III - DATA ANALYSIS

Pollution Sources:

The monitoring stations, shown in Exhibit A, reflect a total drainage area of approximately 88 square miles. The drainage area contributing to the five water level tunnel discharges is approximately 12 square miles, or 13.6 percent of the total drainage area. The effect of tunnel discharges on the water quality of the Catawissa Creek is illustrated by the following example:

Monitoring station CC-7 reflects flow conditions from 28.81 squaremile drainage area, below the discharges of the Audenried Tunnel and Water Level Tunnels 2 and 3. Monitoring Station CC-8 is located downstream of CC-7, reflecting flow conditions from 61.75 squaremile drainage area. Flow measurements and acid concentration on 6/29/1978 were 28.81 cfs and 90 ppm, at CC-7. At CC-8, the recorded flow was 47.81 cfs and 56 ppm concentration of acid. The acid load in lbs/day is derived from the following relationship:

Load (lbs/day) = Flow (MGD) x 8.33 x Concentration (ppm)

Consequently, the acid loads on 6/29/1978 were 13,953 lbs/day and 14,407 lbs/day, at CC-7 and CC-8, respectively. Accordingly, the acid load at CC-3 is predominately attributable to the AMD contribution upstream of CC-7. The acid load contributed by the net runoff between the two stations (47.81 cfs - 28.81 cfs = 19 cfs) is 14,407 lbs. - 13,953 lbs. = 454 lbs., which is equivalent to acid concentration of 454/(19 cfs x 0.646 cfs/MGD x 8.33) = 4.4 ppm.

Due to the limitations described in Section II, direct comparison between stations can only be made during periods of no precipitation, assuming that the recorded stream discharges represent the "base flow" of the stream. Flow records, acid concentration and computed acid load for assumed "base flow" conditions at CC-7 and CC-8 are tabulated as follows:

TABLE 4

DATE	Monitoring Sta. CC-7			Monitoring Sta. CC-8		
	Flow (cfs)	Acidity		Flow (cfs)	Acidity	
		Concentr. (ppm)	Load (lbs/day)		Concentr. (ppm)	Load (lbs/day)
8/24/78	17.01	200	18,307	-	-	-
8/25/78	-	-	-	31.58	92	15,634*
11/15/78	18.53	121	12,065	-	-	-
11/17/78	-	-	-	33.29	69	12,361
12/13/78	58.46	45	14,156	-	-	-
12/15/78	-	-	-	105.56	21	11,928*
3/14/79	87.13	40	18,754	-	-	-
3/16/79	-	-	-	173.50	28	26,142
4/18/79	49.55	46	12,265	-	-	-
4/20/79	-	-	-	88.37	27	12,839

** Negative contribution below CC-7*

Collected data from the Tomhicken Creek also indicate the effect of tunnel discharges on the water quality. Monitoring Station TC-2 reflects flow conditions from 16.33 square-mile drainage area, below the discharges of the Oneida (WLT "0") and WLT 1 tunnels. The approximate drainage areas that contribute to the tunnel discharges are 1.17 and 3.07 square miles for WLT "0" and WLT 1, respectively. Flow measurements upstream of TC-2 are available at stations S-1, WLT "0", WLT 1 and TC-4. In the absence of flow records at TC-2, approximate flow rates can be derived by the following relationship:

Flow at S-1 = +/- Flow (S-2 + WLT "0")

Flow at TC-2=+/- Flow (S-1 + WLT-1 + TC-4)

Accordingly, on 11/29/1978, the acid concentration at TC-2 was 24 ppm. Flow rates at S-1, WLT-1 and TC-4 were 8.52, 8.4 and 2.11 cfs, respectively; whereas acid concentration was 23, 24 and 13 ppm, at the aforementioned stations, respectively. Consequently, the total acid load at TC-2, derived for the flow conditions at 11/29/1978, is:

0.646 x 8.33 (8.52 x 23 + 8.4 x 24 + 2.11 x 13) = 2287 lbs/day and the total flow was 19.03 cfs. Therefore, the computed concentration at TC-2, is

$$\frac{2{,}287 \text{ lbs/day}}{0.646\ (8.52 + 8.4 + 2.11)\text{x } 8.33} = \frac{2287}{102} = 22 \text{ ppm}$$

Whereas, the recorded acid concentration at TC-2 is 24 ppm.

Hydrologic Considerations:

Hydrographs of the continuous flow recording stations, consisting of the five water level tunnels and the combined flow from stations CC-4 & CC-5 are presented in Figures 3 thru 8. Water quality parameters, consisting of pH, Acidity, Total Iron, Sulfates and the computed acid load are plotted as a function of flow, for each of the aforementioned stations. Due to the wide scatter of the plots, points representing winter sampling are designated by (W) and those representing the Fall season are designated by (F). Similarly, the spring and summer test results are shown as (S) and (0), respectively. Precipitation over the study area, based on records obtained from the continuous recording rain gage at the Rod and Gun. Club, is presented in the form of Hyeto-graph (see Figures 3 thru 8). Mass curves, representing accumulative flow at the continuous recording stations, are compared to volume of precipitation and acid load for the 384-day sampling period.

The accumulative combined flow at Stations CC-4 & CC-5 include the discharges from WLT3. For the 384 days of records, this combined flow is approximately 1625 MG (million gallons), or the equivalent of 4987 acre-feet, representing the total runoff from the 3.72 squaremile watershed. During the same period, the total precipitation over the study area was 54.5 inches. Consequently, the total runoff for the period is equivalent to 12 x 4987/(3.72.x 640) = 25 inches. During the same period, the accumulative flow from WLT 3 was 330 MG (see Figure 7). Therefore, the flow attributable to the total runoff, excluding the tunnel's flow, is 1625 MG - 330 MG = 1295 MG (3974 ac-ft) and the net drainage area, excluding the drainage area contributing to the tunnel, is 3.72 - 0.65 = 3.07 square miles. Consequently, the net runoff for the 384 days of record was 12 x 3974/3.07 x 640 = 24.3 inches, or approximately 44.5 percent of the total precipitation. For the same period, the tunnel discharges are equivalent to 29.2 inches of runoff, or approximately 53.6 percent of the total precipitation. The equivalent runoff of the water level tunnel discharges are summarized below:

TABLE 5

Tunnel	Drainage Area Sq. Miles	Accummulative Discharge			% OF TOTAL PRECIP*
		MG	AC-FT	INCH	
WLT 3	0.65	330	1,013	29.22	53.6
Audenried	6.45	4,200	12,890	37.47	68.8
WLT 2	1.00	400	1,228	23.03	42.2
WLT 1	3.07	1,280	3,928	23.99	44.0
Oneida	1.17	750	2,302	36.89	67.7
Total	12.24	6,960	21,361		

* *Total Precipitation = 54.5 inches at the Gun Club.*

The tabulated values indicate that the total discharges from the water level tunnels represent 60 percent of the total precipitation compared to 44.5 inches for a "normal" watershed. Assuming that the runoff from the sub-drainage areas, excluding tunnel discharges, is similar to that represented by the combined flow at CC-4 and CC-5, the effect of the tunnel discharges increase the flow at locations CC-8 and TC-1 from 24.3 inches to 28.9 and 29.1 inches, respectively.

The contribution of each coal basin to the AMD pollution in the receiving streams is tabulated on the following page (page 15).

TABLE 6

Coal Basin	Tunnels	TOTAL DISCHARGE MG	Acid Load Lbs.	Mean Acid Concentration (ppm)
Jeansville	Audenried	4,200	5,900,000	169
South	WLT #1	(1,280)	(360,000)	(34)
Green	WLT #2	(400)	(170,000)	(51)
Mountain	WLT #3	(330)	(65,000)	(24)
		2,010	595,000	36
N. Green Mt.	Oneida	750	520,000	83
Total		6,960	7,015,000	(121)

Out of the total AMD emanating from the abandoned mines, 84% is contributed by the Jeansville Basin through the Audenried Tunnel discharges. The balance of 16% are near-equally divided between the South and the North Green Mountain Coal Basins.

The hydrologic consequence of past mining in the study area watersheds is further demonstrated in Figures 9 thru 14. Flow rates and water quality parameters are plotted from the headwaters of each watershed to the downstream study limits for the highest day of acid concentration, highest acid load and the computed mean acid concentration during the study period. The expected acid and iron concentration after neutralization by limestone treatment and the reduction in acid load, should the pH be raised to 6.5, are indicated by dashed lines in Figures 9 through 14. The projected alkalinity load is not shown. However, at a pH greater than 6.0 for these waters, the alkalinity will exceed the acidity and meet the "non-acid discharge" requirements of the Departments Rules and Regulations.

The mine workings affect the precipitation-discharge relationship by providing a storage effect and causing an increase of both direct

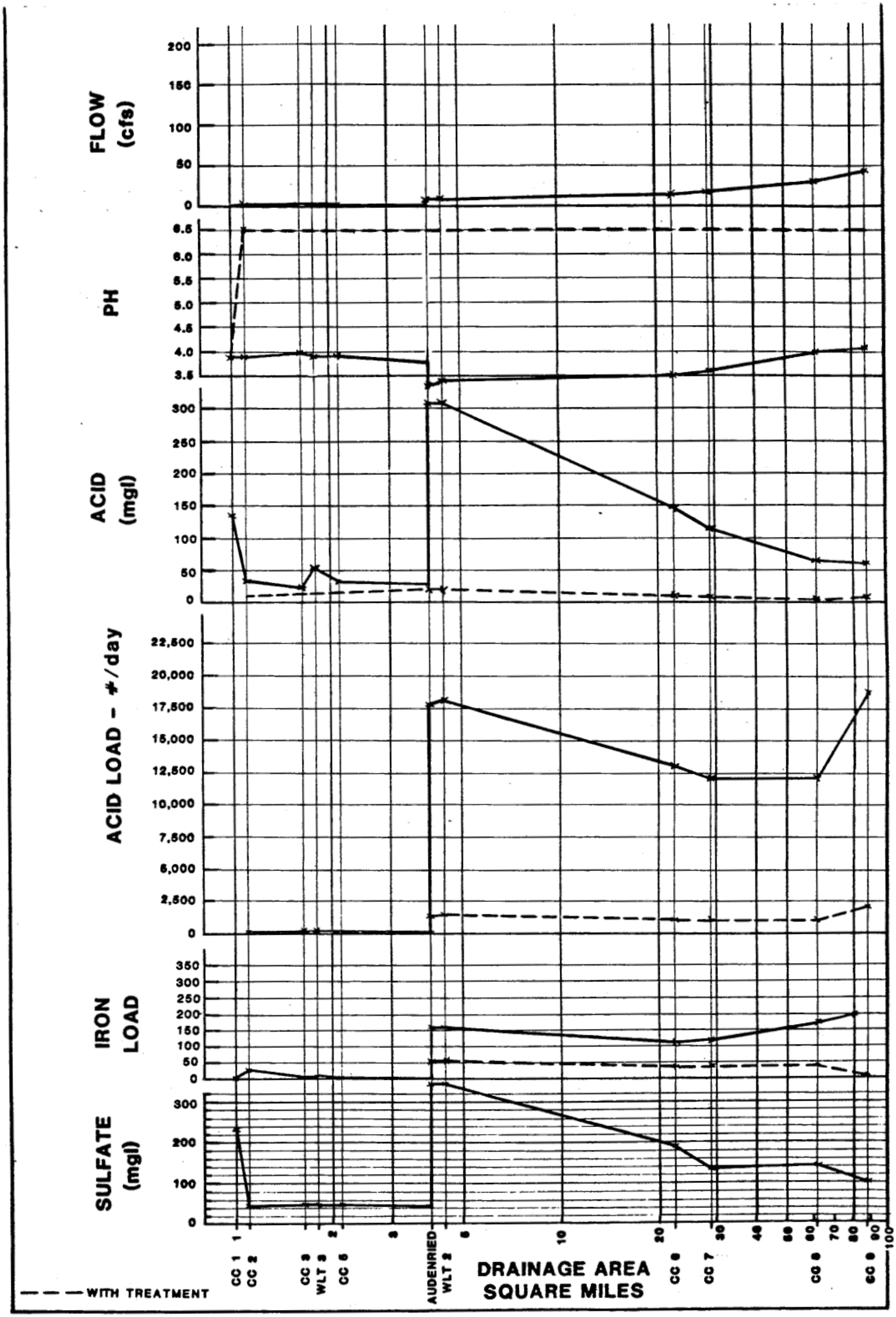

FLOW (cfs)
PH
ACID (mgl)
ACID LOAD - #/day
IRON LOAD
SULFATE (mgl)
DRAINAGE AREA SQUARE MILES
CC 1
CC 2
CC 3
WLT 3
CC 5
AUDENRIED
WLT 2
CC 6
CC 7
CC 8
CC 9
WITH TREATMENT

Figure 9

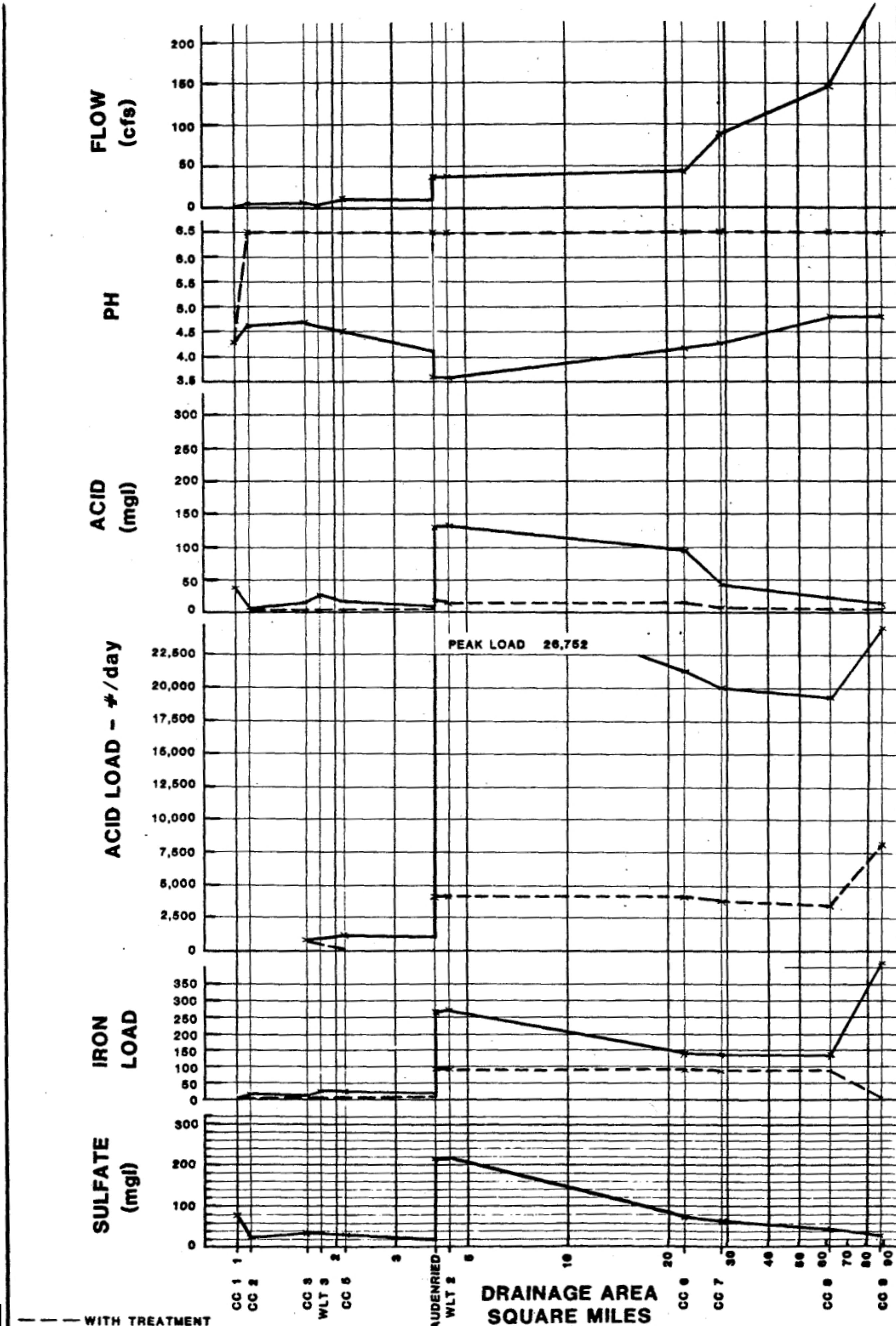

FLOW (cfs)
PH
ACID (mgl)
ACID LOAD – #/day
PEAK LOAD 26,752
IRON LOAD
SULFATE (mgl)
DRAINAGE AREA SQUARE MILES
CC 1
CC 2
CC 3
WLT 3
CC 5
AUDENRIED
WLT 2
CC 6
CC 7
CC 8
CC 9
WITH TREATMENT

Figure 10

Figure 11

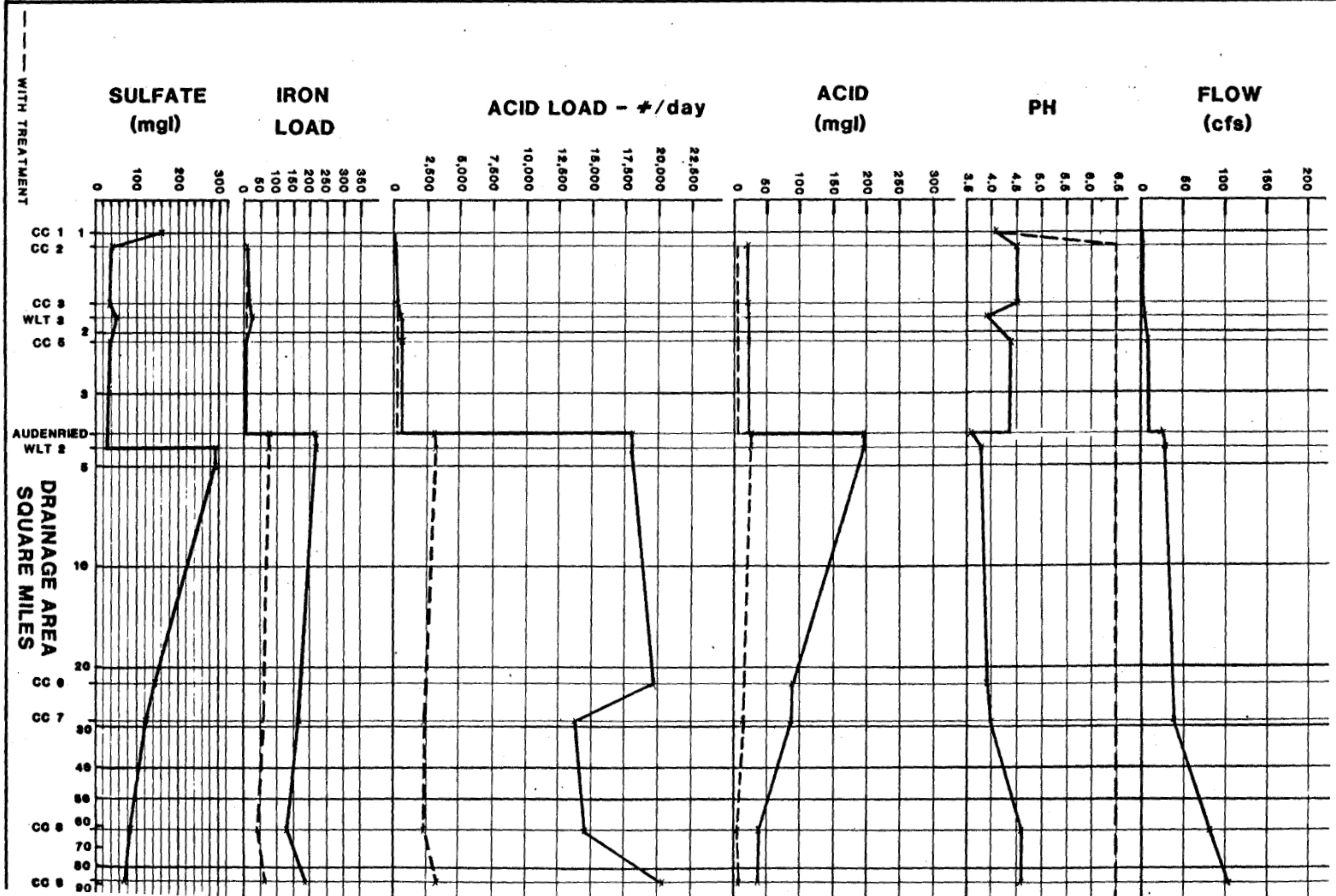


FIGURE 11

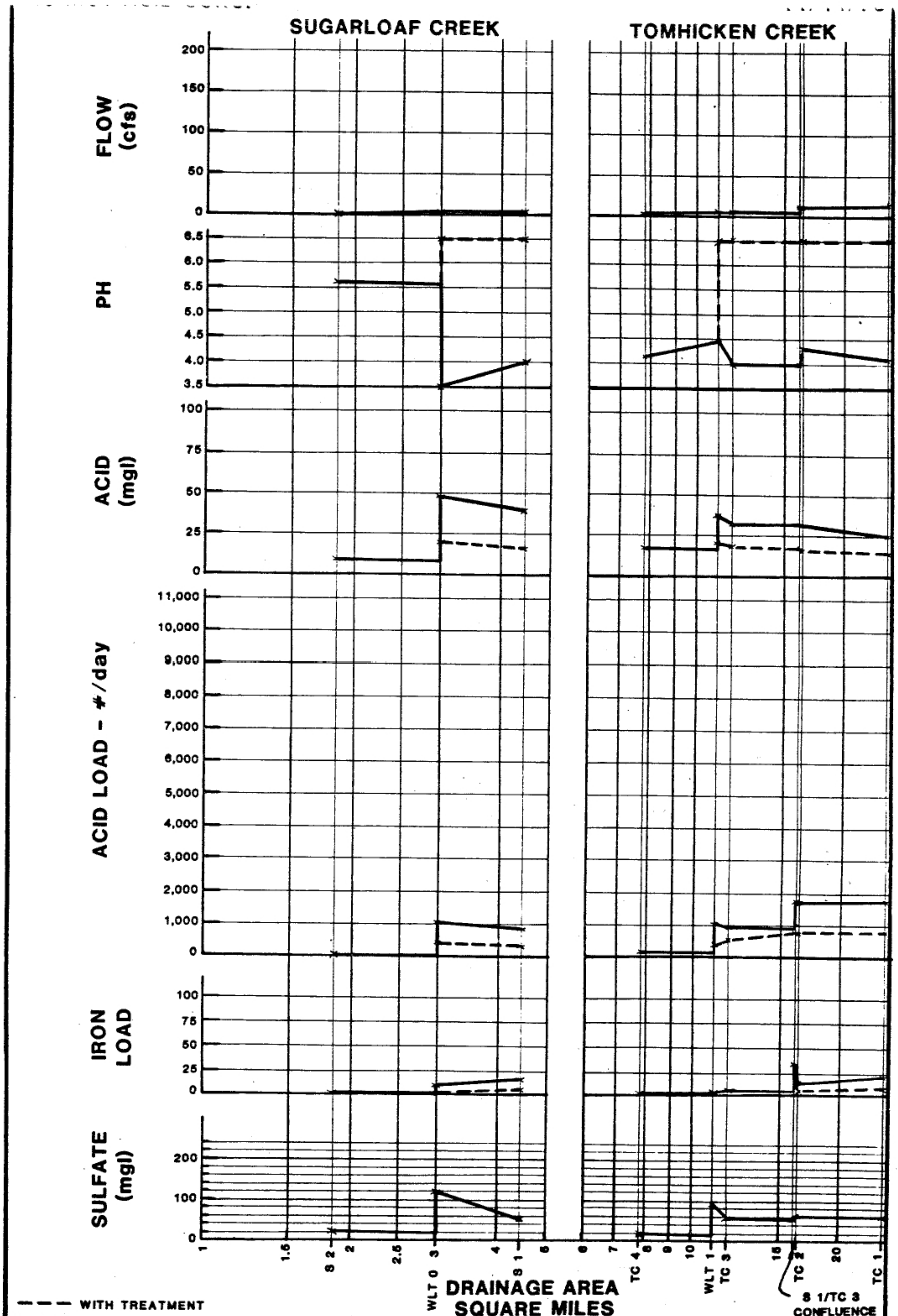

SUGARLOAF CREEK
TOMHICKEN CREEK
FLOW (cfs)
200
150
100
50
0
PH
6.5
6.0
5.5
5.0
4.5
4.0
3.5
ACID (mgl)
100
75
50
25
0
ACID LOAD - #/day
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
IRON LOAD
100
75
50
25
0
SULFATE (mgl)
200
100
0
1
1.5
2
2.5
3
4
5
6
7
8
9
10
15
20
S 2
WLT 0 3
S 1
TC 4
WLT 1
TC 3
TC 2
TC 1
S 1/TC 3 CONFLUENCE
DRAINAGE AREA SQUARE MILES
WITH TREATMENT

Figure 12

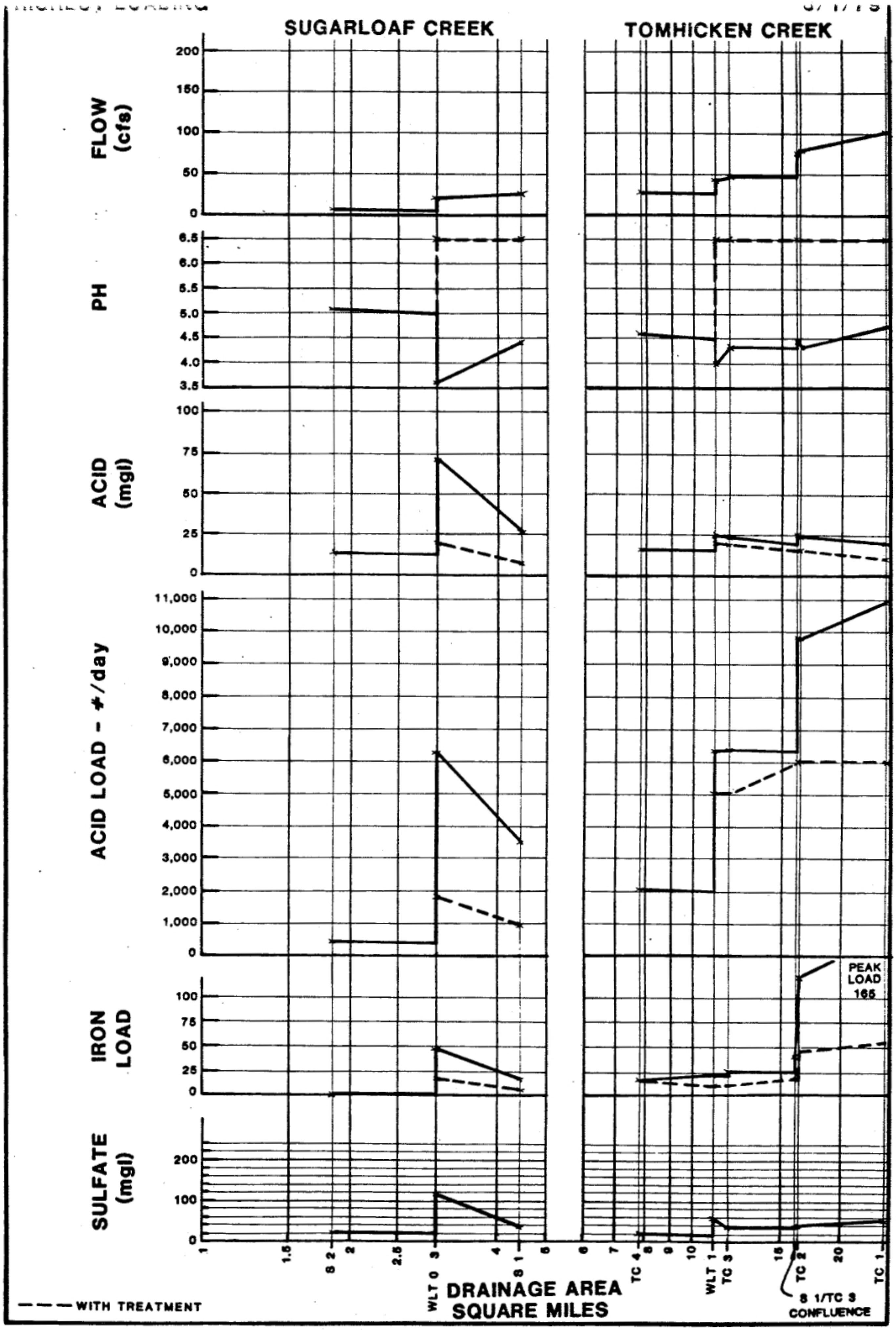

SUGARLOAF CREEK
TOMHICKEN CREEK
FLOW (cfs)
PH
ACID (mgl)
ACID LOAD - #/day
IRON LOAD
SULFATE (mgl)
PEAK LOAD 165
DRAINAGE AREA SQUARE MILES
S 2
WLT 0
S 1
TC 4
WLT 1
TC 3
TC 2
TC 1
S 1/TC 3 CONFLUENCE
WITH TREATMENT

Figure 13

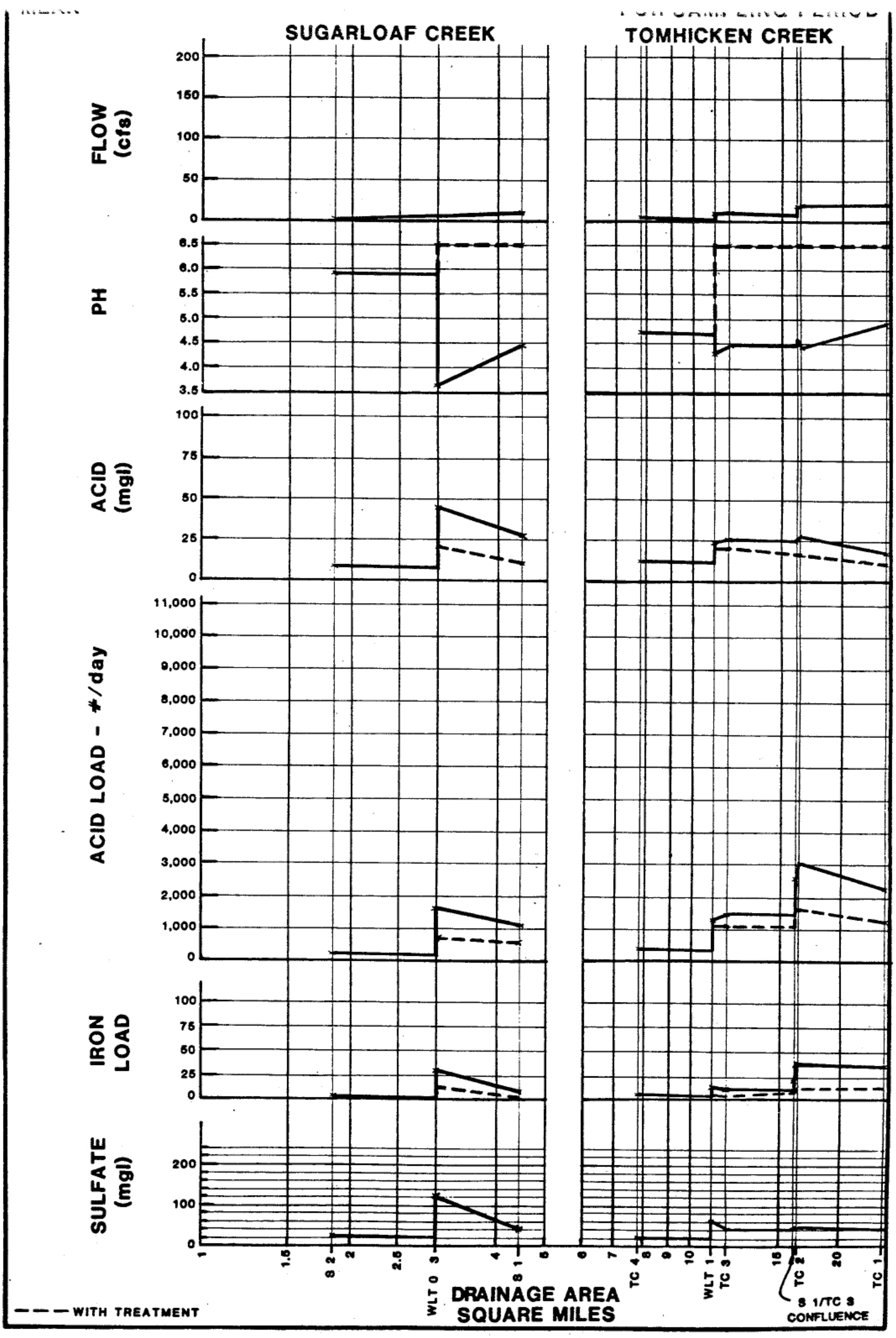

SUGARLOAF CREEK
TOMHICKEN CREEK
FLOW (cfs)
PH
ACID (mgl)
ACID LOAD - #/day
IRON LOAD
SULFATE (mgl)
DRAINAGE AREA SQUARE MILES
S 2
WLT 0 3
S 1
TC 4
WLT 1
TC 3
TC 2
TC 1
S 1/TC 3 CONFLUENCE
WITH TREATMENT

Figure 14

surface inflow and infiltration rates. This is demonstrated by comparing precipitation and discharge ratios for a tunnel and an un-mined watershed of a similar size. The cumulative tunnel discharges represent 60% of the total precipitation for the study year, while stream runoff from un-mined areas discharge approximately 40% of the recorded precipitation.

The storage effect of the mine workings is evidenced by the long lag times observed for the tunnel discharge hydrographs. As a consequence, the frequency of the tunnel discharges is a function of the total volume of precipitation within a period of time, approximately equal to twice the observed lag time. The situation is analogous to a spillway hydrograph of a reservoir with large storage compared to its drainage area. It is also seen in hydrographs for watersheds underlain by cavernous limestone. Examination of the stage recorder charts for high intensity storms indicate the effect of each recharge mode. As shown in Appendix D, a high intensity storm at Audenried produced a triple peaked hydrograph. The peaks corresponded to local runoff between the tunnel portal and the gage, direct surface runoff into the mine and infiltration; with the infiltration causing the highest peaks. This explains the fact that the highest observed peak discharges were associated with snow melt, rather than large storms.

Hydrologic analysis, presented in Appendix D, indicates that 48-hour precipitation events correlate well with the observed peaks from the Audenried Tunnel. A plot of 48-hour precipitation for 2.33, 10 and 100-year recurrence interval on Gumbels Extreme Probability Chart indicated a linear relationship between precipitation and the recurrence interval. Consequently, it is reasonable to assume that a plot of the resulting peak: discharges on Gumbels Extreme Probability Charts will also have linear relationship with the recurrence intervals. Since the above relationship is strictly applicable to rainfall events (excluding snow melt), the selected mean annual peak discharge ($Q_{2.33}$ = 44 cfs, see Appendix D) was the maximum discharge of the Audenried Tunnel during the study period, that was produced strictly due to rainfall. The maximum discharge derived fro a 100-year storm event at the Audenried Tunnel is 85 cfs.

Available records indicate that the largest peaks were associated with rainfall over antecedent snow cover (January 25, 1979), snow melt (Spring thaw) and a tropical storm (May 25-30, 1979). During the 1978-1979 study period, the peak discharge at Audenried resulting from approximately 2.5 inches of precipitation with snow and ice thaw, was 62 cfs. The highest recorded peak discharge during the previous sampling periods (GCF & C Report) was 70 cfs (the latter is also attributable to rain and spring thaw).

The frequency of combined rainfall-snow melt was estimated by comparison with the nearest USGS stream gaging station (Trexler Run at Ringtown). It was assumed that the conditions causing high flow rates during winter and spring thaw would be similar for both locations. Therefore, a frequency analysis was made for Trexler Run using only peak flows during the months of December through April. Fifteen years of records were available for Trexler Run and a summary of the analysis is tabulated below:

RANK M	DATE	FLOW (cfs)	RETURN PERIOD (Gringorten *)
1	04/02/70	73	27.0
2	01/26/76	63	9.7
3	01/24/79	62	5.0
4	01/26/78	55	-
5	12/21/73	53	
6	03/19/75	49	
7	12/08/74	48	
8	03/04/77	44	
9	03/03/72	33	
10	02/13/71	31	
11	03/21/80	28	
12	04/22/79	25	
13	02/13/66	22	
14	03/11/67	14	
15	03/23/68	12	

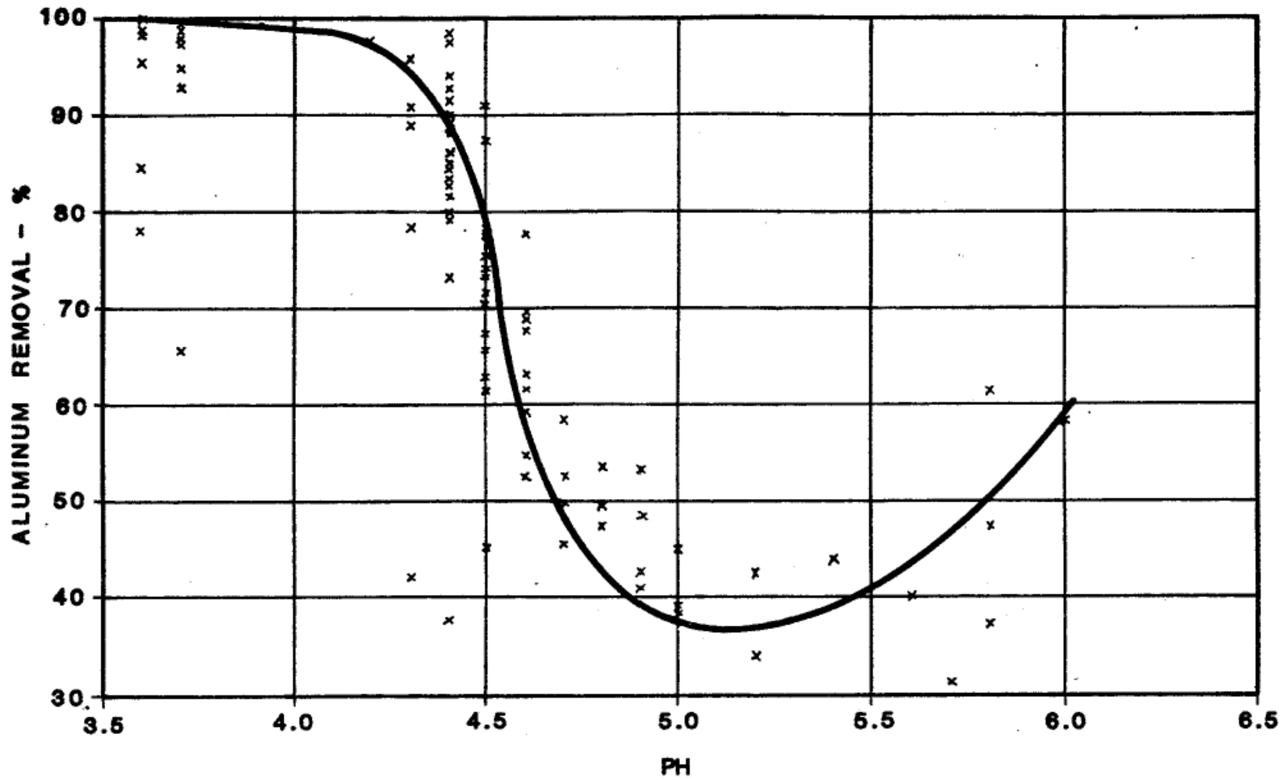
$$^{*}TR = \frac{N+0.12}{M+0.44}$$

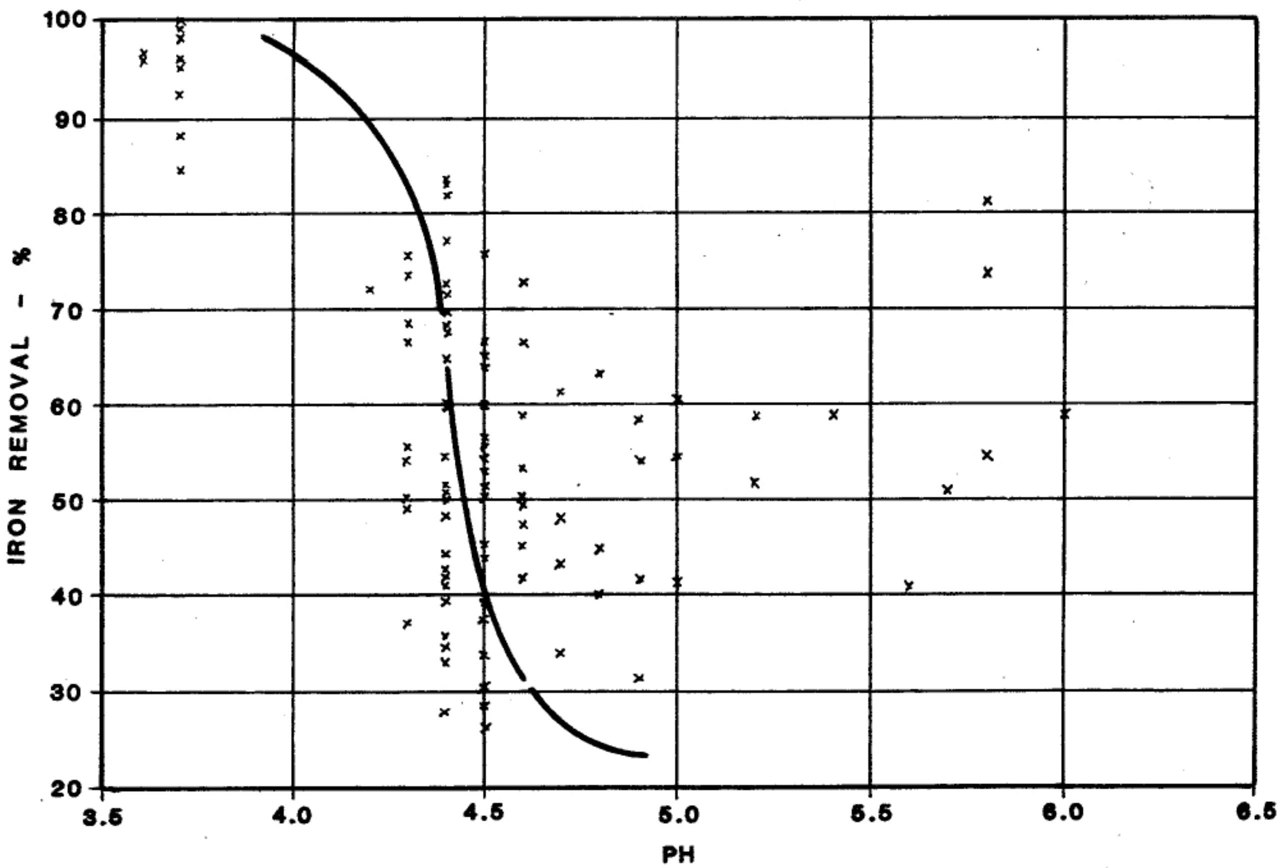
Therefore, the flow of 70 cfs at the Audenried Tunnel was assumed to be a 27-year event and peak tunnel flows during the sampling period are assumed to have a 5-year frequency.

Flow distribution in the form of flow-duration curves, for each of the tunnel discharges, are presented in Appendix D and are summarized below:

TABLE 7

Percent of Time	Tunnels Discharges in cfs which was Equaled or Less for the Indicated % of Time				
	Audenried	Oneida	WLT 1	WLT 2	WLT 3
2%	9.0	1.1	1.4	0.62	0.12
5%	9.8	1.2	1.5	0.64	0.19
10%	10.7	1.4	1.7	0.66	0.22
20%	12.0	1.6	2.2	0.72	0.30
30%	13.5	1.8	2.7	0.82	0.45
40%	15.0	2.0	3.3	0.94	0.70
50%	16.0	2.2	3.9	1.15	1.15
60%	17.2	2.4	4.5	1.60	1.40
70%	18.5	2.7	5.8	1.90	1.60
80%	21.0	3.3	8.0	2.30	2.00
90%	26.0	5.6	10.2	3.30	2.80
95%	32.0	8.2	15.0	4.30	4.00
98%	41.0	9.4	17.0	5.30	4.50





METAL REMOVAL BY DOWNFLOW UNITS

FIGURE 15

IV - ABATEMENT RECOMMENDATIONS

General:

The objectives of the proposed treatment schemes are to raise the pH in Catawissa Creek and its tributaries, Tomhicken Creek and Sugar Loaf Creek to 6.5 and provide excess alkalinity in these streams. The proposed schemes were prepared with consideration for minimizing both capital and maintenance costs as well as environmental impact.

Design criteria for each scheme are based on the results obtained from the demonstration project at the Quakake Water Level Tunnel. Where data from either project is lacking or inconclusive, conservative assumptions are used as the design basis.

The improvement observed in the quality of AMD discharges at Quakake is attributable to the submerged conditions at the portal for the demonstration period. Therefore, all the proposed designs include air lock structures at the tunnel outlets. The airlock is intended to reduce the circulation of air through the mine complex and lower the rate of oxidation of the acid forming pyrites. This technique has been successfully demonstrated by the Bureau of Mines [(6).]

Downflow Bed Design

Downflow beds are proposed wherever practical as they represent the most efficient reagent use and reduce the metal loadings of the AMD. The anticipated removal of Iron and Aluminum is based on results obtained for the down flow units at Quakake, as presented in Figure 15. Design criteria for the downflow units are based on the ratio of the hydraulic-AMD loading to the required stone surface area provided, as shown in Table 10, Appendix D (WLT A). Table 10 presents the solutions of the following equation for varying pH's and carbonate species.

$$\frac{d(H^+)}{d(\frac{At}{V})} = \frac{Kr(H^+)}{1 + \frac{C_T K_1}{[K_1 + (H^+)]^2}} \qquad \text{EQUATION 1}$$

where

(H+) = hydrogen ion concentration, molar (for waters low in dissolved

solids pH = - log (H+))

A = limestone surface area exposed to acid water in sq. ft.

t = time of contact between acid water and limestone in sec.

V = volume of acid water in contact with limestone in ft3.

K = reaction rate constant = 0.333 for clean crushed limestone

containing 1% or less MgCO3, as established from laboratory

studies

r = reactivity coefficient, accounting for the inhibitory effect of metal precipitates formed on the stone, found to be 0.2 in previous field studies

CT = carbonate species concentration, molar

K1 = first dissociation constant for carbonic acid.

All downflow beds presented are presumed to be backwashed daily. However, the units at Tunnels 2 and 3 may require less frequent backwashing as the metal loadings are less than those prevailing at the Quakake, Audenried and Oneida tunnels. All backwash water is intended to be impounded for sludge settlement. It is assumed that the resulting sludge will be treated with a vacuum filter process similar to the Rapid Sludge Dewatering System demonstrated at Quakake by PK Associates, Inc. The Quakake results and description of the system is presented in Appendix I.

Drum Design

The proposed tumbling drum design uses procedures and results developed at the Quakake Project. The basic design procedure consists of determining the required alkalinity production rate over the anticipated flow range and selecting a drum or set of drums which meet or exceed the requirements.

Drum fines production rates are determined by the production of the Quakake drums which generally adhered to the following relationship:

PROD. = 23.5 PI

Where

PROD. = Lbs of Limestone Consumed per Hour

P_I = Power Introduced to the Drum

P_I = EQDy

E= Coefficient of Mechanical and Hydraulic Efficiency of the Wheel (67%).

Q = Infloww in cfs

D = Diameter of the Wheel

Y = Unit Weight of Water

The required production rates were determined on the basis of chemical requirements observed at Quakake. Titration results from the Quakake AMD indicated that for an effluent with a pH of 6.5, an alkalinity content of 40+ mg/l had to be obtained. The Quakake data also indicated that the efficiency of stone consumed relative to alkalinity produced(into solution) varied from 60% to 78%. The required use rate was determined by:

$$U = kc\triangle \text{ alk } Q$$

Where: k = Conversion Coefficient (0.224)

E = Coefficient of Efficiency

$$E = \frac{\triangle \text{ alk in Solution (lbs.)}}{\text{lbs Produced}}$$

$\triangle$ alk = Net Increase in Alkalinity , required

$\triangle$ alk = Mineral Acidity + 40

Q = Flow (cfs)

All proposed drums assume an auger operated axial feed system which will permit easy drum loading and allows for a later modification to an automatic feed system.

The design of the mechanical portions of the drum follows the principles outlined by Pearson(5) that were verified during the demonstration period at the Quakake Tunnel.

Audenried Tunnel:

The Audenried Tunnel has the largest AMD discharge of the tunnels studied. The basic treatment scheme for normal flows utilize 5,000 ft^2 of downflow beds followed by 3 tiers of 4 to 6-foot-diameter drums. This arrangement is designed to treat flows up to 25 cfs which occurs 90% of the time on an annual basis. At higher flows, 2 additional tiers of 2 drums each are employed to bring the total treatment capability up to 60 cfs. The 60 cfs treatment level would be exceeded on a 10-year frequency when rainfall frequencies are considered; however, it will probably be an annual occurrence caused by snowmelt.

Auxiliary features include an emergency spillway, two 750,000-gallon settling lagoons, sludge dewatering facilities and maintenance equipment. The proposed design flow for the emregency spillway is 150 cfs.

Detailed computations and design assumptions are presented in the Appendices for each of the tunnel AMD abatement schemes.

Onieda Tunnel:

The Onieda Tunnel is the second major pollution source, affecting Catawissa Creek. The primary treatment is provided by 2,000 ft^2 of downflow beds followed by a polishing process of 2 tiers of 2 drums. The beds treat flows up to 10 cfs. On an annual basis, the flow is less than 10 cfs 98% of the time. The drums are designed to treat an additional 10 cfs or 20 cfs total.

High discharges are associated with snowmelts, and 20 cfs is estimated to be a 100-yr. event for snowmelt. A 100-year rainfall event is expected to produce only 11+/- cfs.

A 40,000 ft^3 backwash detention lagoon and vacuum sludge dewatering system are proposed.

A major feature of the work includes substantial backfilling of the existing outlet channel. The existing channel is 35+/- feet deep with sideslopes 1:1 or steeper. It is felt that filling the channel to the maximum extent practicable is preferable to cutting the slopes back. The site is in a proposed residential area, and the impact of the proposed construction should be minimized .

<u>Water Level Tunnel #1:</u>
WLT #1 discharges into Tomhicken Creek 6 mi. upstream of its confluence with Catawissa Creek. The proposed design is intended to treat the tunnel flow and low to medium flows of the Tomhicken. The installation is designed to treat combined flows up to 60 cfs. Beyond this flow, it is assumed that the combination partial limestone treatment, dilution and residual limestone fines in the stream will provide an acceptable water quality in the stream. The flows are combined for two reasons. The first is to gain sufficient power for low flow treatment. The second is to treat the Tomhicken at low flow. If the stream is not treated during low flow, the treatment efforts on the tunnel discharge will be partially wasted.

The proposed installation consists of one tier of 2 drums treating only the tunnel discharge. The effluent is then mixed with flow from a partial diversion of Tomhicken Creek and taken through 2 tiers of drums. Each tier having 5 drums; one small low flow drum and four larger high flow drums. The installation will treat an annual event (Q2.33).

Water Level Tunnel #2:

WLT#2 discharges to Catawissa Creek just downstream of the Audenried discharge. The site has physical limitations if construction in the flood plain of Catawissa Creek is to be avoided. The proposed treatment scheme is designed to minimize the area limits of construction.

The treatment scheme consists of 300 ft^2 of downflow beds feeding to a water wheel-drum installation. In order to maximize power and treatment at low flow, a 10' diameter wheel is proposed to drive a 4' diameter drum. This arrangement also minimizes the space requirements compared with a two-tiered installation. The proposed design will treat flows up to and including 10 cfs which corresponds to a 5-year recurrence interval.

The downflow backwash will be discharged to a small settling lagoon. The settled sludge will be pumped to the Audenried installation.

Water Level Tunnel #3:

WLT #3 discharges into the headwaters of the Catawissa Creek approximately 1 stream mile above the Audenried Tunnel. Access to the tunnel site is difficult and the cost of providing access will be high. A decision concerning the location of treatment is required. The proposed design is premised on reclaiming the maximum stream mileage practicable and is not the most economical alternative available.

The proposed design features treatment of the tunnel discharge at the discharge point to include the low-flow of the stream. The treatment units consist of backwashable limestone barriers and then drum treatment. The drum is designed to be similar to the one proposed at WLT #2 in order to provide good low-flow treatment. Treatment is proposed for tunnel flows up to 6 cfs which is slightly greater than mean annual flow. The design also considers minimizing site visits. It is anticipated that drum re-filling and backwashing of the barriers would be required on a 3 to 4-day cycle. It is proposed to pipe the settled backwash sludge to the dewatering facilities at Audenried.

Access to this point on the stream is somewhat better along existing roads and the length of stream treated is maximized.

The acid concentration in the upper reaches of the Catawissa Creek, upstream of the WLT 3 discharge, necessitates additional treatment, beyond the low-flow diversion near WLT 3. Such treatment can be achieved by a 2-tier, 3-drum installation near CC-2.

V - ECONOMIC ANALYSIS

Estimated Cost

Cost estimates of the proposed abatement measures consist of Construction Cost (Fixed Cost) and Operating Cost, including maintenance and replacement costs. The total estimated construction cost for the proposed AMD abatement schemes is $6,310,000. Cost breakdown is presented in Appendix J, and is summarized below:

Audenried Tunnel	$ 2,600,000
Oneida Tunnel	1,300,000
WLT 1	775,000
WLT 2	535,000
WLT 3	1,100,000
Total Construction . . .	$ 6,310,000

The estimated annual operating cost is $534,000, consisting of the following items:

Labor	$ 334,000
Limestone	124,500
Power	3,000
Equipment & Fuel	33,000
Repairs & Replacement	40,000
Total 0 & M Cost	$ 534,000

Cost Analysis:

Replacement of process units at their respective anticipated useful life was accounted for in the annual operating cost. Consequently, the useful life of the project, determined by the least durable permanent construction material, is estimated as 50 years.

The annual fixed cost, based on the amortization of the construction cost is $760,000. Amortization cost (AC) was derived as follows:

$$AC = Px \frac{i(1+i)^n}{(1+i)^n - 1} = 0.1204P$$

Where: P = Principal (fixed construction cost)

i = Interest, substituted by 12% inflation rate

n = Useful Life (50 years)

Therefore, the projected annual cost is:

Annual Fixed Cost	$ 760,000
Operation & Maintenance	$ 534,000
Total Annual Cost	$1,294,000

During the 384-day sampling period, the combined five-tunnel discharges produced 7,015,000 pounds of acid or the equivalent mean discharge of 18,268 pounds per day. The projected mean acid discharges for a 365-day period (June thru May) is equivalent to 15,290 pounds per day. Therefore, the construction cost for the total AMD abatement is equivalent to $413 per pound of neutralized acid per day.

Based on the derived annual cost of 1,294,000, the cost per pound of acid removal is approximately 23 cents (1,294,000/365 x 15,290 = $0.232).

The total AMD discharges for the 384-day study period was 6,960 MG, representing 60 percent of the precipitation over an estimated drainage area of 12.24 square miles. Consequently, the mean precipitation of 47.22 inches, during a 365 day period (June thru May), is expected to produce approximately 0.6 x 12.24 x 640 x 47.22/12 = 18,495 acre-feet of AMD discharges, or the equivalent of 6,030 Million Gallons. Therefore, the estimated annual cost of abatement is also equivalent to a treatment cost of 21.4 cents per 1,000 gallons.

Estimated Benefits From AMD Abatement

The Catawissa Creek is designated by the Department as a cold water stream(8). The present AMD discharges create excess acidity and low pH resulting in degraded water quality of the creek from its source to its mouth. As a result, the present quality of the water is incompatible with the designated water uses for the creek. However, the water quality standards can be met by abating the AMD discharges from the tunnels. Therefore, the present degradation of water quality in the creek, resulting from "man-induced" conditions, is retrievable providing that the AMD abatement is economically justifiable.

The costs and corresponding benefits associated with AMD abatement in the Catawissa Creek Watershed have been reported previously(1). A cost/ benefit ratio of 2.06 was reported for a 5 million dollar investment in 1973.

The cost/benefit ratio was based on the following premises:

i. Successful sealing o f tunnels.

ii. Restoration of areas of high infiltration, including stream channels.

iii. Full. utilization of low-flow augmentation and neutral neutralization.

iv. Eventual treatment of remaining amounts of AMD indicators.

It is therefore reasonable to assume that the 6.31 million dollar construction cost, presently estimated (1981), for the neutralization of the tunnel discharges, will result in equal or larger benefit/cost ratio than that previously cited.

The previously cited benefit/cost ratio was based on sealing the tunnels and flooding the abandoned deep mine workings. Benefits from such a solution would be negated should deep mining in the Middle Anthracite Field be resumed. In addition to sealing the tunnels, surface mine reclamation was also included in the previously cited report. However, if sufficient coal reserves remain within the study area coal basins, the benefits derived from surface mine reclamation would be limited. Should mining be resumed, it is reasonable to assume that the cost of land reclamation will be borne by the mining interests. It should be noted that the major source of tunnel discharges is infiltration which would not be eliminated by surface mine reclamation.

The Catawissa Creek is a potential surface water supply source. It's 153 square-mile watershed encompasses parts of Schuylkill, Luzerne and Columbia Counties which have documented water supply problems. Future use of the Catawissa Creek for water supply purposes is expected to increase the aforementioned benefits and thereby the benefit/cost ratio resulting from AMD abatement. Therefore, the proposed neutralization of the tunnel discharges provide for multi-purpose benefits, including recreation, water supply and allowance for future mining in the project area.

LIST OF REFERENCES

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3. Commonwealth of Pennsylvania, Department of Environmental Resources, Operation Scarlift, Mine Drainage Abatement Measures for the Jeansville Basin, Project Number SL-135, November 1974. (Prepared by GFCC).

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5. U.S. Department of Commerce, National Oceanic and Atmospheric Administration, Climatological Data, Pennsylvania, 1978-1979.

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7. McDonnell, A.J., Pearson, F.H., Limestone Packed Tumbling Drums for Acidity Reduction, submitted for publication in "Journal of Water Pollution Control Federation", 1977.

8. Commonwealth of Pennsylvania, Department of Environmental Resources, Title 25, Part I, Subpart C, Chapter 93, Water Quality Standards, Pennsylvania Bulletin; Volume 9, Number 36, September 8, 1979.