A STUDY OF THE PROPOSED DIAMOND CREEK RESERVOIR

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A STUDY OF THE PROPOSED DIAMOND CREEK RESERVOIR

1. General Descriptions.

Politically the Colorado river is an interstate as well as an international stream. Physically the basin divides itself distinctly into three sections. The upper section from head waters to the mouth of San Juan comprises about 40 percent of the total area of the basin and affords about 87 percent of the total runoff, or an average of about 15 000 000 acre feet per annum. High mountains and cold weather are found in this section. The middle section from the mouth of San Juan to the mouth of the Williams comprises about 35 percent of the total area of the basin and supplies about 7 percent of the annual runoff. Narrow canyons and mild weather preavail in this section. The lower third of the basin is composed of mainly hot arid plains of low altitude. It comprises some 25 percent of the total area of the basin and furnishes about 6 percent of the average annual runoff.

The proposed Diamond Creek reservoir is located in the middle section and is wholly within the boundary of Arizona. The site is at the mouth of Diamond Creek and is only 16 m. from Beach Spring, a station on the Santa Fe railroad. It is solely a power project with a limited storage capacity . The dam which creats the reservoir is of the gravity type to be constructed across the river. The walls and foundation are of granite. For a dam of 290 feet in height, the back water will be about 25 m. up the river.

The power house will be placed right below the dam and Nearly perpendicular to the axis of the river. It is entirely a concrete structure. The power installation would consist of eighteen 37 500 H.P. vertical, variable head turbines, directly connected to 28 000 kwa. 110 000 v. 3 phase, 60 cycle, generators with necessary switching and auxiliary apparatus. Each unit is to be fed by a separate penstock wholly embedded into the masonry.

Concering the power market, the main electric transmission lines would extend to Prescott, Phoenix, Mesa, Florence etc. The mining regions of the mountains of Arizona would be the most adequate market. The demand of power in the above named places might not be large at present. It will, from the ~~obs~~ observation of the writer, rapidly increase with the wonderful advancement of all kinds of industrial development.

All these things being comparatively feasible, there is one difficult problem: that is the silt. At the Diamond Creek dam site the average annual silt discharge is about 82 650 acre feet. The geographical conditions, however, will not permit silt deposites right in the reservoir. So this design will be made under the assumption given in Section 4.

The silt condition and the change of lower course of the Colorado are much like those of the Yellow River in China. But one thing is different. On the Colorado most of the canyon walls are of granite, while those on the Yellow are of alluvial loess: so it is very hard , if not impossible, to get a favorable dam site on the lower part. As a visitor to this country , I should like to see the full development of the Colorado: but how about THE YELLOW!

2. Stream Flow.

The discharge of the Colorado at Yuma, Arizona has been reliably recorded since and including the year 1903. But what is rearly wanted is the discharge at the mouth of the Diamond Creek. In order to do so, the following method has been adopted.

Average discharge of the Colorado at Yuma, 1903-1920=17,000,000.A.
Diverted above by Yuma Project,.................. = 150,000.
Loss by evaporation from river bed below Diamond Creek by assuming an average width of the river to be about 0.2 miles in the 450 miles distance, and an annual evaporation depth of 5 feet ....= 288,000.
Water consumed by irrigation below Diamod Creek with an average demand of 3 acre feet per acre .....= 117,000.
Sum of above ....=17,555,000.

Average discharge of Gila, 1903-1920 ....=1,060,000.
Average discharge of Virgin, 1909-1918 ....= 208,000.
Sum of above ....=1,268,000.

Remainder of water at Diamod Creek=17,555,000-1,268,000
=16,287,000 acre-feet.

Then, $\frac{16,287,000}{17,000,000}$ = 0.956 or 95.6 per-cent.

That is to say the discharge of the Colorado at Diamond Creek is about 95.6 per cent of that at Yuma. But mean annual contribution of Little Colorado, Kanab Creek, Williams, ... to the discharge of the Colorado at YUma ranges from 150,000 to 550,000 acre feet. Sothe error would be the minimum, if we take the percentage as 95 instead of 95.6 . By applying this percentage to the records of discharge of the Colorado at Yuma, the results should be those at the mouth of Diamond Creek as shown in Table 1.

For the purpose of estimating water power no save deduction can be made from average discharge conditions, although a clear knowledge of such conditions is desirable. The information that is needed for the consideration of water power is a complete knowledge of maximum and minimum conditions and of the length of time during which each stage is likely to occure throughout the year, or throughout a period of years. The extreme and average low water conditions generally control the extent of the plant which is to be installed.

In considering the variation in the monthly flow of a stream the flow should be considered in the order of their monthly discharge rather than in the chronological order. InTable 1 , the mean monthly flows are given . These flows are arranged in the chronological order of month. An eximination of this table shows that the minimum monthly flow does not occure in the same month of each year. In table 2 these data have been rearranged. In this arrangement the least flow of any month in a given year is placed in the first line and the flows for other monthsare arranged progressively from minimum to maximum. Theaverage for each month will,by this arrangement ,give abetter criterion of the average water power to be expected in each year.

From table 2,it will be seen that the average minimum mon thly flow is 339.45 acre feet per month or 5.7 cu, ft, per sec. From this table it is not difficult to figure out approximately that during how many per cent of the time of each year, acertain quantity of flow could be obtained without the equalization of a reservoir . If the storage effect and,further, the load factor are both considered, the econemical number of units may be determined therefrom. These considerations will be given in section 6.

In order to construct the mass curve , table3 has been prepared. It also gives the various conditions of storage of the reservoir. One portion of the hydrograph and mass curve of the Colorado at Diamond Creek are shown in Figs. 1and 2.

Table 2.
Discharges of the ColoradoRiver at Diamond Creek
arranged in order of their magnitudes.

	Minimum											Maximum
1903	177.65	180.50	253.65	304.95	357.20	383.80	495.90	634.60	809.4	1970.30	2188.80	3004.85
1904	207.10	212.80	261.25	347.30	349.60	455.05	656.45	680.20	1001.30	1346.15	1617.85	2476.65
1905	266.70	469.30	475.00	678.30	706.80	899.65	1770.80	1482.95	2138.45	2463.35	2952.60	4322.25
1906	400.90	504.45	549.10	661.20	683.p5	1073.50	1121.00	1482.00	1833.50	2280.00	3163.50	4759.95
1907	435.10	610.85	794.20	988.00	1254.00	1311.00	1406.00	1995,00	2194.50	2213.50	5358.00	5633.50
1908	369.55	456.95	555'75	644 10	776.15	929.10	940.50	1007.00	1415.50	1586.50	1900.00	2422.50
1909	491.15	533.90	584.25	733.40	817.95	929.10	1710.00	2384.50	2745.50	3163.50	4645.50	5937.50
1910	348.65	405.65	407.55	443.65	483'55	562.40	858.80	1102.00	1425.00	1624.50	2660.00	3296.50
1911	441.75	503.50	573.95	685.90	705.85	1016.50	1073.50	1149.50	1672.00	2622.00	2926.00	3629.00
1912	314.45	382.85	402.80	552.90	642.20	666.90	777.10	1197.00	1330.00	2384.50	2726.50	6108.50
1913	226.10	320.15	373.35	448.40	498.75	530.10	550.00	603.25	1235.00	1444.00	2261.00	2688.50
1914	438.90	561.45	582.35	613.70	777.10	799.90	876.85	1282.50	1292.00	3011.50	3144.50	6241.50
1915	256.50	336.30	338.20	419.90	538.80	647.90	905.35	1434.50	1700.50	1795.50	2745.50	2793.00
1916	431.30	672.60	701.10	1548.50	1558.00	1586.50	2014.00	2090.00	2147.00	2679-00	3192.00	3363.00
1917	390.00	400.90	418.00	441.75	509.20	533.90	572.85	1368.00	1482.00	2878.50	5082.50	5481.50
1918	306.85	384.75	385.70	428.45	450.30	456.00	675.00	728.65	957.60	1697.65	2527.00	3491.25
1918	219.45	291.65	308.75	378.10	515.85	575.70	621.30	896.80	1162.80	1180.85	1922.80	2109.95
1920	388.00	429.40	475.95	588.05	666.90	1057.85	1057.35	1151.40	2078.60	2514.65	2699.90	7305.50
Ave.	339.45	425.44	465.60	605.94	682.875	749.18	1002.85	1259.44	1590.08	2158.66	2984.10	4170.29

Table 3.

In this table: + sign indicates reservoir gain,

− sign " " loss.

An average evaporation loss of 7 000 acre feet per month has been added to each value of the demand.

(1) Year	(2) Month	(3) Summation of discharges in Table 1.	(4) No. of units in operation	(5) Total demand A.F.	(6) Storage in A.F.	(7) Reservoir gain or loss	(8) Spill
1903	Jan.	180500	6	523522	3076978	/ 343022	
	Feb.	358150	"	"	2731106	− 345872	
	Mar.	715350	"	"	2564784	− 166322	
	April	1524750	"	"	2850662	+ 285878	
	May	3495750	16	1470259	3358703	+ 500041	
	Jun.	6499900	"	"	Full	+ 1534591	14652 9[illegible]
	July	8688700	"	"	"	+ 718541	718541
	Aug.	9323300	9	781723	3272877	− 147123	
	Sept.	9707100	6	523522	3133155	− 139722	
	Oct.	10203000	5	437455	3191600	+ 58445	
	Nov.	10507950	"	"	3059095	− 132505	
	Dec.	10761600	"	"	2875290	− 183805	
1904	Jan.	10974400	"	"	2650635	− 224655	
	Feb.	11181500	"	"	2520280	− 230355	
	Mar.	11531100	"	"	2332425	− 87855	
	April	11986150	"	"	2350020	− 17595	

Table 3.(continued)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	May	13604000	16	1470259	2497611	+ 147591	
	Jun.	16080650	"	"	Full	+1006891	84002
	July	17426800	"	"	3295890	- 124109	
	Aug.	18428100	"	"	2826932	- 468959	
	Sept.	19084550	7	609589	2873793	+ 46861	
	Oct.	19764750	"	"	2944404	+ 70611	
	Nov.	20112450	5	437455	2854649	- 89755	
	Dec.	20373700	"	"	2678444	- 176205	
1905	Jan.	20848700	"	"	2715989	+ 37545	
	Feb.	22331650	16	1470259	2728680	+ 12691	
	Mar.	25284250	"	"	Full	+1482341	791021
	April	27422700	"	"	"		668191
	May	29886050	"	"	"		993091
	Jun.	34208300	"	"	"		2851911
	July	35979100	"	"	"		300541
	Aug.	36685900	5	495556	"		11144
	SEPt.	36952600	"	"	2991044	- 428956	
	Oct.	37421900	"	"	2764688	- 226356	
	Nov.	38100200	"	"	2747332	- 17356	
	Dec.	38999850	"	"	2951330	+ 203998	
1906	Jan.	39400750	"	"	2656574	- 294756	
	Feb.	39905200	"	"	2465368	- 191206	
	Mar.	41387200	16	1470259	2477109	+ 11741	
	April	43220700	"	"	2840350	+ 363241	
	May	46384200	"	"	Full	+1698241	1113591

Table 3 (continued).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Jun.	51144150	16	1470259	Full	+3289691	3289691
	July	53424150	"	"	"		809741
	Aug.	54545150	"	"	3070741	- 349259	
	Sept.	55206350	7	609589	3122352	+ 51611	
	Oct.	55889400	"	"	3195822	+ 73470	
	Nov.	56438500	"	"	3135824	- 60498	
	Dec.	57512000	13	1212058	2996766	- 138558	
1907	Jan.	58766000	"	"	3038708	+ 41942	
	Feb.	59754000	"	"	2814650	- 224058	
	Mar.	59754000	16	1470259	2750391	- 64259	
	April	63155000	"	"	3275132	+ 524741	
	May	65368500	"	"	Full	+ 743241	598373
	Jun.	70726500	"	"	"		3887750
	July	76360000	"	"	"		4163241
	Aug.	78554500	"	"	"		724241
	Sept.	79865500	"	"	3260741	- 159259	
	Oct.	80659000	8	695656	3359285	+ 98544	
	Nov.	81270550	"	"	3274479	- 84806	
	Dec.	81705650	"	"	3013923	- 260556	
1908	Jan.	82075200	"	"	2687817	- 326106	
	Feb.	82851300	"	"	2768311	+ 80494	
	Mar.	83791850	"	"	3013205	+ 244894	
	April	84798850	10	867790	3152415	+ 139210	
	May	86385350	16	1470259	3268656	+ 116241	
	Jun.	88807850	"	"	Full	+ 952241	800897

Table 3(contined)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	July	90707850	16	1470259	Full	+ 429741	429741
	Aug.	92123850	"	"	3365241	- 54759	
	Sept.	92767450	7	609589	3399752	+ 34511	
	Oct.	93323200	"	"	3345913	- 152639	
	Nov.	93780150	"	"	3193274	- 152639	
	Dec.	9470925	"	"	Full	+ 319511	92785
1909	Jan.	95293500	"	"	3394661	- 25339	
	Feb.	96026900	"	"	Full	+ 128811	98472
	Mar.	96956000	"	"	"		319511
	April	98666000	16	1470259	"		239741
	May	101829500	"	"	"		1693241
	Jun.	107767000	"	"	"		4467241
	July	112412500	"	"	"		3175241
	Aug.	114797000	"	"	"		914241
	Sept.	117542500	"	"	"		1275241
	Oct.	118360450	"	"	2767691	- 652309	
	Nov.	118894350	8	695656	2605935	- 161756	
	Dec.	119385500	"	"	2401429	- 204506	
1910	Jan.	120487500	12	1125991	2377438	- 23991	
	Feb.	120971050	5	437450	2423538	+ 46100	
	Mar.	122396050	14	1298125	2550413	+ 126875	
	April	124020550	"	"	2876788	+ 326375	
	May	127317050	16	1470259	Full	+1826241	1283029
	Jun.	129977050	"	"	"		1189741

Table 3 (continued).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	July	130835850	16	1470259	2808541	- 611459	
	Aug.	131398250	6	523522	2847419	+ 38878	
	Sept.	131746900	"	"	2672547	-174872	
	Oct.	132154450	"	"	2556575	-115972	
	Nov.	132198100	"	"	2476703	- 79872	
	Dec.	133003750	"	"	2358831	-117872	
1911	Jan.	133517700	6	523522	2349259	- 9572	
	Feb.	134223550	"	"	2581587	+182328	
	Mar.	135240050	"	"	3019565	+497978	
	April	136389550	"	"	Full	+625978	225543
	May	139011550	16	1470259	"		1151941
	Jun.	142640550	"	"	"	-	2158741
	July	145566550	"	"	"		1455741
	Aug.	146640000	"	"	3050241	-396759	
	Sept.	147143550	"	"	2350482	-966759	
	Oct.	148815550	"	"	2552223	-201741	
	Nov.	149501450	7	609589	2628534	+ 76311	
	Dec.	149543200	6	523522	2546762	- 81772	
1912	Jan.	150257650	"	"	2337690	-209072	
	Feb.	150660450	"	"	2216968	-120722	
	Mar.	151437550	"	"	2570546	+253578	
	April	152634550	16	1470259	2297287	-273259	
	May	155019050	"	"	3211528	+914241	
	Jun.	161127550	"	"	Full	+4638241	4429769
	July	163854050	"	"	"		1256241

Table 3 (continued).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Aug.	165184050	16	1470259	3279741	- 140259	
	Sept.	165736950	7	609589	3223052	- 56689	
	Oct.	166379150	"	"	3255663	+ 32611	
	Nov.	167046050	"	"	3312974	+ 57311	
	Dec.	167428900	"	"	3086235	- 226739	
1913	Jan.	167655000	"	"	2782755	- 383480	
	Feb.	167975150	"	"	2493316	- 289439	
	Mar.	168505650	"	"	2395827	- 79489	
	April	169949650	"	"	3230238	+ 834411	
	May	172210250	"	"	Full	+1651411	1461649
	Jun.	174898750	"	"	"		2078911
	July	176133750	"	"	"		62541
	Aug.	176683750	"	"	3360411	- 59589	
	Sept.	177182500	"	"	3249572	-110839	
	Oct.	177785750	"	"	3243233	- 6339	
	Nov.	178234150	"	"	3082044	-161189	
	Dec.	178607500	"	"	2845805	-236239	
1914	Jan.	179046400	"	"	2675116	-170689	
	Feb.	179660100	"	"	2679227	+ 4111	
	Mar.	180536950	7	"	2946488	+267261	
	April	181828950	"	"	Full	+682411	208899
	May	184973450	16	1470259	"		1674241
	Jun.	191214950	"	"	"		4771241
	July	194226450	"	"	"		1541241
	Aug.	195508950	"	"	3232241	-187759	
	Sept.	196070400	10	867790	2925901	-306340	

Table 3 (continued).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
	Oct.	196870300	10	867790	2858011	- 67890	
	Nov.	197452650	"	"	2472571	- 385440	
	Dec.	198229750	9	781723	2467948	- 4623	
1915	Jan.	198765550	"	"	2225025	- 242923	
	Feb.	200200050	10	867790	2791735	+ 566710	
	Mar.	201105400	"	"	2829295	+ 37560	
	April	202805900	16	1470259	3059536	+ 230241	
	May	205598900	"	"	Full	+1322741	962277
	Jun.	208344400	"	"	"		1275241
	July	210189900	"	"	"		325241
	Aug.	210787800	7	609589	2877222	--189689	38311
	Sept.	211044800	"	"	2606833	- 271889	
	Oct.	211464200	"	"	2805222	- 189689	
	Nov.	211802400	"	"	2605833	- 271389	
	Dec.	212188700	"	"	2332544	- 278289	
1916	Jan.	214817700	16	1470259	Full	+1208741	121285
	Feb.	216366200	"	"	"		78241
	Mar.	218456200	"	"	"		619741
	April	220470200	"	"	"		543741
	May	223662200	"	"	"		1721741
	Jun.	227025200	"	"	"		1892741
	July	229172200	"	"	"		676741
	Aug.	230758700	"	"	"		116241
	Sept.	231459800	"	"	2650841	- 769159	
	Oct.	233017800	"	"	2738582	- 87741	
	Nov.	233690499	10	867790	2543392	- 195190	
	Dec.	234121700	"	"	2301902	- 241490	

Table 3 (continued).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8
1917	Jan.	234655600	6	523522	2312280	+ 10378	
	Feb.	235073600	"	"	2206758	-105522	
	Mar.	235586450	"	"	2256086	+ 49328	
	April	237068450	10	867790	2267827	+ 11741	
	May	239946950	16	1470259	Full	+1408241	256068
	Jun.	245029450	"	"	"		3612241
	July	250510950	"	"	"		4011241
	Aug.	251878950	"	"	3317741	-102259	
	Sept.	252388150	7	609589	3217352	-100389	
	Oct.	252829900	"	"	3049513	-167839	
	Nov.	253230800	"	"	2840833	-208680	
	Dec.	253620800	"	"	2621244	-219589	
1918	Jan.	[illegible]550	"	"	2396405	-224839	
	Feb.	254312400	5	437455	2265800	-130605	
	Mar.	255270000	7	609589	2613811	+348011	
	April	255998650	"	"	2732872	+119061	
	May	257696300	16	1470259	2960263	+227391	
	Jun.	261187550	"	"	Full	+2020991	1561254
	July	[illegible]	"	"	"		1056741
	Aug.	264389550	7	609589	"		65411
	Sept.	264775250	"	"	3196111	-223889	
	Oct.	265225550	"	"	3036822	-159289	
	Nov.	265681550	"	"	2883233	-153589	
	Dec.	266110000	"	"	2702794	-181139	

Table 3 (continued).

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
1919	Jan.	266329450	6	523522	2398722	-304072	
	Feb.	266707550	"	"	2253300	-145422	
	Mar.	267223400	7	609589	2159561	- 93739	
	April	268386200	"	"	2712772	+553211	
	May	270496150	"	"	3352463	+639691	
	Jun.	272418950	16	1470259	Full	+452541	385004
	July	273599800	"	"	3130591	-2[illegible]9409	
	Aug.	274221100	6	609589	3142302	+11711	
	Sept.	274512750	"	"	2824363	-317939	
	Oct.	274821500	"	"	2523524	-300839	
	Nov.	275397200	"	"	2489635	- 33889	
	Dec.	276294000	"	"	2776846	+287211	
1920	Jan.	276960900	"	"	3353256	+576410	
	Feb.	279039500	16	1470259	Full	+608341	
	Mar.	280096850	"	"	3007091	- 412909	
	April	281248250	"	"	2688232	- 318859	
	May	283948150	"	"	Full	+1229641	
	Jun.	291253650	"	"	"		58[illegible]5241
	July	293768300	"	"	"		1044391
	Aug.	294794300	"	"	2975741	- 444259	
	Sept.	295270250	8	695656	2756035	- 219706	
	Oct.	295658250	"	"	2448279	- 307656	
	Nov.	296246300	"	"	2340773	- 107606	
	Dec.	296675700	"	"	2160584	- 180189	

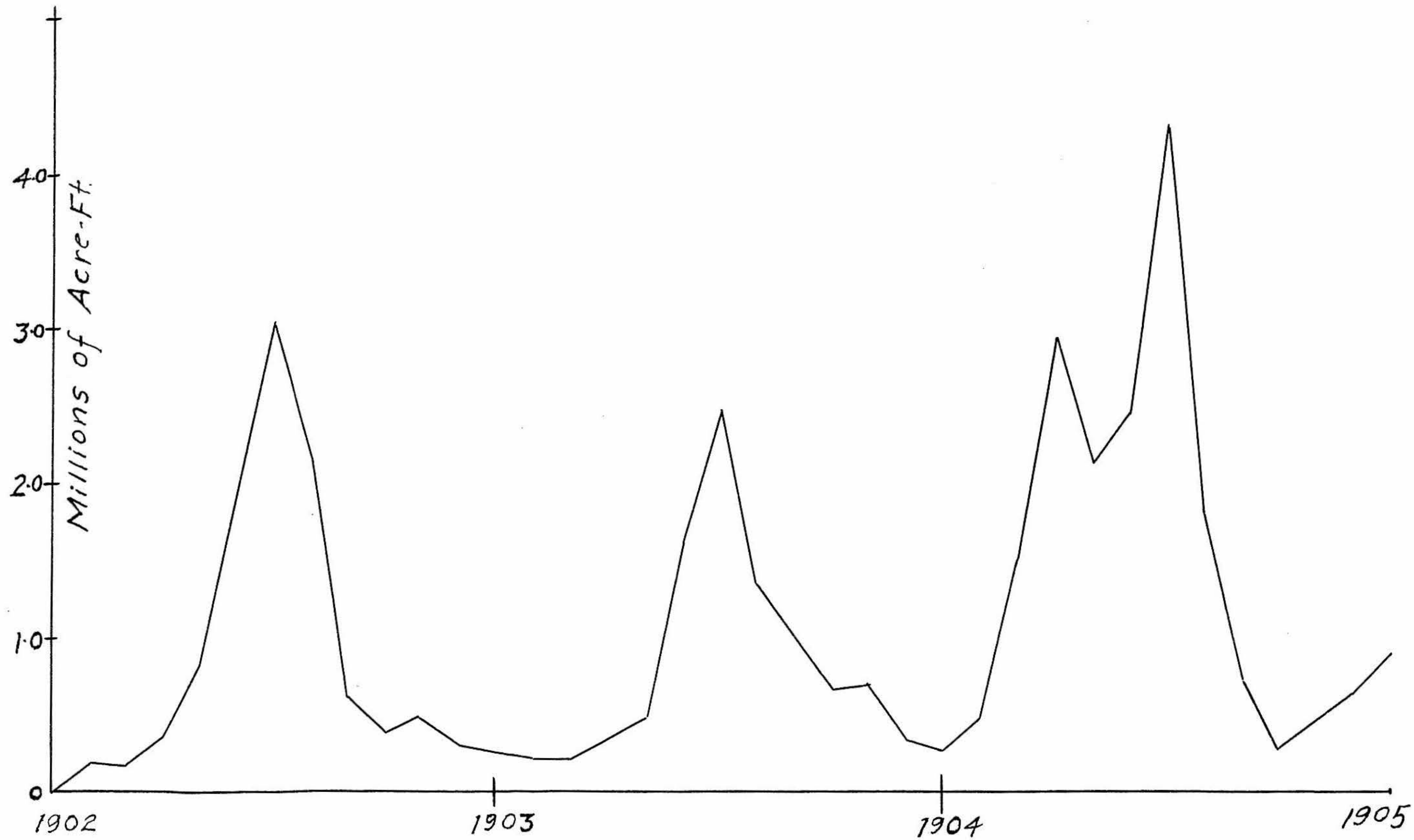
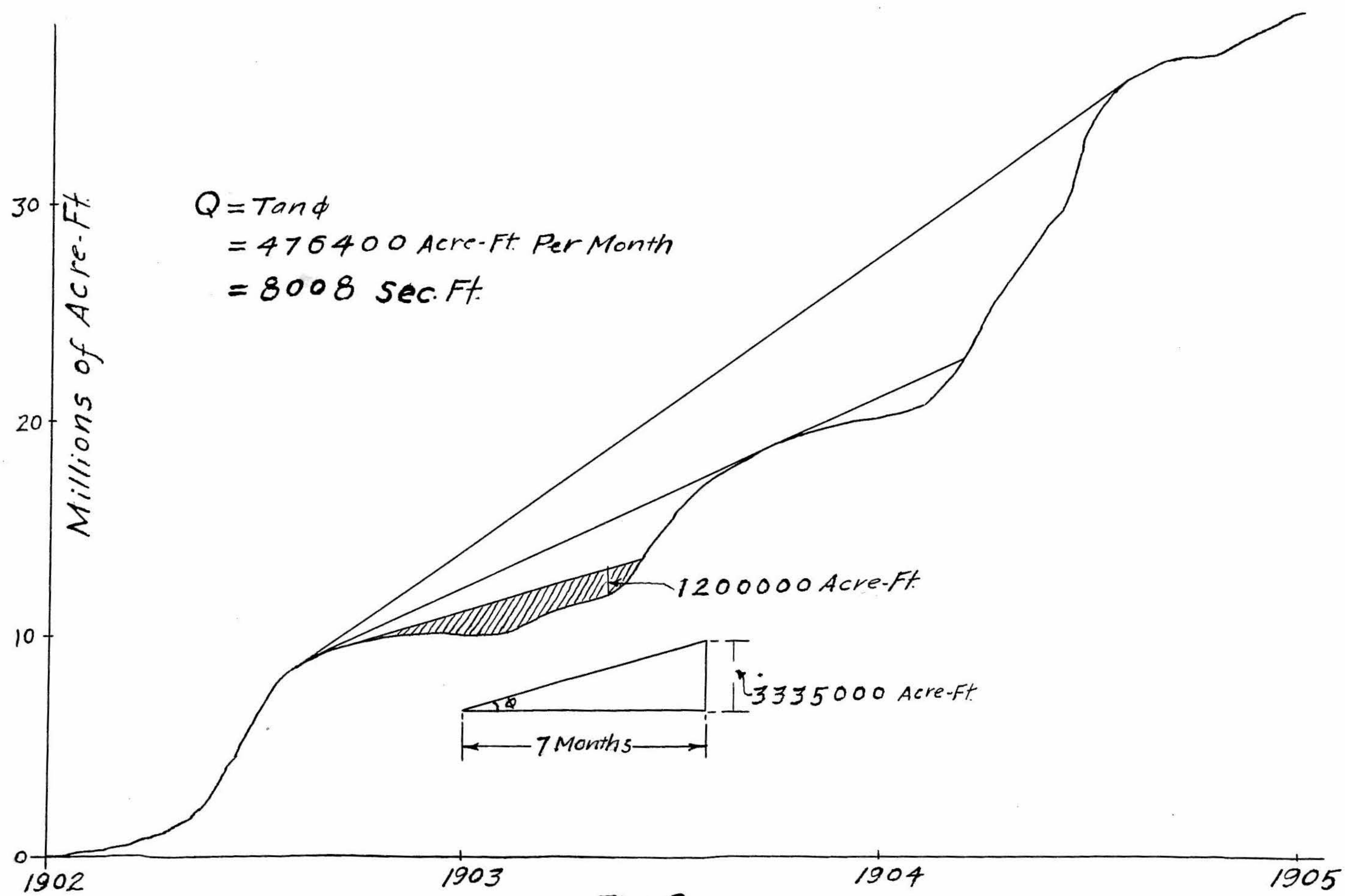


Fig. 1.



Millions of Acre-Ft.
30
20
10
0
Q = Tan φ
= 476400 Acre-Ft. Per Month
= 8008 Sec. Ft.
1200000 Acre-Ft.
3335000 Acre-Ft.
7 Months
φ
1902
1903
1904
1905

Fig. 2.

3. AREA and Capacity.

The 15 minute quadrilateral of middle latitude 35 degree 45 minute 30 second comprises an area of 241.8 square miles. By setting the arm of the plainemeter at a certain length , it gives areading of 1.56 revolutions for this quadrilateral.

Then,

$$m = \frac{241.8}{1.56} = 148 \text{ square miles per rev.,}$$

or, 95000 acres per rev.

THus calibrate the instruement. With this constant m, values shown in Table ~~3~~ 4 have been obtained. The sheet number in this table refers to maps of the U. S. Geological Survey.

At present it is not dif ficult, especially in this country, to get the quadrilateral areas at a definite latitude. So it is an eassy matter to calibrate the instruement. This method is much more convenient and gives less chance to error than the ordinary method.

With these values in the last colum of table 3, we plot an area curve. From this curve, values in colum 2 of table 4 have been obtained by scaling. Values for capacites are computed therefrom. This ~~should~~ method should give a better result than to insert contours between the two contours which are 250 feet apart as indicated in those maps. Fig. 3 shows area and capacity curves .

For adam height of 285 feet, the capacity of the reservoir will be 3,420,000 acre-feet. It is not practicable, however, to have a reservoir drawn down to below 25 percent of its maximum head. Therefore the volume of water effective for water power development will be,

$$\frac{A + A'}{2} \times \frac{H}{4} = 1,200,000 \text{ acre feet.}$$

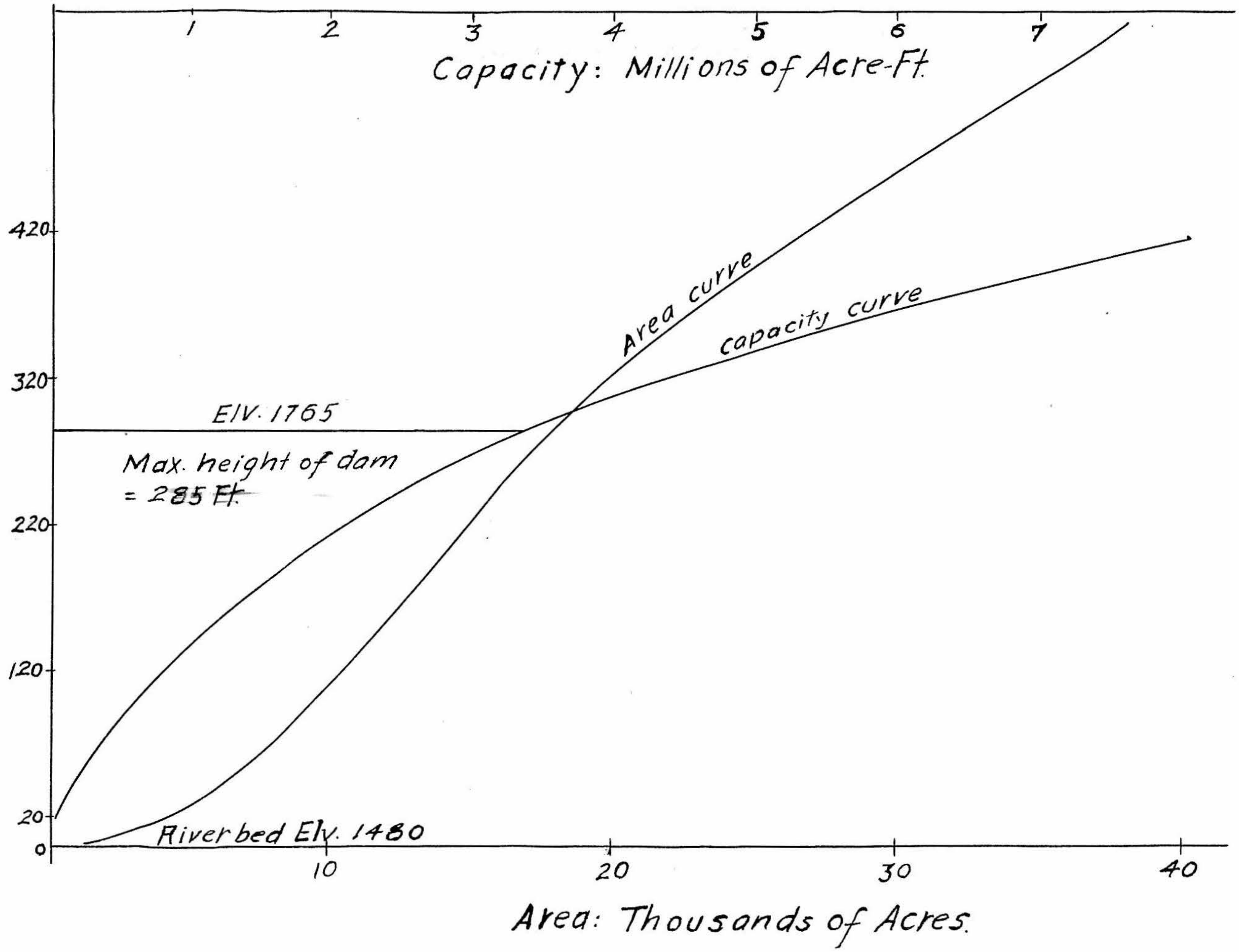
where A and A′ represent the coresponding areas of the reservoir at elevations 1765 and 1693.75 . Apply this quantity to mass curve a constant regulated flow of 8008 sec-feet is obtained. But we could Not take this as abase to figure the number of units as will be seen from the discussions in section 6.

Table 4.

Elevation	Sheet 115.2 No. of Rev.	Sheet 115.3 No. of Rev.	Sheet 115.6 No. of Rev	Area Acres
1480	0			
1500	0.043			4180
1750	0.081	0.063	0.023	15862
2000	0.141	0.083	0.165	37240
2250	0.188	0.118	0.118	51550
2500				

Table 5.

Elv.	Area in acres	Capacities in acre-feet	Summation of Capacities
1480	0		
20	4180	41890	
40	6000	107800	143600
60	7480	134800	278400
80	8500	159800	438200
100	9500	180000	618200
120	10490	199900	818100
140	11460	219500	1037600
160	12700	241600	1279200
180	13250	259500	1538700
200	14000	272500	1811200
220	15000	290000	2101200
240	15700	307000	2408200
260	16600	323000	2831200
280	17800	344000	3175200
300	18600	354000	3529200
320	19750	383500	3912700
340	21000	407500	4320200
360	22300	433000	5753200
380	....		



Capacity: Millions of Acre-Ft.
1
2
3
4
5
6
7
Area curve
capacity curve
Elv. 1765
Max. height of dam
= 285 Ft.
Riverbed Elv. 1480
420
320
220
120
20
0
10
20
30
40
Area: Thousands of Acres.

Fig. 3

X

4. Silt Content and its Influence to Reservoir Li e

The silt content of the Colorado with Gila not in flood, has averaged about 0.5 of 1 per cent at Yuma, Arizona. Since the silt contribution of the Virgin is not appreciable, so this represents approximately the silt condition at the mouth of the Diamond Creek. The discharge of the Colorado at Diamond Creek is estimated about 16530,000 acre feet per year. On this basis, the average annual silt discharge would be,

$$S= 16530000\times 0.005 \quad =82650 \text{ acre feet.}$$

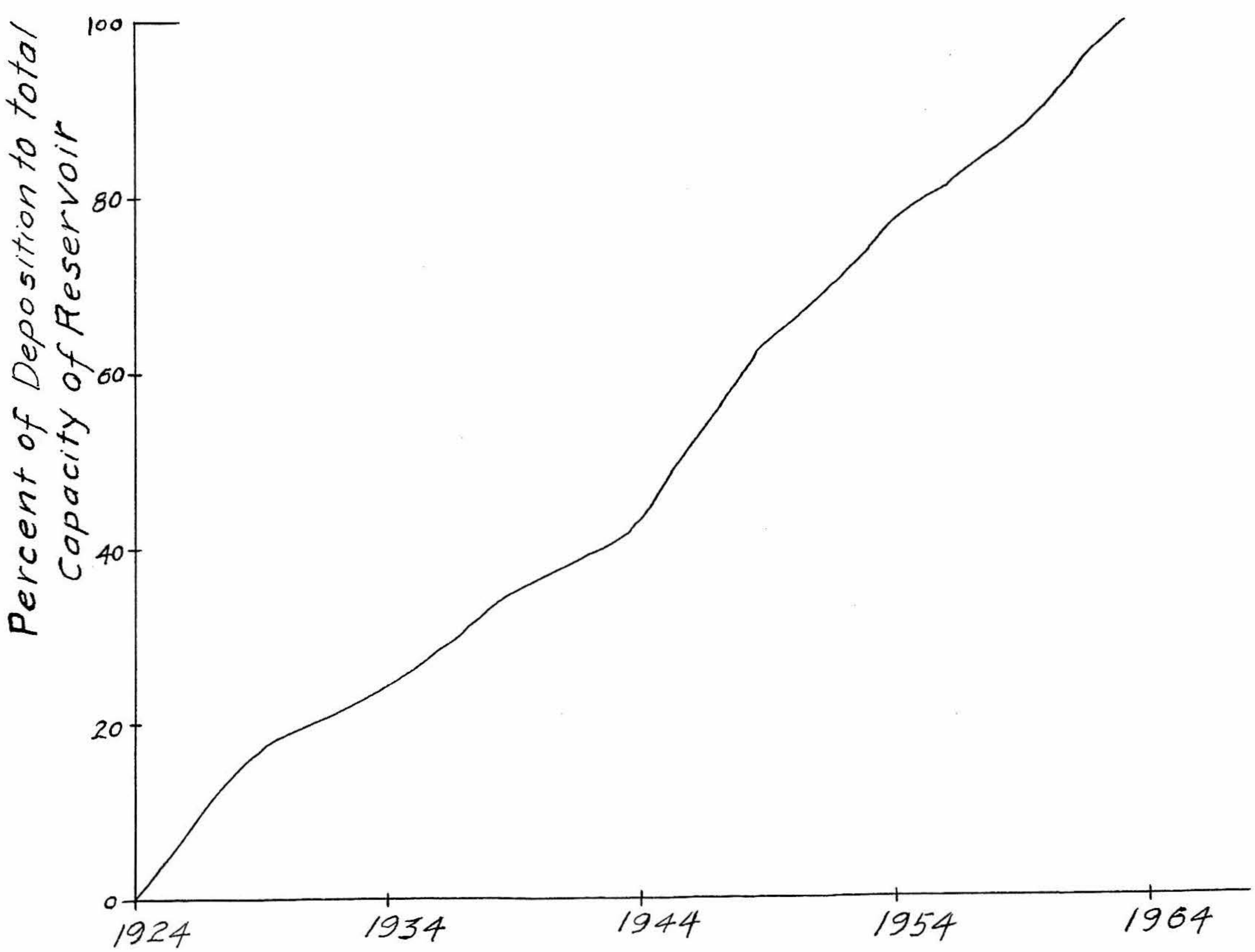
However, the capacity of the reservoir coresponding to a dam height of 285 feet is only 3420000 acre feet. Should allthe silt be allowed to deposite right in the reservoir, only about fourty years this reservoir would be wholly destroyed. This effect is clearly shown in Fig. 4.THE data upon which Fig. 4 was constructed is obtained in the following way;

1. Apply the 0.5 of 1 per cent to the values in Table 1 to get the monthly silt discharges.
2. Add these valves obtained in 1 succesively and ~~rond~~ round over.
3. Get the percentage of deposition compared with t he total capacity of the reservoir.

THen, with time as abssicia and percentages as ordinate , we consteuct Fig. 4.

It is always true that the occurance of large amount of silt in water used in power development involves difficult problems. The solution of this problem must based on a consideration of the extent and charac ter of the silt, on the distribution of silt in the water and on the silt transpor ting power of the water. Now the silt carried by the Colorado has avery fime character. Its specific gravity,as determined by Mr. Forbes, is 2.643 which coresponding to a weight of 165 lbs., per cubic feet. The content of silt is not a function of discharge, nor it is a function of the velocity of flow of the Colorado. This is especially true at Yuma , Arizona, where the Gila river discharge an enormouse amount of silt during its high water. At the mouth of Diamond Creek the writer does hope the discharge of silt is uniform. ~~But anway~~ anyway the amount would not be small.

As the capacity of the reservoir is limited by topography, so it is not justifible to allow the deposition of silt take place right in the reservoir. Meanwhile, the scouling sluces, sand box,....., all are not efective. Finally the writer takes Professor F. Thomas's assumption, that is , the Lees Fery reservoir will be developed first, so that at the Diamond Creek we will practically get clear water . This is equivalent to say that we will neglect the silt efect in the following design. Therefore only several sluce pipes are provided to remove the silt deposits right in front of the dam.



Percent of Deposition to total Capacity of Reservoir
100
80
60
40
20
0
1924
1934
1944
1954
1964

Fig. 4.

5. The Dam

The primary object of adam constructed for water power development is to concentrate the fall of the stream so that the water thus raised ca n be readily delevered to the turbines through raceways and penstocks. In rivers of steep slopes , a wing dam which occupies only aportion of the crossection of the stream may serve the ~~stream~~ purpose , at Niagara Falls. Usually in streams of ~~mod~~-moderate slope the dam must be extend entirely across the stream with a suitable height. Now between the western boundary of Grand Canyon National Park and the mouth of the Diamond Creek , the ~~Co~~ Colorado river falls only about 500 feet . So the dam is necessary to be constructed entirely across the stream.

It is well known that power is a fumction both of head and rate of flow. But, under a given condition, these factores depend upon the height of dam to a large extent. So from one point of view, it might be true that in order to get a maximum value of power the dam must be as high as possible. No; the question would not be so simmple. The overflow of valuable lands, the interference with other vested public rights, ..., and the cost of the structure are all the important factores which would tend to limit the height of the dam. However this project will not interfere any flood control, reclamation or power project on the Colorado river. All water impounded must flow out through outlets without one drop lost. Finally the factor which do limit the height ~~the h~~ of the dam would be the cost with a special reference to the local topograph From this consideration, a height of 285 feet has been adopted.

Fig. 5 . shows ~~the~~ maximum section of the dam. The assumptions made in the design are as follows;

1. An elevation of water of 285 feet or 5 feet beloww crest after construction.
2. On account of the favorable rock foundation, no upleft allowed.
3. The specfic gravity of 1: 3 :6 with plums masonry assumed at $2\frac{1}{3}$.
4. No tension permitted in the masonry.
5. No ice thrust, wave impact,... allowed.

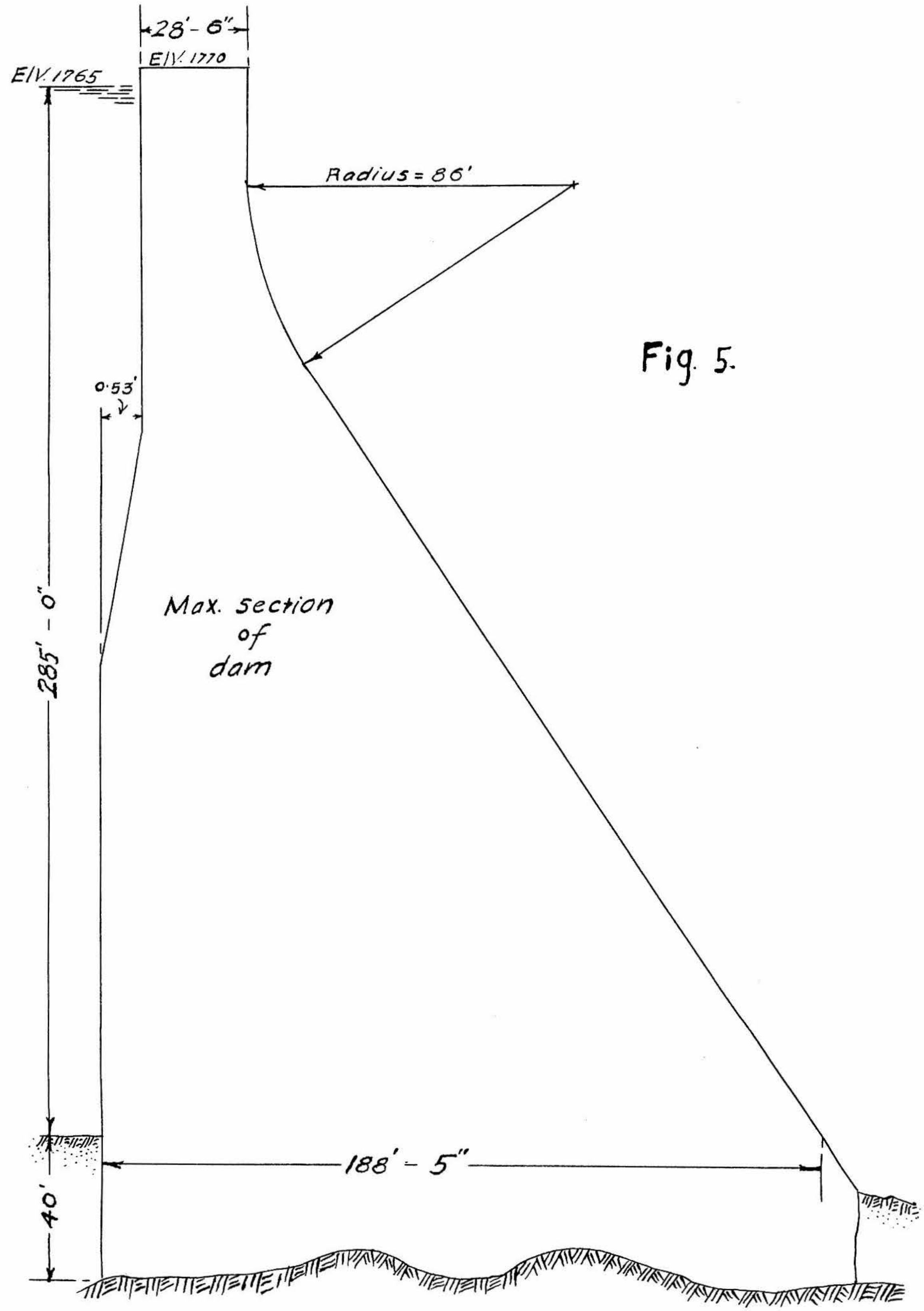
This profile is exactly Mr. Wegman's practical type NO.2:but with a different scale. It is quite satisfactory conserning the economy of material of construction and the condition of stability against sliding, overturing and crushing.

In plane the dam is curved to a radius of about 550 feet. It is founded on solid granite at a depth of about 40 feet below the river bed. Its maximum height from lovest part of the foundation to the top of crest is 330 feet. The roadway on top of dam is ~~about~~ 25 feet wide and the greatest width of foundation is about 200 feet. The dam is provided , at intervals of 100feet, with radial contraction joints. These joints are , however, closed as tight as possible. The necessary figures are shown in Table 6.

Table 6.

Depth of Water below H. L. in ft.	H. Thrust of water in cu. ft of masonry	M. of Water in cu. ft. of masonry	Joint refered to a vertical axis: Left ft.	Right ft.	Total ft.	Area in sq. ft.	Dis. from front face ft.	Dis. from back face ft.	Max. Pressures: Res. Full Tons of 2000lbs.	Res. Empty Tons of 2000lbs.	Coeff. of Friction	Factor of Safty
0.00	0	0	28.50	0.00	28.50	~~14.25~~	14.25	14.25	0.00	0.00	0.00	0.00
26.71	~~1360~~ 152	1360	28.50	0.00	21.50	761.8	12.45	14.25	2.69	1.94	0.20	8.00
42.75	389	5581	30.03	0.00	30.03	1226.5	11.20	14.34	5.24	3.38	0.31	3.4
57.00	696	13226	34.04	0.00	34.04	1679.7	11.39	14.78	7.17	5.03	0.41	2.4
74.05	1175	29000	42.81	0.00	42.81	2328.1	14.35	15.97	7.88	7.00	0.50	2.1
85.50	1565	44646	50.42	0.00	50.42	2861.4	17.47	17.34	7.97	8.02	0.54	"
99.75	2130	70894	59.90	0.88	60.78	3653.7	20.96	20.42	8.46	8.71	0.58	"
114.00	2781	105823	69.37	1.78	71.15	4593.7	24.32	23.80	9.19	9.39	0.61	"
128.25	3521	150674	78.85	2.66	81.51	5681.5	27.63	27.36	10.00	10.10	0.62	2.0
142.50	4347	206689	88.32	3.56	91.88	6916.7	30.97	31.04	10.86	10.84	0.63	"
156.75	5261	275101	97.80	4.45	102.25	8299.9	34.33	34.77	11.77	11.61	"	"
171.00	6262	357159	107.27	5.33	112.60	9829.6	37.71	38.56	12.68	12.38	0.64	"
185.25	7348	454096	116.75	"	122.08	11501.9	41.11	41.48	13.61	13.48	"	"
199.50	8512	567153	126.23	"	131.56	13309.1	44.49	44.46	14.56	14.56	"	"
213.75	9783	697572	135.70	"	141.03	15251.4	47.48	47.48	15.65	15.62	"	"
228.00	11318	846595	145.18	"	150.51	17328.4	51.11	50.13	16.47	16.69	"	"
242.25	12565	1015462	154.66	"	159.99	19541.1	54.44	53.59	17.46	17.73	"	"
256.50	14088	1205406	164.13	"	169.46	21888.5	57.71	56.69	18.45	18.78	"	"
270.75	15696	1417680	173.41	"	178.94	24870.9	60.99	59.78	19.42	19.82	"	"
285.00	17392	1653510	183.08	"	188.41	26988.2	64.27	62.98	20.41	20.88	"	"

Specific Gravity of Masonry = $2\frac{1}{3}$



28'- 6"
El. 1770
El. 1765
Radius = 86'
0.53'
Max. section of dam
285' - 0"
188' - 5"
40'

Fig. 5.

6. Type and Number of Units.
No definite rule can be taken as a guide to the choice of a type of units. Neither is it possible to draw exact conclutions that for a given case , a certain type must be selected. Each case is a separate problem. All that can be done is to broad general conclutions and to decide what have weight and what have not. In this particular case, the 285 feet indicates a somewhat high head: but the storage capacity is rather small. Then the variation in head will not serious. The modern practice shows that this will generally call for a medium speed reaction turbine. From experiments, the efficiency of a reaction turbine is proportional to its capacity. The maximum capacity of this type ever built in this country is 50 000 H. P. The 37 500 H.P. capacity of the same type is comparitively general. So there may be some advantages in getting the latter from the manufacturing companies.

On account of the modern developments of draft tube and the simplicity of the mechanical aspects, the vertical type is superior to the horizontal one. From these considerations, a vertical, single runner, variable head, 37 500 H. P. capacity reaction turbine operation at a speed of 215 r. p. m. may meet the requirments. There is no reason why the speed should be 215 r.p.m. instead of 210 r.p.m. or something else. This rather depends on the designer.

The 28 000 kw. , 11 000 v., 3 phase, 60 cycle generators are mounted right above the turbine.

In deciding the number of units, it is not sufficient to consider the hydraulic conditions only: but also the nature of service , the load factor, the operating expenses, etc. Concering these economical features, the present knowledge is rather absine in accuracy. So it would not be interest to spend much time on those ambiguous facts. However, the 56.04% yearly load factorof the South California Edison company may be worthwhile to be noticed.

The major portion of the cost of a complete development is usually on hydraulic side rather than in the electric side. the electric work rarely exceeds w0% of the hydraulic cost. But the cost of hydraulic work is the most difficult to estimate whithin any degree of accuracy. According the practice in this country the total investment per kw. developed usually ranges between $ 200 to $ 300. This will give a rough measure to the slectio n of machineries.

It is customary that companies usually make two clsses of contracts for power, known as primary and secondary power. The primary power is contracted to continuous supply throughout the year. Evidently the maximum amount of such power , if no steam auxiliaries , is limited by the regulated constant flow. In this case, The 8008 sec. ft. given in Section 3. will serve the purpose. But for this rate of drat , there will be a spill nearly all the year. This means a waste of energy. So it is ~~nec~~ necessary to develop this surplus energy in the term of seconda power. This part of power is only available for a certain time of the year . The continuity of supply , as a rule, is not guaranteed. The rates for secondary power are lower than for primary. The extent of development depends necessarily upon the market of these two classes of service. In this case we suppose the market is adequate and our interst will be to develop the full energy of the stream. However, overdevelopment may entail fixed charges which will make the earring of profit only a speculative possibility of distant future.

So it is necessary to figure out how many percent of the surplus ebergy should be developed in order to make the investment fruitfull. This will give a guide to the selection of economical number of units. In order to do this , we should figure out what is the relative percentage of the rate of flow during a portion of the year , or of a period of years. Either t the mass curve or the hydragraph will serve for the purpose. After this has been done , we get the following results:

Percent of the 18 year period	Sec. Ft.	Percent of the 18 year period	Sec. Ft.
13	34000	52	18000
17	28000	58	12000
25	25000	69	10000
33	20000		

Refering to the year load factor of the South California Edison company, The 18 000 sec.ft. is selected. Upon this basis

$$\text{Power available} = \frac{18000 \times 285}{11} = 466000 \text{ H.P.}$$

The turbine load is generally 1.25 times of that of generator load. Then,

$$\text{no. of units} = \frac{466000 \times 1.25}{37500} = 16.$$

In order to provide for reparations, at least eighteen units should be installed.

7. The Penstock.

Itis not the aim of the report to discuss fully the different theories of flow of water in a penstock or any conduit. So only those formulae which will be used for design are given here. There derivations can be readily found out from any standard treatise on hydraulics.

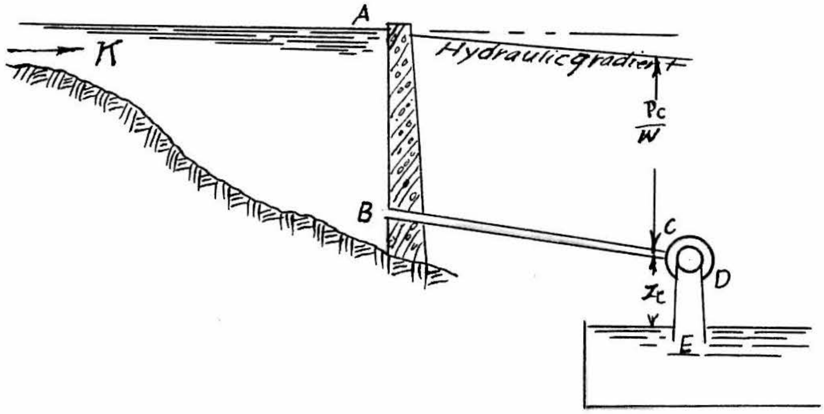


Fig. 6.

Suppose the reservoir is full having a constant head H, maintained by the inflow at K. The water flows out of the fore bay and through the penstock BC, turbine CD and draft tube DE to tailrace. Now this may be considered as one of steady flow. As the area of the reservoir surface compared with that of any outlet opening will be very large, this leads to the assumption that water particles on the reservoir surface will have no appreciable velocity of approach. Practically we may suppose that the whole body of water in the reservoir is at rest. If so, gravity will be the onl external force acting on the fluid. Then apply Bernoulli's theorem to point C, Fig. 6, we get

$$h=H-H'=\frac{P}{\rho}+Z_{\rho}+\frac{V^2}{2g}, \qquad (1)$$

where h represents the net operating head on the turbine. From this equation, we can see readily that the magnitude of H' bears very much upon the operating head. However from the principle of the conservancy of energy, it really represents that part of energy of which one portion is dissipated in the form of heat and another converts into mechanical work. If H' =0, there will be no flow.

In our case the pipe will be uniform and no sharp bends, we ~~ma~~ may write,

$$H'=0.5\frac{V^2}{2g} + f\frac{lV^2}{d2g}, \qquad (2)$$

The value of H' may be any magnitude up to the total head minus the velocity head in the pipe .

From the design of turbine, we know that

$$\frac{V^2}{2g}=0.046h ,$$

where $V_2=\frac{Q}{A}=\frac{37500\times 11\times 4}{285\times\pi(D\delta)^2}=25.2$ ft. per sec.

Consquently , $h=\frac{1}{0.046}\;\frac{(25.2)^2}{2g}=214$ ft.

Then the maximum head lost allowab le is $H'= H-h=71$ ft.

which is less than one third of the total head.

For constant discharge, V varies inversely as the square of the diameter of penstock. It is clear from equation (2), the ~~far~~ larger the diameter of the pipe, the less will be the velocity of flow and the less the head lost. Since lost head means a loss of power , the problem becomes one of determing the proper value to be assigned to this item. This question is rather complicated, because it is not only a function of physical conditions but also of economical features of the development.

So it is necessary to find the economical diameter of the pipe The following method is outlined by R. Muller:

The thickness of pipe shell is determined by the well known formula,

$$t=2.6\frac{HD}{es} \qquad (3)$$

If the joint efficiency, e=0.81 and the maximum allowable stress of steel, s=16000 lbs. per sq. in., then

$$t=\frac{HD}{4980}.$$

The weight of a steel plate 12"×12" and thickness t, weight of steel being taken as 490 lbs. per cu. ft. is w=41t. As the thickness increases in proportion to the head and for the same head increases in proportion tothe diameter. The average weight of one linear foot of pipe line is defined by $W=41t\times\pi D$ or in function of the pressure head by $W=0.026HD^2$. If the weight of the rivets, etc. is considered, 15 percent may be added and the expression becomes $W=0.03HD^2$.

The total weight of the pi e line is denoted by,

$$W_1=0.031HD^2.$$

The annual income from sale of generated power, P is

$$K=aP.,$$

where a is the cost of one H.P. per year.

The interest and depreciation of the penstock has for value

$$F_1=0.031cpHD^2,$$

where c represents the cost of steel per lb. and p the percent of interest.

If F be the fixed charge of the wholl installation, the ~~ann~~ annual expenses are represented by,

$$F+F_1=F+0.031cpHD^2.$$

The annual profits are given by the expression,

$$K'=aP-(F+0.031cpHD^2).$$

Power is denoted by ,

$$P=\frac{62.5}{550}Q(H-H')$$

If the efficiency =80percent, then

$$P=\frac{Q(H-H')}{11} \qquad (4).$$

The Vallot formula for calculating pressue pipes is,

$$D=0.376\left(\frac{Q}{\sqrt{s}}\right)^{\frac{3}{8}} \qquad (5),$$

which is equivalent to

$$s=\sqrt[3]{\left(\frac{\overset{8}{0}.376\sqrt[8]{Q^{3}}}{D}\right)^{16}}$$

$$=0.0054\frac{Q^{2}}{D^{\frac{16}{3}}}$$

But $s=H'/l$, there by substitution:

$$H'=0.0054\frac{Q^{2}l}{D^{\frac{16}{3}}}.$$

Placing the value of H' in (4),

$$P=\frac{Q}{11}\left(H-0.0054\frac{Q^{2}l}{D^{\frac{16}{3}}}\right).$$

The loss of income due to loss of head is,

$$F_{2}=0.0005\frac{Q^{3}al}{D^{\frac{16}{3}}}.$$

The annual net income will be ,

$$K=\frac{QaH}{11}-F\overset{n}{-}\left(F_{1}+0.0005\frac{Q^{3}al}{D^{\frac{16}{3}}}\right).$$

K will be a maximum when the third term of the second member is a minimu m. In order to simplify the calculations, let

$$U=0.031cpH$$

$$V=0.0005Q^{3}al.$$

By substitution the thírd member becomes,

$$X=UD^{2}+\frac{V}{D^{\frac{16}{3}}}.$$

Differentiating and equating to zero:

$$\frac{dX}{dD}=2UD-\frac{16V}{3D^{1\frac{2}{3}}}=0$$

whence,

$$3UD^{2\frac{2}{3}}-8V=0$$

$$D=\left(\frac{8V}{3U}\right)^{\frac{3}{8}},$$

Substituting back the values of U and V and take

$a=\$20$

$p=10\%$

$c=\$0.05$,

we get

$$D=18.4 \text{ ft.}$$

Although this result is theoretically correct, it may offer objection to practice. As the average length of penstock has only a value of about 300 ft. , all the existing ~~examples~~ formulae for long pipes would not be applicable. Refering to ~~so~~ some of the existing examples, a 7'6" diameter penstock is adopted. For this size, the velocity of flow at full gate opening will be 32.8 ft. per sec. Consquently,

$$H'=0.5\frac{(32.8)^2}{2g}+0.022\frac{300}{7.5}\frac{(32.8)^2}{2g}=23 \text{ ft.}$$

This value of H' is nearly twice that obtained by Weisbach's ~~fo~~ formula,

$$H'=\left(0.0144+\frac{0.1716}{\sqrt{V}}\right)\frac{lV^2}{D\,2g}.$$

The numerical computations are not shown here.

Thickness of Pipe

If p=internal pressure in lbs. per sq. in.

r=radius of pipe in inches.

s=working save stress on steel per square in. net area.

e=81 percent for triple riveted joints.

then,

$$\text{Thickness, } t=\frac{pr}{se}=\frac{62.5\times285\times45}{16000\times.81\times144}=0.428 \text{ in.}$$

Suppose we make 50 percent allowance for water harmmer, then $t=1.5\times0.428=0.645$ in. In order to carefore the durability against corrosion,...., a $\frac{11''}{16}$ thickness of pipe will be used.

The Joint

The diameter of a rivet is usually determined by an empirical rule,

$$d=1.2\sqrt{t}.$$

Therefore, $d=1.2\sqrt{11/16}=0.997$ in.

A 1" diameter rivet should be used with thickness of strap $=\frac{9''}{16}$. For this case we have the fpllowing dimensions:

$$p=3\frac{3}{4} \qquad m=3\frac{1}{4} \qquad r=2\frac{1}{4}$$

$$s=3\frac{1}{2} \qquad 0=2\frac{1}{8} \qquad v=1\frac{5}{8}$$

The arrangements are shown in Fig. 7.

Penstock Gates.

The supply of water to each penstock is controlled by a balanced valve of the same design as that at Shoshone reservoir They are located as shown in the detail drawing .

Directly above the spiral cases, Johnson needle valves will be installed as emergency gates. The installations are also shown in detail drawing.

Manholes.

In order to make necessary examinations , when the pipe is ~~em~~ empty, manholes are provided. Such manholes have the cover bolted on which may be removed to allow a man to enter. The cover and its attachment are designed so as to withstand the static pressure and water hammer effect. The diameter is here assumed to be 18".

The static pressure acting on this cover is easy to determine. If, a 50 percent pf pressure is allowed for water hammer, then the pressure acting on the cover will be

$$P=62.5\times15\times\pi\times(.75)^2\times285,$$
$$=47200 \text{ lbs. per sq. ft.}$$

For such a pressure 16-1" diameter bolts will be required.

Expansion Joints.

Owing to the variation of temperaeure, steel pipes undergo a large amount of expansion and contraction. To take care of these changes, expansion joints are necessary . For long pipe li lines , it is customary to put such joints at intervals of 500 feet or so. But for our case only one will be used as shown in the detail drawing. This takes care of the expansion and contraction in its self and allows the pipe to conform with the grade to permit a slight deflection without leakage.

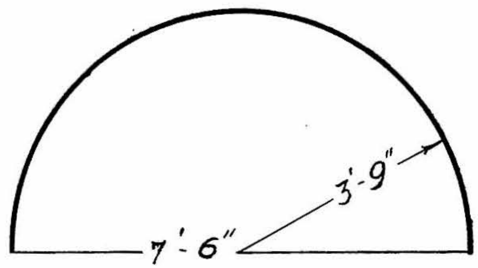
Air Valves.

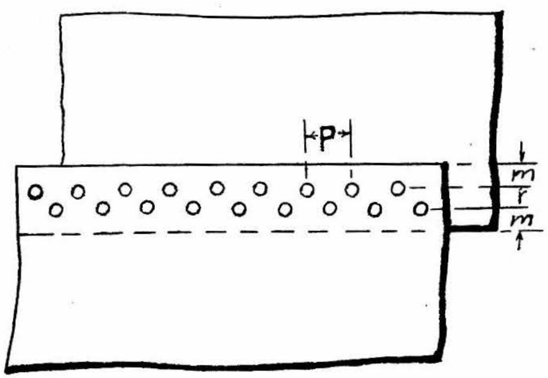
Air valves are generally called relief valves a nd designed to operate automatically being located at the summits of a pipe line. They permit the escape cf air in filling and entrance of air in emptying a pressure pipe. The location of thi valve is shown in detai drawing.

Flanges.

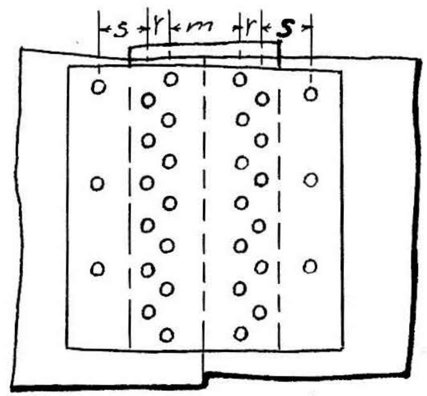
Ptessure pipe flanges must be attached securely and rigidly to the pipe, making a tight joint and eliminating absolutely the po possibility of a leakage. The material of such flanges is to be preferably of forged steel.

Fig. 7.





Circumferential lap Joint
Double riveted



Butt strap Joint
Triple riveted

8. The Case and speed Ring.

The most efficient form of turbine casing in use at present is that of a spiral shape. The materials mostly used are cost iron and cost steel, the choice between them being influenced by consideration of the stresses imposed. As in this case we have rather a high head, cost steel will be used.

The spiral case may begin from any point. For convenience, we assume that it begins ~~from~~ on the transverse axis of the unit. Calling the discharge in the case to the left of unit D_1 and that to the right of unit D_2, the following table shows the discharges to be provided for.

α	180°	225°	270°	315°	360°
D_1	Q	7Q/8	3Q/4	5Q/8	Q/2
D_2	Q/2	3Q/8	Q/4	Q/8	0
Total	3Q/2	5Q/4	Q	3Q/4	Q/2

These are also shown in Fig.8 .

From observations, when the water in a spiral case approaches a turbine runner, a free vortex is formed. The formation of suc a vortex is only by virtue of its own angular momentum previously derived from some source and free from external forces . Hence the angular momentum is constant. Then the tangential velocity is also constant. By allowing 2.5 ft. for th space of speed ring, the distance from crntre of turbine to the centre line of the case may be easily determined. Calling t the distance on right of unit R_1 and that on left of unit R_2, Table 7 gives all of these values.

Table 7.

V=32.8 ft. per sec. Q=1446.5 cu. ft.per sec.

α	180°	225°	270°	315°	360°
R_1	10.58	10.37	10.08	9.79	10.58
R_2	9.45	9.13	8.71	8.16	0
Total	20.03	19.50	18.79	17.95	10.58

9. The Turbine

For a maximum head, ...H=285 feet,
a capacity of each unit, ... =37500 Horse Power,
and a speed, ...N=215 r. p. m.,
we have the specific speed ,

$$N_s = \frac{N\sqrt{HP}}{h^{5/4}} = \frac{215\sqrt{37500}}{285 \times 285^{\frac{1}{4}}} = 35.6$$

With such a value of specific speed, we get, from Fig. 126., R. L. Daugherty: " Hydraulic Turbines",

$$\varphi_e = 0.72$$
$$D_2 = 0.75D$$
$$D_d = 0.99D$$
$$D_t = 1.00D$$
$$B = 0.25D$$

Consquently the diameter of the runner, is

$$D = \frac{1840\,\phi_e\sqrt{h}}{N} = \frac{1840 \times .72\sqrt{285}}{N} = 104 \text{ In. or, } 8.66 \text{ Ft.}$$

Then, $D_2 = 0.75 \times 104 = 78$ In. or, 6.49 Ft.
$B = 0.25 \times 104 = 26$ In or, 2.16 Feet.
$D_d = 0.99 \times 104 = 103$ in or, 8.57 Feet.
$D_t = 1.00 \times 104 = 104$ in or, 8.66 Feet.

All these quantities are clearly shown in Fig. . Also from Fig. 127. of the above mentioned Text, we find,

$$E = 0.875$$
$$E_h = 0.91$$
$$\alpha = 26 \text{ degrees.}$$
$$\beta = 87 \text{ degrees.}$$

$$\frac{V_2^2}{2g} = 0.046H$$

Then,

$$C_r = \frac{0.0000157\,N_s^2}{\phi_e^2\,(B/D)\,E} = 1.75\ \%$$

$$C_u = \frac{E_h}{2\phi_e} = \frac{0.91}{2 \times .72} = 0.632.$$

The direction of the absolute velocity of water entering the runner can be determined by,

$$\operatorname{Tan}\alpha = \frac{C_r}{C_u}$$

The direction of the relative velocity which should also be the direction of the runner vanes is given by,

$$\tan\beta = \frac{c_r}{c_u - \phi_e}$$

The number of guide vanes and runner vanes may be found by Zowsti's rule:

The No. of ~~guide vanes~~ runner vanes,

$$n = K\sqrt{D} = 3\sqrt{104} = 31$$

The No. of guide vanes,

$$n = K\sqrt{D} = 3.5\sqrt{104} = 36.$$

Diameter of Shaft.

In this case the generator is built on top of the turbine case, and so a single continuous shaft will be used both for the turbine and the generator. The diameter of the shaft may be determined by the following formula:

$$D = \sqrt[3]{\frac{100\,\Pi\,0}{N}} = 26 \text{ In.}$$

A 2Ft. 6 In. diameter shaft should be used.

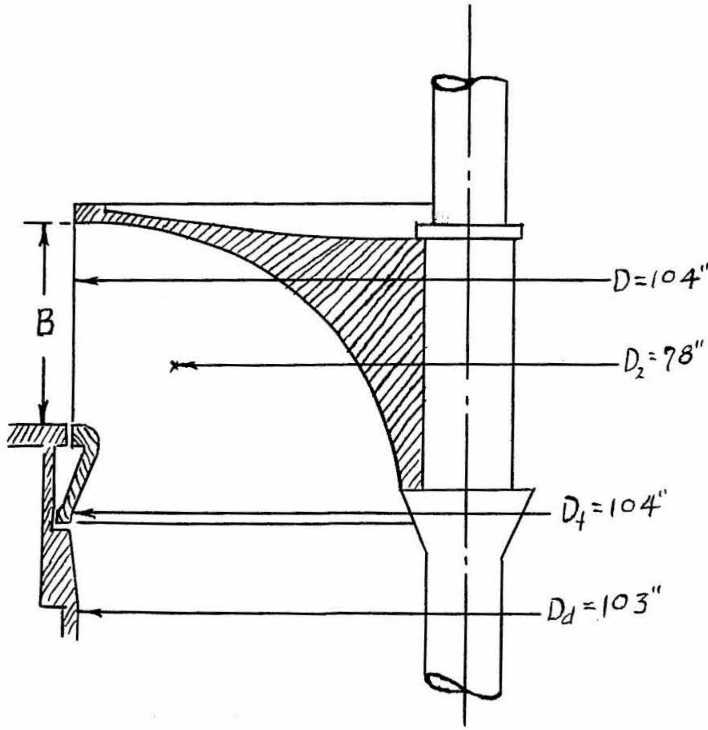


Fig. 9

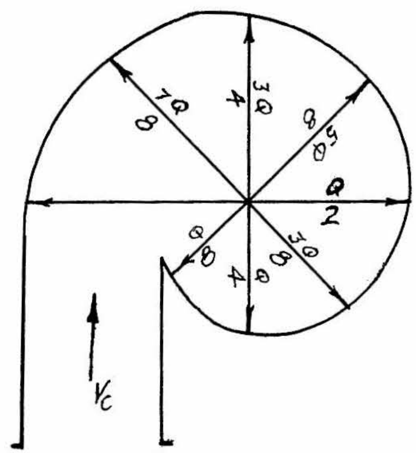


Fig. 8

10. The Draft Tube.

Two alternate designs will be made:

(A). Curved Draft Tube.

The allowable difference in elevation between top of runner band and lowest tail water level may be determined by the equation,

$$E=33.9-K-\frac{V_2^2}{2g},$$

in which K is a constant and varies from 3 to 6 ft. according to the length of the draft tube and V_2=25.2 ft. per sec. as ~~det~~ determined in Section 7.

If K is equal to 4.03 ft, then,

$$E=33.9-4.03-9.87$$
$$=20 \text{ ft.}$$

Assume the velocity of discharge at the end of draft tube

$$V_n=5 \text{ ft. per sec.}$$

The top area of draft tube is,

$$A_1=\frac{Q}{V_2}=57.7 \text{ sq. ft.}$$

The end area of draft tube is,

$$A_n=\frac{Q}{V_n}=289 \text{ sq. ft.}$$

The top area of draft tube must be circular and has D_d=8.57 ft. as its diameter. But the end area may be one of several shapes and here is assumed to be two semicicles connected with tangents. Radius of the semicicle is assumed to be 5', then the length of tangent will be,

$$T=\frac{289-\pi 5^2}{10}=21.05 \text{ ft.}$$

As the outflow velocity is small, the highest point of the end area will be located 1/2 ft. below the lowest tail water level.

The two points on the centre line required in order to locate it can now be known. If the lowest tail water would be taken as one axis and the vertical centre line of the unit as another then one point is 20' abovewhile the other 5.5' below. The horizontal distance from centre of unit to the downstream wall of power house has been determined as 35'.

Generally the design of turbine is such that the bottom flange is about 4' ft. lower than the runner band. It is, therefore, determined to begin curving the draft tube at apoint 4.5' below the runner band.

Hence the difference in elevation between centres of top and bottom areas of the draft tube is,

$$20+5.5-4.5=21',$$

and these two points are 35' apart horizontally.

As a first trial, a parabolic centre line is supposed to be used. In order to simplify the expresion, the system of axis will have its origin at the apex of the parabola. The equation t then reads,

$$Y^2=ax,$$

and as the centre point of the end area must lie on this parabola, then,

$$a=\frac{21^2}{35}=12.6,$$

making $$Y^2=12.6x.$$

The centre line can now be plotted. When this has been done it appears that the angle between lower end of the parabola and the horizontal line is 17°30'as shown in Fig. . This seems to be too greatand necessaties the following alterations.

It seems to be better, if we use a circular centre line with a radius of 21' in connection with a tangent of 14'. By so doing, the total length of centre line will be ,

$$L=4.5+\frac{5\pi}{2}+14=51.25 \text{ ft.}$$

In determing the shape of the tube, areas should be calculated at points usually spaced 2 to 3 ft. apart. In this case 2' ~~spac~~

spacing will be used and the first space is then 1.425'.

As water flows at a constant diminishing velocity, the veloci at any point on its path is given by a parabolic equation,

$$V=b-\sqrt{kz},$$

in which z=thedistance at any point along the centre line from vertex of curve,

v=velocity corresponding to z,

$b=V_2+V_n,$

$$k=\frac{V_2^2-V_n^2}{L}.$$

The distance from vertex of the first and last points can be determined by the following equation,

$$z_1=\frac{V_n^2}{K}=\frac{L}{\left(\frac{V_2}{V_n}\right)^2-1},$$

$$z_n=\frac{V_2^2}{K}=L+z_1.$$

Then

$$K=\frac{25.2^2-25}{51.425}=11.87$$

$$\sqrt{K}=3.46, \qquad b=25.2+5=30.2.$$

$$V=30.2-3.46\sqrt{z}$$

$$z_1=\frac{51.425}{\left(\frac{25.2^2}{5}\right)-1}=2.105$$

$$z_n=51.425+2.105=53.53 \text{ ft.}$$

Since we know Q, the area at any point will be ,

$$A=\frac{Q}{V} \text{ sq. ft.}$$

All the computations are shown in Table 8.

In computing the volume of concrete in a power station, it is necessary to know the volume of draft tube. This is given by,

$$\text{Vol.}=\frac{2Qb}{K}(m-2.\log m)\text{ cu. ft. ,}$$

where $m=1-\frac{\sqrt{k}}{b}(\sqrt{z_n}-\sqrt{z_1})$.

Then, $m=1-\frac{3.46}{30.2}(\sqrt{53.53}-\sqrt{2.105})$,

$=0.337$

Therefore, $\text{Vol}=\frac{2\times1446.5\times30.2}{11.87}(0.337-2.3\log 0.337)$

$=7960$ cu. ft.

Table 8.

Pt. on b	Dis. from upperend of Tube	Z in ft.	V ft/sec.	A sq.ft.	Depth in ft.	Radius in ft.	Tangent in ft.	Total
1	0	2.105	25.20	57.70		4.285		
2	1.425	3.530	23.68	61.00		4.420		
3	3.425	5.530	22.04	68.80		4.720		
4	5.425	7.530	20.70	74.20		4.870		
5	7.425	9.530	19.50	74.20		4.870		
6	9.425	11.530	18.45	78.30	10	5.00	0.01	10.01
7	11.425	13.530	17.43	82.80	"	"	0.42	10.41
8	13.425	15.530	16.55	87.30	"	"	0.88	10.88
9	15.425	17.530	15.70	92.00	"	"	1.35	11.35
10	17.425	19.530	14.90	96.90	"	"	1.84	11.84
11	19.425	21.530	14.12	102.20	"	"	2.37	12.37
12	21.425	23.530	13.38	1p8. 00	"	"	2.95	12.95
13	23.425	25.525	12.66	114.20	"	"	3.57	13.57
14	25.425	27.530	12.00	120.20	"	"	4.17	14 17
15	27.425	29.530	11.38	127.10	"	"	4.86	14 86
16	29.425	31.530	10.70	135.00	"	"	5.65	15 65
17	31.425	33.530	10.17	142.20	"	"	6.37	16 37
18	33.425	35.530	9.55	151.20	"	"	7.25	17.25
20	35.425	37.530	8.95	161.50	"	"	8.30	18.30
21	37.425	39.530	8.40	172.00	"	"	9.35	19.35
22	39.425	41.530	7.95	182.00	"	"	10.35	20.35
23	41.425	43.530	7.35	196.50	"	"	11.75	21.75
23	43.425	45.530	7.00	206.20	"	"	12.77	22.77
24	45.425	47.530	6.30	229.00	"	"	15.05	25.05
25	47.425	49.530	5.80	249.20	"	"	17.09	27 09
26	49.425	51.530	5.32	272.00	"	"	19.35	29.35
27	51.425	53.530	5.00	289.00	"	"	21.05	31.05

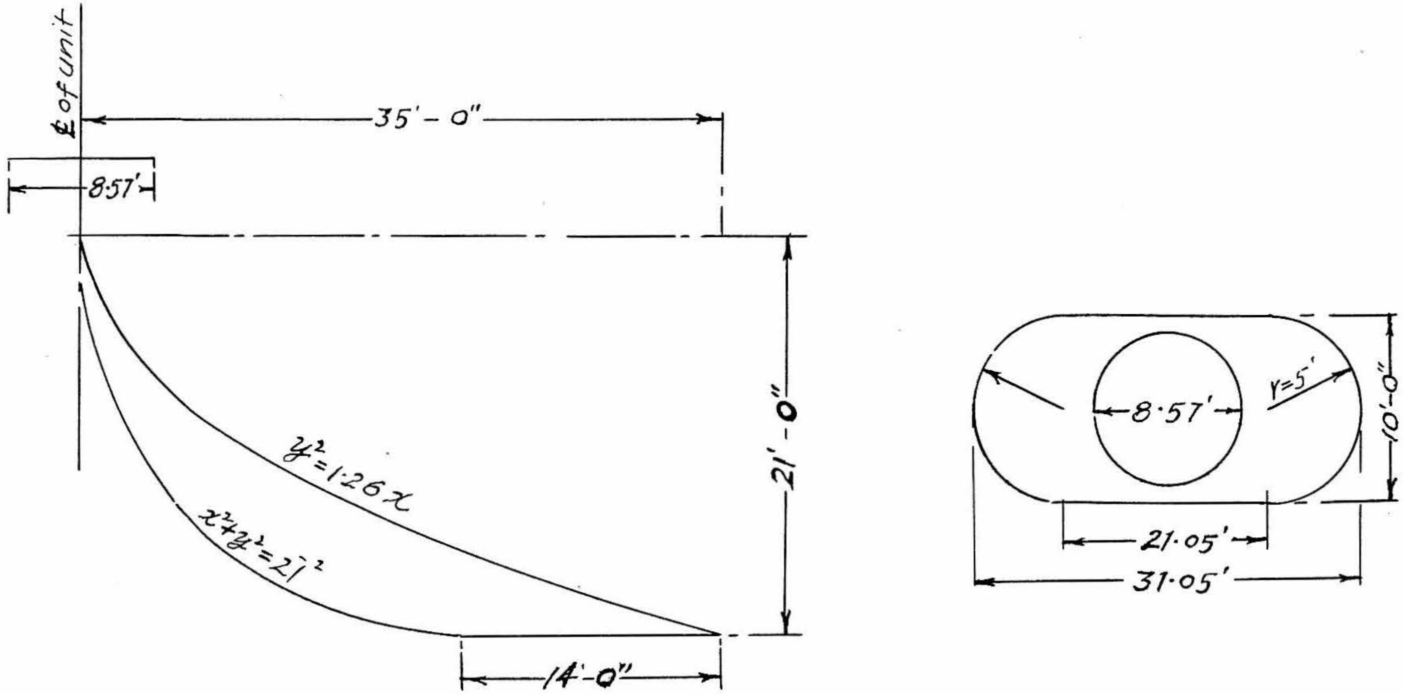


Fig. 10

(B). Draft Tube with Hydrocone.

Draft Tube with Hydrocone.

It is generally a common fact that any hydroelectric power house operation where curved draft tubes are ~~used~~ used has been frequently noticed boiling and disturbance of the water at the outlet from curved tubes. This disturbance indicates the loss of energy or, in other words, the inefficiency of curved tubes. Recent developments have a common tendency to adopt the ~~hydr~~odraft tube with a hydrocone. As the writer could not find out any typical design, or even a definite mathematical treatment, his own considerations will here be given.

Hydroelectric engineers usually accept that a good design of a ~~hyd~~ draft tube must agree with the follpwing four points:

(1). The axis of the tube must be a straight line coincident with that of the turbine.

(2). The crossectional areas of the tube should be enclosed within a diverging surface of revolution about the axis.

(3). Both the axial and radial component velocity of water flowing through the tube should be gradually decelerated.

(4). The outlet end should be of such a form that comforms to what would be the shape of a nonenclosed fluid.

It is evident that the extent ~~that~~ of tubulence depends upon the extent of the first and second factors disagreed. Among all the types of draft tubes the writer considers that Mr. Moody's spreading type meets the requirments to a higher degree.

As in part(A), we take $D_d=8.57'$
$V_2=25.2'/sec.$
$E=20'$

and the transverse centre line of the units is 35' from down stream wall of power house. The axis of the tube is a straight line and coincident with that of the turbine. In order to decelerate both the axial and radial component velocity of the flowing water through the tube, and to convert a high velocity head into pressure head , the enclosed surface of the tube will consists of two parts:

(a). A surface of revolution of a straight line. If we take the intersecting point of a horizontal line through top of runner band and the vertical axis of the unit as coordinates, the equation of the straight line will be

$$Y=\frac{20}{1.715}(x-4.285).$$

At a distance ~~y=20~~, y=-20, the velocity of the water is 12.8'/sec.

(b) Connected with the above surface is one produced by ~~reve~~ revolving one quarter of a circle, about y-axis with its centre at(18, -20).

The surface of the cone is obtained by revolving an arc of a circl about y-axis between limits y=-35 and -28. The centresof the circl is at (11.417, -23).

If we assume the outlet velcity as 4'/sec. ,the height of the opening will be

$$B=\frac{1446.5}{2\pi\times18\times4}=3.2'$$

The other dimensions are shown in Fig. 11.

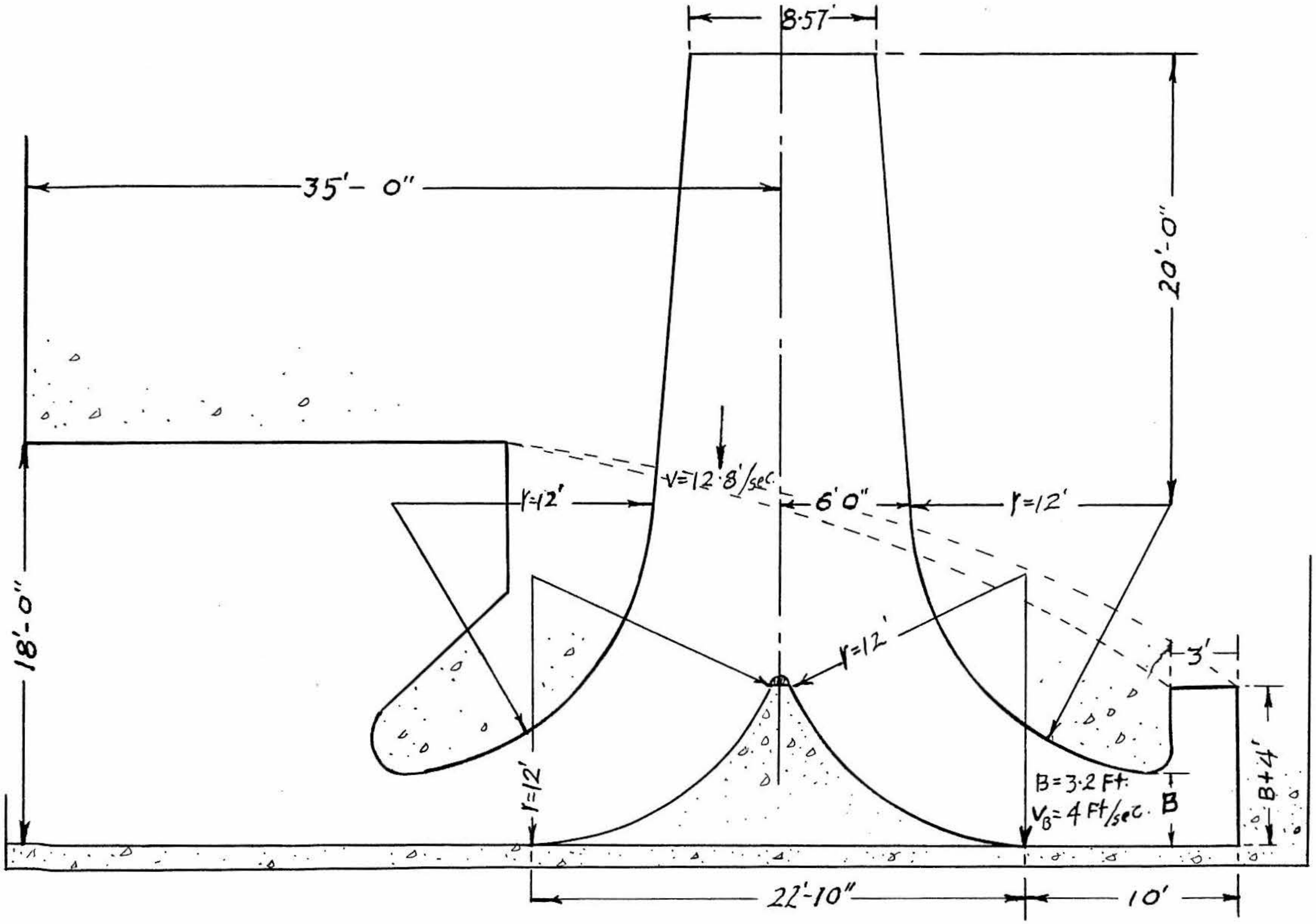

8·57'
35'- 0"
20'-0"
V=12·8'/sec.
r=12'
6'0"
r=12'
r=12'
18'-0"
r=12'
3'
B+4'
B=3·2 Ft.
V_B=4 Ft/sec.
B
22'-10"
10'

Fig. 11

11. The Power House.

The design of a power house differs greatly and is affected to a great extent by local conditions. The type and number of generating units is obviously a determing factor. The accesabilit of a power house sh uld also be considered. In general a high tension transmision is necessary so that provision must be made for housing the transforming and high tension switching apparatus

In this case the entire equipment belonging to one unit, consisting of a generator, high voltage and low voltage switching apparatus and transformers, occupies a 50' length of the station. The eighteen units will require 900' and make the building 1000' long, if 100' allowance should be made for surplus eqquipment. The main generator room will be 80' wide by ~~85~~ 70' high. It will be an entirely reinforced concrete structure.

In order to facilitate the installation of hydraulic and electri equipments, a travelling crane is necessary. The size of the crane depends, of course, upon the maximum load to be carried. But for the latter it is very hard to predict, because neither the weight of the runner northat of the generator is a simple function of its capacity or of its head. The higher the speed , the smaller t the wheel and consquently, its weight. In this case , the speed is medium and so a 200 ton and a 50 ton cranes will be supposed to meet the requirment. For such a size, either four or eight wheels must be used. The runway girders will be of steel resting on reinforced concrete colums. The vertical clearance is 18.5'. Side clearances are 2'.

12. Sluicing Outlets.

Three sluicing outlets are provide. The lengthe of each is about 250 ft. long with its entrance at elevation 1496.5 and outlet at elevation 1486.00. At the upstream face of the dam the outlet is 12 ft. high by 10 ft. wide and with a bottom slope 4:17.and top slope 3:21. It tapers at a distance of 21' 6" to a section of 5' high by 5j wide. The gate seat is located right in there. The 5'×5' section continues 28.5' and changes to a circular sec section of 5' diameter. This section is continued to the end of the conduit.

The unlined portions of the conduit are built of special mixture of concrete,1:1.85:4.5, the cement being composed of 75 percent sand cement and 25 percent straight portland cement.

The gates are made of heavy iron castings. They operated by hydraulic cylinders .The design and general arrangement areabout the sam as those in Elephant Butte reservoir.

13. Regulating Outlets.

It is proposed to provide with a diversion tunnel during the costruction of the dam, around the south end of dam through the canyon wall. This tunnel will finally be made as a sluicing outlet.

From Table 3, we notice that the maximum spill is 5845241 A.F. per month. It is about 3260 cu. ft. per sec.

It will be lined with concrete and provided with three needle valves of 60 inch in diameter, at the end of the tunnel.

14. Estimation of Cost.

This can only be rough and approximate and in no way to make it accurate under such conditions.

Items	466 000 H.P.
Dam and head works Power house	: $ 960 000
(18 wheel pits, curtain wall, steel roof trass, coverings, 24 colums for supporting runway girder, travelling crane and bridges, widows, etc.)	: $ 1000 000
Regulating tunnel and sluicing outlets	: $ 60 000
Hydraulic equipment (18 turbines with necessary auxiliaries, penstocks, riveted steel cases, draft tubes, regulating devices for speed, 18 Johson valves, 18 balanced valves with positive control, 3 sluicing gates with controlling devices, etc.)	: $ 3000 000
Electric equipment (generators, transformers, switching apparatus etc.)	: $ 500 000
Miscellaneous	: $ 8 000
	: $ 5018 000
Engineering, etc.	: $ 500 000
Total capital cost	: $ 5968 000
Capital cost per horse power	: $ 133

15. Conclusion

In conclusion the writer considers that this project is very feasible. But several things must be specially considered. First of all , in this development the secondary power is rather important, and the market conditions must be specially investigated. If the demand of primary power ix greater than that could be supplied , the feasibility of an auxiliary plant must be considered.

Even if it be the case that the Lees Fiery reservoir will be developed first, good care must be taken to the silt deposites. It is so important to the life of the plant.

Bouth the sluicing and regulating outlets must be carried far enough down the stream. Because as soon as the water gets out of the outlets , its velocity reduces and the silt begins to deposite. The capacity of the tailrace may be destroyed.