THE COMPARATIVE COSTS OF TWO DIFFERENT TYPES OF WADING POOLS FOR DODGE BROTHERS PARK NO. 8

THESIS FOR THE DEGREE OF B. S.

R. L. Bowers L. L. Miller

1930

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A THESIS SUBMITTED TO
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BY

R.L. BOWERS L.L. MILLER

CANDIDATES FOR THE DEGREE OF
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THESIS

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PREFACE

This thesis was chosen, because, first of all, it presented a real problem. A problem that had been proposed and was likely to be carried out in the near future.

Health and happiness are the two things which stand foremost in our minds. When it is possible, to change existing conditions, to safeguard health and establish a better means for recreation, it should be considered a great accomplishment.

Because of the limited amount of time available it was impossible to go into detail in the design of all parts of the system. It was the wish of the authors to go into such detail as to effect a sound estimate of costs of different systems.

It is difficult to mention all the sources from which have come help and inspiration for the preparation of this thesis. We are indebted to Mr. Taylor, Engineer in charge of Pontiac Water Works for information and advice concerning the investigation of the Clinton River; Professor Theroux at Michigan State College for advice on design of wading pool and filtration plant; Professor Cade at Michigan State College for advice concerning design of dam and drainage system; Professor Mallmann at Michigan State College for advice concerning analysis and method of purification of the river water.

R.L.B.
L.L.M.

INTRODUCTION

Dodge Brothers State Park No. 8 is located in Macomb County two miles south of Utica or nine miles northwest of Mt. Clemens. This park lies either side of Clinton River and extends along the river a distance of 3,400 feet. It contains 31 acres of ground.

Playground equipment has been installed and additional equipment in the way of tables, benches and garbage cans have been added. At the present time many children are allowed to bathe in Clinton River at this park. Due to the fact that raw sewage is emptied in Clinton River at Utica a distance of less than two miles upstream, the river is in such a sanitary condition that it is unfit for bathing purposes.

A sample was taken April 19, 1930. From this sample 5 tubes of lactose broth each of which had been inoculated with 10 c.c. of the sample shows from 20 to 55% of gas production in 24 hrs. A count of 13,000 bacteria per c.c. was obtained from agar plates. The presence of Escherichia coli was confirmed by use of eosin methylene-blue and Endo's media. Typical colonies were fished from Endo's medium and was placed in lactose broth which showed a gas production of from 15 to 45% in 48 hours.

The presence of Escherichia coli, an intestinal bacteria indicates fecal pollution and waters with fecal pollution are dangerous from a standpoint of the disease, typhoid fever.

Warning can be given to bathers of this water as to its dangerous condition,but until such time that a suitable place for bathing is constructed there is a grave danger of disease from this source.

The purpose of this thesis is to compare economically the estimated costs of the supply for a wading pool. The supply will be furnished either by a direct flow or recirculation system.

In the direct flow systam a dam will be designed to furnish the head. This dam will support a footbridge. The supply in this system will only be chlorinated as a means of purification.

In the recirculation system the supply can either be taken from Clinton River or a well which is in operation at present time. The supply in this system will be filtered through a rapid sand filter and treated with chlorine as a means of purification. Pumps will be installed to furnish required head.

This thesis also deals with design of a drainage system for Dodge Brothers Park #8.

An Estimate of the Cost of an Installation of a Direct Flow Wading Pool.

Design Data

On the following page a table shows total weekly attendance at Dodge Park #8, also attendance on the Sunday of that week and the per cent of a total weekly attendance visiting park on Sunday.

Following is a table showing the attendance at Dodge Park #8 in previous years.

Table No. 1

Year	No Visiting Park during year	% Increase
1927	245,700	
1928	431,400	43
1929	763,108	43.5

From this above table no. 1 assuming a 43% increase for year 1930 the total yearly attendance would be 1,000,000 people.

Table No.

Date	Total Weekly Attendance	Sunday Attendance	% of Total Weekly ~~att~~ Attendance visiting park on Sunday
5/4	150		
5/11	3300	2170	66
5/18	10000	8500	85
5/25	5300	2900	53
6/1	45000	5500	12
6/8	8000	4150	52
6/15	19000	9600	50
6/22	79000	30000	38
6/29	53000	48000	90
7/6	88000	31000	35
7/13	46000	37000	80
7/20	63000	55000	87
7/27	87000	50000	58
8/3	74000	61500	83
8/10	28000	25000	89
8/17	39000	37000	95
8/24	40000	36500	89
8/31	29000	28000	96.5
9/7	38000	6000	16
9/14	4000	3400	85
9/21	2600	2250	87
9/28	1000	650	65
10/5	190	100	53
10/12	570	520	90
Total:	763,110		1,555.5

The average % of a total weekly attendance visiting park on Sunday is 67.6 with a maximum of 96.5% and a minimum of 12%. We will use 75% as a value for design data. With 75,000 as an estimated value for average maximum weekly attendance for year 1929 we have:

763,110 : 75,000 :: 1,000,000 : X

X = 100,000 (approx) average maximum weekly attendance for year 1930.

Using 75% as referred to above we have 100,000 x .75 = 75,000 people; the maximum number attending park on Sunday for maximum year.

Assuming that 1% of the total number in a maximum day will be children using wading pool we have:

75,000 x .01 = 750 children using pool on a maximum day.

From a report of the Joint Committee on Bathing Places of the American Public Health Association and the Conference of State Sanitary Engineers we have stated that, "at large outdoor pools where a considerable proportion of the water in shallow water, we may assume that 50 per cent of the non-swimmers would be on shore. The average space allowance for each non-swimmer in the water is approximately one-half that of the swimmer in deep water. Combining these factors an allowance of 10 square feet per bather should be allowed for this portion of the pool."

In consideration of the fact that this is a wading pool and only children using it we will use 5 square feet per child.

Then 750 x 5 = 3,750 sq. ft. of water surface required.

Maximum depth of wading pool 24"

Assume that 500 children will be the maximum number using the pool per hour. Again from the Report of the Joint Committee as before referred to it states in regard to the frequency of changing water that "the total number of bathers using a swimming pool during any period of time shall not exceed 20 persons for each 1,000 gallons of clean water added to the pool during that period."

Therefore we have $\frac{500}{20}$ x 1,000 = 25,000 gal./ hr.

$\frac{25000}{60}$ = 416 gal./ min. (water to be supplied to pool)

Also the Report of the Joint Committee states that "the slope of the bottom of any part of a pool where the water is less than 6 feet deep must not be more than 1 foot in each 15 feet".

Summing up these specifications for the pool we have:

Maximum number of children using pool in one day 750

Maximum number using pool per hour 500

Total water surface area to be not less than 3,750 sq. ft.

There shall be 416 gal/ min. of fresh water supplied at time of maximum load.

Maximum depth of water to be 24".

Inlets and outlets to be so arranged that there will be no dead ends, no short circuiting and even distribution. Maximum width to be 40' and pool shall be located in a natural ox-bow which exists in park. Pool shall slope to a central outlet and can be drained for cleaning. Pool shall be lined with a concrete slab 6" in thickness and to be reinforced with expanded metal reinforcing. With this design data in mind we will design pool as follows: The pool will be semi-circular, 40' wide with a center line of 210' radius. The inner edge a semi-circle with 190' radius and the outer edge a semi-circle with 230' radius. The pool will be 150' long with semi-circular corners of 12.5' radii. The pool will have a vertical wall around outside 9" in height the top of which is 3" above surface of water. The elevation of the surface of the water in the pool will be 602.40. At one end there will be three inlets, each 1'x3", discharging from a distribution chamber. From this end of pool the bottom will slope down to a point at elevation 600.70 which is 25' along center line from end of pool. Thence the bottom of the will slope to a point 25' from opposite end which is at elevation 600.40. At this point there will be located a main outlet with an auxiliary outlet at opposite end from inlet near the vertical wall. For details see plate No. 4.

At inlet end of pool there will be located a distribution chamber with three outlets into pool. This is to provide even distribution of inlet water over entire length of inlet end of pool See plate No.4 for details.

At a distance of 150' from distribution chamber toward river there will be located the chlorine apparatus inclosed in a small building which will also provide for storage of chlorine cylinders etc. Near this will be located a small chamber in which there will be a float valve which will keep the water surface at a constant elevation of 602.40. The building will be constructed of waterproof concrete as shown on plate No. 4.

The pool thus specified has an area of 5,858 sq. ft of water surface, a maximum depth of 24", width of 40', maximum slope of bottom of 1' in 13½ feet and a circulation system such that there will be even distribution, no short circuiting or dead ends. The velocity of flow is

$$Q = AV$$

$$\frac{416}{7.5 \times 60} = 50 \times V$$

$V = .0185$ ft. / sec. or which is equal to 1.11 ft. / min.

This velocity is low but it is satisfactory. Capacity of tank equals 6,278 cu. ft. or 47,085 gallons.

This concludes the design of wading pool, chlorination plant and piping system.

Stream Flow Investigation of Clinton River

In consideration of drawing the supply for the wading pool from the river, and supplying the necessary head by means of a dam; it will be necessary to investigate the variation of stream flow in the Clinton River. It is hoped to accurately determine the maximum stream flow that will probably occur once in a given number of years.

An exhaustive search for stream flow records was made through the publications of the United States Weather Bureau, and the office of the State Highway Department, but without success. The only existing records were taken at the Water Works in Pontiac; but due to the large difference in the characteristics of the drainage area at Pontiac and that at Utica, Mr. Taylor, Engineer in charge of the Pontiac Water Works advised us against their use in this investigation. Mr. Taylor suggested that we obtain the stream flow records for the Huron River at Ann Arbor as the characteristics of the Huron River drainage area and that of the Clinton River are very similar.

The suggestion of Mr.Taylor was followed. Monthly average stream flow of the Huron River at Borton Dam were obtained from the State Board of Health. The records were complete and extended from 1904 to 1928 inc. The records are tabulated on the following page, and are also plotted on a hydrograph.

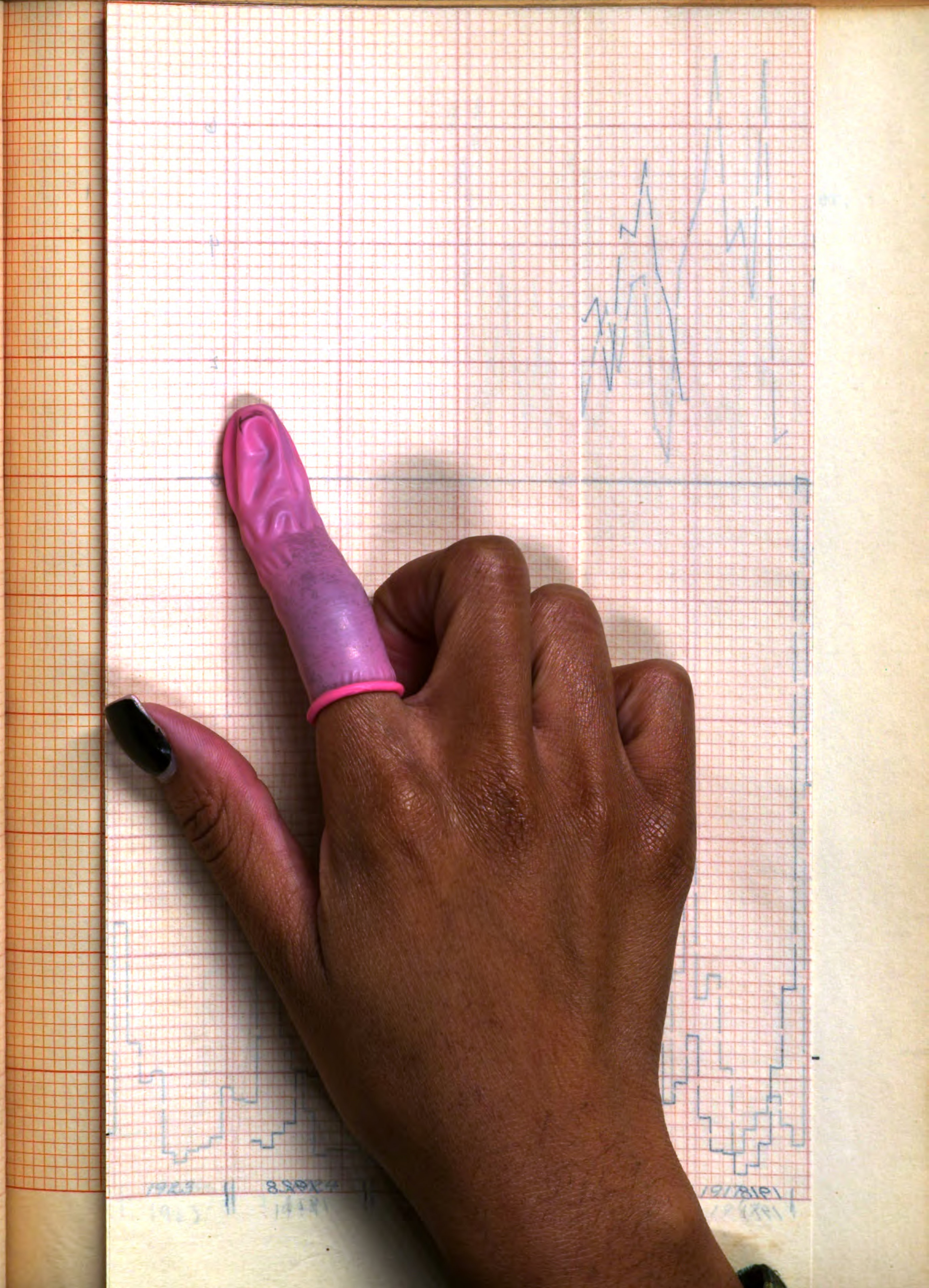
We propose to estimate the probable flow of the Clinton River from the records of the Huron River as follows:-A comparison of the various types of soil that exist in each drainage area will be made by determining what percent each particular type of soil is of the drainage area in which it exists. The percentages of the various soils in the two areas will be compared, and a ratio between the relative perviousness of the two drainage areas will be determined by a method explained later.

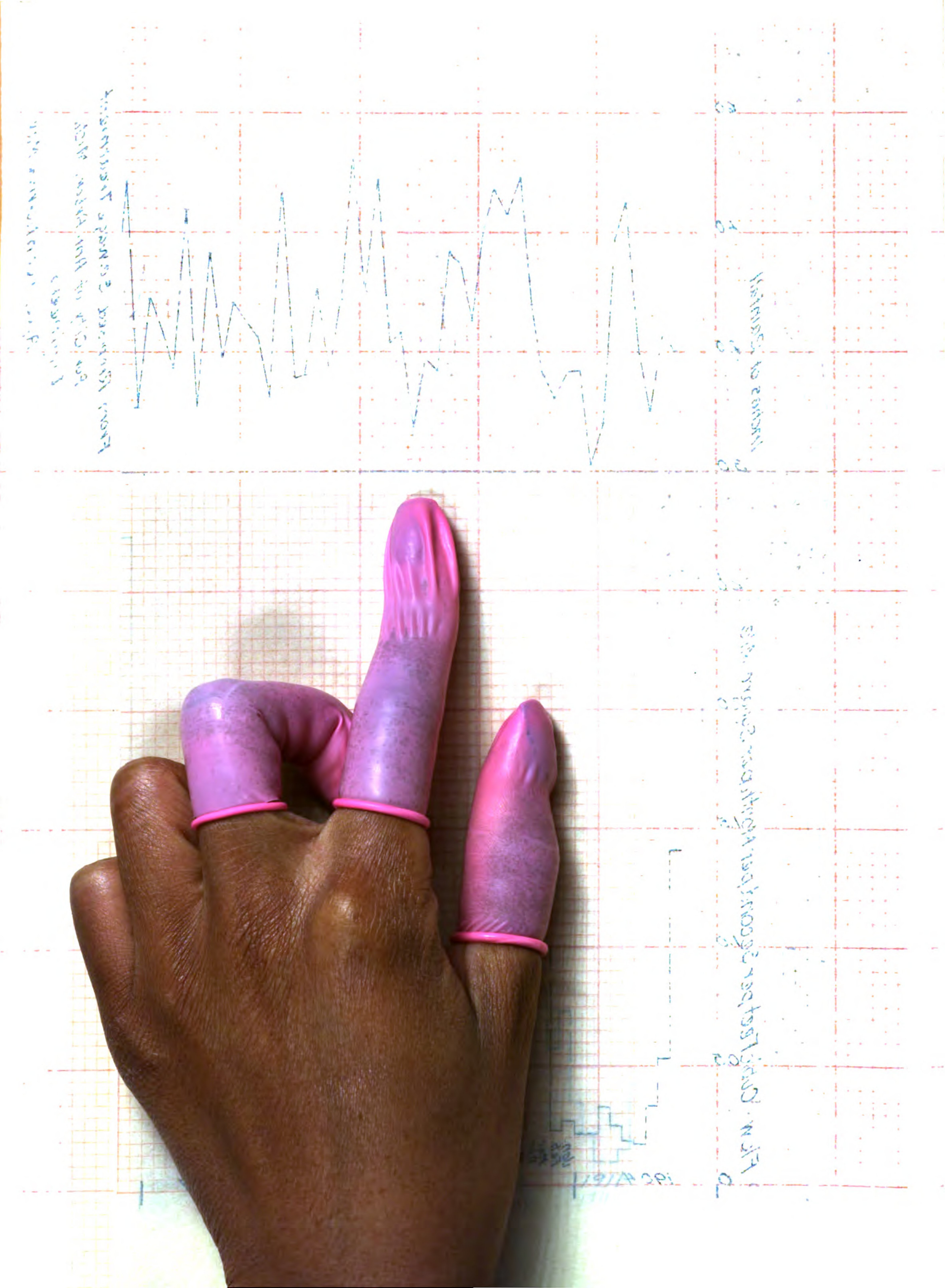
STREAM FLOW RECORD
HURON RIVER at BARTON MICHIGAN
AVERAGE MONTHLY DISCHARGE IN Cu.Ft. per Sec.
DRAINAGE AREA 723 Sq. Miles

Year

Mo.	1904	1905	1906	1907	1908	1909	1910	1911	1912	1913	1914	1915	1916	1917	1918	1919	1920	1921	1922	1923	1924	1925	1926	1927	1928
Jan.		195	655	936	643	288	447	318	240	1186	349	349	888	301	163	343	190	374	401	201	276	131	357	275	576
Feb.		162	407	369	922	788	473	602	214	690	382	1075	816	278	1258	275	205	238	482	188	274	379	461	692	581
Mar		715	492	687	1970	633	791	413	797	1366	630	717	1200	683	2308	976	1210	779	819	823	860	566	1248	695	533
Apr	1020	638	592	820	838	472	466	538	1593	1746	854	404	1553	1025	636	1399	782	764	1617	458	757	322	1617	427	669
May	386	579	434	768	599	1078	730	255	586	1001	1293	368	1012	600	397	995	478	351	662	337	469	187	426	395	358
June	275	1035	308	392	254	463	367	160	237	521	343	387	762	673	161	330	252	227	261	167	374	95	241	404	360
July	124	381	141	229	92	167	117	79	110	220	372	352	331	393	98	193	138	183	140	124	242	60	118	212	211
Aug.	133	284	149	132	257	109	101	79	264	152	208	454	159	140	74	160	138	159	117	93	112	112	181	107	175
Sept.	179	248	79	211	136	109	145	115	336	183	394	731	135	206	134	162	132	365	150	125	124	155	337	147	131
Oct.	240	345	133	302	135	119	153	317	410	233	336	526	206	250	143	228	127	327	168	138	125	410	391	208	155
Nov	156	434	260	346	143	340	166	429	732	367	288	400	245	315	241	358	249	557	186	167	117	682	439	278	
Dec	154	466	427	432	226	386	170	457	445	449	288	384	304	226	487	358	462	682	183	321	167	444	399	788	
TOTAL	2607	5482	4077	5624	6216	4952	4126	3762	5964	8114	5737	6147	7611	5090	6100	5757	4363	5006	5186	3142	5897	3543	6215	4628	
AVER.	290	457	340	469	518	413	344	314	397	679	478	512	634	424	508	480	364	417	432	262	325	275	518	386	
MAX.	1020	1035	655	936	1970	1078	791	602	1593	1746	1293	1075	1553	1025	2308	1399	1210	779	1617	823	860	682	1617	695	
MIN.	124	162	79	132	92	109	101	79	110	152	208	349	135	140	74	160	127	159	117	93	112	60	118	107	

From Proposed Sewage Treatment
for City of Ann Arbor, Mich.
Engineers
Ayres Lewis Norris & May





In the following table, the percentages of the various types of soil as they are measured in each drainage are listed. The areas used in computing these percentages of soil was measured by a planimeter from the soil map published by the Michigan State Department of Agriculture.

The relative perviousness of the various soils are expressed in what one might term as perviousness indices. After reviewing texts on soils it was decided that the perviousness of the soils were related in about the proportion indicated by the perviousness index in the table.

The perviousness ratio between the two drainage areas is determined by dividing the sum of the moments (the product of the percent of a soil in Clinton area, times the perviousness index, times the ratio of soil in Clinton area to soil in Huron area) by the average of the indices. By way of explanation, it is quite logical to assume that the perviousness ratio should be the average of the individual soil ratios and directly proportional to the percent of their existence in the Clinton area, and directly proportional to the perviousness index.

The results of the following calculations indicate that the Clinton drainage area above Utica is 1.18 times as pervious as the Huron River drainage area above Ann Arbor. Hence, in view of these calculations, the discharge of the Clinton River in c.f.s. per square mile at Utica is probably $\frac{1.00}{1.18}$ of the discharge of the Huron River, considering for the present that the rainfall upon the two areas is the same.

Drainage Area Characteristics
of the
Huron and Clinton Rivers

Soil	Perviousness Index	Percent of Huron Drainage Area	Percent of Clinton Drainage Area	Ratio of Percent of Soil in Clinton Area to Percent of Soil in Huron Area	Moment
Out Wash	2	34.2	36	1.05	.756
Moraine	5/3	36.8	46	1.25	.937
Till Plains	4/3	29	13	0.45	.078
Sandy Lake Beds	1		5	—	
	6				1.771

$$\text{Average index} = \frac{6}{4} = 1.50$$

$$\text{Perviousness ratio} = \frac{1.771}{1.50} = 1.18$$

The next factor to be considered is the rainfall over the two drainage areas. From the United States Weather Bureau Report of 1920 (1930 report is not available) the average yearly rainfall over the Huron River drainage area from the year 1880 to 1920 is 31.69". This record is complete. From the same report the average rainfall over the Clinton River Drainage area from the year 1887 to 1920 is 30.01". The records over this area are rather incomplete. For this reason, and because of the fact that there exists very little difference between the two averages, no correction will be made for rainfall in estimating the discharge of the Clinton River at Utica.

There remains only the simple operation of dividing each monthly discharge as plotted on the Huron River hydrograph by the constant 1.18, and the result will be the probable discharge for the Clinton River at that time, expressed in c.f.s per sq. mile. These results are listed on the following table.

Tabulation of the probable stream discharge of the Clinton River. Average monthly discharge in cubic feet per second per square mile.

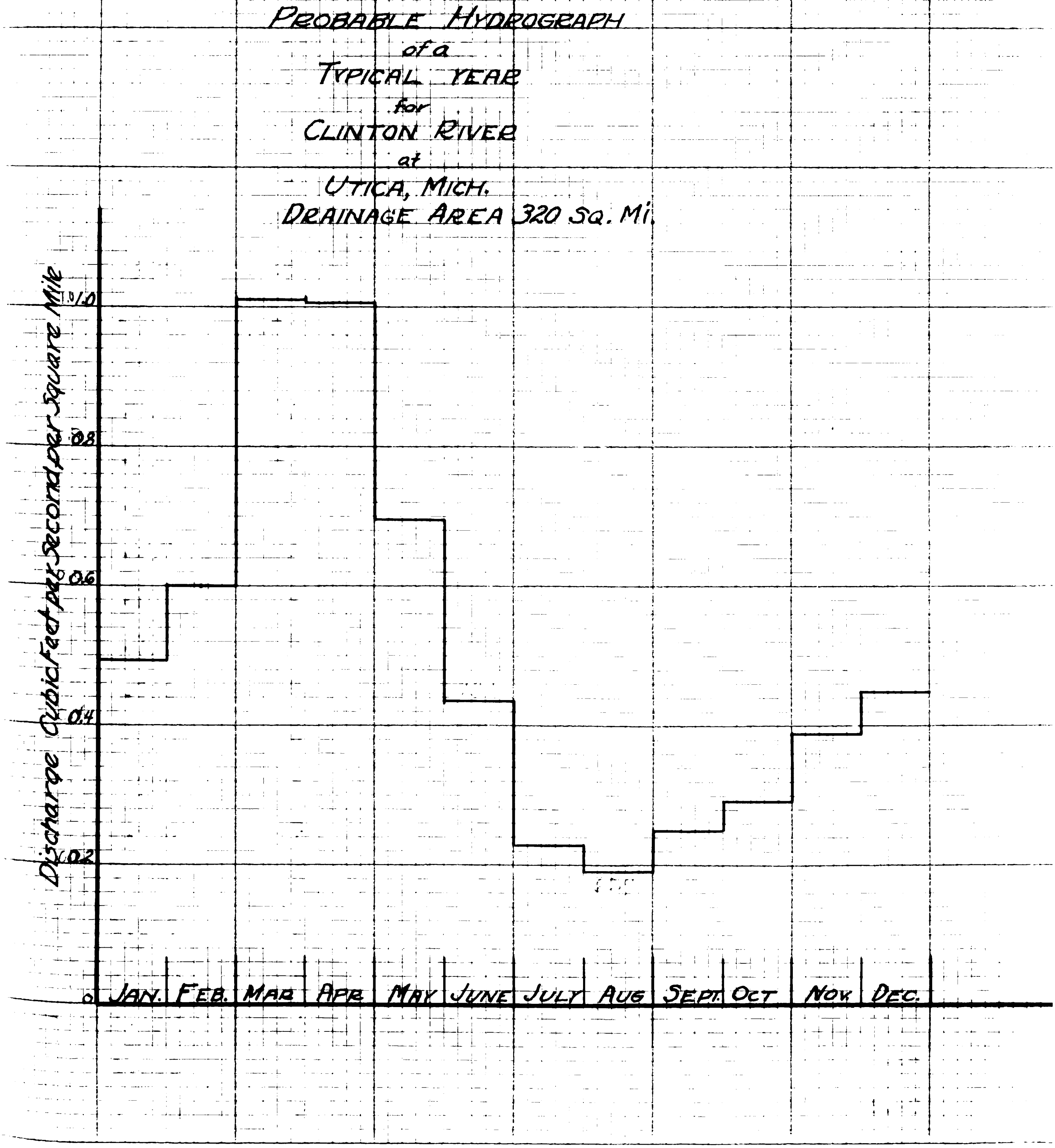
Drainage area = 320 square miles. These values are determined from discharge records covering the period from Apr. 1904 to Oct. 1928 on the Huron River and a study of its drainage area.

Year	Jan	Feb	Mar	April	May	June	July	Aug.	Sept.	Oct.	Nov.	Dec.
1904				1.198	.452	.322	.146	.156	.210	.281	.183	.180
05	.228	.190	.836	.746	.677	1.212	.446	.333	.291	.287	.510	.546
06	.768	.476	.578	.693	.509	.361	.165	.174	.092	.156	.305	.501
07	1.098	.434	.806	.961	.899	.459	.268	.155	.248	.354	.405	.506
08	.754	1.080	2.314	.980	.701	.298	.108	.302	.161	.243	.168	.265
09	.338	.921	.741	.553	1.262	.544	.196	.128	.128	.140	.390	.453
10	.523	.554	.929	.546	.855	.430	.137	.119	.170	.180	.195	.199
11	.342	.706	.475	.631	.299	.188	.092	.092	.135	.371	.503	.535
12	.281	.251	.935	1.870	.688	.278	.129	.309	.394	.566	.856	.522
13	1.389	.809	1.604	2.042	1.173	.612	.258	.178	.206	.273	.430	.527
14	.409	.448	.739	1.001	1.517	.402	.436	.176	.461	.394	.338	.338
15	.409	1.260	.840	.559	.431	.454	.413	.531	.856	.616	.468	.451
16	1.040	.949	1.408	1.822	1.188	.893	.388	.186	.158	.242	.288	.357
17	.452	.326	.800	1.201	.703	.789	.460	.164	.242	.293	.370	.265
18	.191	1.475	2.705	.746	.465	.189	.114	.086	.157	.168	.283	.571
19	.403	.323	1.142	1.640	1.167	.388	.226	.187	.190	.268	.387	.387
20	.223	.241	1.420	.916	.561	.296	.162	.162	.155	.149	.292	.542
21	.439	.280	.912	.895	.497	.266	.214	.187	.428	.484	.654	.800
22	.470	.565	.960	1.891	.776	.306	.164	.137	.176	.197	.218	.215

23	.236	.220	.964	.536	.395	.430	.146	.109	.147	.162	.196	.376
24	.324	.320	1.008	.885	.549	.439	.284	.131	.146	.147	.137	.196
25	.153	.444	.665	.378	.219	.111	.070	.131	.182	.480	.798	.521
26	.418	.541	1.462	1.897	.500	.283	.138	.212	.480	.459	.515	.468
27	.322	.811	.815	.501	.464	.474	.249	.125	.172	.244	.326	.924
28	.667	.682	.710	.784	.419	.422	.247	.205	.153	.181		

From the above table a typical yearly discharge curve is plotted which shows the probably discharge of the Clinton River. The ordinates are obtained by finding the average for each month in the preceding table.

The above data is also arranged into a frequency distribution which shows the number of times each rate of flow would have probably occured during the period covered by the data. The third column in the frequency distribution shows the number of times the particular flow is equal to or greater than its self. The fourth column shows the percent of time and the fifth and sixth the frequency of occurance of each particular rate of flow. The percent of time and frequency in years is plotted on probability paper. The resulting probability curve will be used to determine the size of flood to be expected once in a given number of years.

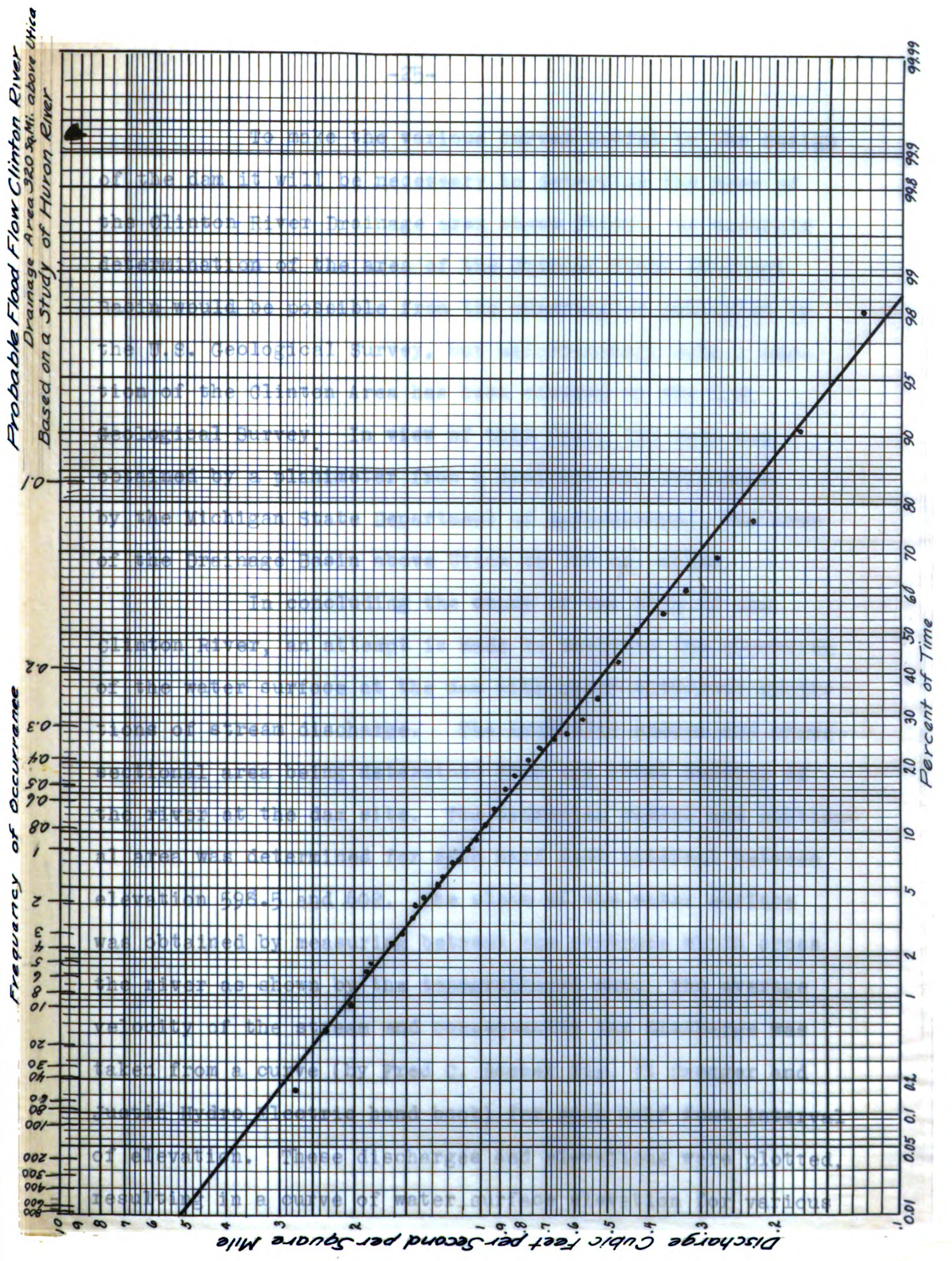

PROBABLE HYDROGRAPH
of a
TYPICAL YEAR
for
CLINTON RIVER
at
UTICA, MICH.
DRAINAGE AREA 320 SQ. MI.
Discharge Cubic Feet per Second per Square Mile
1.0
0.8
0.6
0.4
0.2
0
JAN.
FEB.
MAR
APR
MAY
JUNE
JULY
AUG
SEPT.
OCT
NOV.
DEC.

Frequency Distibution of Clinton River Stream Flow

Average Monthly Discharge c.f.s./ sq.mi.	Number of Occurrences	Summation of Occurrences	Percent of time	Frequency Months	Frequency Years
.100	5	295	99.9	1.001	.0835
.150	23	290	98.1	1.02	.085
.200	42	267	90.4	1.11	.0925
.250	21	225	76.1	1.31	.109
.300	24	204	69.0	1.45	.121
.350	15	180	60.9	1.65	.137
.400	15	165	55.7	1.80	.150
.450	21	150	50.7	1.97	.164
.500	27	129	43.5	2.30	.192
.550	15	102	34.4	2.91	.242
.600	8	87	29.3	3.42	.285
.650	3	79	26.6	3.76	.313
.700	6	76	25.6	3.91	.326
.750	8	70	23.6	4.24	.353
.800	8	62	20.8	4.80	.400
.850	6	54	18.1	5.52	.460
.900	8	48	16.1	6.21	.518
.950	7	40	13.4	7.46	.621
1.000	5	33	11.0	9.09	.756
1.050	3	28	9.32	10.72	.903
1.100	2	25	8.31	12.03	1.003
1.150	1	23	7.63	13.10	1.09
1.200	4	22	7.30	13.70	1.14
1.250	2	18	5.94	16.85	1.40

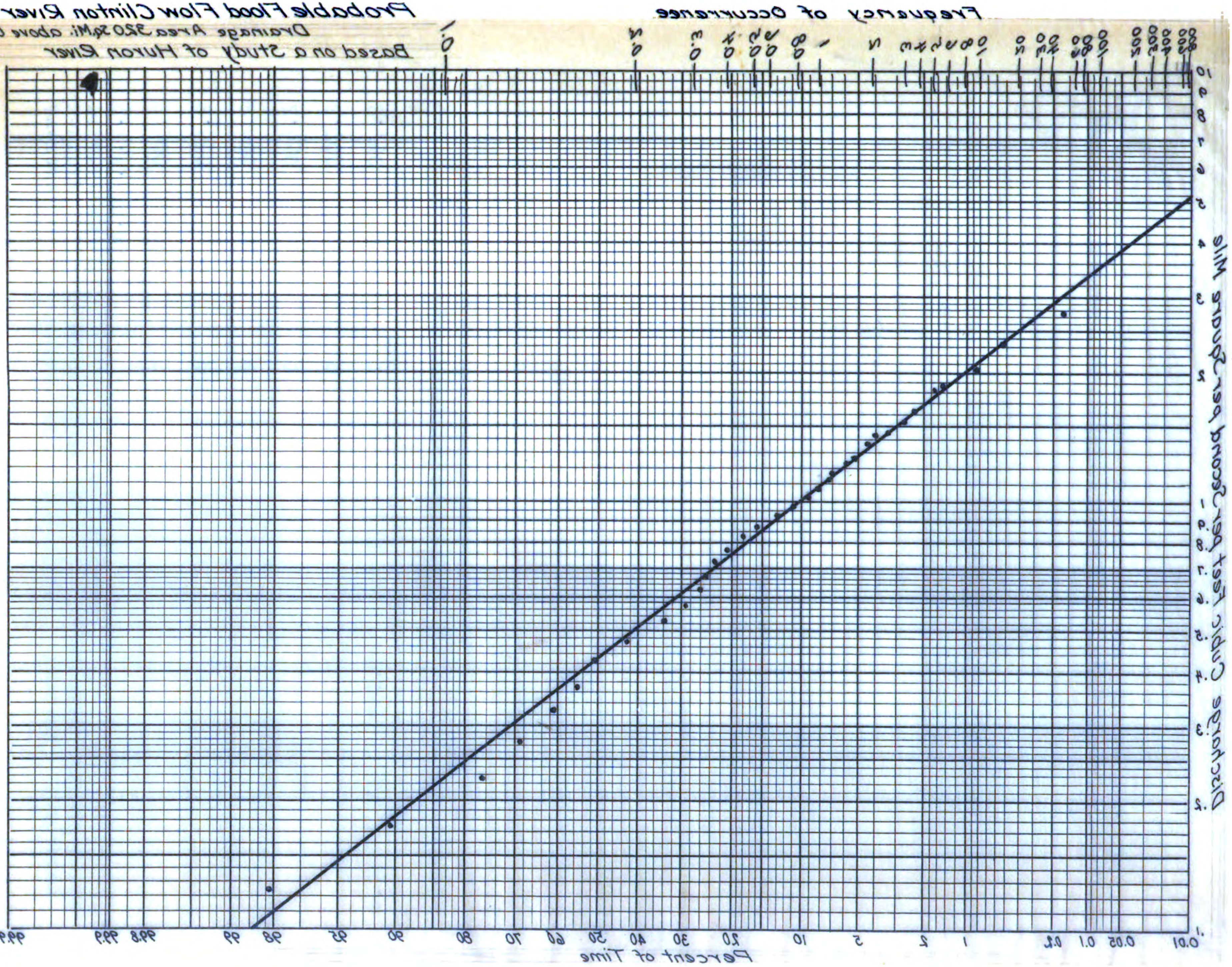
Average Monthly Discharges c.f.s./sq.mi	Number of Occurrences	Summation of Occurrences	Percent of time	Frequency Months	Years
1.300	2	16	5.26	19.00	1.58
1.350	0				
1.400	1	14	4.58	21.85	1.82
1.450	2	13	4.24	23.60	1.97
1.500	2	11	3.56	28.10	2.34
1.550	1	9	2.88	34.85	2.90
1.600	0				
1.650	2	8	2.54	39.40	3.28
1.700	0				
1.750	0				
1.800	0				
1.850	1	6	1.86	53.80	4.48
1.900	2	5	1.53	6.54	5.44
1.950	0				
2.000	0				
2.050	1	3	0.848	118.0	9.83
2.100	0				
2.150	0				
2.200	0				
2.250	0				
2.300	0				
2.350	1	2	0.509	196.7	16.3
2.400	0				
2.450	0				

Average Monthly Discharges c.f.s./sq.mi	Number of Occurrences	Summation of Occurrences	Percent of time	Frequency Months	Years
2.500	0				
2.550	0				
2.600	0				
2.650	0				
2.700	0				
2.750	1	1	0.169	592	49.4
	295				



Probable Flood Flow Clinton River
Drainage Area 920 Sq.Mi. above Utica
Based on a Study of Huron River
Frequency of Occurrence
Percent of Time
Discharge Cubic Feet per Second per Square Mile

Probable Flood Flow Clinton River
Drainage Area 320 Sq.Mi. above Utica
Based on a Study of Huron River



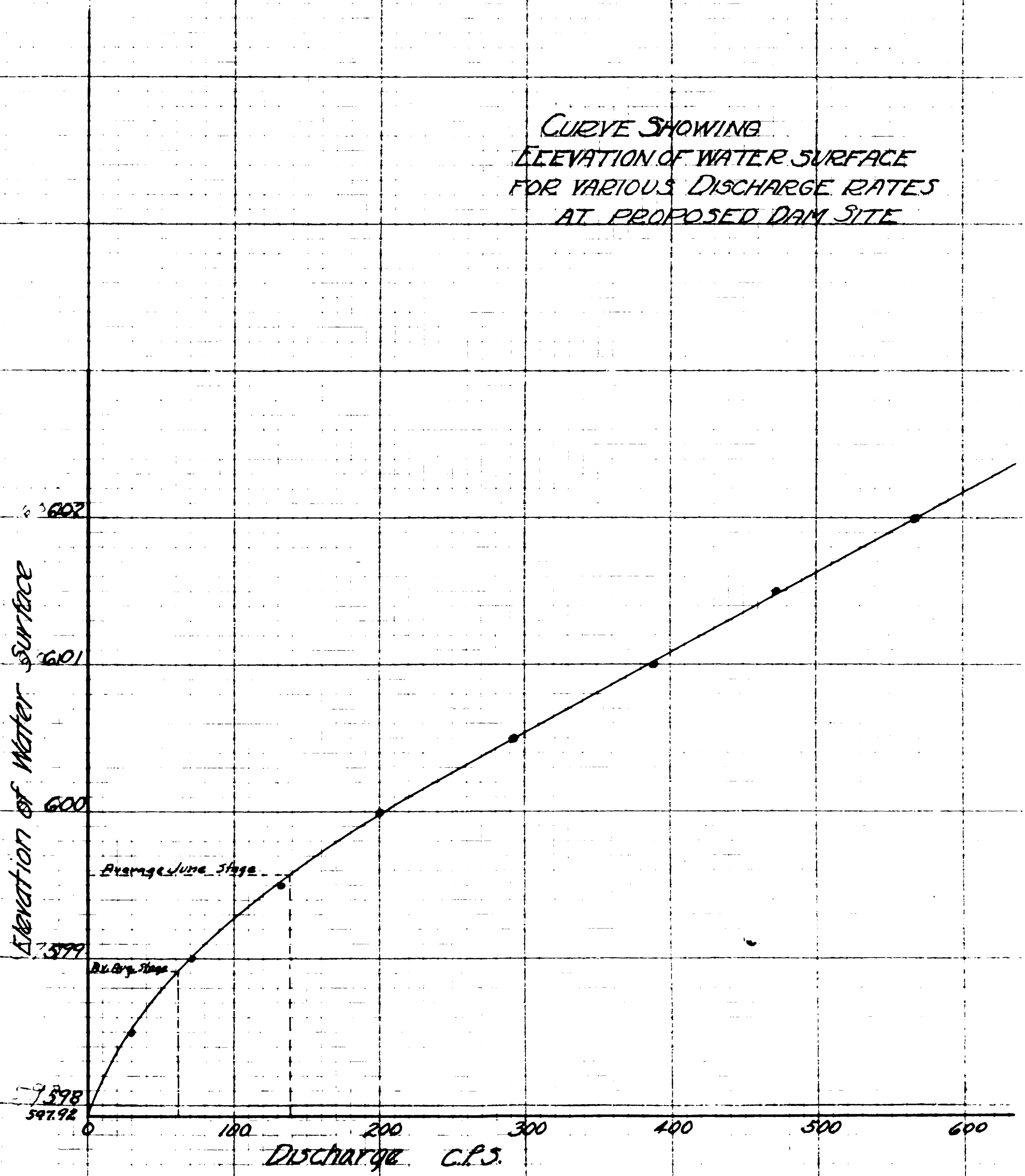
To make the various curves useful in the design of the dam it will be necessary to determine the area of the Clinton River Drainage area above Utica. An accurate determination of the area of the Clinton River drainage Basin would be possible from the quadrangles published by the U.S. Geological Survey, but unfortunately only a portion of the Clinton Area has been covered by the U.S. Geological Survey. In view of this fact, the area was obtained by a planimeter from a drainage map published by the Michigan State Department of Agriculture. The area of the Drainage Basin above Utica is 320 sq. miles

In concluding the study of the flow of the Clinton River, an attempt is made to estimate the elevation of the water surface at the dam site under different conditions of stream discharge. The hydraulic radius and cross-sectional area being determined from the cross section of the river at the dam site. The hydraulic radius and sectional area was determined for each half foot interval between elevation 598.5 and 602. The slope of the water surface was obtained by measuring between the contours which cross the river as shown by the topographical map. The average velocity of the stream and consequently the discharge was taken from a curve (by Fred C. Scobey Fig. 71 Creager and Justin Hydro Electric hand book) for each half foot interval of elevation. These discharges and elevations were plotted, resulting in a curve of water surface elevation for various discharges. We must acknowledge that this curve is based upon the assumption that the slope remains constant and that the slope was determined accurately in the first instance, which of course is not quite true.

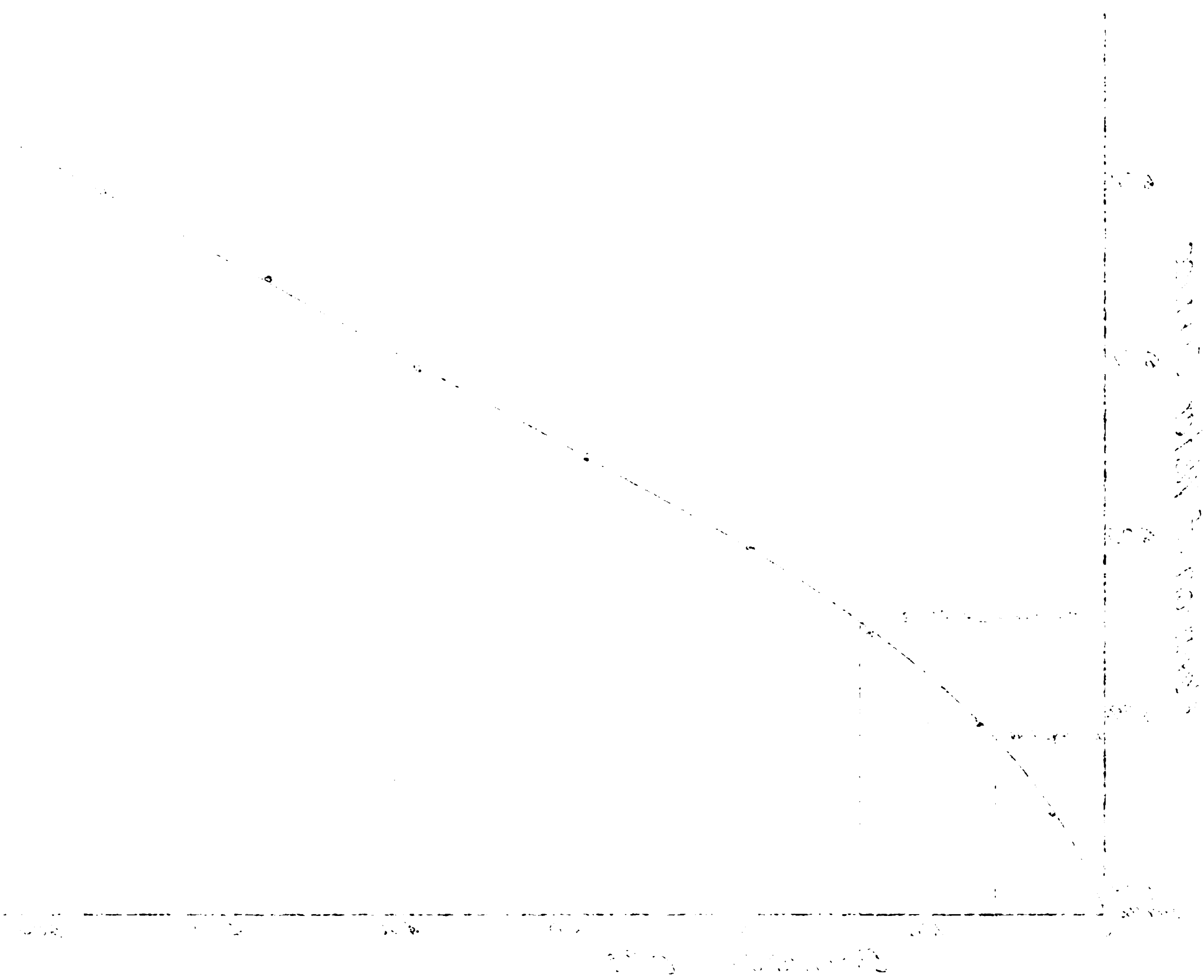
Computations for the gage curve

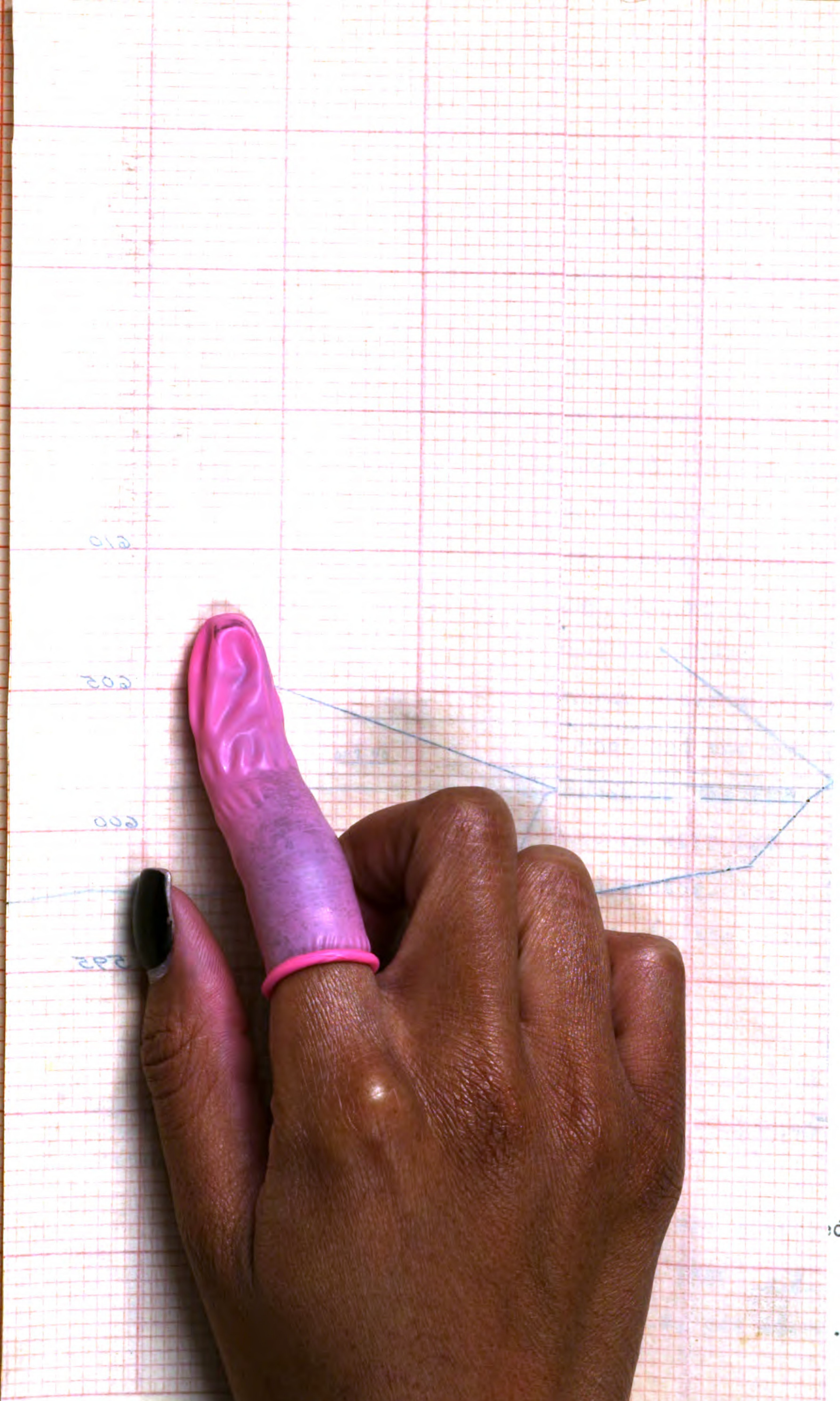
Elevation	Area	P	R	V	Q
598.5	32	43.2	.742	.94	30.0
599	53	46.1	1.15	1.35	71.6
599.5	77.5	47.9	1.62	1.70	132
600	100	49.2	2.04	2.00	200
600.5	124	50.3	2.46	2.35	292
601	146	51.6	2.83	2.67	390
601.5	169	52.9	3.20	2.80	473
602	192	55.7	3.45	2.96	569

The purpose of the curve is to determine the height of the drainage system outlet and to determine as near as possible the elevation.



CURVE SHOWING
ELEVATION OF WATER SURFACE
FOR VARIOUS DISCHARGE RATES
AT PROPOSED DAM SITE
Elevation of Water Surface
602
601
600
599
598
597.92
Average June Stage
0
100
200
300
400
500
600
Discharge C.F.S.





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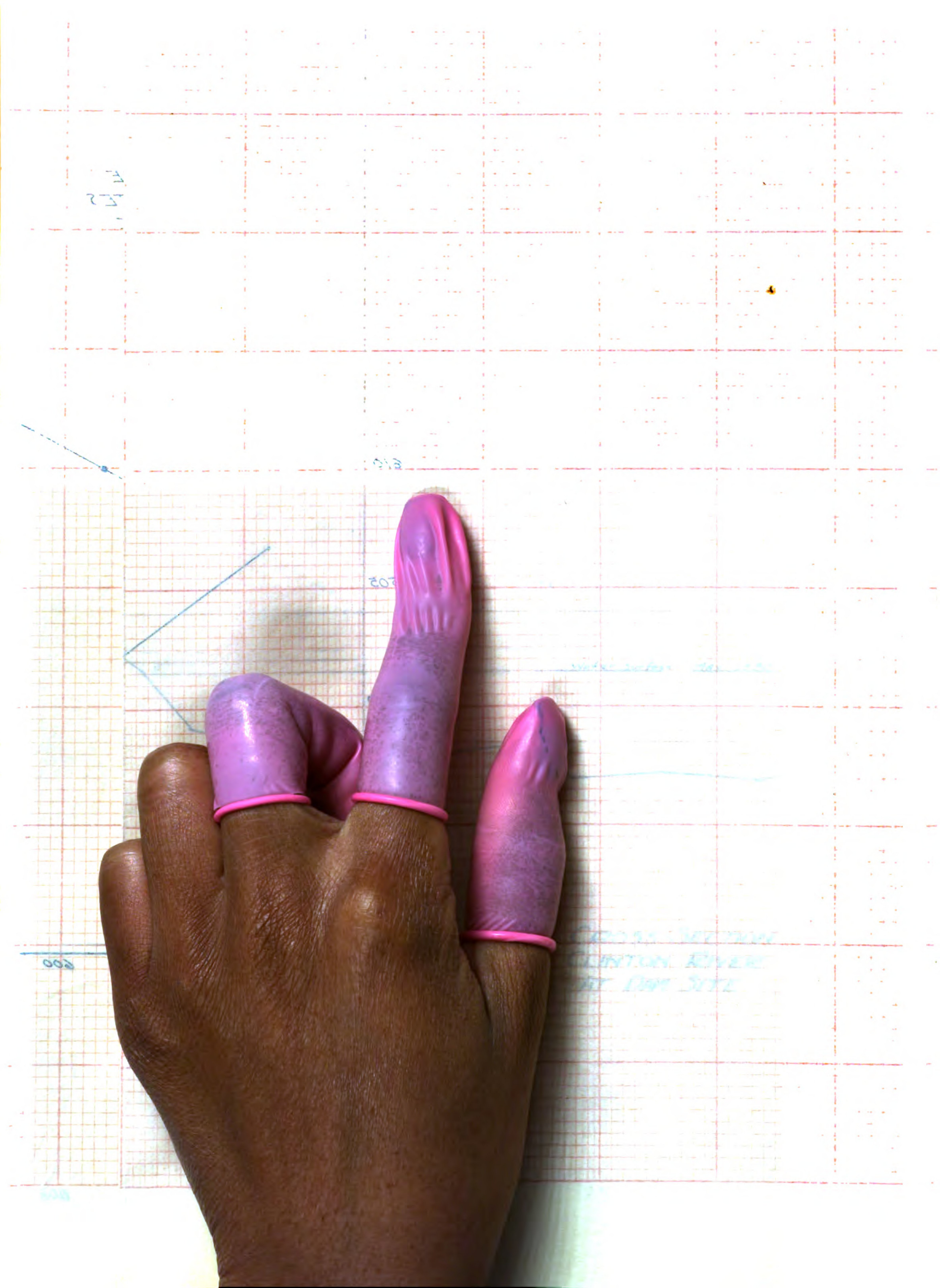

610
605
600
ROSS SECTION
INTON RIVER
DAM SITE

Design of Dam

The dam, as shown on plate No. 2 will be a crest gravity type with spillway and flash boards in the center section. The flash boards and supporting structure will be so arranged as to enable an operator to instantly open the spillway section when such an occasion arises. The flash boards and their supporting structure may also be removed during that part of the year when the park is closed. ~~These~~ There need not remain in place, any thing during the winter except thecrest section and the spillway piers.

In addition to the above provisions, there exists three controlling factors; the minimum water surface elevation to be maintained in the wading pool and the minimum stream flow to be expected over the crest of the dam, the careful consideration of these two factors will determine the correct crest elevation; the third factor, upon which depends the dimensions of the spillway, is the maximum stream flow.

Crest Elevation

Water surface elevation of wading pool=602.40

Min. " " " of pond =602.40
(Head loss between pond and wading pool is negligible)

The probable minimum monthly flow was selected from the probable average monthly flow table on page 17

Probable minimum flow 0.086 c.f.s. per sq.mi.

or .086 x 320 = 27.5 c.f.s.

As there are two crest dams over which the water will flow, $\frac{1}{2} \times 27.5 = 13.75$ c.f.s. over each dam

Determination of depth on crest from Francis' formula

$$Q = cl\,(h \pm h_v)^{3/2}$$

Q = discharge in c.f.s. = 13.75

l = length of crest in ft = 10'

h = head on crest

$h_v = \frac{v^2}{2g}$ = velocity head

C = Coefficient of discharge

From curve 77, Hydroelectric Handbook, by Creager and Justin $C = 3.95$

Velocity head, h_v

Area as measured from cross-section of stream = 217 sq. ft.

Velocity of stream

The velocity head may be neglected

$$13.75 = 3.95 \times 10\,(h)^{3/2}$$

$h =$

The above results would be correct if there were no leakage around the flash boards. But due to the numerous small openings around the flash boards, there will be considerable leakage. It is quite probable, and for the want of more accurate data, that 20% of the minimum flow will leak through the flash boards.

Hence 80% of 13.75 c.f.s = 11.00 c.f.s. discharge over each crest dam.

By Francis' formula

$$Q = C l h^{\frac{3}{2}}$$
$$11.00 = 3.95 \times 10 (h)^{3/2}$$
$$h = \left(\frac{11.00}{3.95 \times 10}\right)^{\frac{2}{3}} = (.278)^{\frac{2}{3}} = .426'$$

Minimum elevation of water surface	602.40
" head on crest dam	.43
	601.97

Let crest elevation at 602.0'

Capacity of Spillway 1st. Trial.

The capacity of the spillway depends upon the maximum flow that will probably occur once in 30 years a frequency consistant with a development of this type. From the stream flow probability curve on page 24, the average monthly flow that will probably occur once in 30 years is 2.7 c.f.s. per sq. mile

Or 320 x 2.7 = 864 c.f.s.

It is also desired to limit the elevation of the water surface when 864 c.f.s. are flowing to 604.00, as this elevation is only slightly lower than the adjoining land.

By Creager & Justin formula (43) for spillways

$$Q = ld\sqrt{2g(h + h_v - d}$$

where in d would equal 2/3 ($h + h_v$)
if the conditions were such that the hydraulic jump would operate. But in this case the down stream slope

through the spillway is not sufficient to cause the hydraulic jump. Hence there remains the possibility of determining d from the probable elevation of the water surface below the spillway. From the gage curve on page ________ the water surface elevation should be 603.5 when the river discharge 864 c.f.s.

hence $h = 604 - 598.5 = 5.5'$

down stream head $h = 603.5 - 598.5 = 5.0'$

598.5 = elevation of spillway floor.

Also assume spillway width = 26'

therefore $d = \frac{h_1}{h} h = \frac{5.0}{5.5} h = .909 h$

$h_v = \frac{v^2}{2g}$; $a = 308$ From cross-section

$v = \frac{864}{308} = 2.8$ Ft. p. Sec

$h_v = \frac{\overline{2.8}^2}{2 \times 32.2} = .164$ Ft

$$Q = d l h \sqrt{2g(h + h_r - d}$$

$$Q = .909 \times 5.5 \times 26 \sqrt{64.4(5.5 + .164 - .909 \times 5.5}$$

$$Q = .909 \times 5.5 \times 26 \sqrt{64.4(.091 \times 5.5 + .164}$$

$$Q = .909 \times 5.5 \times 26 \times 6.54 = 851 \text{ c.f.s.}$$

Flow over crest dams

$$Q = c\, c_1\, l\, (h + h_r)^{3/2}$$

From table 24 Creager + Justin

$c = 3.8$; $c_1 .817$

$$Q = 3.8 \times .817 \times 10 (2.164)^{3/2} = 3.8 \times .817 \times 10 \times 3.18 = 96.3 \text{ c.f.s.}$$

$2 \times 96.3 = 192.6$ over crest dams

851 — thru spillway

1043 c.f.s. total discharge

The above calculations were based upon the assumption that approximately 864 c.f.s. were flowing and that the elevation of the tail water as taken from the gaging curve was 603.5 The results indicate that the elevation of the tail water should be higher.

Capacity of Spillway 2nd Trial

Assume 925 c.f.s.

Elevation of tail water from gaging curve 603.65

$$d = \frac{h_1}{h} h = \frac{5.15}{5.50} h = .935 h$$

$$Q = 0.935 l h \sqrt{2g(.065 h + h_v)}$$

Velocity head

$$V = \frac{925}{308} = 3.01 \text{ ft. per sec.}; \quad h_v = \frac{3.01}{64.4} = .136'$$

$$Q = (.935 \times 5.5 \times 26 \sqrt{64.4(.065 \times 5.5 + .136)}$$

$$Q = 0.935 \times 5.5 \times 26 \times 5.64 = 753 \text{ c.f.s.}$$

Over crest of dams

From table 24 Creager and Justin

C = 3.8; C = .739

Q = 3.8 x .739x10 (2.136) $^{3/2}$ = 87.5

Over crest dams	2 x 87.5	= 175.0
Through spillway		= 753
Total discharge		928 c.f.s.

The open spillway and the drest dams will discharge 928 c.f.s. at a surface elevation of 604.0 which discharge exceeds 864 c.f.s. Hence the assumed spillway dimensions will be adopted.

Spillway width 28'

Floor elevation 598.5

Max. water surface 604.0

Top of spillway piers 605.0

Spillway Design

The flash boards will be held in place by vertical studding secured at the bottom by iron shoes, and at the top by a horizontal steel truss. The truss will be a five paneled Pratt, 28 ft. long and 5' wide.

Selection of flash boards.

Top elevation of boards	=	603.0
Floor elevation	=	598.5
head on bottom board		4.5

Water pressure at bottom 4.5x62.5 = 282 lb/ft

Span of board = $\frac{28}{5} = 5.6'$

B.M. $\frac{1}{8} wl^2 = \frac{282 \times 5.6 \times 5.6}{8} = 1105$ lb. ft.

1105 lb ft = 13300 lb. in

Extreme fiber stress of Douglas Fur from Carnegie Hand book 1200 lb/in^2

$$M = \frac{Is}{c} = 13300$$

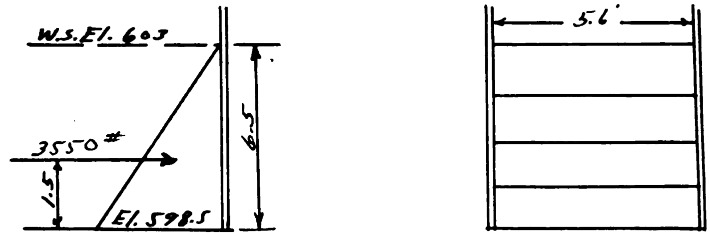
$$\frac{I}{c} = \frac{13300}{1200} = 11.05 \text{ in}^3$$

for 3" plank (measure 2 3/4)

$$\frac{I}{c} = \frac{1.733 \times 12}{1.375} = 15.1 \text{ in}^3$$

3" Douglas Fur timber will be used for Flash boards.

Flash board studding



Resultant water pressure $P = (\frac{1}{2}\ wh^2)\ 5.6$

$$P = \frac{5.6}{2}\ 62.5 \times 4.5 \times 4.5 = 3550\#$$

$$B.M. = (\frac{5}{6.5}\ 3550)1.5 = 4100 \text{ lb ft} = 49200 \text{ lb.in}$$

$$\frac{I}{C} = \frac{M}{S} = \frac{49200}{16000} = 3.07 \text{ in}^3$$

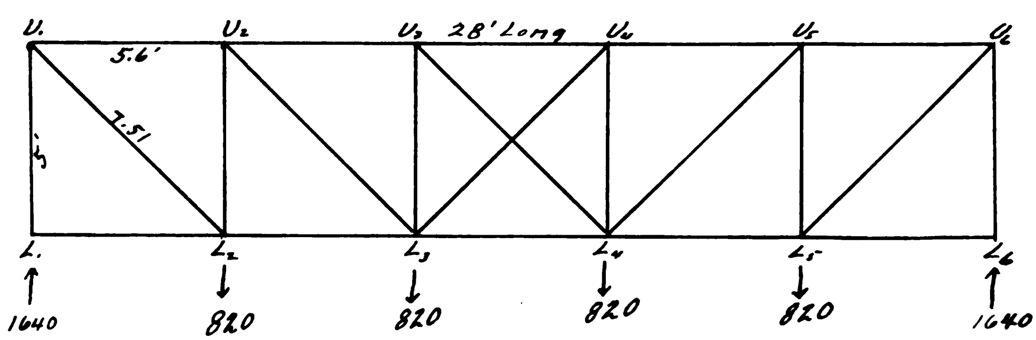
Carnegie Cross Tire M-25 : $\frac{I}{C} = 5.5$

Top reaction $= \frac{1.5}{6.5}\ 3550 = 820\#$

Bottom " $= 3550 - 820 = 2730\#$

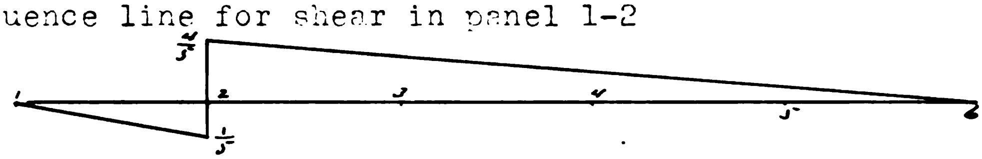
Steel Truss

This truss will be rather unusual in that it must support its own weight in a horizontal position, a dead load of a temporary foot bridge, a live load and impact, and the horizontal panel point load, the loading which makes the truss necessary.

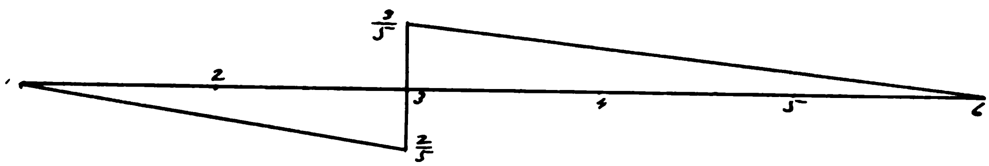


Length of diagonal =

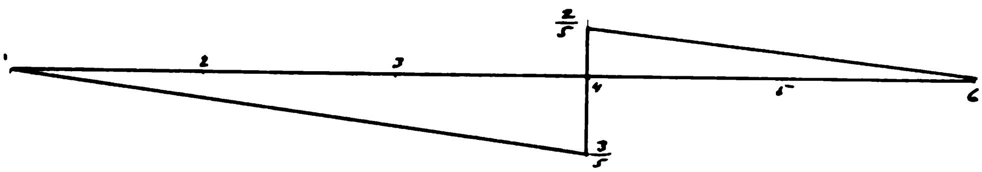
Influence line for shear in panel 1-2



Influence line for shear in panel 2-3



Influence line for shear in panel 3-4



Full load stresses (horizontal)

(by index method)

Member	Index Stress	Factor	Stress
$V_1 - V_2$	-1640	$\frac{5.6}{5}$	-1837
$V_2 - V_3$	-2460	$\frac{5.6}{5}$	-2755
$V_3 - V_4$	-2460	$\frac{5.6}{5}$	-2755
$V_4 - V_5$	-2460	$\frac{5.6}{5}$	-2755
$V_5 - V_6$	-1640	$\frac{5.6}{5}$	-1837
$L_1 - L_2$	0		0
$L_2 - L_3$	+1640	$\frac{5.6}{5}$	+1837
$L_3 - L_4$	+2460	$\frac{5.6}{5}$	+2755
$L_4 - L_5$	+1640	$\frac{5.6}{5}$	+1837
$L_5 - L_6$	0		0
$V_1 - L_1$	-1640	1	-1640

Member	Index Stress	Factor	Stress
V_2-L_2	-820	1	-820
V_3-L_3	0	1	0
V_4-L_4	0	1	0
V_5-L_5	-820	1	-820
V_6-L_6	-1640	1	-1640
V_1-L_2	±1640	$\frac{7.51}{5}$	±2462
V_2-L_3	±820	$\frac{7.51}{5}$	±1231
V_3-L_4	0		0
V_4-L_5	0		0
V_5-L_4	±820	$\frac{7.51}{5}$	±1231
V_6-L_5	±1640	$\frac{7.51}{5}$	±2462

Determination of Maximum Stresses

The maximum stress occurs in top and bottom chord when all panel points are fully loaded. The quantity for the chord members on the preceding page is maximum.

Maximum shear in panel 1-2 occurs when all panel points are fully loaded, as shown by influence line.

Hence values for U_0L_1; U_6L_7; V_1L_2; and $V\ L$ on preceding page are a maximum.

Maximum shear occurs in panel 2-3, with points 3,4 and 5 loaded.

$$V = \frac{1+2+3}{5}\,820 = 985$$

$V_2L_2 = -985$; $V_5-L_5 = -985$ Max.

$V_2-L_3 = \frac{7.51}{5}\,985 = +1480$ Max.

$V_5-L_4 = \frac{7.51}{5}\,985 = +1480$ Max.

Maximum shear occurs in panel 3-4, with points 4 and 5 loaded. $V = \frac{1+2}{5} 820 = 492^{\#}$

$V_3 - L_3$ and $V_4 - L_4 = -492^{\#}$

$V_3 - L_4$ and $V_4 - L_4 = \frac{7.5'}{5'} 492 = +749^{\#}$

V_4-L_4 is Considered as a tension Counter

Maximum stresses

Top chord	-2755
Bottom chord	±2755
$V_1 - L_1$ and $V_6 L_6$	-1640
$V_2 - L_2$ " $V_5 L_5$	- 985
$V_3 - L_3$ " $V_4 L_4$	- 492
$V_1 - L_2$ " $V_6 L_5$	±2462
$V_2 - L_3$ " $V_4 V_5$	±1480
$V_3 - L_4$ " $V_4 L_3$	± 749

Chord Members.

The chord members are subject to two loadings . A vertical load causing bending and tension due to horizontal loading

Vertical load

Foot bridge (see drawing No.2)

Rafters 2"x6"x8' $\frac{12}{21}$	.381 c.f.
Flooring2"x60"x1'	.835 c.f.
Rail & Bracing 2"x4"x6'	.333 c.f.
Floor bracing 1"x4"x5'	.139 c.f

Amount of bridge timber per foot 1.688 c.f.

Weight of yellow pine 44 lb per cu.ft.

44 x 1.688 = 75#/ft

37.5 per ft for each I Beam.

Live Load

Assume 100 lbs per lin.foot

50 lb per foot per I Beam

Live Moment $= \frac{Wl^2}{8} = \frac{50 \times 28 \times 8}{8} = 4900$ lb.ft

Impact $= \frac{300}{300 + \frac{28 \times 28}{100}} = 490 \times .975 = \underline{4780}$

Live Moment + Impact 9680 lb.ft

Assume weight of I Beam as 25 lb per ft.

Total dead load = 25 + 37.5 = 62.5 lb per ft.

Dead moment $= \frac{62.5 \times 28 \times 28}{8} = 6120$ lb ft

Live Moment = 9680

Total Moment 15800 lb ft

" " 189,500 lb in.

Design of Lower Chord

Aloow 4 - 3/8" rivet holes in

The design of this member is exactly similar to that of an escentrically loaded column for which Spofford in Theory of Structures, gives the formula (28)

$$S = \frac{P}{A} + \frac{My}{I + \frac{PL^2}{CE}}$$

in which

M = Bending moment due to vertical load.

P = Axial load

Y = Distance from c.g to extreme fiber

C = Constant; $\frac{1}{C} = \frac{5}{48}$ for uniform load.

S = Maximum stoss

L = Unsupported length

E = Modulus of elasticity

I = Moment of Inertia

Assuming 18.4" 8" I Beam

A area from Carnegia $A = 5.34 - 1.5 = 3.84\ in^2$

$$S = \frac{2755}{3.84} + \frac{189500 \times 4}{56.9 + \frac{2755 \times 5 \times 144 \times 28 \times 28}{28 \times 29000000}}$$

$$S = 717 + \frac{758000}{56.9 + 1.12} = 717 + 13050 = 13767\ lb/in^2$$

Try 7" 20# I

$$A = 5.83 - 1.5 = 4.33$$

$$S = \frac{2755}{4.33} + \frac{189500 \times 3.5}{41.9 + \frac{2755 \times 5 \times 144 \times 28 \times 28}{48 \times 29{,}000{,}000}}$$

$$I = E\ 36 + \frac{663000}{41.9 + 1.12} = 636 + 15430 = 16066$$

18.48# 8" I Beam will be used for lower chord.

Design of Upper Chord

The formula is the same for a compression member with the exception of change in sign, thus

$$S = \frac{P}{A} + \frac{My}{I - \frac{PL^2}{CE}}$$

Try 18.4 # 8" I beam

by formula 22 Spofford's Structures

$$\frac{P}{A} = 16000 - 50\frac{L}{r}$$

$$\frac{L}{r} = \frac{28 \times 12}{3.26} = 103$$

$$\text{or } \frac{5.6 \times 12}{.84} = 80$$

$$\frac{P}{A} = 16000 - 50 \times 103 = 10,850 \text{ \# per in}^2$$

$$S = \frac{2755}{5.34} + \frac{189500 \times 4}{56.9 - \frac{2755 \times 5 \times 144 \times 28 \times 28}{48 \times 29000,000}}$$

$$S = 517 + \frac{758000}{56.9 - 1.12} = 517 \quad 13580$$

$$S = 14097 \text{ \#/in}^2$$

Try 21.8 lb 9" I beam

$$\frac{P}{A} = 16000 - \frac{28 \times 12}{3.67} 50 = 11320 \text{ lb per in}^2$$

$$S = \frac{2755}{6.32} + \frac{189500 \times 4.5}{84.9 - \frac{2755 \times 5 \times 144 \times 28 \times 28}{48 \times 29,000,000}}$$

$$S = 436 + \frac{853000}{84.9 - 1.12} = 436 + 10180$$

$$= 10616 \text{ lb per in}^2$$

21.8 # 9" I Beams will be used for both top and bottom chords.

Design of members $V_1 L_2$ and $V_6 L_5$ load 2462

$$\text{Required Area} = \frac{2462}{16000} = .154 \text{ sq. in.}$$

Try $2\frac{1}{2} \times 2 \times 1/4"$ L = 1.06

$$- 1/4 \times 1/2 = \underline{.13}$$
$$.93 \text{ sq. in.}$$

Try $2 \times 1\ 1/4 \times 1/4 = .75$

$$- 1/4 \times 1/2 = \underline{.13}$$
$$.62 \text{ sq. in.}$$

Use $2 \times 1\ 1/4 \times 1/4$ L^s for diagonals.

Design of Members $V_1 L_1$ and $V_6 L_6$ load 0 1640 compression.

Use Channel for stability of I beams.

Use 6" 8.2 #/ ft channel

$$\frac{1640}{2.39 \text{ in}^2} = 686 \text{ lb per in}^2 \quad \text{adequately safe.}$$

Design of members $U_2 L_2 - U_5 L_5$ 985 lb. compression.

Use 4" 5.4# channel $\frac{985}{1.56} = .631 \text{ lb/in}^2$

Also use same channel for $U_3 L_3$; $V_4 L_4$ $\frac{492}{1.56} = .315 \text{ lb per in}^2$

Design of joints on upper chord.

Use hitch angles 4" x 3" x 1/4" L to connect [to I beams

1 - 3/8 " rivet in S. S. = 1320#

1 - 3/8" " " Bearing = 1800#

$$\frac{1640}{1320} = 1.25$$

However use 4 rivets in each leg.

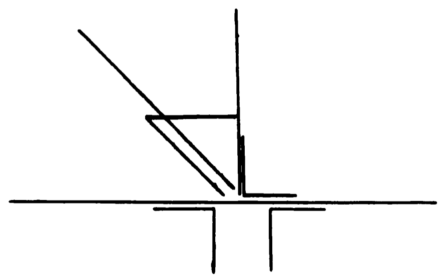
Use 1/4" gusset plate to connect angle to channel.

Load in $V_1 L_2$ = 2462 lbs.

$$\frac{2462}{1320} = 1.87$$

Use 3 rivets in both channel and angle connection.

Design of lower chord joint



Use hitch angle 4 x 4 x 1/4 L

Shear in rivets of channel leg of hitch angle = 820#, However use 4 rivets as are used in upper joints.

Rivets for I beam leg of hitch angle.

Use 4 rivets for shear.

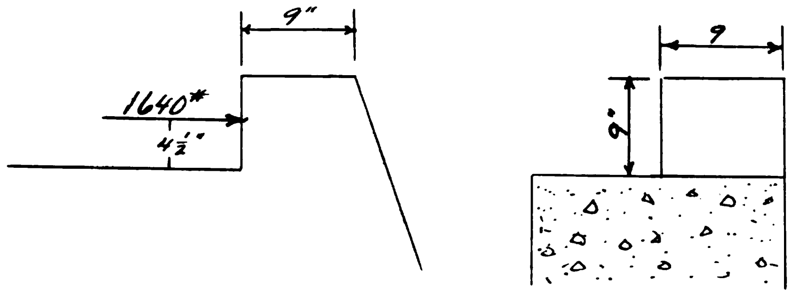
3/8" rivets in tension .11 x 16000 = 1760#

Tension transferred from I beam to connection equals 820#. Use 2 rivets for tension which makes a total of 6 3/8" rivets in horizontal leg of hitch angle.

Use the same gusset plate as is used for upper joints.

Design of Spillway Piers.

Bracket to take horizontal thrust of truss.



Bending moment = 1640 x 4.5 = 7380 lb. in.

v = 60 for shear when web is reinforced

$$d = \frac{V}{6jdv} = \frac{1640}{9 \times .875 \times 60} = 3.5"$$

However it would be advisable to use 6" = d 9" = D

$$A_s = \frac{M}{f_s jd} = \frac{7380}{16000 \times .875 \times 6} = .088 \text{ sq. in.}$$

Use 1/4" ∅ Y Bars for moment.

2 x .049 = 098 sq. in.

$$U = \frac{V}{\Sigma o jd} = \frac{1640}{2 \times .785 \times .875 \times 6} = 199 \text{ lb per in}^2$$

Extend bars 40 bar diameters 10"

Web reinforcing; Use 1/4 ∅ X bars.

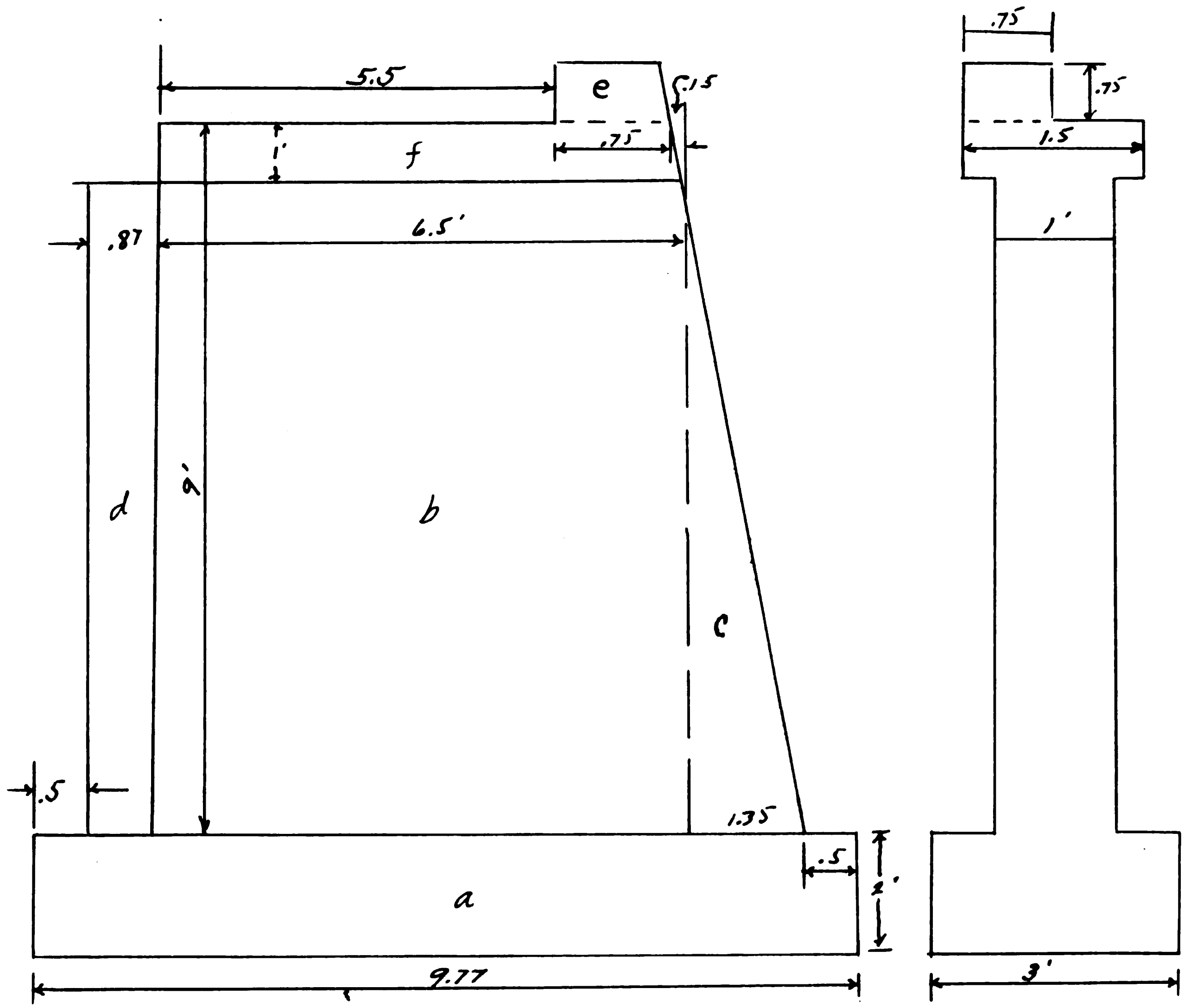
If vertical steel were used, load carried by steel would be (60-40) 9 x 6=

1080 "; at 45° $\frac{1080}{1.41}$ = 765 lb.

Capacity of X Bar = 2 x .049 x 16000 = 1570 lbs.

Piers

After testing piers of various design for overturning, sliding and soil pressure, the following design was found satisfactory.



Moments

Part	Dimensions	Wt.	$\bar{x}$	Moment Plus	Moment Minus
a	2x3x9.77x150	8800	4.89		43000
b	9x6.4x1x150	8650	5.05		46600
c	1.35x4.5x1x150	911	1.40		1280
d	.87x4.5x1x150	588	8.69		5100
e	.75x.75x.75x150	63	2.37		150
f	6.4x1.5x1x150	1440	5.05		7270
Vert. truss		762	5.50		4190
		21214			
Horz. Truss		1640	12.37	20300	
End Water Pressure	$\frac{1}{2}$62.3x36	1120	6	6710	
Pressure on Flashboard	$2.55 \times 1/2 \times 62.\cancel{4x4.5}^2$	1610	3.5	5640	
		4370		32650	104590

Resulting moment = 104590 - 32650 = 71940 lb. ft.

Momtnt distance of Resultant $= \dfrac{71940}{21214} = 3.38$

Eccentricity $c = \dfrac{9.77}{2} - 3.38 = 1.51$

$\dfrac{9.77}{6} = 1.63$

Resultant falls within middle third of base.

Factor of safety against overturning

$\dfrac{104590}{32650} = 3.2$ f.s. for overturning

Factor of safety against sliding

From table on p 14 Hool's Concrete Construction Vol 2, the coefficient of sliding friction for wet clay = 0.33

Sliding force = 4370

Resisting force = 21214 x 0.33 = 7071

$$\frac{7071}{4370} = 1.62 \text{ f.s. for sliding}$$

Soil pressure

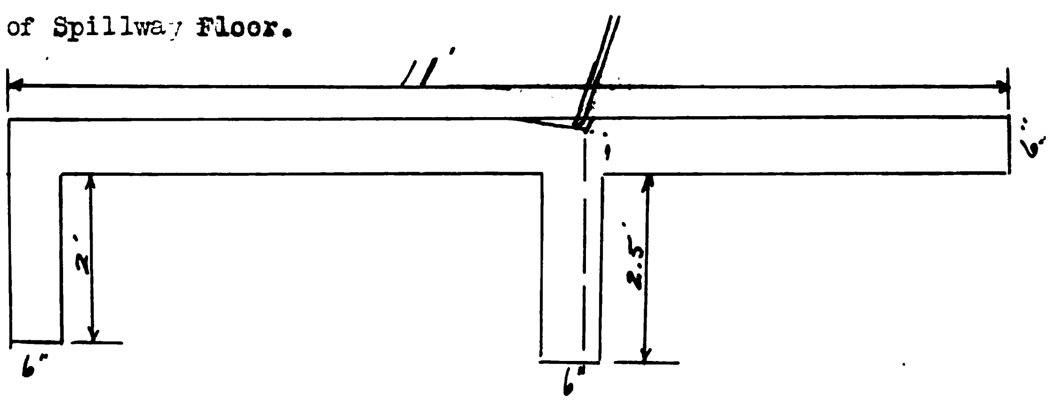
$$p = \frac{w}{t}\left(1 \pm \frac{6c}{t}\right)$$

$$p_1 = \frac{21214}{3 \times 9.77}\left(1 + \frac{6 \times 1.51}{9.77}\right) = 724\ (1 + .922) = 1392\ \#/\ ft^2$$

$$p_2 = 724\ (1 - .922) = 56\ \#/\ ft^2$$

From table on p 13 Hool's Concrete Construction Vol 2 allowable soil pressure for soft clay is 2000 lbs, per sq. ft.

Design of Spillway Floor.



Width between flash board studding 5.6'

Bottom reaction of flash board Stud = 2730#

The floor will be a 6" slab reinforced with wire mesh, with a 2' cut off wall on line of up stream edge of pier. Horizontal thrust of flash board studding to be carried into soil by cantilever wall.

Design of cantilever wall

$$b = 5.6 = 66''$$

Formula for passive earth pressure from Hool's Concrete Construction Vol 2.

Friction angle for clay on page 10 = 25 degrees.

$$P = 1/2\ wh^2 \ \frac{1 + \sin \phi}{1 - \sin \phi}$$

$$\frac{1 + \sin 25}{1 - \sin 25} = \frac{1 + .423}{1 - .423} = 2.47$$

In as much as a factor of safety against sliding of 1.5 if advisable, the pressure P used in formula will be multiplied by 1.5

$$1.5 \times 2730 = \left(\tfrac{1}{2}\ 100\ h^2 \times 2.47 \right) 5.6$$

$$h = \sqrt{\frac{4100}{50 x 2.47 x 5.6}} = 2.44$$

Make depth of wall 2.5'

Bending moment = 4100 x 1.25 x 12 = 61500 lb in.

Use $f_c = 650$, $f_s = 16000$, $K = 107.7$

$$d = \sqrt{\frac{M}{Kb}} = \sqrt{\frac{61500}{107.7 x 66}} = 2.94'' \quad \text{Use } 3''$$

Allowable v = 40 lb per sq in

$$v = \frac{V}{bjd} \ ; \quad d = \frac{V}{bjv} = \frac{4100}{66 x .874 x 40} = 1.78''$$

$$d = 3'' \ ; \ D = 6''$$

$$p = .0077$$

$$A_s = .0077 \times 3 \times 66 = 1.53 \text{ sq in.}$$

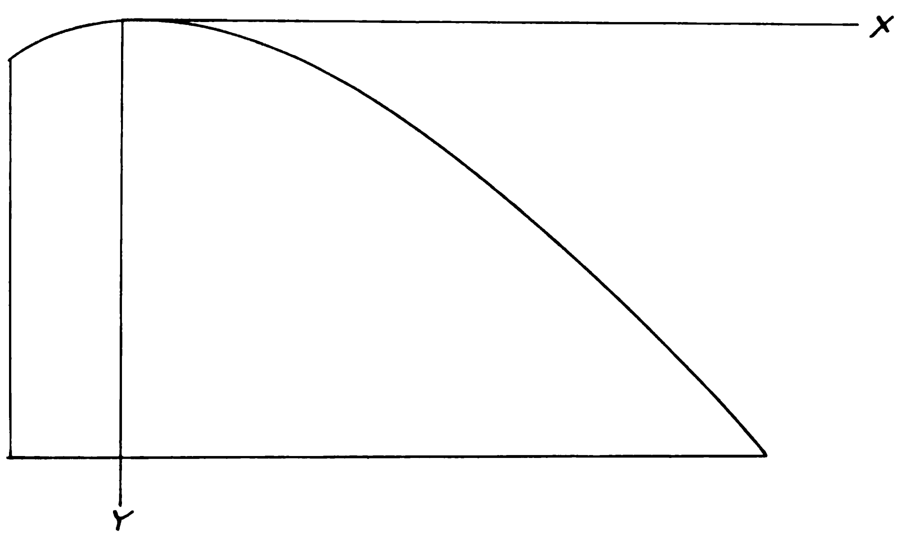
Use 11 - 3/8" ϕ bars @ 6" spacing

$$11 \times .14 = 1.54 \text{ sq. in.}$$

Design of Crest Gravity Dam.

The ends of the dam will be gravity sections over which the entire normal flow of the river will pass. The design of the crest will be in accordance with that suggested on page 208-9 of Hydro Electric Handbook by Creager & Justin. The coordinates of the curve of a crest for unit head as listed on page 209 of the Hydro Electric Handbook, are used in an attempt to arrive at a fairly accurate formula for the curve; as it will be necessary to find the area under the curve and its centroid. Such values will be necessary in determining the weight and moment of the dam.

Curve of crest (Up stream face vertical).



Representative coordinates of curve for unit head and upstream face vertical

$X - 0.7; \quad Y = 0.257 \qquad X = 2.7; \quad Y = 2.82$

The curve is of the form $\qquad X^n = AY$

$$.7^n = .257\ a$$
$$2.7^n = 2.82\ a$$

$$N \log. .7 = \log. .257 + \log. a$$

$$N \log. 2.7 = \log. 2.82 + \log. a$$

Subtracting

$$N (\log. 2.7 - \log. .7) = \log. 2.82 - \log. .257$$

$$N = \frac{\log \frac{2.82}{.257}}{\log \frac{2.7}{.7}} = \frac{1.04}{.587} = 1.773$$

$$(2.7)^{1.773} = 2.82\ a$$

$$A = 2.07$$

Formula $\qquad X^{1.773} = 2.07\ y$

A comparison between the coordinates given in the handbook and those computed by the formula will be made below.

Coordinates from handbook x	y	by formula y
0	0	0
0.1	.007	
0.3	.060	.057
0.5	.142	
0.7	.257	.257
0.9	.397	
1.1	.565	.572
1.4	.870	
1.7	1.22	
2.2	1.96	1.96
2.7	2.82	
3.2	3.82	3.81
3.7	4.93	
4.2	6.22	6.18

The usual head under which this dam will operate is 2 feet, and both coopdinates of the above curve should be multiplied by the head (2 feet) to obtain the proper curve of the crest. Hence the formula will be changed to comply with the 2 ft. head.

Let x. of unit curve equal $\frac{1}{2}$ x of new curve.

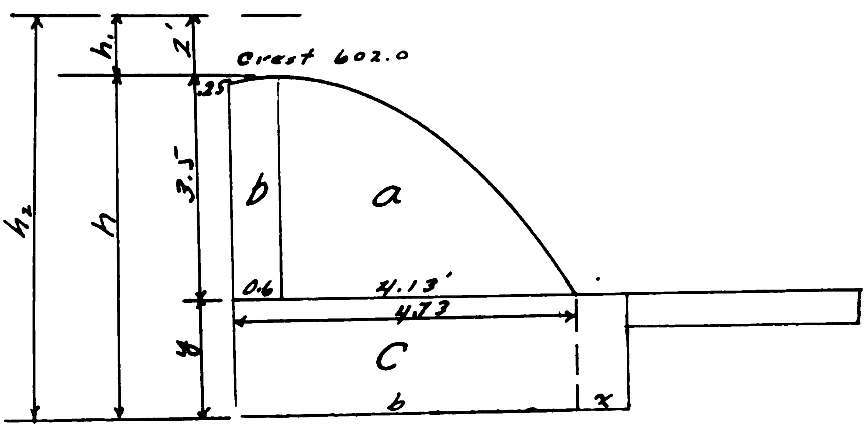
Let y. of unit curve eoual $\frac{1}{2}$ y of new curve.

hence

$$\left(\frac{x}{2}\right)^{1.773} = 2.07 \left(\frac{y}{2}\right)$$

$$\frac{x^{1.773}}{3.42} = \frac{2.07}{2} y$$

$$x^{1.773} = 3.54\ y$$



Coordinates

x		y	
.5	0'-6"	.0825	0'-1"
1.0	1'-0"	.292	0'-3½"
1.5	1'-6"	.579	0'-7"
2.0	2'-0"	.965	0'-11½"
2.5	2'-6"	1.43	1'-5 1/8 "
3.0	3'-0"	1.98	1'-11¾"
3.5	3'-6"	2.61	2'- 7 3/8"
4.0	4'-0"	3.31	3'- 3¾"
4.13	4'-1¼"	3.5	3'-6"

Area under curve.

$$x^{1.773} = 3.54\,y$$
$$x = 2.04\,y^{.564}$$

$$A = \int_0^{3.5} x\,dy = 2.04 \int_0^{3.5} y^{.564}\,dy = 2.04 \, \frac{1}{1.564}\, y^{1.564}\Big]_0^{3.5}$$

$$= 1.304\, y^{1.564}\Big]_0^{3.5} = 1.304 \times \overline{3.5}^{1.564} = 9.26 \text{ sq.ft.}$$

Centroid

$$A\,\bar{x} = \int_0^{3.5} \tfrac{1}{2}\, x^2\,dy = \int_0^{3.5} \tfrac{1}{2}\, \overline{2.04}^2\, y^{2 \times (.564)}\,dy$$

$$= 2.08 \int_0^{3.5} y^{1.128}\,dy = \frac{2.08}{2.128}\, y^{2.128}\Big]_0^{3.5}$$

$$A\,\bar{x} = .976 \times \overline{3.5}^{2.128} = .976 \times 14.4 = 14.07$$

$$\bar{x} = \frac{14.07}{9.26} = 1.52$$

Determination of the dimensions of the footing under the dam so that the resultant will pass through the middle third. Up lift will be considered as being effective under the main portion of the dam only. A moment equation will be set up, and the equation will be solved for additional width of base (x)

M_d = Moment of dam

M_w = " " water

W_d = Weight of dam

C = Unit wt. of Concrete

w = " " " water

b = base width of crest section

y = depth of base.

Uplift = $\frac{1}{2}\, wh_2 b\, \frac{2}{3}\, b$.

$M_d = W_d X + \frac{1}{2} Cy X^2 - M_w - \frac{1}{2} Wh_2 b \frac{2}{3} b$

uplift

$= 1/3 (W_d + cxy) (b + x)$

$M_d + W_d x + \frac{1}{2} cy x^2 - M_w - \frac{1}{3} Wh_2 6^2 =-$

$\frac{1}{3} (W_d b + W_d x + bcyx + cyx^2)$

$(\frac{1}{2} Cy - \frac{1}{3} Cy) x^2 - (\frac{1}{3} W_d - W_d + \frac{1}{3} bcy) x$

$+ (M_d - M_w - \frac{1}{3} W_d b - \frac{1}{3} Wh_2 b^2) = 0$

The equation is now in quadratic form

Valuation of the known quanities.

		Wt.	$\bar{X}$	Moment
a	9.26 x 150	1390	2.61	3630
b	.6 x 3.5 x 150	315	4.33	1365
c	4.73 x 3 x 150	2130	2.36	6030

$W_d =$ 3835 lb. $M_d =$ 10025 lb.ft.

Static Water Pressure $P = \frac{1}{2} W (h_2^2 - h^2)$

$= \frac{1}{2} 62.4 (\bar{9}.5^2 - \bar{6}.5^2 = 1500\#$

Centroid of Water Pressure

$$\bar{Y} = \frac{3\ h.h + h^2}{6\ h. + 3\ h} = \frac{3 \times 3 \times 6.5 + 6.5 \times 6.5}{6 \times 3 + 3 \times 6.5} = 2.68$$

$M_w = 2.68 \times 1500 = 4020$ lb. ft.

Impact $P_1 = \frac{h \cdot w\ v^2}{g} = \frac{62.4 \times 6.5 \times 4 \times 4}{32.2} = 202$ lb.

Considered to act $\frac{1}{2}$ h.

$M_I = 202 \times 1.75 = 354$ lb. ft.

$M_d = 4020 + 354 = 4374$ lb. ft.

Substituting the above values in formula and solving for 'x'

$(\frac{1}{2} 150 \times 3 - \frac{1}{3} 150 \times 3)x^2 - (\frac{1}{3} 3835 - 3835 + \frac{1}{3} 4.73 \times 150 \times 3)x$
$+ (10025 - 4374 - \frac{1}{3} 3835 \times 4.73 - \frac{1}{3} 62.4 \times 9.5 \times 4.73 \times 4.73) = 0$

$75 x^2 + 1846 x - 4809 =$

$$x = \frac{-6 \pm \sqrt{6^2 - 4\ ac}}{2a} = \frac{1846}{150} \pm \frac{\sqrt{1846^2 + 4x75x4809}}{150}$$

$$x = -12.3 \pm \frac{\sqrt{3,410,000 + 1,442,000}}{150}$$

$$x = -12.3 \pm 14.7$$

$x = 2.4$ Or -27.0

The width of the base will be increased 2.4 feet

Resistance to sliding

N = Weight of dam - uplift

$$N = 3835 + 3x2.4x150 - \frac{9.5\ x\ 62.4}{2}\ 4.73 = 3515 \text{ lb.}$$

Coefficient of friction on wet clay = 0.33

$3515 \times .33 = 1171$

Total horizontal force = 1500 + 202 = 1702

$$= \frac{1171}{1702} = 0.69 \quad \text{Not safe}$$

Cantilever cut off wall will be used to prevent sliding.

As it is desirable to have factor of safety against sliding off 1.5, the horizontal force will be multipled by 1.5

To be taken by cantilever wall =

$$1.5 \times 1702 - 1171 = 1389 \text{ lb.}$$

Passive earth pressure $P = \frac{1}{2}\ Wh^2\ \frac{1 + \sin}{1 - \sin}$

Angle of internal friction = 25^o

$$\frac{1 + \sin 25^o}{1 - \sin 25^o} = \frac{1.423}{1 - .423} = 2.47$$

$$P = (\tfrac{1}{2}\ 100\ h^2 \times 2.47\ )$$

$$h = \sqrt{\frac{2 \times 1389}{100 \times 2.47}} = 3.36, \text{ use } h = 3.5'$$

Bending Moment = 1.75 x 12 x 1389 = 29200 lb.in.

$$d = \sqrt{\frac{M}{b\ K}} = \sqrt{\frac{29200}{12 \times 107.7}} = 4.76"$$

$$d = \frac{V}{b\ j\ v} = \frac{1389}{12 \times .874 \times 40} = 3.30"$$

$d = 5"$; $D = 8"$

$p = .0077$ $A_5 = .0077 \times 12 \times 5 = 0.463$ sq.in.

Use ½ " at 6"

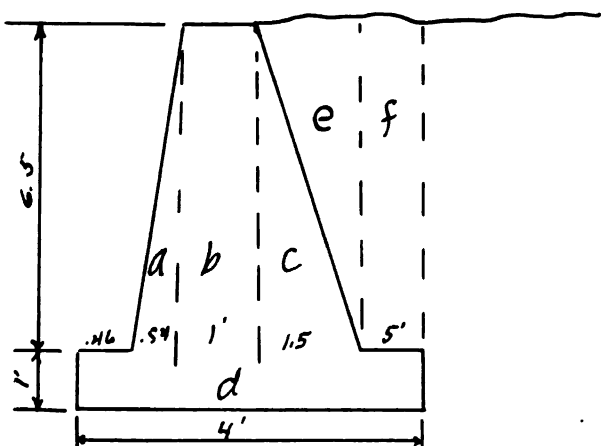
$$u = \frac{V}{20j\ d} = \frac{1389}{3.00 \times .874 \times 3} = 177 \text{ lb. per in}$$

Extend bars 50 bar diameter 25"

Design of Retaining wall.

Gravity Type

After various trials, the following dimensions were adopted.



Moment

Part	dimension	Wt.	$\bar{x}$	Moment
a	$\frac{.54 \times 6.5}{2}$ 150	273	.82	224
b	1 x 6.5 x 150	975	1.5	1463
c	$\frac{1.5 \times 6.5}{2}$ 150	732	2.5	1830
d	1 x 4 x 150	600	2.0	1200
e	$\frac{1.5 \times 6.5}{2}$ 100	488	3.0	1463
f	.5 x 6.5 x 100	325	3.75	1220
		3393		7400

$$P = \frac{1}{2}\, 100 \times \overline{7.5}^2 \quad \frac{1 - \sin 25^\circ}{1 + \sin 25^\circ}$$

$$50 \times \overline{7.5}^2 \times .402 = 1130\#$$

Overturning moment = 1130 x 2.5 = 2820 lb. ft.

$$\text{F.S. of overturning} = \frac{7400}{2820} = 2.62$$

Earth Pressure

$$x = \frac{7400 - 2820}{3393} = 1.35$$

$$C = 2 - 1.35 = 0.65 \text{ ft.}$$

$$P = \frac{3393}{4}\left(1 + \frac{6 \times .65}{4}\right) = 848 \times 1.975 = 1675 \text{ lb.ft.}$$

$$p_2 = 848 \times .025 = 21 \text{ lb. per ft.}^2$$

Sliding

Coefficient of sliding friction for wet clay = .33

Resisting force = 3390 x .33 = 1131 lb.

Sliding force = 1130 lb.

$$\text{Factor of Safety} = \frac{1131}{1130} = 1$$

A cantilever wall will be used to resist sliding. The sliding force will be increased 150%.

1130 x 1.50 - 1131 = 564 lbs.

$$\text{Earth pressure } P = \frac{1}{2} Wh^2 \; \frac{(1 + \sin \phi)}{(1 - \sin \phi)}$$

$$\frac{1 + \text{SIN } 25}{1 - \sin 25} = 2.47$$

$$564 = 50 \times 2.47\ h^2$$

$$h = \sqrt{\frac{564}{50 \times 2.47}} = 2.14;\ 2.5 \text{ ft.}$$

$$\text{B.M.} = 564 \times 2.5 \times 12 = 17000\# \text{ in.}$$

$$d = \sqrt{\frac{17000}{12 \times 107.7}} = 3.6 \text{ use } 4''$$

$$D = 7''$$

$$v = \frac{V}{b\,j\,d} = \frac{564}{12 \times .874 \times 4} = 13.5 \text{ lb in}^2$$

$p = .0077$

$A_s = .0077 \times 4 \times 12 = 0.37$ sq. in.

Use $\frac{1}{2}$" ϕ bars at 6" = .392 sq. in.

Cost of Dam
(exclusive of foot bridge)

Spillway					
2 Piers	10.1 c.y.	@	$20.00	=	$202.00
Steel Truss	762 lb.	@	.06	=	45.72
Flash Board Studding					
4-(M25) Carnegie Cross Ties					
6' - 11½" Long	406 lb	@	.06	=	24.36
Douglas Fir Flash Boards					
	645 bd.ft.	@	90.00 per M	=	58.05
Spillway Floor	6.8 c.y.	@	20.00		136.00
Crest Gravity Dam					
Dam	26.4 c y	@	20.00	=	528.00
Apron	1.5 c y	@	20.00	=	30.00
Retaining Wall	39.2 c.y.	@	20.00	=	784.00
					$1,808.13

Cost of the different factors in the installation of a Direct Flow Wading Pool.

Cost of dam (not including foot bridge)------	$1,808.10
135.7 cu.yd.concrete in place @ 10.70 ------	1,450.00
260 sq.ft. of roof surface @ .25 ------	65.00
8 sheets of 3-9-30,in 12' widths of expanded metal @ $4.30 -----	34.40
1 Type D.B.M Paradon direct feed chlorinator	250.00
2 - 6' x 2' -6" x 3/8" cover plates @ 10.50--	21.00
1-8" globe tank float value ---------	115.00

1 - 8" Standard wedge gate valve - - - - - -	$40.00
1 -12" Standard wedge gate valve - - - - - -	80.00
2 - Floor stands for valves @ $11.00 - - - -	22.00
1 - 22" manhole frame & cover- - - - - - - -	12.00
190' of 8" C.I. pipe in place@ $1.70 - - - -	323.00
1183' of 12" drain tile @ 90¢ per ft. -- - -	1,064.70
2 - Door casing & door @ $8.00 - - -- - - --	16.00
3 Windows complete with shash & glass @ $9.00-	27.00
	$ 5,328.20

An Estimate of the Cost of Installation of a Recirculation Flow Wading Pool

Design Data For Pool

The report of the Joint Committee on Bathing Places of the American Health Association and the Conference of State Sanitary Engineers states that, "In a recirculation or flowing through pool in which the dirty or used water is continually being withdrawn and replaced by fresh or filtered water, purification of the pool water proceeds by consecutive dilution. The first portion withdrawn from the pool will all be dirty water, but, owing to the constant admixture of entering clean water with dirty water remaining in the pool, each succeeding portion of water withdrawn will consist of a decreasing proportion of dirty water mixed with an increasing proportion of clean water. In proportioning the rate at which fresh water should be added to a flowing through pool or the capacity of pumps, filters, etc., for a recirculation pool, this law must be taken into consideration."

In view of this fact and assuming as we did in direct flow pool that 500 children will be maximum per hour, but in this case adding only 500 gal. for each 20 children using pool in an hour, we have

$\frac{500}{20} \times 500 = $ 12,500 gal./hr., the amount of water to be added to pool per hour.

12,500 gal. hr. = 208.33 gal. min.

Inlets for fresh or re-purified water should be located at points so as to produce as far as possible a uniform circulation of water throughout the entire pool. Eight inlets will be used; located at intervals entirely around the perimeter of the pool as shown on plate No. 5. Three outlets will be used which will be located on center line of pool, one at midpoint between the two ends and the others 50' in opposite directions along center line from the one at center of pool.

Total water surface to be not less than 3,750 sq.ft. as was obtained in direct flow data of design.

Max. depth; 21"

Max. wide of pool 40' but there shall be a 6' side walk with low curb on outside of walk all of the way around pool. This walk will be not less than 4" in thickness.

With this design data in mind, the pool will be semi-cirular, 40' wide. The center line will a semi-circle of 210' radius and inner edge of pool to be a semi-circle of 190' radius and outer edge a semi-circle of 230' radius. This pool will have circular corners of 12.5' radius, also about the outside there will be a 6' sidewalk with low curb on outside of walk. This walk will be 4" in thickness. The bottom of pool will slope to central outlet at center of pool and this will be connected to 12" drain pipe. This is to serve the purpose of draining pool for cleaning without pumping the water back to filters. The location and size of inlet and outlet pipes are shown on Plate No. 5

The pool will be 6" in thickness reinforced with expanded metal reinforcing. The elevation of the water surface will be 602.67'.

Rapid Sand Filter

Assuming the filter will take care of 2 gal. per minute for each square foot of surface, the area required would be $\frac{208.33}{2}$ = 104.17 sq. ft. of filter bed surface. We will use a 9' x 12' bed. The sand bed will be 27" thick and will have a uniformity coefficient of not greater than 1.7 and an effective size of 35 mm. The upper edge of the wash water gutters will be 30" above top of sand. There will be 18" of gravel varying in size, the larger stones being at the bottom. The collection system will consist of galvanized iron pipe 3" in diameter with two rows of 3/8" holes spaced 4" center to center along under side. These will connect with a 8" central drain pipe which will empty into the storage tank. To wash the filter the flow will be reversed through the collection system and upward velocity of water will be 18" per min. To maintain this velocity it will require 1326 gal.| min.

The pipe and the throat of the tube is proportional to the square of the rate of flow through the tube. As the foat A drops, it opens the balanced valve so that the level of water in compartment A is the same as in B. The effective head forcing chemical solution through the control valve is, therefore, proportional to the difference of pressure causing the flow through the venturi tube. The rate of flow of chemical is, therefore, proportional to the flow through the Venturi tube.

There will be two pumps, the smaller one of 250 gal.per min.

capacity to be operated whenever filter is supplied with water. The larger pump of 1200 gal. per min. capacity will be operated whenever filter is washed. When water is added to pool after it has once been filled with either be taken from river or from a well which is in operation at the park as their supply is not reliable and, using a rate of 1326 gal. per min. it will be possible to wash filter for a period of 6 min. before supply of stored water is exhausted.

There will be 28 holes of 3/8" diameter in each piece of pipe in collection system. Then $\frac{28 \times 16 \times 3.1416 \times .1875^2}{108 \times 144} =$ .0318%. orifice ratio.

The walls and floor will be 8" in thickness and it will be possible to obtain a head of 5½ ft. above sand before the filter must be washed.

There will be an apparatus for dosing supply to filter with a sufficient amount of alum to effect a layer of schmutzdecke. The successful operation of the filter is dependent upon this layer. A detail cross section of this dosing apparatus is shown on plate No. 5, and it operates as follows: The water to be treated passes through the venturi tube from right to left. The difference in pressure between may contain objectional minerals. The supply from river is the better to use.

The pool will occupy relatively the same position as direct flow pool did as shown on plage No. IV. The building will be located at any convenient place near outlet to pool. For general arrangement of recirculation system, see plate No. V.

Chlorination.

In either the direct flow or recirculation system, a type D B M Paradon Direct Feed Chlorinator will be used. The chlorine will be applied in the small chamber where the float valve is located. In case of direct flow system, the residual chlorine content must be considerably greater than in the recirculation system. Also the dose of chlorine of direct flow must be greater in most cases because of the condition of water than in the recirculation system.

Estimated Cost of a Recirculation System Wading Pool

206 Cu. Yd. of concrete in place @ $11.50	$2369.00
1170 Sq.Ft. of roof surface in place@ .25¢	292.50
1 Type D B M Paradon Direct Feed Chlorinator	250.00
1 Proportional chemical feed apparatus including venturi meter	150.00
Expanded metal reinforcing	34.40
1,250 ft. of 12" drain tile in place@ 90¢	1125.00
1 8" globe float valve	115.00
9.34 cu. yd. of sand @ $5.00	46.70
5.73 cu. yd. of gravel @ $2.00	11.50
1 - 250 gal. per min. centrifugal pump, including motor	1000. 00
1 - 1200 gal. per min. centrifugal pump, including motor	2,000.00
20½ ft. of 2' x 3/8" C.I.coverplate	28.90
9 - 8" gate valves @ $40.00	360.00
1 - 12" gate valve @ $80.00	80.00
4 Windows including gass & frame @ $9.00	36.00
3 Doors including frames @ $8.00	24.00
2650' of 8" C.I. Pipe 100' of 6" C.I. Pipe 83' of 3" perforated pipe 4 - 6" ¼ bend 10- 8" ¼ bend 1 - 8" x 8" Cross 8 - 8" x 3" crosses 3 0 8" x 8" Tees	725.00
Suitable intake constructed of concrete, lump sum	50.00
Total	$8,698.00

Drainage

It is desired to lay out an underdrain system that will drain the entire park. The low land along the river will be drained by a system of drain tile placed 50 feet apart, the high ground will be drained by tile placed 100 feet apart. This spacing of drains as choosen should be entirely adequate as the top 2 or 3 feet of the low land is a sandy loam and quite porous. The higher ground is mostly sand and gravel through which percolation is very rapid.

The low land drainage system is designed to work with in very narrow limits of elevation. It will be noted on the profile of the main draine which is also used to drain the wading pool, that there is less than a foot difference between the invert elevation of the outlet and the elevation of the bottom of the wading pool.

The plan of the drainage system is shown on the topographical map, drawing No. 1.

To determine the size of the mains, it was necessary to determine the rate at which the water is to be removed. It is the usual practice on large drainage systems to arrive at this value by predicting the largest storm that will probably occur once in a predetermined number of years. But in the case of a drainage system of so small an extent and where a rise in the elevation of the river of only two feet would render the system practically useless, a thorough investigation of past rainfall records and large storms is entirely unnecessary.

According to Pickle's Drainage and Flood Control, the average drainage modulus based on years of record is 3/8" of rain to be removed in 24 hours in the State of Michigan. This value is equivalent to 0.0151 cubic feet per second per acre.

By the use of this value, the discharge and slope of the mains will be computed.

An Estimate of the Cost of a Drainage System for Dodge Park #8.

A run-off modulus of .0151 was used for the design of this drainage system. This value was obtained from the study of the Clinton River drainage area. For the main drain, (A b C H I J K L M), a 12" main was used. This main had a slope of .055' / 100' and was designed to carry .984 cu. ft. / sec. To obtain size of all mains a table on page 161 of the 1925 edition of Drainage and Flood Control Engineering by Pickels, was used.

The elevation of 599.25' was chosen as lowest elevation for outlet. 5" drain tile was used for all laterals. For general layout of drainage system see plate No. 1, and for elevations, size, and slope of all mains, see cross section Sheet No. 6

Standard drain tile of clay will be used and following is lengths of the different sized drains with cost of same.

1,183 ' of 12" main @ 90¢ per ft. in place	$1,064.70
212' of 10" main @ 70¢ per ft. in place	148.40
3,694' of 6" main @ 45¢ per ft. in place	1,653.30
17,830' of 5" lateral @ 40¢ per ft. in place	7,132.00
Total	$9,998.40

The above prices in place include all wyes, reducers, and other connections, also with a suitable outlet to be made of concrete, in place.

Grading.

It will be necessary to fill over pipe lines D S P Q R, J O and part of the natural ox-bow; also some filling and grading underneath pool. The average length of haul will be about 500 feet as the earth used in the fill can be obtained from the higher portion of park.

To make these proper fills it will be necessary to haul approximately 5,000 cu. yd. At 80¢ per cu. yd. the cost would be $4,000.

Conclusion.

In conclusion may we present a comparison of the costs of the two different types of wading pools and their respective water supplies. Also a summary of the cost of drainage and grading.

Cost of Wading Pool with direct flow system	$5,328.20
Cost of Wading Pool with indirect flow system	8,698.00
Cost of drainage system	9,998.40
Cost of Grading	4,000.00

In view of the above tabulations the Direct Flow System is the most economical to construct. It is also the most economical to maintain, because no pumping is necessary as the dam furnishes the head.

On the other hand, a more pure water is obtained with the indirect Flow System, because recirculation necessarily requires filtration as well as chlorination.