Research Report Submitted to:

ALBUQUERQUE METROPOLITAN ARROYO FLOOD CONTROL AUTHORITY

(AMAFCA)

and

CITY OF ALBUQUERQUE

PHASE II -

ARROYO TRANSMISSION LOSSES

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FORWARD

This report is submitted to the Albuquerque Metropolitan Arroyo Flood Control Authority and the City of Albuquerque and presents the results of research conducted by the Civil Engineering Department at New Mexico State University. The results of the Phase II research are presented in three separate reports:

1. Arroyo Transmission Losses
2. Rainfall Infiltration of Selected Soils in the Albuquerque Drainage Area, and
3. Energy Dissipator/Grade Control Structures for Steep Channels.

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SECTION 1

INTRODUCTION

General

Runoff in Albuquerque arroyos and sand bed channels may be significantly affected by infiltration into the channel bed and banks. Channel infiltration could be of significant quantity and calculating this transmission loss could result in savings in the design and construction of flood channels and detension dams. Some portion of infiltrated water may percolate to the groundwater table, thus serving as a source of recharge. The magnitude of arroyo transmission losses is presently unknown. The research was performed for the City of Albuquerque and the Albuquerque Metropolitan Arroyo Flood Control Authority (AMAFCA). The research was conducted by the Civil Engineering Department of New Mexico State University in Las Cruces.

Objective of Research

The objective of this research was to obtain an estimate of the magnitude of channel transmission losses that may occur in Albuquerque arroyos and sand bed channels. Losses were estimated by double-ring infiltrometer field tests.

SECTION 2

FIELD TESTING

Equipment

The channel infiltration tests were conducted during the same time period as the rainfall infiltration studies. The necessary equipment consists of a double-ring infiltrometer, a readily available source of water, water depth measuring gauge, and a stop watch.

The double-ring infiltrometer is a set of two separate sheet metal cylinders, open at both ends. The cylinders were each 18-in long by 9 and 14-in diameter, respectively. The water was obtained from the 500 gallon tank on the rainfall infiltrometer device. A vernier hook gauge was used to measure the depth of water in the cylinder infiltrometer, as shown in Figure 1.

Field Procedure

The location of each infiltrometer test was selected to be representative of the undisturbed channel, and was carefully investigated for unusual surface disturbances, animal burrows, large stones, and debris. Areas affected by traffic were avoided.

The ring-infiltrometers were installed by first setting the inner (smaller) cylinder. The cylinder was placed so that the edge would avoid rocks and large gravel and was driven about 4-inches into the ground perpendicular to the surface. The outer cylinder was set concentric with the inner cylinder in the same manner.

The outer ring was filled with water to a depth of about 2-inches which was maintained throughout the test. The water in the outer ring is to serve as a constraint to lateral infiltration of water from the inner cylinder.

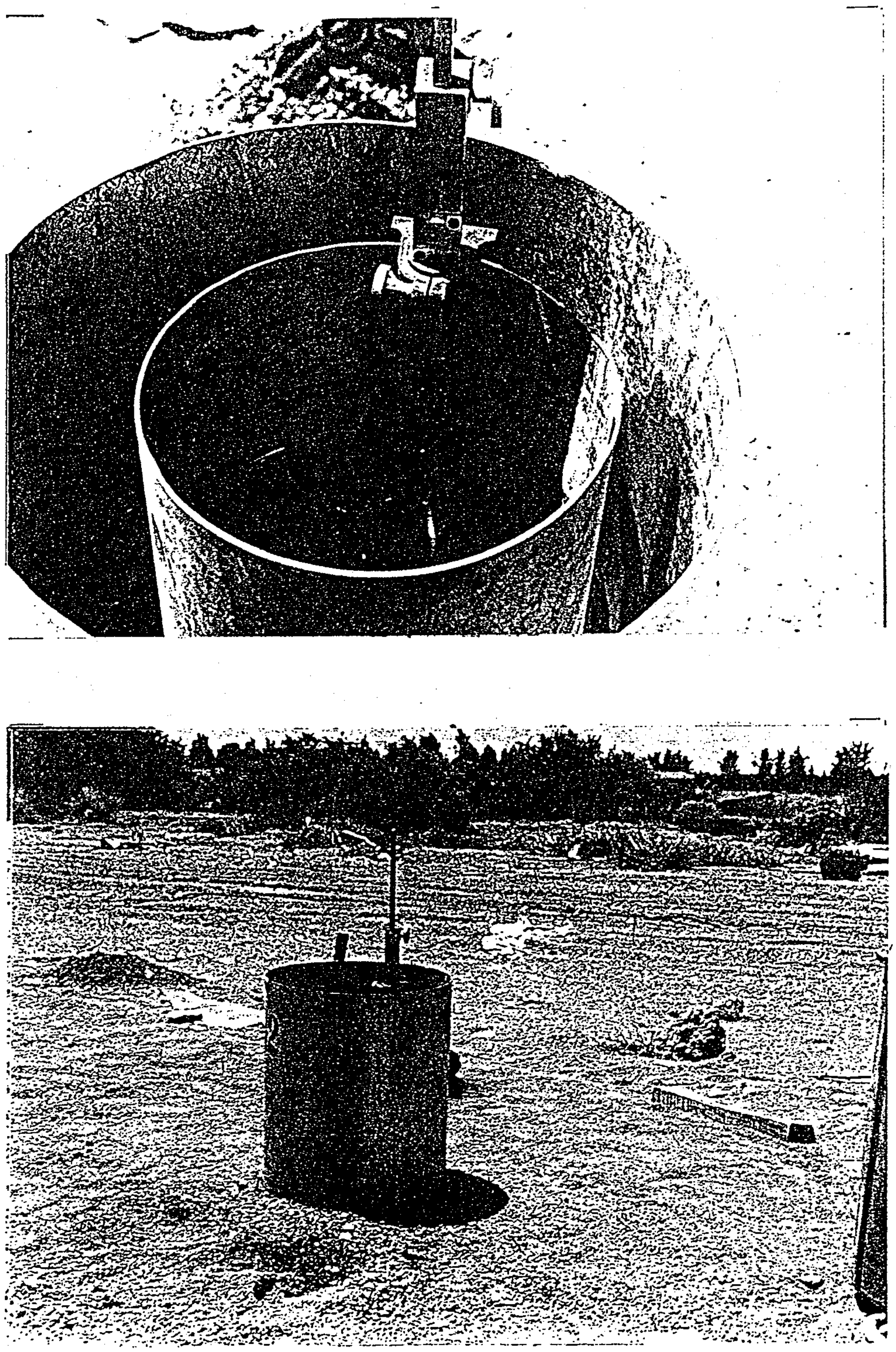

Figure 1.- Photographs of double-ring infiltrometers showing vernier hook gauge for measuring water level in inner cylinder.

A piece of cloth was placed on the ground inside the inner cylinder to avoid excessive disturbance of the soil when water was introduced to the inner cylinder. The inner cylinder was filled with water to a depth of about 12-inches, then the cloth was quickly removed. The initial water surface elevation was recorded with the vernier hook gauge. Periodic measurements were taken of water surface elevation and elapsed time. Water was intermittently added to the inner cylinder to maintain a nearly constant depth. The total time for each test was about one hour.

At the completion of the test the cylinders were removed, and a hole was dug at the location of the cylinders to a depth of about 24-inches. The hole was to insure that no subterranean obstacle to water movement existed, to measure the depth of the wetting front, and to obtain a soil sample.

Location of the Tests

Four infiltrometer tests were conducted in Arroyo del Pino. One test (Site A) was located immediately to the west of the Tanoan Country Club property line, two tests (Sites B and C) were located in the vicinity of Albuquerque Academy, and one test (Site D) was located several hundred yards west of Wyoming Boulevard. Figure 2 indicates the approximate location of the four test sites.

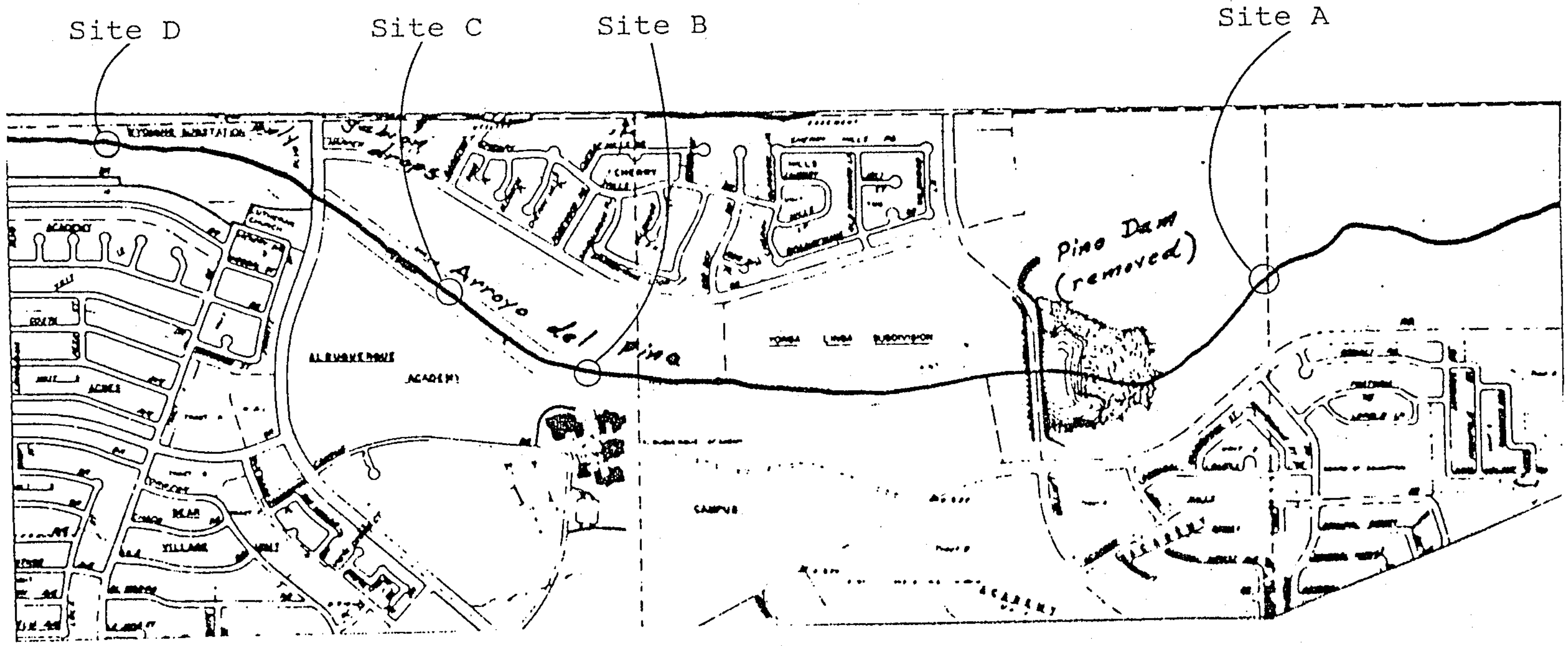

Figure 2.- Location of channel infiltration test sites on Arroyo del Pino.

SECTION 3

DATA

The data were analyzed by considering Darcy's Law:

$$q = kia \qquad (1)$$

where q is flow rate, k is hydraulic conductivity, i is hydraulic gradient, and a is cross-sectional area of flow.

The Darcy equation cannot be solved for hydraulic conductivity using the data because i is not easily determined. The hydraulic gradient is a decreasing function of time due to the increasing length of flow path (advance of the wetted front). The data can be analyzed for the rate of seepage which is defined as:

$$s = \frac{q}{a} = ki \qquad (2)$$

where s is the seepage rate, depth per unit time, at which water infiltrates into the ground.

The seepage rate for the four sites has been calculated and these are presented in Tables 1 through 4. In these tables Δt and Δd are time interval and water surface difference respectively, between two consecutive measurements, and s is the corresponding seepage rate.

The seepage rate (s) was plotted as a function of time (t) for each of the four sites. An exponential decay function of the form:

$$s = Ae^{-Bt} \qquad (3)$$

was fit to each set of data by the method of least squares. The data are plotted in Figures 3 through 6 and the best fit line is shown. The equations of the lines and the coefficients of determination for each are shown in Table 5.

Table 1.- Calculated seepage rates from the double-ring infiltrometer data for Site A.

Measurement times since start of test, in min:sec T_1 (1)	T_2 (2)	Incremental time ($\Delta t = T_2 - T_1$), in hours (3)	Water level decline (Δd), in inches (4)	Seepage rate (s), in inches/hr (5)
0:41	0:57	0.004	1.704	383.4
1:30	2:45	0.012	2.052	164.2
2:43	3:35	0.014	2.136	147.9
4:00	5:07	0.018	2.760	148.3
5:32	6:45	0.020	2.868	141.4
7:13	8:26	0.020	2.904	143.2
8:45	10:11	0.021	2.988	139.7
10:35	11:44	0.019	2.640	137.7
12:13	13:20	0.018	2.952	158.6
13:56	15:38	0.027	3.936	143.1
16:08	17:39	0.025	3.408	134.8
18:22	19:50	0.024	3.588	146.8
20:16	21:43	0.024	3.576	148.0
22:09	23:50	0.028	3.504	124.9
24:15	25:47	0.025	2.964	116.0
26:17	28:06	0.030	3.288	108.6
28:40	30:18	0.027	2.628	96.5
30:44	33:00	0.037	3.624	95.9
33:40	35:52	0.036	3.024	82.4
36:23	39:05	0.045	3.372	74.9
39:43	42:05	0.039	2.760	69.9
42:29	45:26	0.049	3.564	72.4
45:57	48:33	0.043	2.820	65.0
49:00	52:05	0.051	3.336	64.9
52:38	56:06	0.057	3.864	66.8
56:51	60:02	0.053	3.492	65.8

Table 2.- Calculated seepage rates from the double-ring infiltrometer data for Site B.

Measurement times since start of test (t), in min:sec (1)	(2)	Incremental time ($\Delta t=T_2-T_1$), in hours (3)	Water level decline (Δd), in inches (4)	Seepage rate (s), in inches/hr (5)
0:33	3:28	.048	1.680	34.5
3:59	8:25	.073	2.316	31.3
8:25	10:28	.034	0.996	29.1
10:28	13:35	.051	1.200	23.1
14:14	18:00	.062	1.572	25.0
18:00	21:26	.057	1.200	20.9
21:57	27:18	.089	1.932	21.6
27:18	31:23	.068	1.200	17.6
32:00	38:25	.106	2.220	20.7
38:25	42:43	.071	1.200	16.7
43:40	48:38	.082	1.320	15.9
48:38	53:56	.088	1.200	13.5
53:56	59:52	.098	1.200	12.1
59:52	66:09	.104	1.200	11.4
66:09	71:32	.089	1.200	13.3

Table 3.- Calculated seepage rates from the double-ring infiltrometer data for Site C.

Measurement times since start of test (t), in min:sec T_1 (1)	T_2 (2)	Incremental time ($\Delta t=T_2-T_1$), in hours (3)	Water level Decline (Δd), in inches (4)	Seepage rate (S) in inches/hr (5)
0:18	3:21	.050	2.844	55.9
4:05	6:54	.046	2.200	47.0
8:17	12:13	.065	2.832	43.2
13:09	18:05	.082	3.156	38.3
18:52	25:36	.112	3.756	33.4
26:32	30:12	.061	1.908	31.2
32:12	35:28	.054	1.452	26.6
36:19	41:58	.094	2.400	25.4
42:59	48:41	.095	2.604	27.4
49:20	57:15	.131	3.096	23.4
57:15	61:15	.066	1.200	18.0
61:15	68:22	.118	1.200	10.1
68:22	70:21	.033	1.200	36.3
70:21	76:01	.094	1.200	12.7
76:01	82:30	.108	1.200	11.1

Table 4.- Calculated seepage rates from the double-ring infiltrometer data for Site D.

Measurement times since start of test (t), in min:sec T_1 (1)	T_2 (2)	Incremental time ($\Delta t=T_2-T_1$), in (3)	Water level decline Δd, in inches (4)	Seepage rate (s) in inches/hr (5)
0:25	1:20	.015	1.272	83.2
1:20	2:21	.016	1.200	70.8
2:21	3:28	.018	1.200	64.4
3:57	4:50	.014	0.912	61.9
4:50	5:54	.017	1.200	67.5
6:19	6:58	.010	0.780	72.0
6:58	8:12	.020	1.200	58.3
8:42	9:23	.011	0.804	70.6
9:23	10:39	.021	1.200	56.8
10:39	12:03	.023	1.200	51.4
12:35	14:04	.024	1.476	59.7
14:04	15:19	.020	1.200	57.6
15:19	16:42	.023	1.200	52.0
17:12	18:08	.015	0.924	59.4
18:08	19:28	.022	1.200	54.0
19:28	20:52	.023	1.200	51.4
21:23	22:45	.022	1.308	57.4
22:45	24:04	.021	1.200	54.6
24:04	25:46	.028	1.200	42.3
26:20	27:52	.025	1.368	53.5
27:52	29:19	.024	1.200	49.6
29:19	31:01	.028	1.200	42.3
31:33	32:46	.020	1.104	54.4
32:46	34:21	.026	1.200	45.4
34:21	35:59	.027	1.200	44.0
36:28	37:35	.018	0.840	45.1
37:35	39:01	.023	1.200	50.2
39:01	40:46	.029	1.200	41.1
40:46	42:29	.028	1.200	41.9
42:29	44:09	.027	1.200	43.2
44:45	46:45	.033	1.404	42.1
46:45	48:38	.031	1.200	38.2
48:38	50:32	.031	1.200	37.8
51:10	53:40	.041	1.812	43.4
53:40	55:25	.029	1.200	41.1
55:25	57:21	.032	1.200	37.2
57:21	59:31	.036	1.200	33.2
59:31	62:18	.046	1.200	25.8

Table 5.- Equations of seepage rate (s) as a function of time (t) for the four sites.

Site	Equation	Coefficient of Determination (r^2)
A	$s = 175e^{-1.13t}$	.91
B	$s = 32e^{-.93t}$	.91
C	$s = 55e^{-1.22t}$	.93
D	$s = 70e^{-.73t}$	.83

In the line fitting procedure two data points were omitted (outliers) as indicated in the figures. In both cases these outliers were considered to be from data collection errors. The equation of average seepage obtained by taking the geometric mean of the four equations in Table 5 is:

$$s = 68e^{-t} \tag{4}$$

The four seepage rate equations and a line showing the average seepage rate is shown in Figure 7. There is considerable deviation in the four lines initially, however, as time increases the lines converge. Although data are limited the average line should be indicative of the seepage rate for the arroyo in the reach tested.

The soil samples were sieved and the size gradation of each sample is given in Table 6.

Table 6. - Size gradation of the Arroyo del Pino bed material.

Site	% <.074mm	%<2.0mm	%<4.75mm
A	6.2	56.9	82.9
B	3.1	64.4	86.1
C	4.6	64.8	85.3
D	2.6	67.7	90.9

The channel bed material is predominantly fine and coarse sand with some gravel. Very little silt or clay is present. The soil at the four sites were very similar and differences in seepage rate are apparently not related to the overlying bed material size distribution.

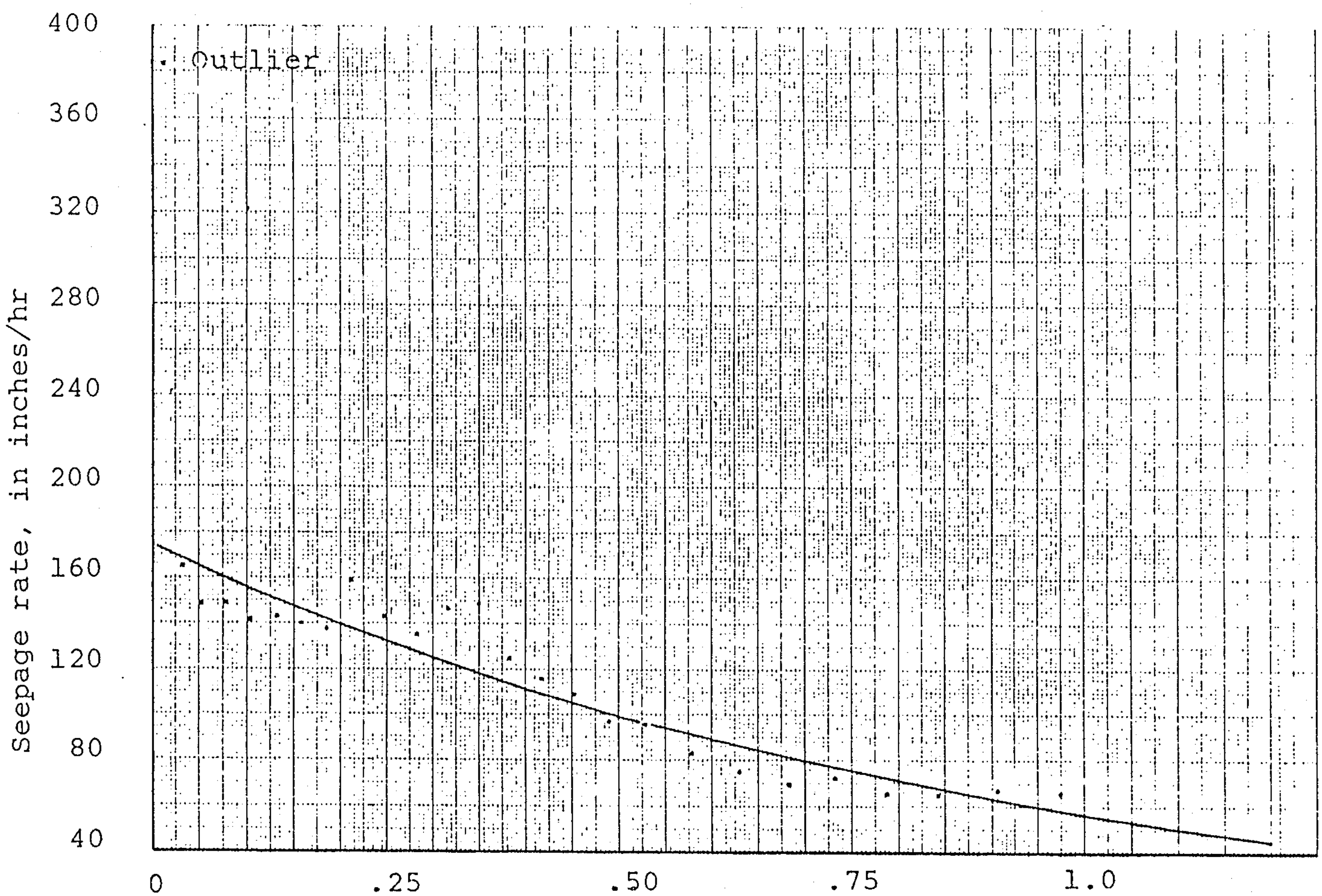

Figure 3.- Calculated seepage rates and line of best fit for Arroyo del Pino Site A.

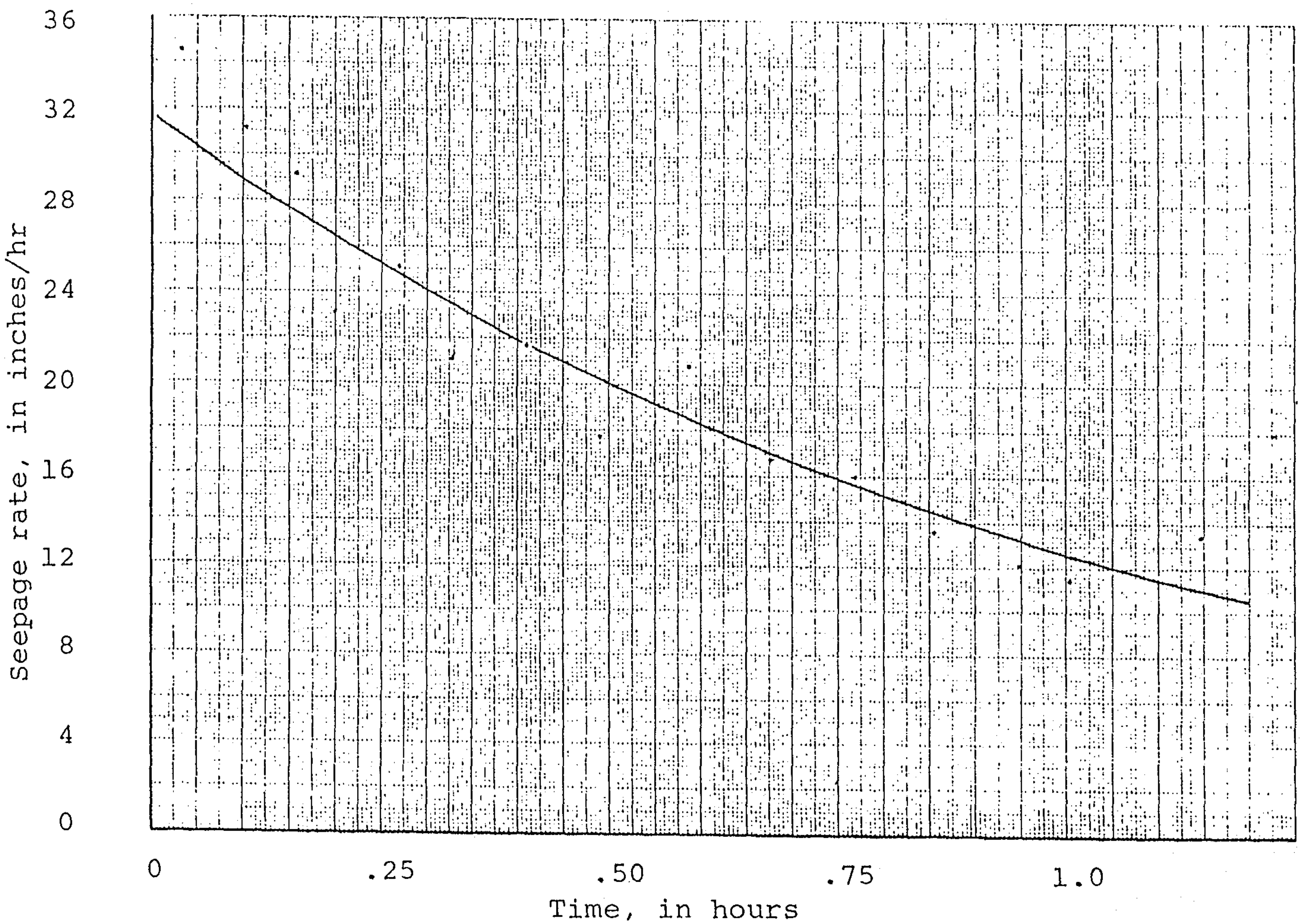

Figure 4.- Calculated seepage rates and line of best fit for Arroyo del Pino Site B.

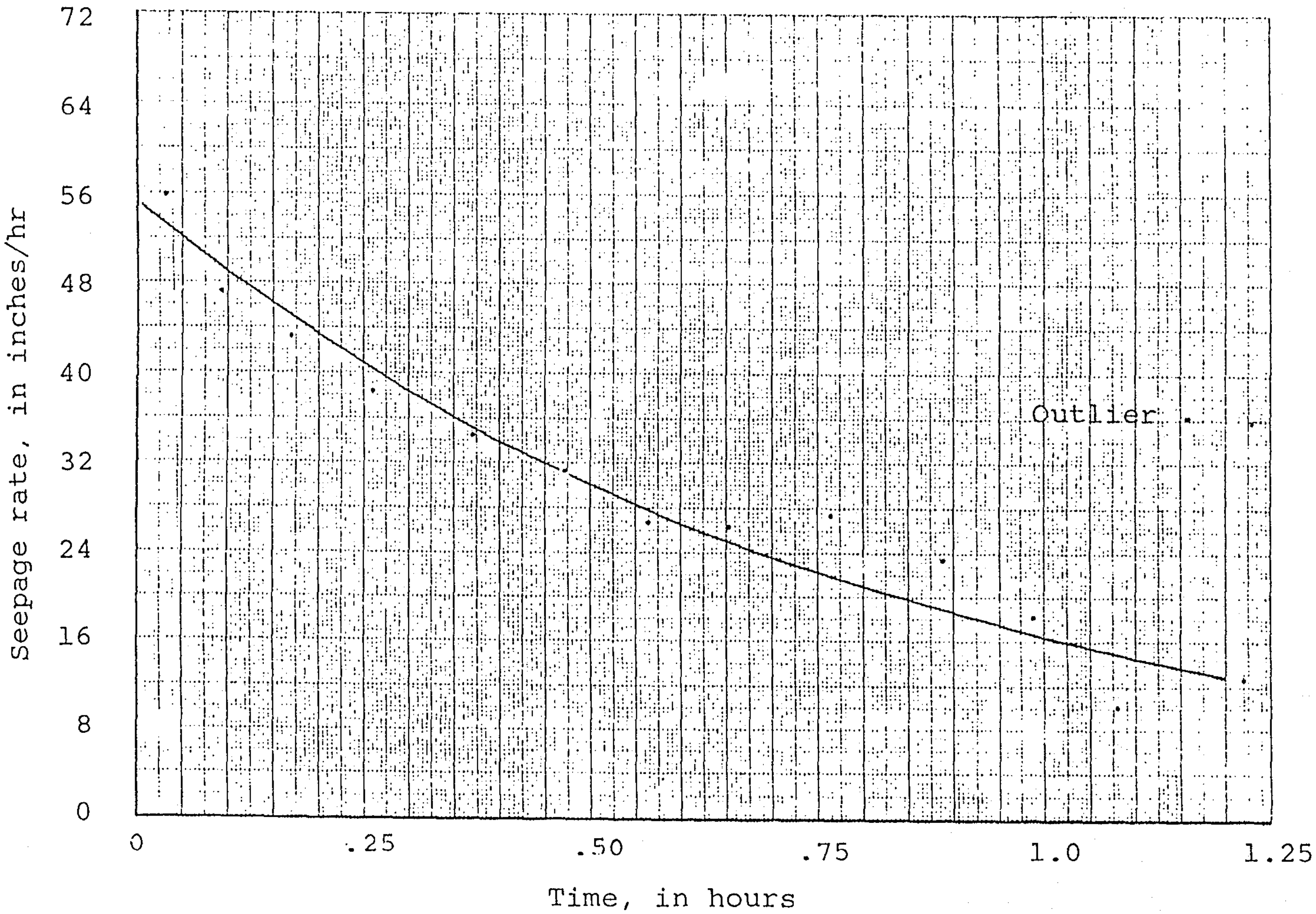

Figure 5.- Calculated seepage rates and line of best fit for Arroyo del Pino Site C.

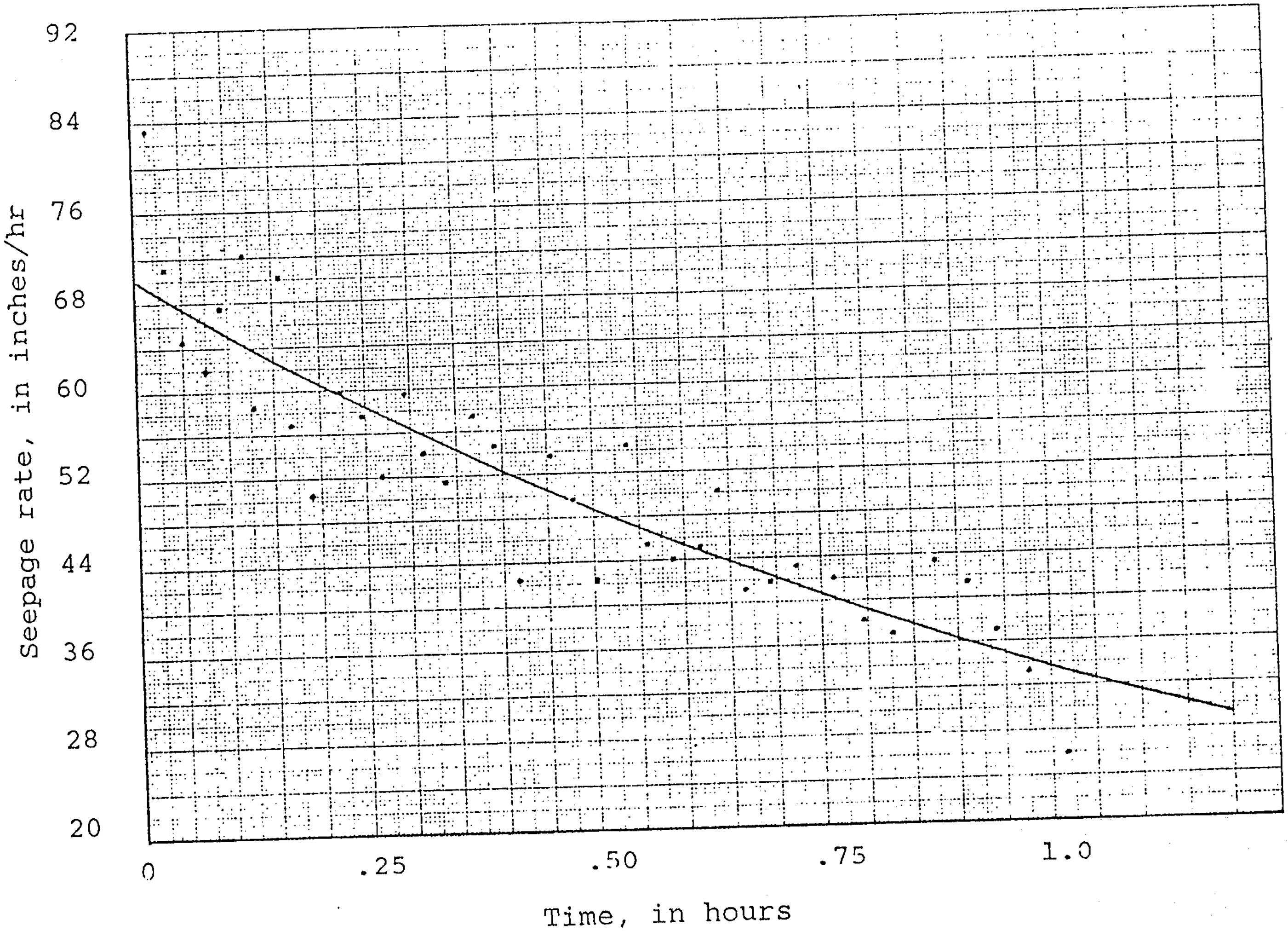

Figure 6.- Calculated seepage rates and line of best fit for Arroyo del Pino Site D.

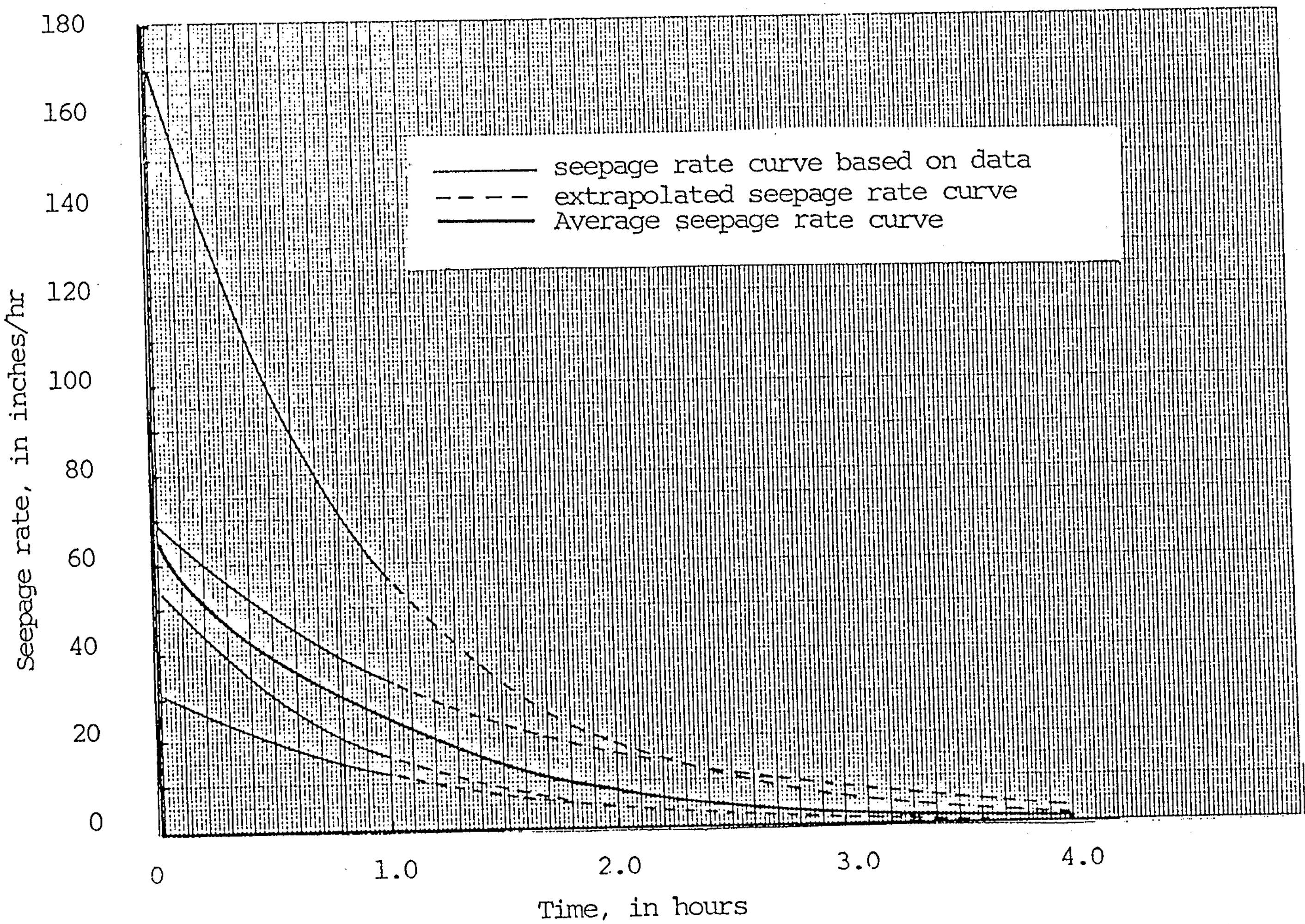

Figure 7.-Seepage rate curves for Arroyo del Pino.

SECTION 4

TRANSMISSION LOSSES

Anyone who has walked the arroyos of the southwest after a heavy rain has seen the evidence of flow that has infiltrated the channel leaving behind the floating debris it was carrying. Sand channel arroyos provide excellent areas for infiltration.

An example of the magnitude of transmission loss in southwestern arroyos is provided by data for the Walnut Gulch Experimental Watershed near Tombstone, Arizona (Renard, 1981). An isohyetal map and runoff hydrographs for the event are shown in Figure 8. The hydrograph from the 57.7-sq mile watershed is presented as Flume I, and the hydrograph from the 3.18-sq mile subwatershed is presented as Flume II. Minor amounts of rainfall fell on the contributing area to the 11.4-mile long dry arroyo between the flumes. The total volume of water lost in the reach was 28.95 a-ft or 2.5 a-ft per mile. The peak discharge was reduced from about 930-cfs to about 140-cfs. The average attenuation of the peak was 69-cfs per mile, a result of both transmission losses and flow resistence. Although Walnut Gulch data cannot be directly used to verify transmission losses of other arroyos, these data do indicate that the magnitude of losses in arroyos can be substantial.

Estimates of channel transmission losses were made for a hypothetical flood event (the 100-yr design flood for Pino Tramway dam) using the infiltration rates as expressed by Equation 4. Several simplifying assumptions were made for these estimates. First the channel length was chosen as three miles, the width a constant 50 feet, the slope three percent and, during the flood, a Manning's value of 0.022 for a plane bed

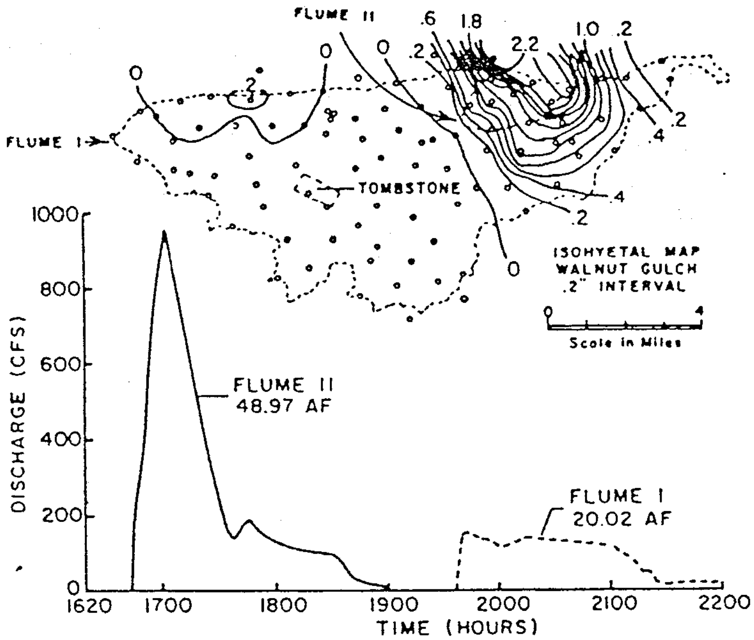

Figure 8.- Storm near upper end of Walnut Gulch on July 30, 1966; each circle shows location of raingage on 57.7-sq mile area.

or antidune condition. The second assumption is that the flood hydrograph could be directly translated through the channel as a kinematic wave; that is, the hydrograph without transmission losses is the same at the entrance and exit of the three mile long channel. The velocity of the hydrograph was determined from Manning's equation. This velocity was used to determine the length of time the hydrograph would be subject to channel infiltration.

Two hydrographs are shown in Figure 9. One is the outflow hydrograph at the exit of the three mile long channel if transmission losses are not considered. This hydrograph is exactly the same as the inflow hydrograph at the entrance to the channel, assuming kinematic routing and no additional inflow to the channel. The second is the outflow hydrograph at the exit of the channel if transmission losses are estimated by Equation 4. Several differences are noticed in the two hydrographs. The initial 0.85 hour is consumed by seepage in passing through the three miles of channel, and the discharge is 1100 cfs at 0.85 hours if seepage is not subtracted from the flow. This implies that for the assumed channel an inflow hydrograph of similar duration and peak discharge of 1000 cfs or less would probably be completely consumed by transmission losses (assuming that flow also spreads to the full 50-ft width of channel). The peak discharge was reduced from 4990 cfs to 4170 cfs. This is a reduction of 5.5% per mile as compared to the 7.5% per mile for the Walnut Gulch example. The volume of runoff was reduced from 458 a-ft to 336 a-ft. This is a loss of 9.0% per mile as compared to the 5.2% per mile for the Walnut Gulch data. The recession limbs of both of the hydrographs look similar and become almost coincident at about four hours. This last observation is expected because seepage rate would have decreased to a relatively low, almost steady state, by

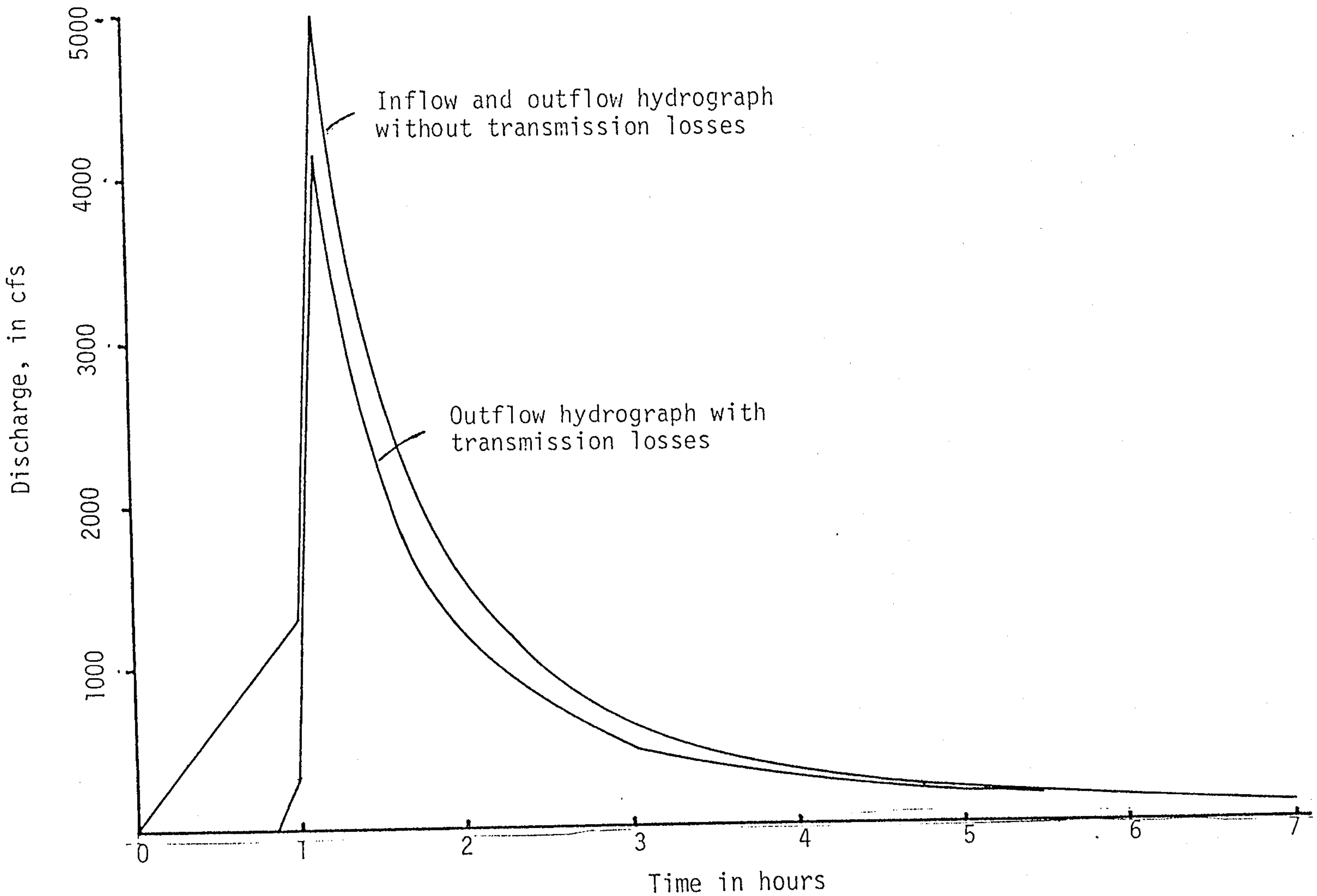

Figure 9. - Example of effects of transmission losses on a flood hydrograph for a simplified three mile reach of Arroyo del Pino near Albuquerque Academy.

that time. These differences are not significant when considering that the simplified example and Walnut Gulch will have different seepage rates, channel characteristics, and flow velocities, and that a more complex routing scheme was not considered in the simplified example. The seepage rate and simplifing assumptions, at least as a rough estimate, are relatively reasonable and do provide an interesting comparison.

Further support of these magnitudes of loss comes from a study by Worstell (1976) on seepage from irrigation channels. He presents results from seepage tests and measurements conducted on numerous canals in various soil types. These results indicate that rates of 0.75 to 1.0 inch per hour are reasonable for sandy soils, and 1.25 to 2.5 inches per hour for gravelly soils. These rates are much lower than those indicated in this study because they represent a condition of saturated flow when infiltration is at a constant rate. For the simplified example above, the average seepage rate for the seven hour period is about 12 inches per hour. After four hours the rate becomes 1.25 inches per hour which is comparable to the rates presented by Worstell. A constant rate of 1.0 inches per hour was used in the simplified example for times beyond 4.22 hours (time at which rate is 1 inch per hour).

The field experiments and previous studies indicate that water loss in arroyos can be an important factor in terms of design. Although limited in scope, the above examples do demonstrate this phenomena.

SECTION 5:

SUMMARY AND CONCLUSIONS

Channel seepage rates have been measured at four sites in Arroyo del Pino. The seepage rates varied with location, especially during the initial periods; however, the seepage rates declined and tended to converge as time increased. The initial seepage rate for a dry channel could be 100 inches per hour or greater. After one hour of flow, the seepage rate decreases to about 37% of the initial seepage rate. The steady state seepage rate of about 1.0 to 2.5 inches per hour occurs later, probably within three to four hours. Transmission losses may result in peak discharge reduction and volume reduction at the rate of 5 to 10% per mile.

The transmission loss in Arroyo del Pino and probably many other Albuquerque arroyos is initially extremely high for dry channel conditions. Many low flows in arroyos would be consumed as channel infiltration. Transmission losses in wide channels during the passage of short duration flood hydrographs, such as caused by high intensity thunderstorms, may significantly reduce the discharge in downstream reaches. During long duration flood flows, such as caused by high intensity general storms, the transmission losses may not significantly reduce the relative runoff volume.

Channel transmission losses are more significant for frequent low-flow events. The lining of upstream channels with impervious material, such as concrete, could result in low-flow maintenance problems in downstream sand bed channels due to the higher frequency of low flows that would have previously been lost through channel seepage.

These conclusions are based on a limited amount of data for one arroyo. A more extensive data collection program could be undertaken with more refined field equipment and a wider range of test arroyos.

Computer models to calculate transmission losses could be used to estimate losses for a range of frequencies of flood flows. This could be used to adjust design hydrographs for transmission losses and to estimate the potential groundwater recharge from unlined channels.

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Worstell, R.V., 1976, Estimating seepage losses from canal systems: Am. Soc. of Civil Engr. Proc., Vol. 102, No. IR1.