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GROUND WATER RECHARGE

PHOENIX URBAN STUDY

FEBRUARY 1977

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FLOOD CONTROL DISTRICT of Maricopa County

3335 West Durango Street • Phoenix, Arizona 85009 • Telephone (602) 262-1501

COMMENTS ON "GROUND WATER RECHARGE PHOENIX URBAN STUDY
February 1977"

Paragraph 1.05.c recommends studies to determine the effect of recharge on subsidence. I think this recommendation should be supported, both for reasons of possible impact on flood control structures and because of the need for water conservation. Linsley, Kohler and Paulhus in Hydrology for Engineers (page 145) state that "... there is no evidence that the ground levels in regions of pronounced subsidence will be recovered if the aquifers are repressurized." Although this statement is made in a discussion of artesian aquifers, it is likely that subsidence is accompanied by a permanent loss in aquifer capacity.

The discussion in Paragraphs 2.07a and 3.05a leads to a preliminary determination that recharge basins are most suitable for the Salt River and New River and Phoenix City Streams projects. I think that the gravel pits which have thus far been considered a liability from the standpoint of aesthetics (Rio Salado) and erosion (bridges) might become a windfall from the standpoint of recharge. In the Handbook of Applied Hydrology (page 13-45), Chow says "Abandoned gravel pits ... are useful situations. ... Sides of pits should be steep enough so that silt settles to the bottom, leaving the sides relatively free for infiltration." Consideration might be given to acquisition of river bottom land by the appropriate agency. This land could be leased for the mining operation with the understanding that it would be used for recharge as needed, or it could be traded for land which has already been mined.

A cursory examination of aerial photographs taken in 1976 and 1977 indicates that there are about 1,000 acres of excavated areas in the Salt River between the Baseline Highway and 35th Avenue. This compares favorably with the Corps estimated area requirement for recharge of 2,000 cfs or 850,000 acre feet/year to allow joint use storage in the Orme Dam flood control pool (paragraph 3.04). That is, it compares with the abilities of the systems shown in Plates 3, 4 and 5.

Examination of aerial photography from 1976 shows about 50 acres of excavations in the Agua Fria River below the New River confluence and another 125 or so just above Glendale Avenue. Considering the recent growth in the areas west and northwest of Phoenix, it is certainly reasonable to expect these excavations to increase in the immediate future.

Use of abandoned gravel pits for recharge is probably more dependent upon flood control measures than use of basins. If uncontrolled flood water entered the pits with a high sediment load, they would probably fill with sediment and lose all effectiveness for recharge after such a flood. However, with the proper management of flood waters, I think they should be considered for recharge.

It should be noted that part of the recharge areas shown on Plates 3, 4 and 5 are located on the Salt River Indian Reservation. While realizing that this is a technical report, I think that consideration should be given to consultation with the Tribal Council, along with other legal considerations discussed in Paragraph 1.04c.

Leslie A. Bond

Leslie A. Bond

BOND/ly

HPD
JAL
BJ

Recharge ACDC waters along Skunk creek + New River?

DEPARTMENT OF THE ARMY
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SPLED-WU

17 March 1977

Mr. Jack Leavitt
Flood Control District of Maricopa County
3335 West Durango
Phoenix, Arizona 85009

Jack. Pls. fill me in.

Dear Jack:

As we discussed in our phone conversation yesterday, I am enclosing the recent report entitled "Ground Water Recharge Phoenix Urban Study, February 1977". The report is preliminary and there is still quite a bit of work that needs to be done. Unfortunately, our funding for this fiscal year does not allow us to continue work on this, however, we are hopeful that we will be able to resume this portion of our study in fiscal year 78.

If I can provide additional information please advise.

Sincerely,

H. W. WORTHINGTON
Study Manager

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GROUND WATER RECHARGE

PHOENIX URBAN STUDY

FEBRUARY 1977

U.S. ARMY ENGINEER DISTRICT, LOS ANGELES

CORPS OF ENGINEERS

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GROUND WATER RECHARGE

PHOENIX URBAN STUDY

I - GENERAL INFORMATION

1.01 PURPOSE. The purpose of this study is to develop a conceptual plan to conserve floodwaters for beneficial use in the Phoenix area. This study is in support of the Phoenix Urban Study overall water management plan. The general intent is to plan for conservation of excess floodwater by recharging the ground water basin.

1.02 SCOPE. This study concentrates on conservation of runoff from two watersheds: the Salt River watershed (see section III and pl. 1) and that collective group of watersheds analyzed under the name of "New River and Phoenix City Streams" (see section IV and pl. 2). Pertinent technical concepts fundamental to ground water recharge are discussed in chapter 2. This report is directed at providing conceptual ideas from which preliminary plan formulation decisions can be made. More detailed studies are needed to make final decisions concerning ground water recharge.

1.03 ASSUMPTIONS. This study addresses future conditions in the Phoenix area. Various water resource projects now in the planning and design stages are assumed in place for this study. Structures assumed to be in place include Orme Dam, New River Dam, Cave Buttes Dam, and Adobe Dam. Descriptions and locations of these structures are included in following paragraphs.

Sufficient data was not available, therefore, estimates were made (see section II) on the infiltration and percolation characteristics of soils in the study area and on aquifer storage capacity.

1.04 CONCLUSIONS.

a. Salt River Basin. Significant quantities of natural surface waters are available for ground water recharge from the Salt River basin. Ground water reserves in the reach from Granite Reef Dam to Tempe are severely depleted. Fortunately, soil conditions in this reach are conducive to recharge and sufficient riverbed land exists, upon which infiltration basins may be located. Dependent upon water rights and existing recharge from the Salt River Project's "additional water", anywhere from an average of 68,000 to 260,000 acre-feet of water could be conserved annually by a ground water recharge system. Potential also exists to store Central Arizona Project water and Salt River Project water underground in this area. Downstream of Tempe, the ground water table gradually rises until it becomes a problem at Buckeye. Intensive recharge is undesirable downstream of the Salt and Gila Rivers confluence. Currently, pumping is taking place near Buckeye to keep the ground water level below the root zone of crops.

b. New River and Phoenix City Streams. Runoff on Cave Creek below the proposed Cave Buttes Dam will be intercepted at the Arizona Canal and put to beneficial use. Therefore, supplemental recharge facilities appear unwarranted. Relatively small and sporadic quantities of runoff are expected on New River and Skunk Creek. Infiltration characteristics near these streams are generally poor. Infiltration basins along these streams would have to be located on the overbanks because the channels are too narrow. Infiltration basins would probably recharge less than 1,000 acre-feet per year of natural runoff into the ground water basin. Although infiltration rates are low, the storage available in the ground water basin could accommodate significant quantities of Central Arizona Project water and Salt River Project water.

c. Arizona Water Law. Current water law in Arizona is not attractive for large scale ground-water recharge. Primarily based on the common law rule, each land owner also owns the water percolating through the underlying soils (reference 19). However, ground water does not legally include water flowing in underground streams with defined beds and banks which are subject to the prior appropriation doctrine of surface waters. Under current laws, surface waters recharged into the ground would become the property of the owner of the overlying land and could not be controlled by the agency responsible for the recharge operation. Until new legislation is inacted the use of aquifers as water storage reservoirs may not be a practical means of water conservation.

1.05 RECOMMENDATIONS.

a. Salt River Basin. Based on this study, infiltration basins in conjunction with upstream storage are recommended for the Salt River. Ground water recharge in the reach between Granite Reef Dam and Tempe is viable because this reach has good infiltration characteristics, adequate surface water availability, and sufficient riverbed area to locate facilities. The potential for silt control in the Granite Reef Dam pool plus the potential storage space in the depleted ground water basin makes this reach a desirable location for ground water recharge. Several conceptual recharge plans are shown on plates 3 through 5. More intense studies, however, are needed to make final decisions on numerous technical factors discussed in this report. Test-infiltration basins are recommended to better define infiltration characteristics and ground water flow patterns. Operation of the proposed infiltration basins by a local agency is recommended; possibly by the agency that will operate Orme Dam.

b. New River and Phoenix City Streams. Ground water recharge facilities on New River and Skunk Creek, solely for natural runoff, are not recommended because the quantities of runoff available for recharge are small and sporadic in occurrence. If imported waters from the Granite Reef Aqueduct, the Arizona Canal and the Arizona Canal Diversion Channel supplemented by natural runoff, were used for recharge, facilities may be feasible. Plate 6 shows a potential location for three alternative infiltration basin plans.

c. Although not further addressed in this report, a ground-water recharge operation may mitigate subsidence problems in the Phoenix area. Studies pursuing the impact of ground water recharge on subsidence are recommended.

II - TECHNICAL ASPECTS OF GROUND WATER RECHARGE

2.01 INFILTRATION. Linsley, Kohler, and Paulhus in reference 1 define infiltration as "the movement of water through the soil surface into the soil." Infiltration is "distinguished from percolation which is the movement of water through the soil." While studying artificial recharge, Tyley in reference 2 has further stated:

> "The major problem associated with evaluation of a recharge site or a method of recharge is the determination of probable long-term infiltration rates. Such rates are critical for determining the method of recharge, the size of the recharge site, and the techniques of operation and maintenance.
>
> Many factors affect infiltration rates, but most are difficult to analyze separately. The composition of surface soils and the geohydrologic conditions discussed above are important factors affecting infiltration rates. The quality of the recharge water and the procedures used in the construction, operation, and maintenance of a recharge project can also affect the long-term infiltration rates. These latter factors can generally be controlled to maintain favorable rates."

This section of the report discusses the factors affecting ground-water recharge in general, some of the available information on recharge in the study area, and the basis for technical assumptions and estimates used in this study.

2.02 INFILTRATION RATE VERSUS TIME PATTERNS.

a. Infiltration rates are known to vary significantly with time. Several infiltration rate-time patterns, defined by researchers, are discussed in the following paragraphs. Muckel in reference 3, presented the typical infiltration functions shown on plate 7. This S-shaped infiltration rate curve for undisturbed soils is indeed well recognized and is presented in hydrology books written by Butler (ref. 4), Todd (ref. 5), and Chow (ref. 6). Actually, infiltration patterns are highly sensitive to local conditions and may not follow Muckel's typical curve at all. However, in the absence of data for a particular site, it appears best to assume Muckel's curve is representative. Muckel quoted Christiansen who described the phenomenon controlling the S-shaped infiltration rate curve for undisturbed soils as follows:

> "a. The initial decrease in permeability or infiltration rate is believed to be caused by dispersion and swelling of the soil particles. This is much more pronounced in some soils than others.
>
> b. The increase in permeability following the initial decrease accompanies the elimination of entrapped air from the soil. This air is slowly dissolved in the water passing through the soil.
>
> c. The gradual decrease in permeability that follows is due primarily to biological activity in the soil."

b. Muckel's infiltration rate curve for disturbed soils is also shown on plate 7. This curve exhibits a continuous decrease in infiltration rates until a lower limit is reached. The infiltration rate for disturbed soil (such as plowed farmland) is primarily controlled by surface conditions, where a sealing layer of fine particles from the disturbed soil forms. If the source of infiltrated water is heavily laden with suspended solids, an infiltration pattern similar to that for disturbed soils will result for any soil. In practice, the effects of the suspended solids is probably the most important factor controlling management of infiltration basins.

c. Tyley (ref. 2) conducted artificial recharge studies in the Coachella Valley of southern California. Infiltration tests conducted by Tyley showed infiltration rates to immediately increase with time to a peak and then drop off (see pl. 8). Tyley essentially explained the difference between his observations and Muckel's typical S-shaped curve as follows: "In many soils, the infiltration rate initially decreases due to dispersion and swelling of soil particles. Figure 8 (pl. 8) shows that this decrease in infiltration rate did not occur in the first test. This was probably due to a very low content of silt- and clay-size particles." The predominant soils were coarse sand and fine gravel.

d. Bouwer (refs. 7, 8, and 9) has conducted extensive studies on ground water recharge for the Flushing Meadows project in Phoenix. This project is located on the north bank of the Salt River near its confluence with the Gila River. Bouwer's work has yielded the infiltration rate curve shown on plate 9. This curve follows the pattern of Muckel's curve for disturbed soils. Bouwer has indicated that "The decrease in infiltration rate during inundation was essentially linear with time and caused by soil clogging. This clogging occurred mainly at the surface . . . Rice has shown that the surface clogging is principally a physical process, caused by the accumulation of suspended solids forming a thin layer with high hydraulic impedance."

2.03 PERCOLATION TO GROUND WATER.

a. As previously mentioned, Linsley, Kohler, and Paulhus (ref. 1) have defined percolation as the "movement of water through the soil." Because this study is concerned with ground water recharge, the percolation of surface waters to the ground water table rather than just surface infiltration is significant. The Arizona Water Commission (ref. 10) defines ground water as "that water occupying all the voids within a volume of rock. The word rock is used by hydrologists to mean both hard, consolidated formations such as limestone, sandstone, or granite; and loose, unconsolidated sediments such as sand and gravel." They also defined aquifers as "layers of rocks which contain ground water and allow its movement in appreciable quantities."

b. Before effective percolation can be achieved, the hydroscopic water requirements for the soils near the surface must be satisfied. The drier soils near the surface are the primary barrier to reaching a deep ground water table in a homogeneous soil. Evapotranspiration losses commonly dry the soils near the surface (in the zone of aeration). When small quantities of water are intermittently infiltrated, they are normally adsorbed in the zone of aeration. Adsorbed water, also known as hydroscopic water, is bonded to the soil by strong surface tension forces and is, therefore, not subject to gravity flow. Once the surface soils (in the zone of aeration) are saturated, water will percolate more freely into less restrictive deeper soils, that are commonly wetter (in the capillary zone). Linsley, Kohler, and Paulhus (ref. 1) have stated: "At the termination of rain, gravity water remaining in the soil continues to move downward and, at the same time, is taken up in capillary pore spaces. Usually the infiltrated water is distributed within the upper few feet of soil with little or no contribution to ground water unless the soil is highly permeable or the zone of aeration very thin."

c. As water percolates freely to the ground water, a bulge or mound in the water table will form below the location of surface infiltration. In reference 11, Bouwer made the following comments concerning stable-mound conditions.

> "The stable-mound condition can develop if there is some form of 'escape' for the water below the water table. This can be in the form of percolation through semi-permeable layers on which ground-water mounds are established, or in the form of a 'control' level of the water table above which the water level cannot rise. Such a control level may be the result of pumped wells in the vicinity of the recharge area, drainage into other basins, 'spilling' of ground water over the edge of discontinuous impermeable layers or lenses that support perched mounds, or, if the water table

comes close enough to the field surface, evapotranspiration or direct outflow. For a constant recharge rate, the water table under those conditions will rise until the total rate of drainage or escape below the water table equals the total recharge rate.

The information that may be desired regarding equilibrium mound positions can be reduced to one or both of the following questions:

1. What is the equilibrium recharge rate at a given maximum height of the center of the mound?

2. What is the equilibrium position of the mound for a certain recharge rate?"

Several years later (ref. 7), Bouwer commented: "The design of a system of recharge area or other collection facilities should be based on avoidance of high water-table mounds beneath the recharge area so that the water does not 'backup' to the soil surface with resulting reduction in infiltration rates."

2.04 EFFECT OF SURFACE WATER DEPTH AND DISTANCE TO WATER TABLE ON PERCOLATION.

a. Percolation in a homogeneous soil layer is generally defined by Darcy's Law:

$$Q = K A \frac{H}{L}$$

WHERE

Q = steady state discharge

K = coefficient of permeability in the direction of flow, at a given temperature

A = is the area perpendicular to the direction of flow

H = hydraulic energy head

L = length of the flow path

b. Darcy's equation may be reviewed to make general conclusions concerning the effect of depth of surface water on percolation during steady state conditions. Discharge varies directly with the energy head "H." Butler, in reference 4, has indicated that in a fairly homogenous soil where the ground water is deep, large increases in the surface depth will result in small increases in percolation. For a simplified example with vertical percolation only; if the ground water table is 300 feet deep and the surface water depth is increased from 1 foot to 10 feet (a 1,000 percent increase), "H" changes from 301 feet to 310 feet (a 3 percent increase). Discharge would similarly increase only 3 percent. If the least permeable soil layer is near the surface, as in a silted recharge basin or if the ground water mound rises close to the surface, "H" may be significantly larger than "L." Under these conditions, increases in surface depth will be followed by significant increases in percolation. Actually for stratified soils (as exist in the study area), the percolation process is considerably more complex.

c. Muckel (ref. 12) has found shallow depths of .2 to .3 foot to infiltrate more water than 1 to 2 feet depths can. "This may have been caused by differences in sunlight reaching the soil, temperature, or other unmeasured factors. These tests showed that certain plants will thrive with the shallow depths of water, whereas they will not survive under greater depths." In practice, it is doubtful that a large series of basins could economically be constructed to maintain .2 to .3 foot depths.

2.05 DRYING RECHARGE BASINS.

a. Infiltration rates versus time curves presented by Muckel, Tyley, and Bouwer (pls. 5, 6, and 7) all indicated that given enough time (at most 2 months), infiltration rates decrease to very low values. Higher rates are almost always restored by simply allowing the recharge basins to dry. Muckel (ref. 12) observed that 1 month of drying following 2 months of infiltration produced good results for his tests. Bouwer (ref. 8) found: "that maximum long-term infiltration or hydraulic loading is obtained with flooding periods in the range of 20 to 30 days and drying periods of about 10 days in the summer and 20 days in the winter" (see pl. 7). The Los Angeles County Flood Control District operates an extensive system of recharge basins. In reference 13, they have stated: "At our spreading grounds with shallow basins where we can alternate wetting and drying the basins, we are able to maintain our infiltration capacities. However, after prolonged continuous wetting at these spreading grounds, the rates gradually deteriorate." They usually keep their basins wet for 15 to 25 days with drying periods of 10 to 20 days.

b. On the extreme, the Orange County Water District of Southern California will keep their basins wet for up to 7 or 8 months between April and November and for about 1 month during the rest of the year. Sedimentation controls their durations of infiltration. During the warmer, drier months, they infiltrate relatively sediment-free imported water, but during the winter months, they frequently incounter sediment-laden storm runoff. It appears to be universal that sedimentation eventually reduces infiltration to the extent that drying has little effect and that only mechanical removal of the sediment layer will restore initial infiltration rates.

c. The ratio of wet time to dry time is pertinent in determining the acreage of recharge basins needed to infiltrate a constant flow. Based on the cases cited above, recharge basins for this study were assumed to require drying for one-third of their operational time.

2.06 EFFECT OF WATER QUALITY ON INFILTRATION.

a. In reference 3, Muckel states that it is common knowledge that hard water is more conducive to rapid infiltration than soft. Ordinarily, water analyzing below 30 ppm of calcium and magnesium is considered soft, from 30 to 60 ppm fairly hard, and above 60 ppm hard. He further stated:

"Sodium is another element known to affect the movement of water into or through a soil. Its importance lies not so much in the quantity present, but in its relation to the elements calcium and magnesium. The sodium percentage, calculated by dividing the quantity of sodium by the sum of the quantities of calcium, magnesium, sodium, and potassium (all in equivalents per million), is the usual way to classify the quality of water for irrigation with respect to the sodium effect on the soil. A water of high sodium percentage tends to deflocculate the colloidal soil particles and, consequently, hinders the movement of water. As is the case with hardness, no inflexible distinction can be drawn between a good and a poor quality of water. Generally, water in which the sodium percentage is above 65 percent is considered to be of poor quality, between 50 percent and 65 percent the quality is questionable, and below 50 percent it is satisfactory both for irrigation and for spreading."

b. The US Geological Survey (USGS) operates the water quality monitoring stations, "Salt River below Stewart Mountain Dam" and "Verde River below Bartlett Dam." Using 11 years of record (1964 to 1974) from these stations, the average hardness and sodium percentage for their combined flow were computed. Hardness was 58 ppm and the sodium percentage was 45 percent. Based on Muckel's statements, the quality of water appears to be satisfactory for infiltration. For the same 11 years, a sodium percentage of 40 percent and a hardness of 119 ppm was computed from data for the USGS water quality station "Colorado River below Parker Dam." This station is a good indicator of the potential quality of water imported to Orme Dam by the CAP. According to Muckel's standards, the quality of the imported water is even superior to Salt River water for infiltration. Thus, mixing of CAP water with Salt River water at Orme would be beneficial for infiltration in terms of water quality. No water quality data is available for runoff from Skunk Creek or New River.

2.07 METHODS OF ARTIFICIAL RECHARGE.

a. Tyley, in reference 2, made a comparison of different methods of recharge. Tyley's comparison (slightly modified to meet conditions of this study) is presented below.

METHODS OF ARTIFICIAL RECHARGE

Method	Advantages	Disadvantages
Injection or Ranney-type collector wells	No evaporation; not susceptible to wind; floods will not harm installation; ease of monitoring.	Very high initial costs; possibility of high maintenance costs; recharge water-treatment costs very high.
Deep pits or shafts	High infiltration rates; minimal siltation and clogging; possibility of multipurpose installation; low relative evaporation.	Higher cost than ponds or basins; rehabilitation more difficult; susceptible to wind-induced wave erosion.
Contour furrows	Inexpensive; efficient use of available land; small power costs.	Continual operation and maintenance problems; relatively high evaporation; possibility of being destroyed by flooding.

Flooding of natural stream channels	Least expensive; efficient use of available land; small power costs.	Very difficult operation and maintenance; highest evaporation; possibility of being destroyed by flooding; difficult to monitor infiltration rates; low infiltration rates.
Ponds or basins	Relatively easy to construct; efficient use of available land; small power costs; easy to rehabilitate; high infiltration rates.	Susceptible to wind-induced wave erosion; diversion structure could be costly; somewhat unprotected from vandalism; floods would harm project in riverbed; vector control may dictate operation of system.

b. The predominant method of recharge in southern California is by the use of infiltration basins. This is primarily because of their superior efficiency in operation and maintenance costs. For this study, only infiltration basins were considered for conceptual schemes. Numerous other factors discussed in paragraph 3.05a would also need to be considered to select a final method of recharge.

2.08 ESTIMATED INFILTRATION RATES ON THE SALT RIVER.

a. By reading the previous paragraphs, one can conclude that infiltration rates are highly variable and depend on local conditions. Todd, in reference 5, lists "representative spreading basin recharge rates" varying from 0.1 to 9.6 feet per day for various locations in the United States. Until actual infiltration tests are made in the areas of interest, the best that can be done is to estimate a long-term rate. Various geologic conditions discussed in Appendix 2 of this report have led to the estimate of 5 feet per day for the Salt River bed between Tempe and Granite Reef Dam. Plate 1 in Appendix 2 shows that the depth to ground water is deep (over 300 feet) in this reach. Because of this great depth, it is less likely that a ground water mound will "backup" and impede surface infiltration unless perched water is experienced. Soil particles are coarse and, again, because of the great depth to ground water, it appears that depths of surface water will have little impact on infiltration rates. Facilities to minimize sediment deposition are considered necessary, although it is recognized that eventually troublesome sediment accumulations will occur and mechanical removal will be required. It seems best to assume that the infiltration rate versus time pattern will react similar to Muckel's typical S-shaped curve for undisturbed soils (pl. 7). After the initial decline there will be a temporary increase in infiltration rates and then the eventual decline as time continues.

b. In conjunction with the Phoenix Urban Studies the Hydrologic Engineering Center (HEC) of the Corps of Engineers has conducted studies to estimate the influence of infiltration on the progression of a flood hydrograph on the Salt River below Granite Reef Dam. Historic flood events were modeled using the St. Venant equations to describe unsteady flow conditions of flood hydrograph progression downstream. A decay function was used to describe infiltration. This function inherently produces an infiltration curve similar to that observed by Bouwer. The final report for this study is presented in Appendix 3. If Muckel's S-shaped curve is considered to describe long-term infiltration, the results of the HEC study might be considered to, at best, describe the first few days of the S-shaped curve. However, the subsequent increase in the S-shaped curve will probably predominate the average long-term infiltration rate in a well managed infiltration basin. The Hydrologic Engineering Center study yielded an average infiltration rate of 2.6 ft/day as a 4-day average. The geological review in Appendix 2 suggests higher long-term rates and so does Muckel's S-shaped curve. Therefore, the average rate of 5 feet per day was considered acceptable.

2.09 ESTIMATED INFILTRATION RATES ON NEW RIVER AND PHOENIX CITY STREAMS.

a. In comparison to the Salt River bed, little geologic and hydrologic information is available for the Skunk Creek-New River area. USGS streamflow records on the Agua Fria River and its tributaries provide insight on infiltration rates in the study area. Plate 10 identifies the location of stream gages used in this study. Significant historical flood volumes on New River, Skunk Creek, and Agua Fria are presented in table 1. In this table, inflow to the lower New River is represented by the sum of discharges recorded at USGS stream gages, No. 5138.35 and 5138.6. Outflow from the reach is represented by gage No. 5139.7. Gage No. 5139.1 indicates flow conditions in the middle of the reach. During some of the recorded floods, New River also received inflow from the Arizona Canal (middle of the study reach). The floods of 1967 and 1970 exhibited relatively large infiltration losses in the lower reaches of the New River (betwee Skunk Creek and the Agua

Fria River) and Agua Fria River (between the New River and Salt River). The flood records for 1970, 1971, and 1972 indicate that local inflow (the exact quantity unknown) occurs between the gages. Thus, total inflow to the reach is undefined for the recorded events. Because of inadequate definition of overflow areas and uncertainty in local inflow, computation of infiltration rates from this data is not possible.

b. A long-term infiltration rate of at least 1 foot per day is suggested in Appendix 2 for fine-grained river deposits, which are indicative of the Skunk Creek-New River area. Reference 14 presents an analysis used to estimate channel infiltration from the September 3-7, 1970 flood on Cave Creek. This analysis yielded an estimated average infiltration rate of 2.5 feet per day (1.25 cfs per wetted acre). Because no other information was available, this rate was used for the New River and Phoenix City Streams study area.

III - SALT RIVER

3.01 DESCRIPTION OF THE SALT RIVER WATERSHED. The Salt River provides a significant contribution to the water supply of the Phoenix area. While the river passes directly through Phoenix, most of the runoff is from higher elevations in the upper watershed, located to the east and northeast of Phoenix. The watershed is shown on plate 1. The major tributary to the Salt River is the Verde River. The drainage area is about 12,900 square miles at the Salt-Verde confluence (about 20 miles east of Phoenix), with roughly equal area from each river. The Salt River continues to the west of Phoenix to its mouth at the Gila River. Here, the drainage area increases another 800 square miles, however, annual runoff from this desert area is negligible. Through this desert area (Salt River Vally, which includes metropolitan Phoenix), the Salt River bed extends about 42 miles from the Verde River to the Gila River. The primary source of runoff is the watershed above the Salt-Verde confluence. Elevations range from 1,300 feet at the Salt-Verde confluence to over 12,000 feet in the San Francisco Peaks. Much of the basin is mountainous and above 6,000 feet. The character of the land changes from dry desert in the lower elevations to pine forests in the higher elevations. Winter snows cover much of the upper basin and result in a significant part of the annual flows on the Salt and Verde Rivers.

3.02 EXISTING WATER SUPPLY SYSTEMS.

a. About one-third of the Phoenix area water supply comes from surface waters of the Salt River. The predominant water supply system, conserving these waters for the Phoenix area, is the Salt River Project (SRP). The SRP's six dams total about 2 million acre-feet of conservation storage. Four of the dams are located on the Salt River and two are located on the Verde River (see pl. 11). Conservation releases from these dams flow down the natural riverbed to Granite Reef Diversion Dam, located about 3-1/2 miles downstream of the Salt-Verde confluence (see pl. 11). At Granite Reef Dam, conservation flows are diverted into the Arizona Canal on the north and the Southern Canal on the south. These two major canals feed the water distribution system for the Phoenix area. The combined capacity of these two canals is about 3,700 cfs. Salt River discharge exceeding this diversion capacity will flow over the ungated crest of Granite Reef Dam and on downstream.

b. About two-thirds of the Salt River Valley water supply is pumped from the ground water basin. Local municipalities, private individuals, water companies, and the SRP all pump water from the underlying aquifer. The SRP alone uses more than 200 wells to supply its canal system. According to the Arizona Water Commission (ref. 10), the Salt River Valley is an example of coordinated use of surface and ground water to maintain a relatively constant total supply. Despite the use of surface waters, pumpage has exceeded the safe yield and the aquifer is depleted more each year. Between 1964 and 1969, the ground water table has declined an average of 1.8 feet per year (ref. 10). Table 2 presents the annual volume of ground water pumpage in the Salt River Valley as estimated by the Arizona Water Commission (ref. 10). Plate 1 of Appendix 2 presents a 1972 ground water profile along the Salt River and exhibits depths exceeding 300 feet to the ground water table between Tempe and the Granite Reef Dam.

c. The City of Phoenix diverts water from the Verde River under agreement with SRP. This water is treated and piped to Phoenix for municipal and industrial distribution.

d. Operational criteria for the SRP are highly complex. Operational decisions are based on available storage, water demands, power availability and demands, ground water availability, and flood forecasting. Most runoff from the Salt River watershed is impounded by SRP reservoirs. Reservoir releases are diverted into the Arizona and Southern Canals at Granite Reef Dam. Therefore, the Salt River bed, below Granite Reef Dam, is usually dry.

e. Occasional large flows will exceed the impoundment capacity of the Salt River Project reservoirs. Whenever large spillway flows are anticipated, the SRP delivers "additional water" to its users in an attempt to draw the reservoi down and put as much water as possible into beneficial use. The SRP (reference 18) has stated, "We refer to excess water deliveries as 'additional water' when such excess water can be delivered to qualified users within the capacity of the canal system without discharging water to the Salt River channel." The amount of "additional water" delivered is limited by the combined capacity (3,700 cfs) for diversions into the Arizona and Southern Canals at Granite Reef Dam. Regular monthly demands (excluding "additional water") for diversions at Granite Reef Dam were estimated by the Corps of Engineers (COE) from USGS records for the years 1959 to 1973. Months with spillway flows at SRP reservoirs were deleted from the computations so that excess "additional water" would not be included. The intent was to isolate "additional water" as an independent value. The SRP (reference 18) also estimated normal delivery demands. SRP described their estimate as follows. "In order to determine how much of that water exceeded the normal delivery demands, the three years immediately preceding 1973 were selected as indicative of representative normal releases for comparison purposes." The estimates by the Corps of Engineers and the SRP are fairly comparable and both listed in table 3.

f. Examination of USGS flow records indicate that even during times of excessive floods, it is doubtful that the "additional water condition" would stimulate capacity use of the canals. For example, during the calendar year of 1973, heavy runoff filled the SRP reservoirs and resulted in the "additional water condition." During April 1973, flow above Granite Reef Dam averaged about 12,000 cfs and the SRP diverted about 2,600 cfs (reference 18), leaving an average unused canal capacity of about 1,100 cfs. During May 1973, flow above Granite Reef Dam averaged about 6,700 cfs and the SRP diverted about 3,200 cfs (reference 18), leaving an average unused canal capacity of about 500 cfs. These examples point out that even though sufficient water was available, capacity canal diversions (3,700 cfs) were not made. During April and May the SRP (reference 18) estimated the average "additional water" to be about 1,300 cfs and 1,800 cfs respectively.

g. "Additional water" is commonly used to leach salts from agricultural land. "Additional water" is usually available for only sporadic and short durations. Its effect on ground water recharge is uncertain. In accordance with the discussions in paragraphs 2.02 and 2.03, significant quantities of "additional water" may be adsorbed as hydroscopic water in the zone of aeration and not recharge the ground water basin. This is likely in areas where the depth to ground water is large. However on the contrary, because of intensive year-round irrigation, the soil's affinity for hydroscopic water may be satisfied resulting in considerable ground water recharge from the infiltrated "additional water." During long periods of excessive flow, such as 1973, some "additional water" would undoubtedly recharge the ground water basin. Considerable data collection is necessary to determine how much "additional water" actually recharges the ground water basin. The SRP (reference 18) has stated, "some water percolates beneath the root zone with every irrigation and maintains hydroscopic water levels within the soil profile. As a result of these areas leaching irrigations of excess waters are subject to percolation and ground water recharge."

h. When local floods in the Phoenix Valley are anticipated to fill the distribution canals, water at Granite Reef Dam or in the canals is diverted into the Salt River to prevent overtopping of these canals. These diversions usually infiltrate in the Salt River bed. However, because the infiltrated waters are relatively small in volume, they add little recharge to the ground water basin.

3.03 PROPOSED ORME DAM. The US Bureau of Reclamation (USBR) is currently developing the Central Arizona Project (CAP) - a major water conveyance system for delivering Arizona's share of Colorado River water to central and southern Arizona. Alternative damsites are being studied to locate a terminal storage facility for CAP water in the Phoenix area. The damsite under most serious consideration is for Orme Dam, just below the confluence of the Salt and Verde Rivers (see pl. 11). The Corps of Engineers has studied the flood control aspect of this proposed structure for the USBR. Storage allocations for Orme Dam are shown on plate 12. The conservation pool will include the first 410,000 acre-feet of the reservoir. The next 950,000 acre-feet will be allocated to the flood control.

3.04 JOINT USE STORAGE OF ORME DAM FLOOD CONTROL POOL.

a. As discussed in reference 15, the Corps of Engineers suggested a seasonal joint use plan for the first 850,000 acre-feet of the flood control pool. Between 1 May and 1 October, 850,000 acre-feet would be available for conservation and the remaining 100,000 acre-feet would be reserved for flood control. Between 1 December and 1 March, the full 950,000 acre-feet would be reserved for flood control. During the transition periods of 1 October to 1 December and 1 March to 1 May the storage allocations would vary linearly with time. This seasonal plan is described in plate 13.

b. Little runoff is expected during the summer months after 1 May. Therefore as design criterion, a maximum of 850,000 acre-feet of water must be evacuated from the joint use pool between 1 May and 1 December according to plan. This would require a steady release of about 2,000 cfs for the 7-month period. A infiltration basin system located below Orme Dam would consequently need a capacity of about 2,000 cfs to assure conservation of the 850,000 acre-feet of joint use storage. The water could be percolated into the ground water basin and extracted as needed.

c. In the future, accurate flood-forecasting systems may be developed that will permit additional conservation use of the flood control pool. Final design studies may also show that modifications to the structure, such as flash boards on the spillway, will permit additional conservation storage. A possibility also exists that another flood control or terminal storage plan may be developed that will eliminate Orme Dam and change the conservation concept presented in this report completely.

3.05 DESIGN OF CONCEPTUAL RECHARGE BASINS.

a. Several conceptual schemes of infiltration basin layouts were developed for this study. Final plans would require considerably more detailed studies. Considerations would have to be given to topography, the final flood control and conservation plan for the Salt River, localized infiltration rates, percolation rates, transmissibility rates, percolation paths, sediment transport, soils design of berms, design of hydraulic structures, water quality, vector control, economics, geologic studies, environmental studies, evaporation, land acquisitions, legal questions, institutional problems, etc. Implementation of a test basin program in the area of concern would greatly help to quantify many of the unknowns encountered in this study.

b. In Section II of this report, an average long-term infiltration rate of 5 feet per day and a drying time of one-third of the operating time were estimated as appropriate. Using these values and the maximum release of 2,000 cfs discussed in the preceding paragraphs, the area of infiltration basins needed was estimated to be 1,200 acres.

c. In the study area, the Salt River bed has a slope of about 1 foot in 610 feet. Basin length is controlled by the riverbed slope and maximum depth of ponding (see pls. 3, 4, and 5). All basins were designed with a minimum depth of 1/2 foot at the shallow end. Maximum depths at the deep end were limited to 5 feet, because greater depths would probably require much more substantial berms than could efficiently be constructed. Greater depths are also associated with longer basin lengths, which are undesirable. Infiltration basins with large fetch lengths may develop troublesome wind waves. Waves will threaten the stability of berms and may also transport unwanted sediments from the berms to the basin floors. Borrowing animals have proven to be a continuous threat to the structural integrity of berms at Los Angeles County Flood Control District infiltration basins making high berms undesirable.

d. Various schemes are presented on plates 3, 4, and 5. For these schemes, infiltration basins were located on the riverbed material, as defined on USGS 1:24,000 scale topographic maps. All of the schemes presented are well upstream of the Rio Salado Plan, which proposes a park-like development of the Salt River bed. Even if additional acreage becomes needed for infiltration, there would be no need for encroachment into the Rio Salado area. It is possible that esthetically pleasing infiltration basins could be designed as part of the Rio Salado Plan, especially if imported water was also used for recharge.

e. Consideration was given to provide a floodflow channel for each plan. The development of the hydraulic capacity of the floodflow channel will be dependent on final flood control design considerations and is currently unknown. Most likely the infiltration basins will be washed away by large floods and will have to be rebuilt. Such considerations lead to the concept that berms should be made as inexpensively as possible, probably of uncompacted riverbed material pushed up by heavy equipment.

f. Delivery of water to the infiltration basins can be achieved in several ways. Plates 3, 4, and 5 each present separate delivery systems. Transporting the least amount of silts possible into the infiltration basins is of prime interest.

g. In general, water released from Orme Dam will flow down the natural riverbed to the small pool behind Granite Reef Dam. At Granite Reef Dam the water will have the opportunity to clarify. From this point, three schemes have been presented on plates 3, 4, and 5.

h. The plate 3 plan calls for releasing water from Granite Reef Dam into the natural riverbed. Collection berms will be used to channel the flow into the infiltration basins. Flow will undoubtedly pick up silts in this plan. Silt-control basins will be required at the recharge facility. Chemical floculents may be introduced in the silt-control basins to precipitate suspended solids. The Granite Reef Aqueduct will siphon under the Salt River just downstream of the Granite Reef Dam. It is unlikely that prolonged low flows will influence the operation of this siphon.

i. The plate 4 plan calls for enlarging about 2 miles of the Southern Canal and constructing about 5 miles of a new canal. Both canals would be lined and convey relatively silt-free water from Granite Reef Dam to the infiltration basins.

j. The plate 5 plan calls for constructing 8 miles of unlined channel. The design would call for low velocities to restrict the transport of sediments. Sediment-control basins may or may not be required.

3.06 USE OF REGULATED FLOW IN THE NATURAL RIVERBED FOR INFILTRATION.

a. The natural riverbed from Granite Reef Dam to the Tempe Bridge may also be used for infiltration. With sustained low flow this normally dry riverbed would undoubtedly change. A confined and meandering low-flow channel would form. The size and shape of such a channel was estimated from a river cross section taken by the USGS at their "Salt River Below Stewart Mountain Dam" stream gage. Here, sustained flows below 2,000 cfs are common. Using their discharge rating curve, a discharge versus wetted area relationship was established and used in conjunction with the infiltration rates discussed in the following paragraph to estimate the river's infiltration capacity. Actually geologic conditions of the riverbed change as it approaches Tempe, and a low flow channel section will change accordingly (probably differently than estimated). However because no additional information is available the USGS-Salt River channel section was used as a best estimate to describe the reach to Tempe.

b. Flows in the natural riverbed below Granite Reef Dam will transport sediments downstream. Because of infiltration, controlled discharges will decrease eventually to zero at some downstream location. Thus, sediments eroded from the upper part of the reach will be deposited in the lower part of the reach causing progressively greater sealing of the riverbed. Infiltration rates may be expected to vary accordingly. Dividing the reach into quarters, the following infiltration rates were assumed: 5 ft/day, 4 ft/day, 3 ft/day, and 2 ft/day (from upstream to downstream). An infiltration capacity of about 400 cfs for the natural riverbed, from Granite Reef Dam to Tempe Bridge was computed. Assuming one-third of the time is needed for drying the bed, an average capacity of about 250 cfs exists. If the use of the natural riverbed is incorporated into any of the plans presented on plates 3, 4, and 5, a reduction of about 150 acres to the infiltration basins may be expected. Use of the natural riverbed only, would not provide sufficient infiltration to recharge the available surface waters.

3.07 WATER AVAILABLE FOR RECHARGE.

a. Historic monthly discharges were analyzed under future "project" conditions to estimate the average annual amount of water that could be conserved in infiltration basins. The computer program "HEC-5C, Simulation of Flood Control and Conservation Systems" was employed to model and future conservation system. USGS flow records were used to establish historic average monthly flows into the SRP reservoirs. Monthly demands for SRP water were estimated from average flows (by COE) in the Arizona and Southern Canals (see para. 3.07e and table 3).

b. Monthly flows from August 1888 to September 1974 were routed through the existing SRP reservoirs based on estimated monthly demands to establish outflows from the SRP system. The computed monthly outflows from the SRP were combined with estimated CAP deliveries (see table 3) to derive inflows to the proposed Orme Dam. CAP demands plus SRP demands (see table 3) were combined to estimate total conservation demands from Orme Dam ("additional water" was not included).

c. Using the 86 years of record combined with SRP and proposed CAP water budgets, HEC-5C was used to determine how much water would be available for recharge. The seasonal joint use storage plan for Orme Dam was used (see pl. 12). Based on the above conditions, two separate sets of storage routings were performed for Orme Dam - one where water for recharge (2,000 cfs maximum) was given priority and one where diversions up to the capacity of the Arizona and Southern Canals (maximum "additional water" plus normal demands - 3,700 cfs) was given priority. This dual analysis was performed because the role of "additional water" in establishing water rights is uncertain at this time. For both routings no water for recharge and no "additional water" were withdrawn from the conservation pool (below 410,000 acre-feet). The conservation pool was used solely to meet regular SRP and CAP demands. Water for recharge and "additional water" were released from Orme Dam only when the pool exceeded 410,000 acre-feet. For the analysis, giving recharge priority an average of 355 cfs or 260,000 acre-feet per year was available for infiltration and 79 cfs or 58,000 acre-feet per year was available as "additional water." Giving "additional water" priority, an average of 350 cfs or 256,000 acre-feet per year was available as "additional water" and 93 cfs or 68,000 acre-feet per year was available for infiltration. It is anticipated that amount of water available for recharge will be somewhere between (but currently undefined) the two values discussed above and will be based upon legal decisions.

d. The analysis revealed that neither the water for infiltration nor "additional water" can be relied on for regular use. Within the 86 years of record, a 16-year period occurred in which no water was available for either infiltration or as "additional water. Recharge of the ground water basin during high runoff years would help reduce the burden of such long droughts on the Phoenix community. A potential also exists for using the recharge basins to infiltrate regular CAP or SRP water thus reducing surface storage requirements at Orme Dam and upstream reservoirs.

3.08 EVAPORATION. If the infiltration basins were operated at full capacity (2,000 cfs all year), about 4,600 acre-feet per year of water will evaporate (based on the rate of 68.76 inches per year at the SRP reservoirs). Because the basins will operate at an average annual flow of about 355 cfs, the ratio of 355/2000, about one-fifth, was used to estimate the true evaporation to be about 900 acre-feet per year. Slightly more should be expected if the natural riverbed is used for infiltration. Actually, water in the infiltration basins is expected to evaporate at a higher rate than water in the SRP reservoirs, because the basins are shallow and will heat up considerably more. Regardless, evaporation off the infiltration basins would be slight compared to the amount of water infiltrated. Of more concern is the control of vegetation in the recharge area. Heavy growth near the infiltration basins could result in large evapotranspiration losses.

IV - NEW RIVER AND PHOENIX CITY STREAMS

4.01 PROPOSED STRUCTURES. Watershed boundaries for the New River and Phoenix City Streams are shown on plate 2. The Corps of Engineers has proposed a comprehensive flood control system for northern Phoenix. This system includes a series of dams and channels that are shown on plate 10 and are described in detail in reference 14. The proposed dams, New River Dam, Adobe Dam, and Cave Buttes Dam, are currently planned for flood control only and not for water conservation. This section presents pertinent runoff data and explores the possibility of installing infiltration basins and using the dams to conserve water. The channels located immediately below the dams will be left in their natural state. The maximum design outflow (for flood control) from Cave Buttes Dam will flow down the natural reach of Cave Creek to the location where flow can be diverted into either the Arizona Canal or the Arizona Canal Diversion Channel (see pl. 10). Because the SRP will divert most Cave Creek flow into the Arizona Canal for conservation use, little benefit would be derived from additional recharge facilities for this water.

4.02 EXISTING STRUCTURES.

a. The Paradise Valley detention basins are located in the drainage basin just east of Cave Creek (see pl. 10). These structures were designed by the US Bureau of Reclamation to retain probable maximum flood volumes and protect the proposed Granite Reef Aqueduct from flooding. Controlled releases of flood water from these basins will be used to supplement flow in the Granite Reef Aqueduct (Central Arizona Project).

b. Dreamy Draw Dam was constructed by the Crops of Engineers in 1973. This ungated structure controls runoff from a drainage area of 1.3 square miles. Runoff from this small desert basin would be negligible in terms of water conservation.

4.03 BASIN CHARACTERISTICS. The adjacent New River, Skunk Creek, and Cave Creek basins are located to the north of Phoenix and are shown on plate 2. All of these streams generallly flow southward from the rugged New River Mountains where snow falls almost every winter. However, most snowmelt infiltrates before reaching the proposed damsites. Elevations range from about 5,000 feet to about 1,400 feet, where the proposed dams are to be located. The lower elevations are fairly flat valley land with regular alluvial slopes and are generally desert in character. The drainage areas above the proposed dams are 164 square miles above New River Dam, 75.6 square miles above Adobe Dam (on Skunk Creek), and 195 square miles above Cave Buttes Dam (on Cave Creek). These basins are sparsely populated and future development is expected to be minimal, therefore, increased impervious cover with associated increases in runoff are not expected. Estimated annual runoff at each damsite is presented in table 4.

4.04 IMPACT OF PROPOSED DAMS ON INFILTRATION. Flood detention achieved by the proposed dams will reduce peak discharges and correspondingly increase the duration of flows on the downstream channels. The effect is to increase the quantity of water infiltrated in downstream channels. Whether the increase in infiltration will result in an increase in ground water recharge is uncertain.

The depth to ground water is about 300 feet (reference 10) in this region. Because streamflow is sporadic (based on USGS records), the underlying soil's affinity for hydroscopic water may not normally be satisfied (see paras. 2.02 and 2.03). The great depth to ground water makes the hydroscopic water requirement appear large. Therefore, most infiltrated water might be adsorbed near the surface rather than recharge the ground water basin. However, extensive agricultural irrigation may keep the underlying soils wetter and, therefore, satisfy this requirement. Additional information would be desirable to determine if increased infiltration will increase ground water recharge or be absored near the surface and enhance vegetation. The answer to this question will be useful in establishing whether concentrated infiltration in a basin or widespread infiltration in the natural channel reaches is desirable as an overall plan. Estimated infiltration capacities for bank-full flow in the downstream channel are given in table 5.

4.05 INFILTRATION BASINS.

a. If infiltration basins are to be located in the study area, the confluence of New River and Skunk Creek may be the most desirable location. Runoff from 315 square miles converges at this location under existing conditions. However, most of the runoffs occur during short times of flooding - a condition that makes use of runoff for recharge difficult. The proposed New River and Adobe Dams will "spread out" flood hydrographs while reducing peak flows, consequently making floodwater more prone to control and recharge. Excess waters in the Arizona Canal could also be used for recharge at this site.

When the "additional water" situation occurs on the Salt River, some of this water could be directed through the Arizona Canal to the recharge facilities. In a similar manner, if excess water is available in the Granite Reef Aqueduct, it could be released into New River and directed to the recharge basins. Such recharge of imported water for later use is, in fact, common place in southern California. Upon completion of the Arizona Canal diversion channel, additional runoff from about 290 square miles will be diverted to the site in interest. This additional area includes the proposed Cave Buttes Dam basin. As mentioned in paragraph 4.01, the SRP will have the option of diverting outflow from Cave Buttes Dam into the Arizona Canal or the Arizona Canal Diversion Channel. If this flow is not needed for immediate use, it could be directed to the recharge facilities for underground storage.

b. Because of the minimal amount of data available for this area it is very difficult to estimate the design capacity of recharge basins and quantity of water that could be conserved by such facilities. For this study, it is assumed that imported water will not be regularly available for recharge (although this may be the only strong reason to provide recharge facilities in the area). Review of USGS records indicates a recharge facility with a capacity of about 200 cfs would conserve most low flows. Even after construction of the proposed dams, the majority of runoff will be from floods with short durations and peak discharges greatly exceeding 200 cfs. Recharge of the higher flows would still not be possible. It appears safe to say that this recharge basin would conserve less than 1,000 acre-feet of water per year on the average (based on review of USGS flow records).

c. Using the infiltration rate of 2.5 feet per day (discussed in para. 2.09), an area of 160 acres would be needed to infiltrate 200 cfs. Drying time was not considered because of the typically short durations of runoff in the area. An additional 20 acres was allowed for silt control. The general slope of the area is 1 foot vertical in about 200 feet horizontal. Using a maximum depth of 5 feet and minimum of 1/2 foot for each basin, three conceptual plans for infiltration basins were developed and are shown on plate 6. Detailed planning and design studies would need to consider many of the items discussed in paragraph 3.05.

d. The overall effect of these infiltration basins will be to conserve relatively small quantities of water, less than 1,000 acre-feet annually, when compared to the communities average demand of over 2-1/2 million acre-feet annually. Furthermore, the effect of the proposed dams alone will be to increase infiltration. This in itself reduces the need for infiltration basins. Thus, infiltration basins, solely for natural runoff, are probably not warranted for this area. Such basins may be highly valuable to recharge imported water.

4.06 MODIFICATIONS TO THE PROPOSED RESERVOIR.

a. Modifications to the proposed reservoirs may permit temporary impoundment of small floods. The water could be released at low rates to the infiltration basins discussed in the previous paragraphs. Each of the proposed dams have part of their reservoir volume allocated to sediment storage. The intent is that at the end of a 100-year period, the volume of sediments accumulated in the reservoir will not restrict the dam's ability to control the design flood. With modifications to the proposed sturctures, part of the sediment-storage volume of the reservoir could be used to impound and conserve floodflows to be released later for ground water recharge. As sediment accumulates in the reservoir, the amount of storage available to impound conservation water will diminish. The modification would essentially consist of raising the ungated flood control outlet to a reservoir storage level that is less than the sediment storage allowance. A small gated conduit connected to a small outlet tower in the reservoir will then allow regulation of impounded waters. Plate 14 illustrates this plan.

b. This modification plan is probably infeasible because of high costs are presumed to be associated with it. In addition to a second outlet works, the embankment and foundation of the proposed dams may need significant structural alterations to accommodate longer impoundment times. Also, by raising the flood control outlet, the reservoirs trap efficiency may be significantly increased, resulting in a need for a larger design sediment allowance and a higher dam. Maintenance costs would also be expected to increase.

TABLE 1
(Runoff volumes in acre-feet)

USGS Stream Gage Number**	Watercourse	12/19-20/67	9/5-6/70	8/3/71	7/17/72	7/21-22/74	8/20/70
		Date of Runoff					
5138.35	New River	12,890	5,810	370	80	0	0
5138.6	Skunk Creek	1,493	2,548	282	207	45	138
Subtotal		14,383	8,358	652	287	45	138
5139.1††	New River	14,297	*8,904		No record		
5139.7	Agua Fria	12,969	7,833	*712	*375	0	0
Gain or Loss***		-1,414	-1,071†	+60	+88	-45	-138

* Includes unknown quantity of inflow from Arizona Canal to New River.
** See plate 10 for stream gage locations.
*** Reach from gage numbers 5138.35 and 5138.6 (subtotal) to 5139.7.
† Difference between gage Nos. 5139.1 and 5139.7.
†† Located in middle of study reach.
NOTE: Gain or loss indicates local inflow or infiltration, respectively.

TABLE 2

Estimated Annual Ground Water Pumpage
in the Salt River Valley

Year	Annual Pumpage (1,000 ac-ft)	Year	Annual Pumpage (1,000 ac-ft)
Prior to 1915	57	1946	1,360
		1947	1,406
		1948	1,670
1915	15	1949	1,644
1916	15	1950	1,852
1917	15		
1918	40	1951	1,910
1919	60	1952	2,020
1920	95	1953	2,300
		1954	2,300
1921	100	1955	2,240
1922	200		
1923	400	1956	2,300
1924	500	1957	2,300
1925	500	1958	2,300
		1959	2,206
1926	500	1960	2,005
1927	500		
1928	500	1961	2,178
1929	600	1962	1,976
1930	650	1963	2,134
		1964	1,972
1931	600	1965	1,500
1932	300		
1933	572	1966	1,350
1934	711	1967	1,763
1935	554	1968	1,264
		1969	1,600
1936	684	1970	1,700
1937	665		
1938	905	1971	1,800
1939	738	1972	1,800
1940	943	1973	1,291
		Total	68,272
1941	444		
1942	1,004		
1943	1,104		
1944	1,017		
1945	1,143		

NOTE: Data from Reference 10.

TABLE 3

Average Monthly Salt River Project and
Central Arizona Project Discharges

Month	Average Salt River Project Demand Estimated by COE* (cfs)	Average Salt River Project Demand Estimated by SRP** (cfs)	Average Central Arizona Project Demand*** (cfs)	Average Estimated Conservation Releases From Orme Dam† (cfs)	Maximum "Additional Water"†† (cfs)	Average Central Arizona Project Inflow to Orme Dam††† (cfs)
January	301	157	24	325	3,399	1,292
February	543	645	88	631	3,157	964
March	1,498	1,124	66	1,564	2,202	492
April	1,399	1,278	370	1,769	2,301	320
May	1,388	1,363	435	1,823	2,312	132
June	1,820	1,624	1,155	2,975	1,880	0
July	1,943	1,846	1,282	3,225	1,757	0
August	1,494	1,304	914	2,408	2,206	0
September	1,354	1,264	1,090	2,446	2,346	0
October	505	615	177	682	3,195	345
November	295	461	88	383	3,405	843
December	383	510	24	407	3,317	1,329

* Canal releases at Granite Reef Dam, based on USGS flow records, 1959 to 1973, excluding "free water."

** Canal releases at Granite Reef Dam, estimated as 3-year average, 1970 to 1972, by SRP (reference 18).

*** Releases from Orme Dam, based on reference 17.

† Sum of Average Salt River Project Demands and Average Central Arizona Project Demands.

†† Based on difference between Average Salt River Project Demands estimated by COE and combined capacity (3,700 cfs) of Arizona and Southern Canals.

††† Based on reference 17.

COE Corps of Engineers
SRP Salt River Project

TABLE 4

Estimated Average Annual Flow at Damsites
(Based on 1967-74 period)

Location	Drainage Area (sq mi)	Estimated Average Annual Flow (ac-ft)
New River Dam	164	4,200
Adobe Dam	90	1,600
Cave Buttes Dam	191	4,900

NOTE: Average annual flow for "Cave Creek at Phoenix" (at Peoria Ave.), D. A. = 252 sq. mi., for period 1958-74 is 2,300 acre-feet.

TABLE 5

Estimated Channel Infiltration Capacity

Reach	Infiltration Capacity* (ac-ft/day) Low-Flow Channel	Total-Natural Channel
New River from Skunk Creek confluence to damsite	260	420
Skunk Creek from New River confluence to damsite	170	440
New River from Agua Fria confluence to Skunk Creek confluence	200	400

* Based on bank-full flow.

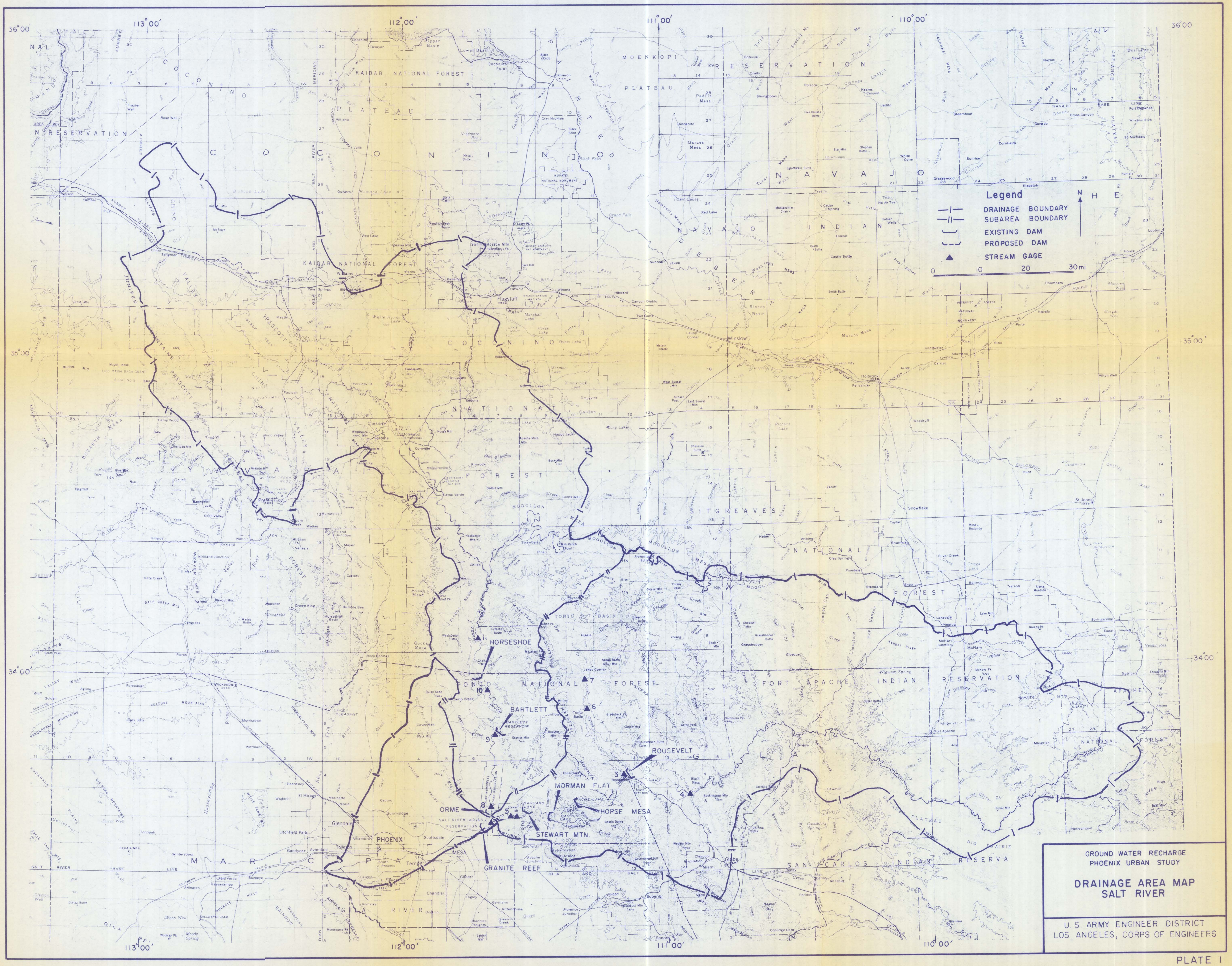

Legend
DRAINAGE BOUNDARY
SUBAREA BOUNDARY
EXISTING DAM
PROPOSED DAM
STREAM GAGE
0 10 20 30 mi
N
HORSESHOE
BARTLETT
ROOSEVELT
MORMAN FLAT
HORSE MESA
STEWART MTN
GRANITE REEF
ORME
PHOENIX
GROUND WATER RECHARGE
PHOENIX URBAN STUDY
DRAINAGE AREA MAP
SALT RIVER
U.S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE I

GROUND WATER RECHARGE
PHOENIX URBAN STUDY

DRAINAGE BOUNDARIES
NEW RIVER AND PHOENIX CITY STREAMS

U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 2


GRANITE REEF AQUEDUCT
SALT RIVER SIPHON
GRANITE
REEF DAM
ARIZONA CANAL
N
GROUP B
GROUP C
Collection Berm
Silt Control Basin
Collection Berm
Collection Berm
Silt Control Basin
Silt Control Basin
SALT RIVER
SOUTHERN CANAL
LEGEND:
BERMS
CANAL
87
GROUP A
158 Ac.
180 Ac.
105 Ac.
150 Ac.
100 Ac
65 Ac.
20 Ac.
27 A
30 Ac.
98 Ac.
136 Ac.
77 Ac.
57 Ac.
2135'
Mc DOWELL ROAD
T-2N
R-6E
R-5E
T-1N
TYPICAL INFILTRATION
BASIN SECTION
(NOT TO SCALE)
Downstream
Freeboard
2'
4'
1/2'
BERM
SLOPE =
1'
610'
2135'
TOTAL AREA of INFILTRATION = 1203 SURFACE ACRES
TOTAL BERM LENGTH =17.2 MILES
SALT RIVER
ARIZONA AVE.
87
NOTE: ONLY TWO GROUPS OF INFILTRATION
BASINS TO BE OPERATED SIMULTANEOUSLY
WHILE THIRD IS DRYING.
SCALE
1:24,000
1
1/2
0
1 MILE
GROUND WATER RECHARGE
PHOENIX URBAN STUDY
SALT RIVER INFILTRATION
BASIN—PLAN I
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 3

GRANITE REEF AQUEDUCT
SALT RIVER SIPHON
GRANITE REEF DAM
ARIZONA CANAL
N
GROUP A
GROUP B
GROUP C
SALT RIVER
LEGEND
BERMS
CANAL
SOUTHERN CANAL
PROPOSED DELIVERLY CANAL
10 Ac.
19 Ac.
19 Ac.
17 Ac.
14 Ac
17 Ac
24 Ac
33 Ac.
50 Ac.
68 Ac.
79 Ac.
91 Ac.
103 Ac.
105 Ac.
105 Ac.
102 Ac.
94 Ac.
83 Ac.
54 Ac.
26 Ac.
22 Ac.
28 Ac.
28 Ac.
87
McDOWELL ROAD
SALT RIVER
ARIZONA AVE.
87
T-2N R-6E
R-5E T-1N
TYPICAL INFILTRATION
BASIN SECTION
(NOT TO SCALE)
DOWNSTREAM
FREEBOARD 2'
BERM
2 1/2'
1/2'
SLOPE = 610' 1'
1220'
TOTAL AREA of INFILTRATION = 1211 SURFACE ACRES
TOTAL BERM LENGTH = 18 MILES
TOTAL LENGTH of PROPOSED DELIVERLY CANAL = 5.2 MILES
UPPER 2 MILES TO BE ENLARGED on SOUTHERN CANAL
NOTE: ONLY TWO GROUPS OF INFILTRATION BASINS TO BE
OPERATED SIMULTANEOUSLY WHILE THIRD IS DRYING.
SCALE
1:24,000
1 1/2 0 1 MILE
GROUND WATER RECHARGE
PHOENIX URBAN STUDY
SALT RIVER INFILTRATION
BASIN — PLAN 2
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:
PLATE 4

TOTAL AREA of INFILTRATION = 1216 SURFACE ACRES
TOTAL BERM LENGTH = 19.8 MILES
TOTAL LENGTH of PROPOSED DELIVERLY CANAL = 8.0 MILES

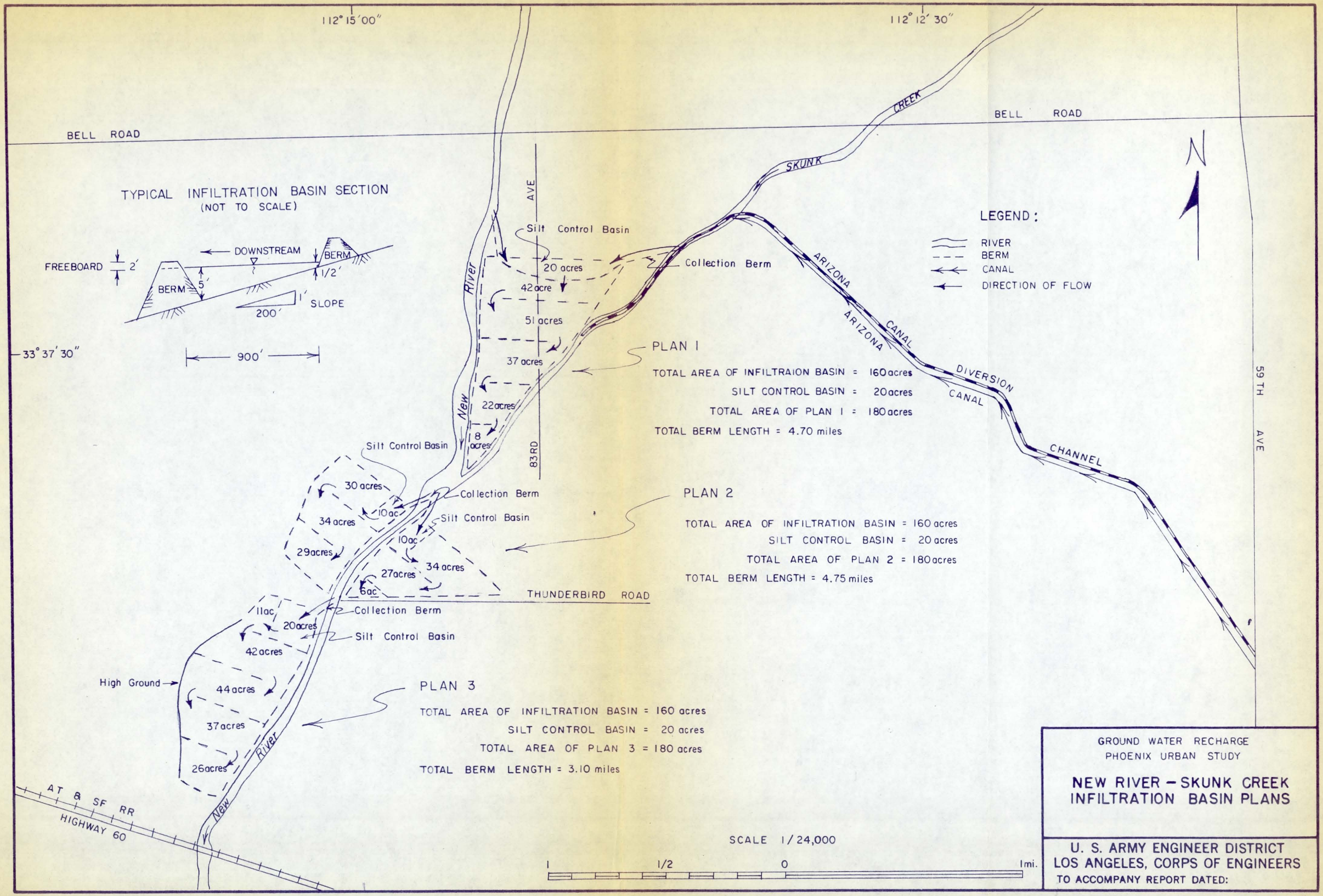
NOTE: ONLY TWO GROUPS OF INFILTRATION BASINS TO BE OPERATED SIMULTANEOUSLY WHILE THIRD IS DRYING.

GROUND WATER RECHARGE
PHOENIX URBAN STUDY

SALT RIVER INFILTRATION BASIN— PLAN 3

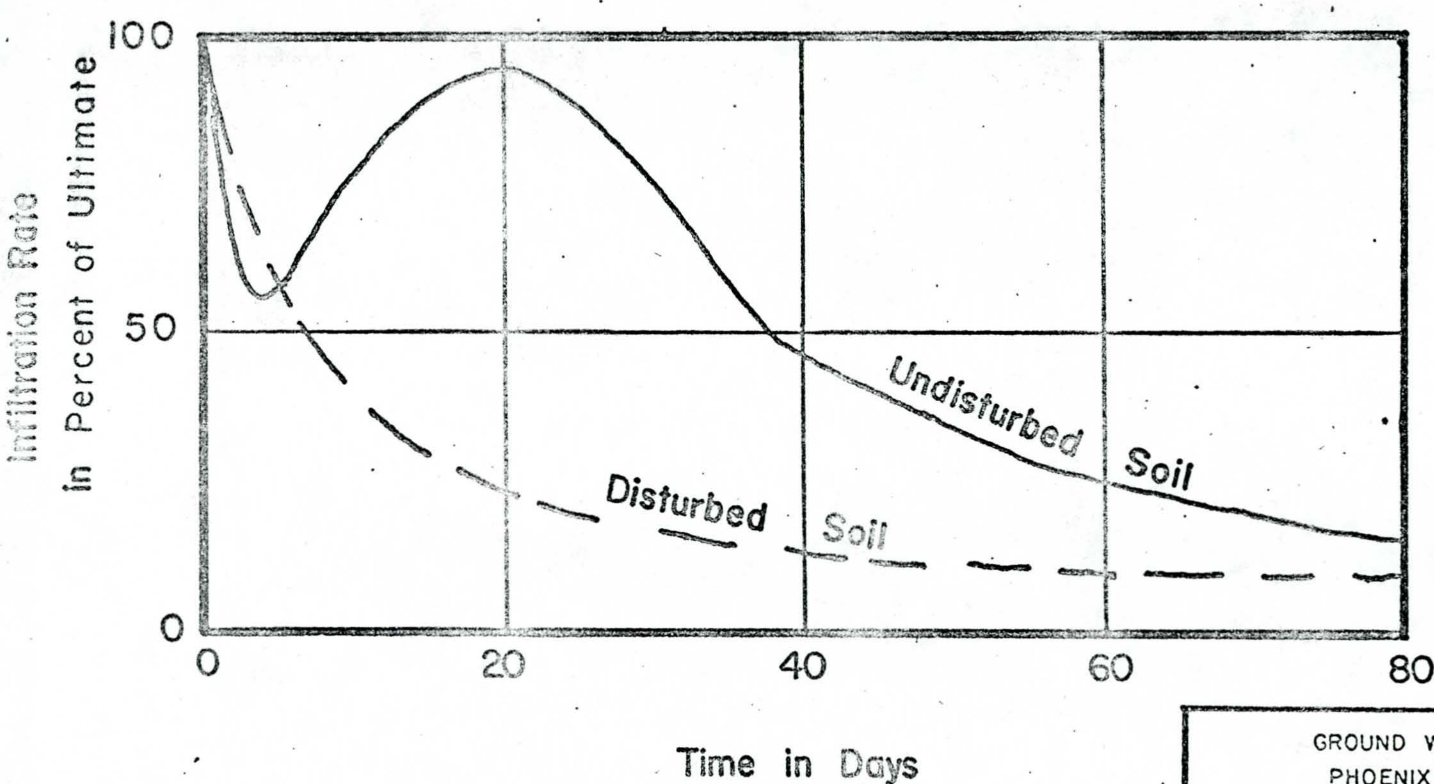
U S ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 5



112° 15′ 00″
112° 12′ 30″
33° 37′ 30″
BELL ROAD
BELL ROAD
SKUNK CREEK
TYPICAL INFILTRATION BASIN SECTION
(NOT TO SCALE)
DOWNSTREAM
FREEBOARD
2′
BERM
5′
BERM
1/2′
1′
SLOPE
200′
900′
83 RD AVE
Silt Control Basin
20 acres
42 acre
51 acres
37 acres
22 acres
8 acres
New River
Collection Berm
ARIZONA
ARIZONA CANAL
CANAL
DIVERSION
CANAL
CHANNEL
LEGEND:
RIVER
BERM
CANAL
DIRECTION OF FLOW
N
PLAN 1
TOTAL AREA OF INFILTRAION BASIN = 160 acres
SILT CONTROL BASIN = 20 acres
TOTAL AREA OF PLAN 1 = 180 acres
TOTAL BERM LENGTH = 4.70 miles
59 TH AVE
Silt Control Basin
30 acres
Collection Berm
10ac
34 acres
Silt Control Basin
10ac
29 acres
34 acres
27acres
6ac
PLAN 2
TOTAL AREA OF INFILTRATION BASIN = 160 acres
SILT CONTROL BASIN = 20 acres
TOTAL AREA OF PLAN 2 = 180 acres
TOTAL BERM LENGTH = 4.75 miles
THUNDERBIRD ROAD
11ac
Collection Berm
20 acres
Silt Control Basin
42 acres
High Ground
44 acres
37 acres
26 acres
New River
PLAN 3
TOTAL AREA OF INFILTRATION BASIN = 160 acres
SILT CONTROL BASIN = 20 acres
TOTAL AREA OF PLAN 3 = 180 acres
TOTAL BERM LENGTH = 3.10 miles
AT & SF RR
HIGHWAY 60
SCALE 1/24,000
1
1/2
0
1 mi.
GROUND WATER RECHARGE
PHOENIX URBAN STUDY
NEW RIVER – SKUNK CREEK
INFILTRATION BASIN PLANS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 6

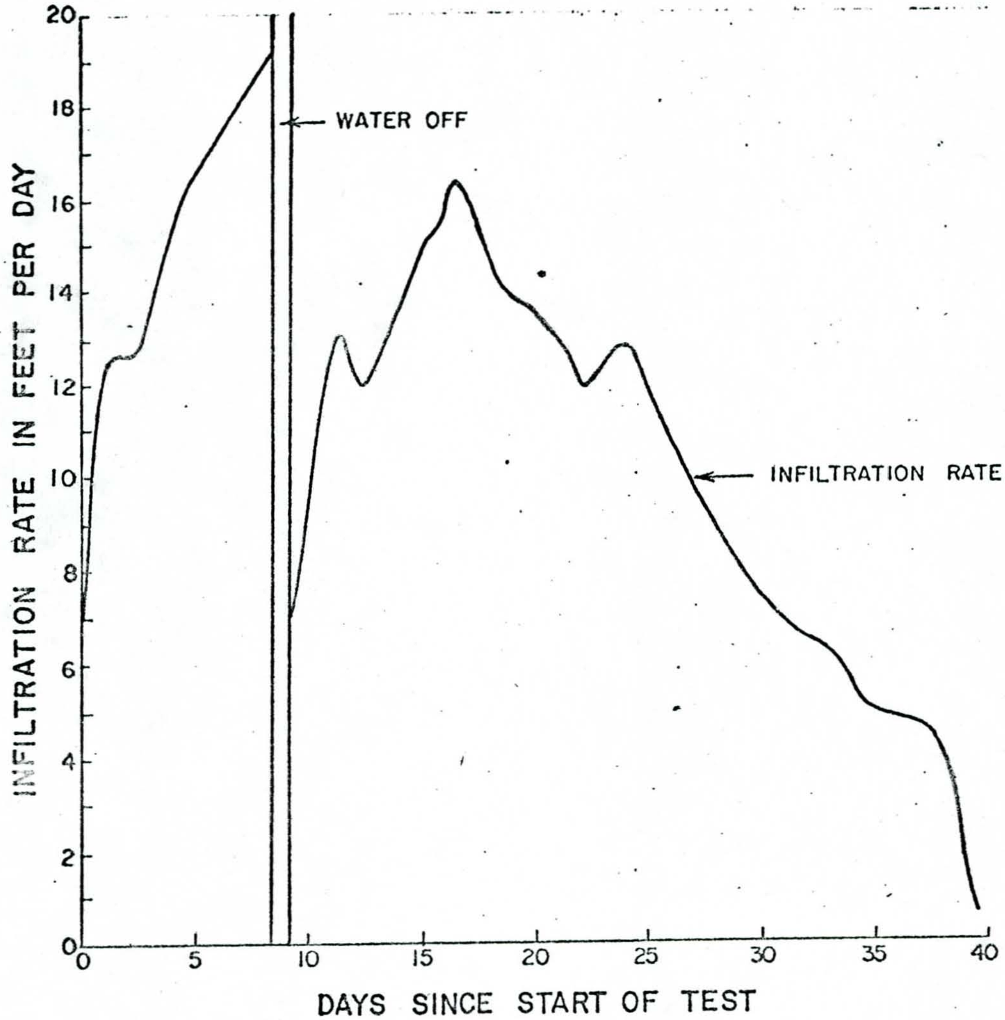


(By Muckel, reference 3)

GROUND WATER RECHARGE
PHOENIX URBAN STUDY

TYPICAL INFILTRATION RATE CURVES

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS



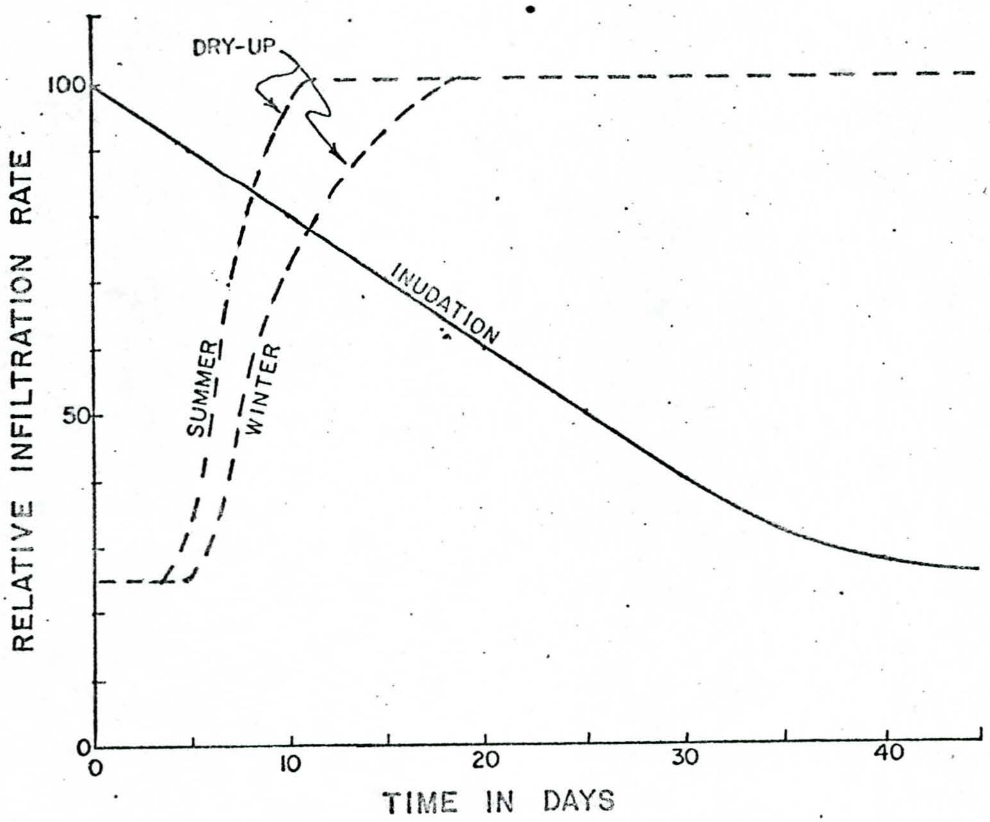
(By Tyley, Reference 2)

GROUND WATER RECHARGE
PHOENIX URBAN STUDY

INFILTRATION RATE CURVE
COACHELLA VALLEY, CALIF.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 8

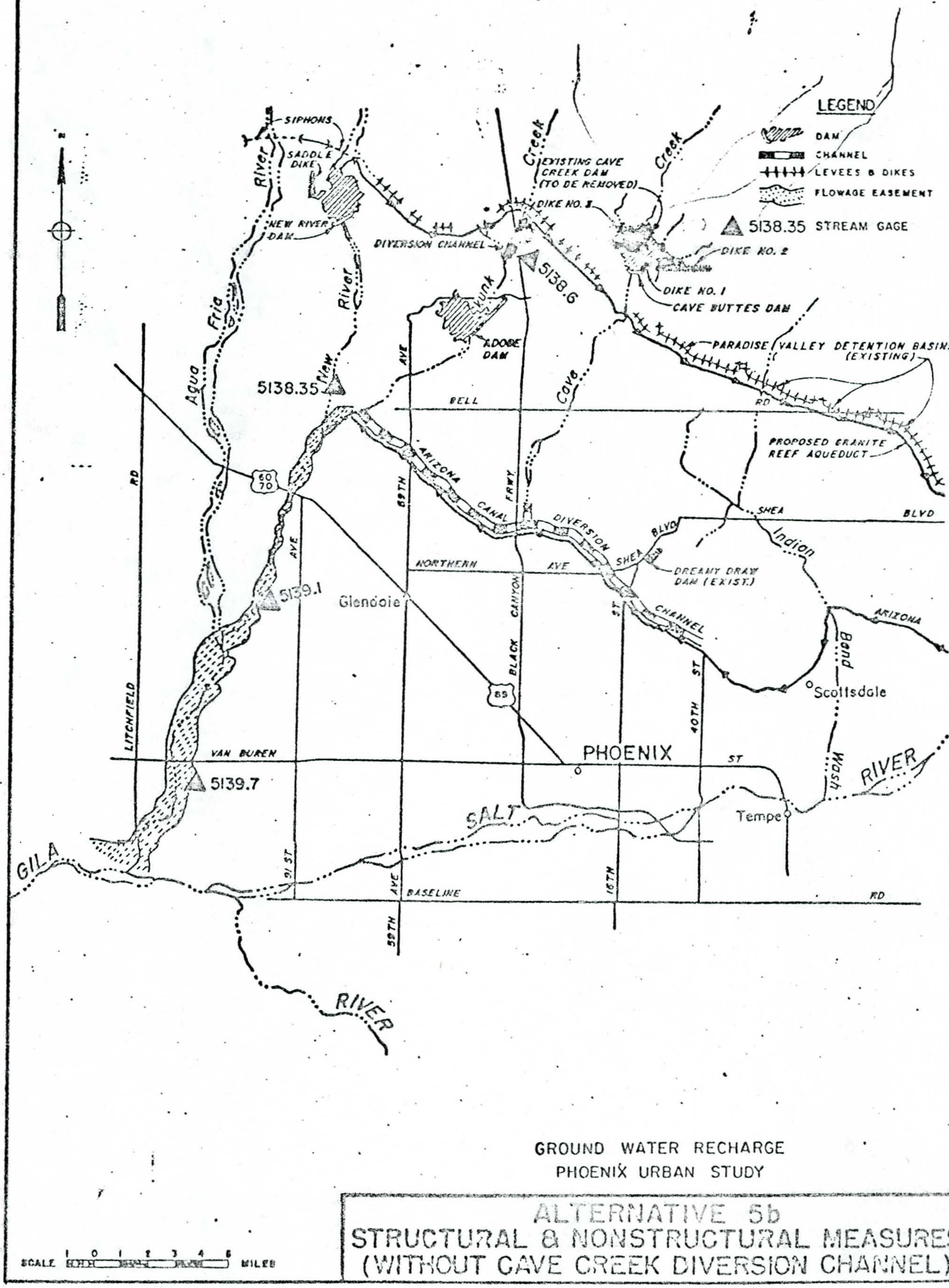


(By Bouwer, Reference 9)

GROUND WATER RECHARGE
PHOENIX URBAN STUDY

INFILTRATION RATE CURVE
FLUSHING MEADOWS PROJECT, ARIZ.

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS



GROUND WATER RECHARGE
PHOENIX URBAN STUDY

ALTERNATIVE 5b
STRUCTURAL & NONSTRUCTURAL MEASURES
(WITHOUT CAVE CREEK DIVERSION CHANNEL)

PLATE 10

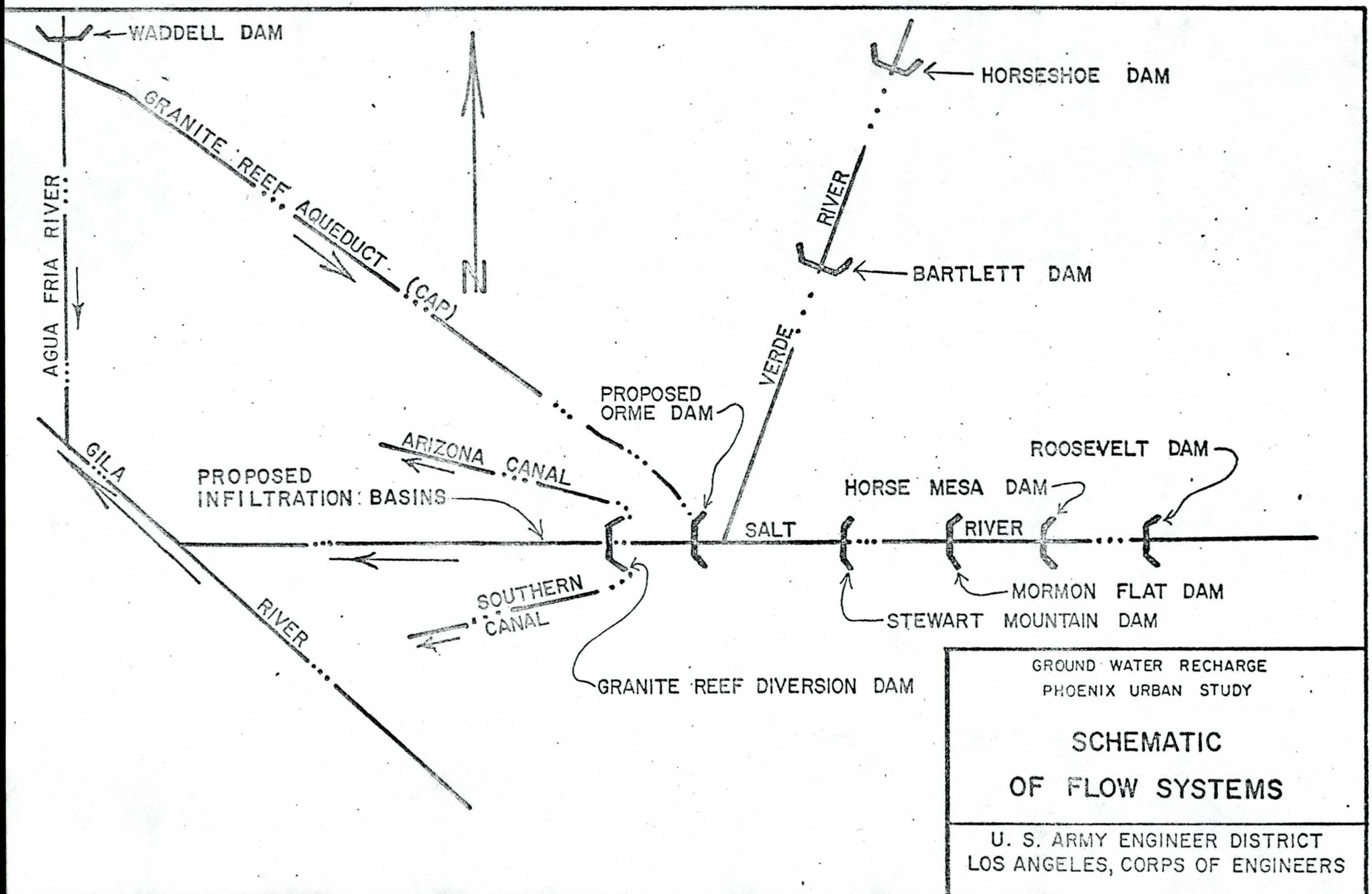
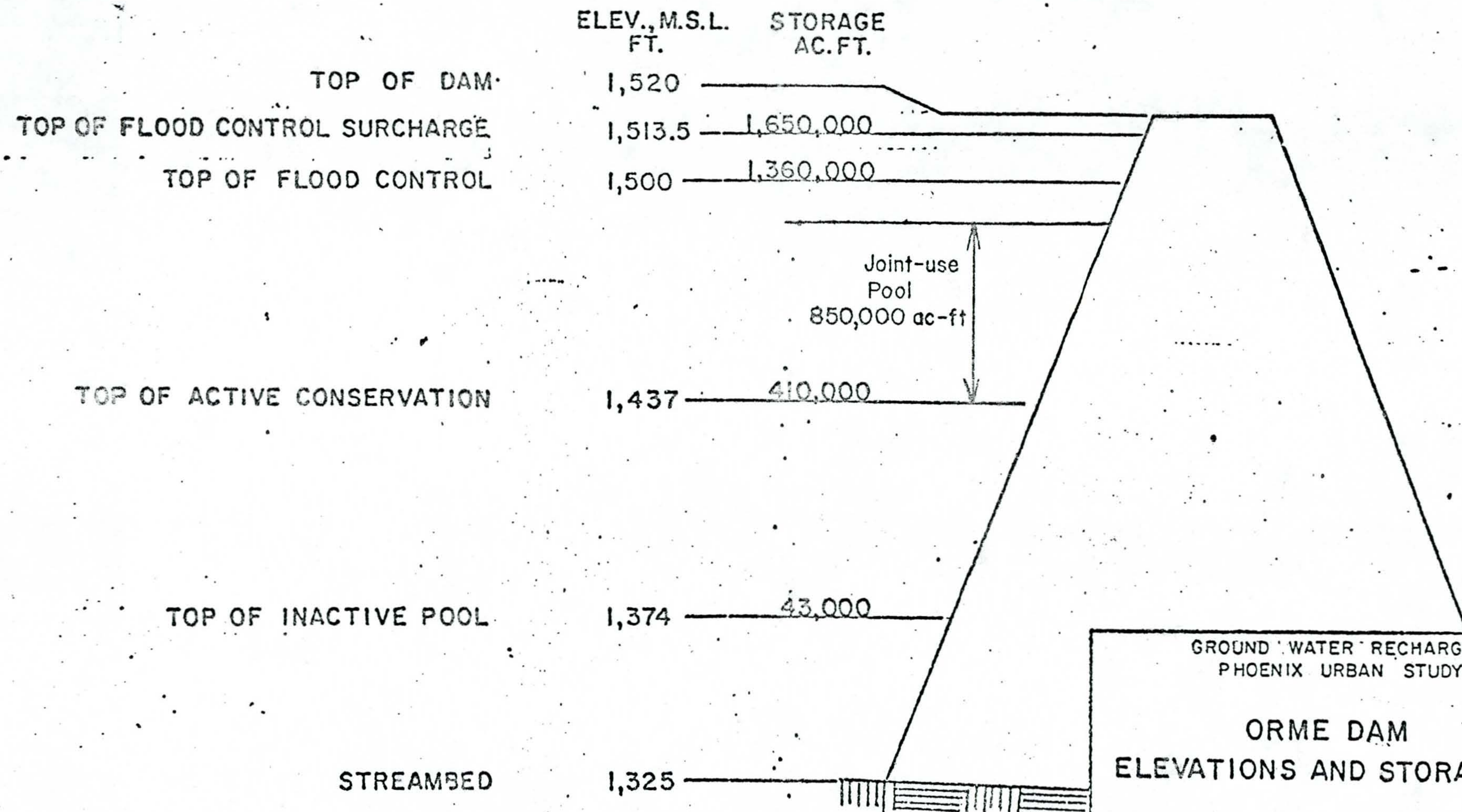

WADDELL DAM
GRANITE REEF AQUEDUCT (CAP)
N
AGUA FRIA RIVER
HORSESHOE DAM
VERDE RIVER
BARTLETT DAM
PROPOSED ORME DAM
ARIZONA CANAL
GILA RIVER
PROPOSED INFILTRATION BASINS
SALT RIVER
ROOSEVELT DAM
HORSE MESA DAM
MORMON FLAT DAM
STEWART MOUNTAIN DAM
SOUTHERN CANAL
GRANITE REEF DIVERSION DAM
GROUND WATER RECHARGE
PHOENIX URBAN STUDY
SCHEMATIC OF FLOW SYSTEMS
U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

PLATE 11

ORME DAM

TERMINAL STORAGE, WATER CONSERVATION AND FLOOD CONTROL

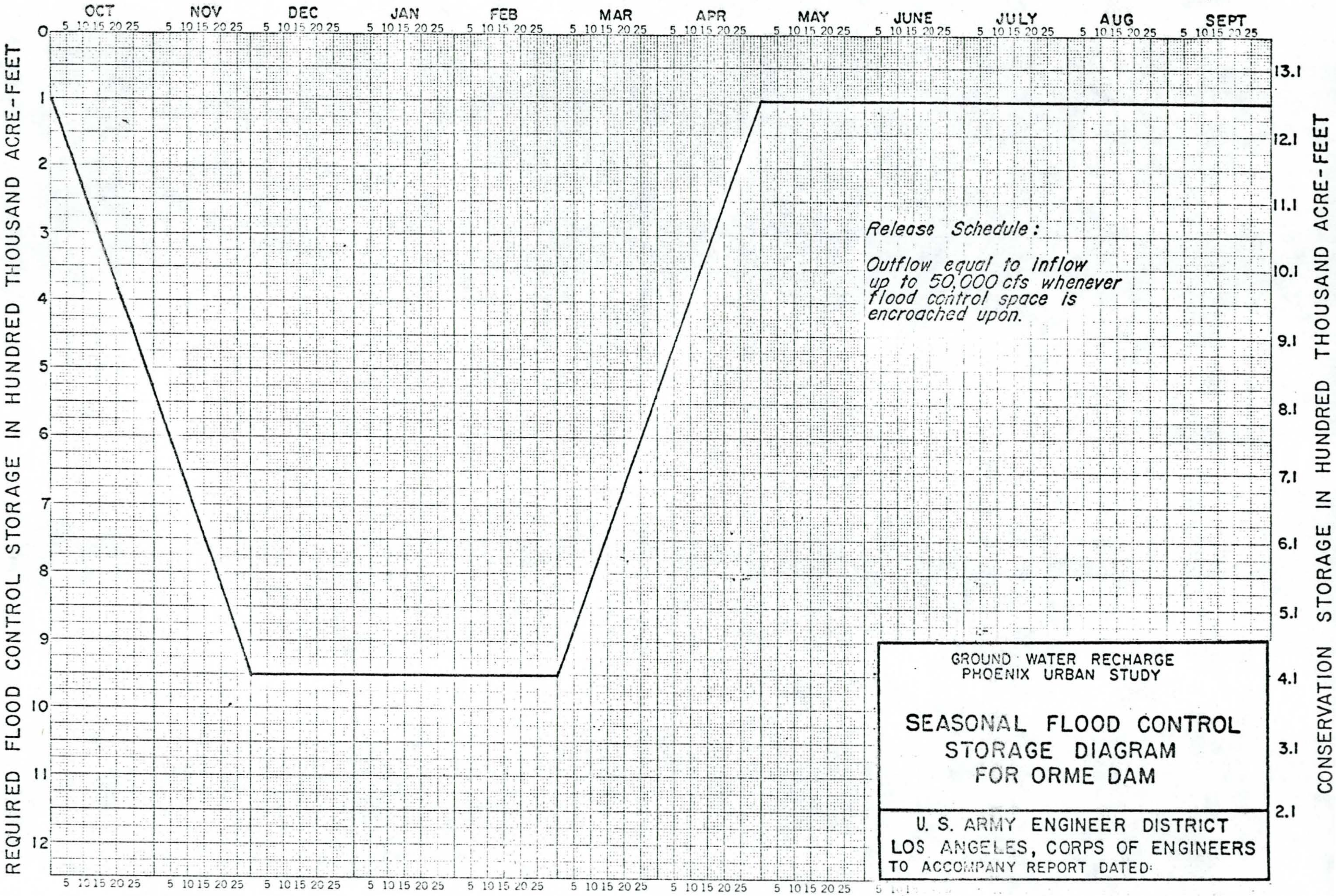


GROUND WATER RECHARGE
PHOENIX URBAN STUDY

ORME DAM
ELEVATIONS AND STORAGES

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED

PLATE 12

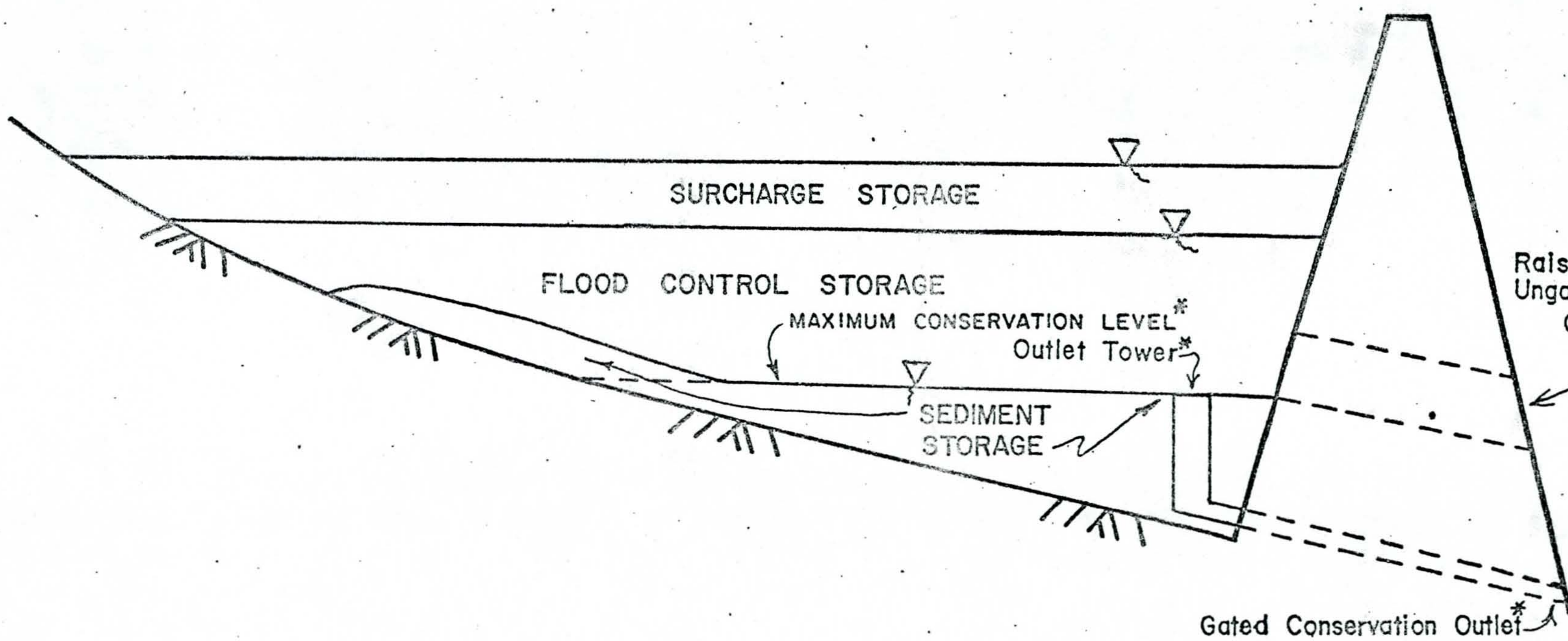


GROUND WATER RECHARGE
PHOENIX URBAN STUDY

SEASONAL FLOOD CONTROL STORAGE DIAGRAM FOR ORME DAM

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

PLATE 13



NOTE: Impoundment Of Storm Runoff In Sediment Storage Must Not Encroach Into Flood Control Storage.

* Modification

GROUND WATER RECHARGE
PHOENIX URBAN STUDY

MODIFICATION TO PROPOSED RESERVOIRS TO INCLUDE CONSERVATION STORAGE

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS

APPENDIX 1

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APPENDIX 2

GEOLOGIC CONDITIONS, INFILTRATION RATES, AND TRANSMISSIBILITY OF THE PHOENIX AREA

Geology Section

Los Angeles District, Corps of Engineers

Even though detailed refinements in the state of the art of infiltration analysis are being made all the time there still exists a need for knowledge of basic hydrologic parameters before recharge can be computed for an area. These parameters include; transmissibility and storage coefficient of the aquifer, description of the ground water basin and long term infiltration rates. "The infiltration rate of the spreading grounds must be high enough to accept the anticipated rate of recharge. The storage capacity of the ground water basin must be adequate to accommodate the anticipated volume of recharge. The transmissibility of the water bearing material must be sufficient to transmit the water away from the recharge site to the area of extraction." (Schaefer and Warner, 1975)

The transmissibility and storage coefficient for the Salt River area have been computed and have been used in an analog analysis of ground water depletion in Central Arizona (Anderson, 1968). East of Tempe Butte the transmissibility along the present course of the Salt River varies from 100,000 to 200,000+ gal/day/ft. An area of high transmissibility, 200,000+, trends south from the river passing nearby Mesa, Gilbert and Chandler before swinging to the west where it joins with a zone of high transmissibility under the Gila River. This zone of high transmissibility is estimated for the upper 1,000 to 1,200 feet of the aquifer and probably delineates the ancient course of the Salt River. West of Phoenix the area of highest transmissibility is coincident with the present course of the Gila River. Transmissibilities for the remainder of the study area range from 10,000 to 200,000+ gal/day/ft. An average storage coefficient of 19% was used in Anderson's analog analysis for Central Arizona and can be assumed to be a conservative value for the study area especially in the coarse river gravel deposits. Vertical and lateral permeabilities of the sand and gravel materials in the Salt River bed have also been measured (Bouwer, 1970). Generally speaking the average permeability in the anisotropic alluvium is approximately 300 ft/day in the horizontal direction and 20 ft/day in the vertical direction. Individual gravel layers are 40 times more permeable than sand layers.

Basins in central and southern Arizona are interconnected by thick accumulations of alluvium. Most hydrologic basins therefore do not have definite physiographic limits and their boundries are arbitrarily chosen. The study areais located in the Salt River Valley which is no exception to the general rule. The Salt River Valley is hydrologically connected with the Lower Santa Cruz Basin to the southeast and the Gila Bend Basin to the southwest. The Central Highlands Geomorphic Province serves as the headwaters for the valley and defines the basin boundary on the north and northeast.

Prior to 1923, the hydrologic system in central Arizona was considered to be in equilibrium (Anderson, 1968). With little exception the water table had the same general form as the surface of the ground, but with a lower gradient (Meizner and Ellis, 1915). In the Phoenix area, ground water moved obliquely to the west parallel to the surface drainages. Since 1923 the ground water has been extensively exploited to the point where water levels have declined as much as 360 feet in 40 years. "The large withdrawl from the ground water reservoir has changed the regional flow pattern from a relatively uniform undisturbed state to a series of individual systems, each one located at a relative center of pumping." (Anderson, 1968)

Although physiographically separate ground water basins do not exist in the study area, the effect of ground water mining has so altered the reservoir that a division in the basin occurs at Tempe. In 1972 east of Tempe along the Salt River the gradient sloped eastward, influenced by deep cones of depression such as those located at Scottsdale and Mesa. West of Tempe the gradient is to the west. The direction of flow is complex, however, being severly affected by areas of extensive pumping such as the one west of Litchfield Park.

The alluvium in the Salt River Valley varies in thickness from zero near bedrock exposures to 2,000 feet near the center of the basins. The valley fill has all sizes of particles ranging from impervious silt and clay to very permeable gravel and boulder beds. "The material is in lenticular layers or beds that apparently are not widely distributed horizontally" (Stulik and Twenter, 1964). Some exception to this general rule is along the present course and ancient courses of the Salt River which are characterized by coarse, permeable alluvium Even so, because the river has shifted its course from side to side in the valley, any vertical section in the alluvium can show a great degree of heterogeneity in material size and permeability.

Lee in his paper, <u>The Underground Water of the Salt River Valley</u> (pp., 128-131), gives a superb description of the character of the alluvium in the Phoenix and Mesa areas. In general, the course, permeable gravel, cobbles and boulders exist at the heads of the broad alluvial fans adjacent to the crystalline bedrock outcrops and along the present and ancient courses of the major streams especially the Salt River. The largest part of the basin is filled with fine grained sediments and evaporites characteristic of a typical intermontane basin (Stulik and Twenter, 1964). The uniform slope of the regional water table at equilibrium indicates that there is some communication between the water bearing formations. The actual path of this communication, however, may be complex. "If the ideal section given in figure 3 represents actual conditions, water from the river would pass laterally six times partly across the valley before reaching the lowest gravels" (Lee, 1905).

Since the downward percolation of water is at least partially delayed by clay, silt, caliche or evaporite layers, semi-perched conditions may locally develop. East of Tempe and north of Chandler a rather extensive perched condition is recognized. It varies in elevation above the regional water table from 20 to 200 feet.

In summary, movement of ground water in the study area at the present time is complex. Ground water levels and probable direction of flow are relatively well documented, however the situation is complicated by the varing pumping rates, the locally perched water and the lenticular nature of the alluvium. Even beneath the existing Salt River channel confining layers exist which serve to perch water above the regional water table (Laney, USGS Phoenix office, oral communication). At the present time the USGS in Phoenix is preparing a hydrogeologic atlas for the East Salt River Valley. The river gravel deposits are being delineated and isopach maps will be constructed which will help in estimating the extent of high permeability alluvium available for ground water storage. At the present time it can be assumed that a dewatered storage basin of considerable size characterized by high vertical and lateral permeabilities exists in the east Salt River Valley.

Long term infiltration rates for the Salt River have been computed using two different methods. Heavy runoff in the spring of 1973 was monitered by the USACE and the USGS. Outflow from Granite Reef Dam was compared to inflow at Painted Rock reservoir. By using the wetted area and correcting for time lag, average infiltration rates were obtained for the entire 107 mile length of the Salt and Gila Rivers between the two dams. Three individual days were looked at; 31 March, 9 April and 4 May. Average percolation rates of .74, .24 and .22 ft/day were calculated for these days - 27, 36 and 61 days after the initial release from Granite Reef.

In another study at the Flushing Meadows project near 91st Ave., long term infiltration rates of 1 to 4 ft/day were obtained under closely controlled conditions in small (20x700') recharge basins (Bouwer, 1970). The surface material in these basins is a loamy sand which is probably the major limiting factor for the infiltration rates. Whether or not these basins are an accurate reflection of infiltration rates further upstream or in coarser surface material is questionable.

These long term values can be compared with short term infiltration rates ranging from 1.4 to 2.5 ft/day obtained on the Salt River from Granite Reef to 7th Ave. (Briggs, 1966). At the present time the data obtained during the 1973 flows down the Salt River along with data obtained during a similar event in 1968 are being evaluated further by the Arizona Water Commission. Before any long term infiltration rate can be assigned to the Salt River channel with any validity the wide variation between rates measured during actual flows and experimental tests must be resolved.

Summary and recommendations. From the study conducted thus far it appears that the Salt River channel, primarily east of Tempe Butte, is a good natural recharge site located within the study area. A report currently being prepared by the USGS will in more detail define the available ground water storage reservoir in this area. Transmissibility of the river deposits are sufficient to carry water away from recharge areas. A more precise estimate of long term infiltration rates is still needed however. Infiltrometer tests, test holes and small scale recharge test basins in the river channel would be of limited value. Only by monitioring the effects of long term controlled releases down the river channel can useful data be obtained. Until this data becomes available it is reasonable to assume long term infiltration rates of at least 1 foot per day in relatively fine grained river deposits and 5 feet per day in coarser deposits if reasonable management techniques are employed.

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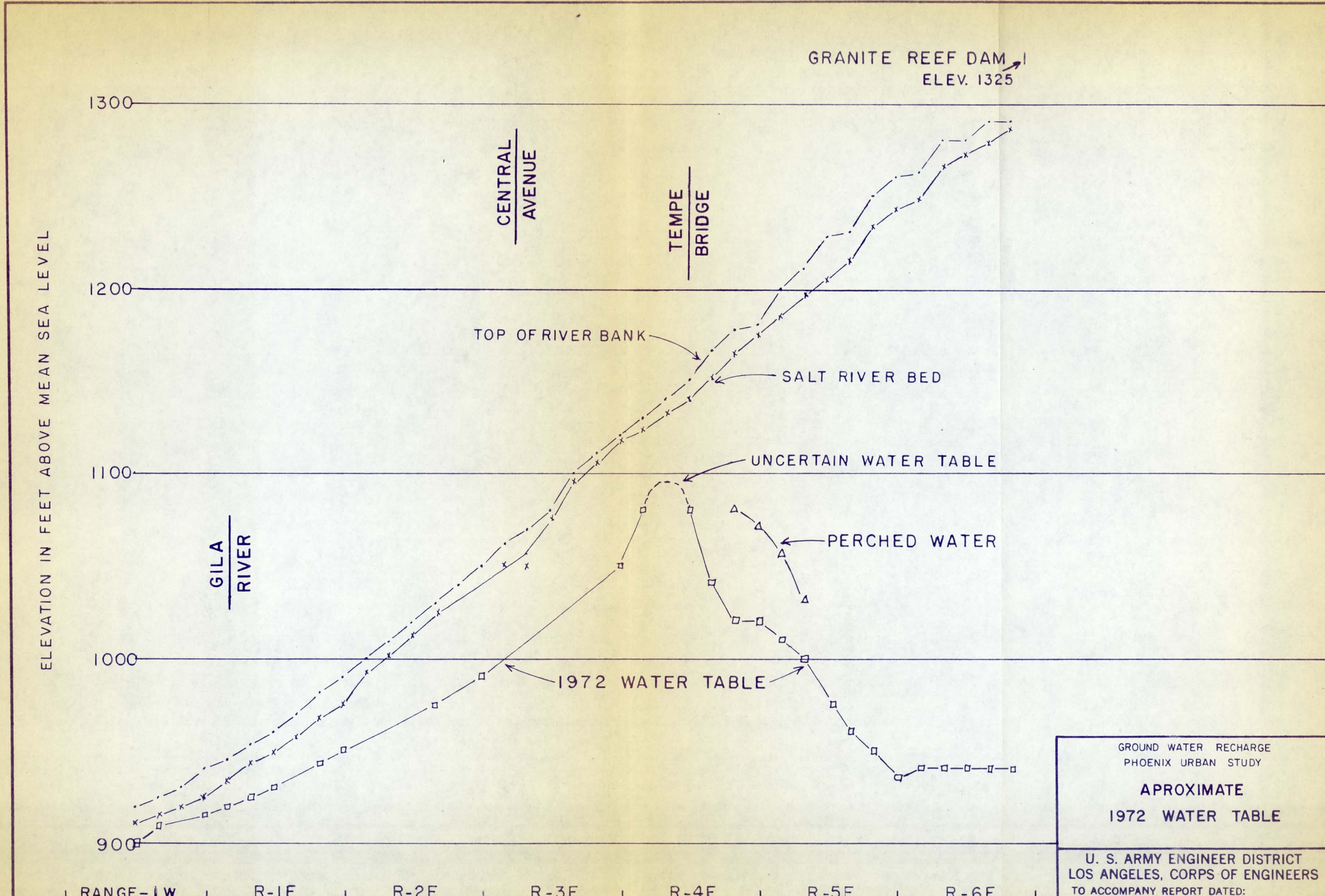


GROUND WATER RECHARGE
PHOENIX URBAN STUDY

APROXIMATE
1972 WATER TABLE

U. S. ARMY ENGINEER DISTRICT
LOS ANGELES, CORPS OF ENGINEERS
TO ACCOMPANY REPORT DATED:

APPENDIX 2 PLATE I

APPENDIX 3

ANALYSIS OF FLOOD ROUTING THROUGH CHANNELS WHICH HAVE MAJOR INFILTRATION LOSSES

Hydrologic Engineering Center

Corps of Engineers

DEPARTMENT OF THE ARMY
SACRAMENTO DISTRICT, CORPS OF ENGINEERS
THE HYDROLOGIC ENGINEERING CENTER
609 2D STREET, DAVIS, CALIFORNIA 95616

SPKHE

26 July 1976

SUBJECT: Analysis of Flood Routing Through Channels Which Have Major Infiltration Losses

District Engineer
US Army Engineering District, Los Angeles
Phoenix Urban Study Office
2721 North Central Avenue, Suite 800
Phoenix, Arizona 85004

1. References:

a. SPKHE Special Projects Memo No. 463, dated 23 March 1976.

b. SPLED Form 2544, dated 14 April 1976.

2. Inclosed are two copies of HEC Special Projects Memo No. 470. This memo constitutes the final report on the work described as "approach (b)" in Reference a.

FOR THE DISTRICT ENGINEER

1 Incl
as

BILL S. EICHERT, Director
The Hydrologic Engineering Center

CF: Dan Norling
Los Angeles District

SPKHE 20 July 1976

SPECIAL PROJECTS MEMO NO. 470

SUBJECT: Infiltration and Unsteady Open Channel Flow: An Approach to Solving the Coupled System

PART I

Problem Statement and Solution Methodology

1. References.

a. SPKHE Special Projects Memo No. 463, dated 23 March 1976

b. SPLED Form 2544, dated 14 April 1976

c. Users Manual for HEC generalized computer program No. 723-G2-L7450, "Gradually Varied Unsteady Flow Profiles", June 1976

d. Amorocho, J., et. al., "Simulation of Runoff from Arid and Semiarid Climate Watersheds," Volume I, University of California, Davis, Water Science and Engineering Paper #3002, June 1973.

2. Introduction. A version of the generalized computer program "Gradually Varied Unsteady Flow Profiles" (referred to herein as "the unsteady flow model"), which calculates infiltration rates and incorporates the effects of infiltration on calculated unsteady flow profiles and discharges, has been developed by The Hydrologic Engineering Center. This memo describes the theoretical basis of the solution technique, application of the model to two flood events on the Salt River near Phoenix, Arizona, and data input requirements and structure. Capabilities and limitations of the technique are discussed.

3. Theoretical Basis of the Model.

a. The governing equations for one-dimensional unsteady open channel flow (the St. Venant equations) can be written:

$$\frac{\partial(AV)}{\partial x} + B\frac{\partial h}{\partial t} - q = 0 \qquad (1)$$

$$\frac{\partial h}{\partial x} + \frac{V}{g}\frac{\partial V}{\partial x} + \frac{1}{g}\frac{\partial V}{\partial t} + \frac{1}{g}\frac{qV}{A} = S_f \qquad (2)$$

Where:

A = cross-sectional flow area

V = mean cross-sectional velocity

x = distance along the channel

B = water surface width

h = water surface elevation

t = time

q = lateral inflow per unit length of channel

g = gravitational acceleration

S_f = friction slope, calculated from Manning's equation

The above two equations are derived from the physical principles of continuity (eq. (1)) and energy conservation (eq. (2)) and are considered generally applicable to one-dimensional rigid boundary open channel flows. Any consistent system of units may be used. This set of partial differential equations cannot, in general, be solved (integrated) analytically to obtain h and V as functions of time (t) and space (x). Several numeric schemes are available, however, for calculating approximate solutions to the St. Venant equations. One such scheme, termed an "explicit, centered difference scheme" provides the basic algorithm utilized by the unsteady flow model. Details are presented in the users manual (Ref. c).

b. Note that the equations of motion that are programmed into the unsteady flow model include a lateral inflow term, q. While this term is usually thought of as representing tributary inflow, it can equally well be thought of as tributary outflow (q negative). Therefore, this term can represent infiltration losses to the channel bed. Consequently, to apply the unsteady flow model to the propagation of flood waves through channels that exhibit major infiltration loss, no change in the basic equations of motion or the algorithm for integrating those equations is necessary. What is required, however, is a method for determining a value for q (i.e., the infiltration loss rate per unit length of channel), which may depend on time, location, etc.

4. Loss Rate Determination. The rate at which water percolates through the soil surface is usually expressed as discharge per unit area, i.e., a velocity. This velocity, f, is generally considered to be a function of several variables:

f = f (duration of wetting, depth of flow, soil characteristics, antecedent conditions, etc.)

For the purposes of this investigation the infiltration rate was assumed to be of the form:

$$f = f_c + (f_o - f_c)\, e^{-kt} \tag{4}$$

Where:

f = instantaneous infiltration rate at any distance, x

f_c = ultimate, constant, infiltration rate

f_o = infiltration rate at time zero

k = a decay coefficient

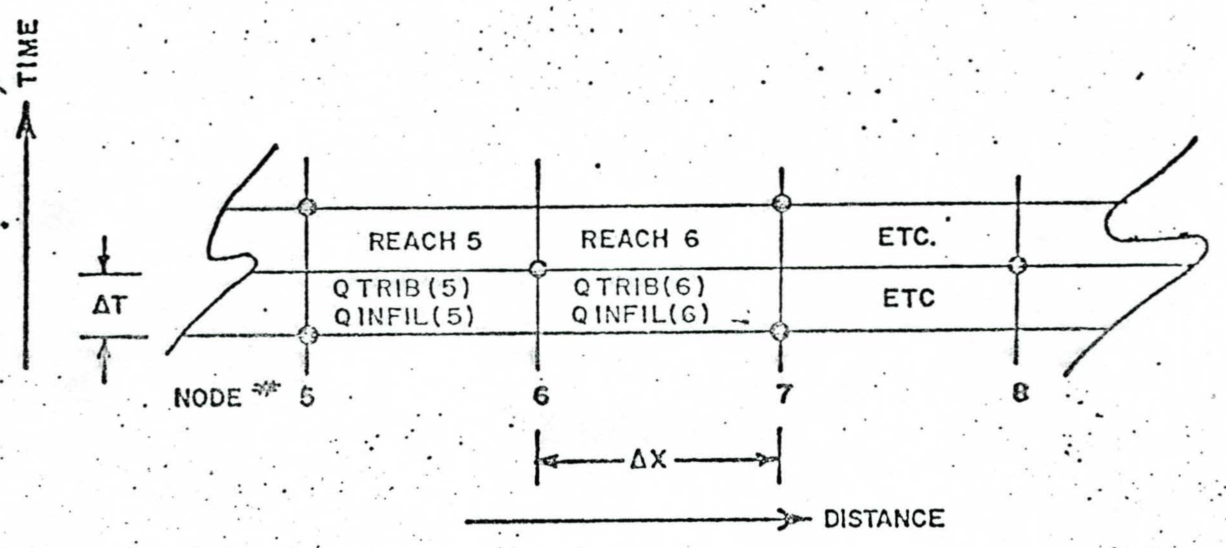
t = time, reckoned from the time at which the flood wave initially reaches location x

This is essentially Horton's formula and has been applied to similar channel loss problems in the arid Southwest (Ref. d). As used in this study, f_c, f_o, and k may vary along the length of the channel being modeled. The time, t^o, also varies along the channel due to the finite travel time of the flood wave. Note, however, that f is assumed to be constant across the cross-section; this is consistent with the one-dimensional equations of motion that are used.

5. Interfacing Loss Rate and Unsteady Flow Calculations.

a. The unsteady flow model solves finite difference approximations of equations (1) and (2). The computations are executed at discrete, evenly spaced, locations along the channel. These locations are termed "nodes". The length of channel between two nodes is termed a "reach". A reach is of length Δx. A different value of the total lateral inflow, Q_{TRIB}, may be assigned to each reach. At interior (non-boundary) nodes the local inflow assigned to any node is that for the reach on the left plus that for the reach on the right (see Fig. I-1). The boundary calculations are more complex and depend upon the type of boundary condition specified.

b. Infiltration is included in this scheme by associating with each reach a total local inflow rate composed of two parts: (1) tributary inflow (or outflow) occurring within the reach, and (2) infiltration which occurs



At Node 6: $q(6) = \frac{1}{2\Delta x}\left(Q_{TRIB}(5) - Q_{INFIL}(5) + Q_{TRIB}(6) - Q_{INFIL}(6)\right)$

At Node 7: $q(7) = \frac{1}{2\Delta x}\left(Q_{TRIB}(6) - Q_{INFIL}(6) + Q_{TRIB}(7) - Q_{INFIL}(7)\right)$

etc.

Where: $Q_{INFIL}(4) = Q_{INFIL}(5) = f(5) \cdot B(5) \cdot \Delta x$

and

$Q_{INFIL}(6) = Q_{INFIL}(7) = f(7) \cdot B(7) \cdot \Delta x$

etc.

FIGURE I-1: Accounting Scheme for Q_{TRIB} and Q_{INFIL}, Interior Nodes in the reach. The first is an input item, provided by the user; the second is calculated from equation (4) as follows:

$$Q_{INFIL} = f \cdot (\Delta x) \cdot (B) \tag{5}$$

where B is a water surface width obtained from the unsteady flow calculations. These calculations, however, only yield results for nodes. It was decided that the appropriate width to use for a given reach is that calculated at the odd node located at one end of the reach. (For a discussion of the significance of odd and even nodes see Ref. c.) The net local inflow or outflow for a reach is then found by subtracting Q_{INFIL} from Q_{TRIB}. The computational procedure for solving equations (1) and (2) is unaltered, only the local inflow term (q) is recalculated to reflect infiltration.

c. The infiltration rate, f, is a function of time. The proper time to use in calculating f is time after passage of the flood wave. At each odd node the time transpired since arrival of the flood is accumulated and the loss rate for the two reaches adjacent to that odd node is calculated from equations (4) and (5).

d. The unsteady flow model requires a finite depth of water at each node, i.e., there can be no "dry" nodes. Therefore, the advance of a flood wave on a dry channel can only be simulated approximately. Version 3.0 of the unsteady flow program (c) contains a procedure for calculating a minimum elevation at each node below which the calculated water surface elevation is not allowed to fall. This is a contrivance that allows some difficult problems, such as the dam break flood, to be analyzed more efficiently. The procedure is equivalent to adding water to the flow and, therefore, does not correspond to the correct solution for the given problem. Appropriate warnings are printed when this occurs and the user must examine the rest of the solution to determine if it is realistic.

e. The minimum elevation concept can be utilized to identify the arrival of the flood wave at each node and "turn on" the infiltration calculation. Initially, a water surface profile for low flow (steady) conditions should be determined. This profile is then specified as the minimum elevation for the flood hydrograph simulation. (The program version developed for this study allows user specification of minimum elevations at each node.) As the flood wave progresses downstream, the water surface elevation at each node increases above the minimum, turning on the infiltration algorithm. If, at any later time, the elevation drops to the minimum again, infiltration is turned off but the time is not reset to zero.

f. It was found necessary to limit the infiltration rate for water surface elevations slightly above the minimum to that which would just drop the water surface to the minimum in one computation cycle. The minimum elevation behaves essentially as a channel bed with respect to infiltration. The presence of a finite depth of water does, however, affect the rate of propagation of the flood wave. This is discussed further in Part II - Test Application.

6. Limitations.

a. Choice of loss rate parameters f_o, f_c, and k may be extremely difficult, particularly if they are allowed to vary with distance along the channel. Choice of appropriate values may also be difficult because the relationships between these parameters and soil characteristics, antecedent conditions, etc. are not well known. Adjustment of infiltration parameters to achieve calibration may prove tedious until adequate experience in applying the model is obtained.

b. The duration of event that can be feasibly analyzed may prove to be on the order of several days. The cost of operating the model for any given duration of simulation depends upon the computation interval and the number of nodes in the model. Explicit solutions of the St. Venant equations are constrained to computation intervals of a maximum size by a stability criterion, which, in turn, depends on hydraulic and geometric parameters. Previous applications of the unsteady flow model have utilized computation intervals ranging from one second to five minutes. Addition of an infiltration loss calculation did not appear to further restrict the computation interval.

c. The necessity of having a finite depth of water in the channel at all times requires the user to carefully construct a low-flow profile before the unsteady event can be analyzed. This step can be extremely helpful, however, in identifying flaws in the geometric model.

7. Summary and Conclusions.

a. The capability for calculating infiltration losses and simultaneously solving the hydraulics of unsteady, one-dimensional, open channel flow has been developed. The system of equations used to describe the open channel flow are complete; no terms have been dropped.

b. The loss rate function used herein provides for an exponential decay of infiltration rate with time after arrival of the flood wave. Since the basic open channel flow equations have been unaltered, the use of other loss rate functions could be explored without major re-programming.

c. The total infiltration rate in any reach is calculated as the rate per unit area times the water surface width times the reach length. Since width is a function of water surface elevation, which varies with time, the changes in bed surface area during passage of the flood hydrograph are incorporated into the infiltration calculations.

d. The program calculates and prints out the instantaneous infiltration rate within each reach and the cumulative volume of water infiltrated in the entire system at any time.

e. Additional data input requirements to operate the infiltration algorithm are minimal.

f. As is shown in Part II, the first application of the model to a system with significant infiltration losses yielded good reproduction of observed discharge hydrographs.

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PART II

Test Application

1. Site Description. The method was applied to an 18-mile length of the Salt River near Phoenix, Arizona (from Jointhead Dam to Granite Reef Dam). See Fig. II-1. Cross-section data at 25 sections was provided by the Arizona Water Commission. Also provided by the Commission were observed discharge hydrographs at Granite Reef and Jointhead Dams for four events in 1968 and a stage-discharge relation at the downstream boundary.

2. Development of the Geometric Model. Cross-section data was transformed into tables of geometric elements vs elevation at 37 evenly spaced nodes by use of the computer program "Geometric Elements from Cross-Section Coordinates". The nodal spacing (Δx) was 2637.5 ft. (approximately 0.5 miles). A Manning's n-value of 0.035 (from the Commission's data) was used. The geometric elements were calculated for an elevation range of 41 feet. A large range is desirable to assure that the calculated water surface elevation is always within the range of the geometric elements table.

3. Development of Initial Conditions.

a. The simulation of unsteady open channel flows requires that initial conditions (water surface elevation and discharge at time zero) be specified at every node. As discussed previously, the appropriate initial condition for this study is a low flow profile. The initial condition was developed by operating the unsteady flow model. Starting with an arbitrary water surface elevation and discharge, the model is run with a constant inflow. Eventually, transients introduced by the arbitrary guess at the profile are dispersed and an equilibrium steady state condition is reached. This condition is approached asymptotically and required simulation of several days of prototype pseudo-time. The discharge chosen for calculation of a steady flow profile was 100 cfs. Later results indicated that a smaller value would have provided a better starting condition for the dynamic flood simulation. During this phase of the study, infiltration was not used. A thalweg plot and low-flow water surface profile are shown in Fig. II-2.

b. The computation interval was initially set at 5 seconds; this was stable, 10 seconds was not. It was found that 6 seconds was usable, and that value was used for the majority of runs.

4. Application of the Model to a Flood Event.

a. The first event simulated was that of 14-18 February 1968. Observed inflow and outflow hydrographs are shown in Fig. II-3. The model was operated with the observed inflow hydrograph as the left (upstream) boundary condition and the stage-discharge relation (Fig. II-5) as the right (downstream) boundary condition. The calculated discharge as a function of time at the downstream boundary was then compared with the observed outflow hydrograph to evaluate the model's performance.

b. Infiltration rates need not be re-calculated every computation interval. The rate was updated every 8 computation intervals (i.e., every 48 seconds) in this study. No investigation of the sensitivity of results to variations of the frequency at which f is re-calculated was undertaken. The purpose of this feature is to save computation time by reducing the number of times an exponential function must be calculated.

c. The computation interval of 6 seconds, developed when calculating initial conditions, was also used when simulating the flood and infiltration flows. The solution remained stable, so it appears that the inclusion of infiltration flows has little impact on the basic stability criterion.

5. Calibration and Results.

a. Calculated outflow hydrographs for several values of the infiltration parameters, f_o, f_c, and k are shown on Fig. II-3 along with observed data. Variation of infiltration parameters with distance along the channel was not attempted. Also shown on Fig. II-3 is the calculated hydrograph for zero infiltration. As expected, the zero infiltration case exhibits higher discharges and smaller travel times than observed. The indication is that significant infiltration is indeed occurring. The set of parameters that best matches the observed data appears to be: f_o = 100 in/hr., f_c = 0.2 in/hr, and k = 1.0/hr. Note that the calculated solutions tend to rise sooner (calculated wave travels faster than observed) and have a lower initial peak than observed. In other words, the rising limb is "smeared" somewhat. It is suspected that this result is a consequence of having a finite water depth initially. However, the rate of rise is also affected by the rating curve used for the downstream boundary. A single run was made with a rating curve displaced downward about 0.5 feet from that shown on Fig. II-5. The behavior of the rising limb was substantially improved by using the modified rating curve (results not plotted).

b. The Arizona Water Commission suggested that an infiltration rate of 1.25 in/hr might be appropriate for the study reach. The steady state rate indicated by this study was 0.2 in/hr. Comparison of these two values may be inappropriate, however, if the 1.25 in/hr is interpreted as being average over the event, thereby including the high initial rate. Averaging equation (4) over the period simulated yields a rate of about 1.3 in/hr with f_0 = 100 in/hr, f_c = 0.2 in/hr, and k = 1.0/hr. This interpretation was suggested by the Los Angeles District.

c. The relative importance of the initial infiltration rate, f_0, and the decay coefficient, k, is difficult to establish because their respective impacts on shape and magnitude of the rising limb of the hydrograph are not easily separable from that of the initial and boundary conditions. Improved initial and/or boundary conditions should be developed and used to better evaluate these parameters.

d. The model was applied to a second event; that of 25-29 February 1968. Results are shown in Fig. II-4. This event appeared to have a somewhat higher infiltration rate, $f_c \simeq 0.3$ in/hr. Again the initial rise was somewhat smoothed. The same initial conditions were used.

e. Tabulated below are calculated maximum water surface elevations for zero infiltration and for the case: f_0 = 5 in/hr, f_c = 0.3 in/hr, and k = 0.5/hr. The values shown are for the period 25-27 February 1968 (event 2). It appears that, during this particular event, the action of infiltration decreased maximum water surface elevations by something less than 0.5 feet.

Mile	Max Elev Zero Infiltration	Max Elev With Infiltration
17.98	1296.27 ft. MSL	1296.12 ft. MSL
15.99	1276.68	1276.35
13.99	1264.59	1264.15
11.99	1250.53	1250.15
9.99	1229.68	1229.26
7.99	1193.71	1193.46
6.00	1186.96	1186.62
4.00	1162.41	1162.20
2.00	1138.69	1138.52
0.0	1135.78	1135.54

6. Computer Costs. The simulations were run on the CDC 7600 located at the Lawrence Berkeley Laboratory. Central processing times and approximate costs to HEC for some typical runs are shown below:

Run	C. P. Seconds	Approx Cost	Remarks
Development of geometric model	1.12	$1.47	
Event 1, 2.67 day simulation	33.66	9.66	
Event 1, 1.67 day simulation	21.28	6.58	
Event 1, 1.67 day simulation	18.93	6.02	No infiltration calculation
Event 2, 1.5 day simulation	17.39	5.67	No infiltration calculation
Event 2, 1.5 day simulation	19.58	6.16	
Event 2, 2.0 day simulation	26.11	7.98	

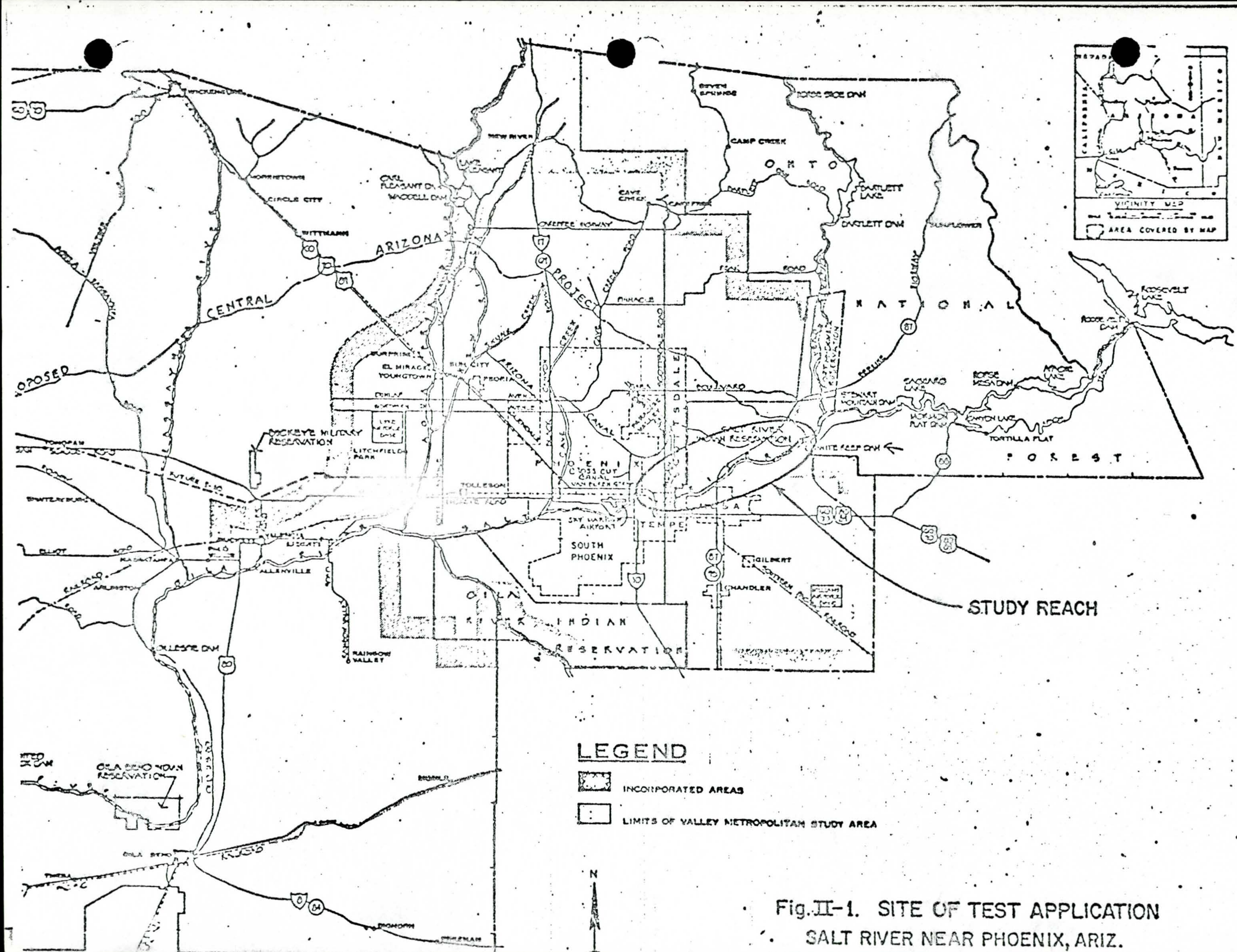

Fig. II-1. SITE OF TEST APPLICATION
SALT RIVER NEAR PHOENIX, ARIZ.

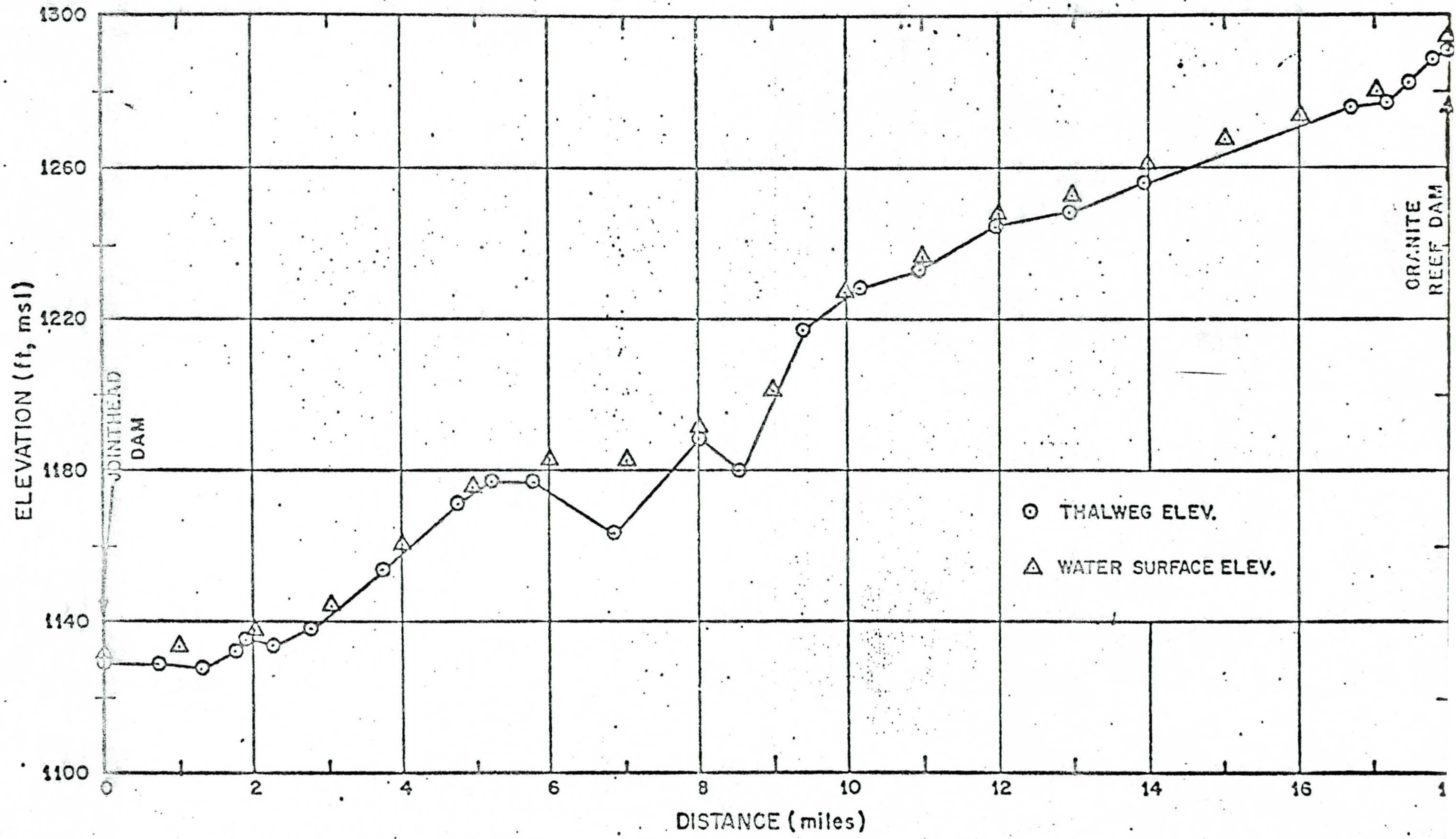

Fig. II-2. THALWEG AND INITIAL CONDITION (Q=100cfs) WATER SURFACE PROFILE

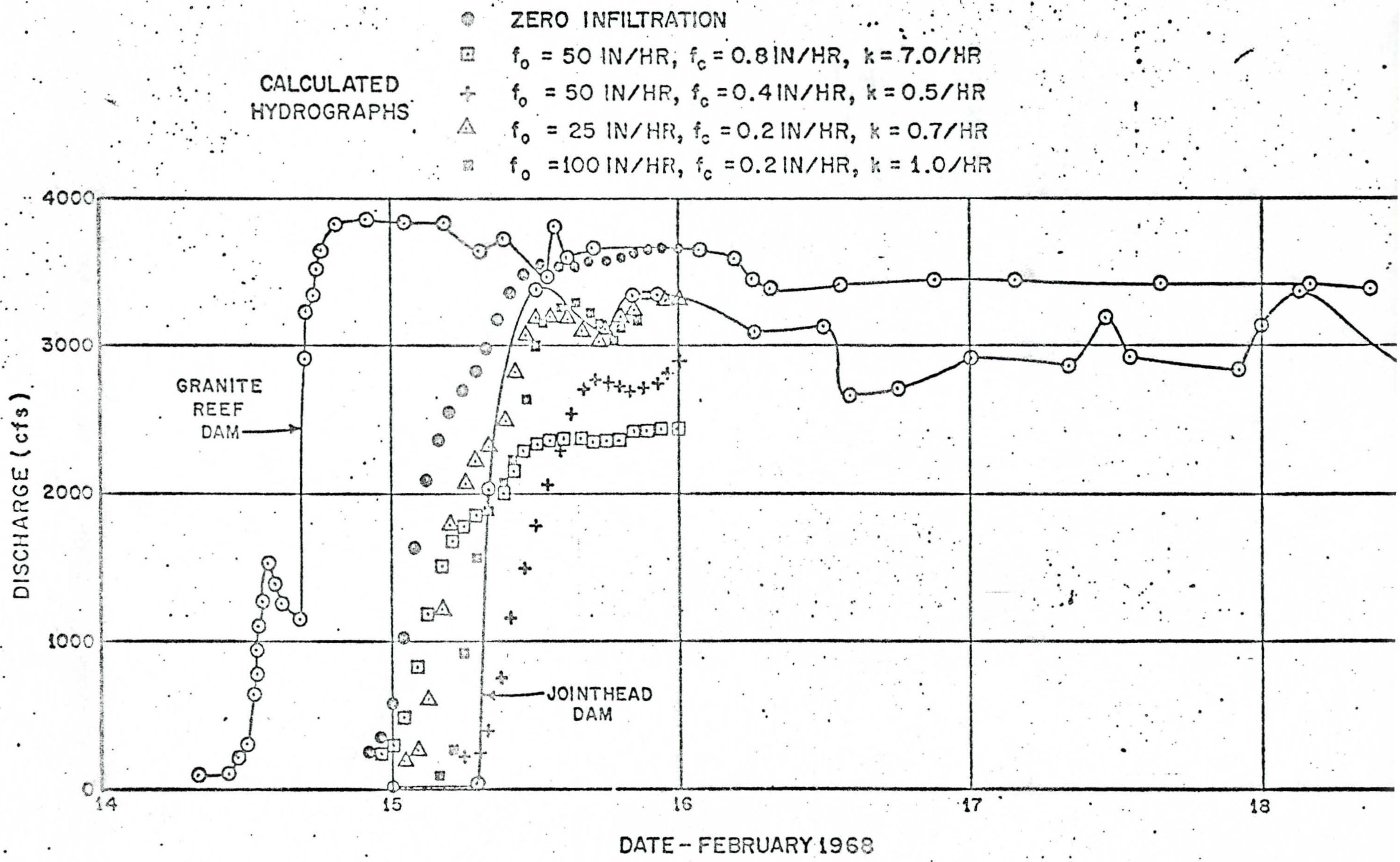

Fig II-3 SALT RIVER, ARIZONA

⊙ - OBSERVED HYDROGRAPHS

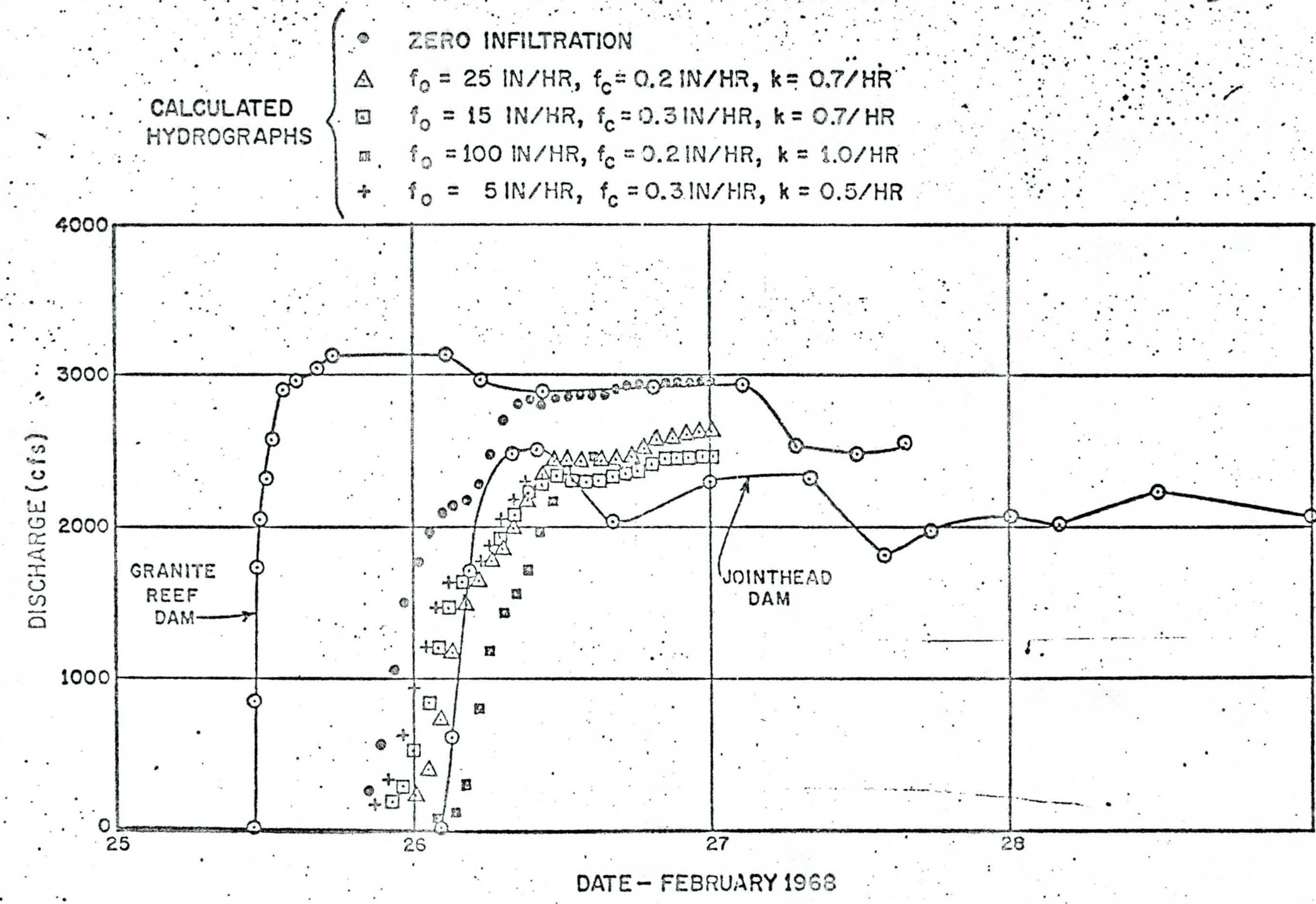

Fig. II-4 SALT RIVER, ARIZONA

⊙ – OBSERVED HYDROGRAPHS

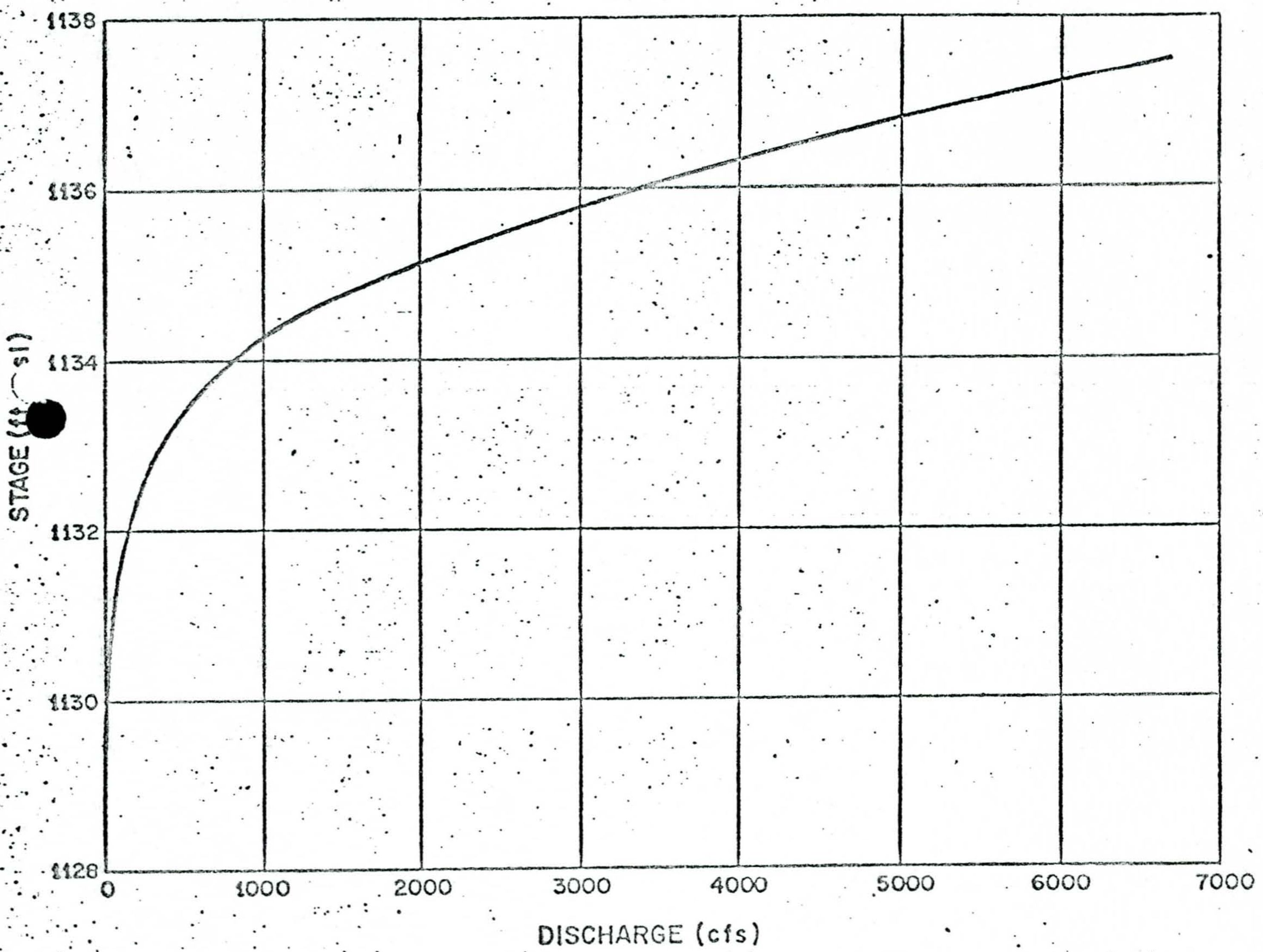

Fig. II-5. , STAGE-DISCHARGE RELATIONSHIP AT DOWNSTREAM BOUNDARY

PART III

Input Data Description

1. General. The input requirements and data formats are identical to those for version 3.0 of the generalized computer program "Gradually Varied Unsteady Flow Profiles" with the additions shown below.

a. Card B2 Optional

Infiltration parameters. Insert this card only if the infiltration option is to be used. It follows B1 cards.

Field	Variable	Value	Description
0		B2	Card identification
2	FC	0,+	Infiltration rate that is approached exponentially as time increases. In inches/hour (converted internally to feet/second).
3	FØ	0,+	Initial infiltration rate, inches/hour (converted internally to feet/second).
4	FK	0,+	Decay coefficient, k, in the infiltration equation, $hour^{-1}$ (converted internally to $second^{-1}$).
5	JCYCLM	2,4,8,12,etc.	Number of computation intervals between recalculations of the infiltration rate. If 2, then the rate is recalculated every time odd nodes are calculated. If not 2, then it should be a multiple of 4.

b. Card B3 Use only when card B2 is present.

Provides user-specified minimum water surface elevations at each node below which the water surface is not allowed to fall. Infiltration is only calculated if the water surface is higher than the minimum. Minimum water surface elevations are calculated on input of geometric data (A3 cards) by Version 3.0 as: (1). The lowest elevation in the elevation table if the area associated with that elevation is greater than zero; if not, then (2). The elevation which is 90% of the distance between the lowest elevation with non-zero area and the next lowest elevation. The minimum elevation at a node used by the program is the larger of that specified on B3 cards or calculated on A3 data input. So, blank B3 cards will cause the program to use the calculated values of minimum elevation (see Fig. III-1).

Field	Variable	Value	Description
0		B3	
2	ELBOT(1)		Minimum elevations for each node starting with upstream-most (left) node. Sequence is the same as B1 cards. Place eight values per card. The number of B3 cards will equal the number of B1 cards.
3	ELBOT(2)		
4	ELBOT(3)		
:	etc.		
9			

Possible tables of geometric elements vs. elevation (A3 cards) for a node.

Case 1			Case 2		
Entry #	Elev	Area	Entry #	Elev	Area
1	100	5	1	100	0
2	101	10	2	101	0
3	102	20	3	102	5
4	103	80	4	103	10
etc	etc	etc	etc	etc	etc
Calculated minimum water surface elevation = 100			Calculated minimum water surface elevation = 101.1		

Note, in each case, a higher elevation specified on the B3 cards for this node will override that calculated on A3 card data input.

FIGURE III-1: Example of Automatic Calculation of a Minimum Water Surface Elevation.

c. Cards B4 **Optional**

The B4 cards provide the user with the option to vary infiltration parameters along the study reach. As infiltration computations are based on odd node conditions only, variable nodal infiltration parameters can be specified only at odd nodes. Hence, if B4 cards are used, there must be one for each odd node in the model. If B4 cards are not present, the values specified on card B2 are used for all nodes.

Field	Variable	Value	Description
0	B4		Card identification
2	FC(1)	0,+	Same as on Card B2
3	FØ(1)	0,+	Same as on Card B2
4	FK(1)	0,+	Same as on Card B2

The first B4 card contains values for node 1, the second for node 3, etc. For an N node model, there will be (N + 1)/2 B4 cards.

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d. Cards C1 - E Same as standard version of program

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e. Cards E3, E4 Required if using infiltration

Code zero tributary inflow for any reach. This merely triggers the lateral inflow computation. If there is tributary inflow, code as usual.