Hartzell - Pfeiffenberger and Associates, Inc.

210 St. Paul Street · Denver, Colorado 80206 · Phone 399-0360

March 3, 1969

PRELIMINARY STORM DRAINAGE PLAN
for
STRATTON HOME PROPERTIES

GATES LAND COMPANY

The purpose of this study is to develop a preliminary plan for the control of surface runoff from the subject property and its adjacent areas. Principal considerations are given to protection of life and property in the area, economy of construction, and esthetics of the final project. Details of the final design are yet to be determined, alternative solutions are possible, and no consideration has been given to local runoff in residential areas except where it would materially affect the design of the primary system. It is believed that a final design embodying the basic features of the recommended plan will provide a safe, dependable system at moderate cost, considering the large volume of flow involved.

Six major drainage basins are involved in the area, some of which contain well defined sub-basins, as indicated on the accompanying map. For reference purposes, stream numbers used here will refer to the principal drainage channel in a given area, including on-stream reservoirs. The general features of the plan provide for: (1) use of natural drainage channels where their capacity is adequate; (2) channel realignment and

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improvement where required; and (3) surcharge flood storage in two major permanent lakes where the flood volumes are too large to handle economically in drainage canals.

It is tentatively planned that all flow will be carried in open channels. These have the advantage of lower initial cost than closed conduits and also have reserve capacity in the form of additional depth, area, and velocity should unforeseen conditions arise. In general, grass channels, rather than concrete lined canals, are recommended. These have advantages of conforming to the natural environment, entail a lower initial cost, and result in velocities low enough that stilling basins and other energy dissipating structures are not required. However, they require some maintenance and large width-depth ratios in order to keep the velocities within allowable limits.

Due to the relatively small areas and steep slopes involved, maximum capacity of the system will be required for storms of relatively short duration. Since no actual runoff data is available for this area, flood hydrographs have been developed from U. S. Weather Bureau records of rainfall intensities and frequencies (reference 1) in conjunction with the unit hydrograph method developed by the U. S. Soil Conservation Service (reference 2). The time of concentration of the runoff for the various basins using this method is found to vary from 11 to 33 minutes depending on the length and slope of the basin. Rainfall records indicate that a storm with a fifty-year return period will produce about 2 inches of rainfall in a one-hour period, but with 10-minute intensities ranging from a total of 1.08 inches to 0.08 inches. This has been selected as the design storm with periods of rainfall arranged in a sequence intended to produce the greatest flood peak, as indicated in Figure 1, with the greatest rainfall occurring during the third 10-minute period.

The amount of runoff to be expected from the storm depends on the soil and watershed conditions and may be determined from curves given in reference 2 (Figure A-4), based on empirical data. For average soil, with pasture or range in fair condition, curve 75 is suggested. In anticipation of additional development of the area the writer prefers the more conservative curve 90, corresponding to a value of "S" of 1.11. Consideration of a one-hour rainfall of 2-inch intensity leads to a runoff of 1.10 inches for the period, equivalent to a runoff coefficient "C" of 0.55. This coefficient is the value recommended for "mountain areas" with loam soil and light vegetation in reference 3, with total losses then equal to 0.90 inches over the one-hour period. With the recommended initial losses equal to 0.20 S, this becomes 0.222 inches or practically equal to the rainfall for the

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first 10-minute period. Distributing the remainder of the losses over the period reduces the design storm to two successive periods of 0.20 inches and 0.90 inches, as indicated in Figure 1.

The storm hydrographs from the above are shown graphically in Figures 2 through 6 and the watershed characteristics tabulated in Table 1. Also shown are the peak flows to be expected, which are important only if no storage is planned, and the total flood volume which is significant only for storage reservoir design. The flood flow in cubic feet per second per square mile (peak value) has also been calculated. As a check on the hydrograph analysis, these peak flows per square mile have been compared with those recommended in reference 3 which are based on a study of 48 streams in and along the front range, and the agreement between the two methods is found to be good. It is therefore felt that the peak flows shown are realistic values to be expected and will be adopted as design values for all channel designs in which appreciably higher flows would not pose a serious threat to either life or property in the area. In consideration of the possibility, however remote, of greater floods occurring at some future date which might result in serious consequences (such as the failure of a dam), it is recommended that a flood volume 50% greater than that above be used for design purposes. It may be of interest to note that in reservoir

No. 2 this can be accomplished with an additional dam height of less than one foot and in reservoir 3 of some 3-4 feet. The additional factor of safety provided by this procedure is considered to far outweigh the small additional cost involved.

Specific recommendations for the various areas are as follows:

Area 1. This is drained by Spring Run and has not been considered as part of this study.

Area 2. The drainage from outside the property is to be intercepted near the west boundary and carried along the south boundary in ditches, eventually into reservoir No. 2. East of Highway 115 the slopes are steep and the channel well defined so that only minor channel improvements should be required. This will provide for some of the internal drainage and relieve the demands on the small natural channels in the area. Since the total flood volume is less than the proposed reservoir area, the flow downstream of the dam can be reduced to any desired value with less than one foot of surcharge in the reservoir. An outflow maximum of 50 cfs is suggested in Figure 2 which will drain the pool to its normal level in about 6 hours.

Area 3. The hydrograph for this area indicates a peak flow of 4400 cfs and a flood volume of 188 acre-feet upstream of the reservoir site.

However this flow enters the area in three rather distinct regions and the above values have been distributed among the three on the basis of the respective areas drained, insofar as this can be determined. The channel designated as 3C has its origin in Fisher Canyon and will carry an expected peak flow of some 1575 cfs. Under natural drainage conditions this flow would bypass the dam site and place excessive demands on the channel downstream. It is therefore recommended that this flow be intercepted near the entrance to the area and carried to a point where it can be discharged directly into the storage reservoir. Since a major portion of this route passes through the proposed golf course, and considering that flows of this magnitude occur at very rare intervals, it should be practical to utilize a portion of the golf fairways as a flood plain for the short runoff period involved. Some minor revisions in the master plan may be desirable to increase the safety of certain areas. An alternative to this channel diversion is the construction of a detention reservoir as shown, allowing the outflow to continue in its present course. Some channel improvements downstream of the reservoir will still be required.

Channel 3A is potentially the largest entering the area with only a small part lying within the property boundaries. The main channels are for the most part capable of carrying high flows, but some overflow may be expected at present in the reach outside the property and directly west of the reservoir site. Considerable channel improvement with the possibility of levees may be necessary in this area in order to protect the buildings lying in the flood plain below. Specific recommendations for this region cannot be made without further study of the nature of the present channel and consideration of various alternatives, such as a detention reservoir where the flow enters the property and where there appears to be a good reservoir site. At the downstream end of the main channel the flood flow should be carried in the golf fairways to the reservoir, with a pilot channel similar to that of channels 3D and 4 (shown in cross section on the accompanying map).

Channel 3B has a relatively small drainage area which can be handled for the most part in the present channels. It appears that the proposed resort hotel site will encroach on the natural channel and a minor relocation is required to avoid the possibility of future difficulties.

Assuming that the above plan is followed, the total flow volume into the reservoir could be about 188 acre-feet with a peak of 4400 cfs. This should be reduced to about 500 cfs downstream of the dam to avoid exceeding the capacity of channel 3D. An outflow curve approximating that shown on Figure 3 will require a reservoir storage capacity of about 160 acre-feet and require a reservoir area and depth somewhat larger than that indicated as a maximum of 18.5 acres in total area. The required increase in depth is on the order of 3-4 ft. and flooding of the surrounding fairways no more than a width of 100 ft. along the shore of the lake. Since this will occur on only very rare occasions and there is no property in the area, this is not considered to be objectionable. This reservoir will return to its normal level in about 8 hours with a peak outflow of 500 cfs.

Channel 3D carries a portion of the flow from an area directly west of Highway 115, all of the flow from the adjacent areas, and the reservoir outflow. It will be seen from the hydrograph in Figure 3 that the peak flow of 500 cfs from the reservoir is delayed by the reservoir in such a way that the outflow from area 3D will have been largely exhausted before the reservoir outflow peak is reached. Thus it is unnecessary to design the channel for both flows. It is recommended that this channel be grass lined and improved to produce a cross section as shown in order to handle a flood flow estimated to be about 1200 cfs from direct runoff. Channels of this type are generally designed with a center pilot channel capable of passing a flow equal to the 2 or 3-year expected maximum, with a wider flood plain channel to provide for the flows which occur less often. Depths in these channels, especially on steep slopes, must be kept small in order to prevent erosion of the ground. The calculated depth is 1.60 feet. The additional height of the banks specified will provide for the "super flood" mentioned above without major damage but with the possibility of some erosion to the channel bottom.

Area 4. The drainage area west of this region is relatively small, with the flood volume running accordingly. Since most of the runoff comes from the area itself there is no economic justification for a detention reservoir. Near the west boundary the channel is capable of handling the flood flow but extensive improvements and realignment on the flood plain are required as shown on the map. The maximum flow expected at the downstream end is about 500 cfs. This can be carried along the south boundary on the east side of Highway 115. The channel cross section is similar to that of channel 3D but somewhat narrower due to the smaller quantity involved. Should the width required prove to be objectionable, it could be replaced with a narrower and deeper channel which could either be concrete lined or provided with check dams at intervals to reduce the velocity and prevent erosion. The lined canal would probably require some sort of stilling pool at the outfall to prevent damage downstream.

Area 5-6. Although these channels carry different amounts of flow, their characteristics are very similar. Both appear to be rather deep, V-shaped channels with steep slopes in the direction of flow. It appears that these channels in their present unspoiled condition are capable of passing without overflow a flood far greater than any that can reasonably be expected to occur. Since the proposed housing density in this area is quite small, and since all can be situated well above any possible flood threat, it is recommended that the existing channels be left completely undisturbed. The most serious threat here would be possible erosion during flood flows. Should this occur remedial measures could be taken to correct the situation. The reservoirs shown on the map are possible alternatives should they prove to be necessary.

Respectfully submitted,

Warren DeLapp, PE

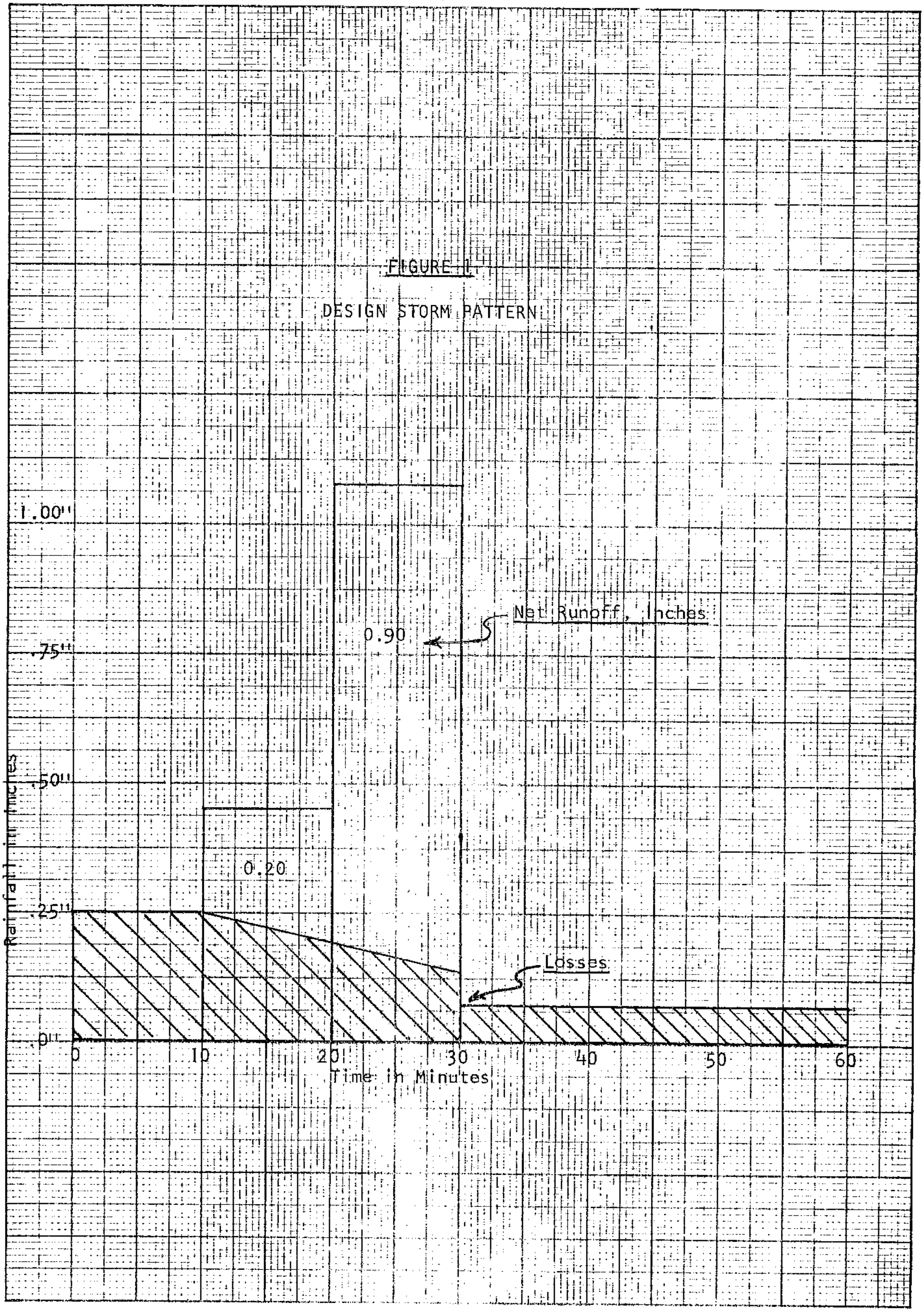


FIGURE 1
DESIGN STORM PATTERN
1.00"
.75"
.50"
.25"
0"
Rainfall in Inches
0.90
0.20
Net Runoff, Inches
Losses
0
10
20
30
40
50
60
Time in Minutes

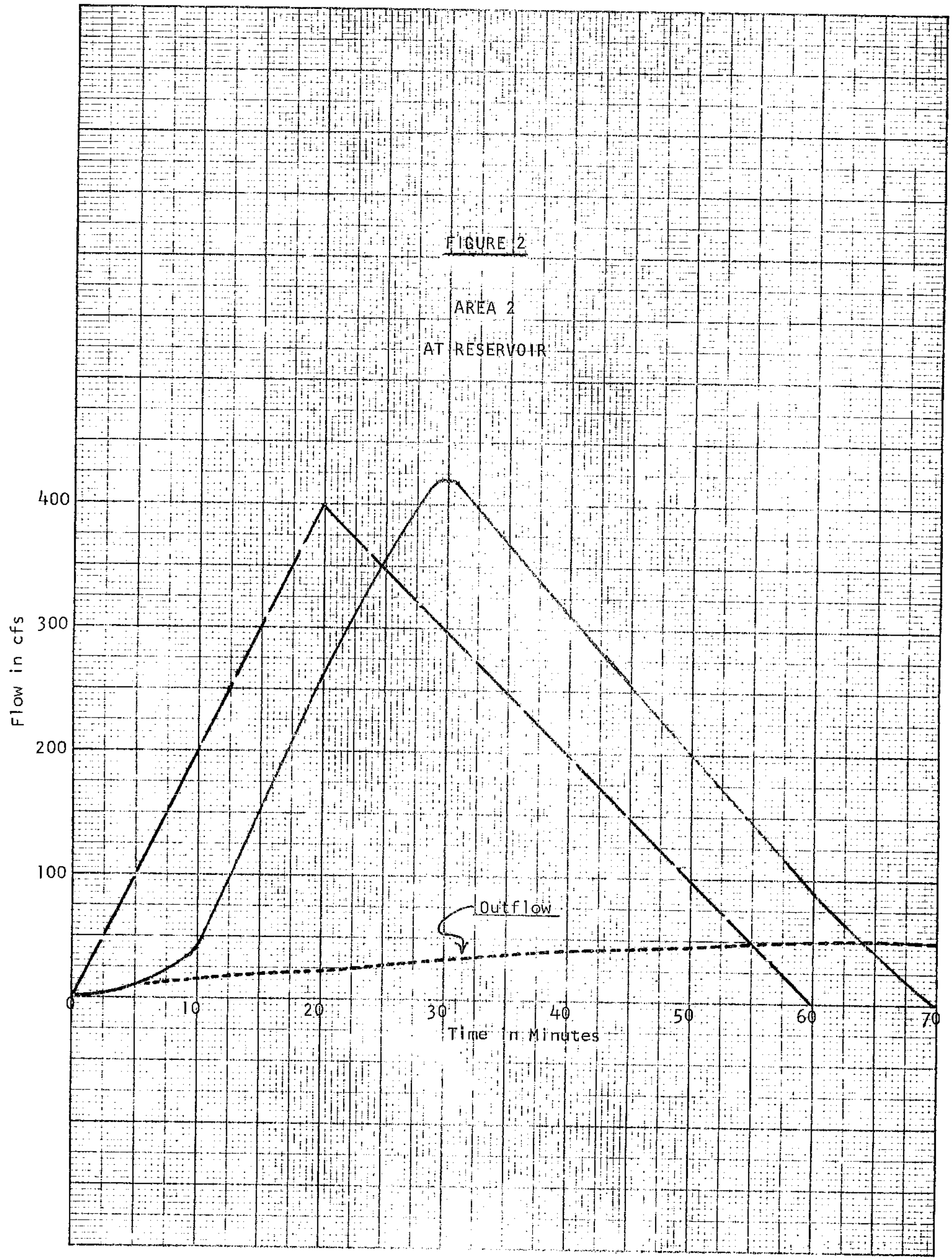


FIGURE 2
AREA 2
AT RESERVOIR
Flow in cfs
400
300
200
100
Outflow
0
10
20
30
40
50
60
70
Time in Minutes

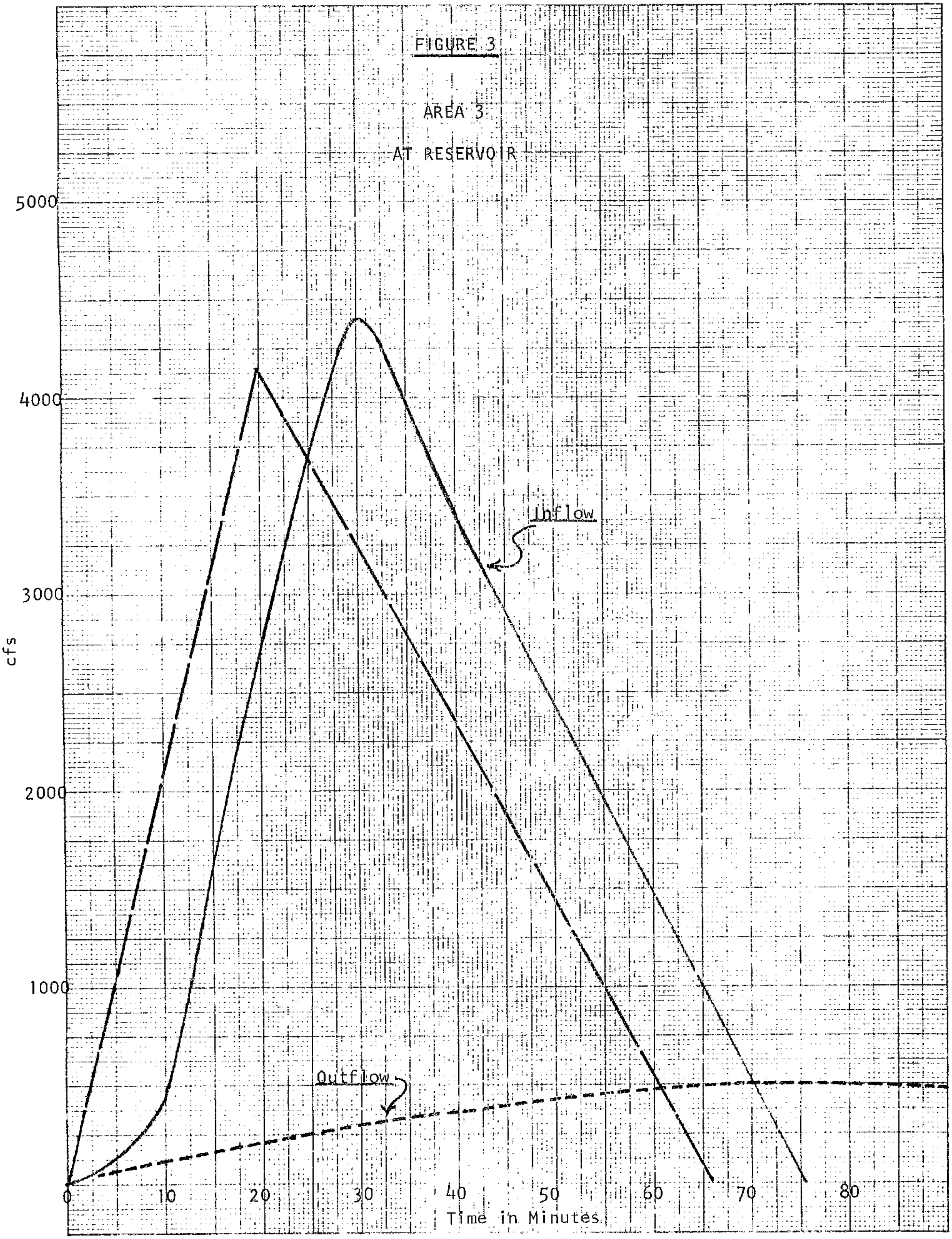


FIGURE 3
AREA 3
AT RESERVOIR
5000
4000
3000
2000
1000
cfs
Inflow
Outflow
0
10
20
30
40
50
60
70
80
Time in Minutes

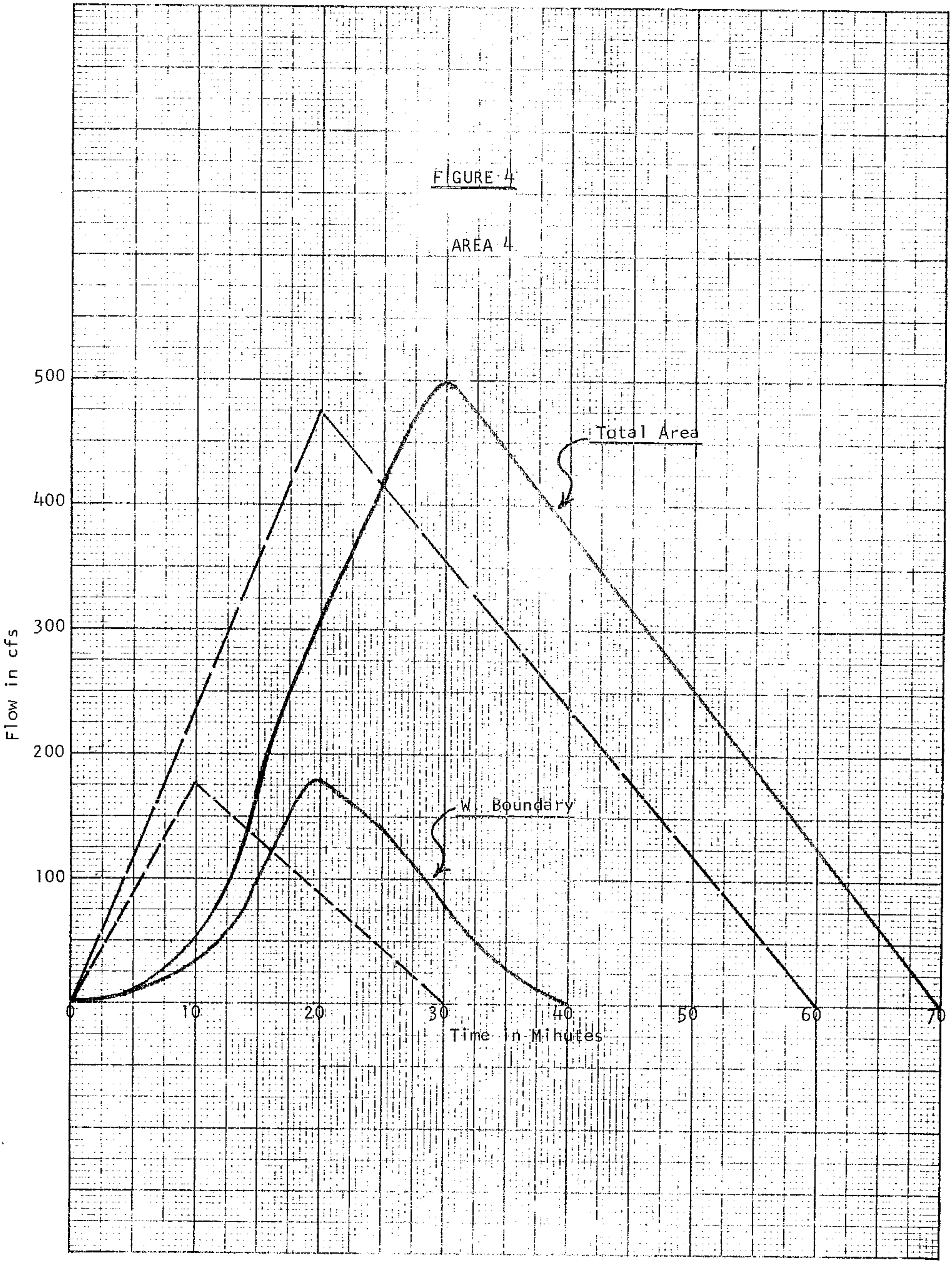


FIGURE 4
AREA 4
Total Area
W. Boundary
Flow in cfs
500
400
300
200
100
0
10
20
30
40
50
60
70
Time in Minutes

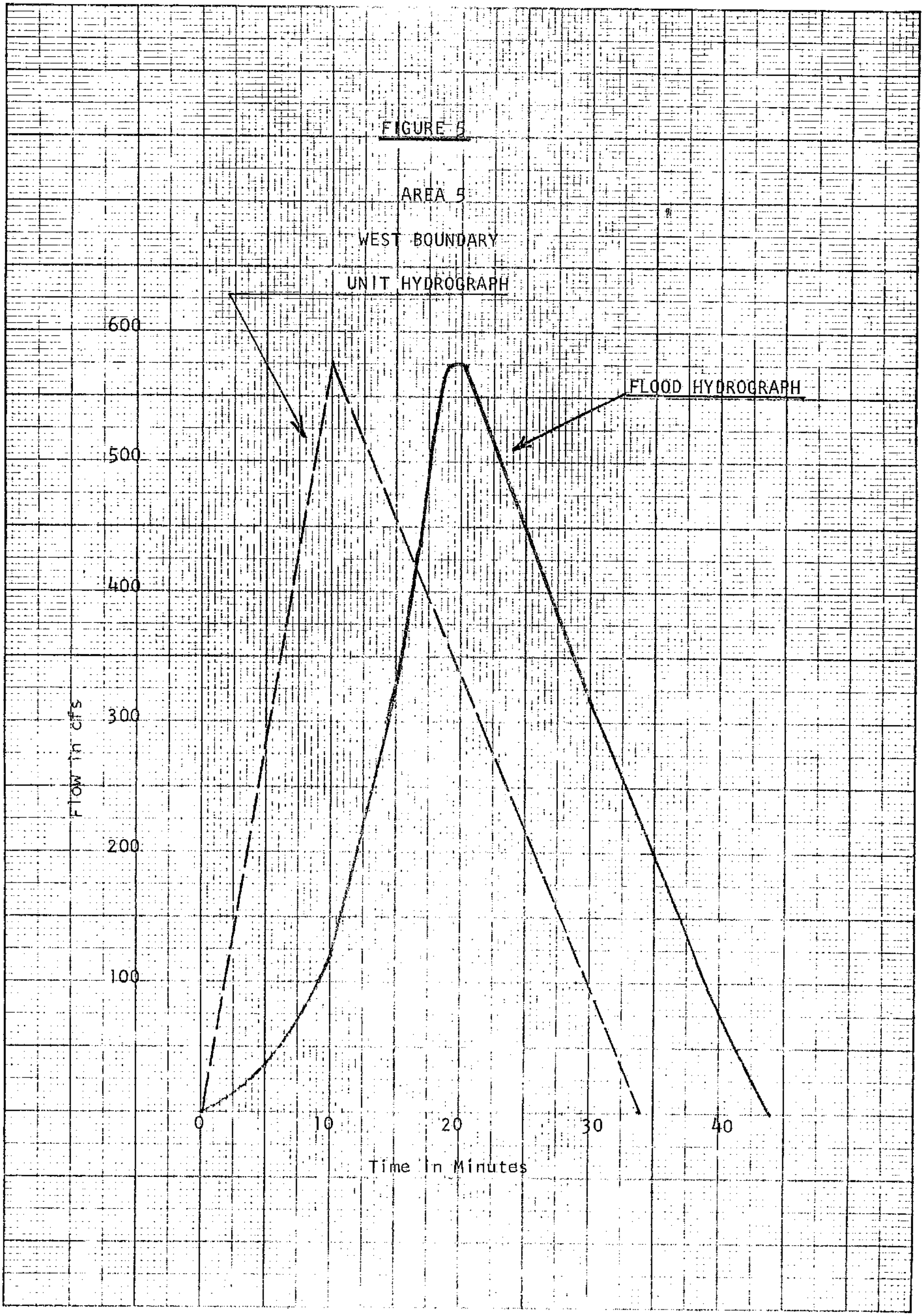


FIGURE 5
AREA 5
WEST BOUNDARY
UNIT HYDROGRAPH
FLOOD HYDROGRAPH
600
500
400
300
200
100
Flow in cfs
0
10
20
30
40
Time in Minutes

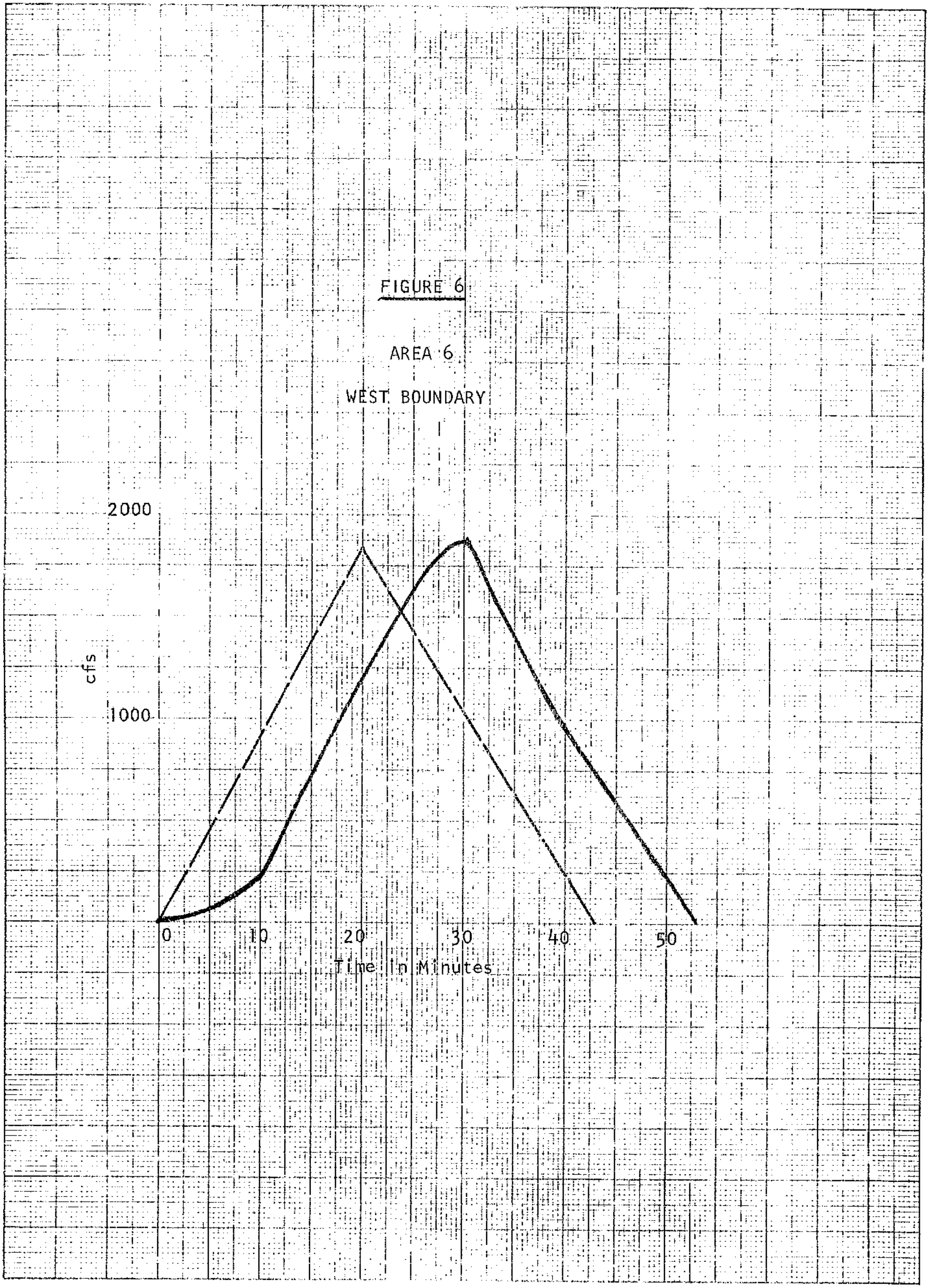


FIGURE 6
AREA 6
WEST BOUNDARY
2000
1000
cfs
0
10
20
30
40
50
Time in Minutes

TABLE I

STREAM HYDROGRAPH SUMMARY

Drainage Area	Location on Stream	Area Acres	Unit Hydrograph T_c	T_p	T_b	Q cfs	Storm Hydr Max Q	Vol Ac-ft	$\frac{cfs}{mi^2}$	2-Yr Max Q
2	Reservoir	200	0.50	0.38	1.01	400	420	18.3	1350	48
3*	Reservoir	2260	0.55	0.41	1.10	4160	4400	188	1240	630
3A	Reservoir	1240					2420	103	1240	345
3B	Reservoir	210					410	18	1240	58
3C	Reservoir	810					1575	67	1240	226
3D	E. Boundary	570	0.50	0.38	1.01	1145	1200	52	1340	138
4	W. Boundary	45	0.18	0.19	0.51	178	178	4.1	2540	21
4	E. Boundary	240	0.50	0.38	1.01	475	499	22.0	1330	57
5	W. Boundary	160	0.22	0.21	0.56	575	583	14.7	2330	69
6	W. Boundary	660	0.32	0.27	0.72	1840	1890	60.5	1835	220

3* Total for 3A, B, C

T_c = "Time of concentration" for drainage Basin

T_p = Time to unit hydrograph peak

T_b = Total length of unit hydrograph

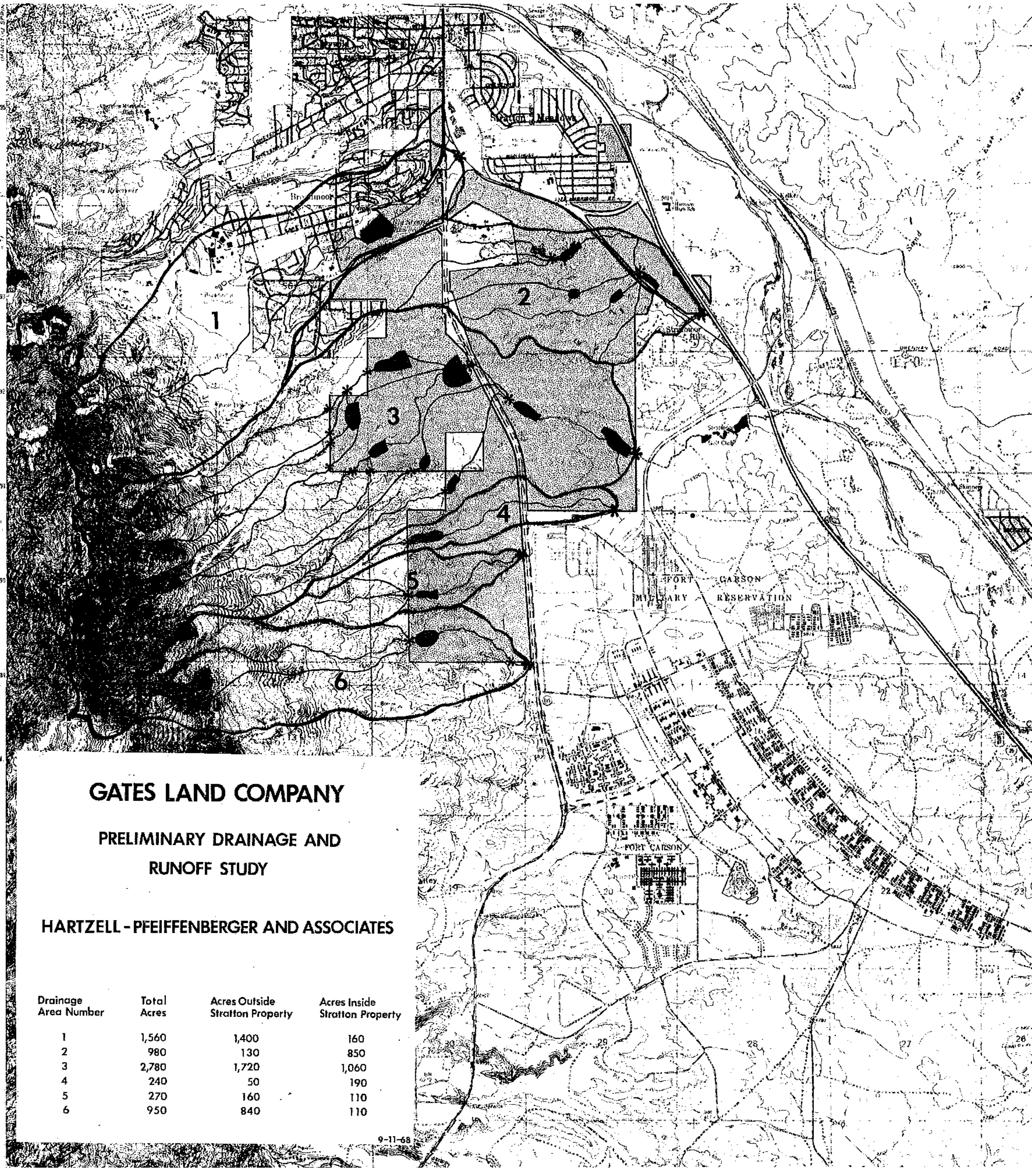
REFERENCES

1. U. S. Weather Bureau Technical Paper No. 25, "Rainfall Intensity-Duration-Frequency Curves".

2. U. S. Bureau of Reclamation, "Low Dams".

3. Ken R. White Consulting Engineers, "Storm Drainage" Report for the Inter-County Regional Planning Commission, Denver, Colorado, 1962.

4. Bureau of Public Roads, "Design of Roadside Drainage Channels". Hydraulic Engineering Circular No. 6.



GATES LAND COMPANY

PRELIMINARY DRAINAGE AND RUNOFF STUDY

HARTZELL - PFEIFFENBERGER AND ASSOCIATES

Drainage Area Number	Total Acres	Acres Outside Stratton Property	Acres Inside Stratton Property
1	1,560	1,400	160
2	980	130	850
3	2,780	1,720	1,060
4	240	50	190
5	270	160	110
6	950	840	110

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