fluid (Freeze and Cherry, 1979, p. 103). This process can produce a measurable flux if the fluid velocities are extremely small. In most zones of active ground-water circulation, however, mixing caused by hydrodynamic dispersion is much more rapid than molecular diffusion. Consequently, the effects of molecular diffusion often are ignored when the length of the flow path is hundreds or thousands of meters.

Use of hydrodynamic dispersion in the model requires specific values for longitudinal and transverse dispersivity, which, simply stated, represent mixing lengths parallel to and normal to the mean flow direction. The dispersion tensor, D, for an isotropic medium is related to dispersivity as follows:

$$D = \begin{bmatrix} D_{xx} & D_{xy} \\ D_{yx} & D_{yy} \end{bmatrix} \tag{4}$$

$$D_{xx} = v^{2}\left(d_L v_x^2 + d_T v_y^2\right), \quad D_{yy} = v^{2}\left(d_T v_x^2 + d_L v_y^2\right),$$

$$D_{xy} = D_{yx} = v^{2}(d_L - d_T)\,(v_i v_j);$$

$$i \neq j,$$

$$i = x,\ y, \quad d_L = \alpha_L v,$$

$$j = x,\ y, \quad d_T = \alpha_T v,$$

where v = average fluid velocity, in meters per second;

d_L = longitudinal dispersion coefficient, in meters squared per second;

v_x = magnitude of x-component of v, in meters per second;

d_T = transverse dispersion coefficient, in meters squared per second;

v_y = magnitude of y-component of v, in meters per second;

α_L = longitudinal dispersivity, in meters; and

α_T = transverse dispersivity, in meters;

For anisotropic media, calculation of the dispersion coefficients becomes more rigorous. Voss (1984, p. 50-54) gives a detailed discussion of the effects of anisotropy on the dispersion equations.

Dispersivity in a heterogeneous aquifer is scale-dependent. Dispersivity will increase with increases in both solute residence time and displacement distance as the dispersion process develops, but it may approach a constant asymptotic value (Sudicky, 1986, p. 2069) that usually is reached when travel distances of hundreds of meters are involved. A constant dispersivity is assumed for the simulations in this study.

The governing equations describing fluxes of fluid and solute mass are solved using standard finite-element approximations. Other non-flux terms are approximated with a finite-element grid using integrated finite-difference methods. Voss (1984, p. 95-129) gives a comprehensive explanation of the numerical technique used to solve the governing equations.

Now that the mechanisms and processes of flow and transport used by the numerical program have been defined, the three conceptual and mathematical models can be developed with a clearer understanding of the information necessary to represent actual processes in the study area. Metric units have been used in developing the mathematical model because chemical quantities used in the model are in metric units. All model output has been converted to inch-pound units for discussion, illustrations, and tables so that consistency is maintained with earlier sections of this report.

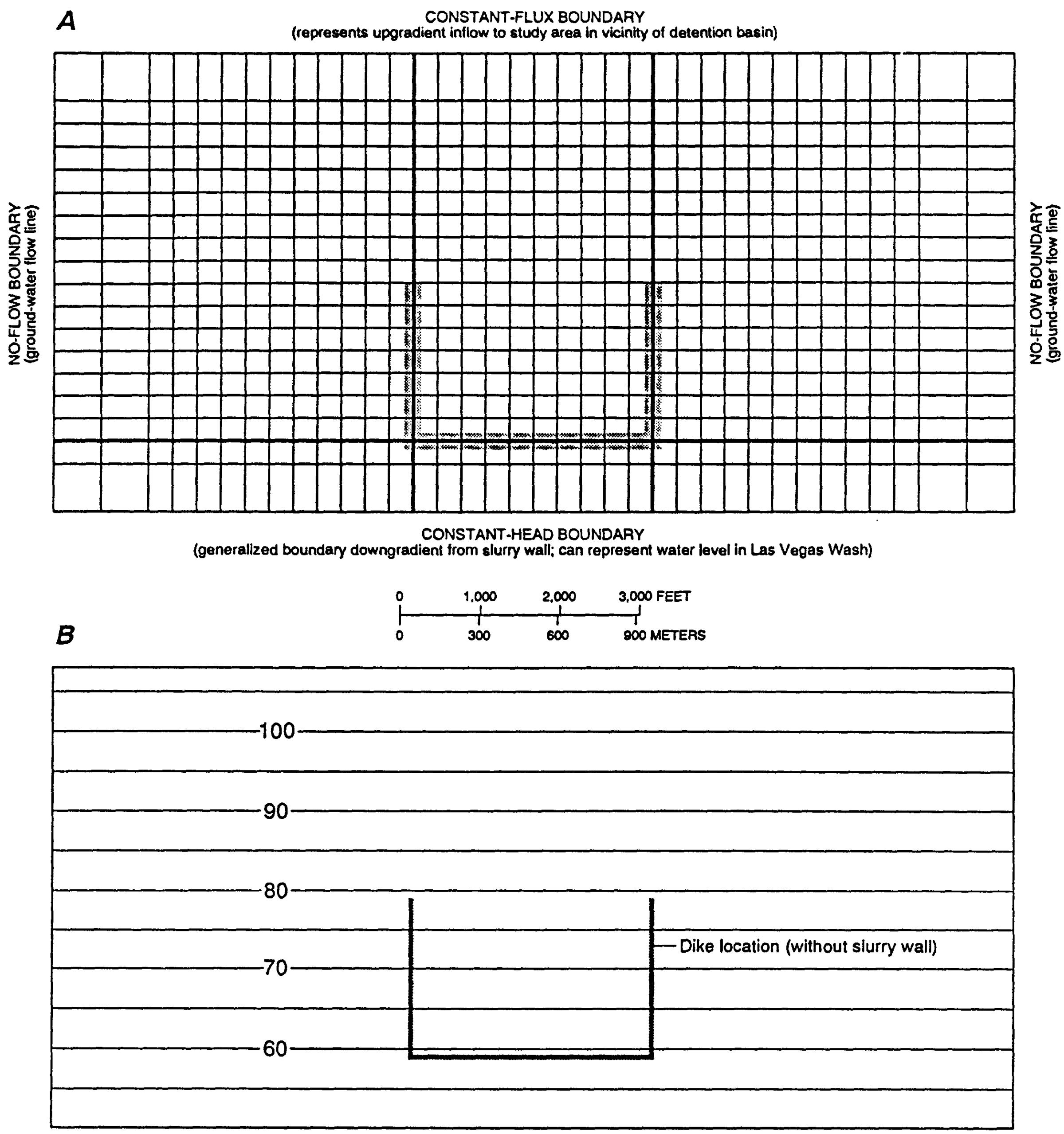
Planimetric Model of Ground-Water Flow

The main purpose of developing a generalized conceptual planimetric flow model was to determine whether the hydraulic heads within the detention basin would be significantly altered by the presence of the slurry wall, resulting in a diversion of flow around the detention basin. The success of the detention basin requires uninhibited inflow; if flow is diverted around the basin, then the potential for salt reduction diminishes.

The generalized conceptual model used to examine the effectiveness of the slurry wall assumes a uniform aquifer thickness throughout the Whitney area. The areal extent of the model was selected to minimize the model's influence on the direction or magnitude of flow or the altitude of water levels in the vicinity of the detention basin. Upgradient ground-water inflow to the area of the detention basin was assumed to be constant. Surface-water inflow to the area was assumed to be insignificant for this particular conceptualization of the system. The average horizontal hydraulic conductivity was estimated to be about 30 ft/d for the entire area, on the basis of slug-test analyses. A "U"-shaped detention basin (open end of U facing upgradient) representing the proposed impermeable dike and slurry wall, similar in size to the actual D-14 detention basin, was incorporated into the conceptual model. The detention basin was designed to release water only by topping the dike, which was assumed to extend 6 ft above the adjacent land surface, on the downgradient side, and to taper to only 1 ft above land surface at its farthest upgradient extent; no outlet structures were incorporated in the conceptual model of the detention basin.

Constructing the Mathematical Model

For these simulations, the governing equation (eqn. 3) for the planimetric model is simplified because only steady-state ground-water flow is considered; thus, all terms involving solute concentration can be ignored. A finite-element grid, consisting of 20 rows and 40 columns of elements connected by 861 nodes, was used in the mathematical model to conceptualize ground-water flow in the Whitney area (fig. 12A). The elements of the planimetric grid differ in areal dimension, but have a constant thickness of 30 ft. Three bands of elements, each 1 ft wide, were used to represent the dike and slurry wall, the permeability of which was varied to analyze its effect on flow fields. A constant-head boundary was specified along the bottom row of nodes in the model grid to approximately represent Las Vegas Wash; that is, the water-table altitude (relative to an arbitrary datum) was specified to be constant at this boundary. This constant altitude allows the fluid-mass flux to vary across the boundary in the simulation. The nodes along the top row of the grid were specified as a constant-flux boundary representing the assumed constant ground-water flow into the study area. Specifically, a constant rate of 1.0 ft^3/s was assigned for total ground-water inflow to this top row of nodes, and it is proportionately divided between each element on the basis of its width.



EXPLANATION

MODEL ELEMENT — All elements have a node at each corner; thus, a node may be common to as many as four contiguous elements

ZONE REPRESENTING TWO ROWS OR TWO COLUMNS OF ELEMENTS — Each row or column has a width of 3.28 feet (1.0 meter). These elements are used in subsequent simulations to represent the dike and hypothetical slurry wall

DIKE AND SLURRY-WALL LOCATION IN GRID

—80— LINE OF EQUAL WATER-TABLE ALTITUDE ON BASIS OF SIMULATION — Interval is 5 feet. Datum is arbitrary

FIGURE 12.--Planimeteric finite-element grid used for model that simulates ground-water flow (*A*), and simulated water-table configuration (*B*) under steady-state conditions without dike and hypothetical slurry wall.

In other words, the inflow assigned to the narrow elements used to represent the dike was less than the inflow assigned to the wider adjoining elements. Of the 1.0 ft^3/s of total inflow specified along the boundary, only 0.2 ft^3/s was assumed to be captured by the detention basin in the simulation; this corresponds to the lowest estimated value of inflow and was calculated earlier in the report. The actual inflow rate is not critical to the analysis when solute transport is not considered. The side boundaries, parallel to ground-water flow and representing streamlines, were specified as no-flow boundaries. To avoid potential negative effects on the flow system caused by these boundaries during simulation, they were placed far enough (5,000 ft) from the elements representing the dike to ensure that they would not influence simulation results in the vicinity of the detention basin.

Results of the Model Simulations

The first simulation was designed to verify that the simulated hydraulic gradient was similar to the observed gradient. A constant hydraulic conductivity of 30 ft/d was assigned to all elements, including those used in the following simulation to represent the dike and slurry wall. The simulated and observed hydraulic gradients proved to be similar (figs. 12B and 7). Both the actual and simulated water levels reveal a constant gradient with a total drop in hydraulic head of about 25 ft from the upgradient to the downgradient extent of the detention basin. The steady-state simulated heads were used as the initial conditions for subsequent simulations incorporating the dike and slurry wall.

For the second simulation, no ground water was allowed to penetrate the slurry wall or over-top the dike. This was accomplished by setting the hydraulic conductivities of the elements representing the dike and slurry wall to 0.0 ft/d. By making the dike and slurry wall impermeable to flow, all of the upgradient inflow was diverted around the detention basin after the water levels within the basin had risen to the levels at the upgradient end of the dike (fig. 13A). Figure 13B shows the simulated flow field as a result of the impermeable dike and slurry wall. The usefulness of this simulation is not limited to its application here with an impermeable dike. If a confining layer of low hydrologic conductivity is assumed to occupy the top part of the aquifer, the resulting water levels and corresponding flow field would be similar to those for an impermeable dike, but perhaps not as pronounced. A silty clay of low hydrologic conductivity does, in fact, occupy the uppermost part of the aquifer (fig. 5) and may cause an increase in water levels behind the dike and slurry wall, resulting in a diversion of some flow around the dike and perimeter.

The third simulation differed from the second in that it permitted water to over-top the dike when the water body within the detention basin was more than 6 ft deep adjacent to the downgradient leg of the dike (this depth coincides with the top of the dike along its downgradient leg). To accomplish this, the computer program was modified (see appendix A) by incorporating a conductance term into the specified-pressure (head) boundary at the top of the dike. The conductance represents the relative resistance to flow and the pressure boundary was selected to equal the altitude of the top of the dike. A conductance value of 1.0 represents no resistance to flow, whereas a value of 0.0 represents no-flow conditions. This modification allowed water levels to rise behind the dike structure until the head within the detention basin reached the height of the dike (the specified value of the pressure-boundary condition). The program then changed the conductance from 0.0 to 1.0 to allow water to flow over the top of the dike (figs. 14A and B). Simulation results indicate that flow was not inhibited from entering the detention basin as long as water was permitted to over-top the dike. In addition, ground water was not deflected around the detention basin in the simulation when over-topping was permitted. Only immediately behind the dike were water levels higher than during pre-slurry wall conditions. This scenario probably is the most accurate representation of how the dike and proposed slurry wall would work.

These three simulations do not address the effects of the migration and distribution of salts, heterogeneities and anisotropy of aquifer properties, surface-water inflow, or seasonal variations in evapotranspiration. These effects are addressed in the following sections that describe cross-sectional models of the Whitney area.

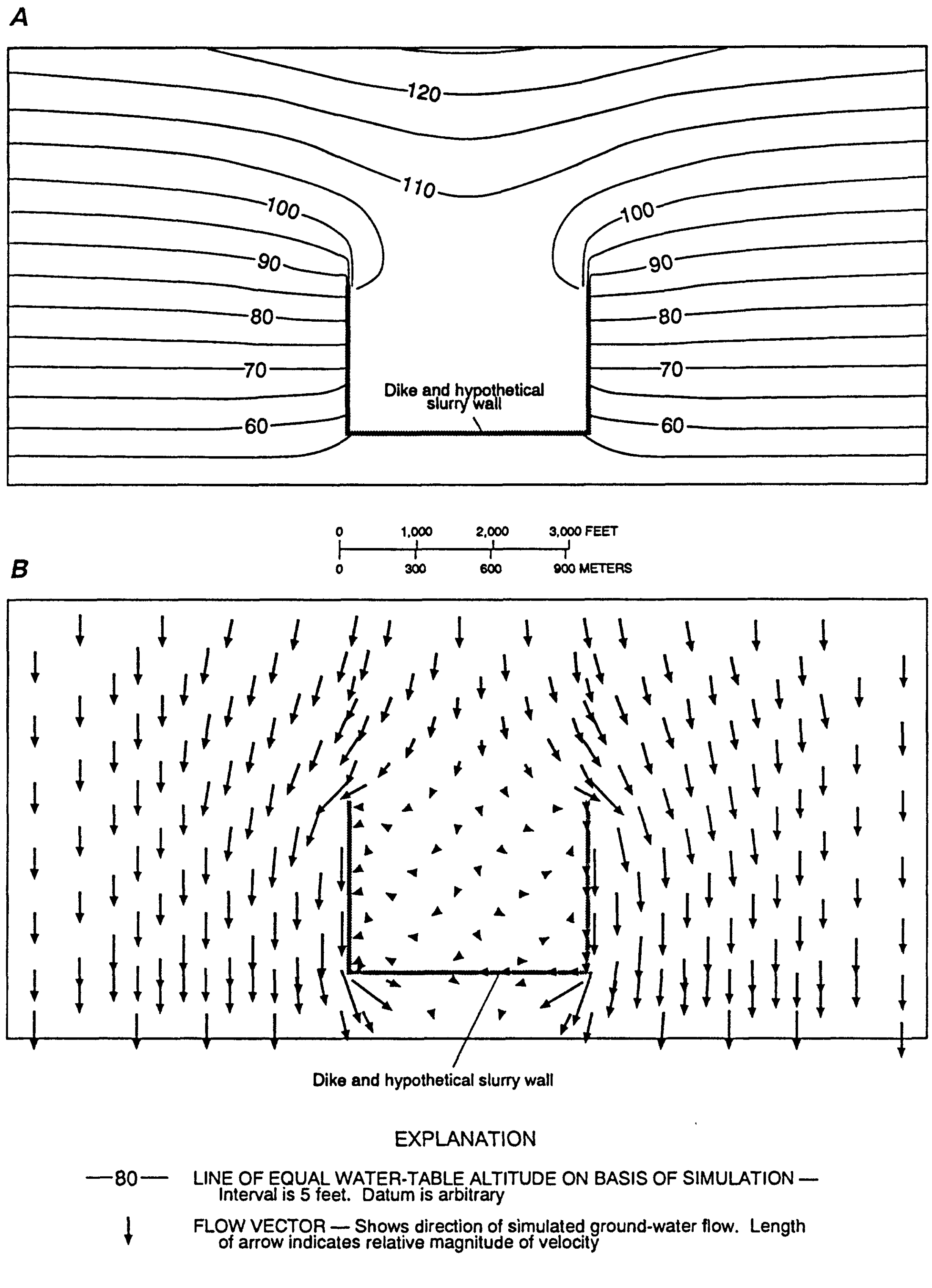
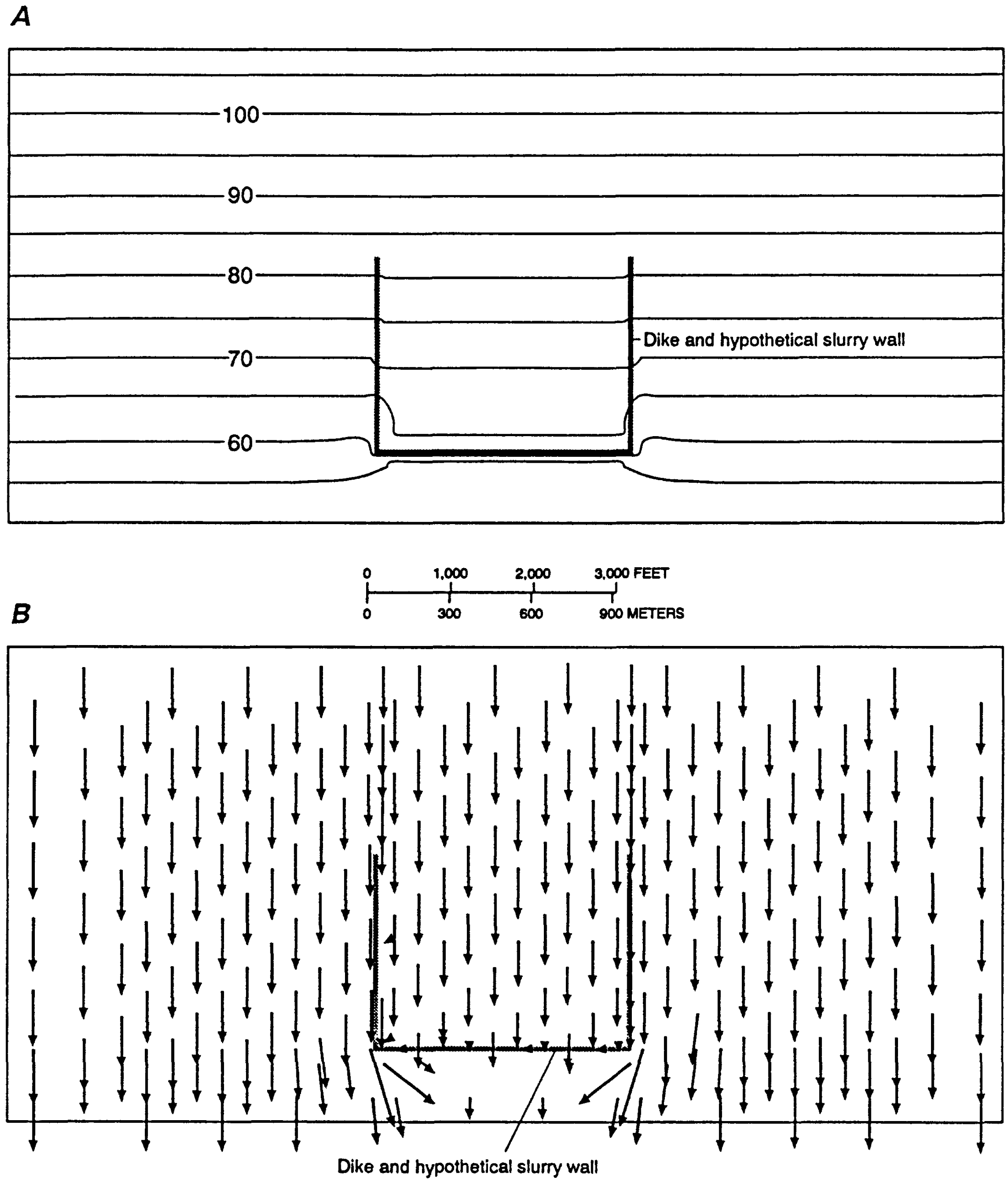


FIGURE 13.--Simulated planimeteric water-table configuration (*A*) and resulting flow pattern for steady-state conditions (*B*), assuming a vertically infinite no-flow boundary coincident with location of dike and hypothetical slurry wall.



EXPLANATION

—80— LINE OF EQUAL WATER-TABLE ALTITUDE ON BASIS OF SIMULATION — Interval is 5 feet. Datum is arbitrary

FLOW VECTOR — Shows direction of simulated ground-water flow. Length of arrow indicates relative magnitude of velocity

FIGURE 14.--Simulated planimeteric water-table configuration (*A*) and resulting flow pattern for steady-state conditions (*B*), with dike and hypothetical slurry wall in place. Top of downgradient dike segment is 6 feet above land surface; height of upgradient segments tapers to land surface at points of farthest upgradient extent.

Cross-Sectional Model of Flow and Solute Transport

Cross-sectional models were developed to determine how the proposed slurry wall and how aquifer and solute properties could affect the migration and distribution of salts in the shallow aquifer of the Whitney area. Under the proposed strategy, the slurry wall was intended to inhibit flow near the base of the aquifer. Flow would occur primarily near the top of the aquifer and would also become fresher over time because most of the flow through the detention basin would occur well above the contact between the potentially salt-laden Muddy Creek Formation and the overlying alluvial aquifer.

The effectiveness of the slurry wall in decreasing the dissolved-solids concentrations (and loads) leaving the detention basin was examined in this cross-sectional model by assuming that the incoming ground water initially contained no dissolved solids but that the Muddy Creek Formation contained dissolvable salts. Under these conditions, albeit somewhat unrealistic, the effect of the proposed slurry wall could be readily observed, because concentration gradients from dissolving salts in the Muddy Creek Formation would be pronounced. Simulations were made to determine the effects of differing the dispersivity, the ratio of horizontal to vertical hydraulic conductivity (anisotropy), the volumetric ground-water inflow, and the aquifer thickness. The results of these tests were used to evaluate the effectiveness of the proposed slurry wall and were used, more specifically, to determine the aquifer or transport property that would be most detrimental to the success of the proposed strategy.

The conceptual model extends along segment X-Y of section W-Z in figures 4 and 11A. The top of the Muddy Creek Formation is just above the bottom of the model and contains a source of dissolvable salts; the water table represents the model top. Ground-water inflow enters the model area from the left upgradient boundary, whereas the slurry wall, with an outflow area at its top, represents the right boundary. Surface-water inflow or evapotranspiration were not included in this simulation, so that they would not complicate the interpretation of the simulation results of the individual aquifer and solute properties. The entire aquifer was assumed to have a constant hydraulic conductivity of 30 ft/d in all the simulations.

Constructing the Mathematical Model

A finite-element rectangular grid having 44 rows and 20 columns connected by 945 nodes was used to simulate ground-water flow and solute transport (fig. 15). Grid spacing was constant with elements having a length (in the direction of flow) of 164 ft and a thickness (vertical dimension) of 0.82 ft. The total length of the modeled region extends 3,280 ft upgradient from the slurry wall (which is the downgradient model boundary). The total thickness of the modeled region is 36 ft, with the upper 30 ft representing the alluvial aquifer and the bottom 6 ft representing the top part of the Muddy Creek Formation. Element thicknesses were selected to be small, to accurately simulate concentration gradients even when vertical mixing was on the order of molecular diffusion; nonetheless, some numerical dispersion may occur. Numerical dispersion is a numerical error that tends to artificially increase mixing due to the type of grid network used for the study. The finer the network, the more accurate the results and the less the numerical dispersion. Because the degree of vertical mixing was important in determining the effectiveness of the slurry wall, a small element thickness was chosen.

The boundary conditions included a constant solute source at the 21 nodes representing the base of the alluvial aquifer (topmost part of the Muddy Creek Formation; fig. 15). Each node was arbitrarily specified to have a concentration of 37,500 mg/L, which is similar to that of seawater. A no-flow boundary was used to represent the water table, which was virtually at land surface. This was considered to be a good approximation because heads remained fairly constant at the water table except immediately upgradient from the slurry wall. A constant atmospheric pressure was applied to the upper right node (node 945) because at least one specified pressure-boundary node needs to be incorporated to correctly calibrate the pressures at the remaining nodes in the model. The boundary along the left (upgradient) side of the grid was specified as having a constant fluid-source flux of 4.63×10^{-5} ft^3/s. This flux is equivalent to taking a volume of unit width along segment X-Y of section W-Z from the same area used to estimate the total

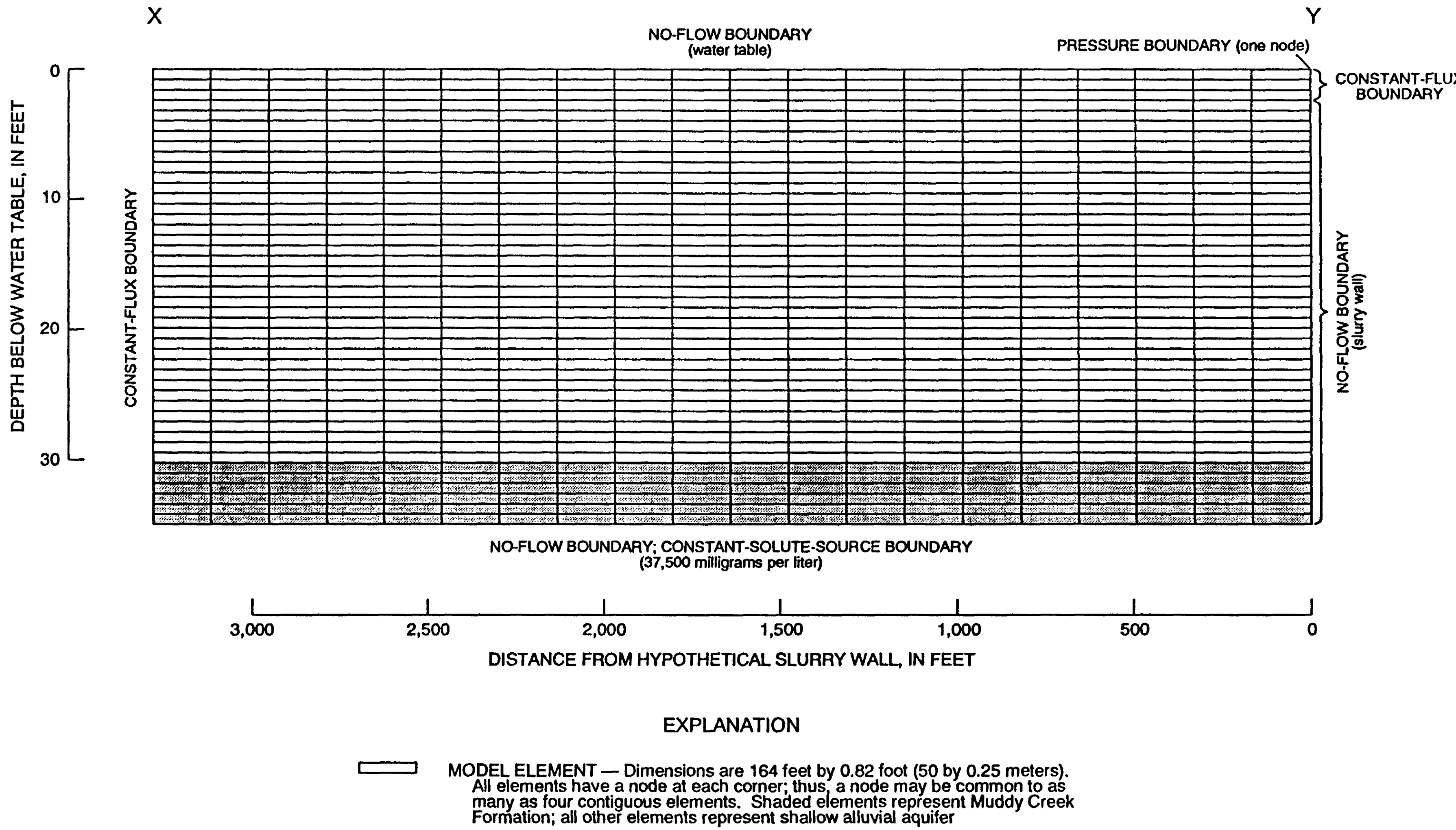


FIGURE 15.--Cross-sectional, finite-element grid used for model that simulates distribution of dissolved solids and ground-water flow. Line of section and points X and Y are shown in figure 4.

ground-water inflow (0.2 ft^3/s) to the detention basin. The fluid flux across each node was distributed on the basis of element thickness and hydraulic conductivity over the thickness of the aquifer. The boundary along the right (slurry-wall) side of the model grid was initially specified as a hydrostatic-pressure boundary corresponding to the water table, so that the steady-state flow field could be established and used as an initial condition for all transient simulations involving the slurry wall. Once the initial pressures for all model nodes were obtained, the right boundary was changed to a no-flow boundary extending upward from the base of the alluvial aquifer (bottom node) to a distance of 2.46 ft below the water table. This no-flow boundary simulates the effect that an impermeable slurry wall would have on the system. The four nodes above the no-flow boundary were specified as having a negative flux equal to the total inflow of 4.63×10^{-5} ft^3/s along the left (upgradient) boundary of the model grid. This type of boundary does not permit variations in flow; hence, the total outflowing salt load is influenced strictly by variations in dissolved-solids concentrations within the detention basin. A specified flux boundary was selected instead of a specified pressure boundary because pressures are density dependent and would, therefore, have to be changed manually during each time increment of the simulation as the solute concentration flowing across the boundary changed.

An initial aquifer hydraulic conductivity of 30 ft/d was specified in both the horizontal and vertical directions. An effective porosity of 30 percent was used for all simulations. A time step of 1 month was used for all simulations; a time step greater than 1 month caused convergence errors that are graphically similar to numerical dispersion.

Results of the Model Simulations

Results of the simulation are reported in three parts. The first part is a discussion describing how estimated properties affect the flow and dissolved-solids distribution within the detention basin and how these properties might influence the effectiveness of the proposed salt-reduction strategy. The second part is a discussion of the importance of aquifer thickness (that is, the scale dependence) in influencing the effectiveness of the proposed strategy. The final part is a discussion of the simulation results using field-estimated properties including evapotranspiration and Las Vegas Wash.

Effects of Varying Aquifer and Solute Properties

Varying the estimated properties of the aquifer and its solutes has profound effects on the migration and distribution of solutes in the shallow aquifer system. Simulation results made to analyze the effects of transverse dispersivity, anisotropy, and reduced inflow are tabulated in table 4. The first series of simulations involved changing the transverse dispersivity, which represents a mixing length perpendicular to the direction of flow; this property is dependent upon aquifer heterogeneities. Transverse dispersivity values of 0.0328 ft and 0.00328 ft were used to examine how mixing due to aquifer heterogeneities can influence the distribution of dissolved solids in the detention basin. A large difference in distribution resulted from differing values of transverse dispersivity (table 4). This large difference indicates that a more precise estimate of potential mixing is needed for the type of deposits found in the study area to ensure valid results.

Transverse or vertical spreading of a solute generally is small in comparison to longitudinal spreading and appears to be controlled mainly by a local dispersion coefficient on the order of magnitude of molecular diffusion. This conclusion is based on the small overall increases in the vertical extent of contaminants indicated in studies by Frind and Germain (1986) and Mackay and others (1986). The local dispersion coefficient, according to Sudicky (1986, p. 2073), is based on velocity fluctuations due to small-scale aquifer heterogeneities. Sudicky reported that field-scale transverse dispersivity probably is greater than stochastically determined dispersivity, the results of which often indicate that transverse mixing is solely by molecular diffusion. Thus, some transverse dispersivity may be important, especially in heterogeneous aquifers. Hence, a value of 0.0328 ft was used for all subsequent simulations, as it most closely represents the type of spreading pattern that would be expected for the alluvial deposits in the Whitney area.

TABLE 4.--*Effects of varying aquifer properties and boundary conditions on ground-water quality*

[Symbols: <, less than; --, not determined]

Aquifer property or boundary condition analyzed	Value chosen	Dissolved-solids concentration of ground-water outflow from study area (milligrams per liter), at end of indicated period of years			Percentage of aquifer containing at least 2,500 milligrams per liter of dissolved solids from hypothetical source at base of aquifer, at end of indicated period of years			
		5	25	50	0	5	25	30
Transverse dispersivity[1] (feet)	0.0328	5,000	7,500	12,500	0	20	50	65
	.00328	5,000	6,000	10,000	0	10	20	30
Anisotropy[2]	1:1	5,000	7,500	12,500	0	20	50	65
	1:10	<2,500	5,000	15,000	0	20	50	65
	1:100	--	4,500	--	0	--	50	--
	1:1,000	--	1,500	--	0	--	50	--
Ground-water inflow[3] (cubic feet per second, per unit width)	4.63×10^{-5}	5,000	7,500	12,500	0	20	50	65
	4.63×10^{-6}	--	3,000	--	0	--	15	--
Aquifer thickness[4] (feet)	30	5,000	7,500	12,500	0	20	50	65
	300	0	0	<100	0	5	15	25
Field-estimated properties and boundary conditions	(5)	0	<1,500	3,500	0	10	20	35

[1] Other values used: anisotropy, 1:1 (vertical and horizontal hydraulic conductivities, 30 feet per day); inflow, 4.63×10^{-5} cubic foot per second; aquifer thickness, 30 feet.

[2] Anisotropy is the ratio of vertical to horizontal hydraulic conductivity (unitless). Ratio of 1:1 indicates isotropic conditions. Other values used: transverse dispersivity, 0.0328 foot; inflow, 4.63×10^{-5} cubic foot per second; aquifer thickness, 30 feet.

[3] Other values used: anisotropy, 1:1 (vertical and horizontal hydraulic conductivities, 30 feet per day); transverse dispersivity, 0.0328 foot; aquifer thickness, 30 feet.

[4] Other values used: anisotropy, 1:1 (vertical and horizontal hydraulic conductivities, 30 feet per day); transverse dispersivity, 0.0328 foot; inflow, 4.63×10^{-5} cubic foot per second for 30-foot thickness and 4.63×10^{-4} cubic foot per second for 300-foot thickness (to obtain equivalent velocities).

[5] Estimated values: anisotropy, 1:10; field-estimated hydraulic conductivities differed from place to place; transverse dispersivity, 0.0328 foot; inflow, 4.63×10^{-5} cubic foot per second; aquifer thickness, 30 feet.

In ground-water systems with moderate to high flow velocities, solute mixing by molecular diffusion usually is negligible compared to mixing caused by dispersion. For molecular diffusion to be an important mixing process in the cross-sectional model described herein, the product of the flow velocity and transverse dispersivity must be similar to the diffusion coefficient of the chloride ion (the dominant ion in the ground water of the Whitney area). That coefficient is 1.7×10^{-9} ft^2/s (Freeze and Cherry, 1979, p. 103). Simulation results indicate that the mean flow velocity is 1.3×10^{-6} ft/s. The product of the flow velocity and the selected range of values for transverse dispersivity estimated in this study (0.00328 to 0.0328 ft) yields values from 4.3×10^{-9} to 4.3×10^{-8} ft^2/s. The smaller value of these two is similar to the diffusion coefficient for chloride; this indicates that the smaller value of transverse dispersivity (0.00328 ft) would produce a similar amount of mixing as molecular diffusion. Larger values of transverse dispersivity tend to minimize the effects of molecular diffusion. However, even if molecular diffusion were the only mechanism influencing spreading in the vertical direction, the quantity of solutes flowing over the slurry wall in the simulations would still make the proposed strategy ineffective. The velocity distribution is such that flow would not be reduced near the bottom of the aquifer; instead, the flow would be diverted upward immediately adjacent to the slurry wall, carrying potentially high concentrations of dissolved solids from the bottom of the aquifer to the outflow area.

The second aquifer property selected in the analysis of aquifer and solute properties was anisotropy. Changes in anisotropy, or the ratio of vertical to horizontal hydraulic conductivity, had minor effects on the distribution of dissolved-solids concentrations in the simulations as listed in table 4. Virtually no differences occur along the flow path except at the slurry wall, where increased anisotropy lowered the dissolved-solids concentration of outflow from the model area.

Restricting outflow by decreasing the vertical hydraulic conductivity results in increased heads in areas upgradient from the slurry wall, indicating that the increased resistance to vertical flow may ultimately inhibit some flow from entering the detention basin. As mentioned earlier in the discussion of the planimetric model, a confining layer of low vertical hydraulic conductivity may result in the increase of heads behind the dike, and ultimately a diversion of flow around it. Results of simulations using four values of anisotropy spanning three orders of magnitude indicated that heads increased only minimally at a distance of 3,280 ft from the slurry wall (at cluster-well site WG035N) when the anisotropy was 1 to 10 (the estimated ratio for deposits in the Whitney area). However, results from simulations in which anisotropy was increased to 1 to 100 and 1 to 1,000 indicated that heads increased at cluster-well site WG035N by 0.43 and 2.00 ft, respectively. This in turn suggests that ground-water flow would be diverted around the dike under such conditions.

The quantity of ground-water inflow along a unit-width section of aquifer was decreased one order of magnitude below the estimated actual inflow (4.63×10^{-5} ft^3/s) to determine whether the reduced flux would in turn reduce the quantity of solutes discharged from the detention basin. Intuitively, a smaller volumetric flux would tend to reduce velocities near the base of the aquifer, and the reduction would inhibit the mixing and migration of salts from the underlying Muddy Creek Formation.

By decreasing inflow one order of magnitude, mixing becomes limited to molecular diffusion because dispersivity is velocity-dependent. Results suggest that a clockwise circular flow pattern is established adjacent to the slurry wall that carries solutes from the bottom of the aquifer over the top of the slurry wall. The circular pattern of flow is caused by mixing along the concentration gradient. Consequently, water in the area where mixing occurs becomes less dense than the underlying saline water, causing the zone of mixed water to move upward into the flow of fresher water. The processes that cause this circular flow are convection and molecular diffusion (Cooper, 1964). The result of this circular action forces more saline water upward and over the slurry wall, and at the same time initiates transverse mixing perpendicular to the circular flow cell. In simulations where dispersivity is increased to values greater than molecular diffusion, larger circular flow patterns are produced, causing mixing farther upgradient from the slurry wall. In previous simulations where flow was not reduced, the larger flow velocities tend to inhibit diffusive forces that help to initiate this circular flow pattern. Results reported in table 4 suggest that reduced flow does not decrease the dissolved-solids concentrations leaving the detention basin, nor does it create a transverse velocity gradient where lower flow velocities develop near the bottom of the aquifer.

Effects of Aquifer Thickness

A principal deterrent to the effectiveness of the proposed detention basin in reducing salt contributions to Las Vegas Wash is aquifer thickness. To impede contamination of the upper parts of the aquifer by upward-migrating salts, as proposed, requires that the slurry wall create almost stagnant conditions near the base of the aquifer within the detention basin. However, because the lateral hydraulic gradient is nearly flat (slope about 1 percent) and the aquifer is thin relative to the flow-path length within the basin (length is 100 times greater than thickness; fig. 11B), the slurry wall would not be expected to appreciably affect flows and distribution of salts upgradient from it.

In this simulation, the aquifer thickness was increased from 30 to 350 ft to determine whether the detention basin would then function as proposed. The aquifer properties and initial conditions used for this simulation were the same as those of the previous simulation--a solute source was specified at the base of the aquifer, the upgradient inflow rate was increased to 4.63×10^{-4} ft^3/s to keep the flow velocities the same as those for the thin aquifer, the vertical hydraulic conductivity was selected to be equal to the horizontal value, and the transverse dispersivity was set to 0.0328 ft.

Simulation results after 25 years, assuming an isotropic aquifer, are shown in figure 16A and 16B Some numerical dispersion in the transverse (vertical) direction would be expected, because the thicknesses of the elements have been increased by an order of magnitude. The increased thicknesses have a tendency to smooth out the distribution of dissolved-solids concentrations across the gradient. The 2,500-mg/L line extends through only about one-third of the aquifer adjacent to the slurry wall. The absence of salts discharging from the detention basin is due to the changed outflow area, which is 350 ft above the bottom of the aquifer in this simulation. The increased thickness has had the effect of reducing the flow velocities at the bottom of the aquifer by slightly less than one order of magnitude at a point half-way between the slurry wall and the upgradient inflow boundary.

After an additional 25 years, the distribution of dissolved solids remained almost the same as the distribution after the first 25 years, suggesting that the reduced flow velocities have inhibited transverse dispersion and convectional mixing. Because of the increased aquifer thickness, this mixing process does not significantly affect the distribution of dissolved solids over the 50-year period. Thus, a much thicker aquifer could reduce flow velocities near the base of the aquifer, and as a result, could inhibit solute outflow from the detention basin for a long time period, leading to reduced salt loads to Las Vegas Wash. Hence, if the solute source were solely the Muddy Creek Formation, and if the aquifer were considerably thicker, the slurry wall might be an effective salt-reduction strategy.

Effects of Using Field-Estimated Hydrologic Properties

A final simulation was made with the mathematical model using field-estimated horizontal and vertical hydraulic conductivity, dispersivity, and upgradient inflow. Hydraulic-conductivity values were estimated on the basis of slug-test analyses. An average dispersivity was assigned on the basis of aquifer heterogeneities and particle-size distributions. Inflow was assigned to the upgradient column of elements on the basis of the estimated total along a unit section, and was distributed among the elements receiving inflow on the basis of their horizontal hydraulic conductivity (that is, elements with high hydraulic conductivity had more inflow assigned to them than elements with low hydraulic conductivity).

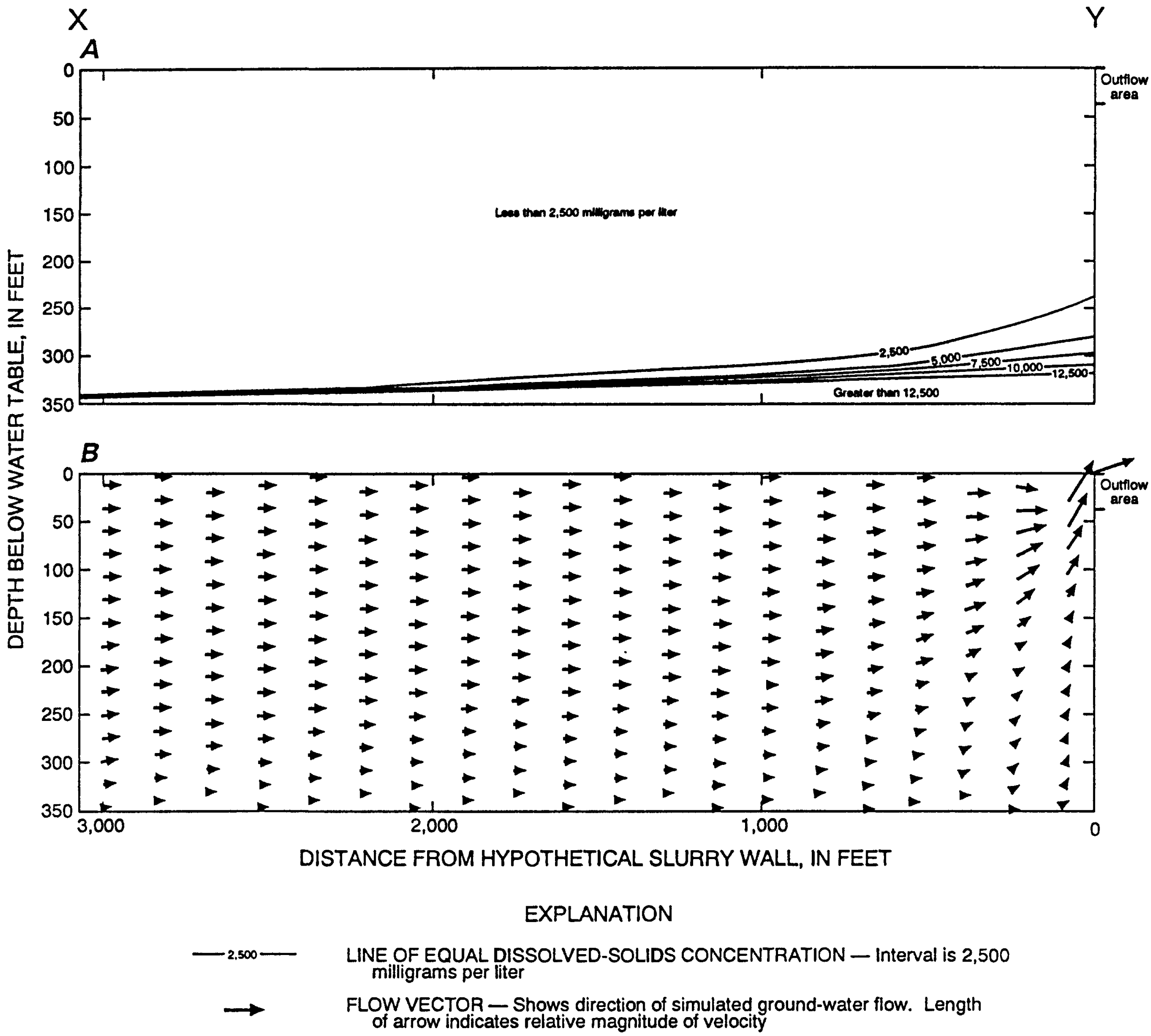


FIGURE 16.--Cross-sectional distribution of (*A*) dissolved solids and (*B*) ground-water flow after 25-year simulation for hypothetical aquifer 360 feet thick. Assumptions: Horizontal and vertical hydraulic conductivities are 30 feet per day; transverse dispersivity is 0.0328 foot; inflow is 4.63×10^{-4} cubic foot per second. Line of section and points X and Y are shown in figure 4.

Two significant conclusions were made on the basis of simulation results shown in figure 17. First, the greatest velocities and most of the ground-water flow were in the mid-depth part of the aquifer (fig. 17D). Consequently, the quantity of inflow in contact with the underlying Muddy Creek Formation, which was suspected of contributing a large quantity of dissolvable salts to the shallow aquifer system, was negligible. Hence, the high dissolved-solids concentrations of the shallow ground water probably are derived from salt-bearing deposits within the aquifer, rather than from the Muddy Creek Formation as hypothesized. Second, the simulated dissolved-solids concentrations discharged from the detention basin were less than for other simulations where a thin 30-ft aquifer was assumed. This was attributed to the prevalence of deposits of low hydraulic conductivity near both the base and top of the aquifer. These zones of low conductivity tend to inhibit mixing by dispersion. The simulation indicated that the slurry wall apparently had no effect on ground-water flow or solute mixing except immediately adjacent to it (fig. 17C-D).

Model Incorporating Evapotranspiration and Las Vegas Wash into the Cross Section

The cross-sectional model discussed above was modified to simulate the effect of evapotranspiration on ground-water flow and solute movement in the Whitney area to account for the discharge of shallow ground water by evapotranspiration during hot summer months. In addition, the model was extended from the site of the proposed slurry wall to Las Vegas Wash to investigate potential changes in flow and dissolved-solids concentrations downgradient from the proposed wall. Analysis of field data suggests that evapotranspiration is the driving mechanism for the accumulation of salts near land surface and within the top part of the aquifer. Naff and others (1975) concluded that phreatophyte transpiration concentrates salts in and above the capillary fringe during the growing season; then, the salts are redissolved in the fall and early winter when the water table rises. The temporary buildup and subsequent flushing of salts near the water table would diminish the effectiveness of the slurry wall in reducing the concentration and load of dissolved solids leaving the detention basin. Ground-water flow may be reduced downgradient from the slurry wall owing to increased evapotranspiration within the detention basin, leading to a temporary reduction in the quantity of solute transported to the wash. However, surface flows from subsequent storms could redissolve the salt residue that had accumulated near land surface, resulting in an increase of the salt load normally carried to the wash during such periods.

The intent of this third and final conceptual model was (1) to verify that processes of evapotranspiration described in earlier sections of this report are influencing the distribution of flow and solutes in the shallow aquifer, and (2) to determine how construction of the proposed slurry wall would affect the distribution of flow and solutes in and downgradient from the detention basin.

Modifying the Existing Mathematical Model

A finite-element grid was elongated to include the cross-sectional area extending through the entire thickness of the aquifer from cluster-well site WG035 through cluster-well site WG071 to Las Vegas Wash, which is about 500 ft southeast of well site WG071 (figs. 4 and 11). The grid contains 10 rows and 40 columns of elements connected by 473 nodes. The elements are variably spaced, as indicated in figure 18. The upper rows are narrow, to present a more accurate simulation of concentration gradients during the evapotranspiration process. The right (downgradient) side of the grid (fig. 18) has thinner elements, to simulate the seepage face associated with the waste-water channel representing Las Vegas Wash.

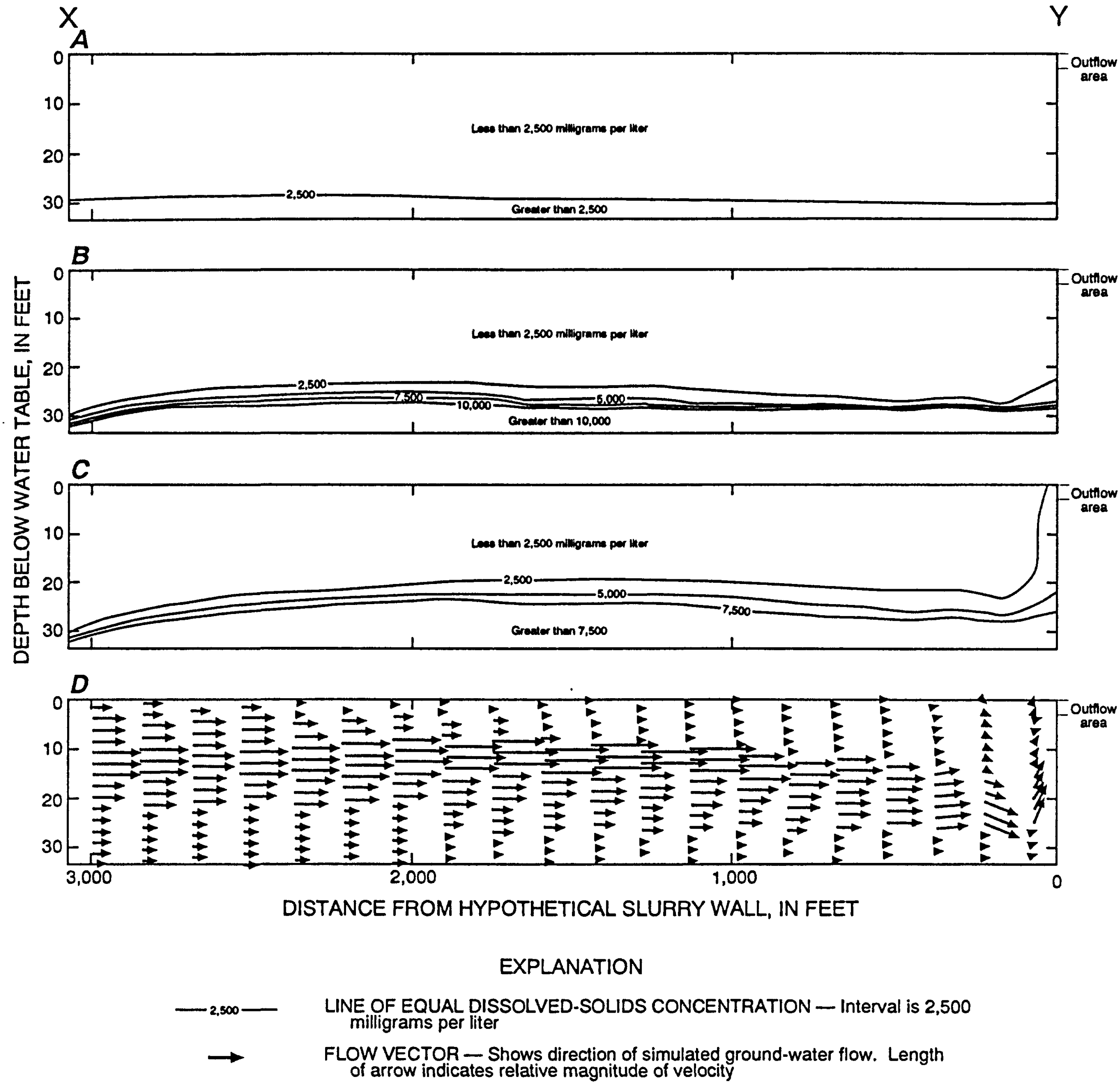
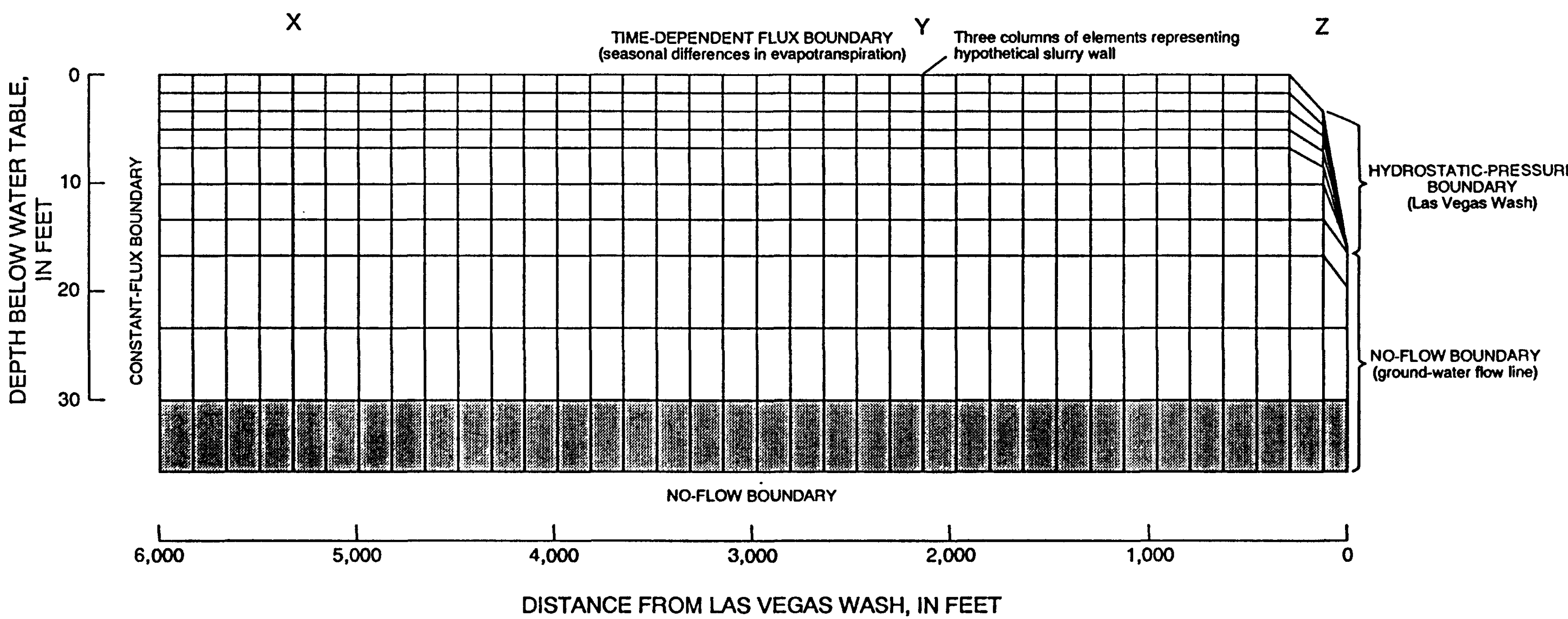


FIGURE 17.--Cross-sectional distribution of (*A*-*C*) dissolved solids after 5-, 25-, and 50-year simulations, and (*D*) ground-water flow pattern after 25-year simulation. Assumptions: Field-estimated horizontal hydraulic conductivity; vertical hydraulic conductivity is one-tenth the estimated horizontal hydraulic conductivity; transverse dispersivity is 0.0328 foot; inflow is 4.63×10^{-5} cubic foot per second. Line of section and points X and Y are shown in figure 4.



EXPLANATION

MODEL ELEMENT — Longitudinal dimensions are 164 feet (50 meters) except for elements representing hypothetical slurry wall, which are 3.28 feet (1.0 meter). Vertical dimension ranges from 1.64 to 6.56 feet (0.5 to 2.0 meters), except adjacent to Las Vegas Wash. All elements have a node at each corner; thus, a node may be common to as many as four contiguous elements. Shaded elements represent Muddy Creek Formation; all other elements represent represent shallow alluvial aquifer

FIGURE 18.--Cross-sectional, finite-element grid used for model that incorporates evapotranspiration and Las Vegas Wash to simulate distribution of dissolved solids and ground-water flow. Line of section and points X, Y, and Z are shown in figure 4.

Boundary conditions associated with the model included, as before, a constant-flux boundary on the left side of the grid representing the upgradient inflow area. The inflow of 4.63×10^{-5} ft^3/s was divided among 11 nodes according to element size and hydraulic conductivity. The right side of the model beneath the seepage face in Las Vegas Wash was specified as a no-flow boundary from the base of the aquifer to the streambed of the wash. A hydrostatic-pressure boundary was used to represent the seepage face. The bottom of the grid was specified as a no-flow boundary (as in the previous model), because vertical flow through the Muddy Creek Formation is assumed to be negligible. The nodes representing the boundary along the top of the grid were modified to incorporate the evapotranspiration process. The modification had to take into account (1) ground-water consumption by phreatophytes and hydrophytes during the growing season, (2) chemical effects on ground water due to the precipitation and dissolution of salts in and above the capillary fringe during water-table fluctuations, (3) surface-water inflow at the inlet structure (fig. 3) from the Monson Road floodway, and (4) near-surface inflow of treated effluent. Both (3) and (4) affect primarily the areas dominated by hydrophytes.

A time-dependent flux boundary in the form of a sinusoidal function was used to simulate seasonal changes in the quantity of evapotranspiration along the uppermost nodes in the model grid. Figure 19 illustrates how the fluid flux was allowed to vary along the boundary. One complete cycle of the sine function represents a full year, with the minimum and maximum values corresponding to winter and summer, when evapotranspiration is least and greatest, respectively. In areas dominated by phreatophytes (areas upgradient from cluster-well site WG053), the rate of ground-water flow and concentration of dissolved solids depend upon the rates of evapotranspiration (which vary seasonally) and ground-water inflow. In areas dominated by hydrophytes (areas downgradient from cluster-well site WG053), distributions of flow and dissolved-solids concentration depend not only on evapotranspiration and ground-water inflow but also on surface-water inflow.

Evapotranspiration is greatest in late summer and least in late winter for both phreatophyte and hydrophyte dominated areas. To simulate the late summer period, evapotranspiration was set equal to the quantity of upgradient subsurface inflow in phreatophyte-dominated areas, and was set equal to one half the quantity of upgradient subsurface inflow in hydrophyte-dominated areas. To simulate the late winter period, no recharge or evapotranspiration at land surface was used in the phreatophyte-dominated areas; whereas recharge equal to one half the upgradient subsurface inflow was used at land surface in the hydrophyte-dominated areas. The shapes of the phreatophyte-dominated and hydrophyte-dominated curves are identical; only the rates differ (fig. 19). The recharge simulated in hydrophyte areas during late winter was assigned a constant dissolved-solids concentration of 3,000 mg/L, approximately equivalent to the concentration of surface or near-surface water entering the detention basin. Two major program modifications, documented in appendix B, were necessary to simulate the sinusoidal evapotranspiration-recharge curve discussed here. The first modification permits no solute-mass discharge from the top row of elements; only fluid mass is allowed to be discharged along the top of the model. The second modification describes the sinusoidal time-dependent boundary at the water table.

The time-dependent flux boundary probably simulates the changes in fluid mass and dissolved-solids concentration from year to year more accurately than it does from season to season. The seasonal inaccuracies in simulated dissolved-solids concentration near the water table result because the unsaturated zone, which includes the capillary fringe, was not accounted for during simulations. Thus, the precipitation of salts in the capillary fringe during the lowering of the water table, and the dissolution of salts from the capillary fringe in the subsequent winter months, were not simulated. Over a full-year period, however, the net loss of solute mass is probably near zero, whereas the net loss in fluid mass by evapotranspiration is significant, as graphically represented in figure 19.

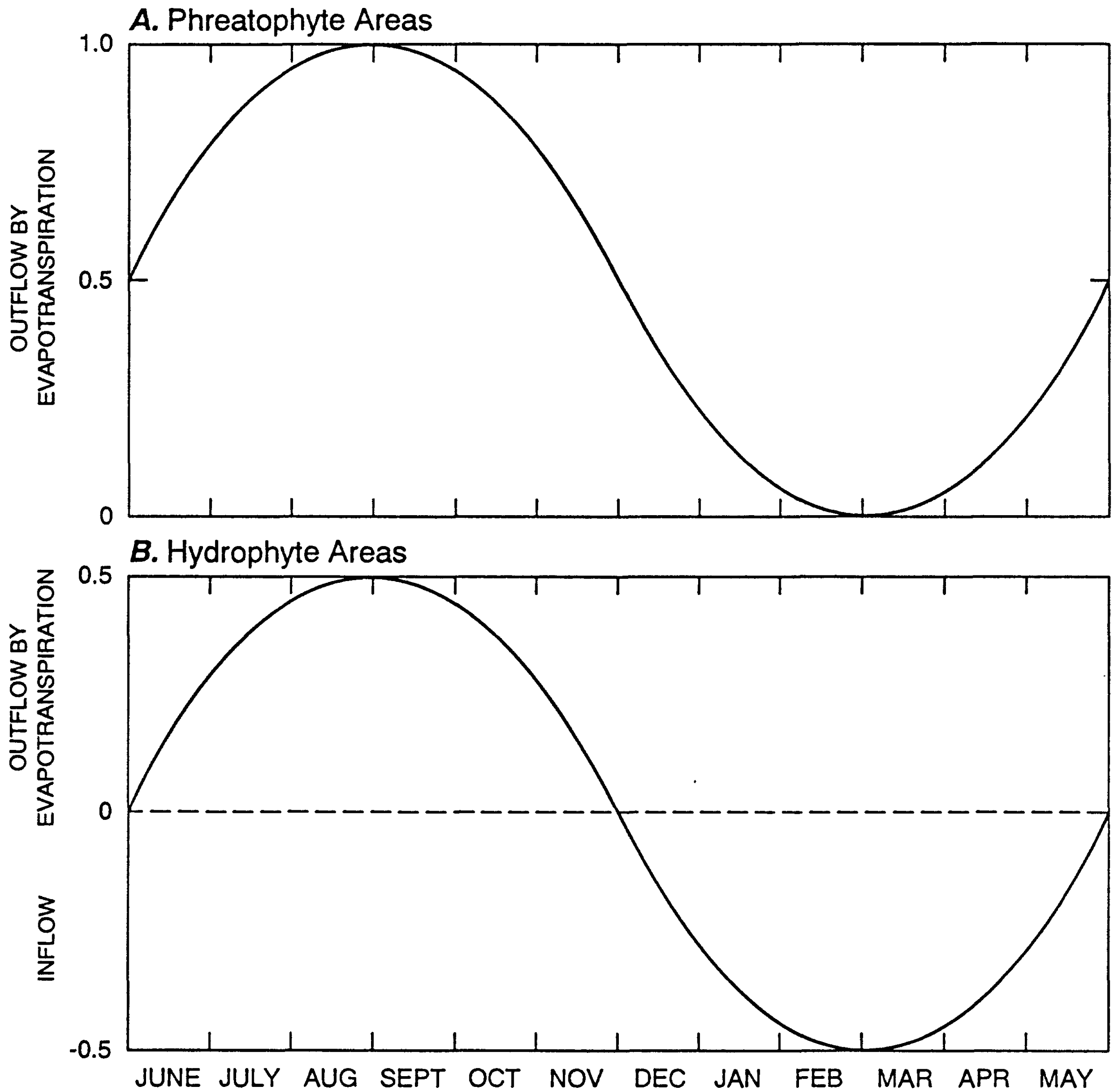


FIGURE 19.--Relation between estimated inflow and total simulated seasonal outflow by evapotranspiration for (*A*) phreatophyte areas and (*B*) hydrophyte areas. One unit of outflow is equivalent to the total subsurface inflow, which is estimated as 4.63×10^{-5} cubic foot per second.

The initial conditions used for the simulations included measured values of dissolved solids obtained from samples at cluster-well sites, ground-water inflow estimated from slug-test analyses, a transverse dispersivity of 0.0328 ft, field-estimated values of hydraulic conductivity, and, as mentioned above, field-estimated rates of evapotranspiration and surface-water inflow. The simulations were executed for a 10-year period, a time-frame considered long enough to verify whether specified boundary and initial conditions were accurate, and long enough to analyze effects of the hypothesized slurry wall on the distributions of flow and dissolved-solids concentrations. Because of the short time step used (1 month), simulations executed for a longer overall period would have been impractical.

The first simulation discussed did not include the effects of the proposed slurry wall in order to verify that the mathematical model sufficiently matched measured and estimated properties. The second simulation examined the effectiveness of the proposed strategy (including the slurry wall) for the Whitney area.

Results of the Model Simulations

Results of the first simulation indicate a sharp gradient of dissolved-solids concentration, similar to the observed gradient (fig. 10), near the top of the aquifer in areas upgradient from cluster-well site WG053 (fig. 20). Simulation results indicate that evapotranspiration influences the distribution of dissolved-solids concentrations more than do possible salt-bearing deposits of the Muddy Creek Formation. Similarly, both the simulated and observed dissolved-solids distributions downgradient from cluster-well site WG053 are

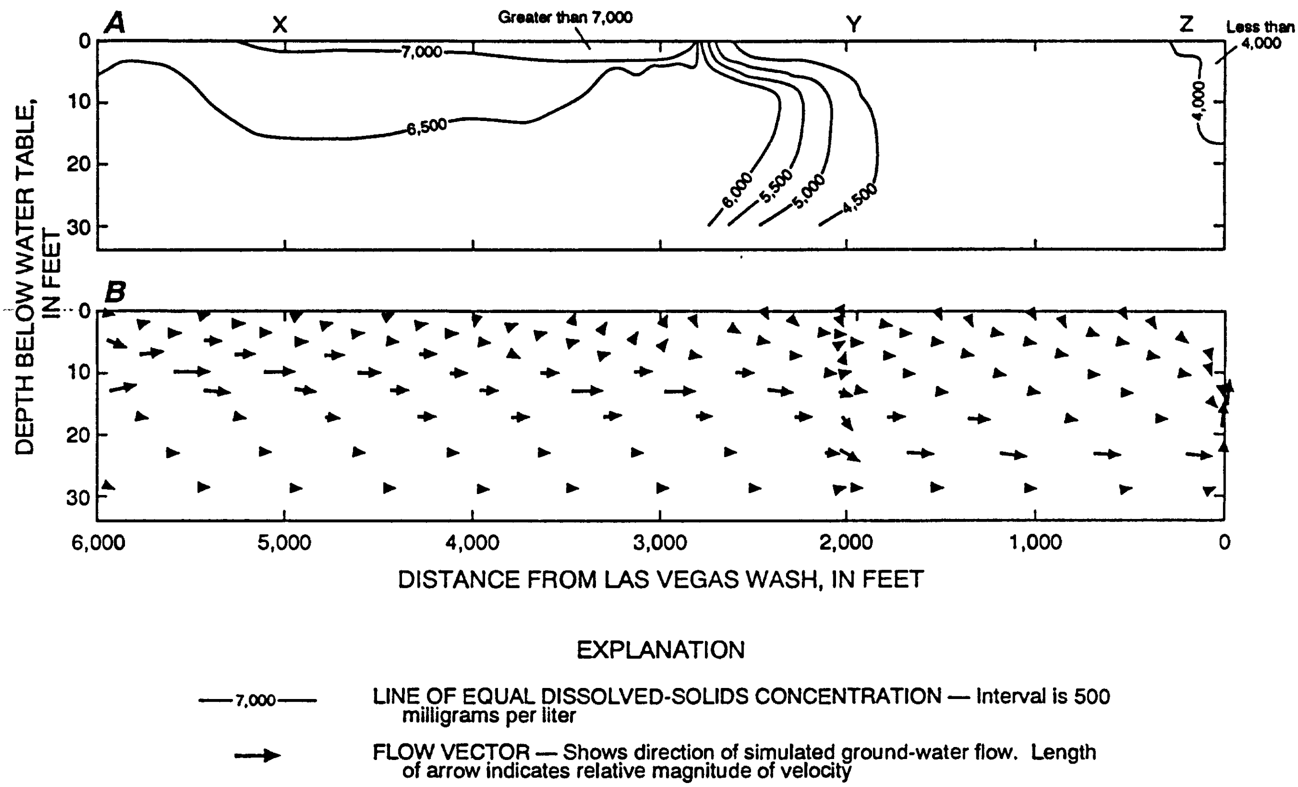


FIGURE 20.--Cross-sectional distribution of (*A*) dissolved solids and (*B*) ground-water flow after 10-year simulation without hypothetical slurry wall. Assumptions: Initial dissolved-solids concentration is 6,200 milligrams per liter beneath phreatophyte areas and 4,300 milligrams per liter beneath hydrophyte areas; incorporates time-dependent flux boundary; horizontal hydraulic conductivity is 30 feet per day; vertical hydraulic conductivity is 3 feet per day; transverse dispersivity is 0.0328 foot; inflow is 4.63×10^{-5} cubic foot per second. Line of section and points X, Y, and Z are shown in figure 4.

uniform throughout the thickness of the aquifer, at about 4,500 mg/L. These lower and uniform concentrations point to the importance of mixing surface and ground waters in the flood-plain area that is dominated by hydrophytes. As in upgradient areas, the influx of fresher water in this area tends to flush out the salts that would otherwise accumulate due to evapotranspiration. Table 5 lists the measured and simulated dissolved-solids concentrations at discrete depth intervals at the cluster-well sites. Comparisons suggest that the initial and boundary conditions used in the mathematical model are sufficiently accurate to simulate the hydrogeologic setting of the Whitney area reasonably well. Therefore, the same model can be used to examine the effects the proposed slurry wall could have on the distribution of flow and dissolved-solids concentrations within and downgradient from the detention basin.

TABLE 5.--*Comparison of measured and simulated dissolved-solids concentrations for selected depth intervals at cluster-well sites*

		Dissolved-solids concentration (milligrams per liter)		
Site and cluster-well designation	Depth interval (feet below land surface)	Average of measured values for 1986-87[1]	Simulated after 10 years[2]	Simulated dissolved solids, as percentage of measured value
WG035AN	4-6	8,500	7,500	88
BN	8-10	6,800	6,500	96
CN	13-15	6,200	6,300	102
DN	16-18	6,000	6,200	103
EN	28-30	5,900	6,100	103
WG044AN	6-8	6,900	6,900	100
BN	10-12	6,500	6,500	100
CN	20-22	6,400	6,400	100
DN	31-33	6,700	6,300	94
WG053AN	6-8	7,800	7,900	101
BN	10-12	6,600	6,400	97
CN	16-18	6,200	6,300	102
DN	18-20	6,200	6,300	102
EN	23-25	6,100	6,300	103
WG062A	4-6	4,900	4,300	88
B	10-12	4,000	4,300	108
C	15-17	4,000	4,300	108
D	20-22	4,300	4,300	100
E	26-28	4,300	4,300	100

[1] Sampling dates: cluster-well sites WG035N, WG044N, and WG053N, October 1986, January and June 1987 (no October sample for well WG035N); site WG062, March and June 1987.

[2] Simulation based on cross-sectional model that incorporates evapotranspiration, surface-water inflow, and Las Vegas Wash, but not the hypothetical slurry wall.

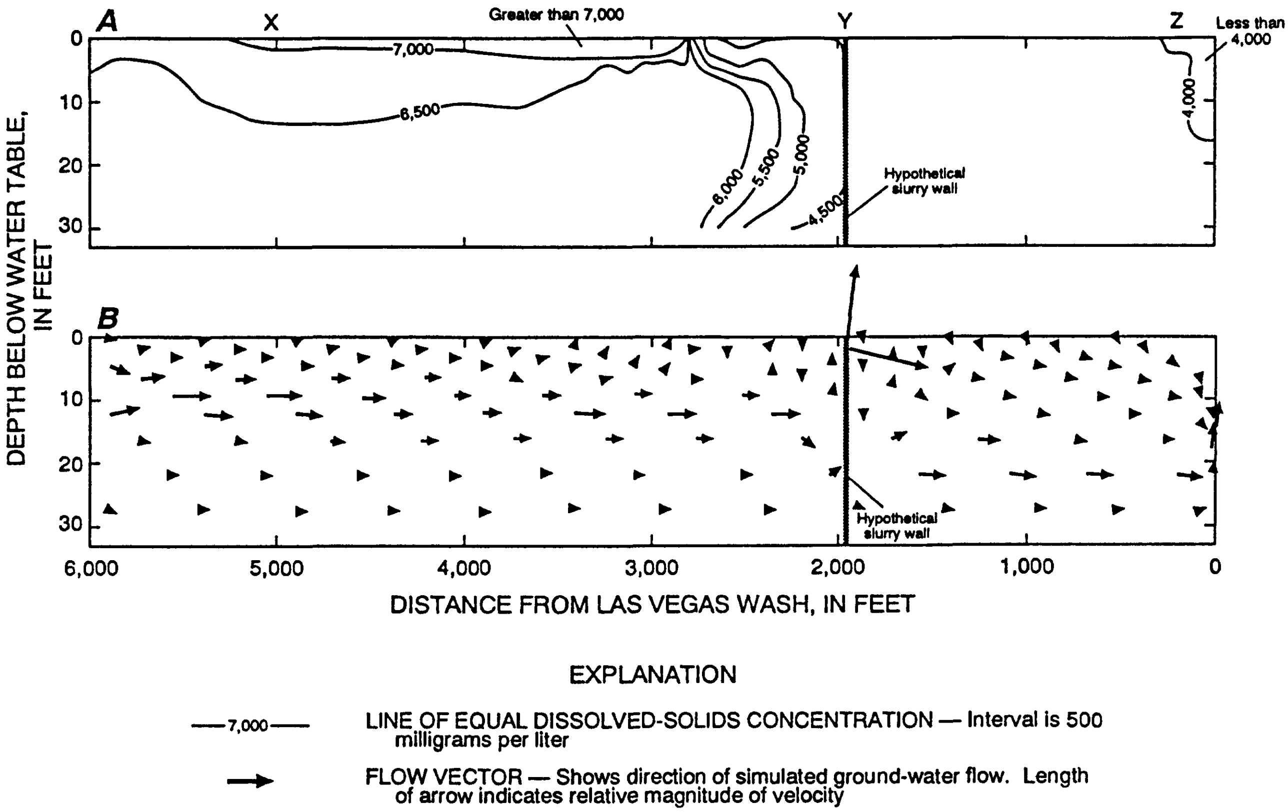


FIGURE 21.--Cross-sectional distribution of (*A*) dissolved solids and (*B*) ground-water flow after 10-year simulation with hypothetical slurry wall. Assumptions: Initial dissolved-solids concentration is 6,200 milligrams per liter beneath phreatophyte areas and 4,300 milligrams per liter beneath hydrophyte areas; incorporates time-dependent flux boundary and field-estimated horizontal hydraulic conductivity; vertical hydraulic conductivity is one-tenth the estimated horizontal hydraulic conductivity; transverse dispersivity is 0.0328 foot; inflow is 4.63×10^{-5} cubic foot per second. Line of section and points X, Y, and Z are shown in figure 4.

Results from the first simulation, after a 10-year period, were used as initial conditions for the final simulation, which incorporated the hypothetical, impermeable slurry wall. The top two rows of elements above those representing the wall were assigned a high hydraulic conductivity to allow flow over the top of the wall. Figure 21 shows the results of the final simulation after a 10-year period. These results indicate that the slurry wall tends to inhibit down gradient movement of dissolved solids in the middle part of the aquifer, where flow velocities are greatest. Immediately upgradient from the slurry wall, dissolved-solids concentrations at the water table increased as a result of upward flow along the impermeable structure. However, the slurry wall had little effect on dissolved-solids concentration distributions in the Whitney area. Ground-water flow velocities were not reduced near the base of the aquifer as hypothesized in the strategy; nor was flow deflected upward toward the water table owing to the presence of the wall. In fact, ground-water flow through the aquifer was unaffected by the presence of the slurry wall until it reached the wall and was forced to flow upward. Intuitively, these results can be understood more readily with the aid of a cross section along the flow path that has no vertical exaggeration (fig. 11B). In summary, the conclusions suggest that the proposed slurry wall would not decrease the total dissolved-solids load leaving the detention basin.

SUMMARY AND CONCLUSIONS

The rapid population growth in Las Vegas Valley has led to increased discharge of treated sewage effluent to the Las Vegas Wash wasteway channel. This increased discharge has caused extensive erosion of the Wash, which in turn has induced seepage of shallow, saline ground water. The alluvial-fan and flood-plain deposits that constitute the shallow alluvial aquifer contain natural salt deposits that are readily dissolved by flowing ground water. The Salinity Control Act of 1974 authorized the U.S. Bureau of Reclamation to develop strategies to control the salt load entering the Colorado River from Las Vegas Wash.

Several strategies to reduce the quantity of saline ground water entering Las Vegas Wash have been proposed by the U.S. Bureau of Reclamation. One such strategy would involve the construction of a surface impoundment, or detention basin, consisting of a dike structure above ground and a slurry wall beneath the dike; the slurry wall would extend to the base of the alluvial aquifer adjacent to the wash in the Whitney area of southeast Las Vegas. The intent of the slurry wall would be to induce surface flow at a greater distance from the wash than under current conditions. As a result, incoming ground water theoretically would be deflected upward due to the slurry wall and would become increasingly fresh, because it would no longer be in contact with the Muddy Creek Formation immediately beneath the aquifer. The Muddy Creek Formation has been identified as a major source of salt-bearing minerals responsible for high dissolved-solids concentrations in the shallow ground water. Thus, more dense saline ground water near the bottom of the alluvial aquifer ultimately would become a stagnant pool, impounded by the slurry wall, while fresher, less dense water would flow over the top of the more saline water. The fresher water would be allowed to flow from the detention basin into the wash or into an adjacent detention basin.

The purpose of the study reported herein was to determine, from analysis of field data and results of simulations using flow and solute-transport models, whether the proposed slurry wall would be effective in reducing the load of salts entering the wash. Water-level data were used to determine seasonal and yearly trends in levels resulting from evapotranspiration and nearby pumping. Slug tests were made at selected cluster-well sites to estimate hydraulic conductivity at discrete intervals through the thickness of the alluvial aquifer. These estimates were used to interpolate hydraulic conductivity through the entire thickness and along a section of aquifer within the detention basin, and were later used in simulations. Water-quality data were analyzed to determine the distribution of dissolved solids along a cross section of the aquifer. Soil samples were collected at specified depth intervals for particle-size and bulk x-ray analysis. Core samples were collected to estimate specific yield and porosity near the water table.

Analysis of field data indicates that the alluvial aquifer is about 30-ft thick and is underlain by a clay-dominated unit tentatively identified as the Muddy Creek Formation. Estimated hydraulic conductivities suggest that a zone of relatively high conductivity (20-150 ft/d) occupies the middle part of the aquifer. Low hydraulic conductivities (0-5 ft/d) dominate the top and bottom parts of the aquifer. A decrease in hydraulic conductivity toward the wash tends to corroborate particle-size data that indicate a trend from alluvial-fan to flood-plain deposits in the study area. X-ray analyses of sediment-core samples indicate that the main source of dissolved solids is dissolution of gypsum that is common in the alluvial deposits of the study area. Water-quality data indicate that the highest dissolved-solids concentrations generally are in the upper part of the aquifer. These high concentrations are due to evapotranspiration, which causes seasonal water-table fluctuations of as much as 3.5 ft and leads to the precipitation of salts near land surface in the summer months and dissolution of these salts during the winter months. Dissolved-solids concentrations of about 9,000 mg/L are common in the upper part of the aquifer during winter months, whereas the average concentration in the aquifer is about 6,000 mg/L.

Inflow to the Whitney study area includes ground water from the northwest, surface water from the Monson Road floodway, and treated effluent from the adjacent flood plain. The fresher surface water mixes readily with the shallow, more saline ground water and tends to dilute ground water in affected areas. Total flow into the proposed detention basin was estimated to range from 0.4 to 1.8 ft^3/s. Ground water is the principal component of inflow, although the surface-water increment can exceed ground-water inflow during periods of high water use in Las Vegas Valley. The surface-water inflow from the Monson Road floodway and adjacent flood plain supports extensive hydrophyte vegetation in the eastern part of the Whitney area. During summer, the rate of evapotranspiration approaches the rate of ground-water inflow to the area. On the basis of estimated inflow and evapotranspiration, the total salt load entering the wash from the Whitney area was about 2,700 ton/yr, or about 1 percent of the total load entering the wash from all sources. This indicates that a large proportion of the total salt load does not originate as ground-water seepage from the Whitney area.

Three conceptual and mathematical models were developed to examine potential effects of a detention basin (with slurry wall) on ground-water flow and dissolved-solids concentration and distribution in the Whitney area, and whether such a basin could potentially reduce salt loads to Las Vegas Wash. The computer program known as SUTRA (Saturated Unsaturated TRAnsport) was used to make steady-state and transient simulations for the three models. The first model was planimetric and was developed to determine the conditions that would cause flow to be diverted around the detention basin. The second and third models were developed for a cross section of the aquifer through the detention basin, oriented along the direction of ground-water flow. The first of the two cross-sectional models was designed to test various aquifer properties and to determine the conditions that would most greatly affect the success of the proposed detention-basin strategy. The second cross-sectional model was designed to closely approximate the dynamic nature of the study area by incorporating seasonal fluctuations in evapotranspiration as well as surface-water inflow, outflow to Las Vegas Wash, and field-estimated aquifer properties.

Principal results of the simulations made using the three models are as follows:

1. Simulations using the planimetric model indicate that ground-water flow would not be diverted around a detention basin with an outlet structure unless the vertical hydraulic conductivity of the uppermost part of the alluvial deposits in the basin was 100 to 1,000 times lower than the horizontal hydraulic conductivity. These low values are considered unlikely owing to the physical nature of the study-area sediments.

2. Simulations using the first cross-sectional model, in which aquifer properties and ground-water inflow were allowed to vary, suggest that the detention basin may not function as hypothesized within a reasonable range of values for aquifer properties. The results also suggest that the slurry wall would not induce surface flow farther upgradient from the wall; instead, flow would be deflected upward immediately adjacent to the wall, carrying salts from the lower parts of the aquifer.

3. Simulations using the first cross-sectional model, in which only aquifer thickness was varied, indicate that the aquifer is too thin and the hydraulic gradient too flat for ground-water flow to be reduced near the base of the aquifer as a result of the slurry wall. For the slurry wall to function as hypothesized, the aquifer would have to be much thicker than it actually is (30 ft) to reduce ground-water flow near the base of the aquifer.

4. Simulations using the first cross-sectional model, in which the distribution of hydraulic conductivity was based on values calculated from slug tests at piezometers along the cross section, indicate that most of the ground-water flow takes place between depths of 10 and 25 ft. Only a fraction of the simulated ground-water flow was below 25 ft.

5. Simulations using the second cross-sectional model, involving evapotranspiration, outflow to Las Vegas Wash, surface-water inflow, and the distribution of hydraulic conductivity determined from slug tests, indicate that the distribution of dissolved solids in areas dominated by phreatophytes is primarily a function of evapotranspiration. In contrast, the distribution of dissolved solids in areas of hydrophyte vegetation is affected by the addition of surface water (primarily treated effluent), which mixes with and dilutes the ground water. Simulations incorporating the proposed slurry wall do not greatly alter the present distribution of dissolved solids in the ground water.

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APPENDIX A

For the planimetric model used herein, several changes have been made to the computer program of Voss (1984), version V12842D, to simulate the effect of the dike on ground-water flow, assuming that the proposed slurry wall has been installed. In the subroutine BCTIME, the parameter GNUN was added to represent the conductance of the dike. The conductance used herein refers to the relative ease with which water can flow. A value of 1.0 refers to no resistance to flow, whereas a value 0.001 refers to greatly restricted flow. GNUN was specified to be zero around the perimeter of the dike (that is, no flow allowed to cross the dike) until the head value in the adjacent upgradient cell reached the height of the dike, which in turn was specified by the PBC(node) as listed in the added code below. The term PBC(node) refers to the pressure-boundary condition at the node specified. In this example, it represents the hydraulic head at the top of the dike. When the calculated head value at a node representing the dike became greater than the specified PBC value at that node, GNUN was changed to 1.0 and water was allowed to spill over the dike with no resistance. These changes allowed for water to be dammed behind the dike to a level equal to the height of the dike, and then to flow over the dike at levels above that height.

SUBROUTINE BCTIME

Add the following to line N130 after IQSOUT:

```
,GNUN,PM1
```

Add the following lines after line N180:

```
COMMON/CONTRL/ GNU,UP,DTMULT,DTMAX,ME,ISSFLO,ISSTRA,TTCYC     N181
NPCYC,NUCYC,NPRINT,IREAD,ISTORE,NOUMAT,IUNSAT                 N182
```

Add the following to line N190 after IQSOU(NSOU):

```
,GNUN(NBCN),PM1(NN)
```

Add the following lines after line N920:

```
PBC(42)=64.0
PBC(43)=67.0
PBC(44)=70.0
PBC(45)=73.0
PBC(46)=76.0
PBC(47)=79.0
PBC(48)=82.0
PBC(49)=85.0
PBC(50)=88.0
PBC(51)=64.0
PBC(52)=67.0
PBC(53)=70.0
PBC(54)=73.0
PBC(55)=76.0
PBC(56)=79.0
PBC(57)=82.0
PBC(58)=85.0
PBC(59)=88.0
PBC(60)=64.0
PBC(61)=64.0
PBC(62)=64.0
PBC(63)=64.0
PBC(64)=64.0
PBC(65)=64.0
PBC(66)=64.0
PBC(67)=64.0
PBC(68)=64.0
```

Add the following lines after N940:

```
   IF((GNUN(IP).LT.GNU).AND.(PM1(-I).LE.PBC(IP))) THEN                N941
      GNUN(IP)=0.0001
   ELSE
      GNUN(IP)=1.00
   ENDIF
```

SUBROUTINE BCB

Add the following to line U80 after QPLITR:

```
   ,GNUN                                                               U80
```

Add the following to line U190 after QPLITR(NBCN):

```
   ,GNUN(NBCN)
```

Modify lines U300 and U310 as shown:

```
   100 GINL = -GNUN(IP)                                                U300
   GINR = GNUN(IP) * PBC(IP)                                           U310
```

SUBROUTINE SUTRA

Modify line B150 as shown:

```
   6      PBC, GNUN, UBC,QPLITR,POBS,UOBS,OBSTIM,GXSI,GETA             B150
```

Add the following to line B330 after IPINCH(NPINCH,3):

```
   ,GNUN(NBCN)
```

Add the following to line B650 after IUBCT:

```
   ,GNUN
```

Modify line B2470 as shown:

```
   2   IPBCT, IUBCT, IQSOPT,IQSOUT,GNUN,PM1)                           B2470
```

Add to line B2640 after QPLITR:

```
   ,GNUN
```

SUBROUTINE BOUND

Add to line F70 after IUBCT:

```
   ,GNUN
```

Add to line F130 after UBC(NBCN):

```
   ,GNUN(NBCN)
```

Add to line F420 after UBC(IPU):

```
   ,GNUN(IPU)
```

Change the following in line F430:

```
   2G20.0 to 3G20.0
```

Add to line F470 after PBC(IPU):

```
   ,GNUN(IPU)
```

Add to line F480:

```
   , 6X, 1PD20.13
```

APPENDIX B

For the cross-sectional model used herein, several changes have been made to the computer program of Voss (1984), version V12842D, to simulate evapotranspiration at the top of the water table. Subroutine BCTIME allows for user-defined, time-dependent boundary conditions to be implemented into the code. A sine-wave function was used to simulate the seasonal fluctuation of evapotranspiration in the study area. The location of the node, size of the adjacent cell, and time of the year influence the amount of outflow from the model area as evapotranspiration or inflow to the model area from surface-water sources. Recharge into the model area was assumed to have a dissolved-solids concentration of 3,000 mg/L. The final program change was in subroutine NODALB, where the nodes representing the top of the water table were modified to allow only fluid flux to be discharged; the solute flux remained in the model and was not allowed to discharge during evapotranspiration.

Because much of the code between lines N1370 and N1480 has been modified, the entire part of code between these lines is included here.

SUBROUTINE BCTIME

```
C     INCORPORATE A SINE FUNCTION FOR ET WHERE THE ABSISSA
          REPRESENTS                                            N1371
C     THE AVERAGE ET RATE FOR THE YEAR.  TIME STEPS MUST
          BE IN ONE                                             N1372
C     MONTH INCREMENTS SO THAT ONE SINE WAVE REPRESENTS
          ONE YEAR.                                             N1373
C                                                               N1374
C                                                               N1375
        ARATE=1.8000E-05/2.                                     N1376
        JJ=IT*30                                                N1377
        THETA=JJ*(3.141592654/180.)                             N1378
        DO 600 IQP=1,NSOPI                                      N1380
        RATE=ARATE*SIN(THETA)+ARATE                             N1381
        I=IQSOP(IQP)                                            N1390
        IF(I.LE.-220) RATE=ARATE*SIN(THETA)                     N1391
        IF(I) 500,600,600                                       N1400
  500 CONTINUE                                                  N1410
C       NOTE :    A FLOW AND TRANSPORT SOLUTION MUST OCCUR
                    FOR ANY                                     N1420
C                 TIME STEP IN WHICH QIN( ) CHANGES.            N1430
        IF((I.EQ.-275).OR.(I.EQ.-308)) RATE=RATE/2.             N1431
        IF((I.EQ.-286).OR.(I.EQ.-297)) RATE=RATE/50.            N1432
        IF(I.EQ.-418) RATE=RATE/1.43                            N1433
        IF(I.EQ.-429) RATE=RATE/3.333                           N1434
        IF(I.EQ.-440) RATE=RATE/6.667                           N1435
        IF(I.EQ.-451) RATE=RATE/12.50                           N1436
        QIN(-I) = -1.0*RATE                                     N1437
C                                                               N1440
C       NOTE :    A TRANSPORT SOLUTION MUST OCCUR FOR ANY       N1450
C                 TIME STEP IN WHICH UIN( ) CHANGES.            N1460
        IF(QIN(-I).GT.0.) THEN                                  N1461
        UIN(-I) =3000.                                          N1470
        ENDIF                                                   N1471
  600 CONTINUE                                                  N1480
```

SUBROUTINE NODALB

Add the following lines immediately after line T790:

```
C       ALLOW ONLY FLUID MASS OUTFLOW AT WATER TABLE (ET)
```

```
360 IF(I.EQ.22) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
      ENDIF
IF(I.EQ.33) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.44) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.55) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.66) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.77) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.88) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.99) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.110) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.121) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.132) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.143) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.154) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
ENDIF
IF(I.EQ.165) THEN
     QUR=0.0D0
     QUL=-CW*QIN(I)
```

```
        ENDIF
    IF(I.EQ.176) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.187) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.198) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.209) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.220) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.231) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.242) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.253) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.264) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.275) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.286) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.297) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.308) THEN
        QUR=0.0D0
        QUL=-CW*QIN(I)
    ENDIF
    IF(I.EQ.319) THEN
        QUR=0.0D0
```

```
      QUL=-CW*QIN(I)
  ENDIF
IF(I.EQ.330) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.341) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.352) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.363) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.374) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.385) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.396) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF

  IF(I.EQ.407) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.418) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.429) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.440) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
  IF(I.EQ.451) THEN
      QUR=0.0D0
      QUL=-CW*QIN(I)
  ENDIF
```