DOCKET NUMBER
PROD. & UTIL. FAC. 50-329,330

316. Describe in detail what aspects of a reactor system design must be completed or you insist upon being completed before the issuance of a construction permit. If in your answer you make reference to other than textual (exclusive of footnote) matter in the PSAR, or reference to other than textual (exclusive of footnote) matter in your Safety Evaluation, then set forth completely the text of each such reference or attach a copy.

As stated in Sections 50.34 (a) (2), (3), and (4) of 10 CFR Part 50 of the Commission's regulations, an application for a construction permit must include a summary description and discussion of the facility and the preliminary design, including (1) the principal design criteria, (2) the design bases, and (3) information relative to materials of construction, general arrangement and approximate dimensions, together with a preliminary analysis and evaluation of design and performance. We do not require that the final design be completed for any specific aspects of the reactor system prior to issuance of a construction permit.

Joseph A. Murphy

Joseph A. Murphy

250. Describe in detail what analysis, specifying each fact, calculation and assumption thereof, was made by you concerning the probable maximum flood and its possible consequences to the proposed plant. During the period of construction of the proposed Midland Units and during your review of the applicant's calculation of the probable maximum flood level, what changes can or are contemplated to be incorporated into the design to insure integrity of the proposed Midland Units, if error is found in flood calculations. Also, state why you wait until construction of the proposed Midland Units is under way to "review the Applicant's calculation" and to assure yourself that "the calculational techniques have been properly employed." In your answer you make reference to other than textual (exclusive of footnote) matter in the PSAR, or reference to other than textual (exclusive of footnote) matter in your Safety Evaluation, then set forth completely the text of each such reference or attach a copy.

[to the extent that information is sought as to why the calculation is not required at this time]

In the course of our review of the Midland application, we and our consultant, the U. S. Geological Survey, have discussed the computational procedures and the hydrologic parameters used in developing the Probable Maximum Flood (PMF) with the applicant. Some details in these areas are included in a draft report entitled "Midland Project - Hydrology and Hydraulics" which was informally provided to the staff by the applicant.*/ We have evaluated the calculational techniques and hydrologic parameters used and have determined that they are acceptable. We consider the applicant's calculated Tittabawassee River PMF peak discharge of 262,000 cfs and the associated water level elevation of 631 ft as presented

*/ This informal report was inadvertently omitted from the compilation of informal submittals previously supplied to the Board and the parties. A copy is attached to this response.

in the draft report to be adequate. This level, plus the increase due to wind wave action not yet determined, will provide the design basis for PMF protection. In lieu of a completed and reviewed calculation, the applicant has stated in Amendment 10 that vital structures will be designed to withstand the PMF which will be calculated by the applicant using techniques and parameters already evaluated by the staff. We consider this criterion acceptable. During our review of the Final Safety Analysis Report, we will assure that this criterion has been met and the design of the vital structures is adequate. We consider that a commitment by an applicant to meet design criteria found acceptable by the staff is adequate for issuance of a construction permit.

Joseph A. Murphy

Joseph A. Murphy

Dwight E. Nunn

Dwight E. Nunn

Midland Project - Hydrology and Hydraulics

The main concern hydrology-wise regarding the safety of vital installations is the probable maximum water level at the plant site. Four parts are involved in this problem:

1. What would be the natural probable maximum flood (PMF) discharge?

2. What would be the effect of upstream dam failures at the time of the PMF?

3. What would be the maximum water level resulting from the PMF (or PMF + dam failure)?

4. Could a Bullock Creek PMF with a 100-year flood on the River give a higher water level?

Probable Maximum Flood

The hydrograph of the Tittabawassee River PMF at the plant site was developed using:

1. The unitgraph shown in Figure 1.

2. Probable maximum precipitation (PMP) and infiltration losses as shown in Figure 2.

Rainfall excesses obtained by subtracting the losses from the rainfalls were applied to the unitgraph to obtain the PMF hydrograph of Figure 2.

Dam Failure

It was requested that a reasonable mode and time of failure be assumed to evaluate the effect of dam failure upon the maximum water level at the plant site. The worst condition would be if the four dams on the river were to fail successively downstreamward. It was understood that the total storage behind all four dams could be treated as concentrated at the dam farthest downstream, - Sanford Dam.

Study of the Sanford Dam design and records, and a brief site investigation indicated that failure would probably be by overtopping, - on which there is a dearth of usable information. A theoretical approach was tried for getting the logical mode and duration of failure by using the bed load transport ability of flowing water. However, this did not give logical results after the first 20 to 30 minutes of the failure process. A search of the technical literature of the past 15 years regarding dam failure by overtopping yielded only 2 instances where maximum discharge and failure duration were given. Based on these a failure pattern in 10-minute increments was assumed as shown in Figure 3.

It seems logical that failure would start as the headwater level rises over the low point in the crest at the right end of the dam, and that the water level would continue to rise over the remaining length of the crest due to two factors:

1. A high rate of inflow to the reservoir from upstream dam failure, in addition to the natural flood. (The rate of inflow from upstream failures was not studied since all storage to be released was assumed concentrated at Sanford.)

2. Increasing restriction on the outflow thru and over the dam, - from high tailwater levels that would result from a constriction in the channel about 1/2 mile downstream at the C&O Railroad.

It was found that the latter condition would reduce the headwater-tailwater differential at the dam to less than 11' so that peak outflow thru the break would be less than 210,000 cfs. It would also limit the amount of total stored water released thru the break to about 167,000 ac.-ft. Figure 4 shows how this value was arrived at, and Figure 5 shows the hydrograph resulting from the dam failure. The rising limb of the failure hydrograph is a result of weir overflow calculations based on the assumed mode and duration of failure, and the recession limb has been determined by trial, assuming all storage is released in about 24 hours.

The next step was to determine a reasonable point in time at which overtopping could be expected. Logically, this would occur when the rate of inflow to the reservoir exceeded the ultimate capacity of the spillway. This capacity was determined to be 25,000 cfs with headwater .5 feet over the low point in the dam crest. Sanford Dam spillway has 6 gated bays, - 2 @ 25.35' wide and 4 @ 22' wide. Its ogee crest is 13.5' lower than the embankment low point and probably has a discharge coefficient of 4.0 at that head. Tailwater would be about 2.5' over the spillway crest and its effect is still negligible at that discharge.

The PMF hydrograph at Sanford Dam was then developed to obtain the time (from start of runoff) at which the discharge from Sanford would begin to exceed 25,000 cfs. It was assumed outflow=inflow since the storage capability is small with respect to the flood volume. The hydrograph was developed using:

1. The unitgraph shown in Figure 6.

2. PMP and infiltration losses as shown in Figure 7.

Rainfall excesses were applied to the unitgraph to obtain the PMF hydrograph at Sanford Dam as shown in Figure 7. The hydrograph shows that overtopping would start about 1.5 days after the start of runoff.

The failure hydrograph in Figure 5 was routed by the coefficient method thru the Tittabawassee River to the Midland plant site, using an average velocity of 3.5 ft./sec., time increments of 6 hours, and channel storage coefficient of .10. The assumed velocity and coefficient are considered appropriate for the PMF condition. Since the flooded area would be extensive, average velocities would be low and storage effects would be high. The channel-routed hydrograph is shown on Figure 8 together with the natural PMF at the plant site. Figure 8 also shows that when the failure hydrograph is added to the PMF in proper phase, the peak discharge at the plant site would be approximately 262,000 cfs (8,000 less than was estimated previously).

Maximum Water Level

It has been determined that the water level that would result at the plant site from the PMF peak of 262,000 cfs in the Tittabawassee River, would be about El. 630.1, - say 631 in round numbers, - under post-project conditions. This is the result of calculations made using conservative assumptions regarding channel and flood plain flow resistance and downstream water levels. Calculations were made using a USCE-originated computer program which uses the standard step backwater method. For the calculations, the following were used:

1. Cross-sections obtained from four sources - USGS 7.5 minute quadrangles with 5 ft. contours, 1 ft. contour maps by Abrams Aerial Survey, USCE surveys of 1948-49, and Bechtel surveys of April, 1970.

2. Channel and flood plain roughness values as on each cross-section shown on Figure 9.

3. Starting water surface elevations for 5 different discharges arrived at by several trial runs for each discharge, selecting the higher of the pair of runs showing the best convergence.

For the cross-sections, the USGS maps were used generally above El. 610, sometimes higher. The use of USCE survey information was limited to the channel proper plus 200 to 300 ft. of the flood plain. Abram's maps were used to fill in between USGS and USCE data, and Bechtel surveys were used for sections 7 thru 10. In all, there were 11 cross-sections used to study pre-project conditions, and 13 cross-sections for computation of post-project water levels. The cross-section locations are shown in Figure 9.

For the purpose of developing judgement for "n" values in the area of concern, calculations were made to duplicate the water surface profile of the record flood of March, 1948, (34,000 cfs), checking at 4 points where the water level had been determined by the U. S. Corps of Engineers, Detroit District. The observed and the calculated water levels for pre-project conditions are listed in Table 1, together with Manning "n" values used in the calculations.

However, roughness values indicated by this study could not be applied blindly to the post-project conditions. A study of recent aerial photographs showed that for some reaches of the eastern flood plain it would be on the unsafe side to use the roughness indicated in the pre-project study. In other reaches it would be ridiculously conservative to apply the "n" value determined in the pre-project study to the entire eastern flood plain. The cross-sections were therefore divided into sub-sections and conservative but reasonable "n" values assigned to the eastern flood plain portions. (Fig. 9)

The "n" value applied to the western flood plain is also conservative because construction of the cooling pond dikes eliminates the shallow, high "n" flood plain to the west as a flood flow area, leaving the deeper, lower "n" area to the east so that the right overbank "n" should be lower than in any historical flood that inundated areas to the west. Also, because the computed pre-project water levels were slightly lower than the observed levels (Table 1), the channel "n" for cross-sections 10 thru 6 was raised to .028 for post-project conditions.

To be conservative it was assumed that the new railroad bridge and the Smith's Crossing bridge would not be destroyed by the PMF prior to the flood peak. However, Smith's Crossing bridge would almost certainly be destroyed. The result would be a lower-than assumed water surface elevation at Section 10. It was assumed that the railroad bridge piers would gather debris so that the effective thickness of each pier would be about 30 ft.

Bullock Creek PMF

Computations were made using 5 cross-sections and the standard step method to determine the maximum water level that would result if there were to be a Bullock Creek PMF concurrent with the arrival at the plant site of the 100-year flood peak on the Tittabawassee River. That maximum level was determined to be about El. 620 at the west dike of the cooling pond and poses no threat to vital installations since they are at El. 634.

The Bullock PMF was developed using:

1. The 2-hour unitgraph shown in Figure 10.

2. PMP and infiltration loss rate shown in Figure 11.

Application of the maximum 24-hr. rainfall excess to the unitgraph resulted in the PMF hydrograph in Figure 11.

The corresponding Bullock Creek water surface profile was calculated using the same computer program as used in the river backwater computations, with cross-sections located as shown in Figure 12.

TABLE 1

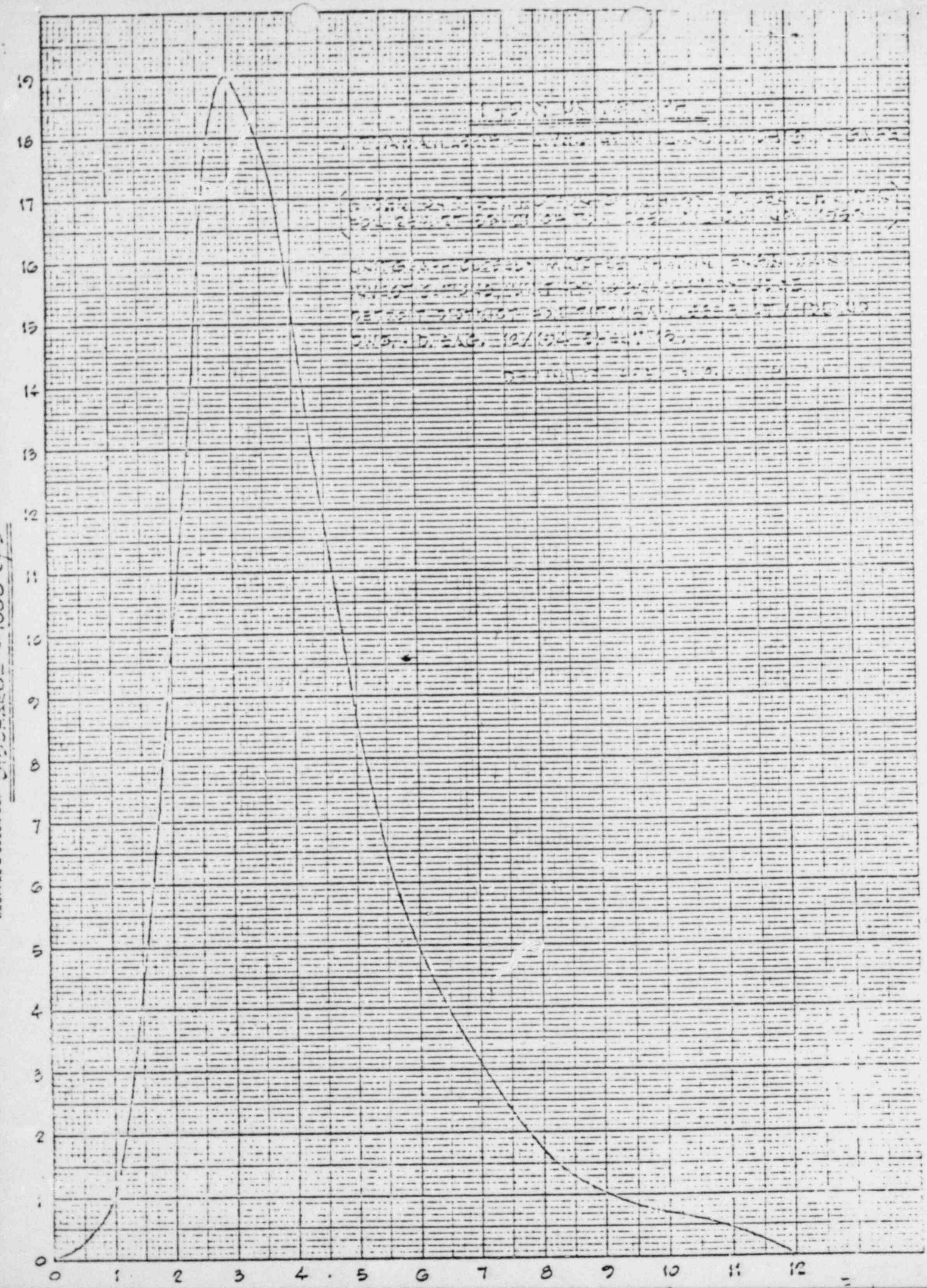
OBSERVED AND CALCULATED WATER SURFACE ELEVATIONS
SMITH'S CROSSING TO PLANT SITE AND USGS GAGE

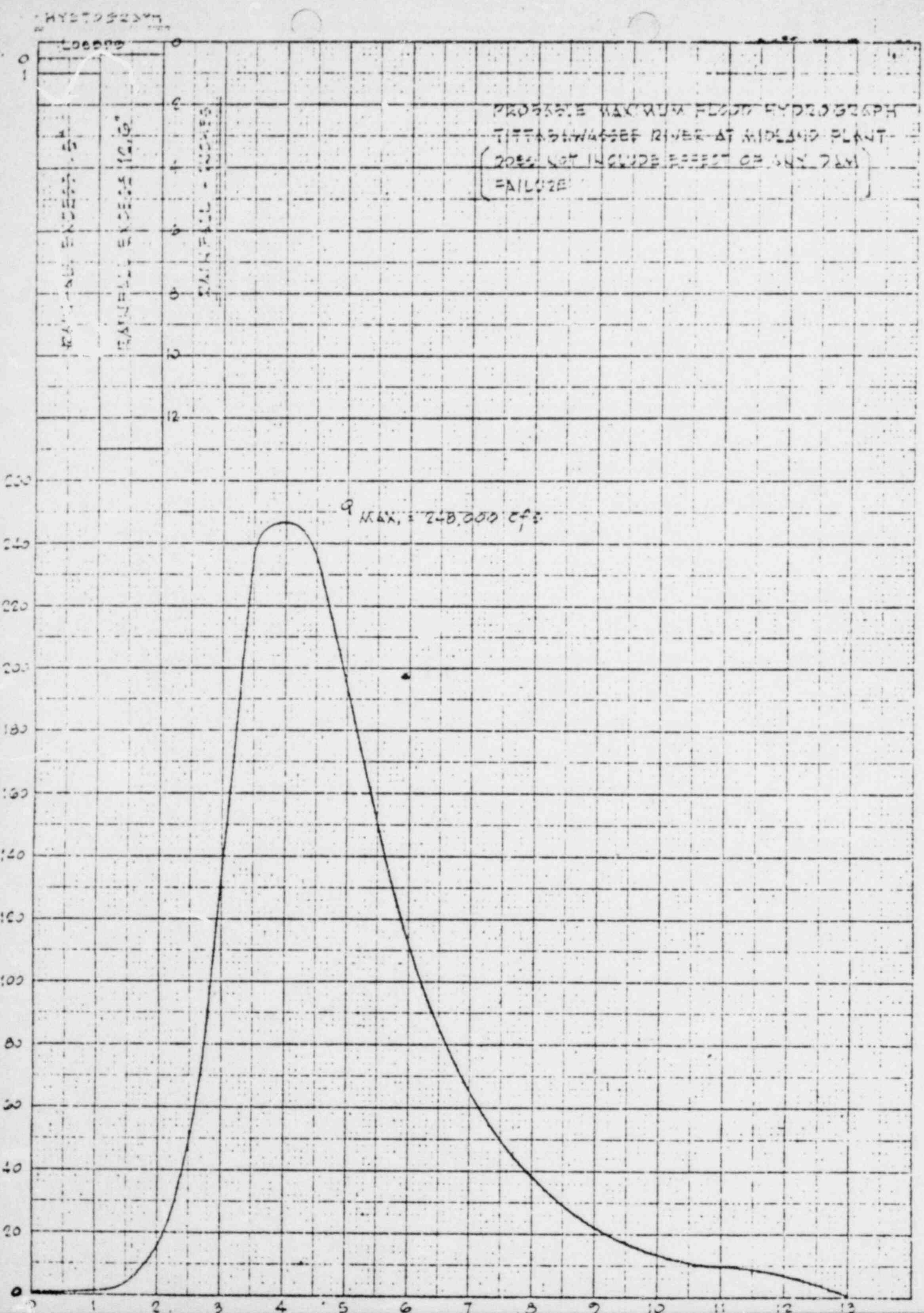
Section No.	Station	Pre-Project W.S. Elevations 34,000 cfs Obsrv.	Pre-Project W.S. Elevations 34,000 cfs Calc.	Pre-Project Manning's "n" LOB	Pre-Project Manning's "n" CH	Pre-Project Manning's "n" ROB	Post-Project W. S. Elevations 55,000	Post-Project W. S. Elevations 75,000	Post-Project W. S. Elevations 100,000	Post-Project W. S. Elevations 170,000	Post-Project W. S. Elevations 270,000
10[1]	0+00	606.59	606.59	0.150	0.027	0.045	608.0	609.0	613.5	619.0	625.0
9	10+00		606.79				608.2	609.3	613.7	619.2	625.2
8	26+50		607.32				609.2	610.6	614.3	619.7	625.6
7	41+50		607.62				609.7	611.3	614.8	620.2	626.1
7.05	52+00		607.70				610.0	611.7	615.0	620.2	625.8
7.1	62+00		608.22				610.6	612.5	615.6	620.8	626.1
6	76+00	608.56	608.39	0.040	0.027	0.045	611.0	613.0	616.2	621.7	627.7
6.05	87+50		608.57				611.2	613.3	616.6	622.4	628.6
6.07	90+50						611.1	613.2	616.4	621.9	628.1
6.1 ds	93+50						611.1	613.1	616.3	621.7	628.0
6.1 us	94+00						611.4	613.7	619.8	623.2	628.0
5	102+50		608.51				611.6	614.0	620.3	624.1	629.1
5.1	121+00		609.03				612.1	614.6	620.7	624.8	630.0
4[2]	144+50	609.10	609.15	0.040	0.027	0.06	612.2	614.8	620.9	625.1	630.4
2[3]	179+50	609.78	609.63								

[1]Smith's Crossing

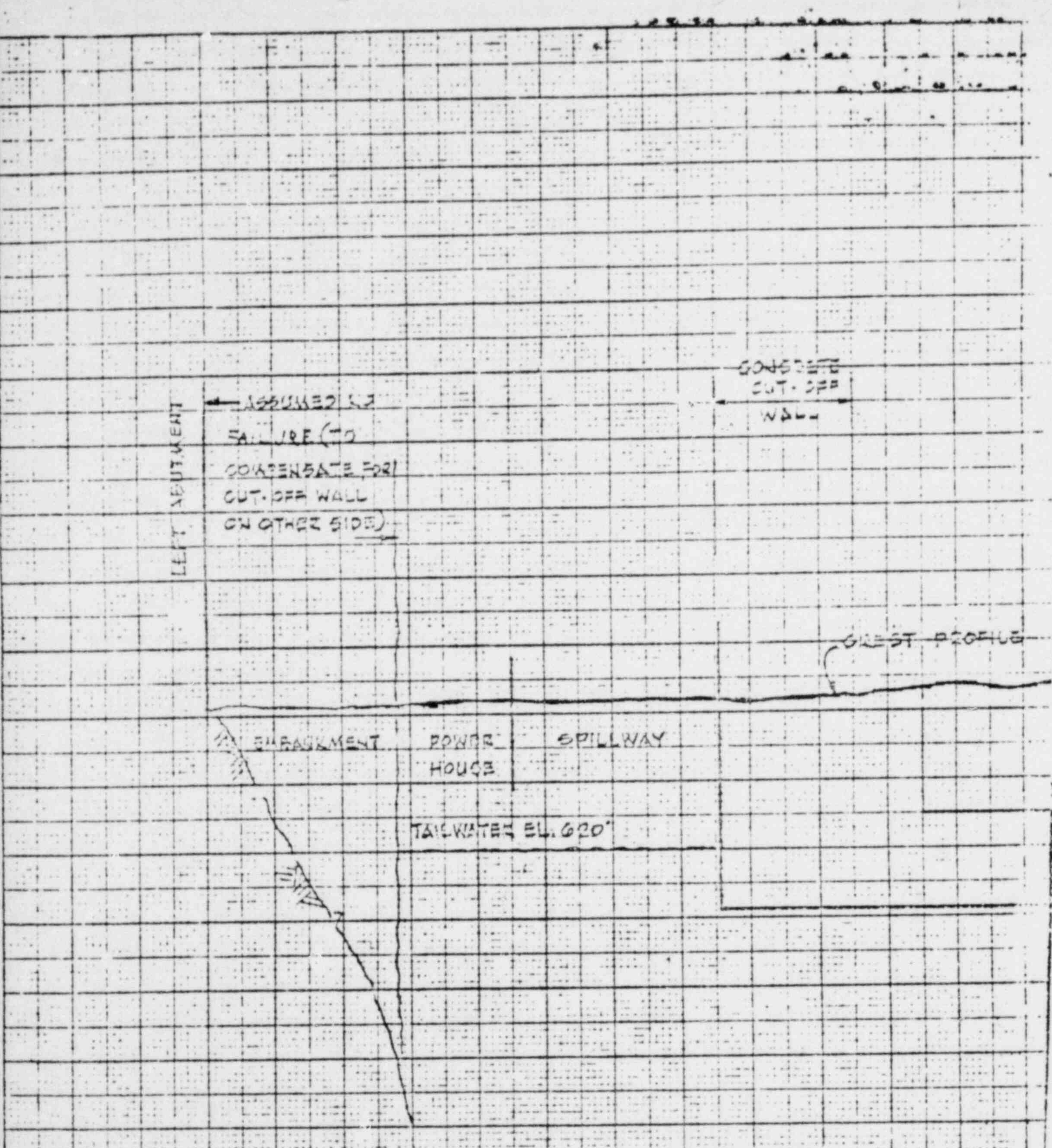
[2]Plant site

[3]USGS gage


DISCHARGE - 1000 cfs
0
1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
KEUFFEL & ESSER CO.



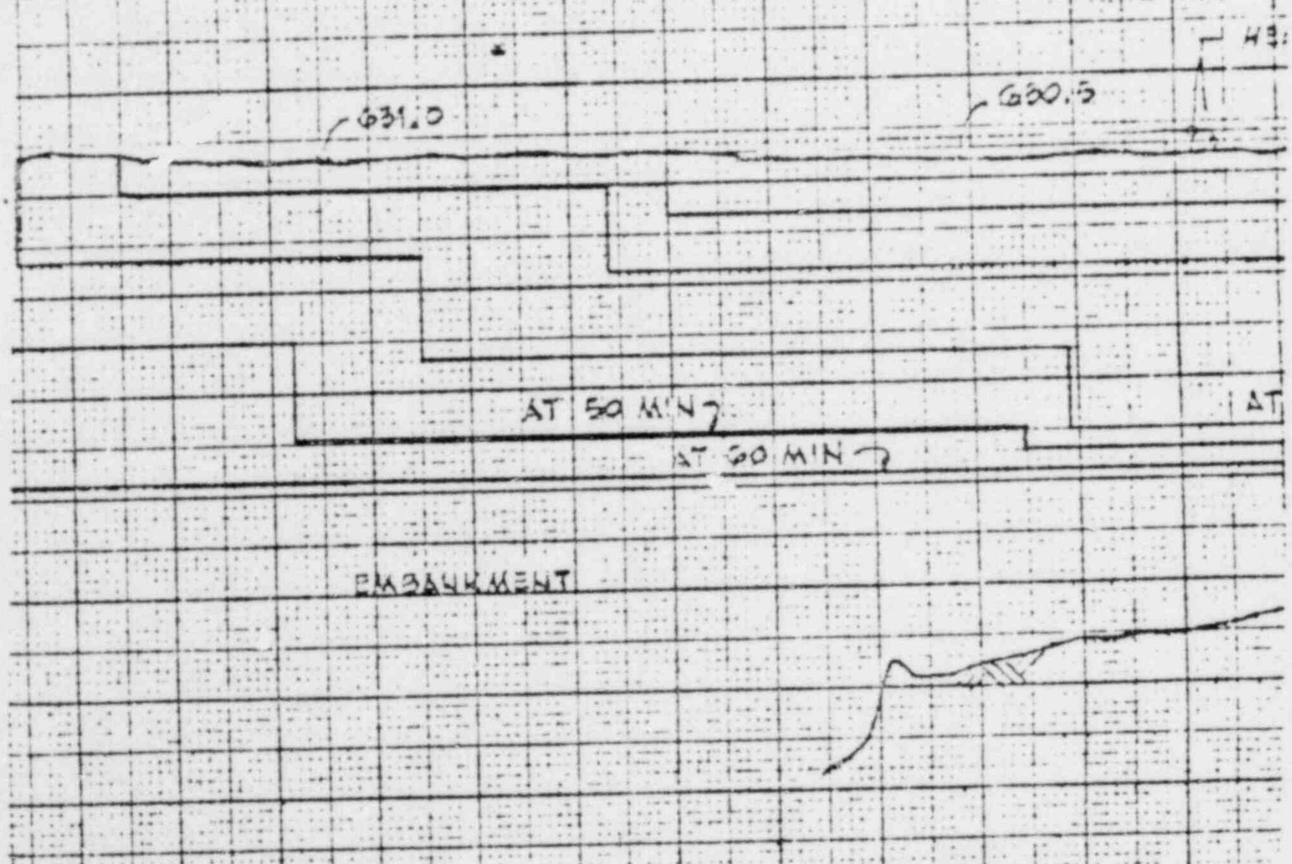
PROBABLE MAXIMUM FLOOD HYDROGRAPH
TITTABAWASSEE RIVER AT MIDLAND PLANT
(DOES NOT INCLUDE EFFECT OF ANY DAM FAILURE)
Q MAX. = 248,000 cfs
0
2
4
6
8
10
12
0
20
40
60
80
100
120
140
160
180
200
220
240
260
0
1
2
3
4
5
6
7
8
9
10
11
12
13



LEFT ABUTMENT
ASSUMED
FAILURE (TO
COMPENSATE FOR
CUT-OFF WALL
ON OTHER SIDE)
CONCRETE
CUT-OFF
WALL
CREST PROFILE
EMBANKMENT
POWER
HOUSE
SPILLWAY
TAILWATER EL. 620'

NOTES

1. WOLVERINE POWER CO. DATUM ABOUT THAN USGS DATUM (W.P. CO. DATUM
2. STATIONING IS EQUAL TO THE REC MARCH 1979 SURVEY.



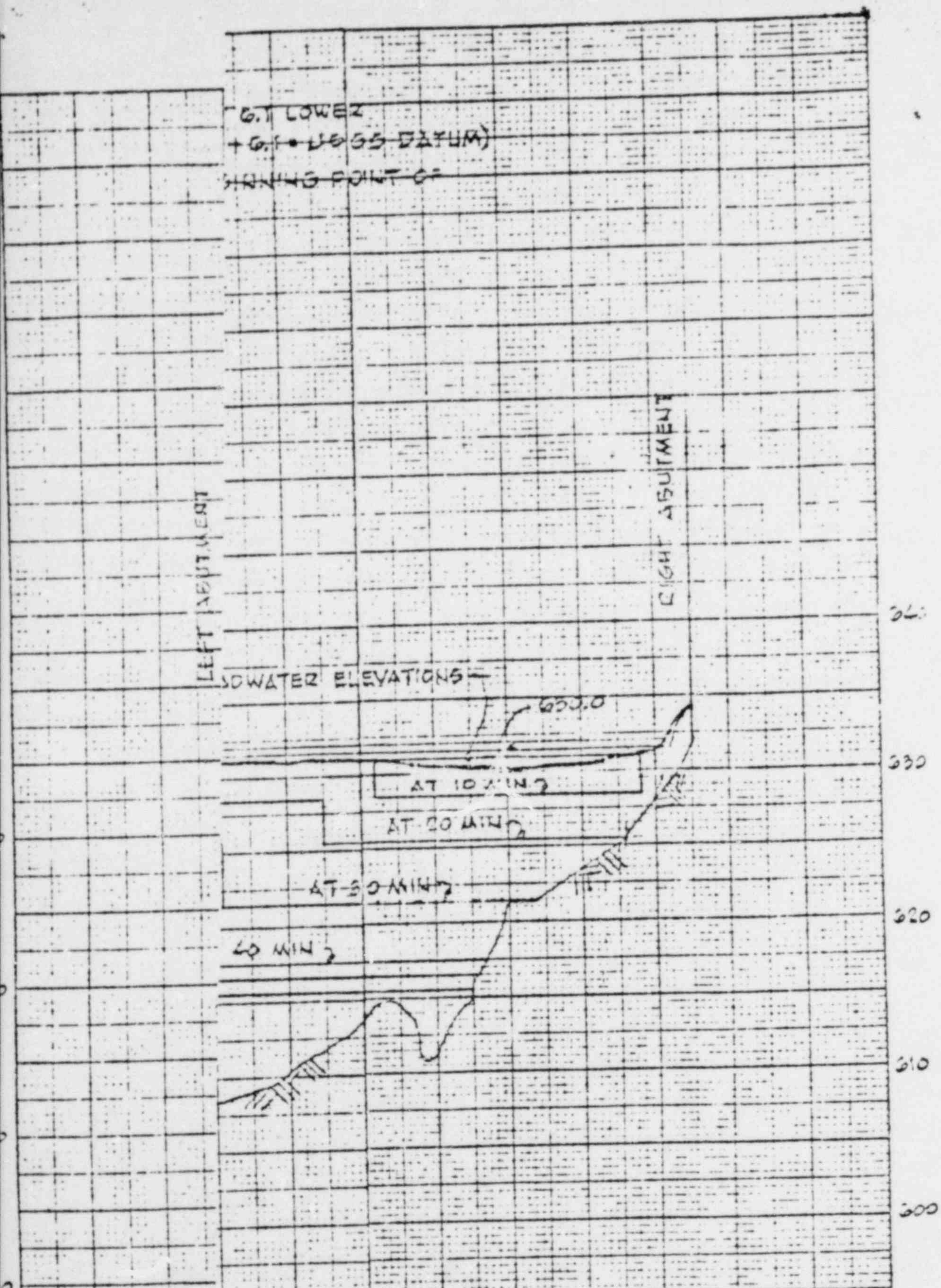
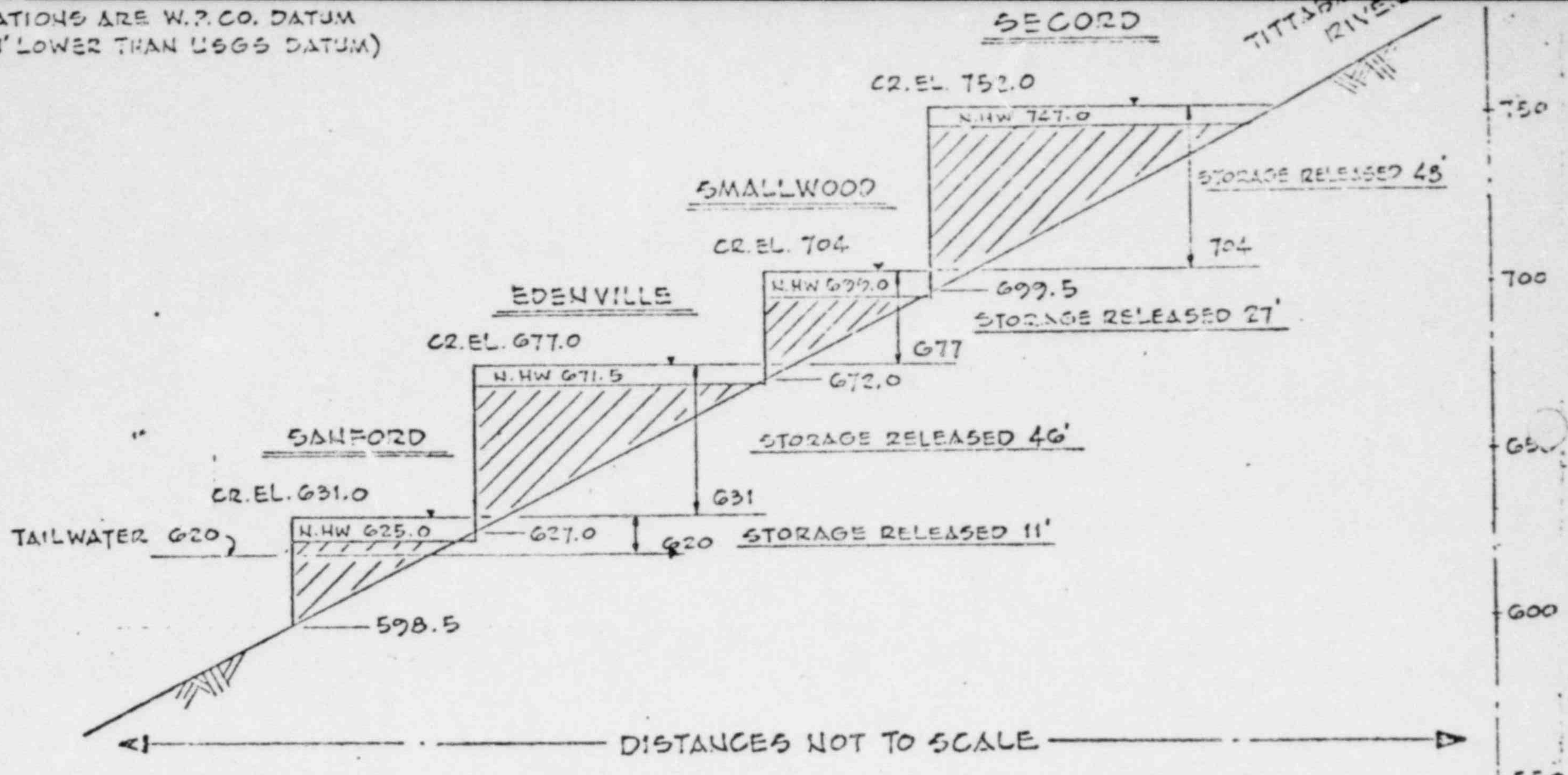


FIGURE - 3

SANFORD DAM
ASSUMED FAILURE MODE
HORIZONTAL 1" = 100'-0"
VERTICAL 1" = 12'-0"

ALL ELEVATIONS ARE W.P.CO. DATUM
(ABOUT 6.1' LOWER THAN USGS DATUM)



Sanford

2115 ac. Surf. Area
× 11' depth
23,265 × 2/3 = 15,510 af

Edenville

3658 ac.
× 46
168,000/2 = 84,000 af

Smallwood

1286 ac
× 27
34,700/2 = 17,350 ac

Secord

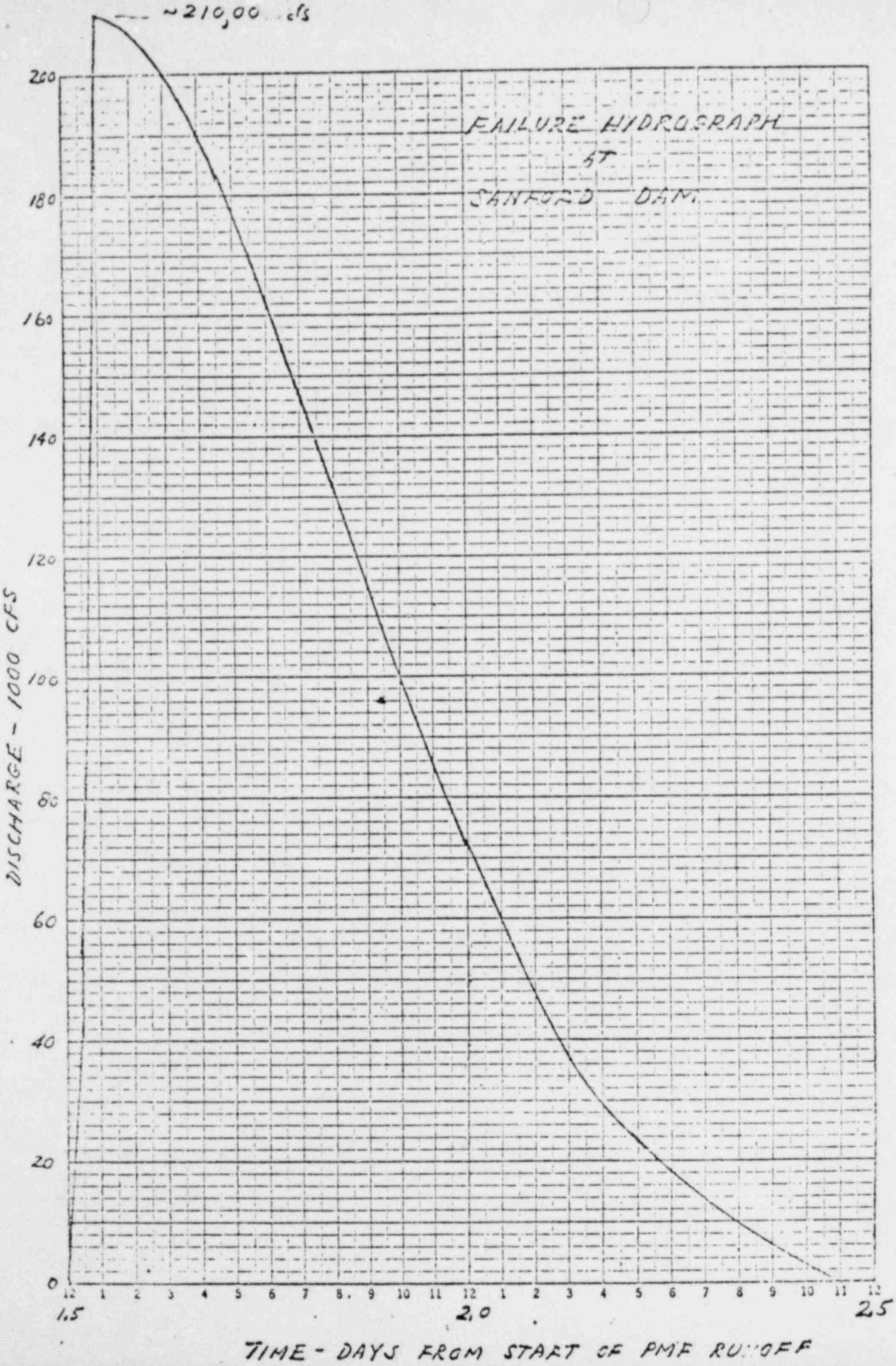
2087 ac
× 48
100,000/2 = 50,000 af

50,000 af
17,350
84,000
15,510

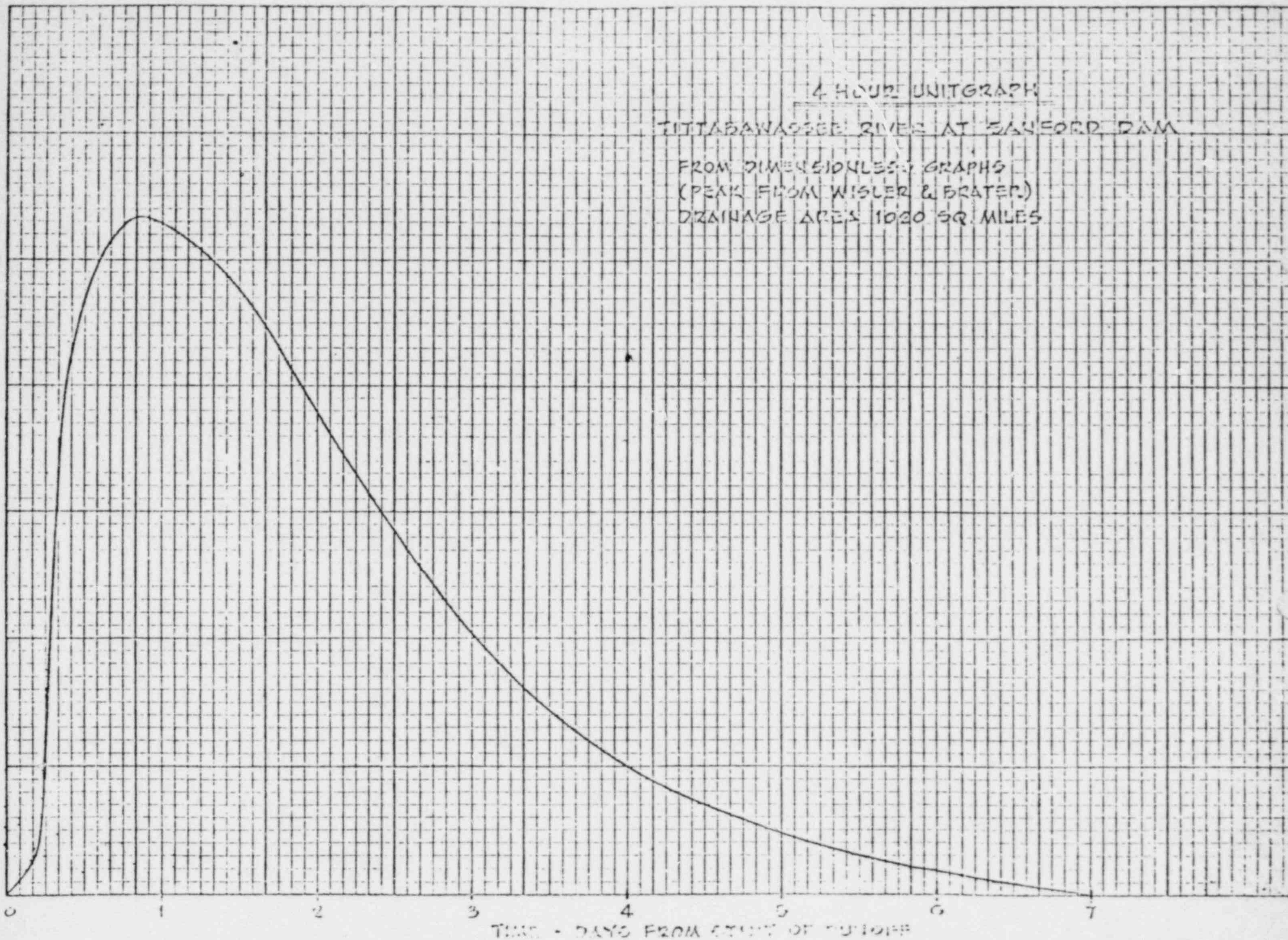
ΣS = 166,860 af released

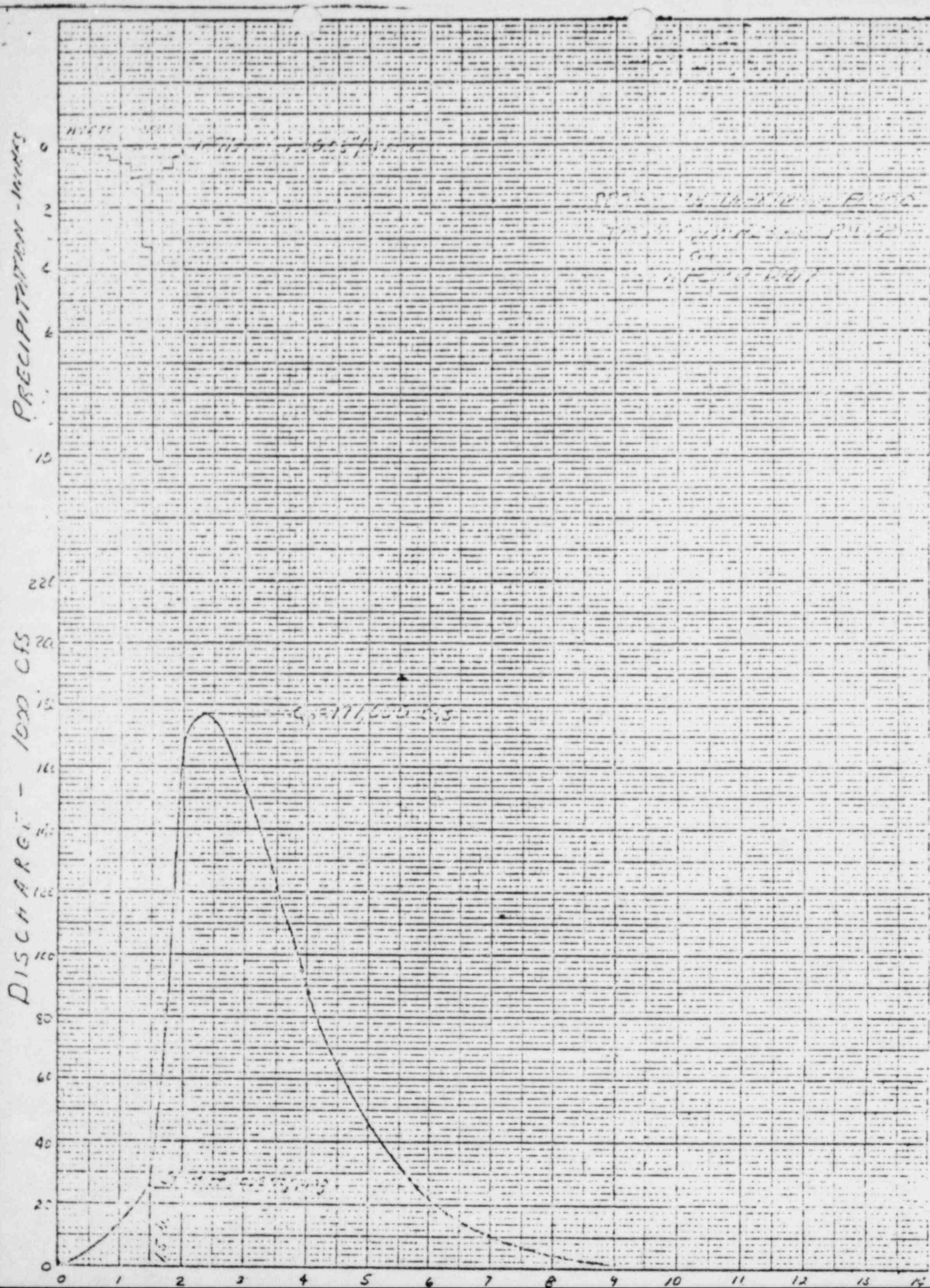
SCHEMATIC OF WOLVERINE DAMS
ON THE
TITTABAWASSEE RIVER ABOVE MIDL

FIG. 4

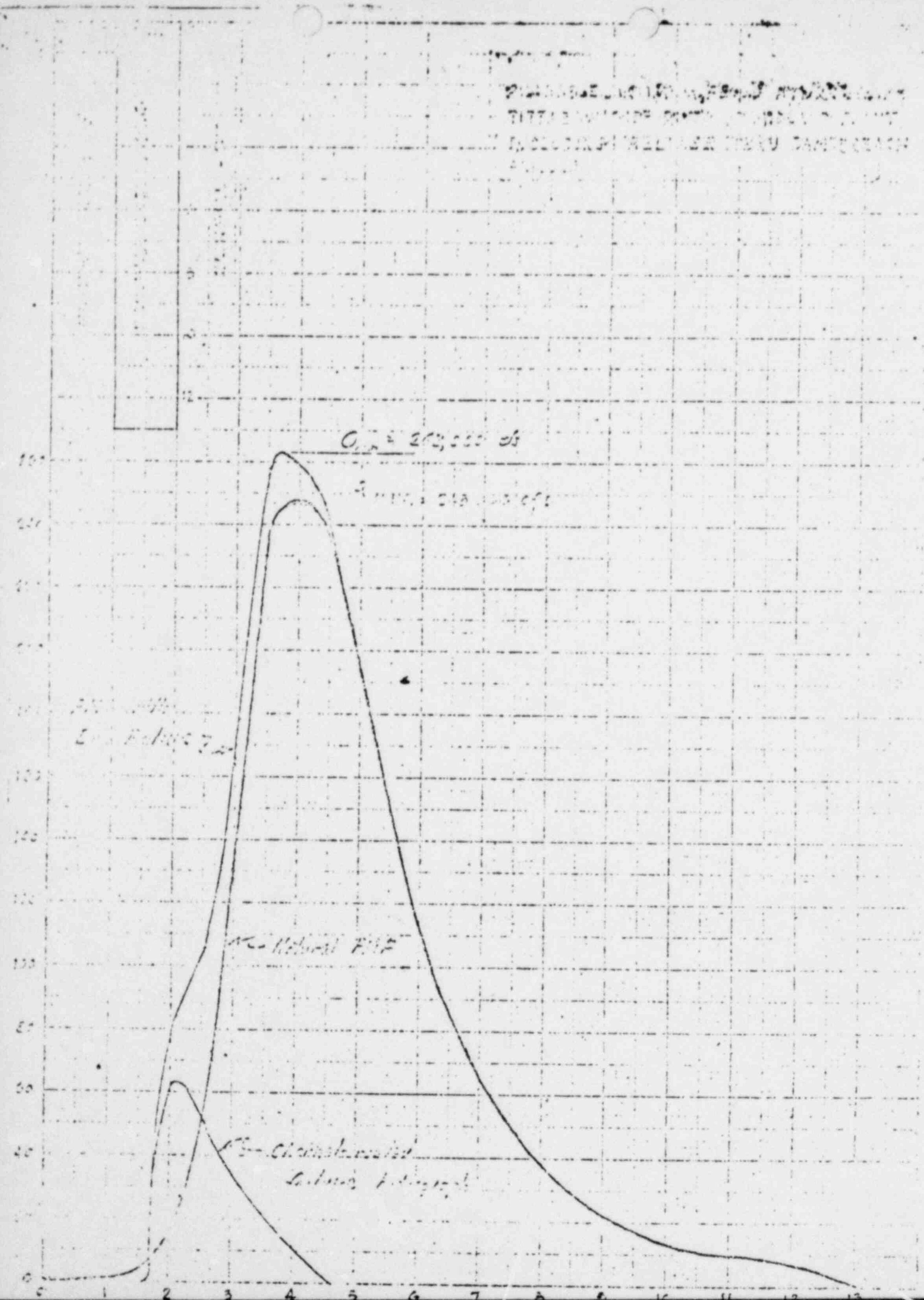


~210,00 cfs
FAILURE HYDROGRAPH
AT
SANFORD DAM
DISCHARGE - 1000 CFS
200
180
160
140
120
100
80
60
40
20
0
12
1
2
3
4
5
6
7
8
9
10
11
12
1
2
3
4
5
6
7
8
9
10
11
12
1.5
2.0
2.5
TIME - DAYS FROM START OF PMF RUNOFF


4 HOUR UNITGRAPH
TITTABAWASSEE RIVER AT SANFORD DAM
FROM DIMENSIONLESS GRAPHS
(PEAK FROM WISLER & BRATER)
DRAINAGE AREA 1020 SQ. MILES
0
1
2
3
4
5
6
7
TIME - DAYS FROM START OF RUNOFF



PRECIPITATION - INCHES
DISCHARGE - 1000 CFS
0
2
4
6
10
20
40
60
80
100
120
200
0
1
2
3
4
5
6
7
8
9
10
11
12
13


0 1 2 3 4 5 6 7 8 9 10 11 12 13
60
40

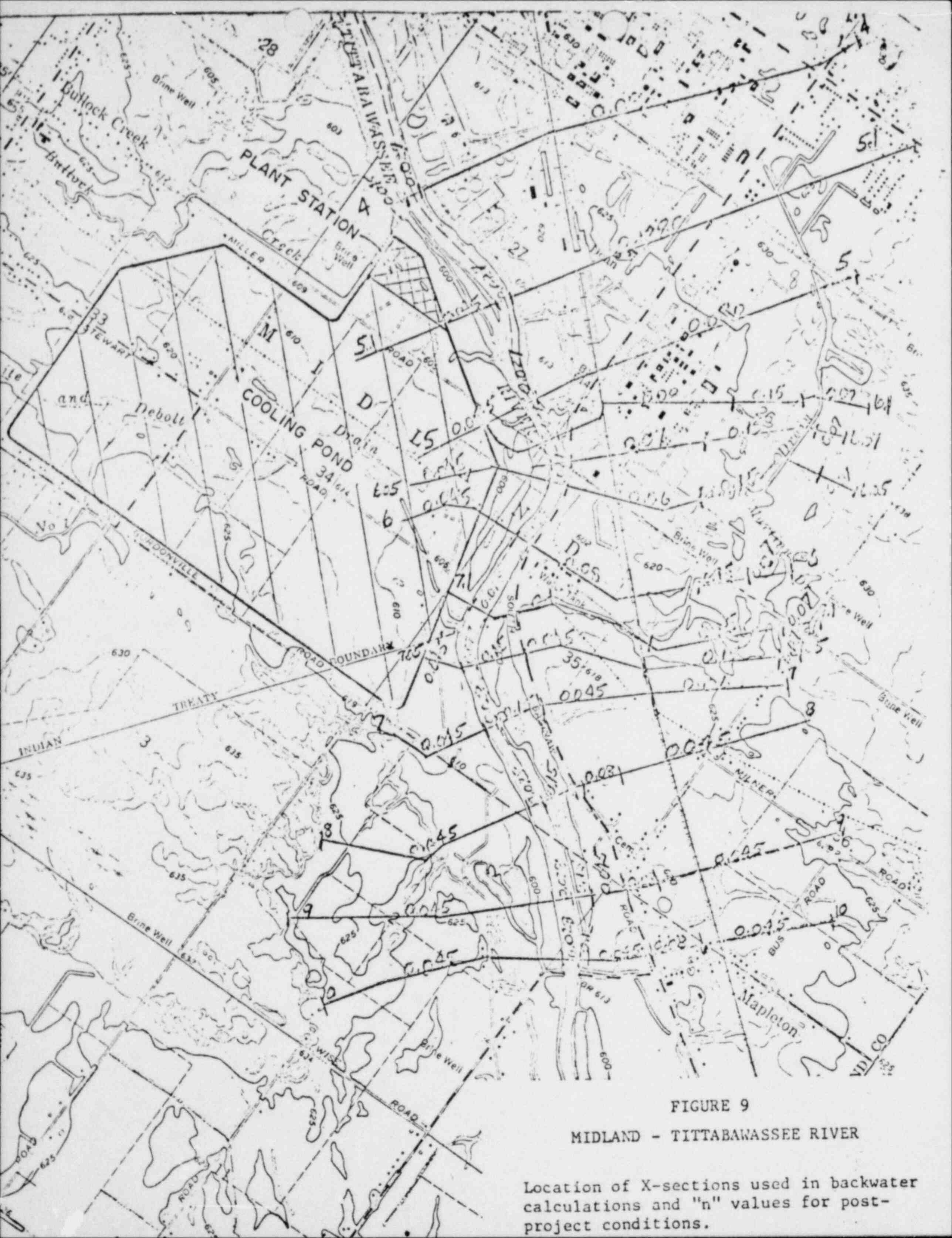


FIGURE 9

MIDLAND - TITTABAWASSEE RIVER

Location of X-sections used in backwater calculations and "n" values for post-project conditions.

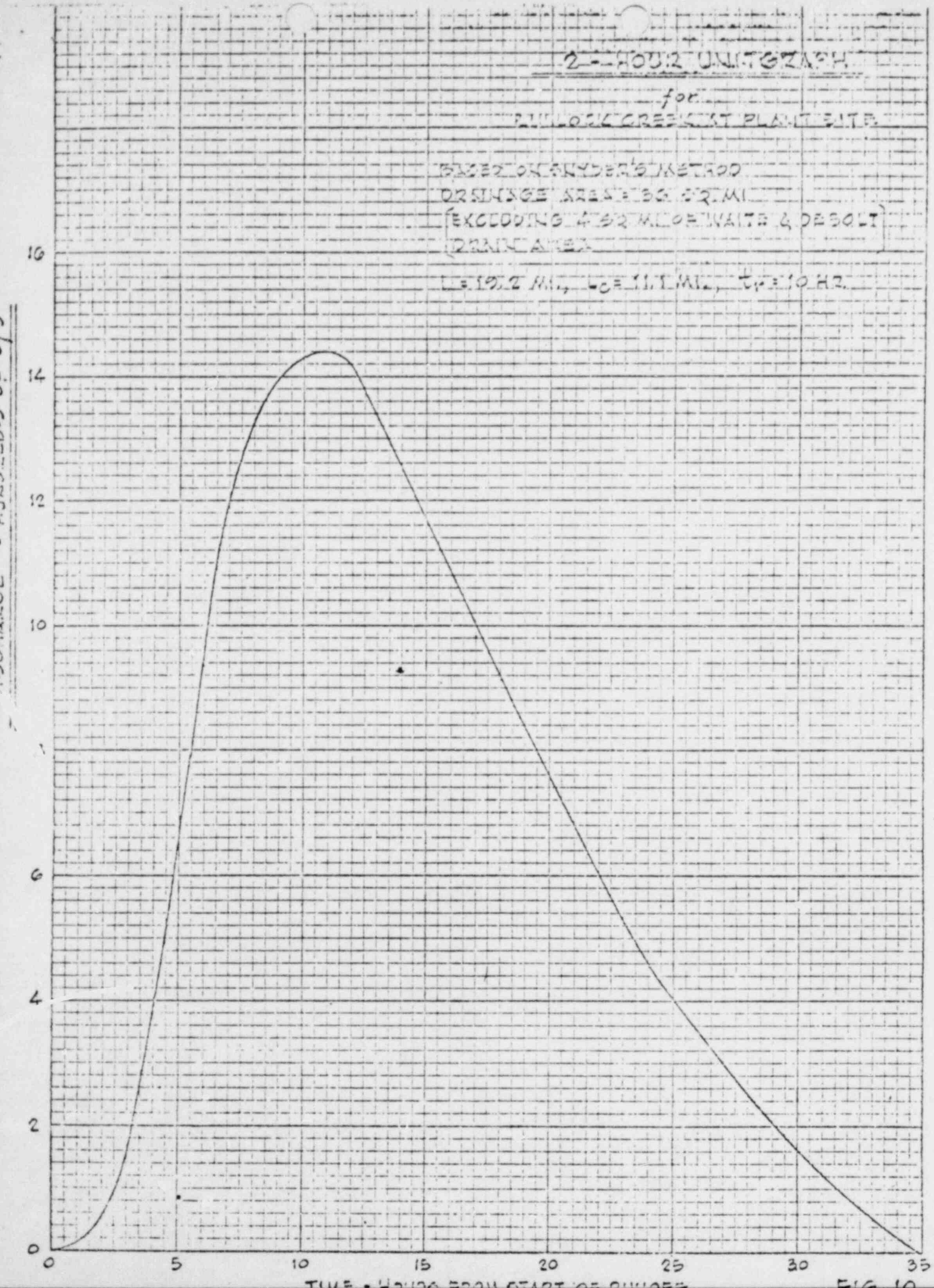
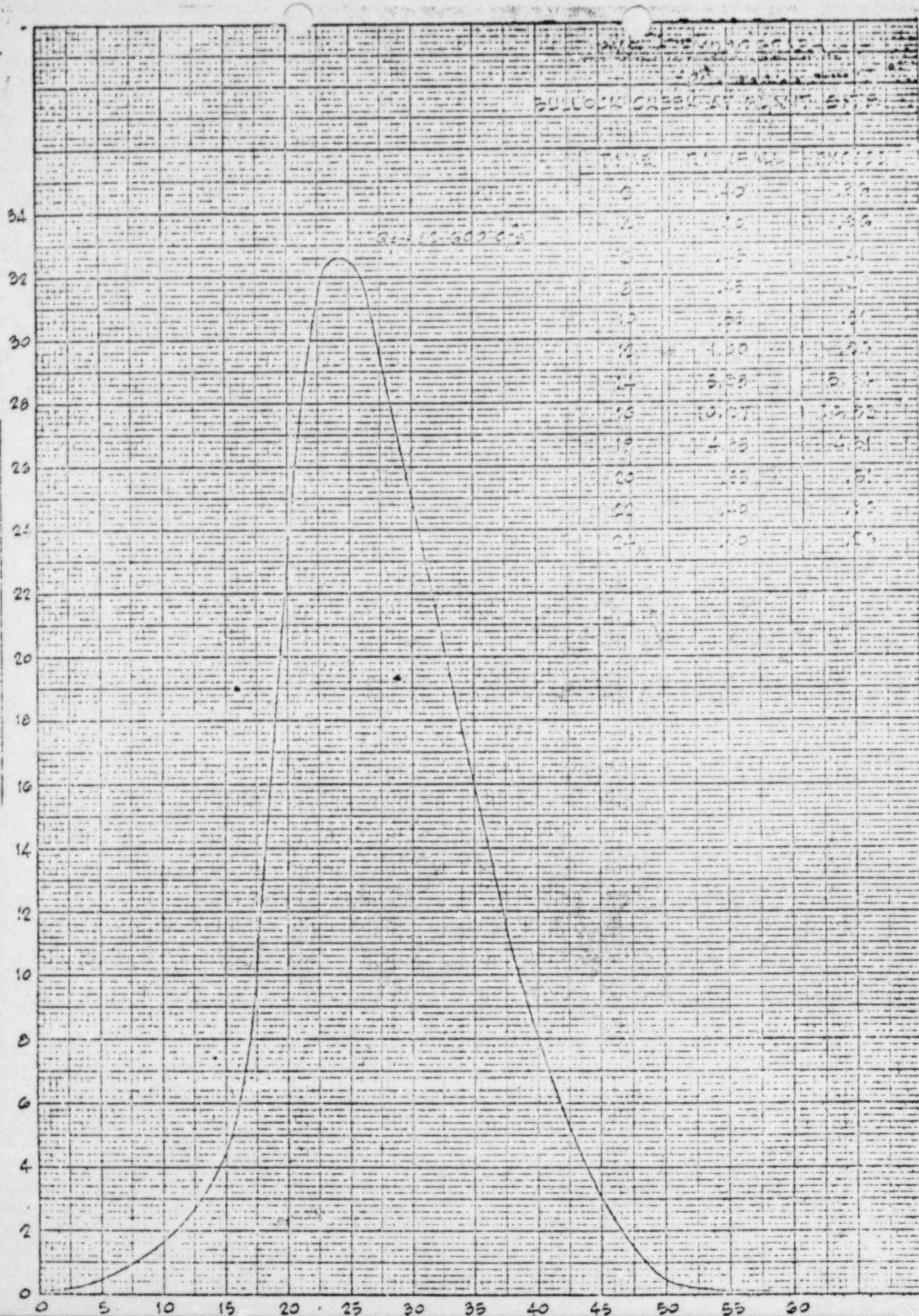
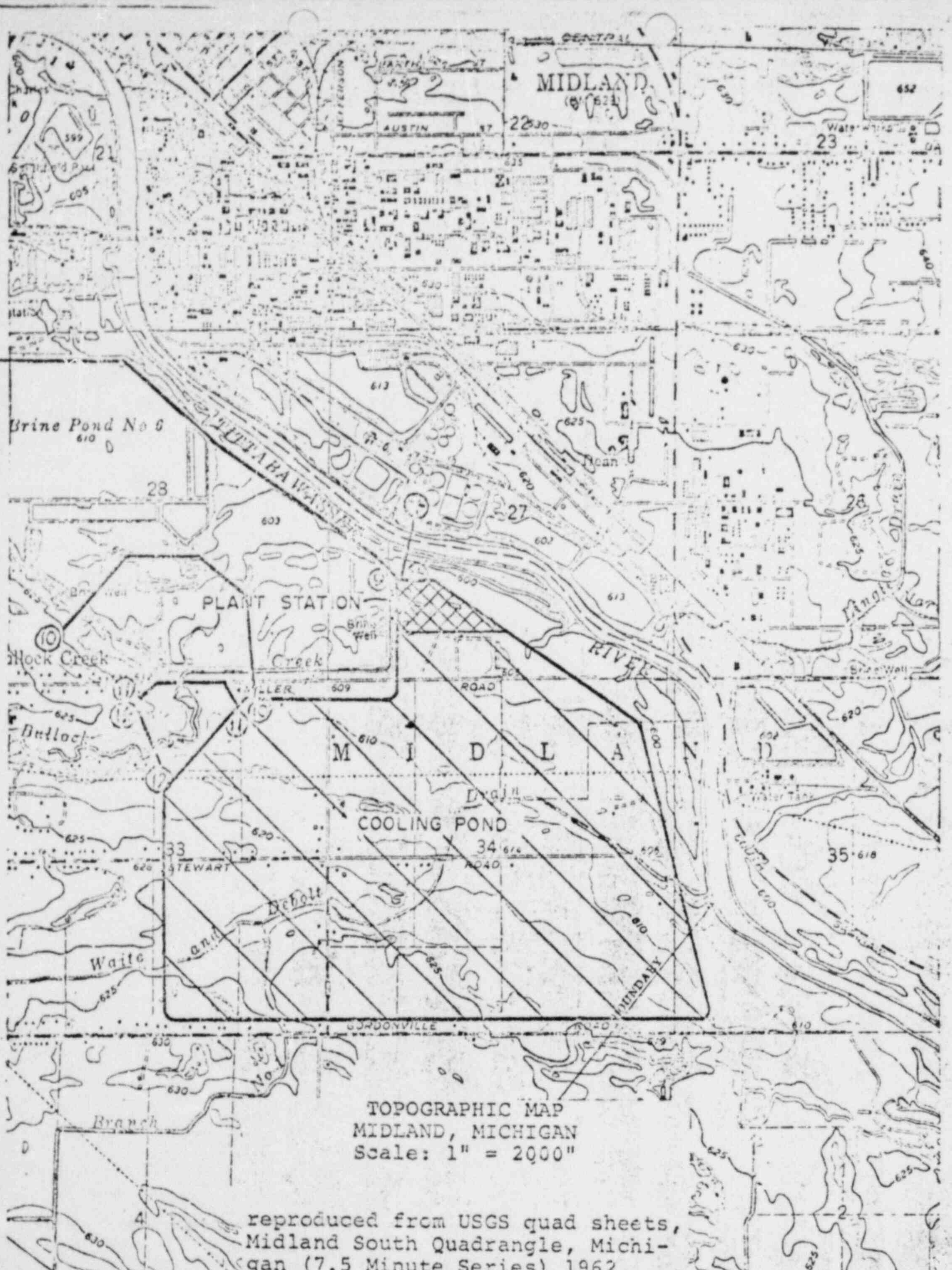

2 - HOUR UNITGRAPH
for
BULLOCK CREEK AT PLANT SITE
BASED ON SNYDER'S METHOD
DRAINAGE AREA = 36 SQ. MI
(EXCLUDING 4 SQ MI. OF WAITE & DEBOLT DRAIN AREA)
L = 19.2 MI., Lc = 11.1 MI., tp = 10 HR.
DISCHARGE - HUNDREDS OF cfs
0
2
4
6
8
10
12
14
16
0
5
10
15
20
25
30
35
TIME - HOURS FROM START OF RUNOFF

FIG. 10



KEUFFEL & ESSER CO.



TOPOGRAPHIC MAP
MIDLAND, MICHIGAN
Scale: 1" = 2000"

reproduced from USGS quad sheets,
Midland South Quadrangle, Michigan (7.5 Minute Series) 1962

FIGURE 12