STORM WATER MANAGEMENT PLAN

SOUTH VENICE GARDENS AREA

FINAL ENGINEERING REPORT FOR SARASOTA COUNTY, FLORIDA

PROJECT NO. 9250-1-RT

JUNE, 1983

CAMP DRESSER & McKEE INC.
6221 14th STREET WEST, SUITE 302
BRADENTON, FLORIDA

environmental engineers, scientists, planners & management consultants **CDM**

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SOUTH VENICE GARDENS AREA

FINAL ENGINEERING REPORT

FOR

SARASOTA COUNTY, FLORIDA

By

Bernard L. Golding, P. E.
Area Water Resources Manager

June 1983

CAMP DRESSER & McKEE INC
Environmental Engineers & Scientists
6221 14th Street West, Suite 302
Bradenton, Florida

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EXECUTIVE SUMMARY

The Engineering Report presents a plan for improving the existing poor drainage in the 180 acre area just south of Briarwood Road and east of State Route 50 in the southern portion of Sarasota County referred to therein as the South Venice Gardens area. The area currently drains through an existing poorly maintained outlet canal to a branch of Alligator Creek.

As part of the work involved in the preparation of this Report, two principal alternative storm water management plans for improving drainage in the general area were considered. The first alternative involved improving the existing outlet canal which runs easterly from State Route 41 to a branch of Alligator Creek. The second alternative involved the construction of a new storm sewer system running westerly to the Intracoastal Waterway via Shamrock Drive.

The second alternative was predicated on the assumption that the proposed drainage system and detention basins, being constructed as part of the proposed Venetian Plaza Shopping Center, would still allow a substantial flow to enter the upper (western) end of the outlet canal during a major rainfall. Thus, the construction of a new storm sewer system to the Intracoastal Waterway, to eliminate or lessen the impact of runoff from the shopping center on the outlet canal, would have been required. However, upon analysis, the proposed detention basin and outlet storm sewer discharging to State Route 41 was found to be adequate such that actually no runoff to the outlet canal from the shopping center area would result from a 25 Year Frequency - 24 Hour Duration Rainfall. Thus, construction of the Venetian Plaza Shopping Center storm drainage system is actually an improvement over the prior situation (predevelopment conditions) where some runoff from the shopping center area did actually enter the outlet canal.

Therefore, based on the adequacy of the proposed shopping center storm water management facilities and the very high cost of construction of a new storm sewer system, which cost was estimated to be in excess of $500,000, the first alternative storm water management plan, improving the existing outlet canal, was adopted. This alternative was studied in detail and is recommended in the Engineering Report. The estimated construction cost of the recommended storm water management plan is $390,500.

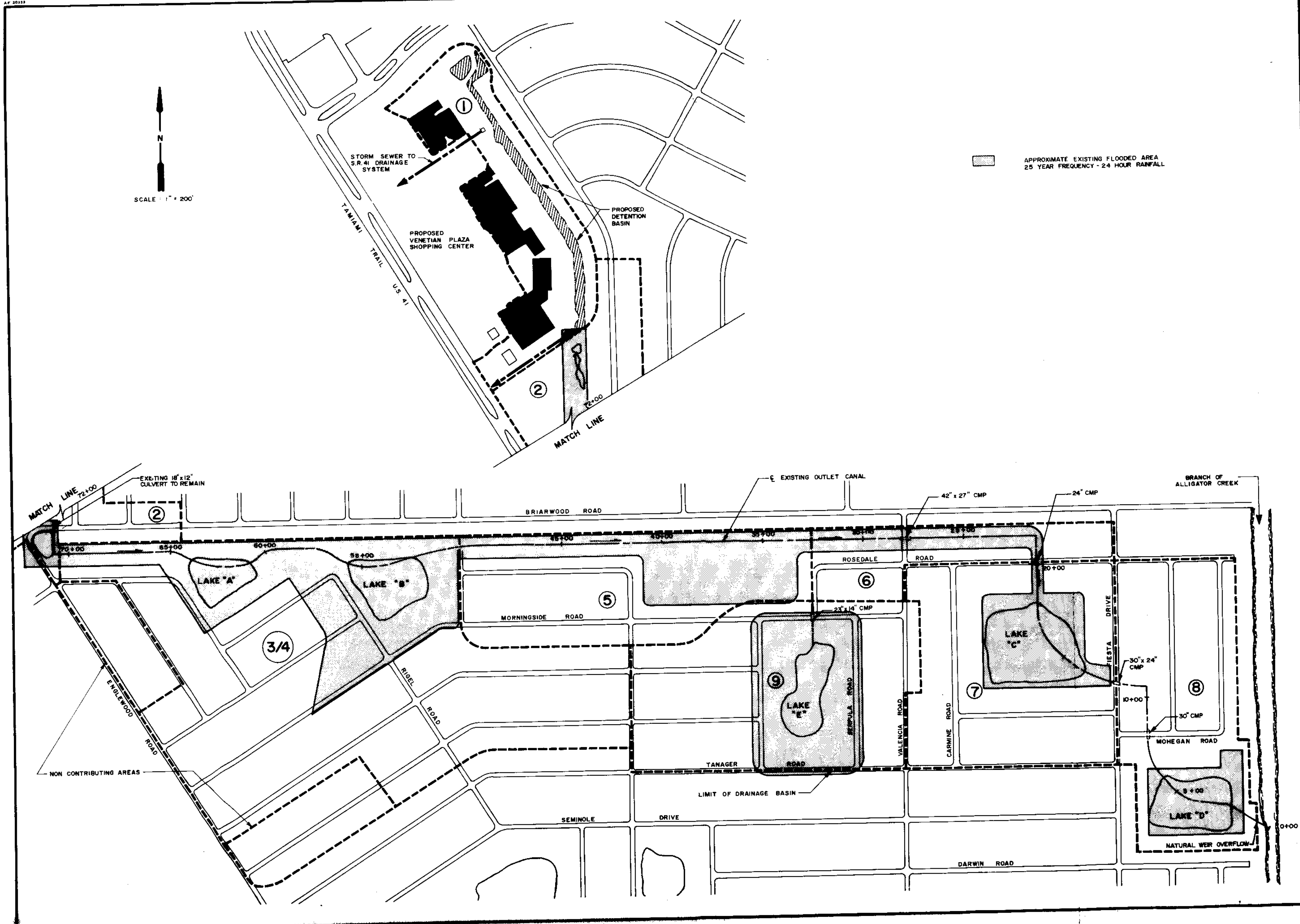
The recommended storm water plan presented in the Report, in addition to enlarging and deepening of the existing outlet canal will also require the lowering of the existing water surface level of four existing lakes (designated Lakes A, B, C and D) through which storm drainage passes, and an additional lake (Lake E) in the area which outlets by a ditch to the outlet canal. The lowering of these lakes will provide the additional reservoir storage for peak flow attenuation over that which is now available in the lakes.

The proposed recommended improvements will not have any adverse impact on the environment. It is not anticipated that the lowering of the lake levels of approximately 1.5 to 2 feet will adversely affect the shoreline vegetation. Also, in 90 percent of the rainfalls which occur, the runoff will be conveyed in approximately the same manner and at approximately the same rate as before. In major rainfall events, the total runoff will necessarily be conveyed at a higher rate to Alligator Creek than previously, but with the area being almost entirely developed and extremely flat, only a minimum amount of erosion will occur.

The location of the outlet canal, designated in the Report as the South Venice Gardens Outlet Canal, and the five lakes involved are shown on Drawings No. 1 and No. 2. The approximate area subject to flooding from the 25 Year Frequency - 24 Hour Duration Rainfall has been delineated on Drawing No. 1. Drawing No. 2 shows a plan of the proposed improvements to the existing Outlet Canal. Also presented is Drawing No. 3 which consists of a profile along the improved Outlet Canal showing the existing and proposed canal bottom and water surface elevations therein during the 25 Year Frequency Rainfall and the average existing and proposed water surface elevations in the various existing lakes. The average existing and proposed water surface elevations in the various existing lakes are tabulated below:

Lake Designation	Average Existing Water Surface Elevation	Proposed Water Surface Elevation	
		Low Water	25 Year Frequency
A(1)	13.0±	11.5	14.8
B(1)	13.0±	11.5	14.8
C	10.0±	8.5	12.5
D	9.0±	7.0	11.1
E	12.5±	11.5	13.9

(1) Under the proposed storm water management plan, Lakes A and B will be connected and act as a single lake.



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CDM

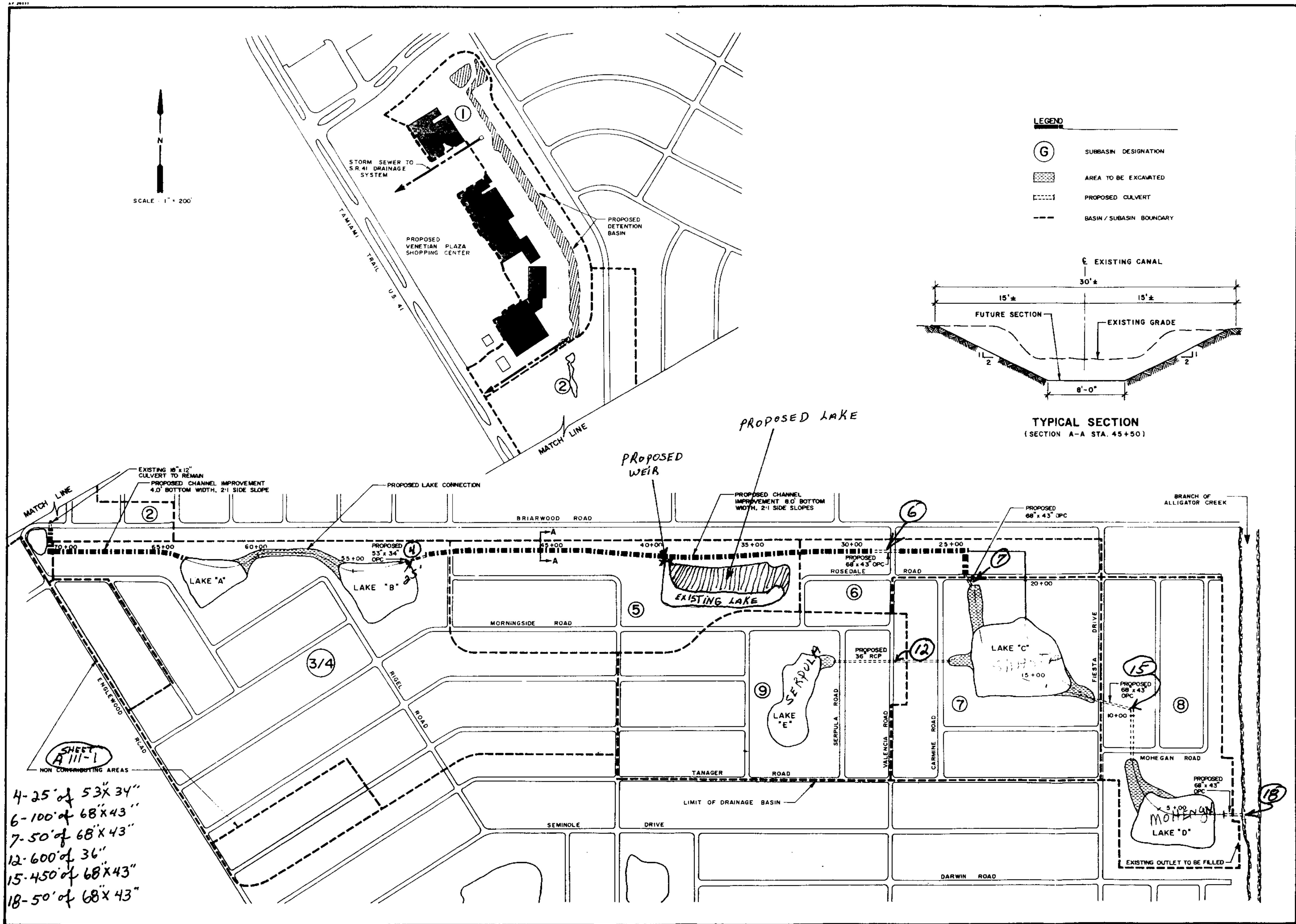
DESIGNED BY SLG
DRAWN BY
CHECKED BY SLG
APPROVED BY SLG
DATE 6/18/83

REV. NO	DATE	DRWN	CHKD	REMARKS

GENERAL PLAN
EXISTING FLOODING CONDITIONS
SOUTH VENICE GARDENS AREA

PROJECT NO. 9250-I-RT

DRAWING NO. 1



4-25' of 53" x 34"
6-100' of 68" x 43"
7-50' of 68" x 43"
12-600' of 36"
15-450' of 68" x 43"
18-50' of 68" x 43"

50,962.50 PIPE

CAMP DRESSER & McKEE INC.

CDM

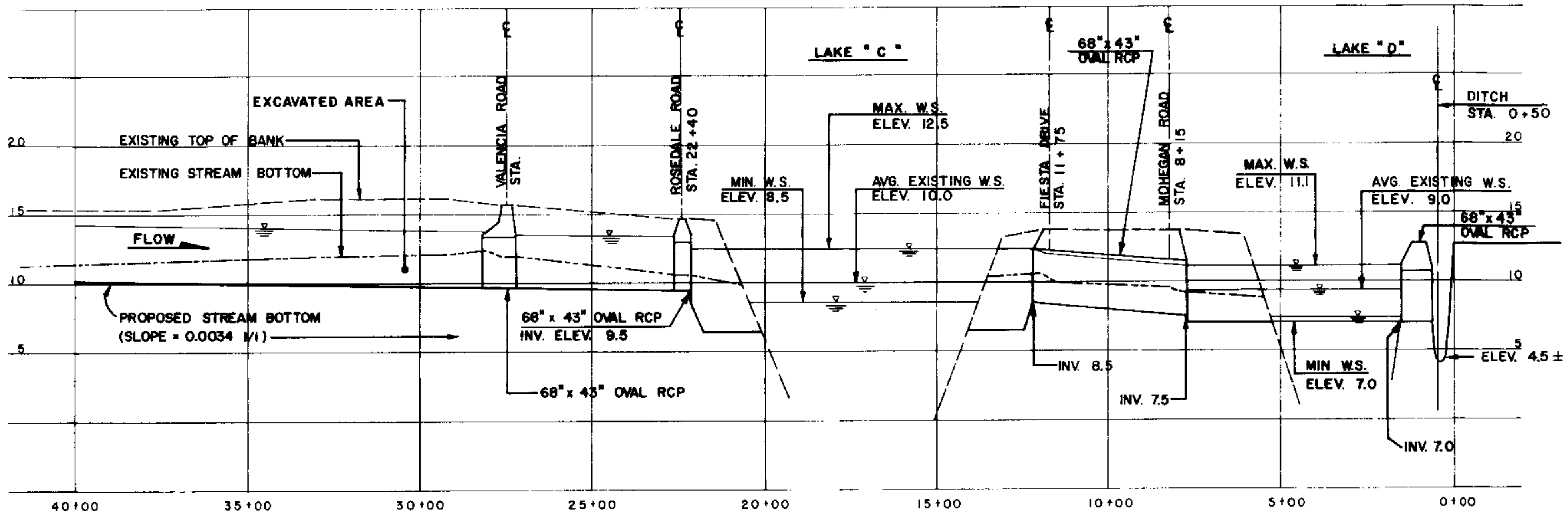
GENERAL PLAN
PROPOSED IMPROVEMENTS TO OUTLET CANAL
SOUTH VENICE GARDENS AREA

DESIGNED BY BLG
DRAWN BY BLG
CHECKED BY BLG
APPROVED BY BLG
DATE 6/10/83

REV NO | DATE | DRWN | CHKD | REMARKS

PROJECT NO
9250-I-RT

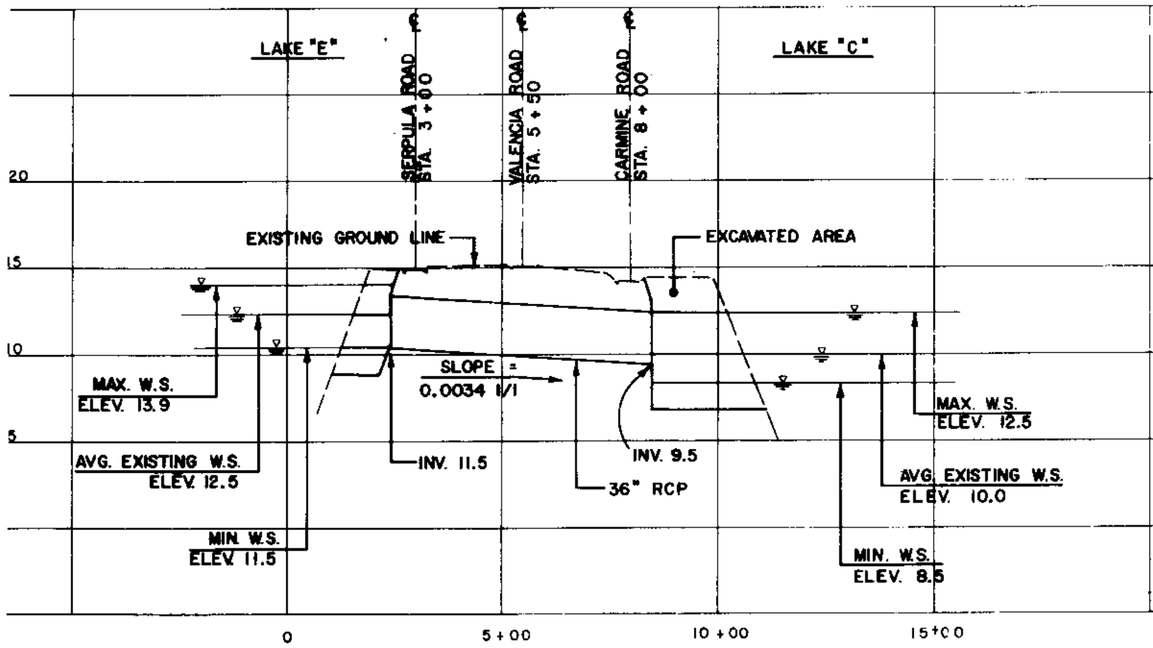
DRAWING NO.
2



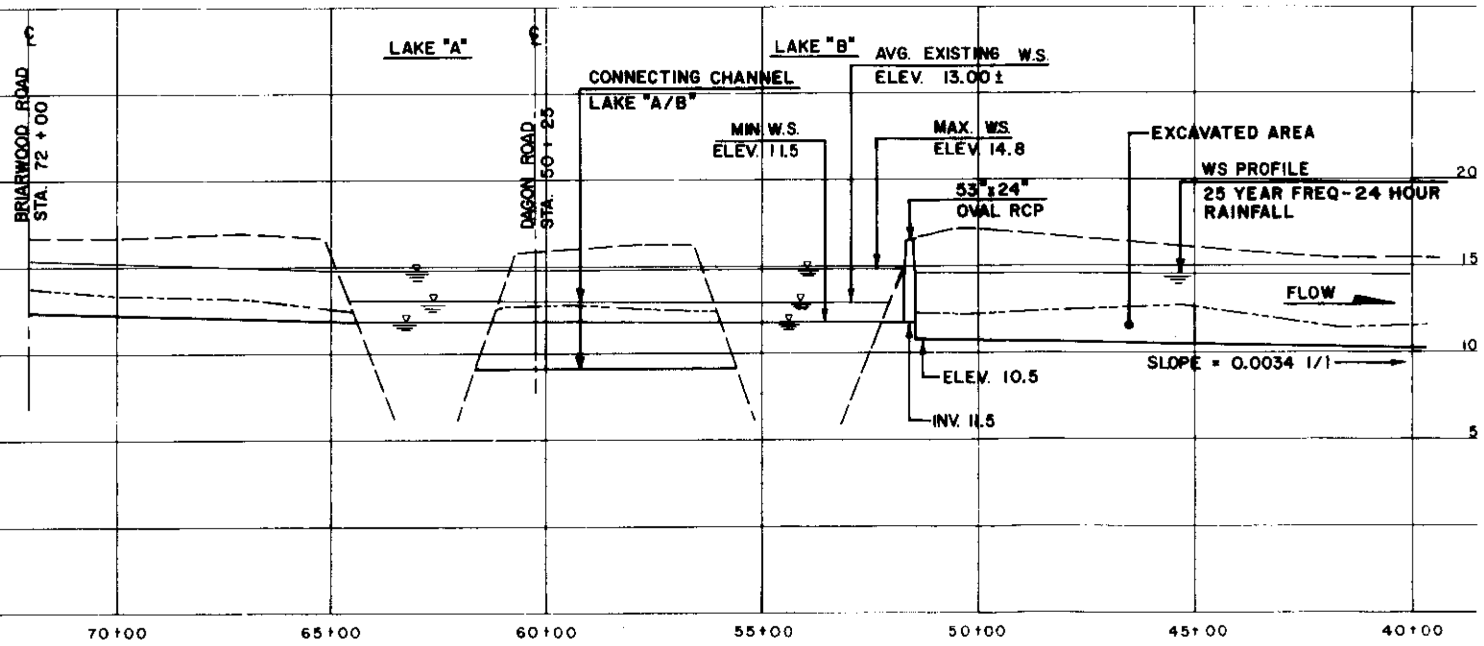
OUTLET CANAL PROFILE

1" = 5' VERTICAL

1" = 200' HORIZONTAL



LAKE "E" OUTLET PROFILE



OUTLET CANAL PROFILE

REV NO | DATE | DRWN | CHKD | REMARKS

DESIGNED BY
DRAWN BY
CHECKED BY
APPROVED BY
DATE

CAMP DRESSER & McKEE INC.

CDM

PROFILE
OUTLET CANAL – LAKE "E" OUTLET
SOUTH VENICE GARDENS AREA

PROJECT NO. 9250-I-RT

DRAWING NO. 3

Under the proposed storm water management plan presented in the Engineering Report, the following changes to the existing drainage system would be made:

1) The existing South Venice Outlet Canal would be improved (4.0 feet bottom width, 2:1 side slopes) from its origin at Station 71+80 just south of Briarwood Road at the intersection of Briarwood and Banya Drive to Station 64+00 at which point it enters Lake A.
2) Lakes A and B would be connected with a new channel (8 feet to 20 feet bottom width, 2:1 side slopes) so that the Lakes would act as a single storage unit (Lake A/B).
3) The normal dry weather level of Lake A/B would be lowered from Elevation 13.0± (the present water surface level) to Elevation 11.5 to provide additional reservoir storage.
4) A 25' long 54"x34" oval concrete pipe would be placed at the outlet of Lake A/B at Station 41+50 (Invert Elevation 11.5). This outlet structure would control the water surface level of Lake A/B.
5) The existing South Venice Outlet Canal would be widened and deepened (8.0 feet bottom width, 2:1 side slopes) from the proposed control structure a Station 41+50 (bottom elevation 10.5) just west of Dagon Road to a point just south of Rosedale Road at Station 22+50. A new (short-cut) channel for the Outlet Canal into Lake C would be constructed in this area.
6) The existing culvert under Valencia Road would be replaced with a 100 feet long 68"x43" oval pipe culvert.
7) A new 50 feet long 68"x43" oval pipe culvert would be placed under Rosedale Road at the new location of the Outlet Canal.

8) A new wide channel connection from the exit of the proposed new culvert under Rosedale Road to Lake C would be excavated.

9) A new wide approach channel connection from Lake C to the proposed outlet structure under Fiesta Drive would be excavated.

10) The normal dry weather level of Lake C would be lowered from Elevation 10.0$\pm$ to Elevation 8.5 to provide additional reservoir storage and to permit a higher head (additional potential energy) at the inlet of the proposed outlet structure.

11) The normal dry weather level of Lake E would also be lowered from Elevation 13.0$\pm$ to Elevation 11.5 to provide additional reservoir storage and to permit a higher head (additional potential energy) at the proposed new outlet structure.

12) A new outlet structure from Lake E to Lake C (600 feet of 36" RCP) from Serpula Road to Carmine Road would be installed. The existing clogged outlet running north to the Outlet Canal would be sealed.

13) A new wide approach channel connection from Lake E to the proposed entrance of the new outlet structure at Serpula Road would be excavated.

14) A new channel connection at the exit of the new outlet structure from Lake E would be excavated in Lake C.

15) The existing open channel outlet from Lake C to Lake D would be replaced with a 450 feet long, 68"x43" oval pipe, the inlet of which (Invert Elevation 8.5) would act to control the level of Lake C. This pipe would carry controlled flow from Lake C under Fiesta Drive and Mohegan Road to Lake D.

16) A new channel connection from the exit of the above mentioned outlet structure at Mohegan Road to Lake D would be constructed.

17) The normal dry weather level of Lake D would be lowered from Elevation 9.0± to Elevation 7.0 to provide additional storage and to provide more head on the outlet structure.

18) A 50 feet long 68"x43" oval pipe would be constructed at the outlet of Lake D (leading to the arm of Alligator Creek), the inlets of which (Invert Elevation 7.0) would act to control the level of Lake D.

19) The existing outlet from Lake D would be filled and the immediate area regraded.

The above listed improvements to the existing Outlet Canal and the lowering of the lakes will eliminate long duration flooding of those "lower" areas in the vicinity of the lakes. However, local street flooding (to a depth of several inches) and swales flooding will still occur for short periods of time due to the limited capacity of the transport system. Improvements to local street drainage (i.e., new larger culverts under certain driveways) should be constructed.

The above listed improvements to the existing Outlet Canal will require the acquisition by the County of a construction and maintenance easement approximately 40 feet in width from Dagon Road to Rosedale Road (30 feet for the canal plus 10 feet for a maintenance easement). In addition to this easement, the purchase of three double lots would be required.

In addition to the above proposed improvements, the Engineering Report recommends that two small areas, one 5.7 acres in size and one 4.3 acres in size (both adjacent to Englewood Road) be drained separately by the construction of small detention basins therein with outlets to the State Route drainage system as these areas were noncontributary to the Outlet Canal. The construction of detention basins in each of these areas would also require the purchase of double lots in each for the basins.

PART A

PROJECT DESCRIPTION

SECTION I

BASIN DESCRIPTION

General:

The drainage basin studied in this report consists of an 183 acre essentially urbanized, flat sandy area just east of State Route 41 and north of Alligator Creek in the south Venice area of Sarasota County.

This drainage basin is presently drained by a canal (ditch), henceforth referred to as the South Venice Gardens Outlet Canal or Outlet Canal, which starts just east of State Route 41 and flows easterly through four lakes approximately 7,500 feet to a branch of Alligator Creek and hence therein south to Alligator Creek itself. The drainage basin and outlet channel are shown on Drawings No. 1 and 2 which are located at the end of this report.

The drainage basin at the present time is essentially developed consisting principally of small single family residences located on 80 feet x 100 feet double lots except for an approximate 16 acre area on the extreme western end of the drainage basin which is presently being developed as a part of the Venetian Plaza Shopping Center.

As presently proposed, runoff from the 16 acre eastern area of this proposed shopping center will enter a detention basin to be constructed therein. Outflow from this detention basin will be by small storm sewer pipes to the existing State Route 41 sewer system.

Prior to development, runoff from the 16 acre area flowed slowly toward the previously mentioned Outlet Canal[1].

An additional 10 acres of the proposed Venetian Plaza Shopping Center will flow westerly toward the existing State Route 41 storm sewer system and hence therein southerly to Alligator Creek. This 10 acre area did not and will not contribute runoff to the previously mentioned South Venice Gardens Outlet Canal.

For purpose of computing hydrographs for storm water management plan analysis, the 183 acre drainage basin contributing to the Outlet Canal was subdivided into 8 subbasins designated Subbasins No. 1 and 2, Subbasin No. 3/4 and Subbasins No.5 through 9 as shown on Drawings No. 1 and 2.

Physical Characteristics:

Both the 183 acre drainage basin contributing to this South Venice Outlet Canal and the 10 acre area contributing to the State Route 41 storm sewer system are extremely flat with no predominate slope in any direction. However, this ground level does generally fall from Elevation 16.0 at the eastern end of the 183 acre basin at the start of this Outlet Canal to Elevation 14.0 at the outlet thereof; a drop of 2 feet in 7,000 feet - a slope of 0.0002 ft/ft.

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(1) Available gradient was so small that long term ponding probably resulted with much of the ponded runoff being removed by evapotranspiration.

Interior drainage of the basin is by roadway swales on both sides of the various roadways in the area toward the Outlet Canal and/or lakes in the basin during small rainfalls. However, in a portion of the basin, the swales in front of the house have been filled in and the small diameter limited capacity, culvert pipes (installed to convey water adjacent to the roads) are partially or completely blocked. Therefore, during major rainfalls, the swales adjacent to the roadways actually act as small detention basins with the roads additionally acting as 30-40 feet wide shallow depth conveyance channels which transmit runoff to the Outlet Canal and lakes.

In certain areas, the swales adjacent to the roadway and the roadway themselves slope in opposite directions (however so slight) such that storm water runoff could and probably does actually flow in two directions during major rainfall events.

For purposes of hydrograph computation, as will be subsequently discussed, the excess storage in the roadway swales was treated as depression storage and the depression storage used in the model for hydrograph computation increased from the standard 0.1 inch to 0.5 inch to account for this situation.

The increased home construction in the basin over the last ten year period has eliminated many vacant lots of lower elevation where runoff from other adjacent areas in the basin was temporarily stored. This process is obviously continuing and will ultimately compound the drainage problems in the general area.

The drainage basin contributary to the Outlet Canal and the subbasins thereof shown on Drawings No. 1 and 2 were delineated by actual field survey and visual inspection. However, because of the extreme flatness of the terrain, the indicated boundaries must be considered approximate.

Land Use:

For purposes of computing hydrographs of runoff from each of the subbasins of the 183 acre basin required for determining existing flooding conditions and for evaluating the proposed storm water management plan for the area, the existing total impervious area and directly connected impervious area (DCIA) of each subbasin was computed and the percent of each type land cover and adjusted growth[1]. The results of these computations are shown on Table 1. Also listed on Table 1 are the computed times of concentraion of each of the subbasins.

Non-contributary Areas:

Based on the field survey and visual inspection, two areas, one 5.7 acres in size and one 4.3 acres in size on the western end of Subbasin No. 3/4 both adjacent to Englewood Road were found to be non-contributary to the Outlet Canal. Rain falling on these areas ponds in roadway swales or adjacent vacant lots until removed by infiltration and/or evapotranspiration. As both areas were located a considerable distance from any lake such that altering the general area to make run off from these areas flow eastward was deemed

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(1) The principal adjustment for future growth was made in Subbasin No. 8, the least built up of the subbasins.

TABLE 1

DRAINAGE BASIN PROPERTIES

Subbasin Number	Area (acres)	Total Imp. Area Acres	Total Imp. Area %	DCIA[1] Acres	DCIA[1] %	Lake Area Acres	Lake Area %	T_c Hours
1[2]	16.0	10.2	64	3.6[3]	23	--	--	0.50
2	12.0	4.7	39	3.5	29	--	--	0.60
3/4	56.9	9.9	17	5.7	10	3.6	6	0.75
5	20.7	3.1	15	1.5	7	--	--	0.60
6	8.3	1.7	20	0.9	11	--	--	0.60
7	23.5	3.1	13	2.1	9	3.7	16	0.50
8	20.0	3.4	17	1.6	8	2.1	11	0.55
9	25.2	4.2	17	3.1	12	1.9	8	0.75

[1] Directly Connected Impervious Area

[2] Eastern Subbasin of Venice Gardens Shopping Center

[3] Includes Detention Basin

impractical; the construction of small detention basins in each area with outlets to the State Route 41 drainage system is subsequently recommended to resolve drainage problems in these areas.

Existing Drainage Pattern:

As previously stated, the existing 183 area drainage basin is drained by a canal, the South Venice Outlet Canal, which starts just east of State Route 41 and flows easterly approximately 7,500 feet to a branch of Alligator Creek. In this 7,500 feet length, the Outlet Canal passes through four lakes (designated Lakes A through D, as shown on Drawing No. 1 and 2). These lakes serve as reservoir storage to attenuate flows in the canal system. Also, a 25.2 acre central portion of the basin flows to another lake designated Lake E. Outflow from Lake E is by a small ditch which flows north to the South Venice Gardens Outlet Canal. However, for all practical purposes, this ditch from Lake E to the Outlet Canal is blocked at the present time.

The South Venice Gardens Outlet Canal and the Lake E outlet ditch pass under roadways and embankments via culverts. These culverts are identified by size, type, location and invert elevation on Table 2. Also listed on Table 2 are the approximate bottom elevations of the Outlet Canal at the various lake inlets and outlets.

The existing South Venice Gardens Outlet Canal is approximately trapezoidal in shape with 2:1 side slopes. The bottom of the canal varies in width from approximately 5 feet at its start near State Route 41 (Station 71+80) to 10 feet at Valencia Road (Station 28+00).

TABLE 2

CULVERT AND LAKE ELEVATIONS

Station	Description	Location or Designation	Types	Inv./El. In	Inv./El. Out	
5+00	Lake	D	--	9.4	8.9	
8+00	Culvert	Mohegan Rd.	36'-30" CMP	9.62	9.33	
12+00	Culvert	Fiesta Dr.	40'-24"x30" RCP	10.71	10.04	
16+00	Lake	C	--	10.4	10.7	
20+00	Culvert	Rosedale Rd.	38'-24" CMP	10.79	10.38	.38↓
28+00	Culvert	Valencia Rd.	36'-27"x42" CMP	12.34	11.98	2.08↓
55+00	Lake	B	--	12.5	12.2	
63+00	Lake	A	--	12.4	12.8	
64+30	Culvert	Inlet Lake A	10'-27"x42" CMP	12.36	12.36	
72+00	Culvert	Briarwood Rd.	36'-12"x18" RCP	13.68	13.68	
Lake E Outlet	Culvert	Morningside Road	38'-14"x23" RCP	12.20	12.35	
Lake E Outlet	Culvert	Driveway	20'-12"x18" RCP	12.01	12.05	
Lake E	Lake	E	---	-	12.50	

From the Lake C outlet to Lake D, the bottom width of the Outlet Canal is approximately 4 feet with 1:1 side slopes.

The approximate existing surface area of each of the five lakes is listed below:

Lake	Area (Acres)
A	1.4
B	2.2
C	3.7
D	2.1
E	2.5

For purposes of determining existing flooding condition, Lakes A and B were assumed to be a single lake - Lake A/B. Also for purposes of determining existing flooding conditions, it was assumed that because the ditch is for all practical purposes blocked, no flow passes from Lake E via the Lake E outlet ditch to the South Venice Gardens Outlet Canal.

Shopping Plaza Drainage System:

As previously stated, the proposed Venetian Plaza Shopping Center was divided by the shopping center consulting engineer into two subbasins; one approximately 16 acres in size, hereinafter referred to as the Eastern Subbasin, and one approximately 10 acres in size, hereinafter referred to as the Western Subbasin. Both subbasins include the rear portion of the lots of adjacent single family residences.

The plan of the Venetian Plaza Shopping Center (now under construction) and the proposed drainage system thereof are shown on Drawings No. 1 and 2.

Storm water runoff from the Eastern Subbasin, which is Subbasin No. 1 of the Venice Gardens Outfall Canal basin, will flow by overland sheet flow and storm sewers easterly to a detention basin to be constructed in this subbasin at the rear of the proposd Venetian Plaza Shopping Center.

As will be subsequently stated, this detention basin is large enough (7.2 acre feet at Elevation 16.5 - the overflow elevation) to contain runoff from the 25 Year Frequency - 24 Hour Duration Design Rainfall, such that no discharge from Subbasin No. 1 to the Outlet Canal will occur.

As previously stated, storm water runoff ponded in this detention basin will oulet in two pipes, one 15" in diameter and one 18" in diameter. These pipes will transmit runoff across the drainage divide to the existing SR 41 storm drain system through the drainage system being constructed to drain the 10 acre Western Subbasin of the Shopping Center. Water in this detention basin may be ponded for a significant period of time before completely running out through this system.

Runoff from the 10 acre Western Subbasin will flow westerly and enter the State Route 41 storm sewer system through two pipes, one 15" in diameter and one 18" in diameter, which pipes act to control flow to this system.

Design Hydrographs:

The hydrographs of runoff from each of the eight subbasins comprising the total 183 acre basin from the 25 Year Frequency - 24 Hour Duration Design Rainfall are shown on Figures 1 through 8 which constitute Section 1 of the Appendix of this Report. The procedures used in the computation of these hydrographs are subsequently described in Part B of this Report. As previously stated, the hydrograph of runoff from Subbasin No. 1, shown on Figure 1, as the 16 acre Eastern Subbasin of Venetian Gardens Shopping Plaza, which amounts to 4.2 inches, will be completely contained in the detention basin to be constructed therein.

The design hydrographs of runoff from each subbasin as shown on Figures 1 through 8 in the Appendix were used to evaluate both the existing flooding condition and the proposed stormwater management plan (as only alterations to the existing receiving lakes and conveyance system was contemplated).

SECTION II

EXISTING FLOODING CONDITIONS

General:

In order to ascertain the affect of the 25 Year Frequency - 24 Hour Duration Design Rainfall on the 183 acre drainage basin under existing conditions (no improvements to the existing South Venice Outlet Canal but with the proposed Shopping Center in place), hydrographs of runoff from each of the eight subbasins from the 25 Year Frequency - 24 Hour Duration Rainfall were summed and/or routing through lake storage (by the storage-indication working curve method) or through channel storage (by the Muskingum Method) to determine the peak flows at various locations along the existing Outlet Canal and to determine the inflow hydrographs to and the outflow hydrographs from the various existing lakes.

The results of these actions are shown on Figures 9 through 20 in Section 2 of the Appendix to this Report. The approximate areas flooded by the 25 Year Frequency rainfall are shown on Drawing No. 2.

SECTION III

STORM WATER MANAGEMENT PLAN

General:

Under the proposed storm water management plan, the following changes to the existing drainage system would be made:

1) The existing South Venice Outlet Canal would be improved (4.0 feet bottom width, 2:1 side slopes) from its start from Station 71+80 just south of Briarwood Road at the intersection of Briarwood and Banya Drive to Station 64+00 at which point it enters Lake A.

2) Lakes A and B would be connected with a new channel (8 feet to 20 feet bottom width, 2:1 side slopes) so that the Lakes would act as a single unit (Lake A/B).

3) The level of Lake A/B would be lowered from Elevation 13.0± (the present water surface level) to Elevation 11.5 to provide additional reservoir storage.

$36.09 foot $902.25

4) A 25 feet long ~~54~~ 53"x34" oval concrete pipe would be placed at the outlet of Lake A/B at Station 41+50 (Invert Elevation 11.5). This outlet structure would control the water surface level of Lake A/B.

5) The existing South Venice Outlet Canal would be widened and deepened (8.0 feet bottom width, 2:1 side slopes) from the proposed control structure at Station 41+50 (Bottom Elevation 10.5) to a point just

south of Rosedale Road at Station 22+50. A new (short-cut) channel for the Outlet Canal into Lake C will be constructed in this area.

6) The existing culvert under Valencia Road would be replaced with a 100 feet long 68" x 43" oval pipe culvert.
#57.21 foot
$5,721.00

7) A new 50 feet long 68"x43" oval pipe culvert will be placed under Rosedale Road at the new location of the Outlet Canal in this area.
57.21 foot = $2,860.50

8) A new wide channel connection from the exit of the proposed new culvert under Rosedale Road to Lake C would be excavated.

9) A new wide approach channel connection from Lake C to the proposed outlet structure under Fiesta Drive would be excavated.

10) The level of Lake C would be lowered from Elevation 10.0$\pm$ to Elevation 8.5 to provide additional reservoir storage and to permit a higher head (additional potential energy) at the inlet of the proposed outlet structure.

11) The level of Lake E would be lowered from Elevation 13.0$\pm$ to Elevation 11.5 to provide additional reservoir storage and to permit a higher head (additional potential energy) at the inlet end of the proposed new outlet structure.

$13,776.00

$22.96 foot

12) A new outlet structure from Lake E to Lake C (600 feet of 36" RCP) from Serpula Road to Carmine Road would be installed. The existing clogged outlet running north to the Outlet Canal would be sealed.

13) A new wide approach channel connection from Lake E to the proposed entrance of the new outlet structure at Serpula Road would be excavated.

14) A new channel connection at the exit of the new outlet structure from Lake E would be excavated in Lake C.

15) The existing open channel outlet from Lake C to Lake D would be replaced with a 450 feet long, 68"x43" oval pipe; the inlet of which (Invert Elevation 8.5) would act to control the level of Lake C. This pipe would carry controlled flow from Lake C under Fiesta Drive and Mohegan Road to Lake D.

$57.21 foot $25,744.50

16) A new channel connection from the exit of the above mentioned outlet structure at Mohegan Road to Lake D would be constructed.

17) The level of Lake D would be lowered from Elevation 9.0± to Elevation 7.0 to provide additional storage and to provide more head on the outlet structure.

$57.21 foot $2,860.50

18) A 50 feet long 68"x43" oval pipe would be constructed at the outlet of Lake D (leading to the arm of Alligator Creek), the inlets of which (Invert Elevation 7.0) would act to control the level of Lake D.

19) The existing outlet from Lake D would be filled and the immediate area regraded.

The above listed improvements to the existing Outlet Canal and the lowering of the lakes will eliminate long duration flooding of those "lower" areas in the vicinity of the lakes and conveyance system. However, as also previously stated, local street flooding (to a depth of several inches) and swales completely filled with water will still occur for short periods of time due to the limited capacity of the transport system. Improvements to local street drainage (i.e., new, larger culverts under certain driveways) could, and probably should, be placed. These improvements should be done judiciously to prevent accelerating storm water runoff to the lakes and Outlet Canal, thus compounding the flooding problem. However, the lowering of the lakes and improvements to the Outlet Canal will increase the hydraulic gradient and so undoubtedly have a beneficial affect on local street flooding.

The above listed proposed changes to the existing Outlet Canal drainage system are shown on Drawing No. 2. A profile along the center line of the improved Outlet Canal which shows the proposed bottom of the improved Outlet Canal channel, the water surface elevation in this channel and the various lakes resulting from the 25 Year Frequency - 24 Hour Rainfall is shown on Drawing No. 3.

Analysis (Flood Control Effectiveness):

In order to determine the effectiveness of the above proposed system, the hydrographs of runoff from the various subbasins from the 25 Year Frequency - 24 Hour Duration Rainfall were summed and/or routed through lake storage (by the storage-indication working curve method) or through channel storage (by the Muskingum Method) to determine the peak flows at various locations along the Outlet Canal and to determine the inflow hydrographs to and the outflow hydrographs from the various lakes. The results of these various summations and routings are shown on Figures 22 through 32. These figures constitute Section 3 of the Appendix of this Report. The peak flow rates of the various hydrographs are listed on Table 3.

As can be observed from Drawing No. 3, the peak flows from the 25 Year Frequency - 24 Hour Duration Design Rainfall will be almost completely contained within the banks of the improved Outlet Canal and lakes.

Alternative Plan:

In addition to the above plan, a second alternative plan was studied. This plan involved the construction of a new storm sewer system running westerly to the Intracoastal Waterway via Shamrock Drive. This second alternative plan was predicated on the assumption that the proposed drainage system and detention basin being constructed as part of the proposed Venetian Plaza Shopping Center would still allow a substantial flow to enter the upper (western) end of the Outlet Canal during a major rainfall. Thus, the construction of a new storm sewer system to the Intracoastal Waterway to eliminate or lessen the impact of runoff from the shopping center on the Outlet Canal would have been required. However, as previously stated, the

TABLE 3

PEAK FLOWS IN OUTLET CANAL

FUTURE CONDITION

No.	Location	Time Hours	Peak Flow CFS
1	Outlet Lake A/B (Station 51 & 50)	17.0	60
2	Station 41 & 50	17.0	60
3	Station 41 & 59	17.0	77
4	Station 28 & 00	17.0	84
5	Station 18 & 00	17.0	84
6	Station 19 & 00	16.5	104
7	Entrance Lake C	17.0	128
8	Outlet Lake C	18.0	108
9	Entrance Lake D	17.5	118
10	Outlet Lake D	18.0	115

proposed detention basin being constructed as part of the shopping center will fully contain all runoff from a 25 Year Frequency - 24 Hour Duration Rainfall. As also previously stated, this construction is an improvement over the existing predevelopment situation. The adequacy of the proposed shopping center storm drainage system, plus the estimated cost of a new storm sewer system (estimated at over $500,000), ruled out this possible alternative.

PART B

HYDROLOGIC DESIGN

SECTION I

SOIL CONDITIONS

General:

The soil mantle in the drainage basin has been classified by the Soil Conservation Service (SCS)as predominately Leon fine sand. Small pockets of Rutledge fine sand and Plummer fine sand are also located in the basin. These soil pockets were considered to be too small to have any significant effect on the general hydrologic characteristics of the basin.

The Leon fine sand located within the basin is made up of layers of fairly pervious sand to a depth of approximately 50 inches. The top 18 to 30 inches is quite pervious (2-5% passing the No. 200 sieve) and so has been classified by the SCS in the Unified Soil Classification System as SP. However, immediately below the top 18-30 inches is a less pervious semi-hard organic layer 3 to 6 inches in thickness (7-15% passing the No. 200 sieve), which has been classified by the SCS in the Unified System as SP, SP-SM. This layer tends to reduce vertical percolations and cause perched water tables in the basin at certain times of the year. Underneath the top 50 inches of fine sandy soil is a relatively impervious clayey layer classified by the SCS in the Imified System as SM.

Hydrologic Soil Classification:

For the purposes of computing direct runoff(1), the Hydrologic Soil Classification system of the Soil Conservation Service was used. In this

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(1) Direct runoff (rainfall-excess) is that portion of rainfall which actually runs off the land and becomes sheet flow and ultimately stream flow.

system, the Soil Conservation Service classifies soils into four groups, depending on their runoff potential. This potential is measured by two factors; infiltration rate into the soil which is controlled by surface conditions and transmission rate through the soil which is controlled by the soil horizons. These four hydrologic soil groups are as follows:

A. (Low runoff potential.) Soils having high infiltration rates even when thoroughly wetted and consisting chiefly of deep, well to excessively drained sands or gravels. These soils have a high rate of water transmission.

B. Soils having moderate infiltration rates when thoroughly wetted and consisting chiefly of moderately deep to deep, moderately well to well-drained soils with moderately fine to moderately coarse textures. These soils have a moderate rate of water transmission.

C. Soils having slow infiltration rates when thoroughly wetted and consisting chiefly of soils with a layer that impedes downward movement of water, or soils with moderately fine to fine texture. These soils have a slow rate of water transmission.

D. (High runoff potential.) Soils having very slow infiltration rates when thoroughly wetted and consisting chiefly of clay soils with a high swelling potential, soils with a permanent high water table, soils with a clay pan or clay layer at or near the surface, and shallow soild over nearly impervious material. These soils have a very slow rate of water transmission.

The Leon fine sand has been given the dual classification A/D by the SCS. The first letter (A) applies to the drained condition as would be the expected situation in sandy soils with a small amount of fines when the groundwater table is below the surface. The second latter (D) applies to the undrained condition which would normally be the condition in a rural (undeveloped) very flat basin during the wet season when the groundwater table is near or at the ground surface (or water is standing on the ground surface). Under such circumstances the placing of the soil in Hydrologic Group D classification would be mandatory since most of the rainfall occurs durng the summer season when the groundwater table is high.

However, the urbanization of an area does lower the groundwater table as a result of grading and paving thereof which causes rainfall to run off more rapidly rather than ponding on the ground surface and supplying the groundwater table. Also, the construction of lakes, detention ponds, storm sewers, sanitary sewer systems, water mains, ditches, underdrains, etc., which drain the upper portion of the soil mantle act to keep the groundwater table several feet below the surface by providing positive drainage of the upper layers.

Therefore, for purposes of computing runoff under existing developed conditions, the soils in the drainage basin were considered to be the Hydrologic Soil Group C category, except for the soils in Subbasin No. 1 (Venetian Plaza Shopping Center Area). These soils were assumed to be in Hydrologic Soil Group A for the developed condition. The latter developed condition classification in Subbasin No. 1 was the result of the proposed extensive underdrain/stormdrain system to be constructed therein.

Groundwater Level:

For purposes of computing runoff from Subbasin Nos. 2 through 9 (from those subbasins with Group C soils), it was assumed that urbanization thereof would keep the groundwater table (top of saturated zone) a minimum of 2.0 feet below the ground surface at all times. For the purposes of computing runoff from Subbasin No. 1 (from that subbasin with an assumed Group A soil), it was determined that urbanization thereof would keep the groundwater table a minimum of 2.5 to 3.0 feet below the ground surface at all times.

The computed available water storage volume in the soil (in inches) was computed by multiplying the estimated water holding capacity of the soil (which equals 0.25 in/in) by the depth to the saturated zone in inches is as follows:

Depth to Water Table	Cumulative Water Storage
2.0	6.0"
3.0	9.0"

SECTION II

HYDROLOGIC DESIGN PROCEDURES

General:

The procedures used in the computation of flows (surface runoff) from the various drainage basins considered in this report followed the general procedures currently used in the computation of flows from most small drainage basins; the establishing of a design rainfall for a particular frequency; the computation of direct runoff (rainfall-excess increments) from the design rainfall and the computation of a design hydrograph of runoff utilizing the computed rainfall-excess increments.

The computed design hydrographs were then combined (summed), routed through reservoir storage in the lakes by the storage-indication working curve method, and through storage in the channel reaches by the Muskingum Method, to obtain the final design hydrographs at various points along the Outlet Canal.

The actual method used to compute the design hydrographs of runoff was the HNV-Santa Barbara Urban Hydrograph Method (HNV-SBUH Method), a tested simulation model, which is described in detail in the subsequent section of this part of the report.

Computer programs were actually used to compute the design hydrographs and to perform the necessary flood routings and hydrograph summations.

Design Rainfall:

For purposes of analysis and design, and in accordance with the requirements of Sarasota County's Land Development Regulations, a 25 Year Frequency Rainfall was used in hydrograph computation.

From a study of past major storms that have occurred in the southeastern portion of the United States and in the southwestern section of the State of Florida in particular, it was apparent that a large portion of the total rainfall of major storms fell within a 24 hour period such that, for purposes of this engineering report, this time period was adopted as the time period for the 25 Year Frequency Rainfall. The total rainfall depth for the 25 Year Frequency - 24 Hour Duration Rainfall taken from Weather Bureau Technical Paper No. 40(1) amounted to 9.5 inches.

In order to be able to establish a reasonable design rainfall intensity, mass rainfall curves of past major rainfalls that have occurred in the West Central Florida area were plotted as dimensionless ratios and are shown on Figure 1.

In order to compare these actual intensities with the standard rainfall intensities used by the Corps of Engineers, and the Soil Conservation Services, dimensionless ratios of the various design rainfalls used by

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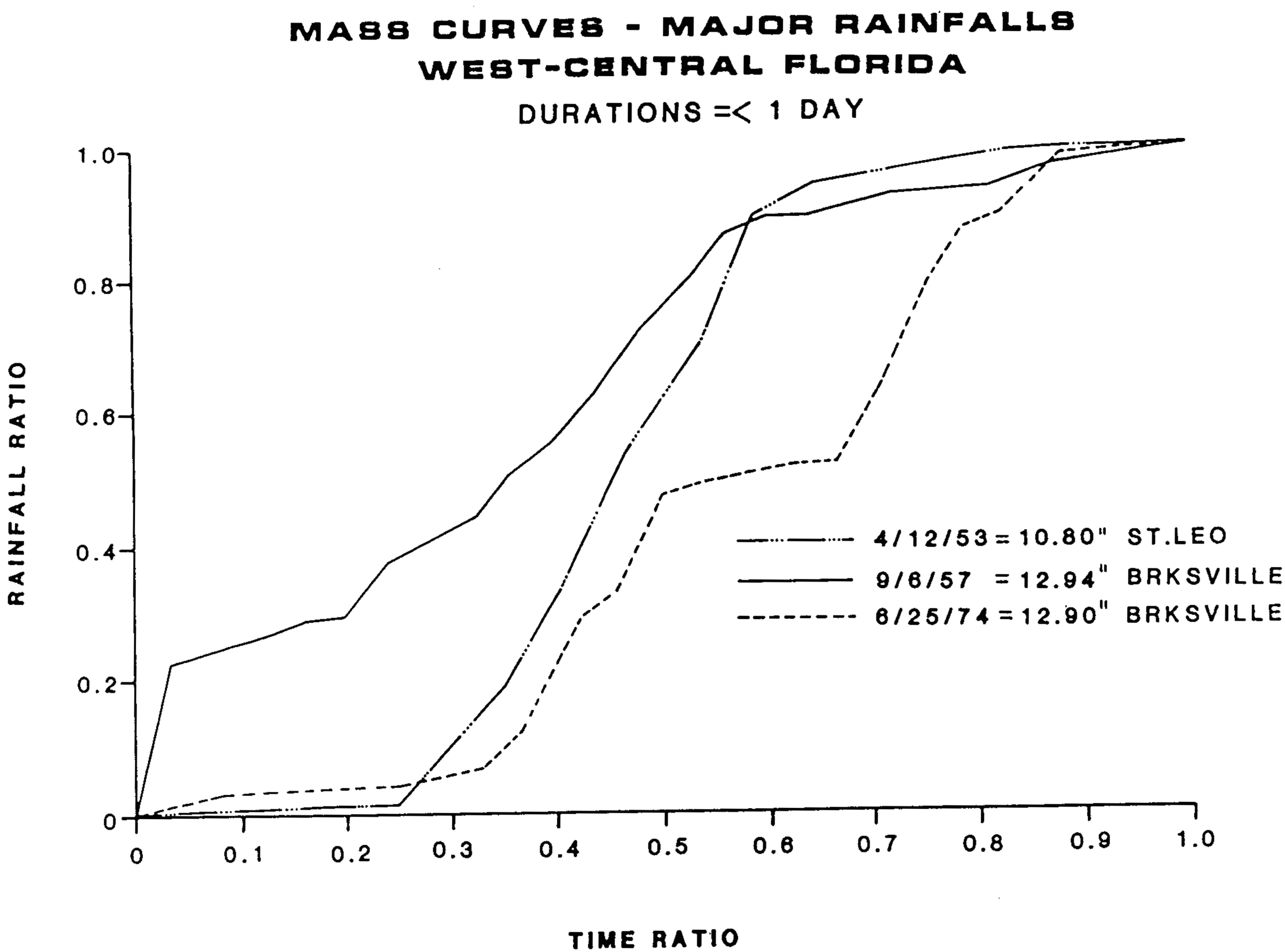
(1) Technical Paper No. 40, Rainfall Frequency Atlas of the United States for Durations from 30 Minutes to 24 Hours and Return Periods from 1 to 100 Years", Weather Bureau (Now NOAA), U.S. Dept. of Commerce, Washington, D.C., May 1961.



MASS CURVES - MAJOR RAINFALLS
WEST-CENTRAL FLORIDA
DURATIONS =< 1 DAY
RAINFALL RATIO
1.0
0.8
0.6
0.4
0.2
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
TIME RATIO
4/12/53 = 10.80" ST.LEO
9/6/57 = 12.94" BRKSVILLE
6/25/74 = 12.90" BRKSVILLE

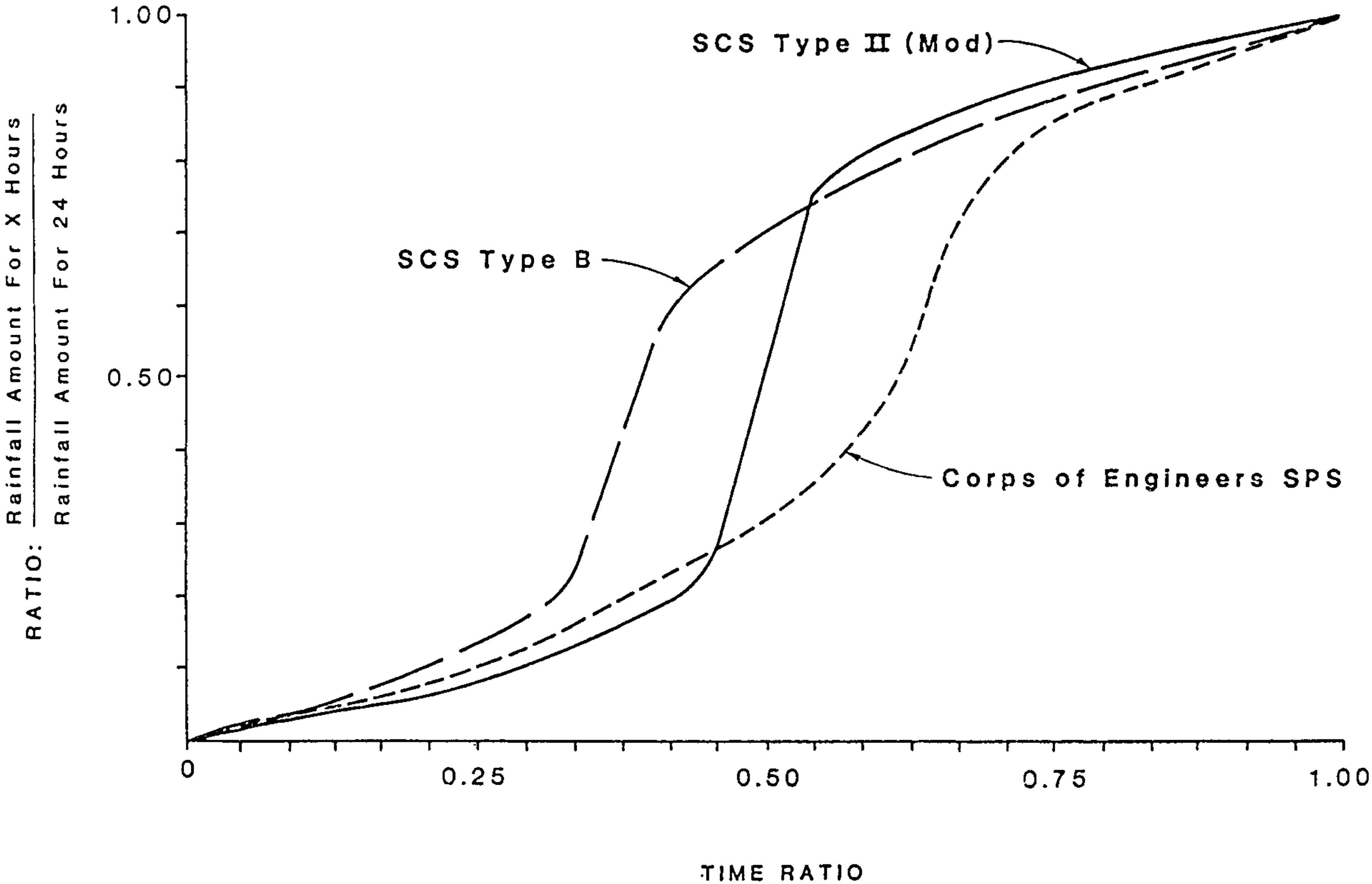
FIGURE 1

these agencies, or suggested by these agencies for general use, were also computed and plotted, and are shown on Figure 2 for a one-day duration.

From this Figure, it was immediately apparent that the rainfall intensity of the Corps of Engineers Standard Project Storm (SPS) distribution more nearly matched the rainfall intensities of past major events for the one-day duration. Based on this comparison the rainfall distibution of the Corps of Engineers as shown on Figure 2 was adopted for use in computing the design flood hydrographs in this report.

Successive thirty minute rainfall increments (Δ P's) for the 25 Year Frequency - 24 Hour Duration Rainfall and summation (Σ P's) of these rainfall increments distributed in general accordance with the Corps of Engineers Standard Project Storm (SPS) distribution are listed on Table 4. A hyetograph of these rainfall increments is plotted on Figure 3.

24 HOUR DESIGN STORM DISTRIBUTION



FIGURE

TABLE 4

25 YEAR FREQUENCY - 24 HOUR RAINFALL

(ONE HALF HOUR INCREMENTS)

Time Hours	Σ P Inches	Δ P Inches	Time Hours	Σ P Inches	Δ P Inches
0					
0.5	0.07	0.07	12.5	3.18	0.23
1.0	0.14	0.07	13.0	3.42	0.24
1.5	0.21	0.07	13.5	3.77	0.35
2.0	0.28	0.07	14.0	4.13	0.36
2.5	0.35	0.07	14.5	4.58	0.55
3.0	0.42	0.07	15.0	5.23	0.55
3.5	0.49	0.07	15.5	6.11	0.88
4.0	0.56	0.07	16.0	6.98	0.97
4.5	0.66	0.10	16.5	7.39	0.41
5.0	0.76	0.10	17.0	7.79	0.40
5.5	0.88	0.12	17.5	8.01	0.22
6.0	1.00	0.12	18.0	8.22	0.21
6.5	1.12	0.12	18.5	8.36	0.14
7.0	1.24	0.12	19.0	8.50	0.14
7.5	1.40	0.16	19.5	8.62	0.12
8.0	1.56	0.16	20.0	8.74	0.12
8.5	1.72	0.16	20.5	8.84	0.10
9.0	1.89	0.17	21.0	8.94	0.10
9.5	2.06	0.17	21.5	9.04	0.10
10.0	2.23	0.17	22.0	9.14	0.10
10.5	2.40	0.17	22.5	9.23	0.09
11.0	2.57	0.17	23.0	9.32	0.09
11.5	2.75	0.19	23.5	9.41	0.09
12.0	2.95	0.19	24.0	9.50	0.09

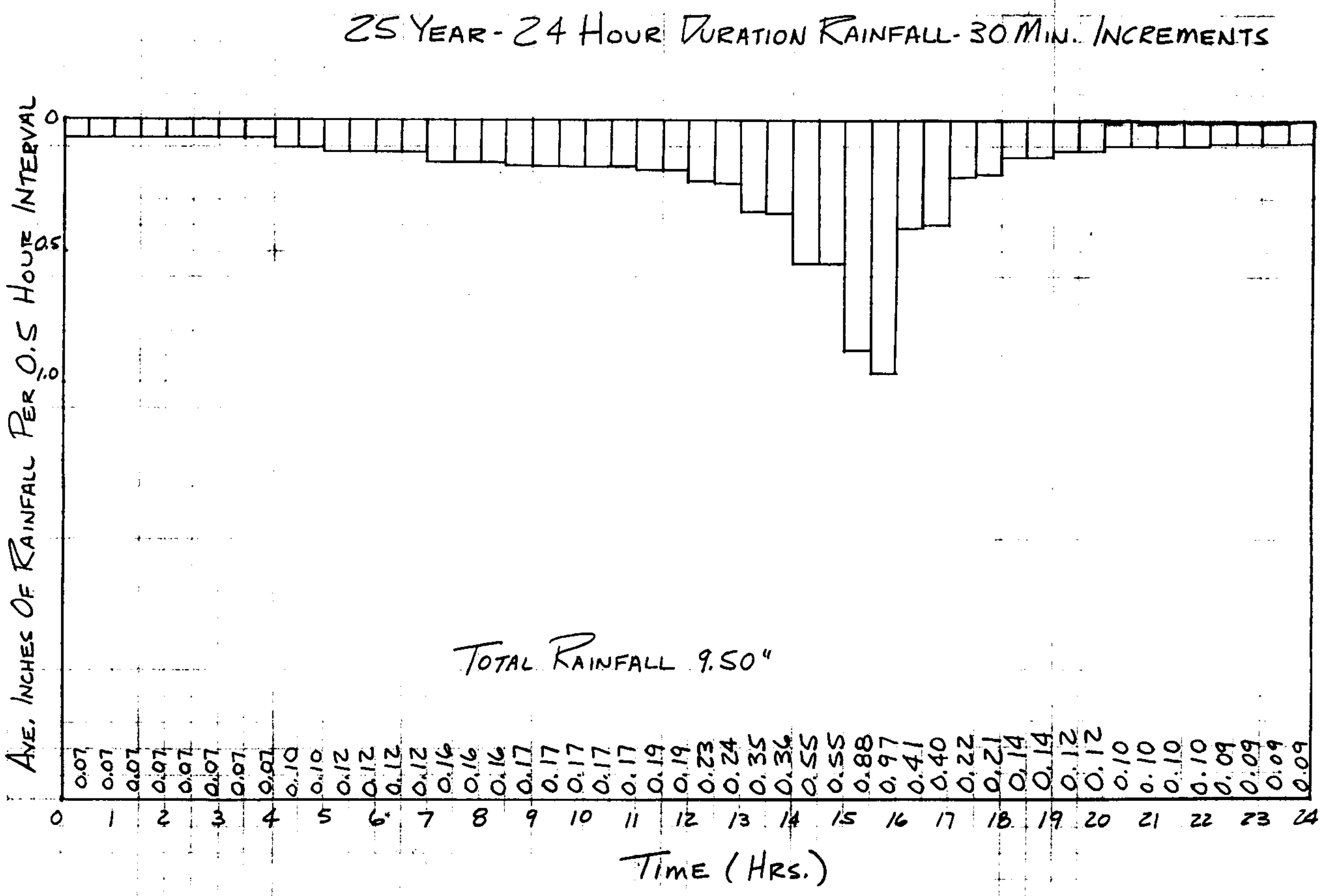

25 Year - 24 Hour Duration Rainfall - 30 Min. Increments
Ave. Inches Of Rainfall Per 0.5 Hour Interval
0
0.5
1.0
Total Rainfall 9.50"
0.07 0.07 0.07 0.07 0.07 0.07 0.07 0.07 0.10 0.10 0.12 0.12 0.12 0.12 0.16 0.16 0.16 0.17 0.17 0.17 0.17 0.17 0.19 0.19 0.23 0.24 0.35 0.36 0.55 0.55 0.88 0.97 0.41 0.40 0.22 0.21 0.14 0.14 0.12 0.12 0.10 0.10 0.10 0.10 0.09 0.09 0.09 0.09
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time (Hrs.)

Figure 3

SECTION III

HNV-SANTA BARBARA URBAN HYDROGRAPH METHOD

General:

The HNV-Santa Barbara Urban Hydrograph Method (HNV-SBUH Method) is a modification of a method originally developed by Mr. James M. Stubchaer, F. ASCE, of the Santa Barbara County (California) Flood Control and Water Conservation District.[1] The method as described herein computes a hydrograph directly without going through any intermediate process, as does the unit hydrograph method.

The HNV-SBUH Method is in many respects similar to some of the time-area-concentration curve procedures for hydrograph computation in which an instantaneous hydrograph in a basin is developed and then routed through an element of linear storage to determine basin response. However, in the HNV-SBUH Method, the final design (outflow) hydrograph is obtained by routing the instantaneous hydrograph for each time period (obtained by multiplying the various incremental rainfall excesses by the entire water-shed area in acres) through an imaginary linear reservoir with a routing constant equivalent to the time of concentration of the drainage basin. Therefore, the difficult and time consuming process of preparing a time-area-concentration curve for the basin is eliminated.

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(1) Stubchaer. J. M., "The Santa Barbara Urban Hydrograph Methods", Proceedings, National Symposium on Hydrology and Sediment Conrol, ORES Publication College of Engineering, University of Kentucky, November 1975.

Model Description:

A step-by-step description of the basic HNV-SBUH Method is given below:

1. Runoff depths for each time period are calculated using the following equations:

   Directly Connected Impervious Area Runoff -

   $R(0) = I\ P(t)$ (inches) (1)

   Pervious Area Runoff -

   $R(1) = P(t)\ (1-I-I_1)-f(1-I_t-I_1)$ (inches) (2)

   Lake Area Runoff -

   $R(2) = P(t)I_1$ (inches) (3)

   Total Runoff Depth -

   $R(t) = R(0) + R(1) + R(2)$ (inches) (4)

where

$P(t)$ = Rainfall depth during time increment t (inches)

f = Infiltration during time increment t (inches)

I_t = Total impervious portion of drainage basin (decimal)

I = Directly connected impervious portion of basin (decimal)

I_1 = Lake Area (decimal)

Δt = Incremental time period (hours, i.e., 0.25, 0.50, etc.)

The directly connected impervious area (I or DCIA), sometimes referred to in literature as the hydraulically effective impervious area, are those impervious areas where runoff therefrom does not flow over a pervious area before reaching and entering an element of the drainage system (streets with curbs, catch basins, storm drains, etc.).

In the derivation of Equation (2) and as shown on Figure 4, the rainfall on the pervious area is considered to be made of two parts, the rainfall on the non-directly connected impervious area $(I_t-I)P(t)$ which is assumed to run off uniformly onto and to be distributed uniformly over the pervious area and the rainfall on the pervious area $(1-I_t-I_1)P(t)$ which, when added together, gives the equation $(1-I-I_1)P(t)$. The runoff from the pervious area is then obtained by subtracting infiltration f $(1-I_t-I_1)$ from this total rainfall or $R(I) = P(t)(1-I-I_1)-f(1-I_t-I_1)$.

2. The instantaneous hydrograph of runoff from the land area is then computed by multiplying the total runoff depth R(t) in each time increment by the drainage basin area A in acres and dividing by the time increment t in hours.

$$I(t) = R(t) \cdot A/\Delta t \qquad \text{(cfs)} \qquad (5)$$

As in the Rational Method, the conversion factor 1.008 was ignored.

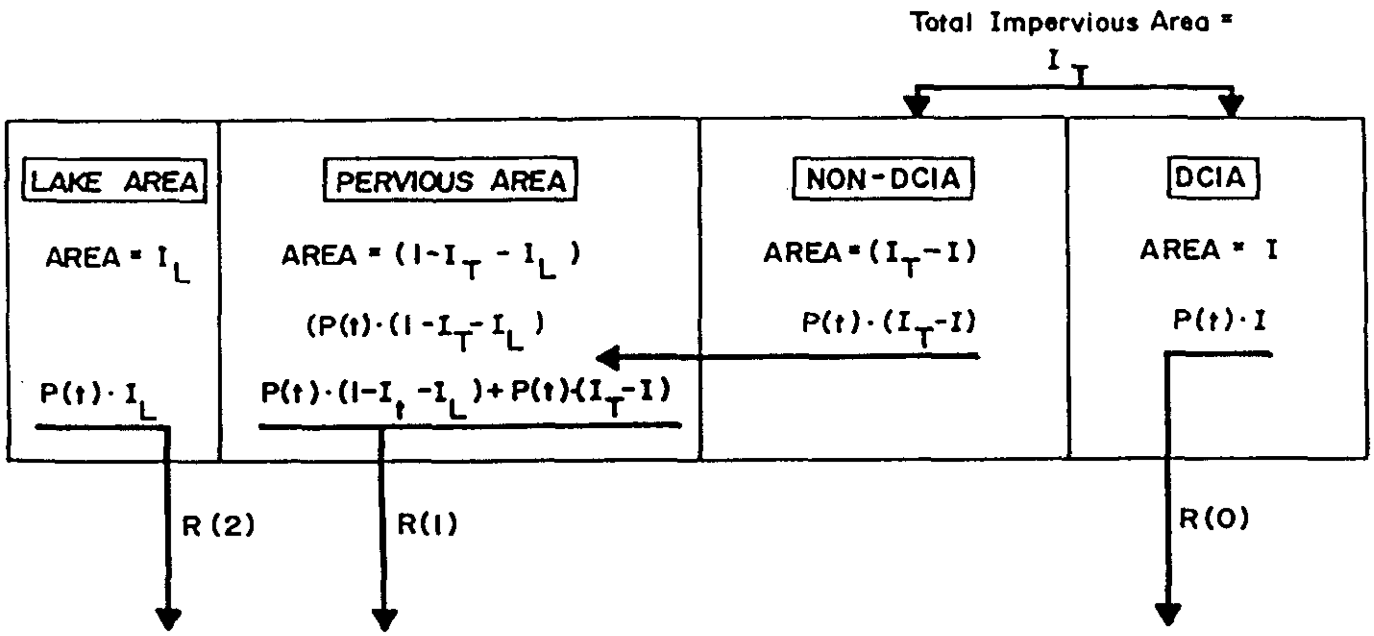
3. The final runoff hydrograph Q(t) from the land area is then obtained by routing the instantaneous hydrograph I(t) through an imaginary reservoir with a time delay equal to the time of concentration (Tc) of the drainage basin. This flood routing may be done by use of the following equation which is subsequently derived on Table 5.

$$Q(t) = Q(t-1) + K\,[I(t-1)+I(t)-2Q(t-1)] \qquad \text{(cfs)} \qquad (6)$$

where

$$K = \frac{\Delta t}{2T_c + \Delta t}$$

and where T_c = time of concentration of the basin (hours).



$$\text{AREA} = (1-I_T-I_L)+(I_T-1)+I+I_L = 1$$

$$R(0) = P(t)\cdot I$$

$$R(1) = P(t)\cdot(1-I_T-I_L)+P(t)(I_T-I)-f(1-I_T-I_L)$$

$$R(1) = P(t)\cdot 1-P(t)I_T-P(t)I_L+P(t)I_T-P(t)I-f(1-I_T-I_L)$$

$$R(1) = P(t)(1-I-I_L)-f(1-I_T-I_L)$$

$$R(2) = P(t)\ I_L$$

$$R(t) = R(0)+R(1)+R(2) = P(t)\,I+P(t)(1-I-I_L)-f(1-I_T-I_L)+P(t)\,I_L$$

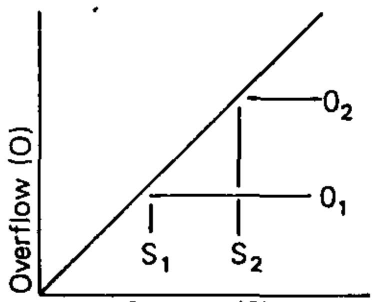
DERIVATION OF EQUATIONS

HNV-SANTA BARBARA URBAN HYDROGRAPH METHOD

FIGURE 4

Table 5
Santa Barbara Urban Hydrograph Method
Derivation of Routing Equation

1. Consider a linear reservoir with a definite storage S such that the storage is directly proportional to the outflow, $\Delta S \propto O$



$$\text{Time}^{*} \cong \frac{S_2 - S_1}{Q_2 - Q_1}$$

2. Slope of Storage Curve $T_c = \dfrac{\Delta S}{Q_2 - Q_1}$ = Flow Travel Time * in Reach = Time Delay in Reservoir

$$\Delta S = T_c\,(Q_2 - Q_1)$$

where T_c = Time of concentration

3. Substituting in the General Storage Equation —

$$T_c\,(Q_2 - Q_1) = \Delta S = \frac{(I_1 + I_2)}{2}\,\Delta t - \frac{(Q_1 + Q_2)}{2}\,\Delta t$$

4. From which can be derived —

$$Q_2 = Q_1 + K\,(I_1 + I_2 - 2Q_1)$$

Where $K = \dfrac{\Delta t}{2T_c + \Delta t}$ and where

T_c = time of concentration of the basin (hours)

---

* Flow Travel Time = K in Muskingum Method

4. The rainfall falling on the lake area $P(t)(I_1.A)/\Delta t$ in each time increment is then added to the routed outflow hydrograph from the land area in that increment to give the total final outflow design hydrograph from the drainage basin.

In the HNV-SBUH Method, normally all of the rain which falls on the land area is considered runoff except for the first 0.1 inch (depression storage) which is automatically subtracted from the rainfall in the programs.

Infiltration:

In using the HNV-SBUH Method to compute design flood hydrographs, infiltration is computed in accordance with the relationships illustrated on Figure 5 as first described by Holtan and developed and presented by Terstriep and Stall.(2) In this methodology, it is assumed that an initial (Maximum) Infiltration Rate f_o in inches per hour is available in the soil mantle when rainfall starts which drops off exponentially with time as the voids in the soil become filled with water to some Final (Constant) Infiltration f_c - in inches per hour. The total cross-hatched area below the Infiltration Curve and above the Final (Constant) Infiltration Rate line (=S) represents the total available water storage in the soil mantle. As shown on Figure 5, this Total Available Water Storage Capacity, S, in the soil in inches is divided into parts; 1) the water already stored in the soil when rainfall begins (F) which is the result of previous rainfalls, and 2) the actual available Storage

(2) Terstriep, M. L. and Stall, J. B., "The Illinois Urban Drainage Area Simulator "ILLUDAS" Illinois State Water Survey, Bulletin 58, State of Illinois, Department of Registration and Education, 1974.

in the Soil (S-F). Of course, if a long dry period preceded the start of rainfall, the total area (actually a volume) under the Infiltration Curve (=S) would be available for water storage. In the computer programs, the infiltration f shown on Figure 5 is computed by the Horton equation:

$$f = f_c + (f_o - f_c)e^{-kt} \tag{7}$$

where

- f = infiltration rate at some time t after the start of rainfall, inches per hour
- f_o = initial infiltration rate, inches per hour
- f_c = final constant infiltration rate, inches per hour
- k = a shape (decay) factor (=2)
- e = base of natural logs

In using the HNV-SBUH Method to compute hydrographs, the Standard Infiltration Curves for each of the four general Soil Conservation Service (SCS) groups (A, B, C and D) shown on Figure 6 as originally established by Terstriep and Stall and as modified by Golding,[3] are used to compute infiltration. Standard Antecedent Moisture Conditions for these four types of soils, as also established by Terstriep and Stall and which are subsequently

(3) Golding, B. L., Personal communication with M. L. Terstriep, January 18, 1978.

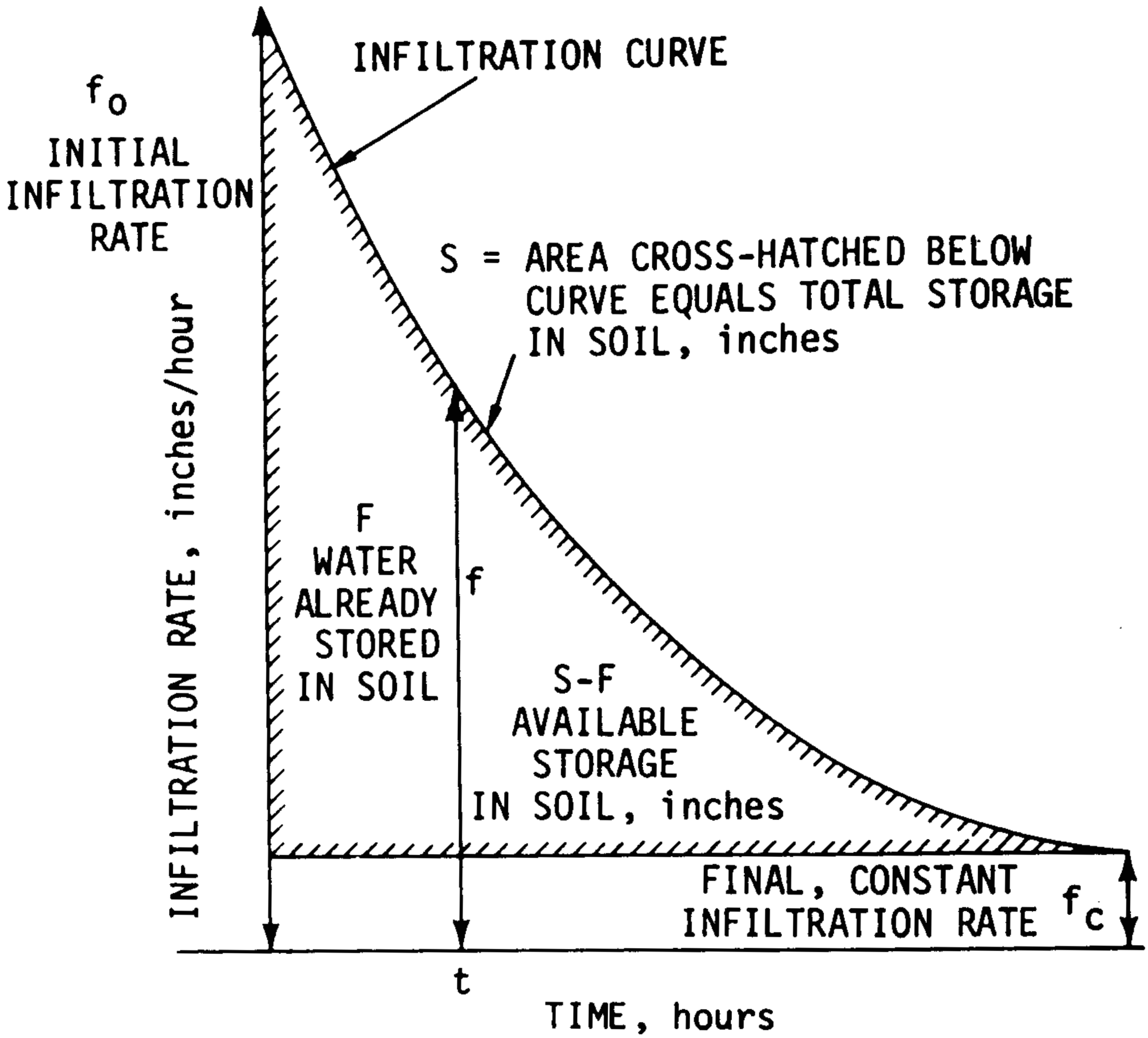


Figure 5 Diagram of infiltration curve and infiltration rates as related to storage in soil

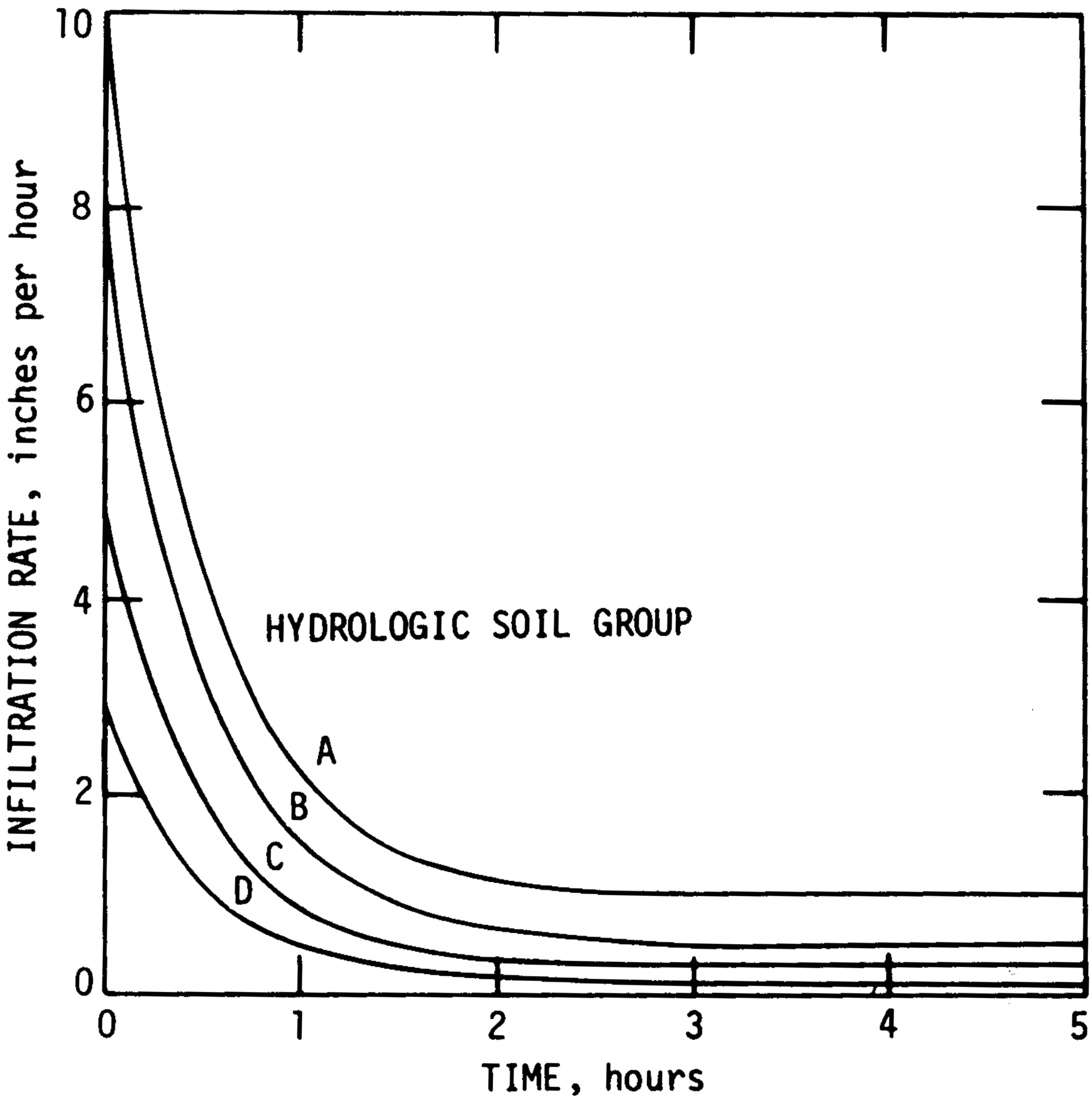


Figure 6 Standard infiltration curves for bluegrass turf

discussed, can be used to determine F. The various factors used to compute the Standard Infiltration Curves shown on Figure 6 for each of the four SCS Standard Hydrologic Soil Groups are listed on Table 6.

The four antecedent moisture conditions listed in this table - Bone Dry (Condition 1), Rather Dry (Condition 2), Rather Wet (Condition 3) and Saturated (Condition 4), are dependent on the total rainfall that occurred during the five days preceding the particular storm (Antecedent Moisture Condition) as shown on Table 7.

In using the HNV-SBUH Method to compute hydrographs, the infiltration f, at some particular value of F, is first computed for the particular Antecedent Moisture condition by first summing up incremental volumes under the curve for increments of 0.01 hour, computing a second value of infiltration f_2 at incremental time Δt later and then averaging the two computed infiltration values. To compute rainfall-excess from the pervious areas, the average infiltration over the Δt time period is then normally applied to that rainfall-excess increment during which time element a total rainfall of 0.1 inch has fallen (depression storage.

Theoretically, the initial depression storage should be subtracted from the rainfall and the infiltration curve applied only at that subsequent time after rainfall has started, at which time the total volume of rainfall and the total volume of infiltration storage under the infiltration curve are equal. However, in using the HNV-SBUH Method to compute hydrographs, the infiltration curve is applied immediately after rainfall begins and the depression storage has been satisfied. Therefore, a storm with small initial amounts of rainfall

TABLE 6

FACTORS USED FOR CALCULATING THE STANDARD INFILTRATION CURVES FOR GRASSED AREAS

Item	Value			
Hydrologic soil group USDA designation	A	B	C	D
Final constant infiltration rate, f_c, in. per hour	1.0	0.50	0.25	0.10
Initial infiltration rate, f_o, inches per hour	10	8	5	3
Shape factor, k, or infiltration curve	2	2	2	2
Available storage capacity, S, in soil mantle,				
inches, for four antecedent conditions				
Bone dry, Condition 1	4.3	3.4	2.3	1.3
Rather dry, Condition 2	3.4	2.8	1.8	1.1
Rather wet, Condition 3	1.9	1.4	1.0	0.6
Saturated, Condition 4	0	0	0	0
Infiltration accumulated, F, in soil mantle,				
inches, at start of rainfall				
Bone dry, Condition 1	0	0	0	0
Rather dry, Condition 2	1.9	1.4	1.0	0.6
Rather wet, Condition 3	3.4	2.8	1.8	1.1
Saturated, Condition 4	4.3	3.4	2.3	1.3

TABLE 7

ANTECEDENT MOISTURE CONDITIONS FOR PERVIOUS AREAS (GRASS)

Condition	Description	Total Rainfall During 5 Days Preceding Storm (inches)
1	Bone Dry	0
2	Rather Dry	0 to 0.5
3	Rather Wet	0.5 to 1
4	Saturated	Over 1

falling over an extended period of time will cause a premature reduction of the infiltration rate (a decay of the infiltration rate with time), which will result in higher runoff than would actually be the case. However, since the infiltration accumulated in the soil mantle due to antecedent rainfall (=F) is very broadly defined and since the pervious area contribution is generally small, this induced error is normally not significant.

As previously stated, normally 0.1 inch depression storage is subtracted from the initial rainfall increments before the average infiltration rate is applied thereto. In this particular drainage basin, the initial depression storage was increased to 0.5 inch to account for the large amount of storage in the adjacent roadway swales.

Design Flood Hydrographs:

The Design Flood Hydrographs of runoff from the various subbasins in this report (resulting from the various design rainfalls falling on the subbasins) are shown on Figures 1 through 8 in Section 1 of the Appendix to this report.

SECTION IV

FLOOD ROUTING

Lake Flood Routing:

As previously mentioned, the effect of reservoir storage in the lakes for both the existing and proposed future conditions was determined by flood routing using the "Modified Puls Method" which flood routing was performed by use of a computer program.

The computer program for the "Modified Puls Method" used in the report was based on the following equations:

$$I - 0 = \Delta S \qquad (1)$$

where

I = volume of inflow into the reservoir for a given time interval

0 = volume of outflow ourt of the reservoir for that time interval

S = change in the volume of storage for that time interval

This equation is also written as:

$$\frac{(I_1+I_2)}{2}\Delta t - \frac{(0_1+0_2)}{2}\Delta t = S_2-S_1=\Delta S \qquad (2)$$

where

Δt = time interval ($\Delta t = t_2-t_1$)

I_1 = the inflow at time 1 (rate)

I_2 = the inflow at time 2

0_1 = the outflow at time 1

0_2 = the outflow at time 2

S_1 = the storage at time 1 (volume)

S_2 = the storage at time 2

ΔS = the change in volume of storage for the time interval.

For the purposes of this report, the preceding equation was rewritten in the following :

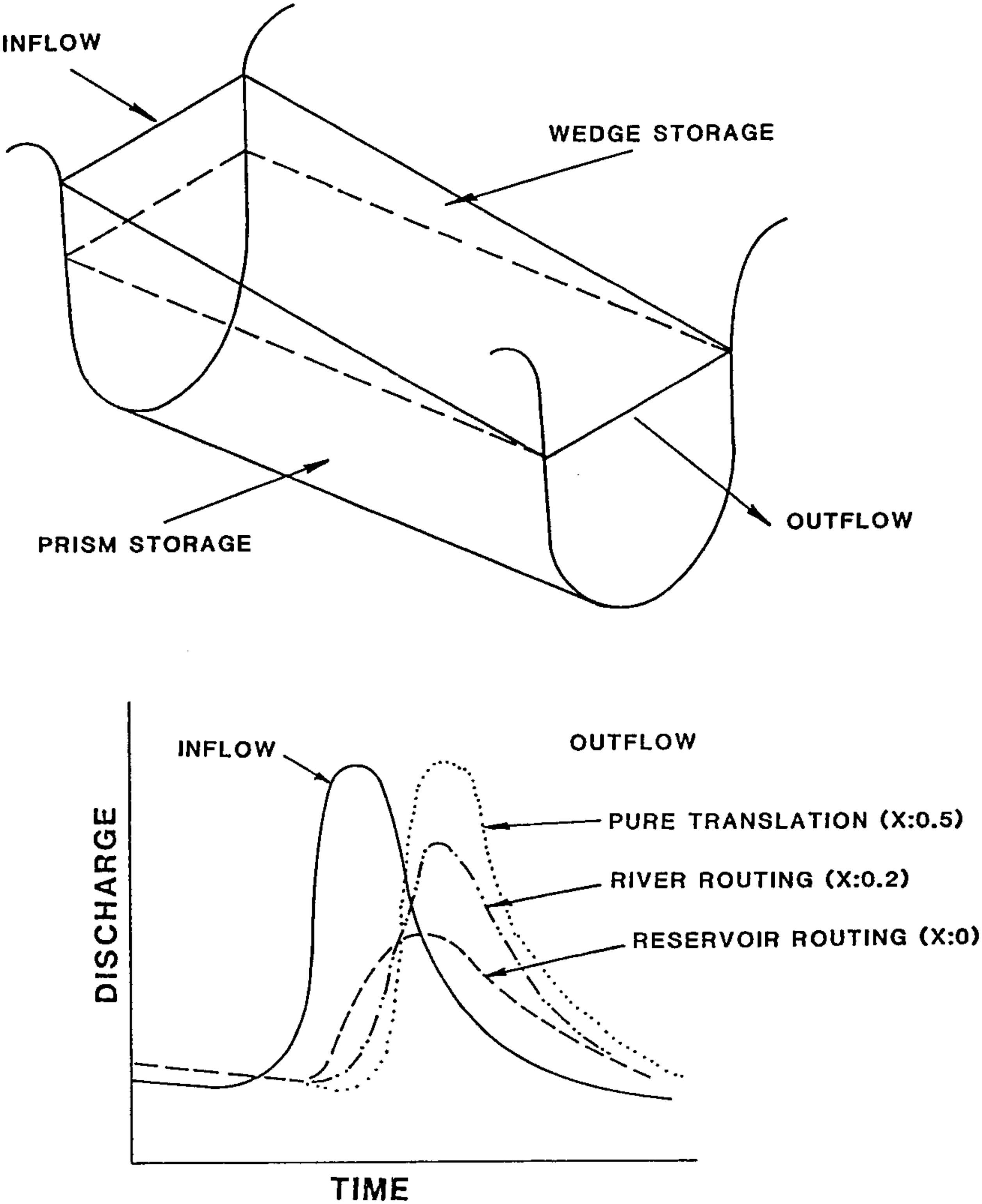
$$\frac{I_1 + I_2}{2} + \frac{S_1}{\Delta t} - \frac{O_1}{2} = \frac{S_2}{\Delta t} + \frac{O_2}{2} \qquad (3)$$

In routing the Design Flood through reservoir storage in a lake by this procedure, the storage in the lake is assumed directly propotional to the outflow form the lake. Therefore, knowing the relationship between the water surface elevations (heads) and discharge for the particular outlet pipe (the spillway rating curve) and the elevation-storage curve for the lake, it was then possible to plot a curve showing the relationship between outflow through the spillway and the storage indication factor $\frac{S}{\Delta t} + \frac{O}{2}$ which is the storage indication working curve for the lake. The storage-indication working curves made flood routing through the lakes directly solveable as all known terms of Equation (3) are on the left side of the equation and all unknown terms on the right.

Channel Flood Routing:

Flood routing through channel storage in the various reaches of the Outlet Canal was accomplished by the Muskingum Method.

In this method, wedge and prism storage are used to relate outlfow to storage and inflow (see Figure 7). During the advance of the flood wave inflow always exceeds outflow thus producing the wedge storage shown in Figure 7. Conversely, during the recession of the flood wave, outflow exceeds inflow resulting



Schematic of Muskingum Routing Technique

FIGURE 7

in negative wedge storage. The wedge storage is represented by KX(I-O) and the prism storage by KO; therefore, the total storage S is equal to KO plus KX(I-O).

$$S = KO+KX(I-O) \tag{6}$$

where

S = storage in the channel reach

I = inflow to the channel reach

O = outflow from the channel reach

K = discharge-storage proportionality factor

X = Discharge weighting factor (storage parameter)

Combining the above equation with the storage-continuity equation results in the traditional Muskingum flood routing equation.

$$O_2 = C_0I_2+C_1I_1+C_2O_1 \qquad \text{CO} \tag{7}$$

where

$$C_0 = \frac{0.5T-KX}{K-KX+0.5T} \tag{8}$$

$$C_1 = \frac{KX+0.5T}{K-KX+0.5T} \tag{9}$$

$$C_2 = \frac{K-KX-0.5T}{K-KX+0.5T} \tag{10}$$

and where

$$C_0+C_1+C_2=1 \tag{11}$$

Equation (11) must be equal to 1.0 if the required steady state flow condition exists.

In the Muskingum Method, K, the discharge storage portionality factor, has the dimension of time and is generally assumed to be equal to the flow travel time in the reach which was determined by dividing the reach length by the velocity in that reach of the Outlet Canal. Velocities in all cases were determined by water surface profile computation. For both the existing and proposed coditions, a discharge weighting factor (X) of 0.3 was used.

SECTION V

WATER ACCOUNTING

In order to ascertain if the assumed amount of water that is expected to infiltrate into pervious areas of the various subbasins during the Design Rainfall actually does so, an accounting of the available storage in the soil voids and the water assumed to infiltrate therein was made for Subbasin No. 3/4. This analysis is required to determine if there is enough storage in the soil in order to accommodate the water assumed infiltrating therein during the 25 Year Frequency-24 Hour Duration Rainfall such that 100% runoff would not occur This water accounting, summarized on Table 8 below, showed that there is just sufficient storage to accommodate the assumed total infiltration.

TABLE 8

BASIN NO.	SOIL GROUP	CONSTANT	STORAGE REQUIRED (INFILTRATION) (INCHES)	AVAILABLE STORAGE (INCHES)
3/4	C	1.08	6.22	6.0"

The computation of the total volume of rainfall contributing to storm water runoff and infiltration on the pervious surfaces during the 25 Year Frequency-24 Hour Duration Rainfall event was made by multiplying the incremental rainfalls by a constant to account for the storm water flowing to the pervious areas from the nondirectly connected impervious areas. The constant used in this analysis is as follows:

$$C = \frac{1 - I}{I - I_t}$$

where

I = Directly connected impervious area (decimal)

I_t = Total impervious area (decimal)

The constant as computed for Subbasin 3/4 is given on the above table.

The total amount of water entering the groundwater table from the pervious portion of the subbasin and the total runoff from the previous portions are computed on Table 6 for Subbasin 3/4. The procedure as illustrated in this table uses the previously described Horton infiltration equation to determine the infiltration - which infiltration rate varies with time. As can be observed from Table 8, runoff (rainfall-excess) only occurs from the pervious areas when the incremental rainfall increment (increased by the above derived constant) is greater than the infiltration rate. As can be observed from Table 9 (Subbasin 3/4, Soil Group C), runoff from the pervious portion of this subbasin only occurred during the 12 hour period (Hour 7.5 through Hour 19.0).

As summarized on the bottom of Table 9, the total amount of water entering the groundwater table during the design storm in Subbasin 3/4 amounts to 5.12 inches.

However, the above analysis, which computes volumes, in inches, for a unit area of pervious land, does not consider the available water holding capacity of the soil profile located under the impervious land segments. This additional storage volume is generally ignored in storm water runoff computations, as is the case with this study. Since this is a significant and viable storage volume, the water holding capacity of the soil profile under the impervious areas can be considered to increase the water holding capacity of the pervious land segment soil profile.

TABLE 9

Computation of Inflow to G.W.T. and Runoff, Basin 3/4[1] Soil Type C

Condition 3 - Rather Wet

$f_o = 500,\ f_c = 0.25,\ F = 1.80$

Time (Hrs)	ΔP (In)	ΔP_1 (In)	Inf. Rate (In/0.5 hr)	Water to G.W.T. (In)	Runoff (In)
0					
0.5	0.07	0.08[2]	--	--	0
1.0	0.07	0.08[2]	--	--	0
1.5	0.07	0.08[2]	--	--	0
2.0	0.07	0.08[2]	--	--	0
2.5	0.07	0.08[2]	--	--	0
3.0	0.07	0.08[2]	--	--	0
3.5	0.07	0.02[2]			
		0.06	0.50	0.06	0
4.0	0.07	0.08	0.26	0.08	0
4.5	0.10	0.11	0.18	0.11	0
5.0	0.10	0.11	0.14	0.11	0
5.5	0.12	0.13	0.13	0.13	0
6.0	0.12	0.13	0.13	0.13	0
6.5	0.12	0.13	0.13	0.13	0
7.0	0.12	0.13	0.13	0.13	0
7.5	0.16	0.17	0.13	0.13	0.04
8.0	0.16	0.17	0.13	0.13	0.04
8.5	0.16	0.17	0.13	0.13	0.04
9.0	0.17	0.18	0.13	0.13	0.05
9.5	0.17	0.18	0.13	0.13	0.05
10.0	0.17	0.18	0.13	0.13	0.05
10.5	0.17	0.18	0.13	0.13	0.05
11.0	0.17	0.18	0.13	0.13	0.05
11.5	0.19	0.21	0.13	0.13	0.08
12.0	0.19	0.21	0.13	0.13	0.08

(1) Basin Area = 56.9 acres, I_t = 17%, I = 10%

(2) 1st 0.5" to depression storage, C = 1.08
$\Delta P_1 = \Delta P \times C$

TABLE 9 - Cont'd

Computation of Inflow to G.W.T. and Runoff, Basin 3/4[1] Soil Type C

Condition 3 - Rather Wet

$f_o = 500$, $f_c = 0.25$, $F = 1.80$

Time (Hrs)	ΔP (In)	ΔP_1 (In)	Inf. Rate (In/0.5 hr)	Water to G.W.T. (In)	Runoff (In)
12.5	0.23	0.25	0.13	0.13	0.12
13.0	0.24	0.26	0.13	0.13	0.13
13.5	0.35	0.38	0.13	0.13	0.25
14.0	0.36	0.39	0.13	0.13	0.26
14.5	0.55	0.60	0.13	0.13	0.47
15.0	0.55	0.60	0.13	0.17	0.47
15.5	0.88	0.95	0.13	0.13	0.82
16.0	0.97	1.05	0.13	0.13	0.92
16.5	0.41	0.44	0.13	0.13	0.31
17.0	0.40	0.43	0.13	0.13	0.30
17.5	0.22	0.24	0.13	0.13	0.11
18.0	0.21	0.23	0.13	0.13	0.10
18.5	0.14	0.15	0.13	0.13	0.02
19.0	0.14	0.15	0.13	0.13	0.02
19.5	0.12	0.13	0.13	0.13	0
20.0	0.12	0.13	0.13	0.13	0
20.5	0.10	0.13	0.13	0.11	0
21.0	0.10	0.11	0.13	0.11	0
21.5	0.10	0.11	0.13	0.11	0
22.0	0.10	0.11	0.13	0.11	0
22.5	0.09	0.10	0.13	0.10	0
23.0	0.09	0.10	0.13	0.10	0
23.5	0.09	0.10	0.13	0.10	0
24.0	0.09	0.10	0.13	0.10	0
		Totals	=	5.10"	4.83"

Accordingly, for a total percent impervious area of 17% for Subbasin 3/4, and assuming that 75% of the water holding capacity of the soil profile under these impervious areas is available for the storage of storm water, the computed volume of storm water stored in the soil profile of a pervious land segment may by reduced by the following constant:

$$\frac{(1-I_t)}{(1-I_t) + 0.75\ I_t}$$

The constant for Basin 3/4 was computed to be 0.87.

Therefore the total volume of storm water (infiltration) required to be stored in the soil profile of the subbasin from the 25 Year Frequencey-24 Hour Duration Rainfall is computed as follows:

5.10" x 0.87 = 4.42"

Added to this volume must be the water already assumed to be there or infiltration F in the soil profile at the start of design rainfall for the Rather Wet Condition as listed on Table 8. This addition is as follows:

4.42" + 1.80" = 6.22"

This volume is the total volume for which storage must be available in the soil voids and is listed in Table 6. It should be noted that this computation does <u>not</u> take into account the evapotranspiration (= .10 in.) that occurs during the 24 Hour Duration Rainfall period.

PART C

COST ESTIMATE AND CONCLUSIONS

SECTION I

COST ESTIMATE

The construction cost estimates presented for the improvements to the Venice Gardens Drainage System were computed in the standard manner of multiplying the various computed quantities of work by suitable unit prices for each quantity and referenced by Engineering News Record (ENR) construction cost index. The estimated costs for the construction of improvements to the Outlet Canal, proposed in this report, are summarized below:

	Item Description	Cost
1.	Clear and Grub	$ 9,000
2.	Channel Excavation	31,300
3.	Fine Grading (Channel)	3,800
4.	Sodding (Channel)	15,000
5.	36" RCP	46,400
6.	53" x 34" Oval RCP	1,800
7.	68" x 43" Oval RCP	89,600
8.	Special Manhole	2,000
9.	Concrete Slope Protection	1,700
10.	Concrete in Headwalls	8,500
11.	Dredge Excavation	57,000
12.	Pavement Removal	600
13.	Pavement Restoration	2,900
14.	Sheet Pile Retaining Walls	14,000
15.	Fine Grading (Lawns)	2,400
16.	Seeding and Mulching (Lawns)	500
17.	Relocate/Restore - Fences	5,000
18.	Relocate/Restore - Structures	10,000
19.	Dewatering	20,000
20.	Utility Relocation	9,000
21.	Property (3 lots @ $10,000 per lot)	30,000
	Subtotal Cost	$360,500

The estimated cost of constructing drainage facilities to serve the two small noncontributary aeareas are summarized below:

	Item Description	Cost
1.	Excavation	$ 5,000
2.	15" RCP	4,000
3.	Utility Relocation	1,000
4.	Property Acquisition (2 lots @ $10,000 per lot)	20,000
	Subtotal Cost	$ 30,000
	Total Cost	$390,500

The above costs do <u>not</u> include the costs of acquiring any drainage, construction or maintenance easements.

SECTION II

CONCLUSIONS

1. Runoff from the 16-acre eastern portion of the proposed Venetian Plaza Shopping Center from a 25-Year Frequency - 24 Hour Duration Rainfall will be contained within the proposed detention basin being constructed therein. Ponded runoff from this detention basin will flow westerly to the existing State Route 41 drainage system in storm sewer to be constructed as part of the shopping center drainage system and then southerly in the State Route 41 drainage system to Alligator Creek. This detention basin and outlet pipes that discharge to the State Route 41 system will actually improve the overall drainage situation as some runoff from this area did in the past ultimately contribute to the South Venice Gardens Outlet Canal from the shopping center area.

2. Accelerated home building in the 180-acre South Venice Gardens area studied in this report has eliminated many of the low vacant lots where runoff from adjacent areas in the basin was temporarily stored and also increased the amount of imperious area. This results in increased flooding of the overall area.

3. The enlarging of the outlet canal as described in this Report will, along with lowering of the lakes to provide additional water storage, eliminate flooding of the areas adjacent to the lakes and along the Outlet Canal from a 25-Year Frequency - 24 Hour Duration Rainfall.

APPENDIX

SECTION 1

SUBBASIN HYDROGRAPHS

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 1

25 YEAR FREQUENCY - 24 HOUR RAINFALL

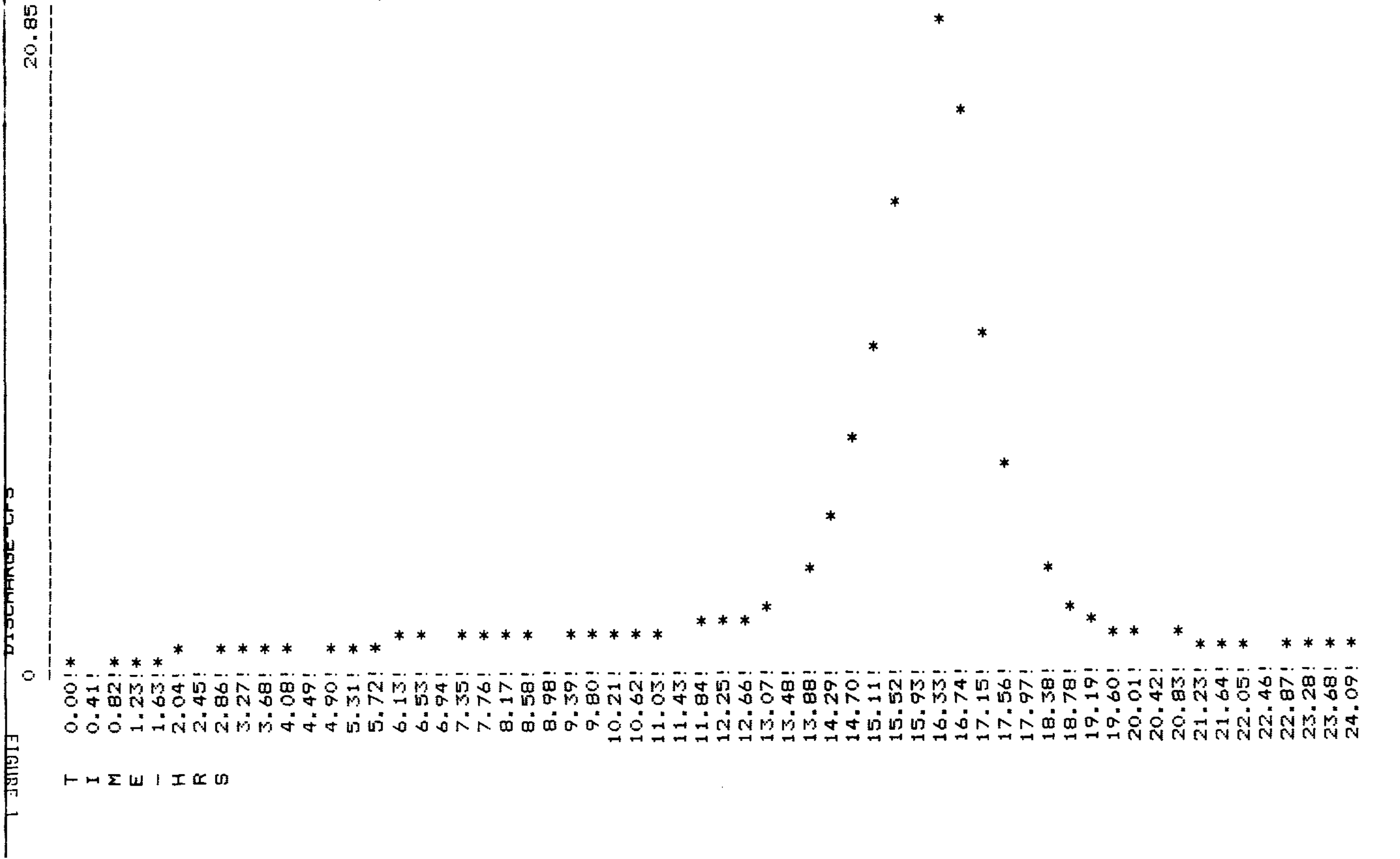


FIGURE 1

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 2

25 YEAR FREQUENCY - 24 HOUR RAINFALL

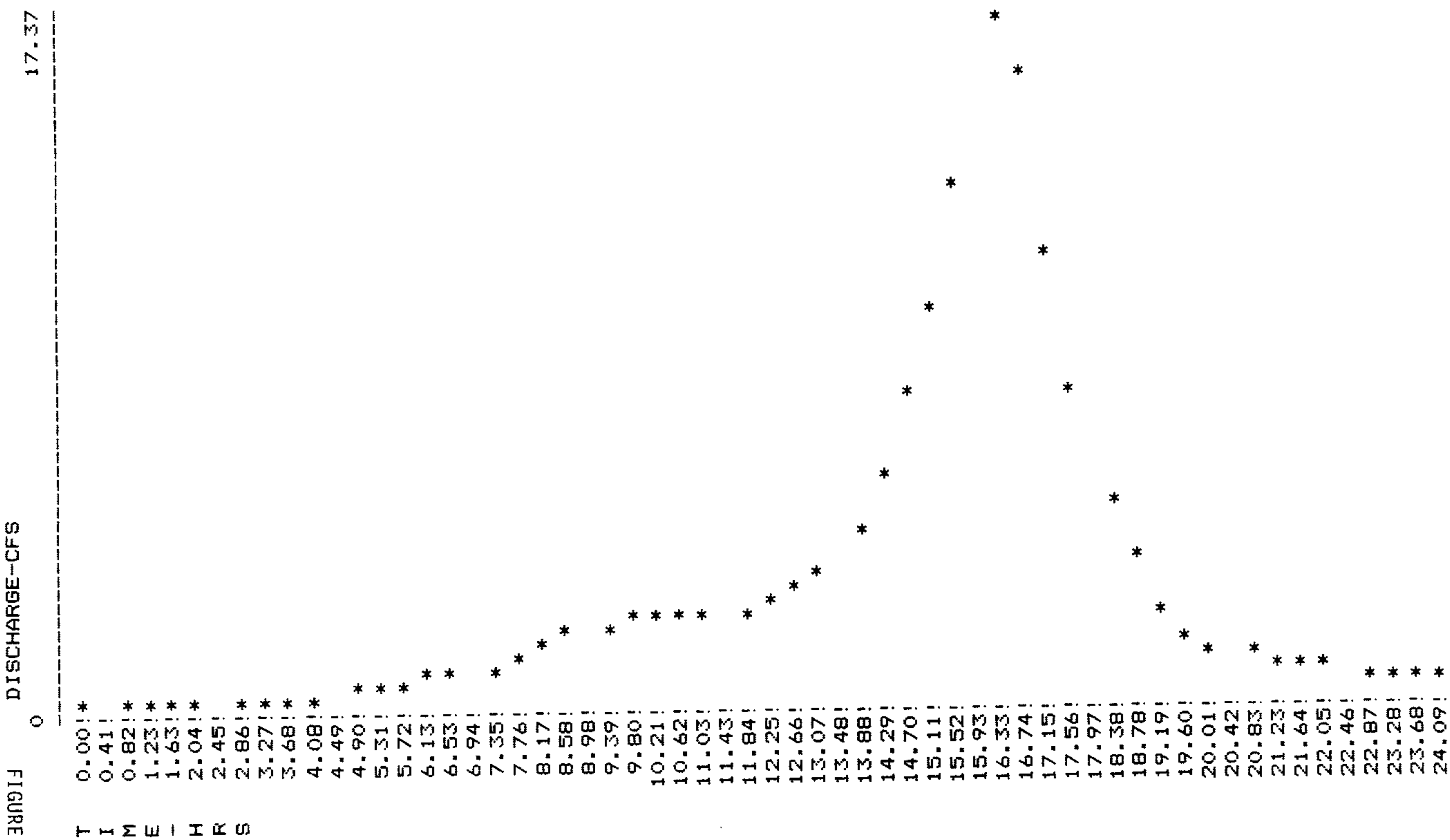


FIGURE 2

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 3/4

25 YEAR FREQUENCY - 24 HOUR RAINFALL

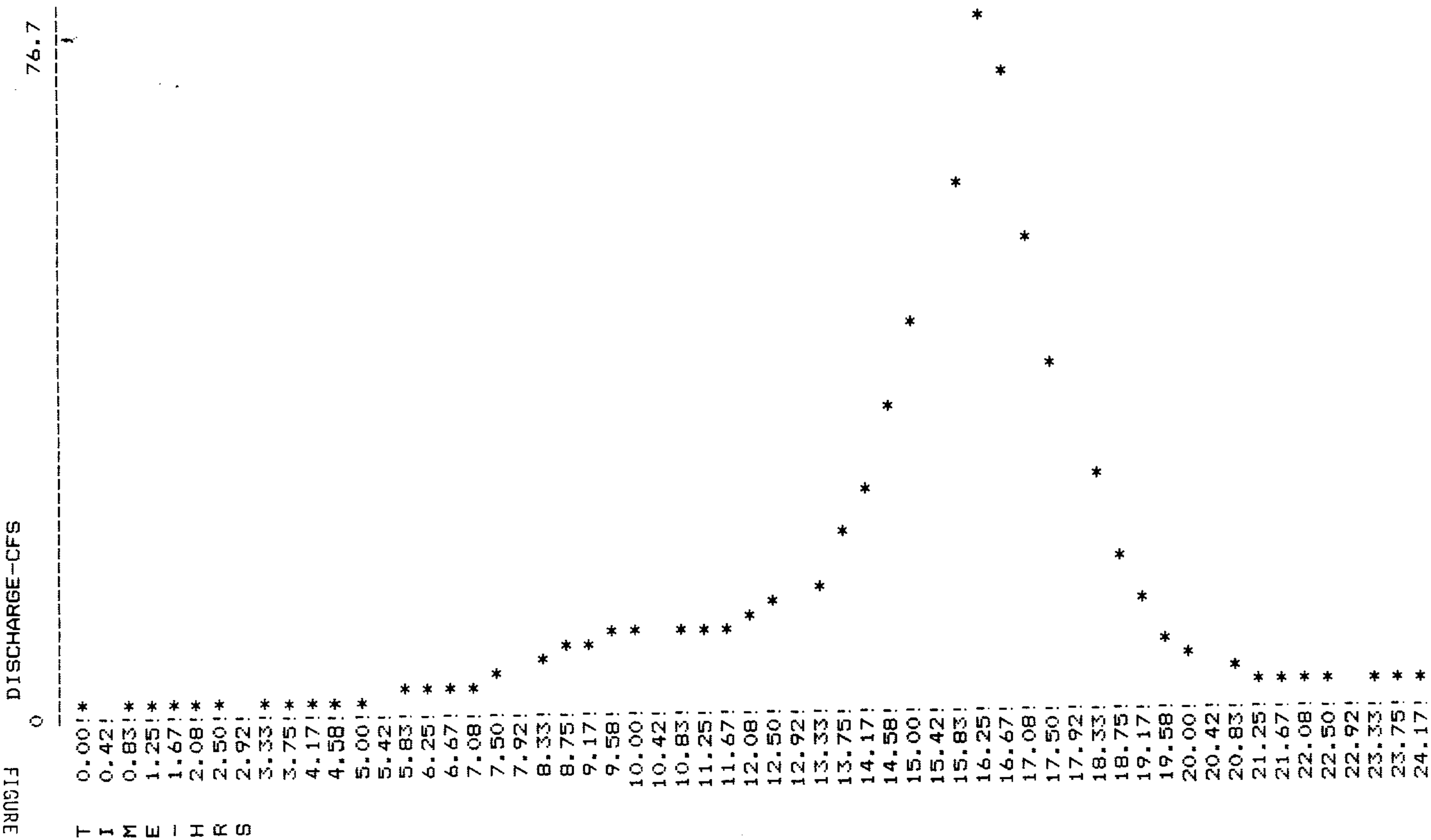
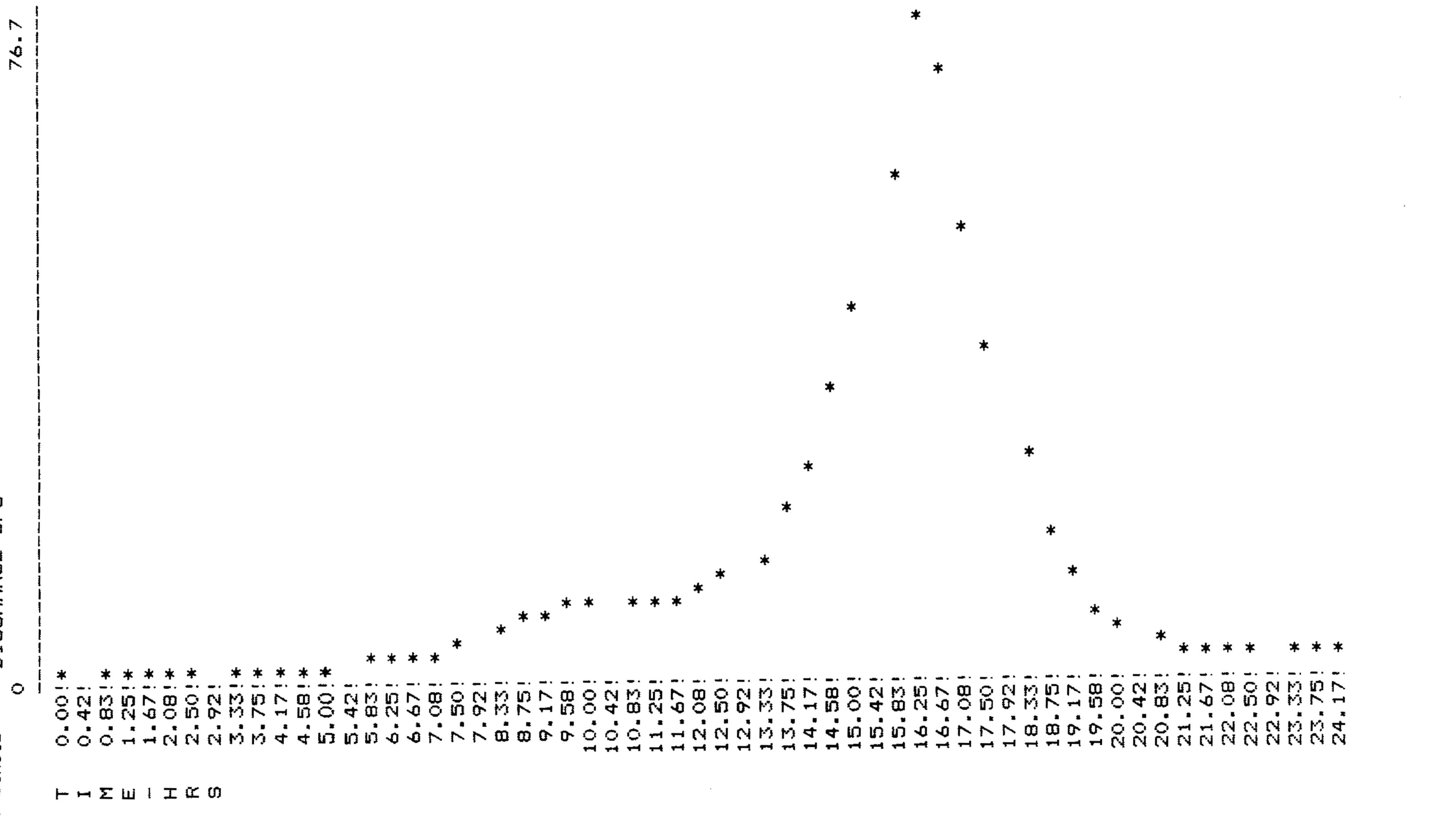


FIGURE 3

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 3/4

25 YEAR FREQUENCY - 24 HOUR RAINFALL



DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 5

25 YEAR FREQUENCY - 24 HOUR RAINFALL

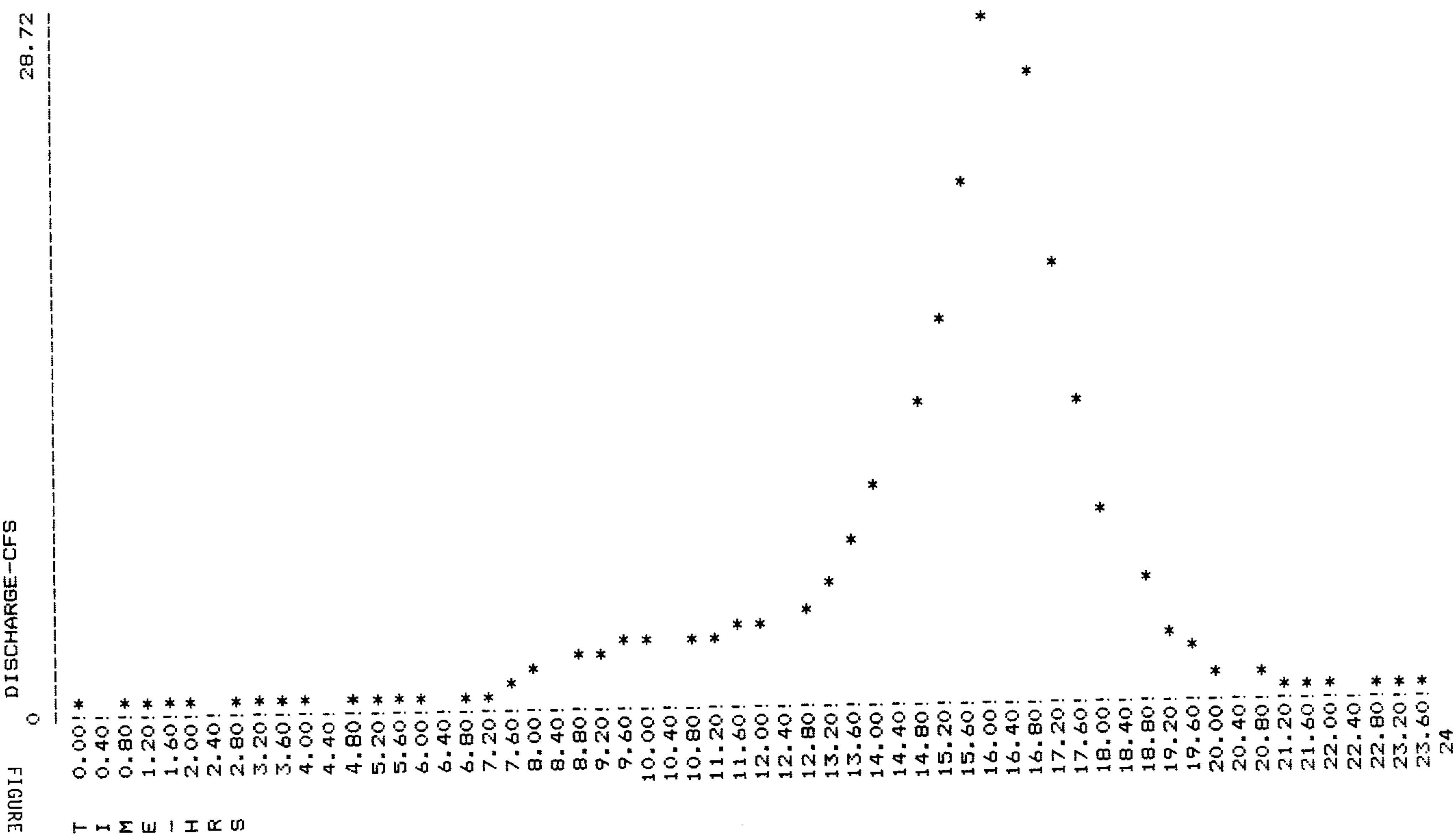


FIGURE 4

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 6

25 YEAR FREQUENCY - 24 HOUR RAINFALL

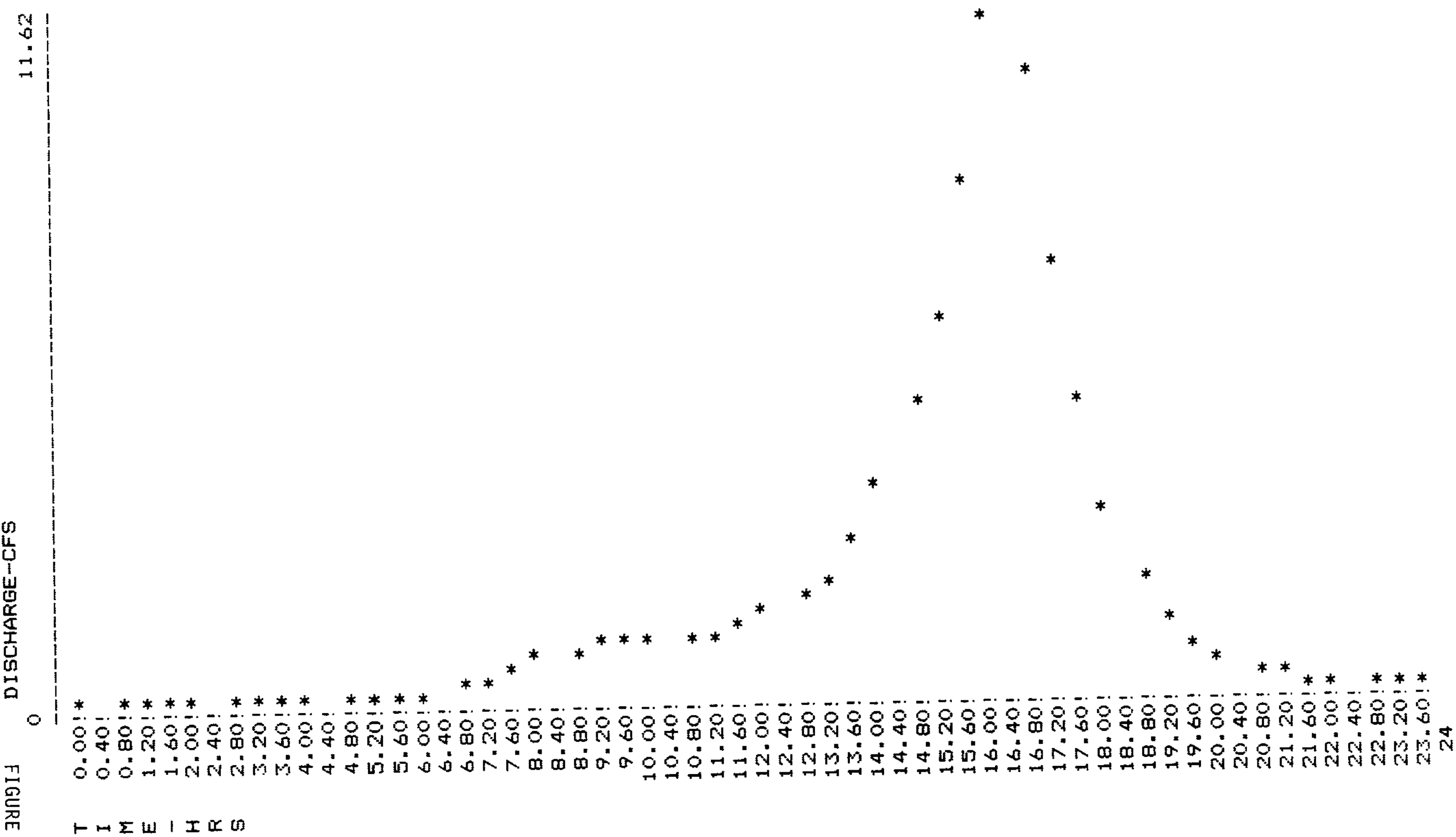


FIGURE 5

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 7

25 YEAR FREQUENCY - 24 HOUR RAINFALL

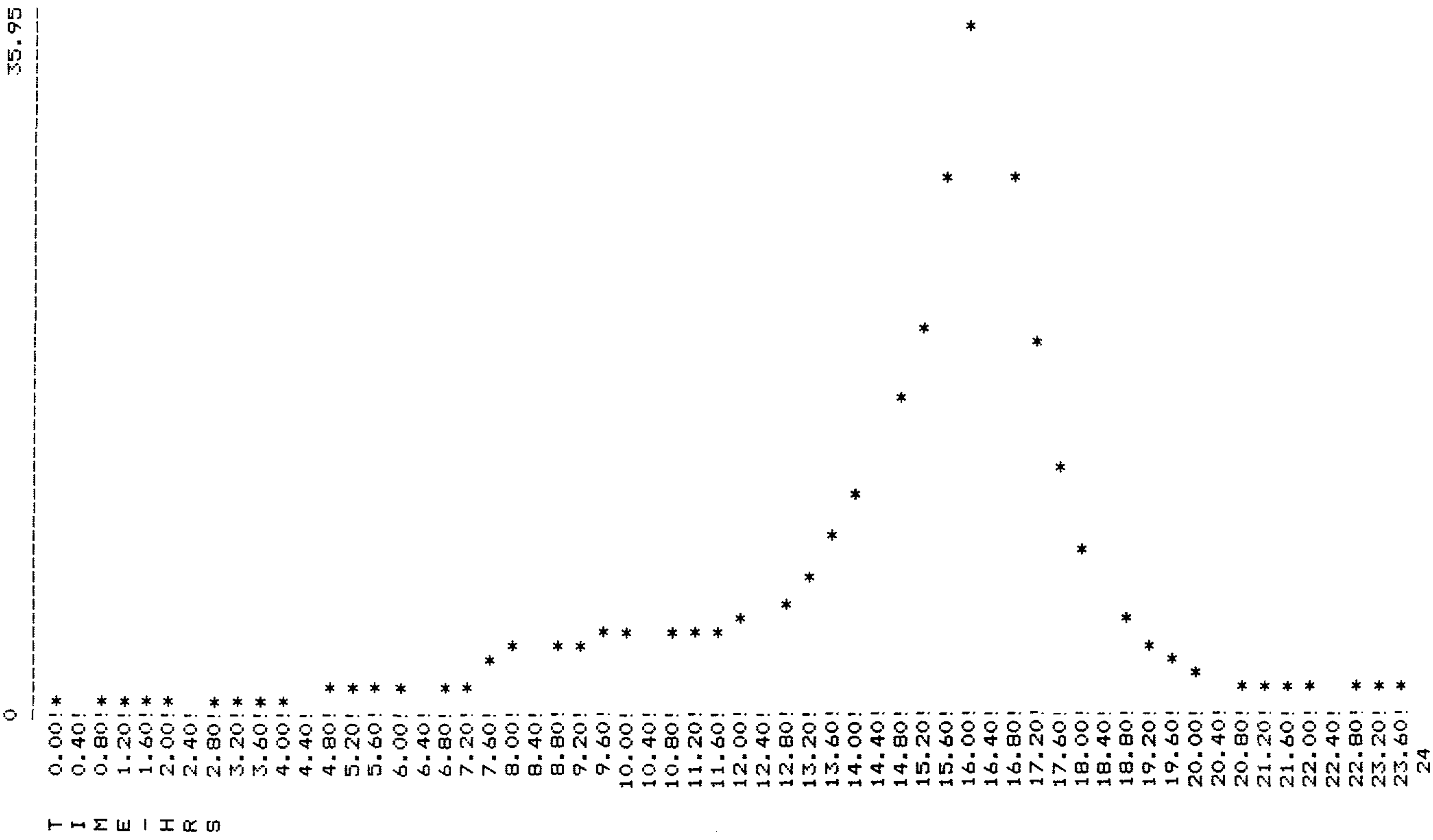


FIGURE 6

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 8

25 YEAR FREQUENCY - 24 HOUR RAINFALL

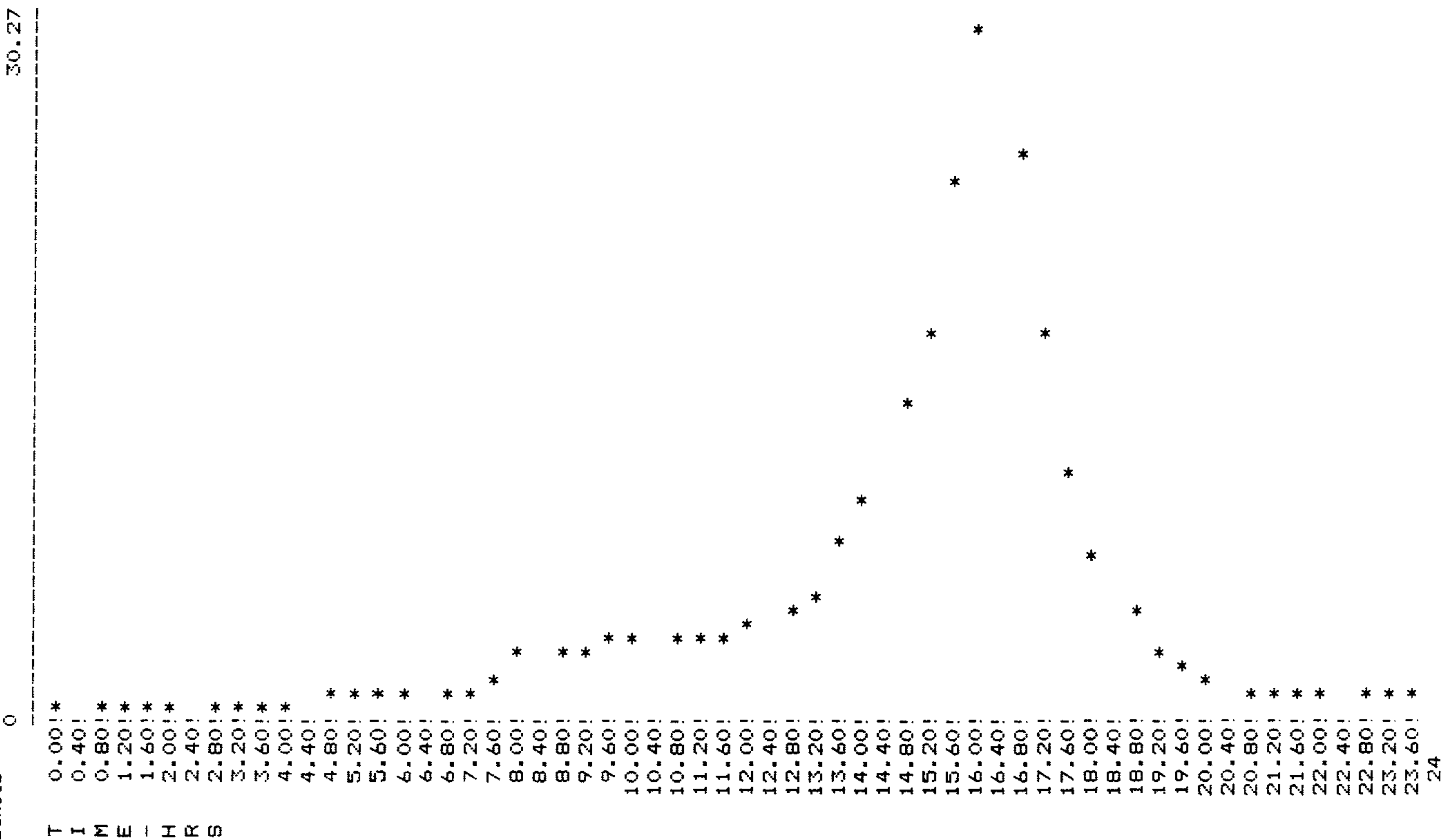


FIGURE 7

DESIGN FLOOD HYDROGRAPH

SUBBASIN NO. 9

25 YEAR FREQUENCY - 24 HOUR RAINFALL

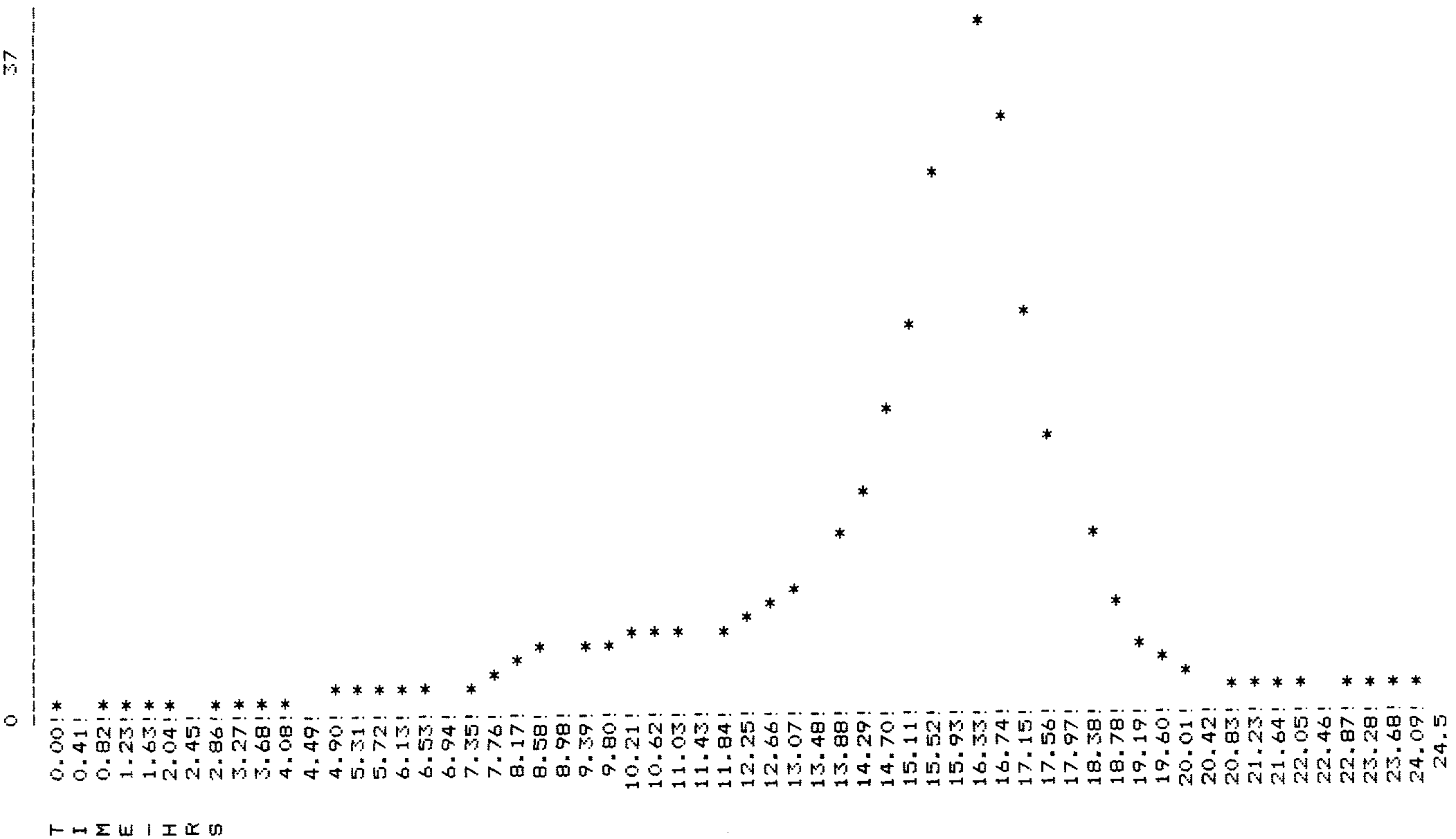


FIGURE 8

SECTION 2

DESIGN FLOOD HYDROGRAPHS

EXISTING CONDITIONS

DESIGN FLOOD HYDROGRAPH

INFLOW HYDROGRAPH TO LAKE A/B

SUBBASINS NO. 2 & 3/4

25 YEAR FREQUENCY - 24 HOUR RAINFALL

<u>EXISTING CONDITIONS</u>

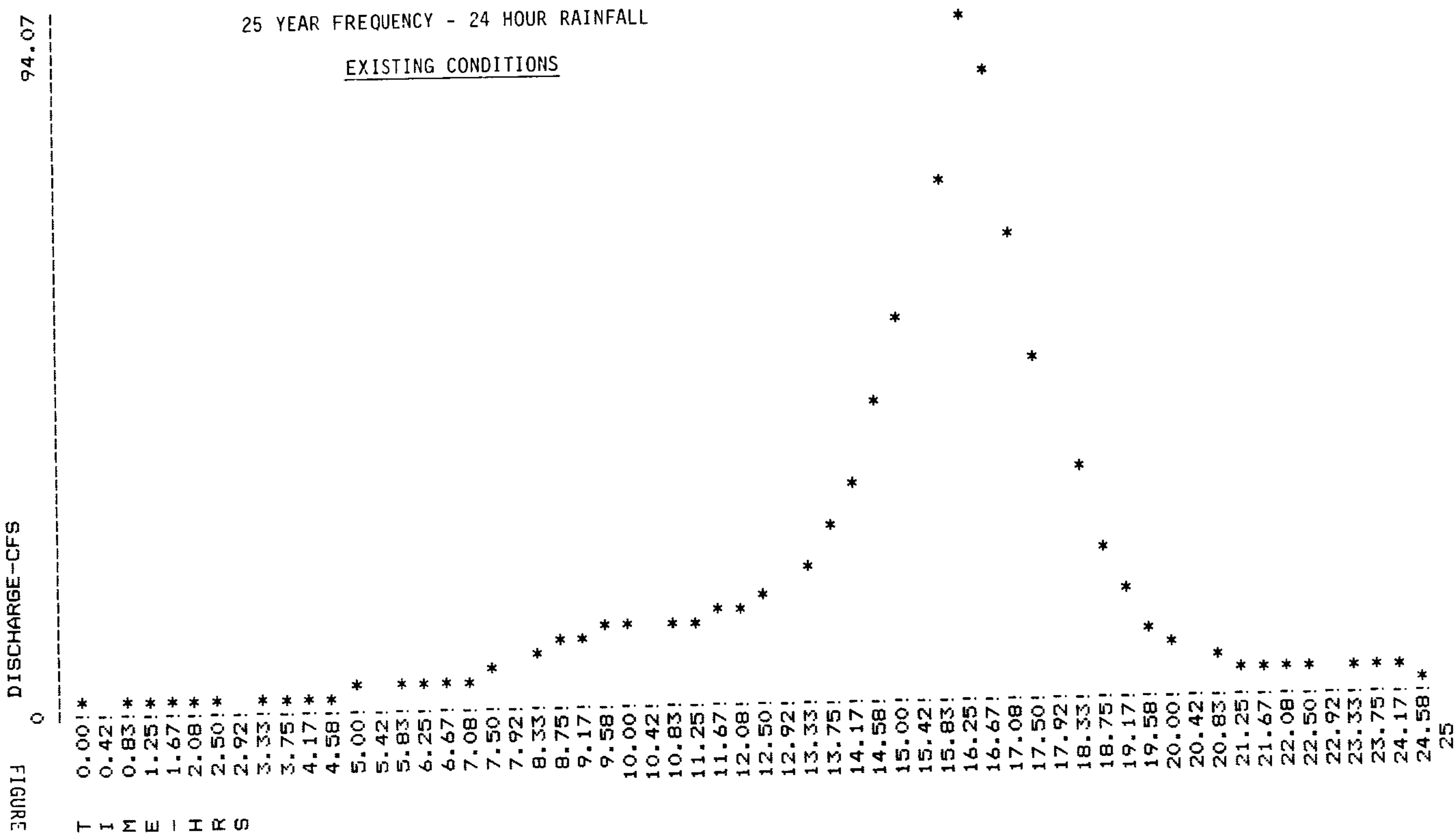


FIGURE 9

DESIGN FLOOD HYDROGRAPH

OUTFLOW HYDROGRAPH FROM LAKE A/B

SUBBASINS NO. 2 & 3/4

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

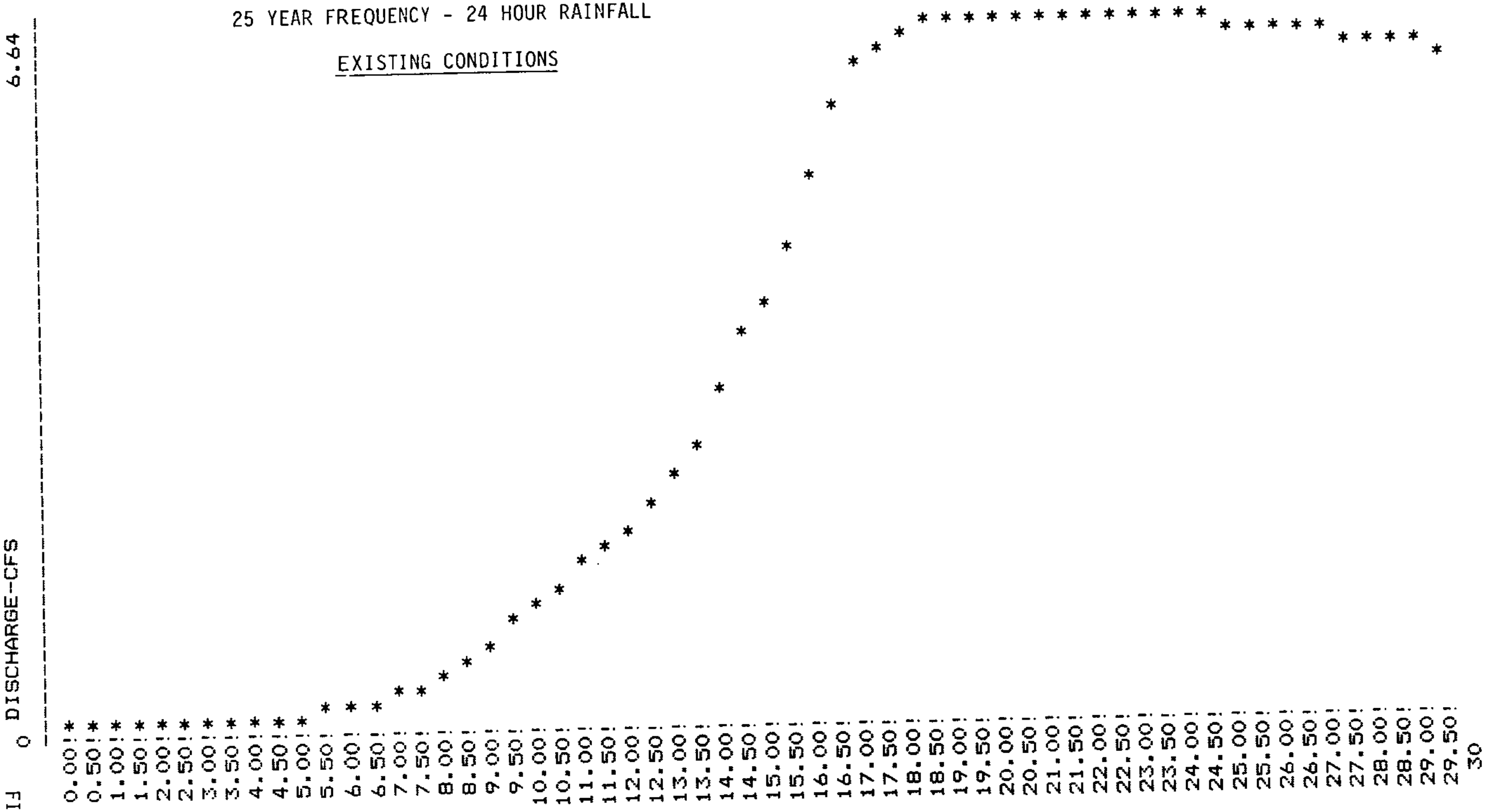


FIGURE 10

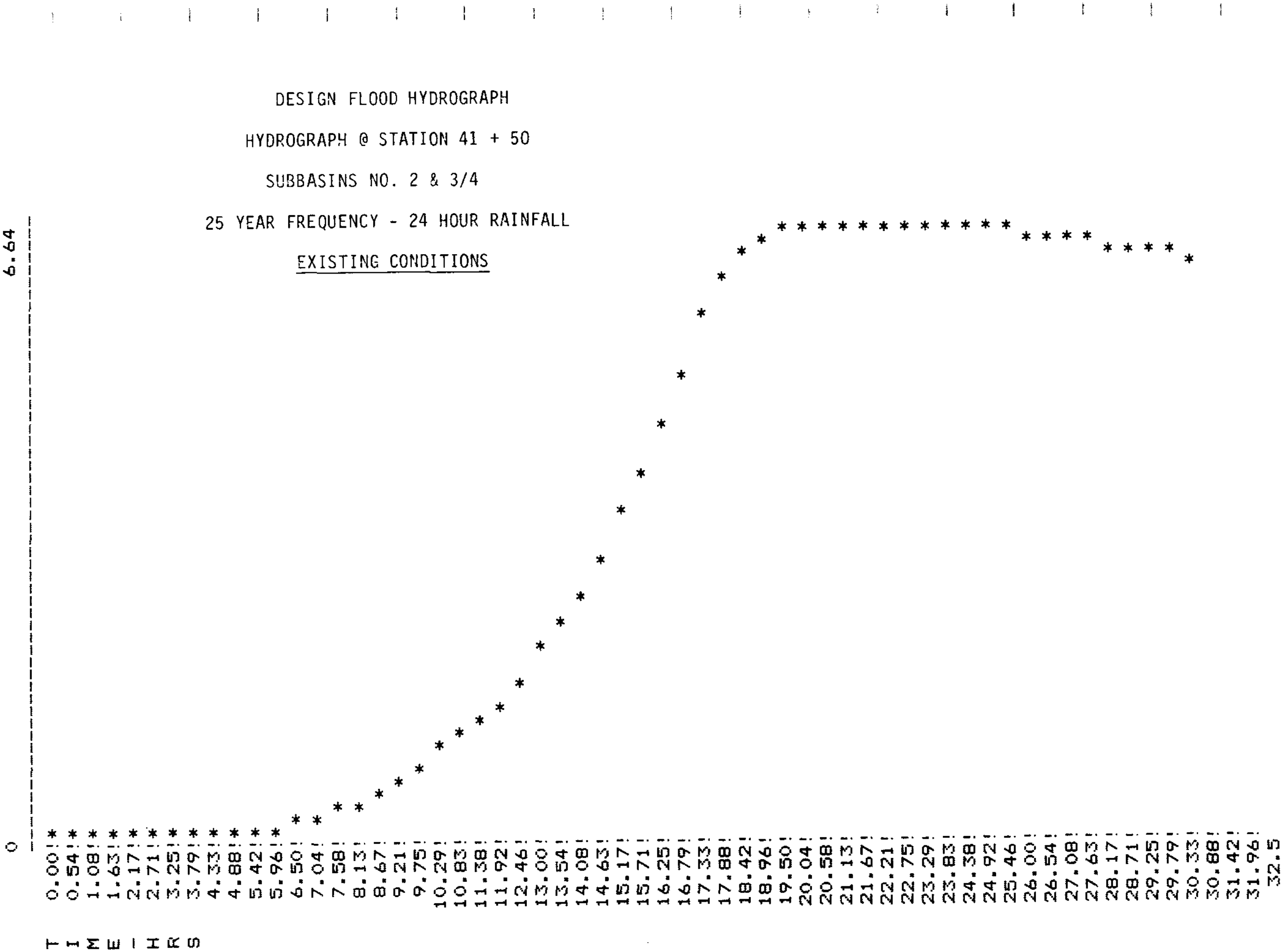

DESIGN FLOOD HYDROGRAPH
HYDROGRAPH @ STATION 41 + 50
SUBBASINS NO. 2 & 3/4
25 YEAR FREQUENCY - 24 HOUR RAINFALL
EXISTING CONDITIONS
DISCHARGE-CFS
0
6.64
TIME-HRS
0.00 0.54 1.08 1.63 2.17 2.71 3.25 3.79 4.33 4.88 5.42 5.96 6.50 7.04 7.58 8.13 8.67 9.21 9.75 10.29 10.83 11.38 11.92 12.46 13.00 13.54 14.08 14.63 15.17 15.71 16.25 16.79 17.33 17.88 18.42 18.96 19.50 20.04 20.58 21.13 21.67 22.21 22.75 23.29 23.83 24.38 24.92 25.46 26.00 26.54 27.08 27.63 28.17 28.71 29.25 29.79 30.33 30.88 31.42 31.96 32.5

FIGURE 11

DESIGN FLOOD HYDROGRAPH

OUTFLOW HYDROGRAPH FROM SUBBASIN NO. 5

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

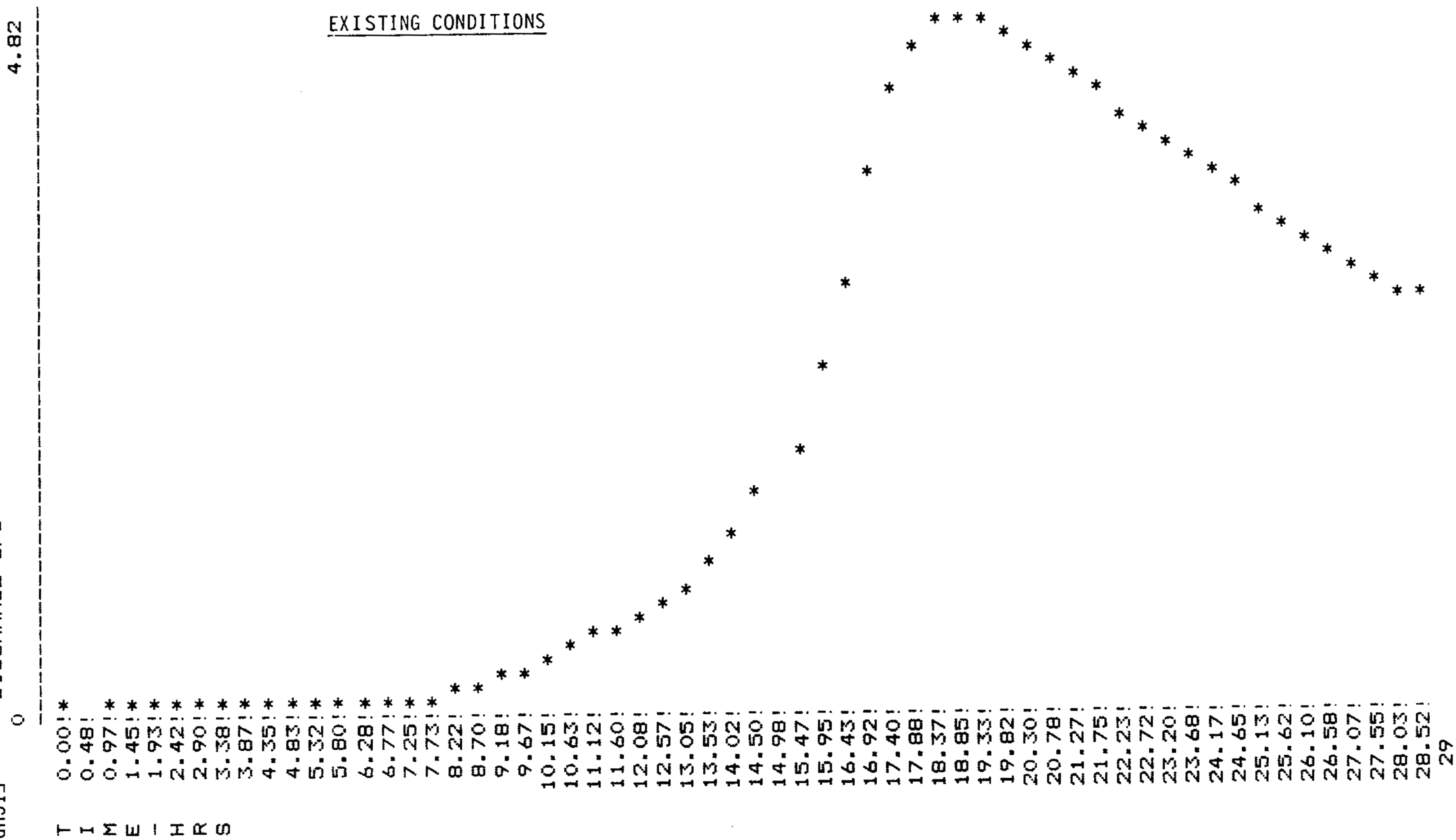


FIGURE 12

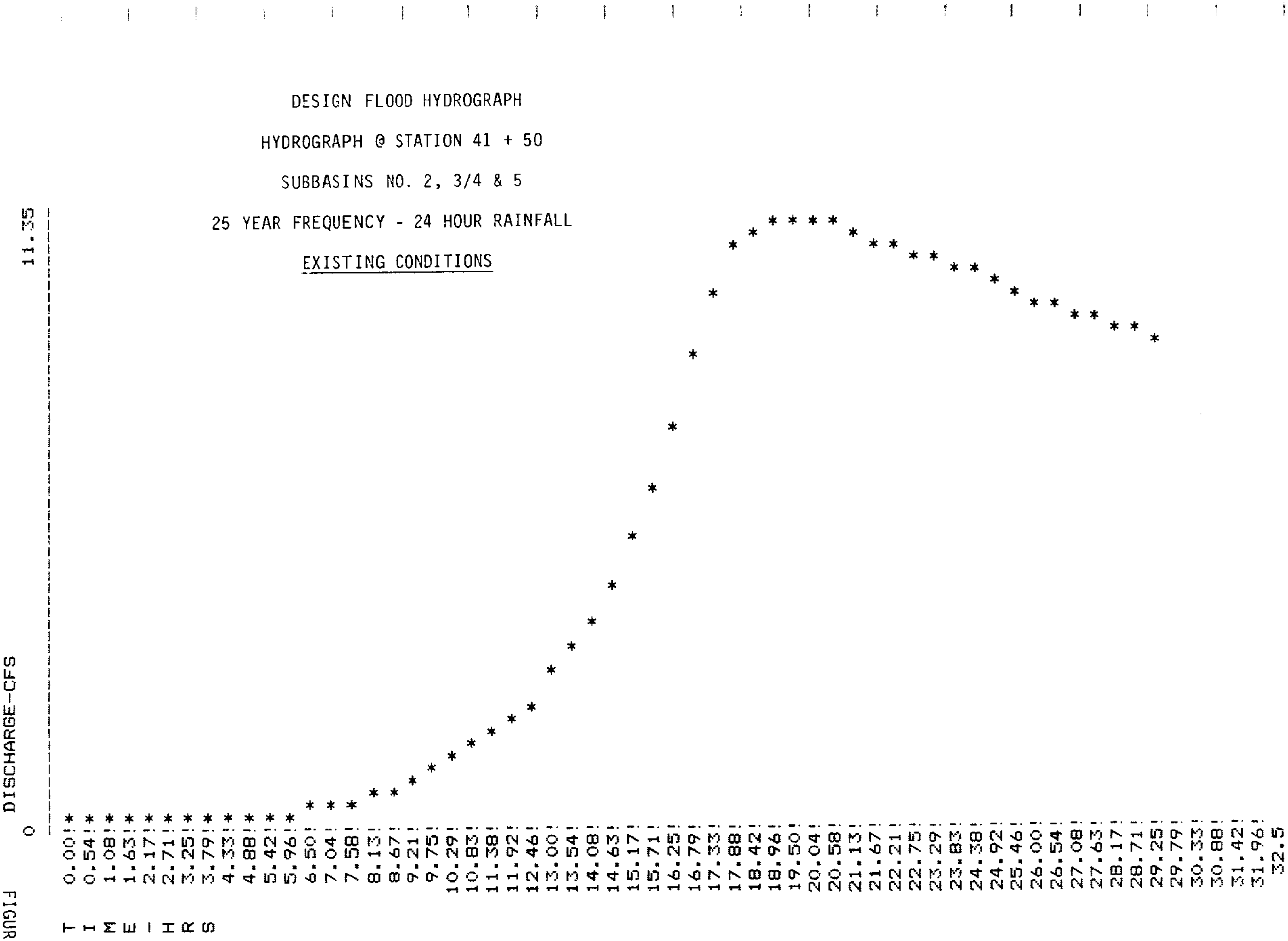

DESIGN FLOOD HYDROGRAPH
HYDROGRAPH @ STATION 41 + 50
SUBBASINS NO. 2, 3/4 & 5
25 YEAR FREQUENCY - 24 HOUR RAINFALL
EXISTING CONDITIONS
DISCHARGE-CFS
0
11.35
TIME-HRS
0.00 0.54 1.08 1.63 2.17 2.71 3.25 3.79 4.33 4.88 5.42 5.96 6.50 7.04 7.58 8.13 8.67 9.21 9.75 10.29 10.83 11.38 11.92 12.46 13.00 13.54 14.08 14.63 15.17 15.71 16.25 16.79 17.33 17.88 18.42 18.96 19.50 20.04 20.58 21.13 21.67 22.21 22.75 23.29 23.83 24.38 24.92 25.46 26.00 26.54 27.08 27.63 28.17 28.71 29.25 29.79 30.33 30.88 31.42 31.96 32.5

FIGURE 13

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 28 + 00

SUBBASINS NO. 2,3/4 & 5

25 YEAR FREQUENCY - 24 HOUR RAINFALL

<u>EXISTING CONDITIONS</u>

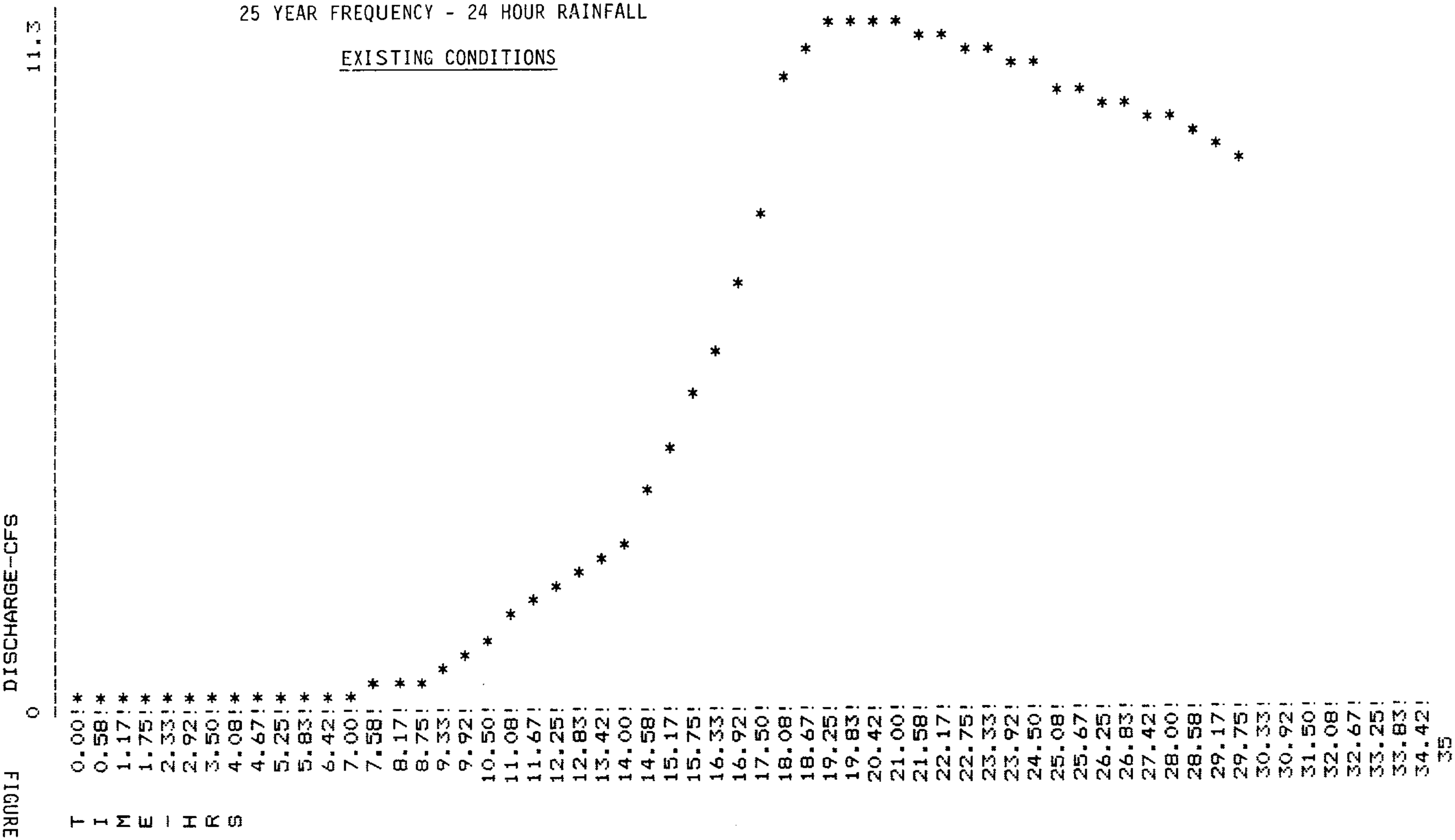


FIGURE 14

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 18 + 00 (LAKE C)

SUBBASINS NO. 2,3/4,5 & 6

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

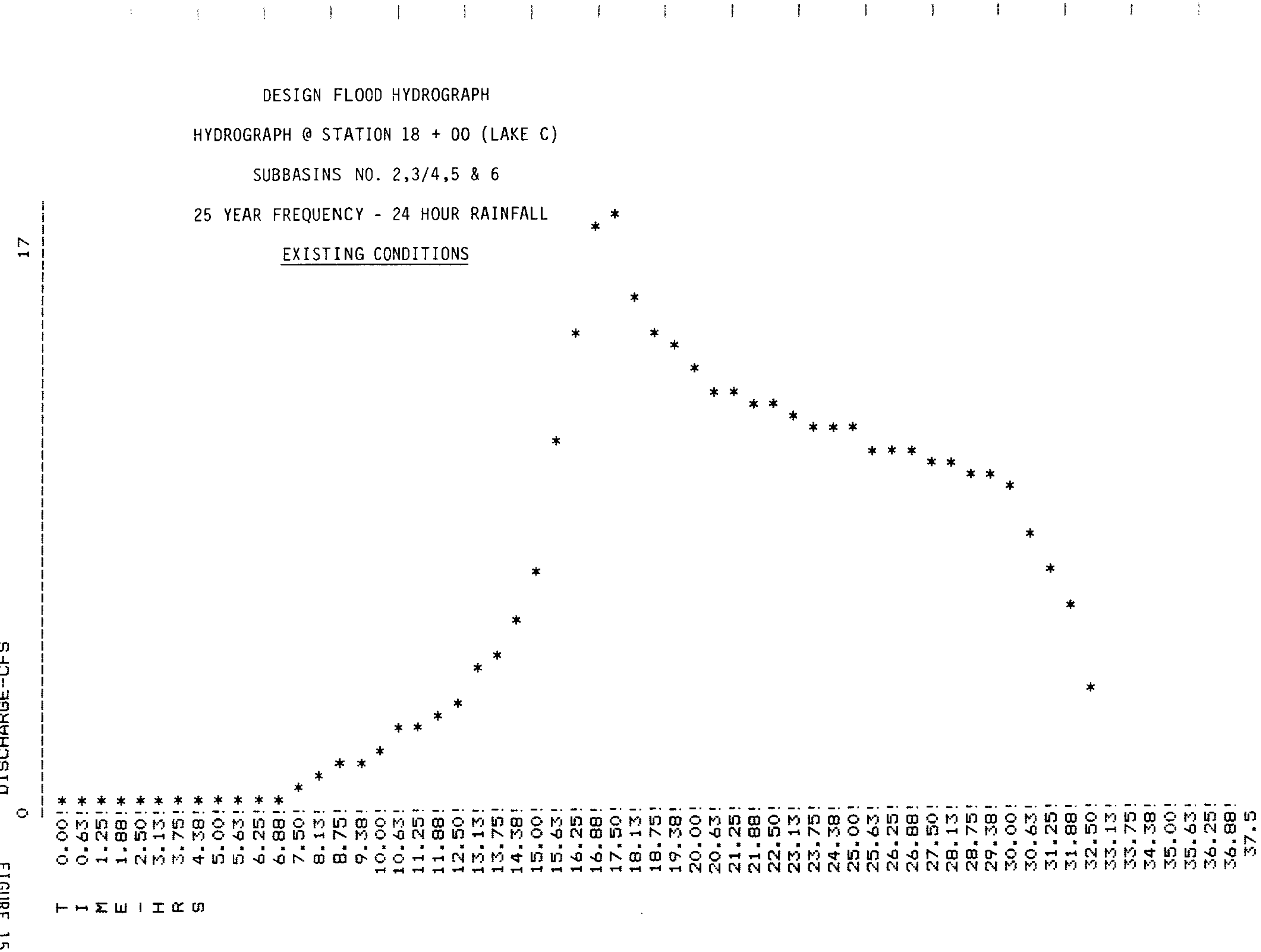


FIGURE 15

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 28 + 00

SUBBASINS NO. 2,3/4,5 & 6

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

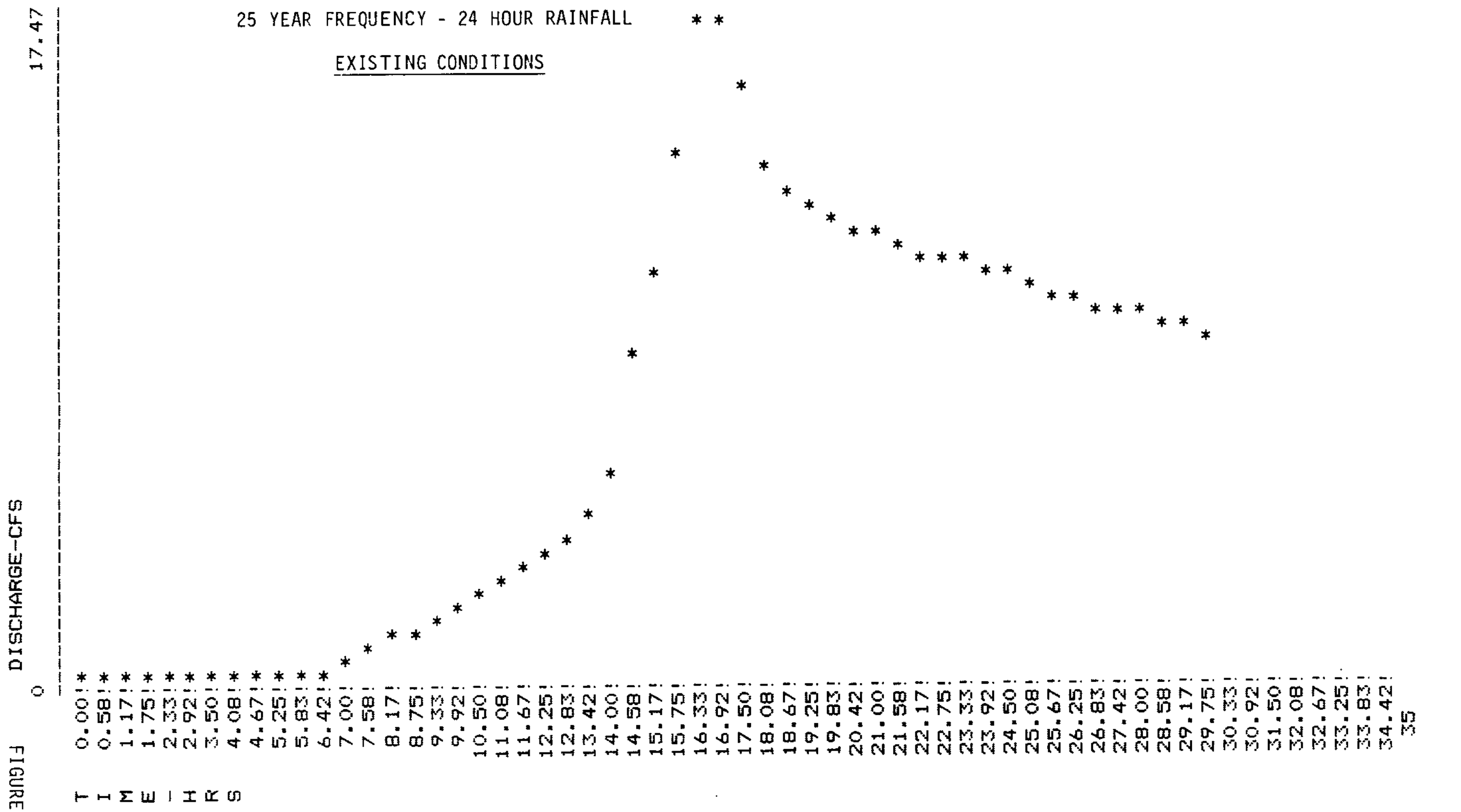


FIGURE 16

DESIGN FLOOD HYDROGRAPH

TOTAL HYDROGRAPH ENTERING LAKE C

SUBBASINS NO. 2,3/4,5/6 & 7

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

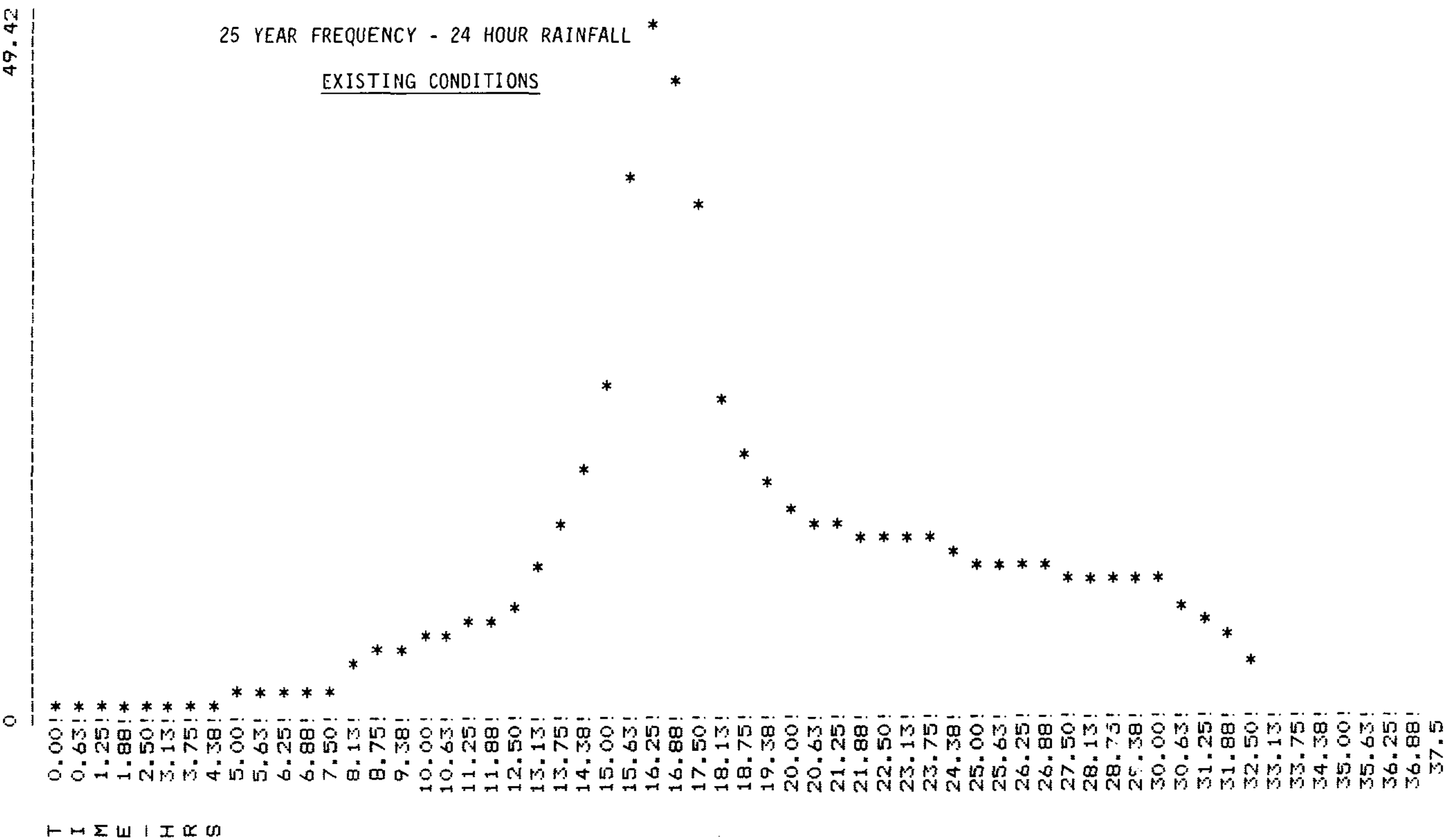


FIGURE 17

DESIGN FLOOD HYDROGRAPH

ROUTED HYDROGRAPH LEAVING LAKE C

SUBBASINS NO. 2,3/4,5,6 & 7

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

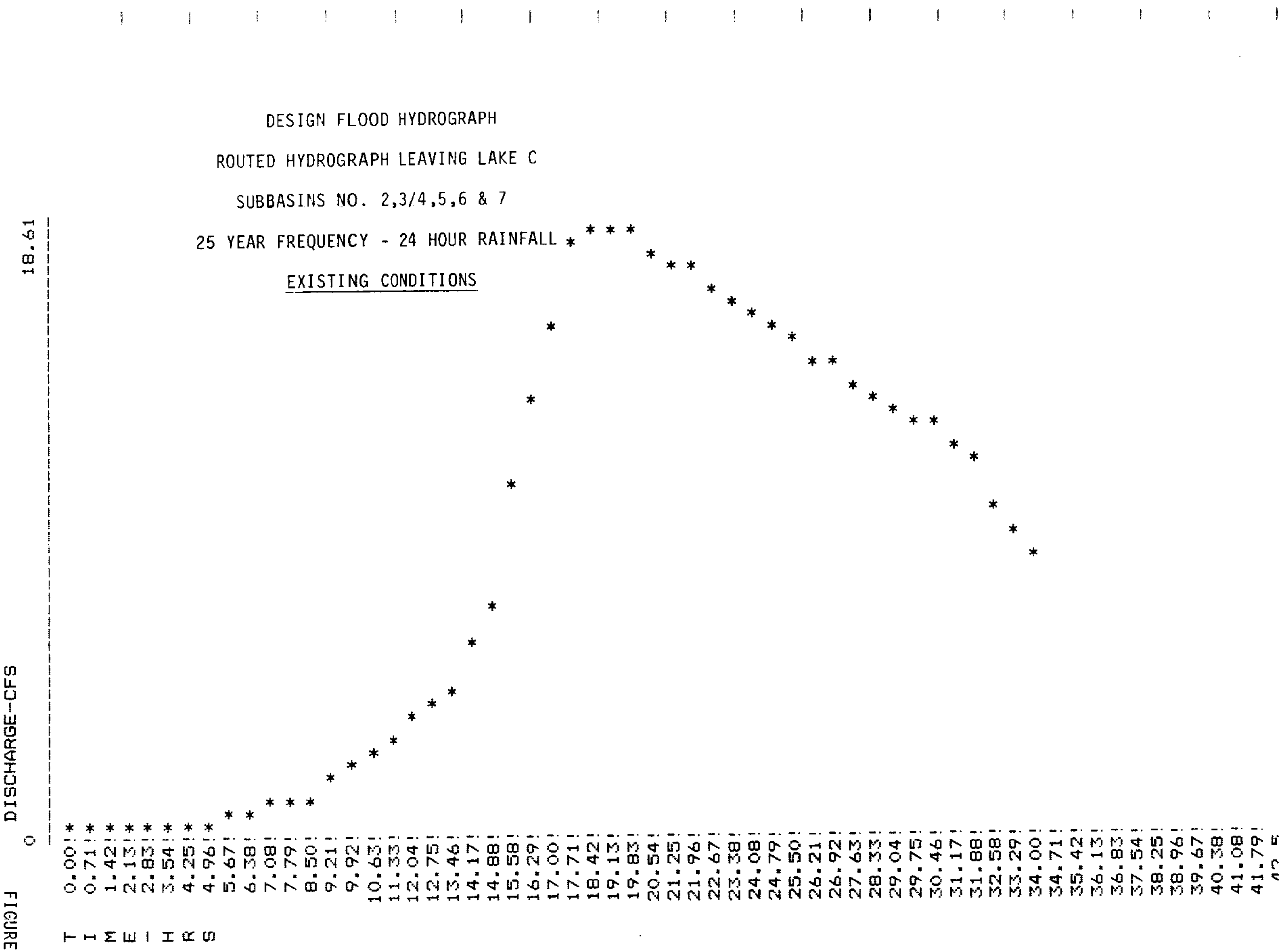


FIGURE 18

DESIGN FLOOD HYDROGRAPH

TOTAL HYDROGRAPH ENTERING LAKE D

SUBBASINS NO. 2,3/4,5,6,7 & 8

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

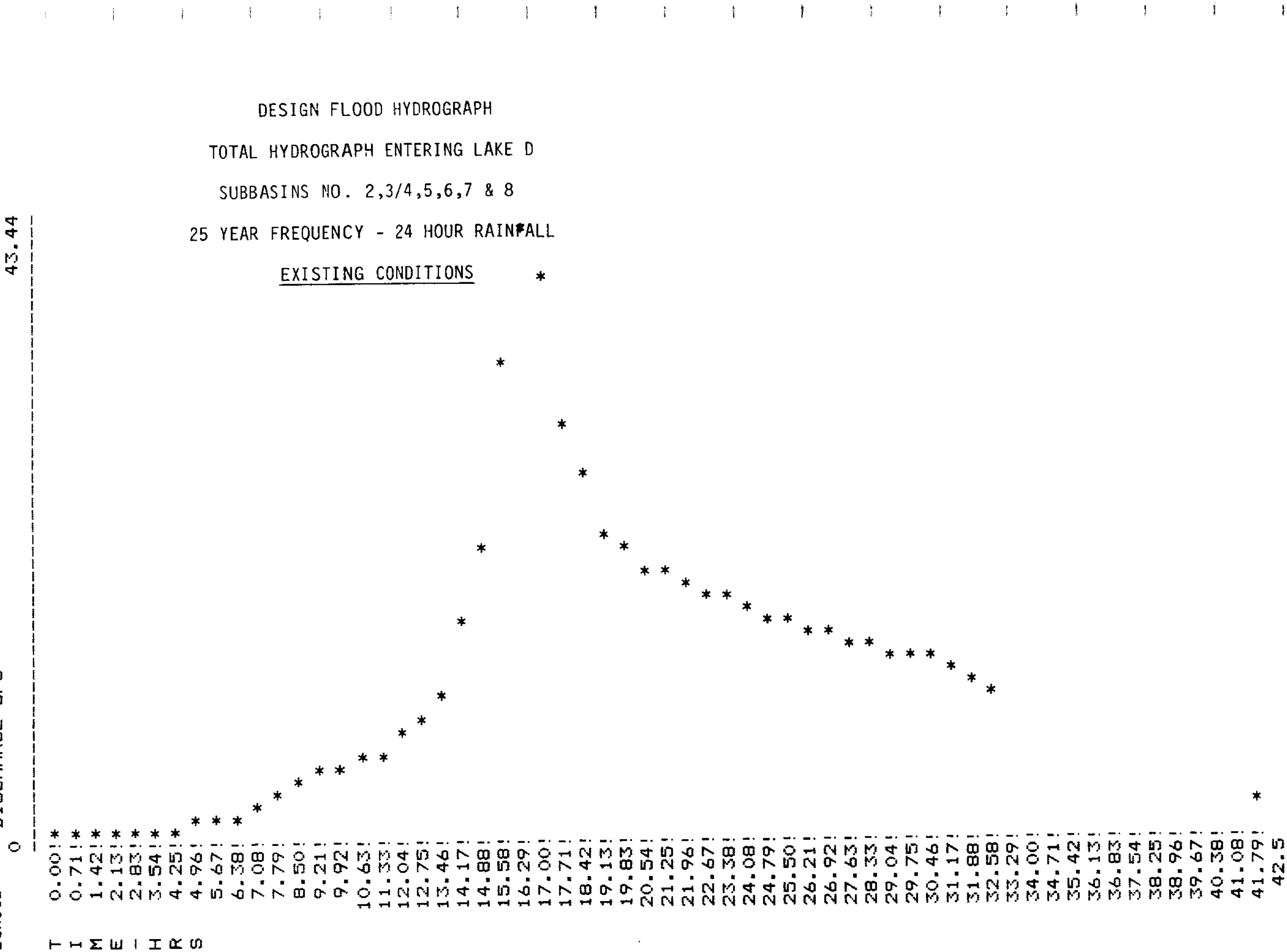


FIGURE 19

DESIGN FLOOD HYDROGRAPH

ROUTED HYDROGRAPH LEAVING LAKE D

SUBBASINS NO. 2,3/4,5,6,7 & 8

25 YEAR FREQUENCY - 24 HOUR RAINFALL

EXISTING CONDITIONS

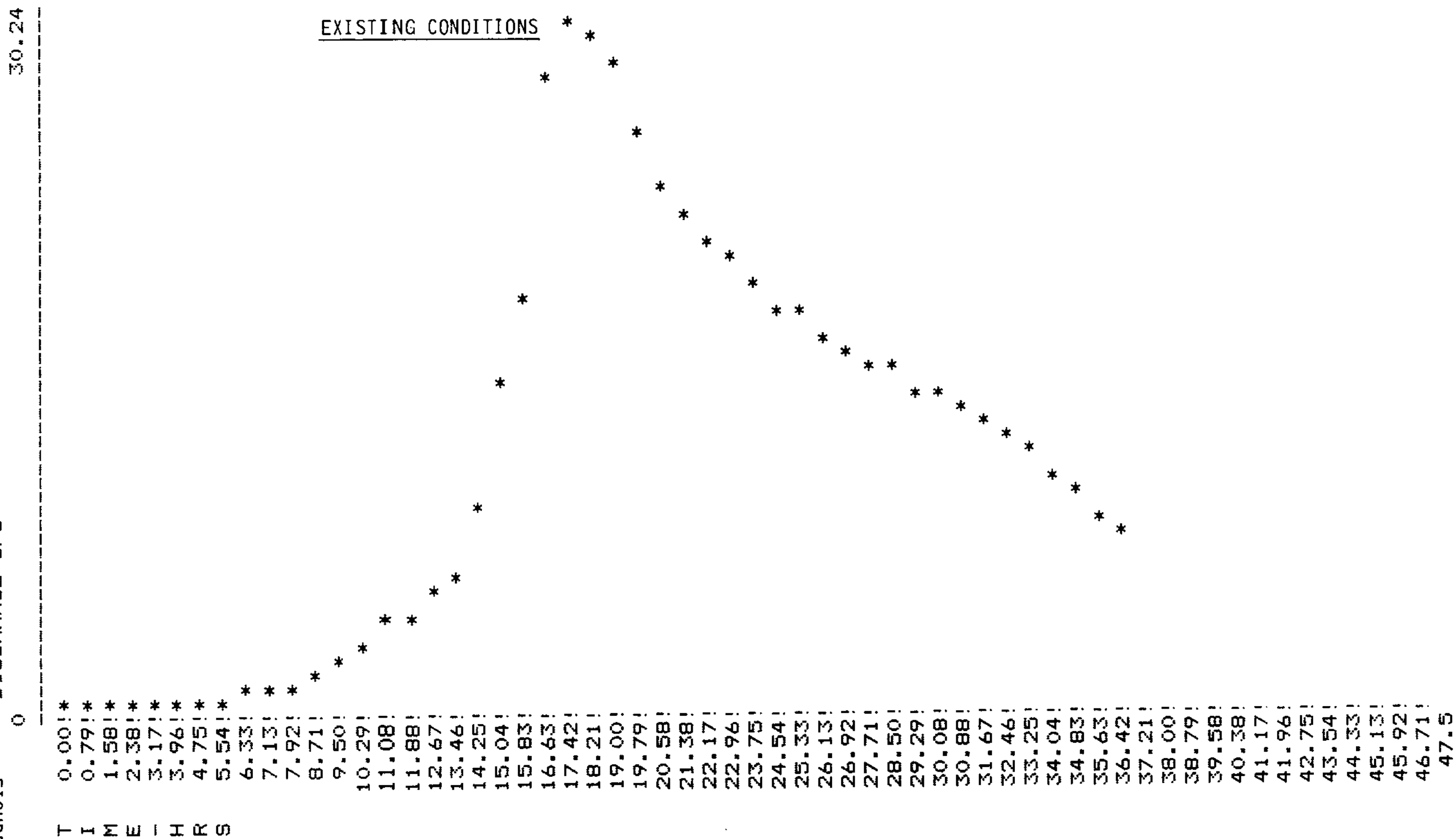


FIGURE 20

SECTION 3

DESIGN FLOOD HYDROGRAPHS

FUTURE CONDITIONS

DESIGN FLOOD HYDROGRAPH

INFLOW HYDROGRAPH TO LAKE A/B

SUBBASINS NO. 2 & 3/4

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

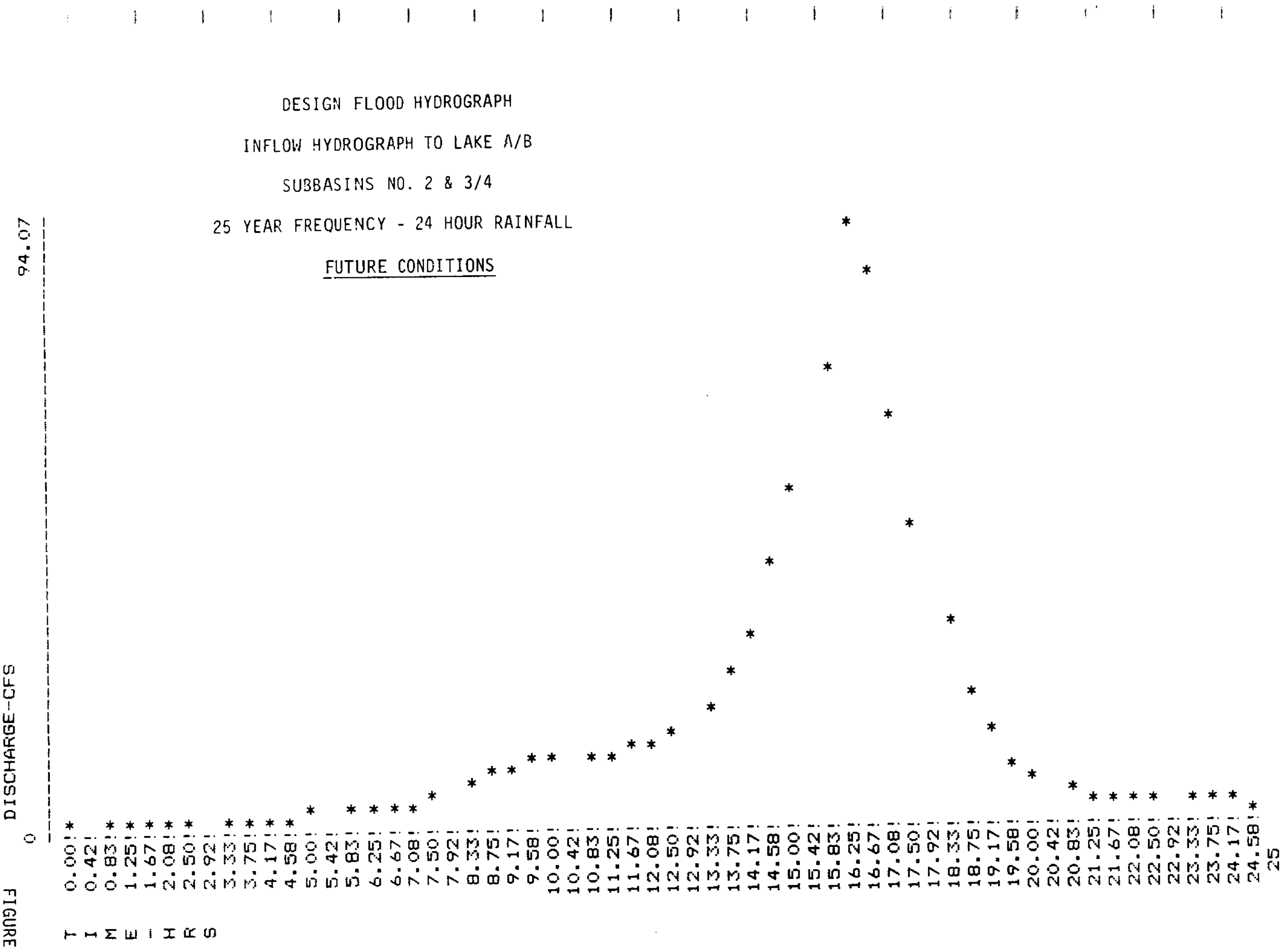


FIGURE 21

DESIGN FLOOD HYDROGRAPH

SUBBASINS NO. 2 & 3/4

AT OUTLET LAKE A/B

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

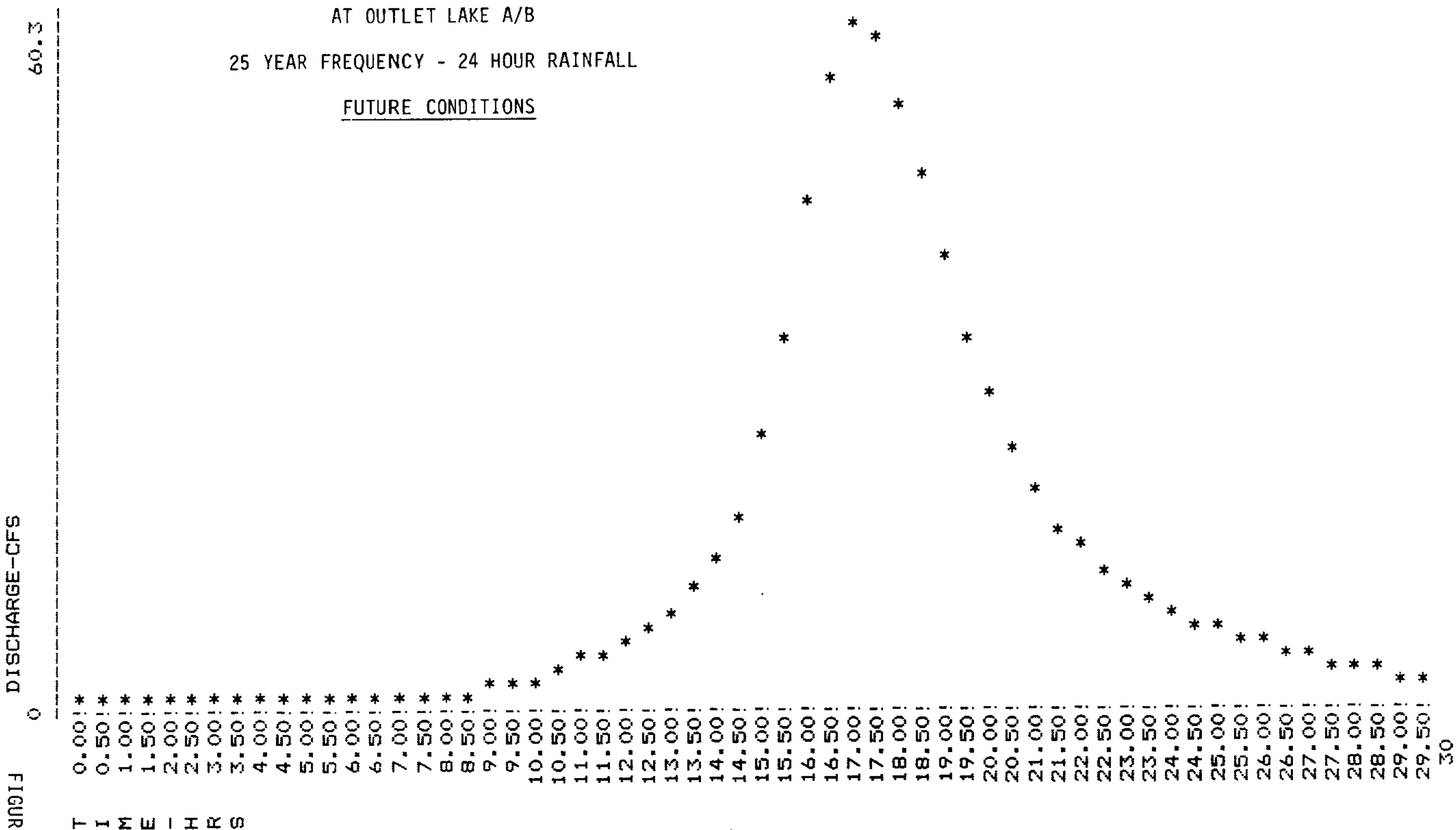


FIGURE 22

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 51 + 50

SUBBASINS NO. 2 & 3/4

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

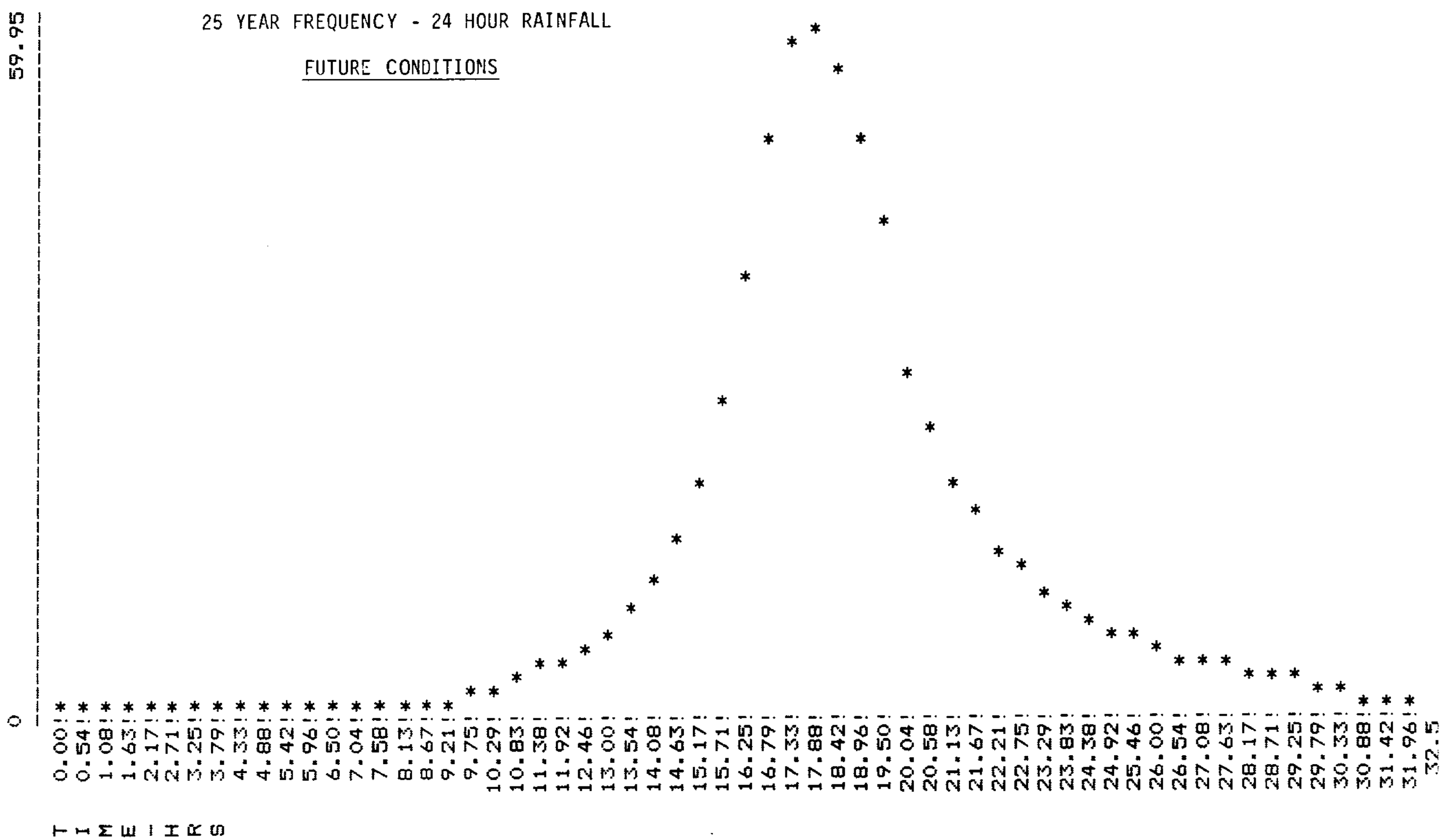


FIGURE 23

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 41 + 50

SUBBASINS NO. 2,3/4 & 5

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

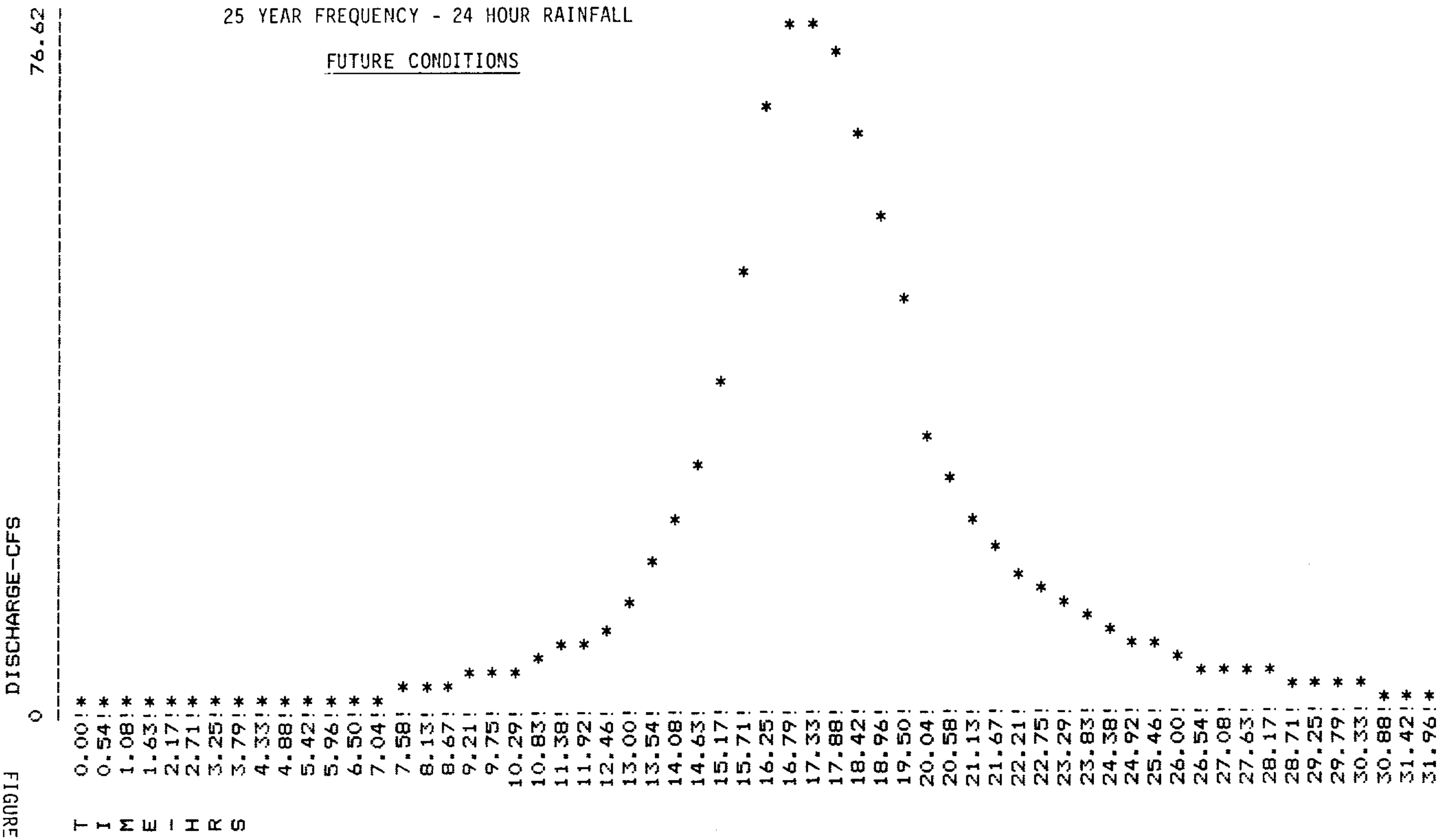


FIGURE 24

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 28 + 00

SUBBASINS NO. 2,3/4 & 5

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

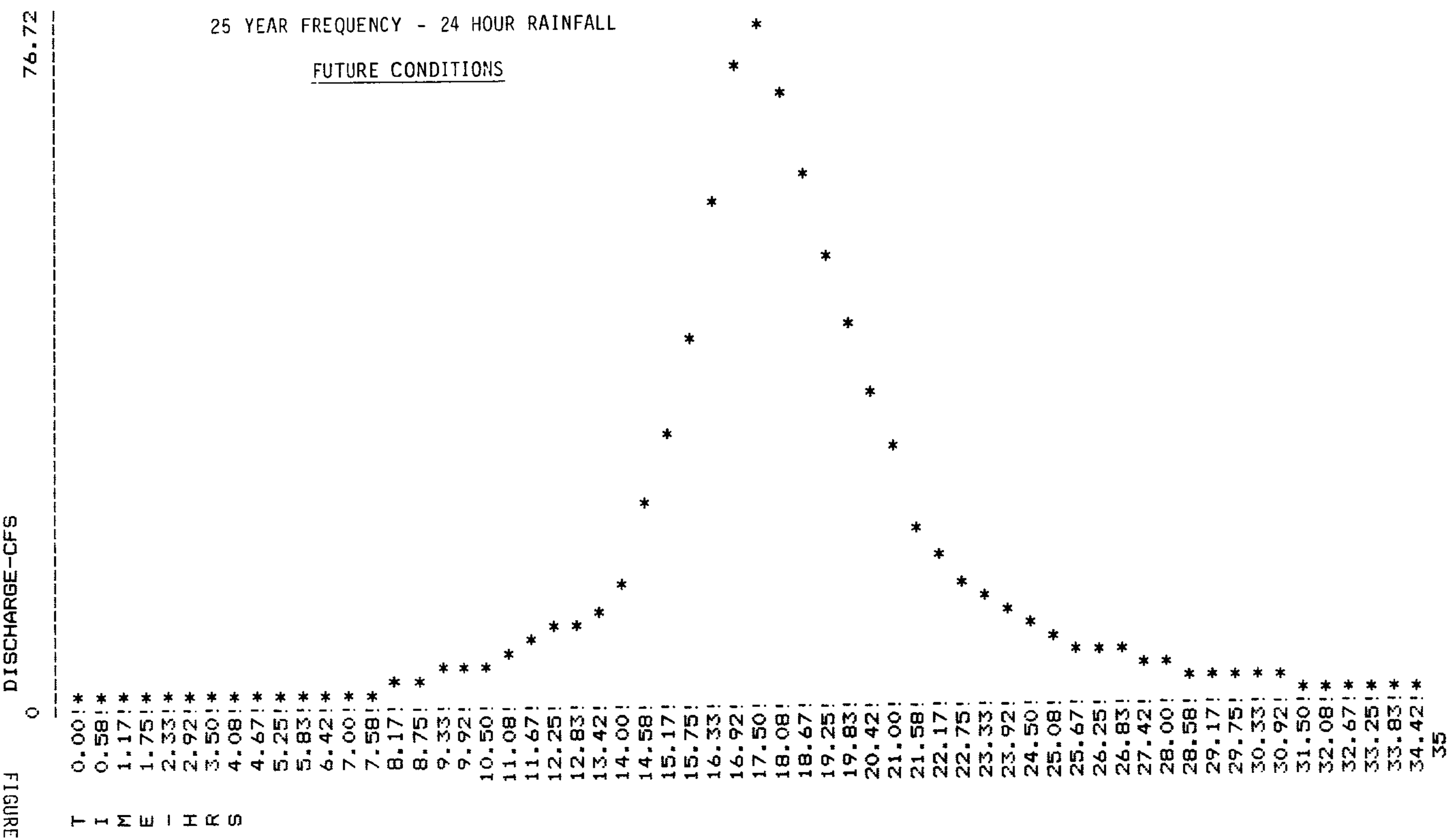


FIGURE 25

DESIGN FLOOD HYDROGRAPH

HYDROGRAPH @ STATION 28 + 00

SUBBASINS NO. 2,3/4,5 & 6

25 YEAR FREQUENCY - 24 HOUR RAINFALL

<u>FUTURE CONDITIONS</u>

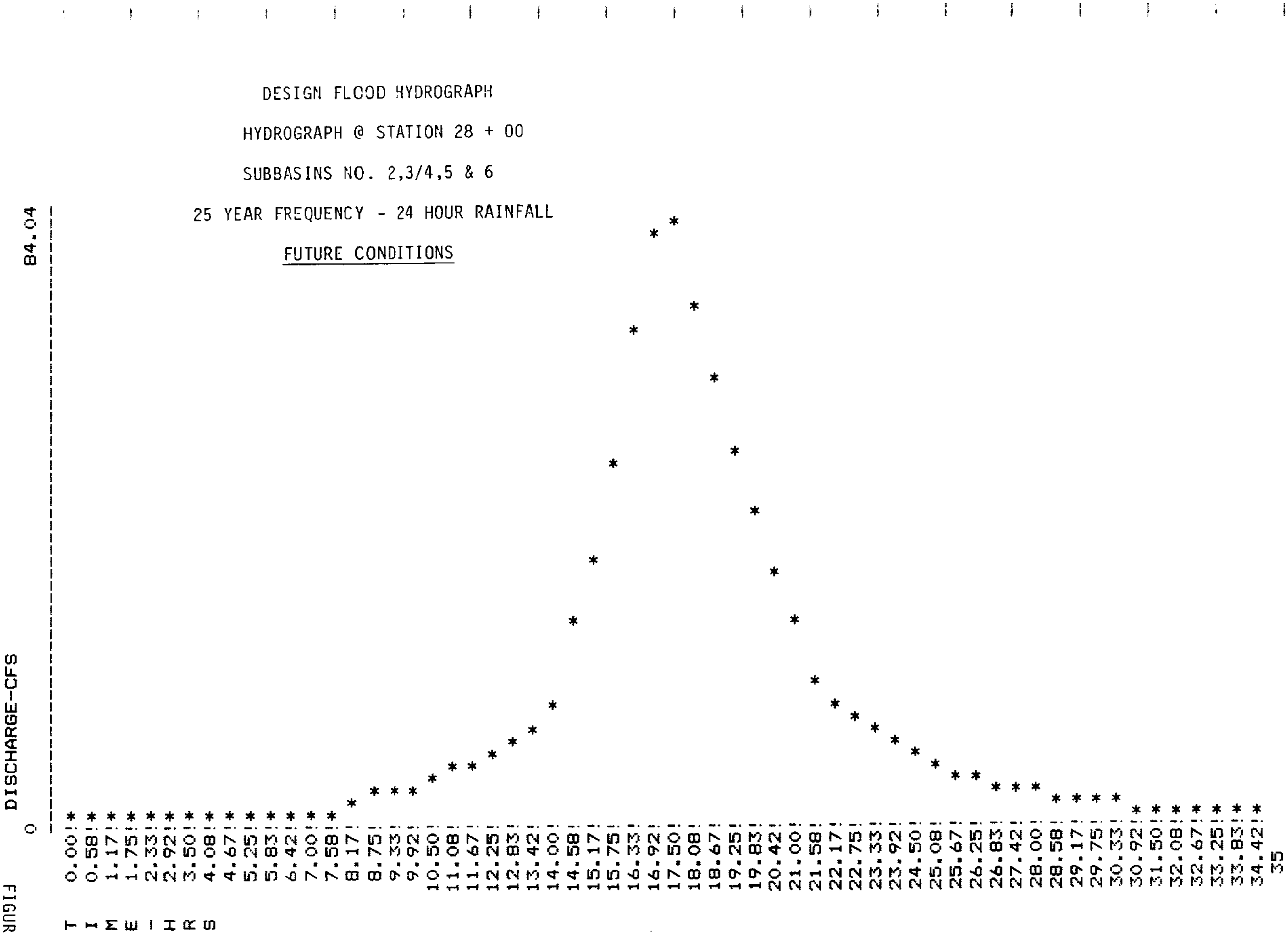


FIGURE 26

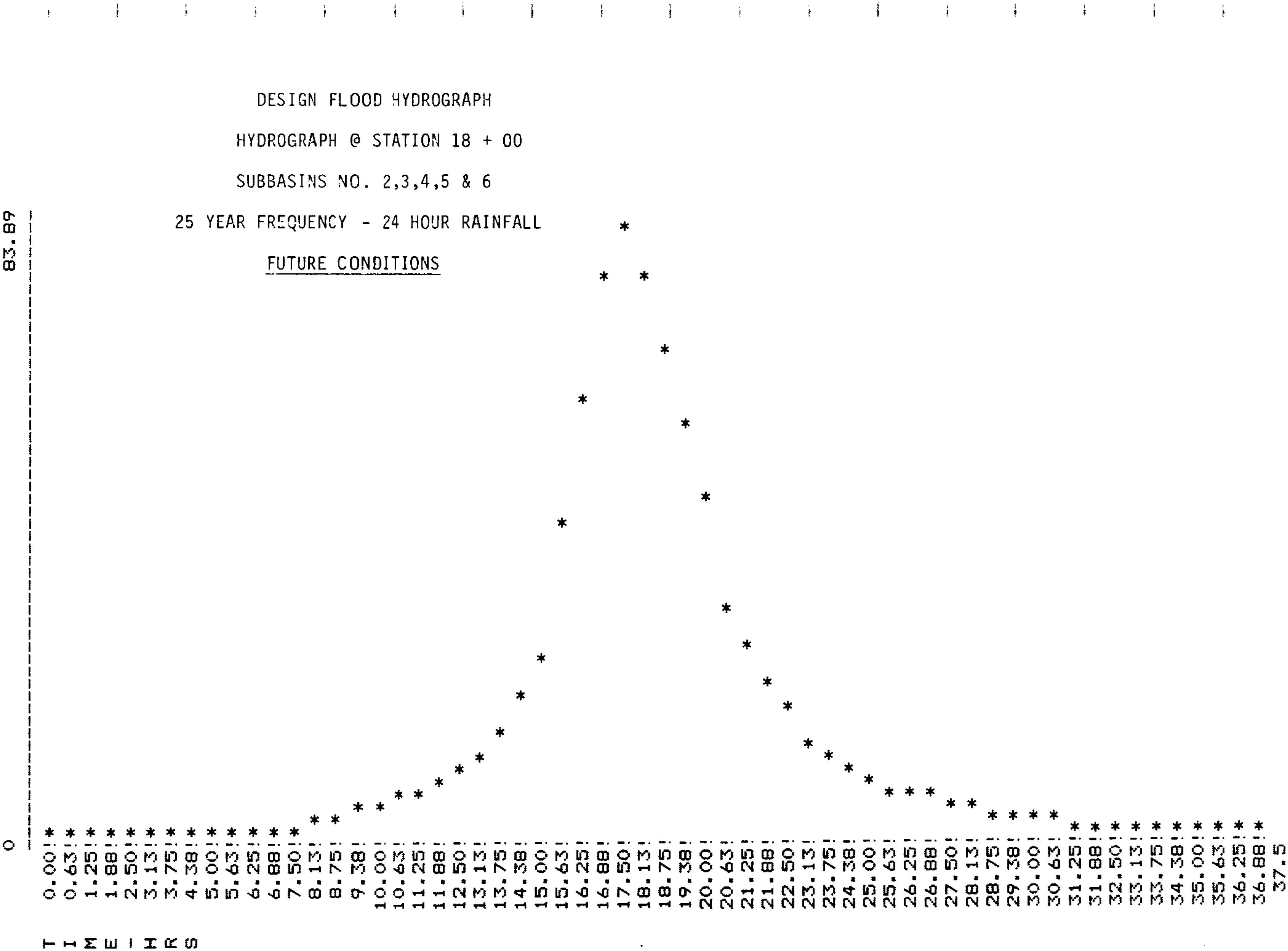

DESIGN FLOOD HYDROGRAPH
HYDROGRAPH @ STATION 18 + 00
SUBBASINS NO. 2,3,4,5 & 6
25 YEAR FREQUENCY - 24 HOUR RAINFALL
FUTURE CONDITIONS
DISCHARGE-CFS
83.89
0
TIME-HRS
0.00
0.63
1.25
1.88
2.50
3.13
3.75
4.38
5.00
5.63
6.25
6.88
7.50
8.13
8.75
9.38
10.00
10.63
11.25
11.88
12.50
13.13
13.75
14.38
15.00
15.63
16.25
16.88
17.50
18.13
18.75
19.38
20.00
20.63
21.25
21.88
22.50
23.13
23.75
24.38
25.00
25.63
26.25
26.88
27.50
28.13
28.75
29.38
30.00
30.63
31.25
31.88
32.50
33.13
33.75
34.38
35.00
35.63
36.25
36.88
37.5

FIGURE 27

DESIGN FLOOD HYDROGRAPH

SUBBASINS NO. 2,3/4,5,6 & 7

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

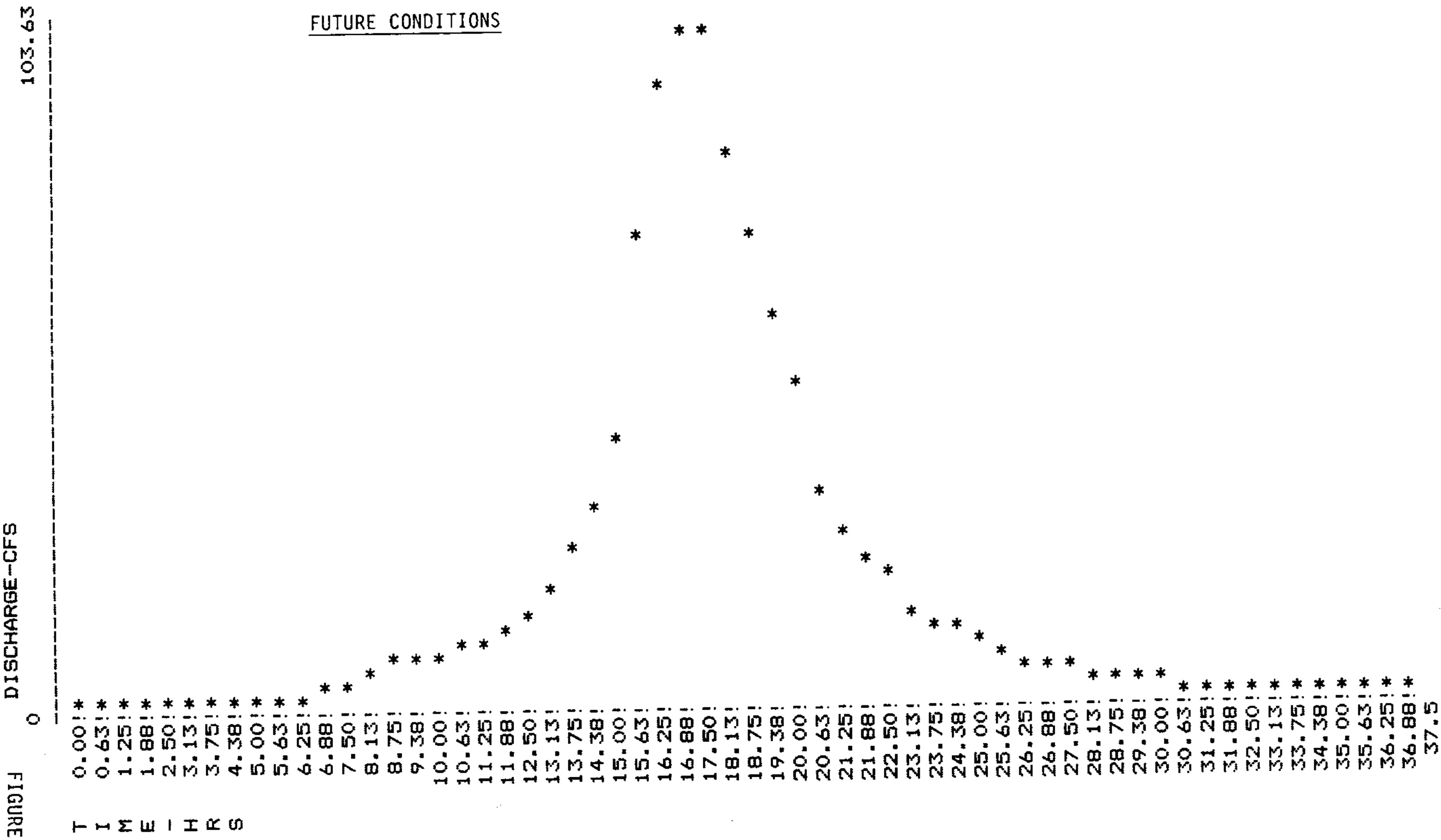


FIGURE 28

DESIGN FLOOD HYDROGRAPH

ENTERING LAKE C

SUBBASINS NO. 2,3/4,5,6,7 & 8

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

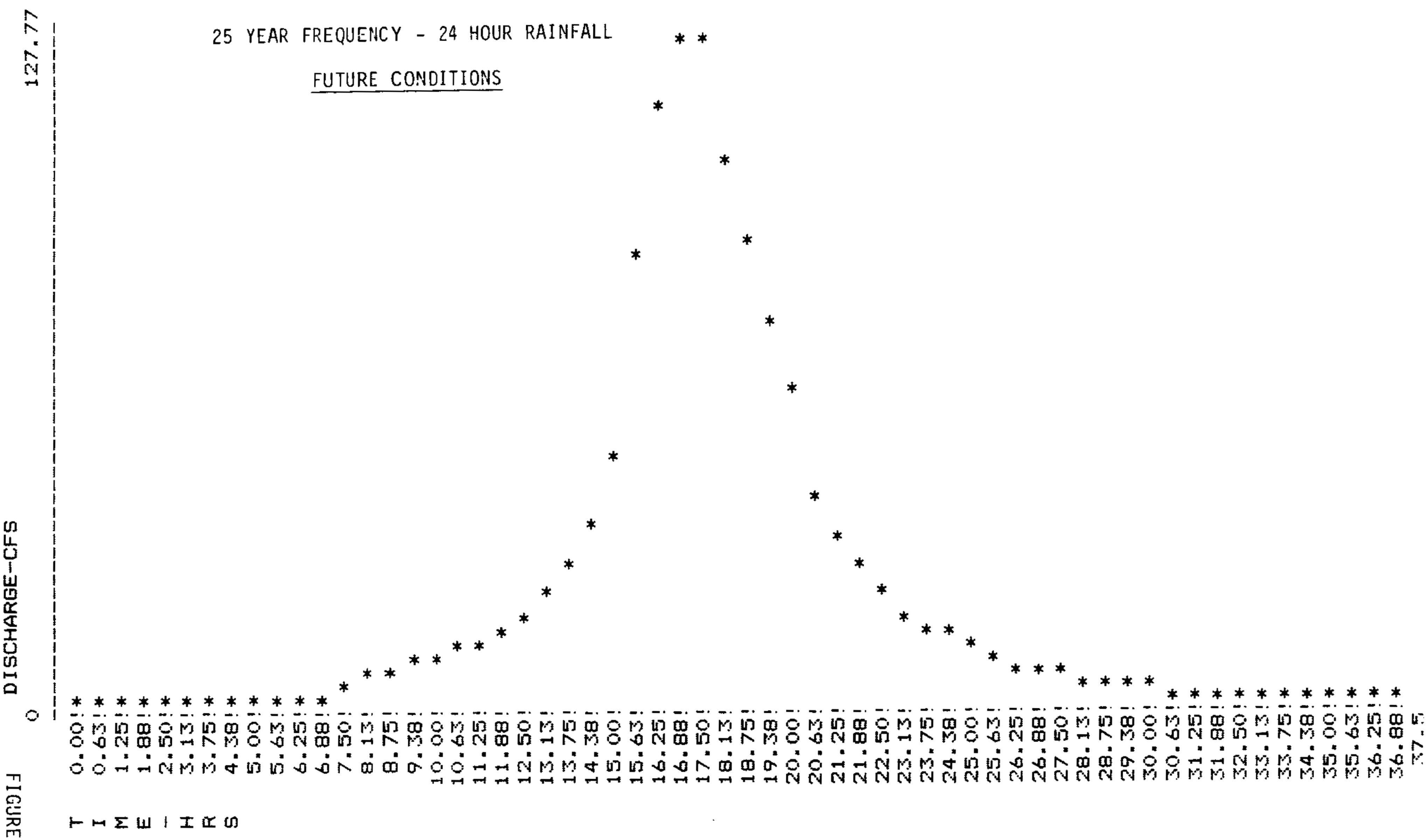


FIGURE 29

DESIGN FLOOD HYDROGRAPH

ROUTED HYDROGRAPH LEAVING LAKE C

SUBBASINS NO. 1,2,3/4,5,6,7 & 8

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

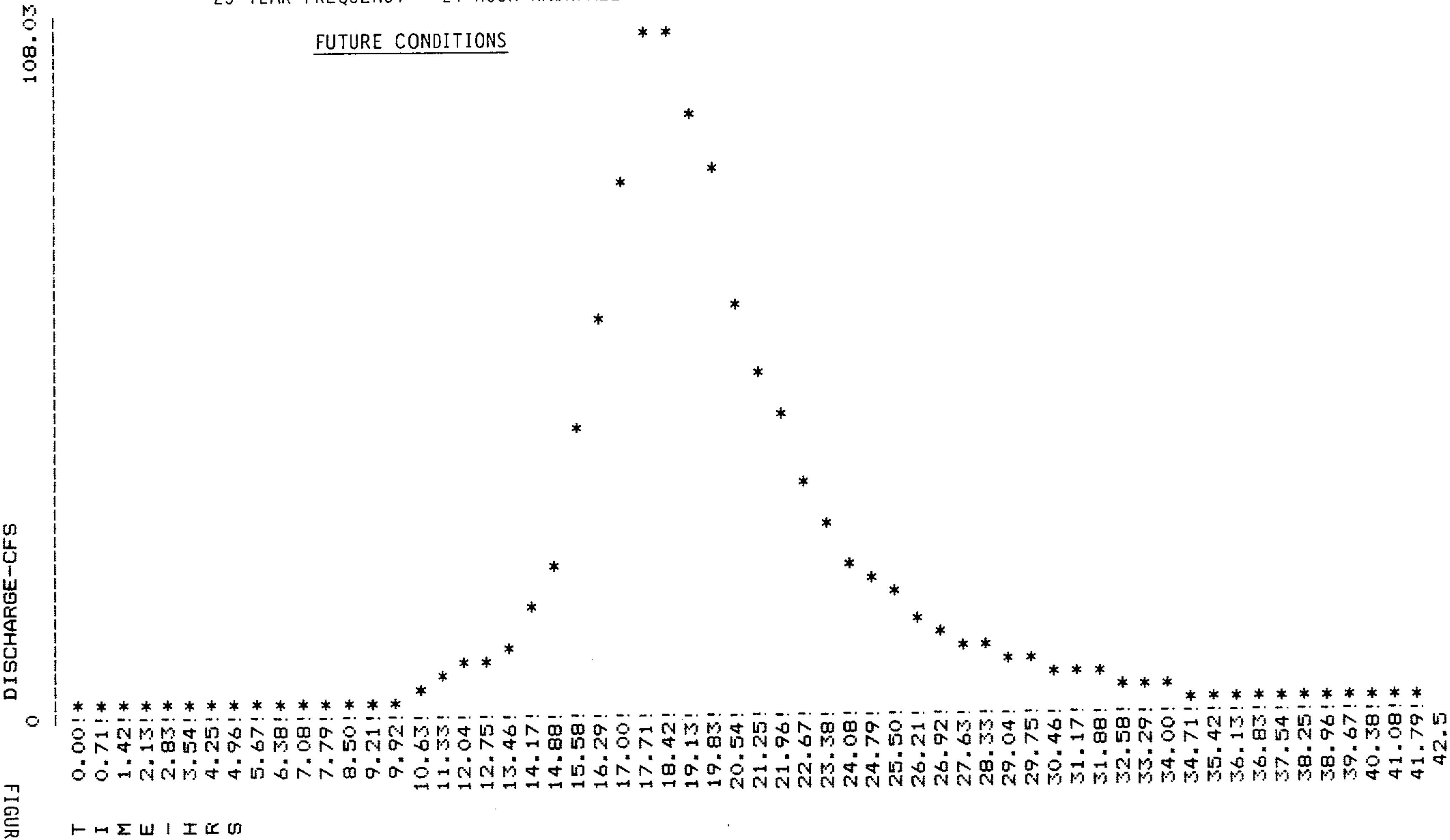


FIGURE 30

DESIGN FLOOD HYDROGRAPH

TOTAL HYDROGRAPH ENTERING LAKE D

SUBBASINS NO. 2,3/4,5,6,7,8 & 9

25 YEAR FREQUENCY - 24 HOUR RAINFALL

FUTURE CONDITIONS

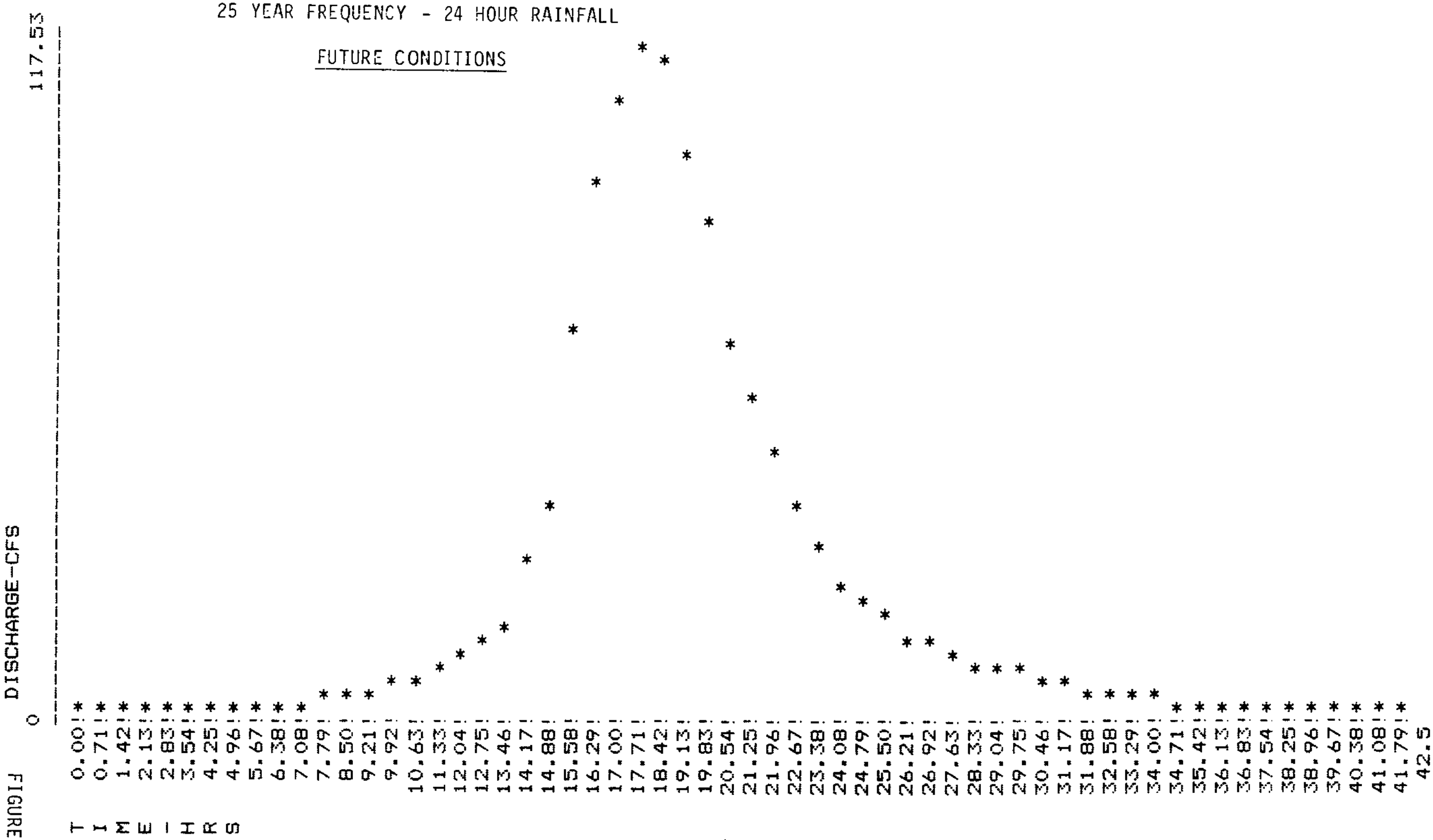


FIGURE 31

DESIGN FLOOD HYDROGRAPH

ROUTED HYDROGRAPH LEAVING LAKE D

SUBBASINS NO. 2,3/4.5.6.7.8 & 9

25 YEAR FREQUENCY - 24 HOUR RAINFALLL

<u>FUTURE CONDITIONS</u>

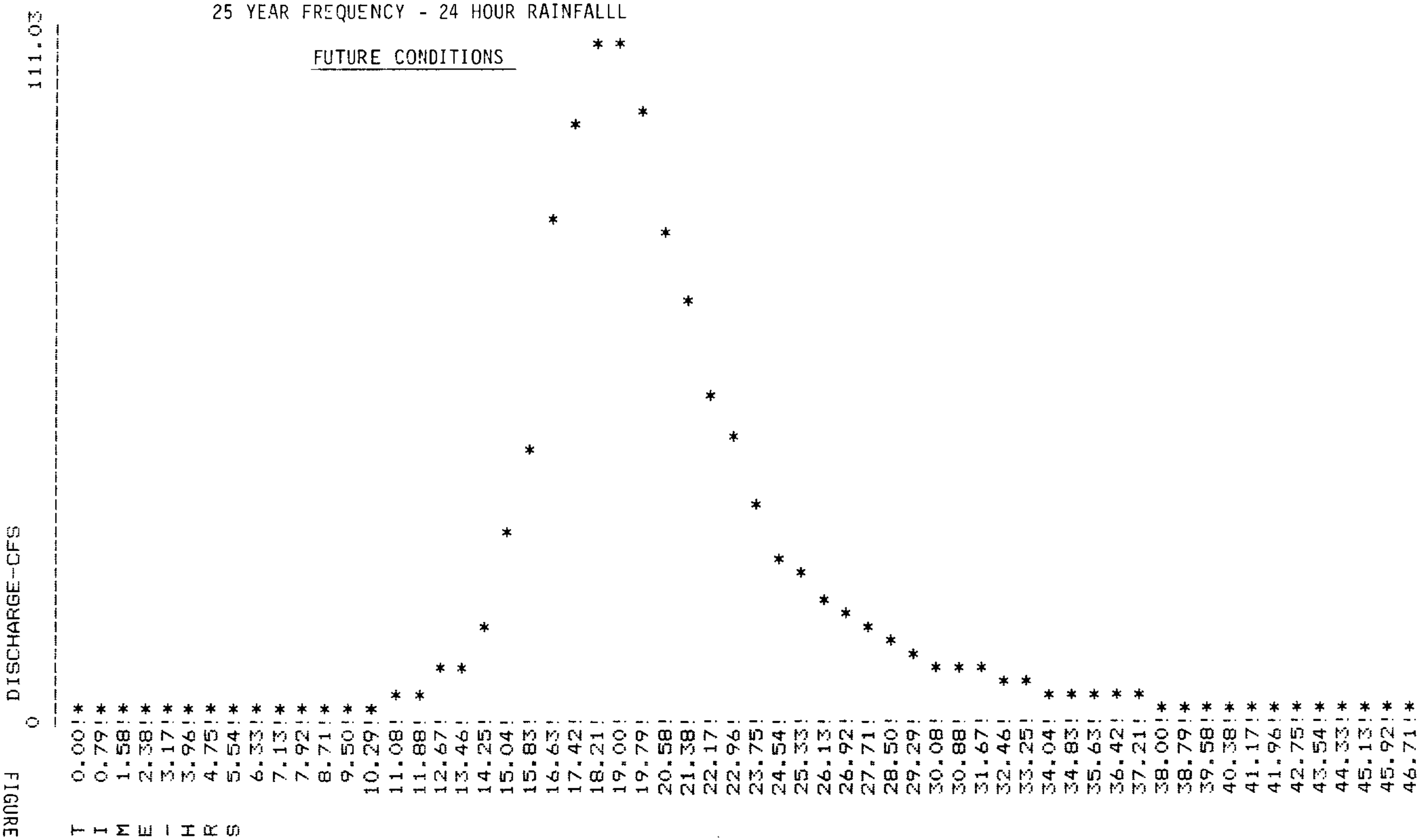


FIGURE 32

DRAWINGS

DRAWINGS

HOLE, MONTES
& ASSOCIATES, INC.
Engineers - Planners - Surveyors
2100 S. Tamiami Trail, Suite B
VENICE, Florida 34293

(941) 492-2450

LETTER OF TRANSMITTAL

TO:
Drainage Operations
Sarasota County
100 Cattlemen Road
Sarasota, FL 34232

ATTENTION: Les McKinney
RE: Stormwater Management Plan

DATE: March 27, 1998
HMA JOB NO. 1995090x

WE ARE SENDING YOU: ☒ Attached ☐ Under separate cover via__________ the following items:

☐ Shop drawings ☐ Prints ☐ Plans ☐ Samples ☐ Specifications
☐ Copy of letter ☐ Change order ☐ ____________

COPIES	DATE	No.	DESCRIPTION
1	3-27-98		South Venice Gardens Area Stormwater Management Plan - Final Report
			by CDM June, 1983

THESE ARE TRANSMITTED as checked below:

☐ For approval ☐ Approved as submitted ☐ Resubmit____copies for approval
☐ For your use ☐ Approved as noted ☐ Submit____copies for distribution
☒ As requested ☐ Returned for corrections ☐ Return____corrected prints
☐ For review and comment ☐ ____________
☐ FOR BIDS DUE __________ 19 ______ ☒ RETURNED AFTER LOAN TO US

REMARKS:

COPY TO: Scott Passmore, P.E.

SIGNED: Kreg Maheu
Kreg E. Maheu, P.E.

N
SCALE : 1" = 200'
STORM SEWER TO S.R.41 DRAINAGE SYSTEM
PROPOSED VENETIAN PLAZA SHOPPING CENTER
TAMIAMI TRAIL U.S. 41
PROPOSED DETENTION BASIN
1
2
MATCH LINE
LEGEND
G SUBBASIN DESIGNATION
AREA TO BE EXCAVATED
PROPOSED CULVERT
BASIN / SUBASIN BOUNDARY
℄ EXISTING CANAL
30'±
15'±
15'±
FUTURE SECTION
EXISTING GRADE
2
1
2
1
8'-0"
TYPICAL SECTION
(SECTION A-A STA. 45+50)
EXISTING 18"x12" CULVERT TO REMAIN
PROPOSED CHANNEL IMPROVEMENT 4.0' BOTTOM WIDTH, 2:1 SIDE SLOPE
PROPOSED LAKE CONNECTION
PROPOSED CHANNEL IMPROVEMENT 8.0' BOTTOM WIDTH, 2:1 SIDE SLOPES
BRANCH OF ALLIGATOR CREEK
PROPOSED 68" x 43" OPC
BRIARWOOD ROAD
70+00
65+00
60+00
55+00
45+00
40+00
35+00
30+00
25+00
20+00
15+00
10+00
5+00
PROPOSED 53" x 34" OPC
PROPOSED 68" x 43" OPC
A
A
LAKE "A"
LAKE "B"
RIGEL
LAKE "C"
LAKE "D"
LAKE "E"
ROSEDALE ROAD
MORNINGSIDE ROAD
3/4
5
6
7
8
9
PROPOSED 36" RCP
PROPOSED 68" x 43" OPC
PROPOSED 68" x 43" OPC
FIESTA DRIVE
MOHEGAN ROAD
RIGEL ROAD
ENGLEWOOD ROAD
SERPULA ROAD
VALENCIA ROAD
CARMINE ROAD
TANAGER ROAD
SEMINOLE DRIVE
NON CONTRIBUTING AREAS
LIMIT OF DRAINAGE BASIN
EXISTING OUTLET TO BE FILLED
SHASTA
MOHAWK
Road Outfall
Road Outfall
Bike Path
Bike Path
Opening
REMARKS
REV NO
DATE
DRWN
CHKD
DESIGNED BY BLG
DRAWN BY
CHECKED BY BLG
APPROVED BY BLG
DATE 6/18/83
CAMP DRESSER & McKEE INC.
CDM
GENERAL PLAN
PROPOSED IMPROVEMENTS TO OUTLET CANAL
SOUTH VENICE GARDENS AREA
PROJECT NO.
9250-I-RT