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Frictional Resistance in Artificial Waterways

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Frictional Resistance in Artificial Waterways

The use of empirical formulas plays an important part in the design of channels for carrying water. Practically all such formulas contain a factor which represents the resistance to the flow of water offered by the material forming the channel, and consequently there are almost as many different values for this factor as there are kinds of material. The engineer designing an irrigation channel must exercise good judgment in choosing the value of this coefficient or the resulting section will be too large or too small to carry the volume of water desired. If the channel is too large, the construction as well as the maintenance cost will be excessive, while if too small, there may be heavy crop losses because of insufficient water for irrigation.

In order to add to the available information on proper values of the coefficient of roughness in various types of artificial waterways, a series of field experiments was conducted in Colorado during the irrigation seasons of 1912 and 1913. The principal types of channels experimented upon were metallic, reinforced concrete and timber flumes, concrete-lined canals, earth canals, concrete and timber chutes, and inverted siphons of wood stave pipe, located on irrigation systems in the Cache la Poudre, Arkansas, Grande, Gunnison, Uncompahgre and San Luis Valleys.

PURPOSE OF INVESTIGATION. 1. To determine the coefficient of roughness in empirical formulas for several types of open channels; and also to determine whether such coefficient changes with variations of discharge of the channel.

2. To determine the loss of head in water flowing through siphons, and to compare Kutter's formula with the ordinary friction formula as adapted to pressure pipes.

3. To make current meter measurements throughout cross-sections of several channels to permit the vertical velocity curves to be plotted, and, from a standpoint of accuracy, to compare the integration and single point methods with the multiple point method.

EQUIPMENT AND METHODS USED. The same current meter was used in all the experiments, being a light type of the Price rod meter. This instrument was rated at Berkeley, California, just prior to beginning the experiments, and upon their completion was rated by the Bureau of Standards, Washington, D. C. The former ratings were used for the season of 1912 and the latter for 1913. A high grade wye level, target leveling rod reading to thousandths of a foot, standard steel tape, and graduated hard wood stick with tapered edge, were used.

In choosing the waterways upon which to conduct the experiments, particular care was exercised in selecting those typical of the class or group being investigated. Those possessing the slightest changes in the value of the hydraulic elements throughout the length of the section were best adapted for the purpose of experiment. Sufficient length was selected to accurately obtain the slope of the water surface. Other conditions being uniform, the length required varied inversely as the fall of the water surface. The distance required to develop the true fall was not so long, however, as to introduce a broken profile of the water surface gradient. Wherever possible, sections were selected on tangents, to avoid effect of curvature.

As repeated experiments were made upon some of the conduits as a check, or to determine any change in the value of the coefficient with the variation in discharge, permanent bench marks were established at convenient points along the section; from which future elevations of the water surface could be determined, or the wetted areas could again be cross-sectioned.

In all of these tests particular care was exercised in obtaining the correct discharge by current meter measurement, but at the same time the fact was not overlooked that measurement of the cross-sectional elements at the several stations was of equal importance in obtaining the coefficient of friction for any given waterway. In most cases the discharge was likely to fluctuate slightly while the test was being made, hence the importance of a simultaneous determination of the slope, cross-sectional elements and discharge, or where this was impossible, as but two men were in the field, slope and cross-sectional measurements were taken immediately after the current meter measurement near the head of the section, proceeding from the upper to the lower end. Hence, under such conditions it seemed inadvisable to spend time in making more than two determinations of the discharge, especially on a large channel where much time was consumed in doing so. The integration method was preferred for measurements in shallow streams. Independent meter measurements checked with an error seldom greater than 1.5 percent, and usually close to 0.5 percent.

check measurements being made by two methods. In some cases the discharge measurements were not made within the experimental section, but at some convenient support near the upper or lower end of the section. In every case, however, the flow was measured at a point near enough to make the seepage loss a negligible quantity.

In some places there was no foot bridge across the channel and wading had to be resorted to, which introduced a slight error in the discharge measurements, but it will be shown later that where the coefficient of roughness alone is desired, a small error in the determination of the discharge has an inappreciable effect upon the value of the coefficient. In all current meter measurements, cross-sections were chosen in which the filaments of flow apparently approached and receded from the section in parallel lines.

Values for the coefficient of roughness in the general formula of Ganguillet and Kutter, were secured from the field data by graphic methods for slopes up to .0621 feet per foot, and by computation for greater slopes.

METALLIC FLUMES

All of the metallic flumes upon which tests were made are of the semi-circular type, and may be divided into the following three classes, according to the characteristics of their interior surfaces:

GROUP 1. Flumes whose connections at the joints are countersunk to the plane of the sheet metal, and which present a smooth, unobstructed water face.

GROUP 2. Flumes whose joint connections protrude into the waterway beyond the plane of the sheet metal.

GROUP 3. Flumes constructed of corrugated metal and whose corrugations at right angles to the line of flow offer the only frictional resistance.

In most of these flumes tie beams extended across the waterway, upon which bench marks were established at the upper and lower ends of the sections, and at intermediate points where necessary. Because of the fluctuations of the water surface, the vertical distance from these benches to the water was measured directly with the rod, readings being taken at the highest and lowest levels. In this way it was possibly by averaging the two readings to minimize the error in obtaining the slope of the water surface.

The weight of the water caused distortion of the original semi-circular cross-sections, and consequently it was necessary to make two sets of measurements to obtain the wetted cross-sectional area and wetted perimeter. The mean maximum depth of water in the center of the channel was measured with a graduated hardwood stick, having a beveled edge. By use of plumb bob and tape the mean length of wetted chord was determined, and having

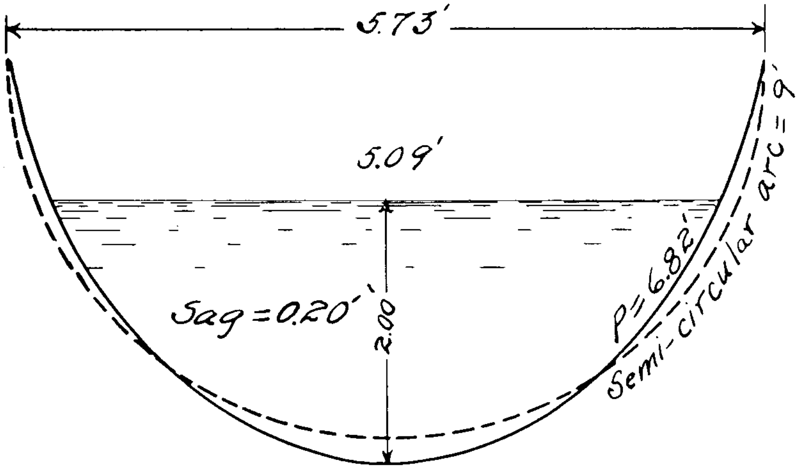

Fig. 1. Cross-section of Metallic Flume, Uncompahgre Project, Montrose.

measured the semi-circular arc of the flume, the wetted perimeter was determined by measuring the dry arcs directly with the tape.

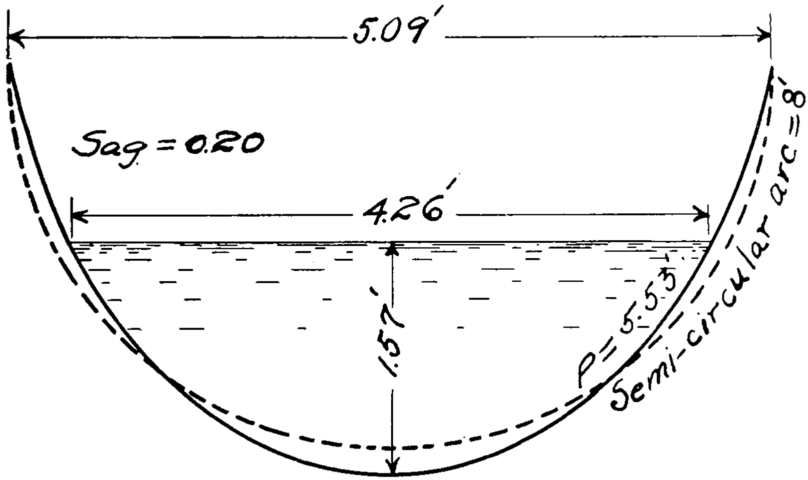

Fig. 2. Cross-section of Metallic Flume, Uncompahgre Project, Montrose.

These elements, viz., diameter, semi-circular arc, dry arc, wetted perimeter, wetted chord and maximum depth of water, when fitted together diagramatically, determined the form of curve assumed by the sheet metal under the weight of the water. Wetted areas were obtained from the diagram by use of the planimeter.

Figures 1 and 2 show cross-sections of flumes on the King Lateral Extension on the Uncompahgre Project, near Montrose. The dashed lines represent the semi-circular shape of the flume when empty, and the parabolic form of curve assumed by the sheet metal under the weight of the water is represented by the solid lines. The bottoms of these flumes were depressed 0.2 feet when about one-third full of water.

Figure 3 shows a cross-section of a flume on the Garland Canal, Blanca. A depression of 0.11 ft. on the bottom takes place when this flume is one-eighth full. Sufficient measurements could not be made to determine the distorting effects of other heads because of the difficulty of having the quantity of water varied. However, these serve to illustrate the fact that allowance should

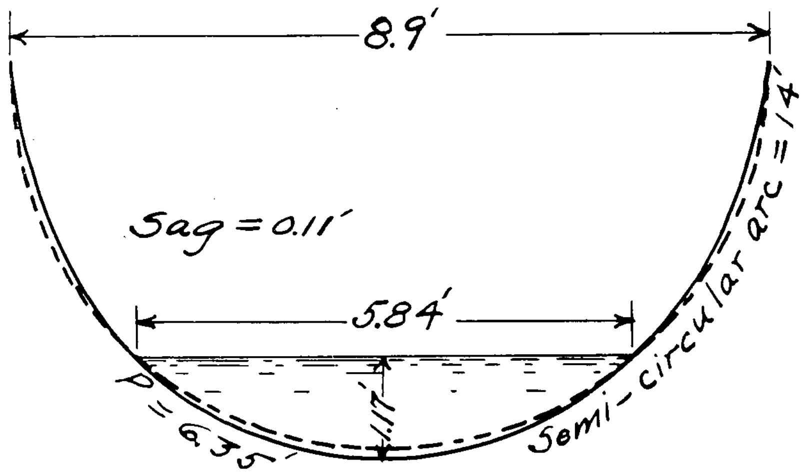

Fig. 3. Cross-section of Metallic Flume, Garland Canal, Blanca.

be made for this sag if the proper grade and elevation of the flume in relation to the channels of approach and recession be maintained when water is passing through. The change in the amount of cross-sectional area due to this distortion, is inappreciable and would not affect the carrying capacity of the flume.

The corrugated flumes experimented upon showed no change under weight of water from the semi-circular form.

GROUP NO. 1. SHORT FLUMES WITH WASTE-WAYS. On the Willcox canal, of the Grand Valley Irrigation District, near Grand Valley, are a number of rather short flumes which span the arroyos leading from the mesas and serve as wasteways in case of breaks in the banks of the canal. The wasteway consists of a 10x3 feet hole, cut lengthwise in the bottom of the flume near the middle of its length, and opening into a wooden box encasing the bottom of the outside of the flume. Wasteway gates are built into one side of this box. The flumes experimented upon are from 70 to 90 feet in length and with semi-circular arcs of 168 inches. Duplicate experiments were made upon each flume as a check upon the value of the coefficient. Table 1 shows the results of these experiments.

TABLE 1—HYDRAULIC ELEMENTS IN FLUMES WITH WASTEWAYS.

Length of Section Tested	Discharge sec-ft.	Area of Wetted Section sq. ft.	Mean Velocity ft. per sec.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
79.6	15.72	5.83	2.70	7.02	0.83	0.00216	63.6	0.0211
79.6	27.68	9.50	2.91	8.32	1.14	0.00168	66.4	0.0219
69.6	14.05	9.12	1.54	8.23	1.11	0.00043	70.6	0.0202
69.6	22.22	12.93	1.72	9.53	1.36	0.00043	71.2	0.0210

The results of these experiments indicate that the effect of the wasteway in the flume of short length, is to reduce its carrying capacity by increasing the coefficient of friction. For flumes of this class, less than 100 feet long and having wasteways similar to the above type, a mean value for (n) of 0.021 is indicated.

FLUMES ON TANGENTS WITH NO IRREGULARITIES. About 3,000 feet of flume had been constructed in the winter of 1912-13 upon the Garland canal of the Trinchera Irrigation District, near Blanca. The length of semi-circular arc was 168 inches. Because of the excellency of the construction, uniform grade and perfect alignment it was especially well adapted to purposes of experiment. (Plate I.) .

Tests were made upon the tangents between curves in April, 1913.

Experiments were also made upon a flume of the Minnesota canal near Paonia, in October, 1913. The grade of this flume was not uniform throughout its entire length, so that only two sections were tested, but these had uniform grade and sufficient fall to allow of an accurate determination of the slope. No transverse bracing whatever existed in this flume, and during the three seasons it had been in operation sag had taken place in cross-section and caused a displacement of the uprights and drawing together at the top. For this reason the mean wetted area was obtained by taking cross-sectional depths, which were plotted, and the wetted perimeter was measured with a pair of dividers.

Table 2 gives the results of these tests:

TABLE 2

HYDRAULIC ELEMENTS IN FLUMES HAVING NO IRREGULARITIES.

Name of Canal	Length of Section Tested ft.	Discharge sec-ft.	Area of Wetted Section sq-ft.	Mean Velocity sec. ft.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
Garland	450	19.59	3.62	5.41	5.95	0.608	0.0028	131.1	.0109
Garland	650	19.59	3.66	5.34	6.00	0.610	0.0023	142.3	.0101
Garland	325	19.59	4.45	4.40	6.44	0.691	0.0022	112.5	.0126
Minnesota	70	7.04	1.17	6.01	3.12	0.374	0.0069	118.1	.0111
Minnesota	200	7.04	1.17	6.01	3.13	0.374	0.0052	136.2	.0099

The value of (n) indicated in this class of flumes is 0.011.

GROUP NO. 2. FLUMES ON TANGENTS WITH NO IRREGULARITIES. All of the flumes of this class upon which experiments were conducted, are located upon the King Lateral Extension of the Uncompahgre Project, United States Reclamation Service, near Montrose. The flumes are of excellent construction, there being no irregularities in alignment or grade. The grade of each being quite heavy, no difficulty was experienced in getting the true slope of the water surface. Several cross-sectional areas and water surface elevations were taken throughout the length of each. The experimental results are given in Table 3, upon flumes with semi-circular arcs of 108 inches, 120 inches and 96 inches respectively.

TABLE 3—HYDRAULIC ELEMENTS IN FLUMES OF GROUP NO. 2.

Length Section Tested feet	Discharge sec-ft.	Area of Wetted Section sq. ft.	Mean Velocity ft. per sec.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
300.5	39.19	7.32	5.35	6.82	1.072	0.00386	83.1	.0179
189	33.30	5.77	5.77	6.30	0.916	0.00537	82.3	.0177
635	23.72	4.65	5.10	5.53	0.841	0.00411	86.6	.0166

A mean value of 0.0174 is indicated for this class of flumes.

GROUP NO. 3. CORRUGATED FLUMES. Experiments were conducted upon the corrugated flumes on the Stewart, and Fire Mountain canals near Paonia, in October, 1913.

A metal flume having a semi-circular arc length of 132 inches, and a length of 1,745 feet, carried the water of the Stewart canal around a steep hillside and contained numerous curves and tangents, a portion of which is shown in Plate II. Because of the rather poor alignment of tangents, nothing definite could be done

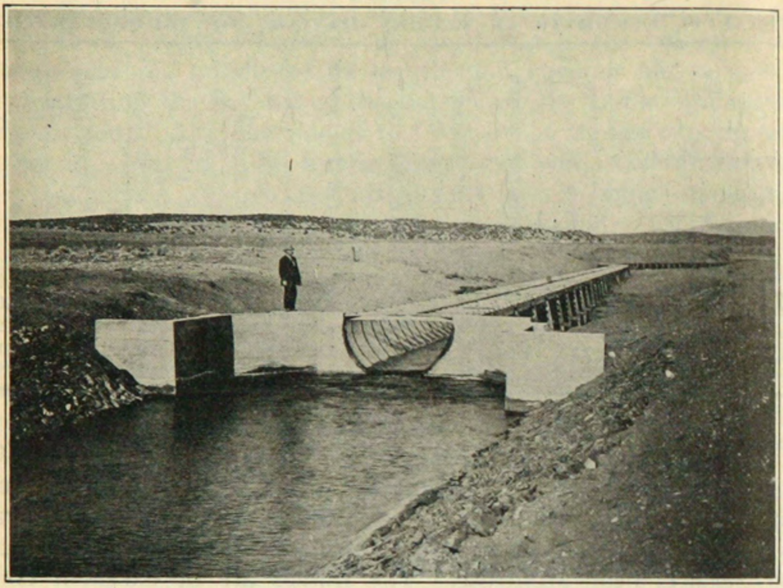

Plate I. Metallic Flume, Garland Canal, Blanca.

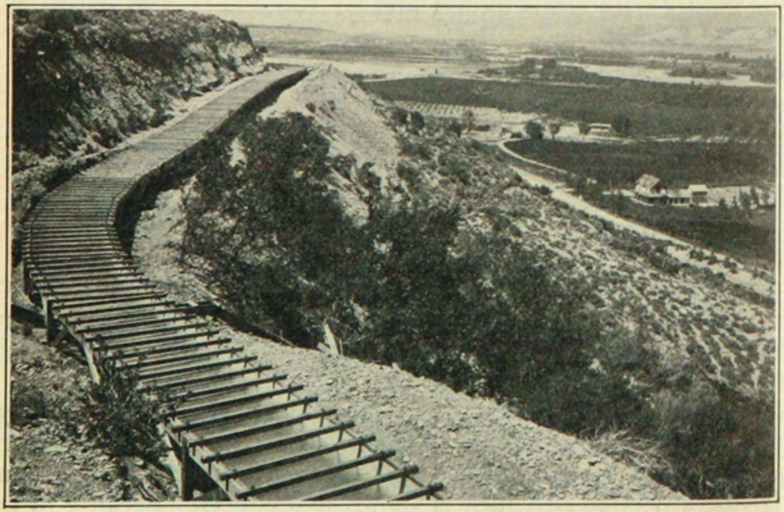

Plate II. Corrugated Flume, Stewart Canal, Paonia.

in the way of finding the true effect of curvature in retarding the flow. It was with the idea of obtaining the value of the coefficient for this particular flume that the tests were made. Elevations of the water surface and the cross-sectional elements, were determined at intervals throughout the entire length. Figure 4 shows the alignment of this channel. Individual sections upon which determinations of (n) were made, are indicated by the alternate solid and dashed lines. The values of the coefficient for each section are written opposite. Two independent experiments were made upon this flume on different days. The results are as shown on the diagram, Fig. 4, and in Table 4, which gives mean values for the entire length of the flume.

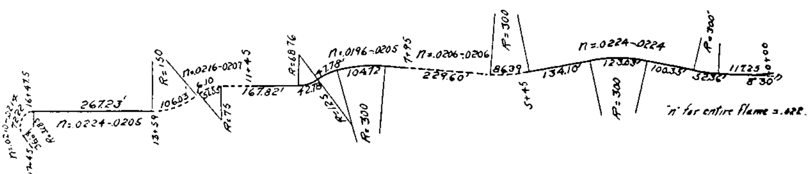

Fig. 4. Corrugated Metallic Flume, Stewart Canal, Paonia.

Figure 5 shows the alignment and results of tests on the flume of the Fire Mountain canal over Hubbard Creek. The mean

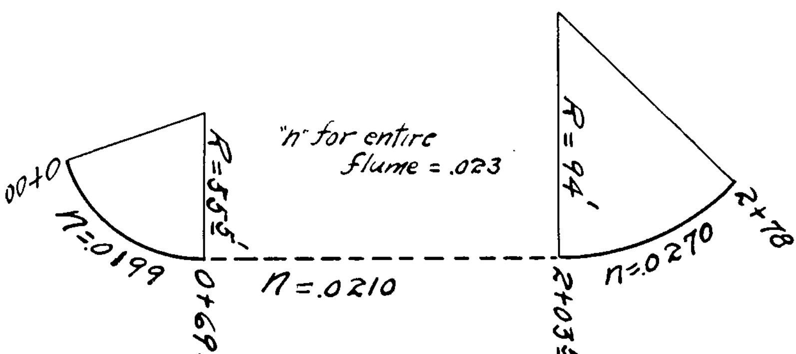

Fig. 5. Corrugated Metallic Flume, Fire Mountain Canal, Paonia.

hydraulic elements for this flume, taken as a whole, are shown in Table 4. Its semi-circular arc is 144 inches.

Because of the short length and slight fall so much weight cannot be attached to the experiment on the Fire Mountain flume as to the ones on the Stewart flume.

In determining the carrying capacity of corrugated flumes these data indicate a safe value of (n) to be 0.0225.

TABLE 4—HYDRAULIC ELEMENTS OF CORRUGATED FLUMES.

Name of Canal	Discharge sec-ft.	Area of Wetted Section sq-ft.	Mean Velocity ft. per sec.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
Stewart	14.7	7.67	1.918	7.35	1.043	0.000892	62.8	0.0222
Stewart	14.5	7.65	1.897	7.37	1.038	0.000879	62.8	0.0222
Fire Mountain	16.88	8.84	1.910	7.96	1.110	0.000871	61.4	0.0230

TIMBER FLUMES

BENCH FLUMES ON ORCHARD MESA POWER CANAL. About a mile above Plateau Creek on Grande River, is located the headworks of the Orchard Mesa Power Canal. Owing to the close proximity of the river to the bluffs it was necessary to use an aggregate length of several miles of rectangular wooden flumes, interspersed with earth canal, to convey the water around the hillside to the power house below the Orchard Mesa at Palisade (Plate III). This flume was built in the winter of 1909-1910.

The flume is irregular in alignment, being made up in sections varying in length from less than 100 feet to over 600 feet, connected by sharp angles. The waterway averages twelve feet in width, with a depth of six feet, and is laid on a grade of about 4.5 feet per mile. The sides and bottom are of planed lumber with tongue and groove joints. In nearly every section has been placed two-inch by four-inch posts vertically along the sides at intervals of about 100 feet, the purpose of these being to check the flow at low heads that the sides of the flume might be kept from drying out and leaking badly when larger heads were turned in.

A line of levels was run along each division of the flume, and benches established at points on the sides at the upper and lower ends of the sections to be experimented upon. The elevation at each side along the bottom was also obtained. It was found that the flume had settled in many places as much as 0.3 feet below grade. During a time when the water was out of the canal accurate measurements were made of the widths at the top and bottom, at the upper and lower ends of each section, and although the flume was apparently designed for a width of 12 feet, this dimension did not prevail exactly throughout.

It was necessary in choosing the proper place for the ends of the section to consider the effect of the angle upon the flow of the water entering and leaving the section. Usually the condition was that of eddies and whirls and the banking of the water on the outside. Hence usually as much as 30 feet above or below the turn was allowed for the water to regain its flow in an undisturbed condition.

Several experiments were made at heads varying from 60 to 230 second feet. The unevenness of the water surface necessitated the taking of two readings from the bench marks to the water at its lowest and highest stages, and the mean of these was used in the computations for the slope. Table 5 shows the results of 75 experiments on 22 sections of this canal.

Of the 22 sections experimented upon, 17 show a higher value of (n) for the highest than for the lowest discharges. Of these 17, 14 have a greater slope of water surface for the highest than for the lowest heads. Only eleven, however, indicate a gradual increase of the value of (n) with that of the hydraulic radius. At the bottom of Table 5 is given the mean values of the elements obtained for each discharge. As will be noted there is no apparent change in the value of (n) with that of the hydraulic radius. However, to make the change apparent, of the ten mean values obtained, arranged in order of corresponding discharge from least to greatest, an average of 1-4, 4-7 and 7-10 indicate (n) to be .0117, .0121, and .0124, respectively. This set of experiments indicates the possibility of a variation in the value of (n) with the change in hydraulic radius for a rectangular flume of this type at least, and that this value increases with the discharge. These data are not positive proof of such variation, but they are presented here merely as a suggestion. Even if this change in the value of (n) does occur, it is probably not of great practical concern.

An average value for the coefficient of roughness of this flume is 0.0122.

There follows a brief description of other timber flumes investigated, and Table 6 gives the hydraulic elements as determined. The results are of value merely in illustrating the condition of some flumes after a period of use. Many of these may not originally have been built upon the proper grade to have the same carrying capacity as the channel leading to or away from them, or if formerly well designed and constructed, subsequent conditions may have caused sagging or irregular alignment.

REDLANDS MESA FLUME, GRAND JUNCTION. The flume is in excellent condition as regards the character of the material of which it is constructed, the lining being tongue and

TABLE 5

HYDRAULIC ELEMENTS FOR THE ORCHARD MESA POWER FLUME.

Length of Section Tested	Discharge sec-ft.	Area of Wetted Section sq-ft.	Velocity ft. per sec.	Wetted Perimeter	Hydraulic Radius	Slope ft. per foot	Coefficient (c)	Coefficient (n)
132	136.1	29.41	4.63	16.88	1.742	.000687	133.9	.0123
132	169.5	33.33	5.09	17.53	1.901	.000695	139.9	.0120
132	209.3	37.90	5.52	18.32	2.068	.000680	146.2	.0117
132	219.4	40.05	5.47	18.70	2.140	.000838	129.3	.0131
200	136.1	27.59	4.94	16.59	1.663	.001003	120.7	.0127
200	169.5	29.72	5.70	16.91	1.757	.000883	144.7	.0115
200	209.3	35.55	5.89	18.41	1.930	.001242	120.1	.0137
200	219.4	37.53	5.86	18.24	2.058	.001342	111.4	.0151
300	136.1	27.47	4.96	16.53	1.661	.000832	133.1	.0124
300	169.5	31.90	5.32	17.30	1.843	.001046	121.0	.0138
300	209.3	36.08	5.80	18.00	2.004	.000693	155.7	.0110
300	219.4	38.24	5.74	18.36	2.083	.000626	156.6	.0109
200	136.1	25.70	5.30	16.18	1.588	.000980	134.4	.0121
200	209.3	34.10	6.14	17.65	1.932	.001016	138.7	.0121
200	219.4	35.59	6.17	17.89	1.988	.001266	122.8	.0139
360	136.1	26.92	5.06	16.49	1.631	.000747	145.0	.0114
360	169.5	30.57	5.55	17.09	1.789	.000761	150.3	.0111
360	209.3	34.47	6.07	17.84	1.932	.000720	162.9	.0104
360	219.4	36.09	6.08	18.01	2.004	.000636	170.3	.0100
196	136.1	27.09	5.03	16.50	1.642	.000617	158.0	.0105
196	169.5	31.28	5.42	17.20	1.820	.000633	159.5	.0105
196	209.3	35.30	5.93	17.87	1.974	.000954	136.7	.0123
196	219.4	36.79	5.96	18.12	2.030	.000903	139.3	.0122
144	136.1	28.73	4.74	16.79	1.711	.000554	153.9	.0108
144	169.5	32.38	5.24	17.40	1.860	.000665	148.8	.0113
144	209.3	35.44	5.91	17.90	1.981	.001157	123.3	.0136
144	219.4	36.46	6.02	18.07	2.019	.000949	137.6	.0122
208	136.1	25.32	5.38	16.22	1.560	.000853	147.4	.0111
208	169.5	28.63	5.92	16.77	1.708	.000945	147.7	.0112
208	209.3	31.56	6.64	17.28	1.825	.000974	157.3	.0108
208	219.4	33.12	6.62	17.53	1.891	.001151	142.1	.0119
220	136.1	24.91	5.46	16.54	1.506	.000728	163.7	.0100
220	169.5	29.12	5.82	16.87	1.725	.000842	152.8	.0109
220	209.3	33.40	6.27	17.58	1.899	.000847	156.7	.0109
220	219.4	34.78	6.31	17.82	1.952	.000801	159.5	.0107
244	136.1	27.30	4.99	16.64	1.640	.001025	120.2	.0137
244	169.5	29.97	5.66	17.02	1.758	.001041	132.1	.0127
244	209.3	34.92	6.00	17.84	1.958	.000984	136.7	.0123
216	169.5	31.08	5.45	16.59	1.875	.001102	119.8	.0139
216	209.3	36.84	5.68	18.16	2.030	.001052	123.1	.0138
216	219.4	37.02	5.93	18.19	2.019	.001191	129.7	.0140

TABLE 5—Continued.

Length of Section Tested	Discharge sec-ft.	Area of Section sq-ft.	Velocity feet per sec.	Wetted Perimeter	Hydraulic Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
116	136.1	26.85	5.06	16.56	1.622	.001101	119.6	.0137
116	169.5	30.42	5.57	17.09	1.780	.001325	114.8	.0140
116	209.3	36.46	5.74	18.09	2.015	.001222	115.6	.0145
116	219.4	35.98	6.10	18.02	1.995	.001394	115.8	.0145
172	169.5	29.70	5.71	16.28	1.824	.000877	142.7	.0118
172	209.3	35.49	5.90	17.93	1.980	.001005	132.2	.0129
172	219.4	34.58	6.34	17.78	1.943	.001180	132.3	.0128
208	136.1	24.72	5.52	16.22	1.521	.001466	116.9	.0139
208	169.5	29.58	5.74	16.95	1.742	.001192	126.1	.0131
208	219.4	33.64	6.52	17.64	1.908	.001754	112.8	.0149
324	63.3	18.84	3.36	15.14	1.244	.000500	134.5	.0117
324	139.5	30.36	4.60	17.06	1.780	.000673	132.7	.0125
324	214.9	39.06	5.51	18.51	2.110	.000846	130.7	.0130
324	228.8	40.50	5.65	18.75	2.160	.000794	136.3	.0126
136	63.3	15.30	4.14	14.55	1.051	.001051	124.6	.0121
136	139.5	26.58	5.25	16.43	1.617	.000934	135.0	.0122
136	228.8	38.34	5.97	18.39	2.085	.001007	130.3	.0130
147	139.5	26.28	5.31	16.38	1.605	.000557	177.7	.0093
147	214.9	36.12	5.96	18.02	2.004	.000543	180.6	.0095
147	228.8	38.16	6.00	18.36	2.079	.000645	163.7	.0106
581	63.3	15.73	4.02	14.63	1.074	.000725	144.1	.0108
581	139.5	26.28	5.31	16.36	1.606	.000675	161.5	.0102
581	214.9	38.04	5.65	18.34	2.075	.000959	126.8	.0133
581	228.8	39.52	5.79	18.59	2.126	.001045	122.8	.0139
308	63.3	17.23	3.67	15.86	1.087	.000564	141.6	.0109
308	139.5	28.20	4.95	16.70	1.688	.000726	141.5	.0118
308	214.9	38.94	5.52	18.49	2.105	.000768	137.3	.0122
308	228.8	40.74	5.64	18.79	2.170	.000801	135.1	.0127
404	177.8	33.54	5.30	17.57	1.909	.000577	159.8	.0106
404	213.1	41.21	5.17	18.86	2.188	.000421	170.4	.0101
305	177.8	33.88	5.26	17.64	1.919	.000879	128.1	.0130
305	213.1	42.86	4.98	19.13	2.239	.000696	126.2	.0137
420	177.8	36.12	4.92	18.03	2.003	.000555	147.6	.0115
420	213.1	45.44	4.68	19.58	2.320	.000548	131.4	.0131
1	63.3	16.77	3.80	15.05	1.114	.000710	136.2	.0114
2	136.1	26.83	5.09	16.51	1.624	.000883	137.2	.0120
3	139.5	27.54	5.08	16.59	1.659	.000713	149.7	.0112
4	169.5	30.59	5.56	17.00	1.798	.000924	138.5	.0121
5	177.8	34.51	5.16	17.75	1.944	.000670	145.2	.0117
6	209.3	35.21	5.96	17.91	1.964	.000965	138.9	.0123
7	213.1	43.17	4.94	19.19	2.249	.000555	142.7	.0123
8	214.9	38.04	5.66	18.34	2.074	.000779	143.8	.0120
9	219.4	36.14	6.08	18.03	2.002	.001079	134.7	.0126
10	228.8	39.45	5.81	18.57	2.124	.000858	137.6	.0126

groove, mill-planed lumber. The grade, however, was insufficient to prevent considerable deposition of silt on the bottom, the sediment being as much as 0.5 feet deep in places. The cross-section is 5.62 feet by 2.8 feet, and the length tested is 296 feet.

FLUME NO. 42, WILLCOX CANAL, GRAND VALLEY. This flume is approximately twenty years old, and in 1912 had been repaired and lined with a patent roofing material, shown in Plate IV. This consists of flexible cement, reinforecd in the center with imported burlap, backed with highly compressed saturated felt and surfaced with flake mica. The lining is well joined and presents a very smooth surface. The alignment and grade of the flume are very poor and sediment is deposited in several places due to sagging. The width of waterway is 3.5 feet.

FLUME OVER CHICOSA CREEK, OXFORD CANAL, FOWLER. This structure is twenty-three years old and is still in excellent condition of grade and alignment. The channel is 4.35 feet by 9.7 feet. The sides are formed of 2x12-inch boards, with one-fourth by three inch crack battens on the inside of the flume, as shown in Plate V. The flooring is composed of 2x12-inch boards laid at right angles to the line of the flume, but these are so regular as to apparently form a very smooth surface. However, the data indicates a high frictional resistance. With the flooring of this type there is a tendency for silt to be deposited in the cracks, thus preventing leakage, an advantage in this respect over that of flooring placed longitudinally.

EAST CANAL CHUTE, UNCOMPAHGRE PROJECT. This flume is of practically the same age and cross-section, and has the same type of lining as that of the High Mesa Chute. The fall is eleven feet per 100 feet. It also has a low value of (n).

TABLE 6—HYDRAULIC ELEMENTS IN TIMBER FLUMES AND CHUTES.

Name of Flume	Length of Section Tested ft.	Discharge sec-ft.	Area of Wetted Section sq-ft.	Velocity ft. per sec.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
No 42	360	6.33	3.74	1.69	5.46	0.685	.000610	83.4	.0162
Chicosa Creek	1712	16.01	10.41	1.54	11.23	0.928	.000274	96.4	.0150
High Mesa Chute	175.6	8.45	0.48	17.61	2.48	0.194	.1331	109.4	.0104
East Canal Chute	169.5	4.17	0.36	11.60	2.29	0.157	.1133	86.9	.0118

See also Table 5, Orchard Mesa Power Canal Flume, pages 14 and 15.

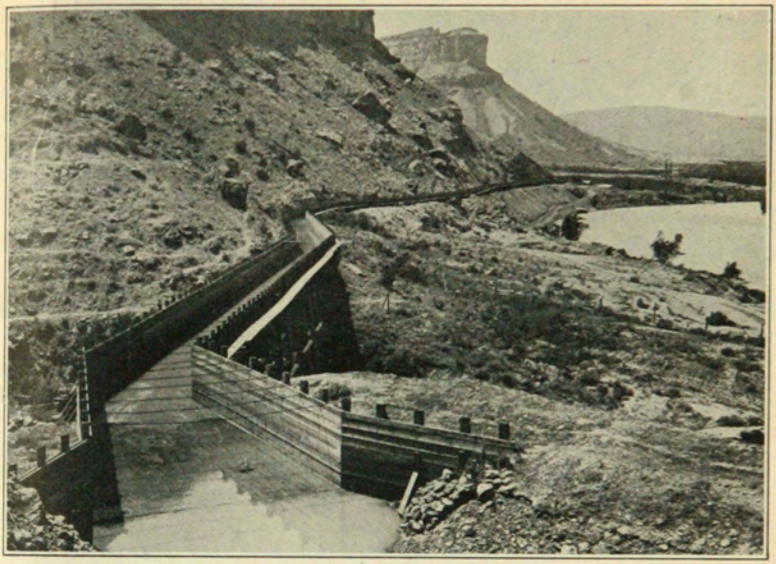

Plate III. Timber Flume, Orchard Mesa Power Canal, Palisade.

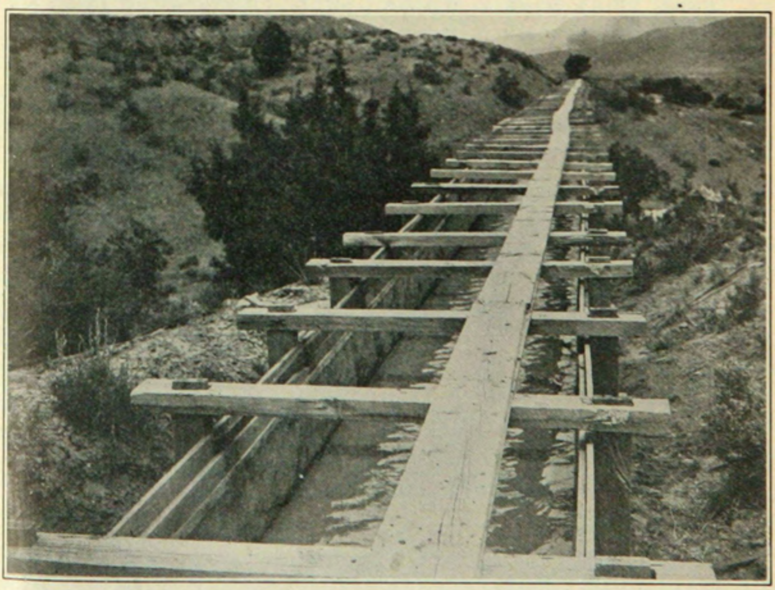

Plate IV. Timber Flume, Lined With Patent Roofing, Willcox Canal, Grand Valley.

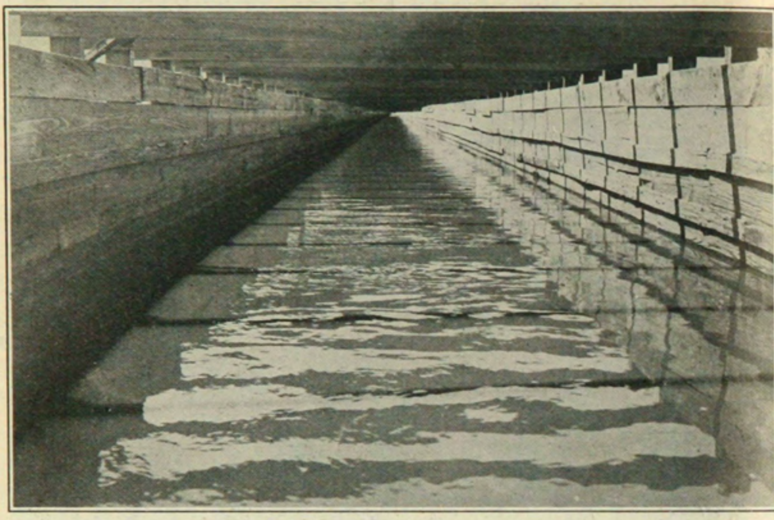

Plate V. Timber Flume Across Chicosa Creek, Fowler.

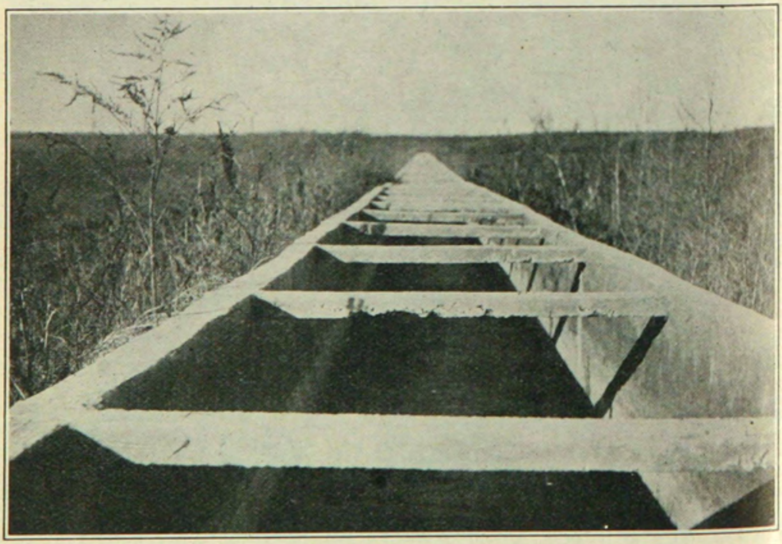

Plate VI. Concrete Flume, Long Pond, Fort Collins.

HIGH MESA CHUTE, UNCOMPAHGRÉ PROJECT. This chute was built in the spring of 1913 and the experiment was made upon it in June. The waterway is composed of 2x12-inch boards on each side, joined by 2x12-inch boards at the floor, giving ten inches by two feet as the dimensions of the cross-section. The lumber is mill-planed. It has a fall of about thirteen feet per 100 feet. As will be noted the coefficient (n) is rather low for this type of lining.

CHUTES

Since from a construction standpoint practically all chutes may be classified as flumes or lined channels, the data has been placed under those headings. Experiments were made upon two timber and three concrete-lined chutes, included in Tables 6 and 7.

CONCRETE-LINED CHANNELS

LONG POND REINFORCED CONCRETE CHUTE, FORT COLLINS. Long Pond Reservoir is situated on a ridge to the north of the Cache la Poudre River, near Fort Collins, and is used for the storage of water to be supplied to the Larimer & Weld canal during the latter part of the irrigation season. The slope from the reservoir to the Larimer & Weld canal is such as to prohibit the use of an earth section to convey the water, hence a reinforced concrete chute, resting on piers from one foot to five feet high, was provided. (Plate VI.) This was constructed in the fall of 1910. The chute is rectangular in cross-section, being 4.4 feet wide by 2.7 feet deep, and having a grade of three percent. The velocity of the water varies from 12 to 20 feet per second, depending upon the discharge. The channel has a smooth cement lining and is in very good condition. The structure is over 1,000 feet in length, but owing to curves and change of grade at inlet and outlet, only a 600-foot section was experimented upon. During a period when the channel was not in use, levels were run over this section and benches established upon the sides 100 feet apart, cross-sectional elevations of the bottom being taken opposite these points.

During August and September, 1912, five experiments were made upon the flow of water in this flume, at different heads, for the purpose of determining any variation of (n) with the change in hydraulic radius. The current meter measurements were made near the inlet in a section of light grade where the water had not yet attained a high velocity. Readings were taken to the water surface from all benches, the mean of the highest and lowest pulsations being used in computing the slopes and other hydraulic elements.

Table 7 gives the results of these experiments. To make any variation in the value of (n) apparent, the experiments are arranged according to discharge from least to greatest. An average of 1-3, 2-4, 3-5 indicate mean values of (n) to be .0122, .0117 and .0121, respectively, from which a comparison can be made. These limited data do not throw any light upon the variation in the value of (n) with the discharge. The mean coefficient of roughness for this chute is .012.

REINFORCED CONCRETE FLUME OVER DRY CREEK, HANDY CANAL, LOVELAND. This flume has a maximum elevation above the creek bed of about 30 feet, and is supported upon concrete columns. The total length is approximately 650 feet, and it is built in two sections of different grades, the upper being about 130 feet long with a fall of 5 feet, and the lower 520 feet long with a fall of 8 feet. The cross-section is rectangular, being 7.9 feet wide by 3 feet deep. The channel is lined with cement mortar, trowel finished, and is in very good condition, giving a velocity in the lower section of about 13 feet per second. Because of the short length of the upper section it was discarded and the lower one chosen for the purpose of experiment, 514.5 feet of which was tested. This flume was built in the spring of 1906.

SOUTH CANAL, U. S. RECLAMATION SERVICE, UNCOMPAHGRE PROJECT. Although much of this canal is concrete-lined, great difficulty was experienced in selecting suitable sections for the purpose of experiment, owing to the short distance between curves, and the roughness of the water surface, which rendered an accurate determination of the slope quite difficult. Two short sections of heavy grade, and one long one of light grade, were finally chosen as being typical of this type of canal, and as presenting the most favorable conditions for determining the hydraulic elements. The canal is designed for a carrying capacity of 1,300 second feet, but the 60 to 100 sec. ft. conveyed at the time of the tests is believed to be sufficient to permit of a reasonably accurate determination of the frictional resistance for this type of concrete lining. The surface of the channel has been left in the condition given it by the forms, the boards on the face of which having been placed longitudinally in the direction of the channel. The concrete lining was placed during 1906 and 1907.

Two chutes were experimented upon. The one at Mile Post No. 2 has a bottom width of 10.1 feet, side slopes 1 to 4, and a grade of seven percent. At Mile Post No. 9 the chute has a bottom width of 13 feet, side slopes 1 to 2 and a grade of seven percent. The length of section tested on the former was 201 feet and on the latter 142 feet. There was also a section 730 feet in length between stations 489+45 and 496+75, which was tested. This has a bottom width of 13 feet, side slopes 1 to 2, and a grade of .15 percent. In Table 7 the sections are designated as (a), (b), and (c) respectively.

Fortunately, near the upper and lower ends of the chutes, beams had been placed over the channel as a means of reinforcing the sides against earth pressure, and from these, cross-sectional and slope measurements were taken. (Plate VII.) For the other section the rodman used a fireman's ladder placed against the side slope and held from above, as a means of getting the elevation of the bottom of the channel and the water surface. On chutes of heavy grade, depths in a section were measured perpendicularly to the bottom, rather than in a vertical direction.

The results of these tests show an average value of (n) to be .0161 for this kind of lining, which, as would naturally be expected, is somewhat higher than that of the cement, or mortar-faced lining.

TABLE 7

HYDRAULIC ELEMENTS OF CONCRETE-LINED CHANNELS AND CHUTES.

Name of Channel	Length of Section Tested ft.	Discharge sec-ft.	Area of Wetted Section sq-ft.	Velocity sec-ft.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
Long Pond Chute	600	35.79	2.78	12.87	5.68	.489	.02978	106.6	.0125
Long Pond Chute	600	78.32	4.29	18.26	6.36	.674	.02968	128.9	.0113
Long Pond Chute	600	100.38	5.61	17.89	6.96	.806	.02943	116.1	.0128
Long Pond Chute	600	104.47	5.17	20.21	6.76	.765	.02990	133.5	.0111
Long Pond Chute	600	122.94	6.24	19.70	7.24	.862	.02971	122.9	.0123
Dry Creek Flume	514.5	154.00	9.76	15.78	10.35	.942	.01459	134.5	.0115
South Canal (a)............	201	89.80	5.78	15.52	11.28	.514	.07180	80.8	.0158
South Canal (b)............	142	59.68	5.28	11.32	13.92	.379	.07230	68.4	.0171
South Canal (c)............	730	111.30	23.66	4.71	16.82	1.406	.00151	102.1	.0155

EARTH CHANNELS

These investigations were made chiefly upon some of the larger canals of the Arkansas, San Luis and Grand Valleys. The main object in view was to ascertain the amount of frictional re-

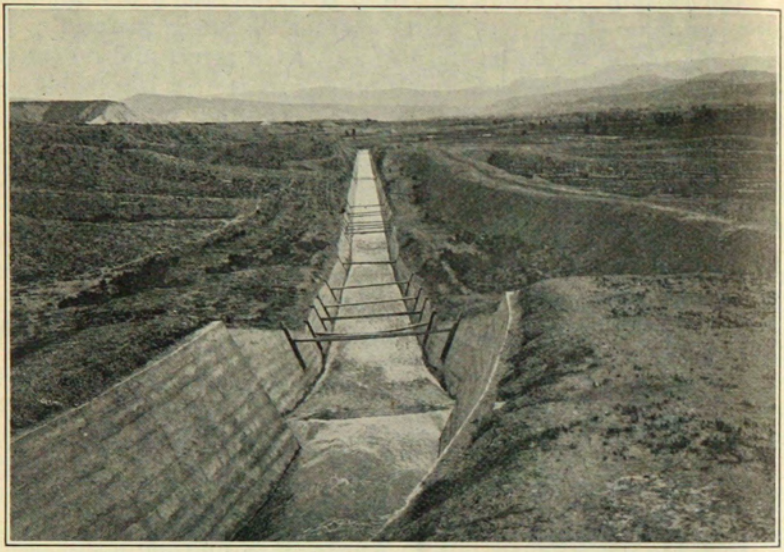

Plate VII. Concrete Chute, Uncompahgre Project, Montrose.

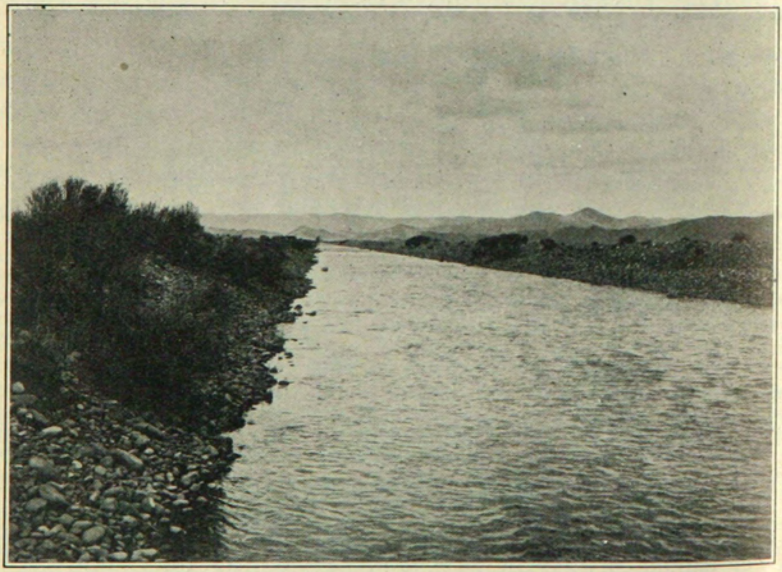

Plate VIII. Lateral No. 1, Rio Grande Canal System, Del Norte.

sistance offered by various kinds of material (gravel, rock, soil, or vegetation) covering the sides and bottom of the channel.

Sections on tangents were selected for experiment, and these were made as long as possible, that an accurate determination of the true slope of the water surface might be made. At the same time due care was exercised in choosing sections in which values of the hydraulic elements were to all appearances fairly constant throughout the channel tested. In short sections of 600 feet or less, cross-sectional measurements of water surface elevations were determined at the ends and middle of the section, while for longer sections these measurements were taken regularly from 300 to 800 feet apart, depending upon the uniformity of the section. In securing the elevations of the water surface two stakes were driven into the channel, so that their heads were below the water surface. Into the top of one stake was driven a nail whose head was flush with the water surface at the lowest pulsation, and similarly a nail was driven into the other stake to mark the highest level of pulsation. Opposite each point at which measurements were to be made, permanent bench marks were established and the elevations of the cross-sectional profile from bank to bank were tied into these. To eliminate the error due to a slight fluctuation in the discharge during the test, the position of the water surface at the various sections was marked simultaneously with the taking of the current meter measurements, or immediately thereafter. In the latter instance elevations of the water surface were marked beginning at the upper end, rather than by proceeding from the lower to the upper end. Cross-sectional elevations were taken of the bed of the canal at intervals from 0.5 feet to 4 feet, depending upon the regularity of the profile. The profiles were subsequently plotted and from these the wetted perimeters were measurd with a pair of dividers. Wetted areas were computed from the depths rather than by measurement with the planimeter. The mathematical mean of these values was used in the computations.

The accompanying diagrams, Figs. 6 to 17 inclusive, illustrate typical cross-sections assumed by the canals after years of use. Along with some of these profiles are shown dimensions and form of the canal as originally constructed, but unfortunately with most of them there is no record concerning the shape and dimensions of the original cross-section. The cross-sections shown were obtained by plotting on a large scale the several cross-sections taken in the field, and from these a new perimeter was adopted to represent an average.

Table 8 giving the results of the experiments on earth canals, was computed from the mean of all the wetted areas and perimeters in each of the sections.

CANALS OF THE RIO GRANDE SYSTEM, DEL NORTE. The main canal of this project diverts water from the Rio Grande river where it enters the San Luis Valley, west of Del Norte. About a mile and a half below the diversion dam is the first bifurcation which is at the head of Lateral No. 1. About two miles east of this point Lateral No. 1-c takes out of Lateral No. 1.

The slope of the land of the San Luis Valley is exceptionally uniform for miles, consequently the canals are built on tangents for great distances. Under these conditions an excellent opportunity was presented for making hydraulic experiments of this nature. The material composing the bed of the canals varies from fine gravel to smooth, rounded, water-worn rocks six inches or more in thickness. Near the foot-hills, in the main canal, the waterway is very rough, being formed by the larger sized stones, while out farther in the valley, as in Lateral No. 1-c, the rocks are smaller and fewer in number, and there is more gravel, which offers less resistance to the flow of the water.

The section selected on the main canal was just above the first bifurcation and had a length of 2,000 feet with a fall of six feet.

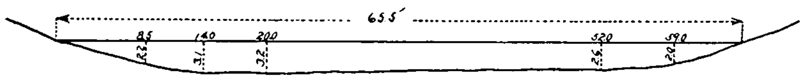

Fig. 6. Main Canal, Rio Grande System, Del Norte.

(Fig. 6.) Benches were established at the outer edge of the bank every 500 feet opposite the places at which measurements were to be made in determining the hydraulic elements. On July 20, 1912, elevations of the water surface were taken opposite these points, but owing to the large head of water and the swiftness of the current in the channel, wading was impossible at the time. Just below the bifurcation are two concrete rating stations with swing bridges, one in connection with the Main canal and the other in Lateral No. 1, from which the current meter measurements were made. The sum of the two discharges, 707 second feet, was taken as the discharge of the Main canal in the 2,000-foot length.

The upper end of the experimental section on Lateral No. 1 was about a mile below the first bifurcation. This was an even mile in length and presented ideal conditions for experimental work. (Plate VIII.) The fall was about 19.5 feet to the mile. The cross-section is shown in Figure 7. A line of benches was established 600 feet apart, and on July 20 elevations of the water surface were taken opposite these places. The water was too swift, however, to wade, so that the discharge was measured at the rat-

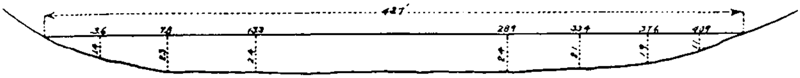

Fig. 7. Lateral No. 1, Rio Grande System, Del Norte.

ing station at the head of the canal, two percent being arbitrarily deducted for seepage loss in transit, leaving the discharge 380 second feet.

Lateral No. 1-c conveys the water down a steeper slope than do the other two canals, causing drops to be constructed about every 1,300 feet, and thereby limiting the length of test section on this canal to 1,238 feet. The cross-section is shown in Figure 8. As on the other canals, the water was too swift for taking the cross-sectional elements in July, but benches were established about 400 ft. apart and water surface elevations taken opposite these, while the gaging was done just below the drop at the head of the section where the current was not too swift to prevent wading.

On October 5 and 6, 1912, other experiments were conducted upon these canals, which at the time carried only low heads, these being 85 second feet, 33 second feet and 23 second feet respectively. On Lateral No. 1 measurements of the discharge were made at both the lower and upper ends of the mile section, to determine the loss or gain, if any, by seepage. The results were as follows:

Station	Time	Sec. Ft.
0+00	1:30 P. M.	33.306
52+80	3:00 P. M.	33.345

For a small head the change is inappreciable, but with a large discharge it is believed that a deduction of two percent as used for the measurement in July, is sufficient to cover all loss by seep-

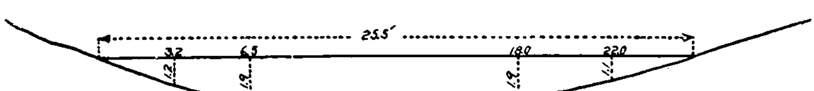

Fig. 8. Lateral No. 1-c, Rio Grande System, Del Norte.

age to the river in the two miles of section from the gaging station to the lower end of the section tested. At this time, also, cross-sectional elevations were taken opposite the bench marks, the channel being in such stable condition that it was believed that no appreciable change of cross-section had taken place from July to October.

On April 24, 1913, two more experiments were made upon the Main canal and Lateral No. 1, there being no water in Lateral No. 1-c. The discharges were 73 and 26 second feet, respectively.

The results of these tests are shown in Table 8. These canals show a higher coefficient of roughness for low heads than for high, differing in this respect from the timber flumes on the Orchard Mesa Power Canal. In other words, these rough channels gave a value for (n) which varies inversely as the hydraulic radius, which would indicate that the roughness in earth canals has a greater influence in retarding the flow of small heads of water than the same degree of frictional resistance exerts on high heads. In designing large canals in this class of materials, therefore, the lowest values of (n) given in the table should be used, while for smaller channels use should be made of the greatest values of (n).

BESSEMER CANAL, PUEBLO. This canal has been in operation about twenty-three years. From the few records available as regards the original cross-sections, the bottom width was five and one-half feet, side slopes one and one-half to one, and maximum depth of water eight feet. As the canal was operated at high stages of water, a decided tendency was developed to deposit sediment on the slope near the water surface, and to scour out at the bottom of the slope. In the sandy-loam, mesa soil, or in the extremely sandy soil of the river bottom near the headgate, the tendency has been to widen to a shallow channel with almost vertical sides. This tendency is less marked in the adobe or clayey soils. The diagrams of the cross-sections, Figures 9, 10, 11 and 12, illustrate this. In cleaning the channels on mesa lands a section was adopted which seemed to be naturally formed by the water. This was gradually widened to 8 feet on the bottom, the sides adjusting themselves in a general way to a slope of from 1 to 1, to 1¼ to 1, which section has seemed to maintain itself. The diagrams show a comparison of the original section, with the section worked to in cleaning the canal, and the section which the water seems inclined to establish. It has a high hydraulic radius and approaches an egg shape.

Four sections of this canal were experimented upon with the idea of noting the change in the value of the coefficient with the variation of the kind of material composing the bed. In each of these sections the channel is in excellent condition, perfectly straight and of uniform cross-section and grade. Elevations of the water surface and cross-sectional area measurements were made at intervals throughout their lengths. The water surface pulsated .01 feet.

Section (a), Fig. 9, borders on Stone Street, Pueblo, and is 1,495 feet in length. Its bed is of smooth, waterworn adobe, from which project very fine rootlets from the cottonwood trees planted in rows on either side of the canal.

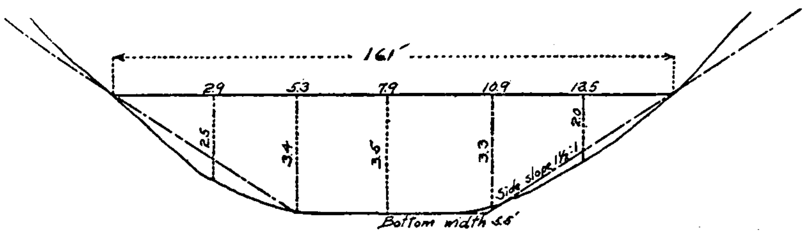

Fig. 9. Section (a), Bessemer Canal, Pueblo.

Section (b), Fig. 10, also bordering on Stone Street, Pueblo, has a lining of smooth, waterworn adobe, but the absence of rootlets accounts for the low value of (n) in comparison with section (a). Its length is 1,600 feet.

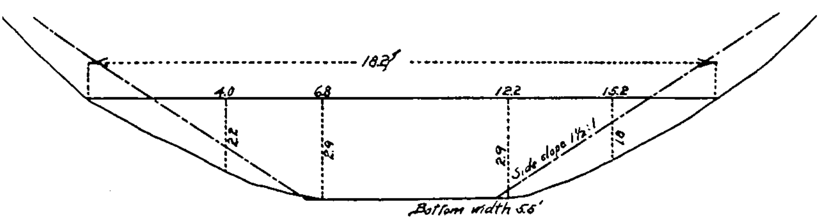

Fig. 10. Section (b), Bessemer Canal, Pueblo.

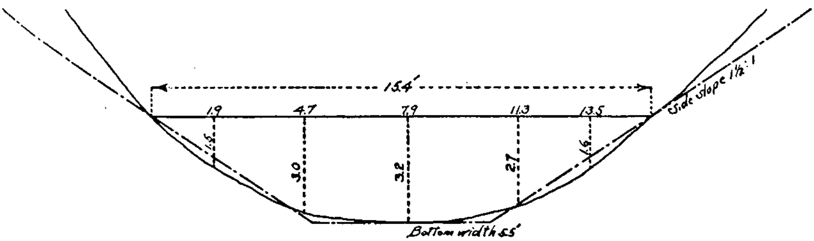

Fig. 11. Section (c), Bessemer Canal, Pueblo.

Section (c), Fig. 11, and (d), Fig. 12, located on the north side of Adams Street, Pueblo, have beds composed of fine silt merging into clays, with a liberal sprinkling of loose stones from 1 inch to 3 inches in thickness. The sections are 1,194 feet, and 2,002 feet in length, respectively.

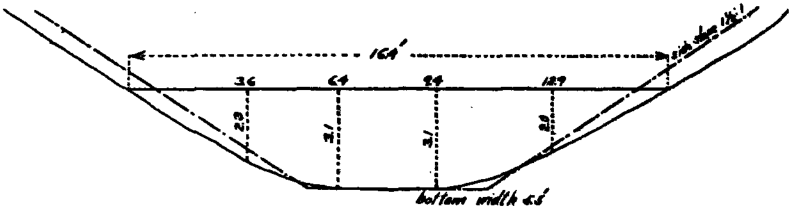

Fig. 12. Section (d), Bessemer Canal, Pueblo.

ROCKY FORD CANAL, ROCKY FORD. Two sections were tested, and in both, cross-section measurements and water surface elevations were taken at both ends and at intermediate points.

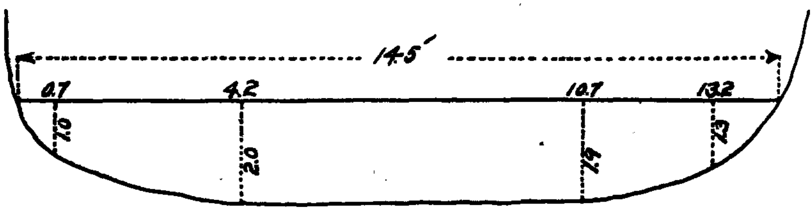

Fig. 13. Section (a), Rocky Ford Canal, Rocky Ford.

Section (a), Fig. 13, along Sycamore Street, Rocky Ford, is 1,000 feet in length, and has a bed of fine, loose sand. The sides are of clay with fine grass roots projecting. Some grass overhangs the bank into the water. This portion of the canal is somewhat crooked and the banks irregular.

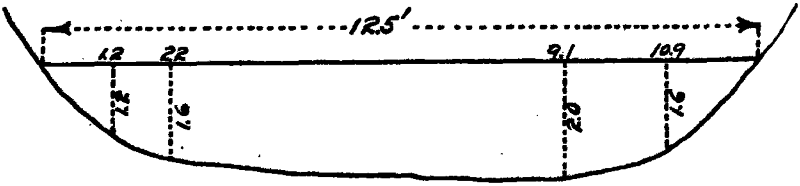

Fig. 14. Section (b), Rocky Ford Canal, Rocky Ford.

Section (b), Fig. 14, 1,350 feet in length and located one-half mile east of Rocky Ford, has a straight alignment. The bed is of fine sand, with sides of clay. Some grass overhangs into the water. The pulsation of the water surface was .01 feet.

The friction coefficients of these two sections are almost identical.

FORT LYONS CANAL, LA JUNTA. This canal, Fig. 15, one of the largest in Colorado, was built to carry 2,000 second feet of water, and is over 113 miles in length. The experiment was made in August, at a season when the canal was carrying little

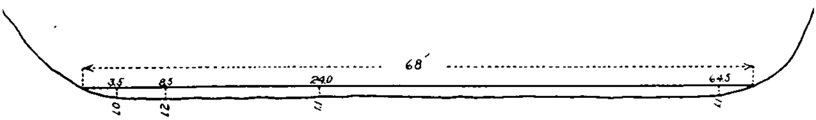

Fig. 15. Fort Lyons Canal, La Junta.

more than its first priority. The 137.26 second feet conveyed completely covered the bed of the canal for a width of from 65 to 70 feet, and from a depth of from 1 to 1.5 feet, making the conditions for such a test very favorable. The 2,600-feet section investigated, is located about five miles east of La Junta. It is constructed on a tangent, except for possibly 100 feet at the ends which are slightly curved. The canal bottom is composed of very fine silt merging into fine sand, and this in places is quite boggy. It is exceptionally smooth and regular and free from any impediments to check the flow of the water. This accounts for a lower coefficient than ordinarily assigned to earth channels. The pulsation of the water surface was inappreciable.

JARBEAU POWER CANAL, RIFLE. An experiment was made upon this waterway just a few days after it had been put in operation for the first time and before much change in the cross-

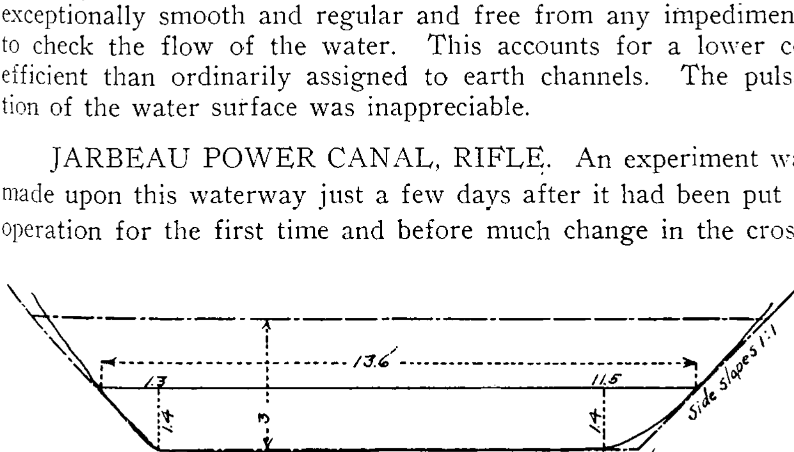

Fig. 16. Jarbeau Power Canal, Rifle.

section had taken place, as shown in Fig. 16. In the 900-foot section investigated, the upper 300 feet had a bed composed of clayey loam with a few scattering, smooth, waterworn stones projecting. The bed of the other part of the section was of clayey loam upon which aquatic moss was beginning to grow. This canal also had an exceptionally low frictional resistance. The section was designed as follows: Bottom width, 11 feet; water depth, 3 feet; side slopes, 1 to 1. The hydraulic elements used were:

Wetted area	42.0 sq. ft.
Wetted perimeter	19.5 ft.
Hydraulic radius	2.15
Slope	.0005
Velocity	2.2 ft. per second
Discharge	93.7 sec. ft.
(n)	0.025

MESA LATERAL, GRAND VALLEY CANAL, GRAND JUNCTION. The sections investigated were 550 and 600 feet in length and at the time of experiment were carrying close to their full capacity. The bottom was smoothly lined with a fine sediment, while the sides were composed of a rather uneven surface of clay loam. Short grass grew on the bank which was submerged 0.5 feet when the canal was full, as in this experiment.

WILLCOX CANAL, RIFLE. The portion of this canal experimented upon was a 400-foot stretch just below the concrete siphon under Rifle Creek. The bed consists of fine silt, sand and pebbles, with a thin scattering of rounded, waterworn stones about 6 inches in diameter. This canal was first put into operation in

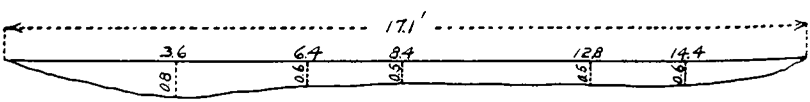

Fig. 17. Willcox Canal, Rifle.

the spring of 1912, and its cross-section is shown in Fig. 17. The head carried at the time of the test in September, 1913, was less than one-twentieth of the designed capacity of the channel.

Table 8 gives the results of all the experiments on earth channels.

TABLE 8—HYDRAULIC ELEMENTS OF EARTH CHANNELS.

Name of Channel	Length of Section Tested ft.	Discharge sec. ft.	Area of Wetted Section sq-ft.	Velocity ft. per sec.	Wetted Perimeter lin. ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)
Rio Grande System—									
Main Canal	2000	707.00	146.05	4.84	65.7	2.225	.00307	58.4	.0284
Main Canal	2000	85.13	45.81	1.86	54.3	0.844	.00299	37.0	.0342
Main Canal	2000	75.07	39.06	1.92	52.1	0.749	.00300	40.4	.0309
Lateral No. 1	5280	380.00	81.62	4.66	43.7	1.868	.00366	56.3	.0284
Lateral No. 1	5280	33.34	22.71	1.47	34.5	0.659	.00368	29.8	.0386
Lateral No 1	5280	27.16	19.52	1.39	32.8	0.596	.00362	29.9	.0221
Lateral No. 1c	1238	143.60	37.22	3.86	26.2	1.422	.00220	68.8	.0221
Lateral No. 1c	1238	22.60	12.70	1.78	19.6	0.648	.00204	48.9	.0249
Bessemer Canal a	1495	57.98	37.15	1.56	18.15	2.045	.00036	57.5	.0281
Bessemer Canal b	1600	57.98	37.27	1.55	19.87	1.877	.00024	72.9	.0219
Bessemer Canal c	1194	42.35	36.65	1.16	17.78	2.063	.00026	50.0	.0321
Bessemer Canal d	2002	42.35	33.56	1.26	18.55	1.808	.00028	55.9	.0280
Rocky Ford Canal a	1000	41.20	24.44	1.68	16.08	1.521	.00058	56.5	.0266
Rocky Ford Canal b	1350	34.33	19.79	1.73	13.98	1.415	.00069	55.3	.0269
Fort Lyons Canal	2600	140.55	75.60	1.86	66.80	1.132	.00038	89.5	.0165
Jarbeau Power	900	32.27	16.43	1.96	14.75	1.114	.00049	83.8	.0176
Mesa Lateral a	550	42.16	27.91	1.51	16.52	1.689	.00022	78.3	.0200
Mesa Lateral b	600	40.32	27.40	1.47	16.46	1.663	.00026	70.6	.0220
Willcox Canal	400	15.29	8.78	1.74	26.30	0.334	.00334	52.1	.0205

GENERAL SUMMARY OF (n).

Semi-Circular Metallic Flumes.

	(n)
Smooth, unobstructed water face; with wasteways, and of length less than 100 feet	0.021
Smooth, unobstructed water face	0.011
Joint connections protruding into the waterway beyond the plane of the sheet metal	0.0174
Corrugations at right angles to the line of flow	0.0225

Timber Flumes.

Lined with patent roofing material	0.016
Planed lumber, battens on the sides; flooring placed transversely	0.015
Planed lumber; battens on the sides; flooring placed longitudinally	0.012
Planed lumber; no battens; perfect alignment	0.011

Concrete-lined Channels.

Smooth concrete or mortar-finished surface	0.012
With surface left unplastered in condition given it by the forms	0.016

Earth Channels.

Bed exceptionally smooth and of fine silt; very uniform in cross-section; perfectly straight alignment	0.017
Bed of hard, water-worn adobe	0.022
Bed of coarse gravel and small, loose stones	0.024
Banks of smooth clay; bottom of fine sand; grass over-hanging banks into water	0.027
Bed of hard, water-worn adobe; many fine roots projecting	0.028
Bed of fine silt, merging into clay; liberally sprinkled with large, loose stones	0.030
Bed of little gravel but composed mainly of smooth, rounded stones, 6 inches or more in thickness	0.032

SIPHONS

During the summer of 1913 a number of experiments were made to determine the coefficient of friction in wood stave pressure pipe under actual field conditions. By far the greatest difficulty encountered in the making of these tests was the determina-

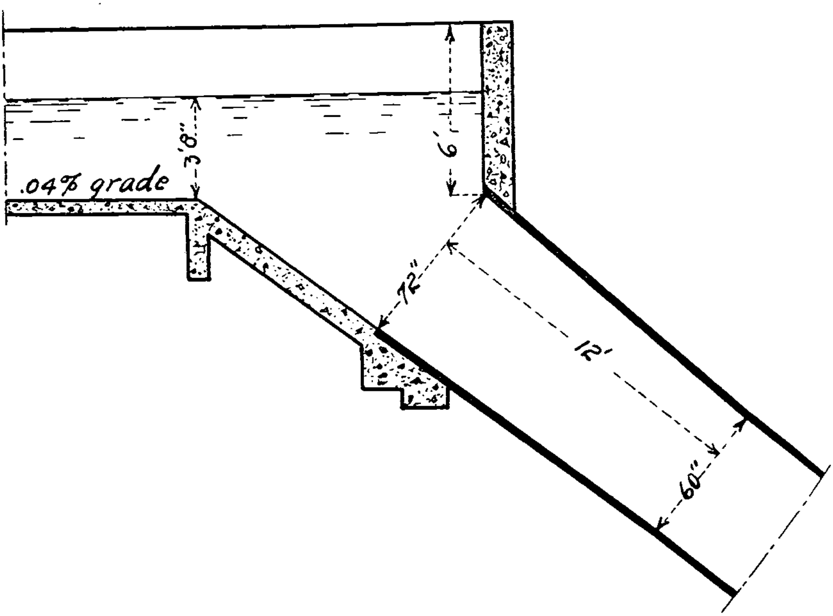

Fig. 18. Siphon Inlet, Arkansas Valley Conduit, Pueblo.

tion of the elevation of the mean water level at each end of the siphon, specially at the inlet where there was an eddying motion and boiling effect of the water. This was satisfactorily overcome, however, by siphoning water from the wood stave pipe, over the in-take wall and into a bucket which rested on a firm foundation.

A weight placed on the end of the half-inch siphon hose kept it well below the water surface. The elevation was then taken from the water in the bucket.

The discharge was measured with the current meter in the open channel leading to or away from the siphon. Data relative to profile, alignment, length and diameter of pipe was obtained from the engineering offices in connection with the projects.

ARKANSAS VALLEY CONDUIT, PUEBLO. This channel, which supplies the water to the Minnequa steel plant of the Colorado Fuel & Iron Company, at Pueblo, heads on the Arkansas River about thirty miles above Pueblo, and closely follows the

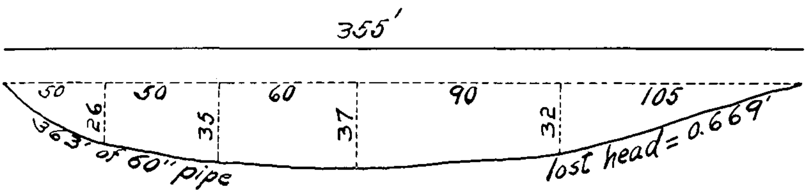

Fig. 19. Siphon No. 13, Arkansas Valley Conduit, Pueblo.

bluffs until it reaches the open mesa a few miles below Pueblo. To prevent seepage losses in the shaley material characteristic of that region, a considerable portion of the canal was concrete-lined. Numerous arroyos cross the right-of-way and at these places inverted siphons were constructed, varying from a few hundred feet

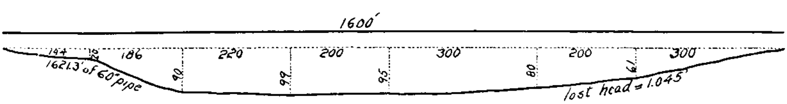

Fig. 20. Siphon No. 14, Arkansas Valley Conduit, Pueblo.

to over 2,700 feet in length, and with diameters from 48 inches to 60 inches. Plans and elevations of several of these siphons are shown in Figs. 19 to 27, inclusive. These are of wood stave pipe, but after only three or four years of service dry rot began and all of the twenty-five siphons were encased in reinforced concrete.

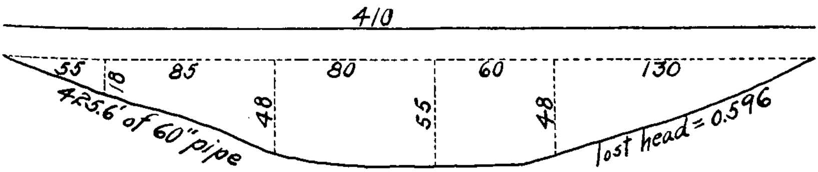

Fig. 21. Siphon No. 12, Arkansas Valley Conduit, Pueblo.

The interior, or water-surface of the pipe, is in good conditio The inlet and outlet structures are substantially constructed of r inforced concrete, to which the pipe is rigidly joined, as shown i Fig. 18. For the purpose of minimizing, or entirely overcoming loss of head due to contraction at inlet, the end of the pipe is flare at the rate of one inch to the lineal foot until the diameter at th

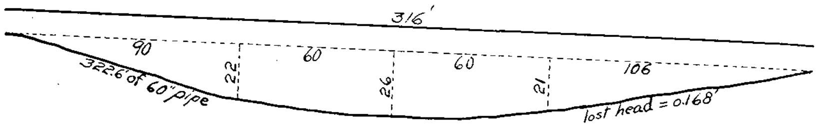

Fig. 22. Siphon No. 6, Arkansas Valley Conduit, Pueblo.

entrance is increased by one foot over the diameter of the main pipe. Wasteways are provided in the channels just above the intakes, from which are discharged surplus flood waters collected from the hillsides in times of heavy rain. There are no grating

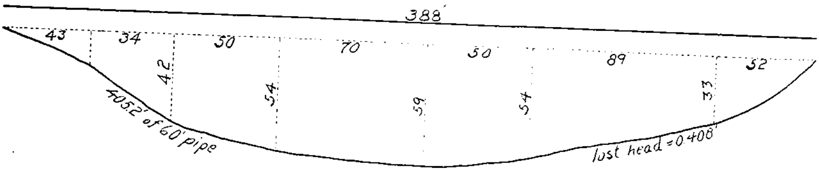

Fig. 23. Siphon No. 8, Arkansas Valley Conduit, Pueblo.

screens at either end of the siphon, for the reason that little debris of a floating nature is carried by the canal, and in freezing weather ice would form over the screen so as to completely block the flow; neither are there any blowoffs at the lowest points along the pipe

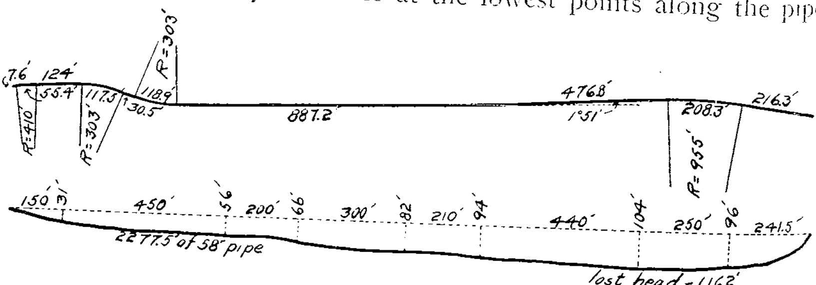

Fig. 24. Siphon Nos. 18 and 19, Arkansas Valley Conduit, Pueblo.

line, it evidently being believed that the velocities of from 2.5 to 3.5 feet per second on the conduit would effectually prevent the deposition of sediment. No information exists as to what extent, if any, gravel or particles of shale obstruct the pipe at the bottom

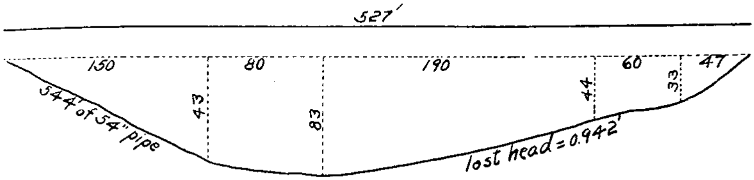

Fig. 25. Siphon No. 17, Arkansas Valley Conduit, Pueblo.

of the depressions, but the high values of (n) obtained for some of the siphons indicate the presence of debris or other unfavorable condition within those pipes.

Experiments were conducted upon this series of siphons on August 10 and 11, 1913. The structures had been in use since the

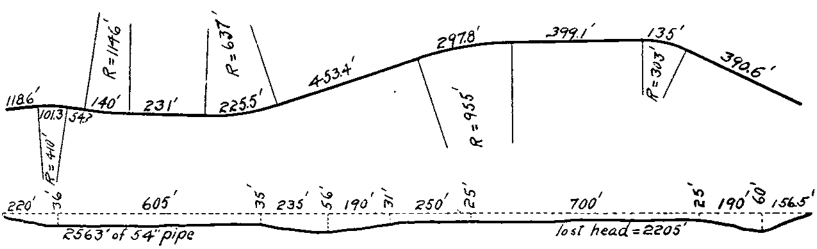

Fig. 26. Siphon No. 7, (lower), Arkansas Valley Conduit, Pueblo.

spring of 1907. There are over twenty-five siphons on this canal, but with the method used in determining the loss of head, tests could not be made upon those having very turbulent condition at the inlet, due to the drop into the penstock or having high walls over which the water could not be siphoned with the hose. Furthermore, the flow into the conduit was practically constant which prevented the determination of the effect of velocity upon the friction factors. Elevations of the water surface were referred to the company's bench marks established upon the concrete inlet and outlet structures.

Regarding the effect of ice in retarding the flow of the water in siphon No. 23, which is 50 inches in diameter, 2,797.6 feet in length, and under a maximum pressure head of 147 feet, Mr. R. M. Hosea, Chief Engineer of the Colorado Fuel & Iron Company, says: "The intake end of siphon No. 23 troubles us in winter time particularly by the formation of anchor ice inside the pipe, which appears to take place on the ascending leg of the pipe to such an extent as to check the discharge with a plug of ice particles. When the plug is forced through in advance of the water, on one or two occasions we have had it fill the ditch section so that the water ran over the sides of the ditch on top of the ice. What takes place inside the pipe is only surmised, but we believe that the ice floats in the upper half of the pipe until it accumulates sufficiently to form

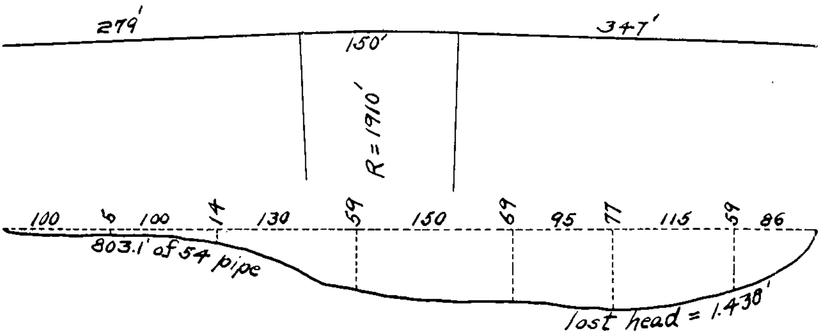

Fig. 27. Siphon No. 9, Arkansas Valley Conduit, Pueblo.

a plug, and this plug is forced out after the water backs up in the approach to the pipe and increases the pressure sufficiently to break the plug."

Table 9 shows the factors of flow in these siphons under conditions found in them during August, 1913.

REDLANDS MESA SIPHONS, GRAND JUNCTION. These are of 48-inch bore throughout, buried beneath about two feet of earth, except at the lowest points where a few feet are exposed. (Figs. 28 and 29.) As there are no blowoffs, it is believed that the velocity of 1.6 feet per second, which was found in the test made June 15, 1913, was hardly sufficient to prevent some

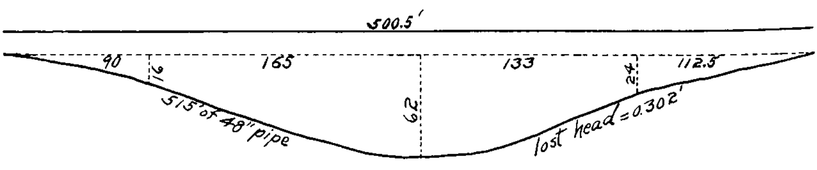

Fig. 28. Siphon (a), Redlands Mesa Canal, Grand Junction.

deposition of the red sandy sediment characteristic in that region of the water diverted from the Gunnison River. No doubt, however, when the canal is carrying its full capacity, sufficient velocity is developed to remove this deposit of silt, though coarser material may still remain. One instance is reported of a dead cow having been carried through the siphon which would indicate sufficient

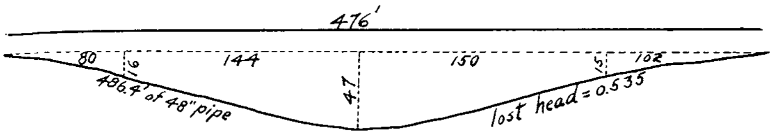

Fig. 29. Siphon (b), Redlands Mesa Canal, Grand Junction.

force of current to remove large obstructions of a floating nature.

Neither end of the pipes were flared, and at the time of these tests while the outlet ends were entirely submerged the water at the intake ends was only about three feet deep. The cross-section of the pipe was completely filled at about four feet from the entrance. A deposit of silt about 0.5 feet thick covered the bottom of the pipe at the outlet. The water at inlet and outlet was very quiet.

SIPHON UNDER PURGATOIRE RIVER, LAS ANIMAS. The Jones Extension of the Las Animas Consolidated Canal conveys water across the Purgatoire River by means of a 36-inch wood stave pipe. The pipe is buried about ten feet beneath the shifting sands of the river bed, and has a maximum pressure head, at its lowest point, of about 20 feet. No provision is made for draining the siphon at its lowest point. This conduit was tested for less of head on August 8, 1913. It was newly constructed, having been in operation only a portion of that season. Conditions were especially favorable for this experiment as the water entered and left the channel very quietly, and it is thought the 2.16 feet velocity obtained therein, together with the short period it had been in use, is reasonable evidence that no debris or silty deposits existed in the bore.

GENERAL RESULTS. Table 9 gives the data relative to the dimensions of pipe and maximum pressure heads, together with the hydraulic elements arranged in the order of corresponding mean velocities in the bore. In columns 9, 10 and 11 are given values of the coefficients commonly used in estimating the carrying capacity of pipe lines. Column 8 enables a comparison to be made of the loss of heads in the siphons on a basis of a 1,000-foot length of pipe. The other columns are self-explanatory.

The wide range in the amount of lost head per 1,000 feet of pipe, and the values of the coefficients hardly justify a close comparison.

As was previously stated, the high values of (n) obtained for some of the siphons indicates a deposit of debris at the low point of the siphon, or other condition which retards the flow of water. There were no means for inspecting the interior of the pipe, but as there are no known reasons for doubting the accuracy of the experimental data, it is probable that the pipes in question were not clean, and these data indicate their interior condition. It is not to be understood that values for (n) of 0.027, or even 0.019, are recommended for wood stave siphons properly constructed and in good condition of alignment and interior. The siphons which were expected to be in good condition did show low values of (n), but the entire data indicate the condition that may be obtained in siphons after a few years of operation where no provision is made for cleaning them.

TABLE 9—HYDRAULIC ELEMENTS OF WOOD STAVE SIPHONS.

Name of Siphon	Diameter (in.)	Length of Pipe (ft.)	Maximum Pressure head	Discharge sec-ft.	Mean Velocity ft. per sec.	Total loss of Head	Loss of Head per 1000 ft.	(c)*	(n)**	(F)***
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)
Redlands Mesa (b)	48	486	51	20.34	1.62	0.497	1.021	49.4	.0271	.1005
Redlands Mesa (a)	48	515	66	20.44	1.63	0.202	.392	82.2	.0172	.0380
Jones Extension	36	815		15.25	2.16	0.515	.632	98.6	.0140	.0262
Arkansas Val. Conduit No. 13.	60	363	37	49.70	2.53	0.669	1.843	52.8	.0275	.0925
Arkansas Val. Conduit No. 14.	60	1621	99	49.70	2.53	1.045	0.644	89.7	.0169	.0324
Arkansas Val. Conduit No. 12.	60	426	55	50.73	2.58	0.596	1.400	61.8	.0238	.0675
Arkansas Val. Conduit No. 6..	60	323	26	53.99	2.75	0.368	1.141	72.7	.0204	.0486
Arkansas Val. Conduit No. 8..	60	405	59	53.99	2.75	0.408	1.007	77.6	.0193	.0429
Arkansas Val. Cons. 18 & 19.	58	2278	104	52.68	2.88	1.162	0.510	116.0	.0134	.0192
Arkansas Val. Conduit No. 17.	54	544	83	52.68	3.31	0.942	1.732	75.0	.0195	.0458
Arkansas Val. Conduit No. 7..	54	2563	60	53.99	3.39	2.205	0.861	109.1	.0139	.0220
Arkansas Val. Conduit No. 9..	54	803	77	53.99	3.39	1.438	1.790	75.6	.0193	.0449

*Value of (c) in Chezy Formula $V = c\sqrt{rs}$.

**Value of (n) in Kutter's Formula.

***Value of (f) in formula for lost head due to friction and curvature.

$$h = f \frac{L}{d} \frac{V}{2g}$$

THE EFFECT UPON "n" OF VARIATION IN HYDRAULIC FACTORS

Many times in determining the carrying capacity to be assigned to a channel under construction, doubt arises as to the proper coefficient of roughness to use, or as to what safety factor to apply for any possible future variation in the value of the coefficient.

Table 10 illustrates the manner in which the discharge fluctuates with a variation in (n). Four typical classes of lining were selected to show this, viz., metal, concrete, timber and earth. The middle row of each group is from data actually observed in the field, while the others are computed from values of (n) changed by five units in the fourth decimal place, the hydraulic radius and slope, of course remaining constant in all cases. From the percentage values which refer to each group, it will be observed that the discharges vary in the inverse ratio as the coefficient of roughness, the hydraulic radius and slope remaining constant.

In this connection there may also be noted the accuracy of current meter measurement required to produce an erroneous value of (n). For a low value of (n) a 4 percent error in current meter measurement changes the coefficient .0122 to .0117, for instance. Or, with high value of (n) a 2 percent error in meter measurement changes .0285 to an erroneous value of .0280. Hence it is believed an error of 1.5 percent, or less, in current meter measurement was allowable in obtaining the correct discharge of the various channels under test.

TABLE 10—EFFECT UPON "n" OF VARIATION IN HYDRAULIC FACTORS.

Name of Channel	Comparative Percentages of Q	Discharge sec-ft.	Area of Wetted Section sq-ft.	Velocity ft. per sec.	Wetted Perimeter lin-ft.	Hydraulic Mean Radius	Slope feet per foot	Coefficient (c)	Coefficient (n)	Comparative Percentage of (n)
King Lateral	103	34.45	5.77	5.97	6.30	0.916	.00537	85.05	.0170	103
Metallic	100	33.30	5.77	5.77	6.30	0.916	.00537	83.21	.0175	100
Flume	97	32.26	5.77	5.59	6.30	0.916	.00537	79.65	.0180	97
Long Pond	104.5	128.92	6.24	20.66	7.24	0.862	.02971	128.80	.0117	104.5
Concrete	100	122.94	6.24	19.70	7.24	0.862	.02971	122.82	.0122	100
Chute	95.5	117.48	6.24	18.83	7.24	0.862	.02971	117.36	.0127	95.5
Orchard Mesa	104	228.76	36.79	6.22	18.12	2.030	.000903	145.28	.0117	104
Timber	100	219.40	36.79	5.96	18.12	2.030	.000903	139.32	.0122	100
Flume	96	210.88	36.79	5.73	18.12	2.030	.000903	133.92	.0127	96
Rio Grande	102	720.61	146.05	4.93	65.70	2.222	.00307	59.59	.0280	102
Main Canal	100	707.00	146.05	4.84	65.70	2.222	.00307	58.45	.0285	100
Earth	98	694.61	146.05	4.76	65.70	2.222	.00307	57.44	.0290	98

RELATION OF CURRENT METER METHODS TO FRICTIONAL RESISTANCE IN WATERWAYS

If there was no frictional resistance within a carrying channel, the filaments of flow would be parallel and of uniform velocity. In other words, if the sides and bottom offered no resistance to flowing water, the velocity would be the same throughout all parts of the cross-section. A chip floating on the surface would indicate the true velocity, which, multiplied by the cross-sectional area of the channel would give the exact flow. One current meter measurement at any point in the water would be sufficient, and stream rating would be a very simple operation.

However, the lines of flow are not parallel in either the horizontal or vertical plane, nor are their characteristics exactly the same in any two channels, because of the variation in cross-sections, and the wide range in the frictional resistance offered by the many materials of which waterways are constructed. Such irregularities have brought the current meter into universal use for measuring the velocity of water in open channels where it is impractical to install a permanent device, such as a weir.

Of the four principal methods of stream gaging with a current meter—multiple point, vertical integration, six-tenths, and the two- and eight-tenths methods—many hydrographers acquire the habit of using one method for practically all conditions. Some even assert that the method they use meets all types of channels and all conditions of flow with a uniformly nice accuracy. For the purpose of throwing some light on this subject, experiments were made on several types of channels. These were gaged in such a manner as to permit their vertical velocity curves to be plotted, as in Figs. 30 to 33 inclusive, and also to give a comparison of the several methods.

In measurements made to determine velocities in cross-section and to compare the integration and multiple point methods of measurement, the meter was held at points of 0.2, 0.3, 0.4, 0.6 and 0.8 of the depth in each vertical plane, and in a few instances at every 0.2 foot depth. These vertical velocity curves were taken at points approximately 1/10 the width of the channel for canals having greater widths than 10 feet. For narrow channels points were chosen about 0.5 feet apart. The meter was held at each point about 40 seconds. In taking the integration measurements a complete number of trips was made from top to bottom of the channel rather than endeavoring to operate the meter during a fixed number of seconds. The meter was moved slowly and at a uniform rate in the vertical plane. Gage readings of the level of the water in the channel were taken before and after making the current meter measurements, in order to note any change.

To obtain the maximum surface velocity of some sections, bits of sticks were thrown into the swiftest current and allowed to travel the length of the section. They did not all cover the distance in the same length of time, but the time interval of the fastest float was recorded.

The methods used in the experiments for comparing current meter methods, agree essentially with those outlined by Mr. F. C. Scobey, Irrigation Investigations, U. S. Department of Agriculture.

VERTICAL VELOCITY CURVES. Of the several channels in which experiments were made, four have been chosen for the purpose of illustrating the distribution of velocities in the cross-

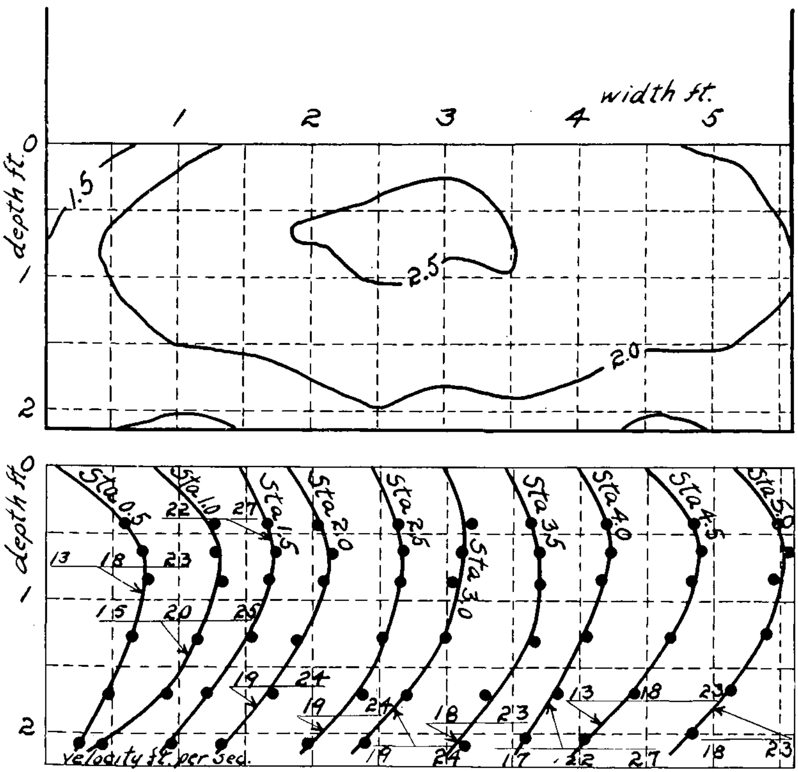

Fig. 30. Timber Flume, Pueblo Waterworks, Pueblo.

section. These are graphically represented in Figs. 30, 31, 32, and 33. The upper diagram in each figure shows the cross-section of the waterway, with lines of equal velocity, while the lower diagram shows the vertical velocities for each station in the section.

Figure 30 represents the condition of flow in a timber flume on

TABLE 11

CURRENT METER MEASUREMENTS IN TIMBER FLUME. PUEBLO WATER WORKS, PUEBLO.

Station	Depth Ft.	Discharge Sec. Ft.				Relative Percentages			
		Mult. Pt.	Integration	0.2 & 0.8	0.6	Mult. Pt.	Integration	0.2 & 0.8	0.6
0.5	2.15	2.93	3.11	2.95	3.14	100.0	106.1	100.8	107.1
1.0	2.15	2.16	2.20	2.25	2.30	100.0	101.9	104.1	106.4
1.5	2.15	2.27	2.27	2.30	2.43	100.0	100.0	101.2	107.1
2.0	2.16	2.39	2.41	2.47	2.46	100.0	100.9	103.2	102.9
2.5	2.16	2.51	2.51	2.60	2.64	100.0	100.0	103.6	101.1
3.0	2.16	2.41	2.53	2.47	2.58	100.0	105.0	102.5	107.1
3.5	2.16	2.36	2.34	2.42	2.53	100.0	99.2	102.5	107.1
4.0	2.11	2.28	2.26	2.35	2.38	100.0	99.1	103.0	104.3
4.5	2.11	2.17	2.18	2.26	2.31	100.0	100.4	104.1	106.5
5.0	2.11	3.78	3.87	3.89	4.07	100.0	102.3	102.9	107.8
Totals ...		25.26	25.68	25.96	26.84	100.0	101.7	102.8	106.3

the Pueblo Municipal Waterworks System. The cross-section was taken near the middle of a tangent about 1,000 feet in length, giving a uniform distribution of the velocities which varied from 25 feet per second midway between the sides and at about 0.3 of the depth below the surface, to 1.5 feet per second at points along the bottom and sides.

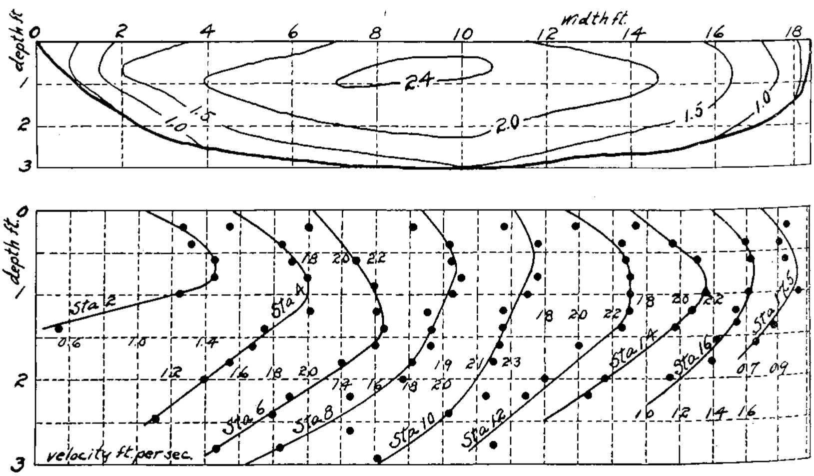

Fig. 31. Earth Section, Larimer County Canal No. 2, Fort Collins.

TABLE 12

CURRENT METER MEASUREMENTS IN LARIMER COUNTY CANAL NO. 2, FORT COLLINS.

		Discharge Sec. Ft.				Relative Percentages			
Station	Depth Ft.	Mult. Pt.	Integration	0.2 & 0.8	0.6	Mult. Pt.	Integration	0.2 & 0.8	0.6
2	1.70	3.84	4.39	3.40	4.28	100.0	114.2	88.6	111.5
4	2.50	8.05	7.85	8.15	8.70	100.0	97.6	101.1	108.0
6	2.81	10.68	10.62	10.40	11.69	100.0	99.7	97.6	109.3
8	2.97	12.65	12.12	13.54	13.19	100.0	95.9	107.1	104.1
10	3.01	12.82	12.58	13.06	13.18	100.0	97.9	101.7	102.6
12	2.85	11.23	11.06	9.92	11.51	100.0	98.4	88.2	102.4
14	2.65	9.59	9.70	9.43	9.80	100.0	101.1	98.3	102.1
16	2.32	6.44	6.71	6.71	7.18	100.0	104.3	104.2	111.6
18	1.76	3.21	3.64	3.14	3.46	100.0	113.2	97.9	107.9
Totals ...		78.51	78.67	77.75	82.99	100.0	100.2	99.0	105.7

Figure 31 shows an unusually symmetrical condition of flow, as found in an earth section of the Larimer County Canal No. 2, near Fort Collins. This constitutes an ideal cross-section for current meter work, so far as uniformity in variation of velocities is concerned. The velocity at the bottom of the deep portion of the section is almost 1.0 feet per second greater than that near the banks where the water is shallow.

Figure 32 also represents an earth section. This gaging was taken in the Hottel Mill Canal, Fort Collins, at a point on a long easy curve, which accounts for the area of maximum velocity being somewhat off center. The swiftest current is thrown toward the other bank. This same effect is shown in the cross-sectional diagram of the timber flume on the Redlands Power Canal, Grand Junction, Fig. 33. The gaging was made about 30 feet below where the canal joined the flume at a very slight angle, and hence the greater velocity on one side. However, this inequality of velocity distribution was not in the least apparent to the eye.

The effect of frictional resistance to the flow is represented by the drag at the lower part of the vertical velocity curves. As will be observed, the curves for the timber flumes are flatter than those for the earth sections, and approach more nearly a vertical position, which indicates the variation in roughness of the bed. In general it may be said that the amount of distortion of the vertical curves from a vertical line varies directly as the roughness of the bed.

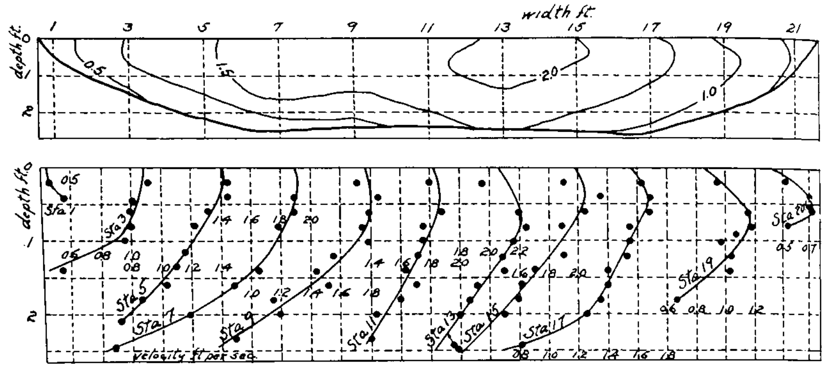

Fig. 32. Earth Section, Hottel Mill Canal, Fort Collins.

TABLE 13

CURRENT METER MEASUREMENTS IN HOTTEL MILL CANAL, FORT COLLINS.

Station	Depth Ft.	Discharge Sec. Ft.				Relative Percentages			
		Mult. Pt.	Integration	0.2 & 0.8	0.6	Mult. Pt.	Integration	0.2 & 0.8	0.6
1	0.51	0.37	0.49	0.35	0.38	100.0	132.3	94.7	102.8
3	1.45	3.05	2.99	2.96	3.29	100.0	98.1	97.1	107.9
5	2.26	4.94	4.94	5.59	4.85	100.0	100.0	113.1	98.2
7	2.44	7.38	7.38	7.73	7.82	100.0	100.0	104.7	106.0
9	2.42	7.42	7.61	7.57	7.28	100.0	102.7	102.1	98.3
11	2.43	8.21	8.36	8.36	8.02	100.0	101.8	101.9	97.7
13	2.48	9.84	9.99	9.99	10.04	100.0	101.3	101.4	102.1
15	2.56	8.60	8.55	9.11	8.50	100.0	99.4	105.9	98.9
17	2.50	6.97	6.52	7.26	6.92	100.0	93.6	104.2	99.3
19	1.92	3.75	4.02	3.60	4.06	100.0	107.2	96.0	108.2
20.6	1.08	0.77	.84	.61	0.87	100.0	109.1	79.3	113.0
Totals ...		61.30	61.71	63.13	62.02	100.0	100.7	103.0	101.2

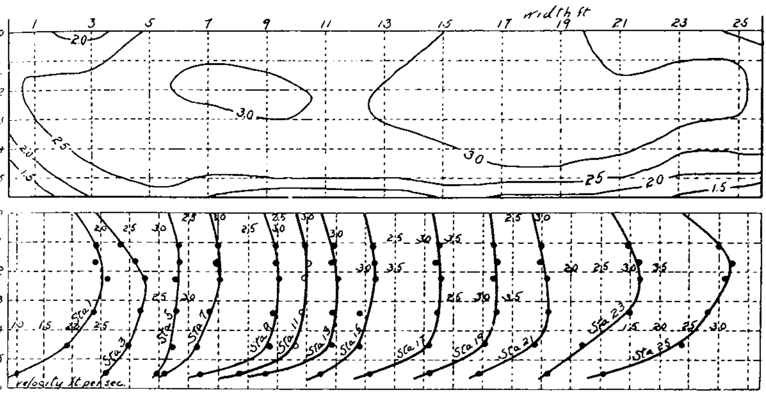

Fig. 33. Timber Flume, Redlands Power Canal, Grand Junction.

TABLE 14
CURRENT METER MEASUREMENTS IN TIMBER FLUME, REDLANDS POWER CANAL, GRAND JUNCTION.

Station	Depth Ft.	Discharge Sec. Ft.				Relative Percentages			
		Mult. Pt.	Integration	0.2 & 0.8	0.6	Mult. Pt.	Integration	0.2 & 0.8	0.6
1	5.68	24.08	23.74	24.76	27.26	100.0	98.6	102.6	113.5
3	5.68	27.38	27.49	27.38	30.67	100.0	100.5	100.0	112.1
5	5.68	30.22	31.24	31.58	31.58	100.0	103.3	104.2	104.1
7	5.68	30.79	31.58	32.15	32.38	100.0	102.5	104.3	105.1
9	5.68	30.90	30.67	33.40	33.40	100.0	99.4	108.1	108.1
11	5.68	30.90	30.56	32.83	33.06	100.0	98.9	106.2	107.1
13	5.68	30.67	31.13	32.72	33.28	100.0	101.5	106.7	108.5
15	5.68	32.60	31.81	34.31	32.94	100.0	97.6	105.1	101.0
17	5.68	33.85	32.94	35.90	36.47	100.0	97.3	106.0	107.8
19	5.68	33.40	34.76	35.33	36.69	100.0	103.9	105.8	109.9
21	5.68	31.47	32.04	33.40	35.33	100.0	101.8	106.2	112.1
23	5.68	29.54	29.20	29.51	33.97	100.0	98.9	100.0	115.0
25	5.68	30.44	32.94	30.67	31.92	100.0	108.1	100.1	104.7
Totals ...		396.24	400.10	413.94	429.05	100.0	100.97	104.47	108.28

COMPARISON OF CURRENT METER METHODS

In the following discussion it will be assumed that the multiple point method of determining the flow in open channels is the most accurate, because it calls for the greatest number of velocity determination in each vertical section. It must be admitted, however, that it is not always the most practical. In Tables 11, 12, 13

TABLE 15
COMPARATIVE RESULTS OF CURRENT METER MEASUREMENTS.

Channel	Mean Depth of Water ft.	Mean Velocity Ft. per sec.	Discharge Sec. ft.				Relative Percentages			
			Mult. Pt.	Integration	0.2 & 0.8	0.6	Mult. Pt.	Integration	0.2 & 0.8	0.6
Redlands Power Flume	5.7	2.8	396.24	400.10	413.94	429.05	100.0	100.97	104.47	108.28
Pueblo Water Works Flume	2.2	2.1	25.26	25.68	25.96	26.84	100.0	101.7	102.8	106.3
Delta Mill Flume	2.5	3.4	47.74	48.04	48.57	49.52	100.0	100.7	101.9	103.9
Church Canal Rating Flume	1.6	2.8	53.43	53.31	53.52	57.60	100.0	99.8	100.2	107.9
Agricultural Canal Rating Flume	1.0	2.4	31.29	32.87	32.12	34.48	100.0	105.1	102.6	110.1
Farmers' High Line Rating Flume	1.6	4.3	172.45	170.48	169.13	169.65	100.0	98.9	98.2	98.3
Delta Mill Canal	2.2	1.4	44.45	45.98	45.54	48.60	100.0	103.3	102.4	109.3
Hottel Mill Canal	2.0	1.3	61.30	61.71	63.13	62.02	100.0	100.7	103.0	101.2
Hottel Mill Canal	2.0	1.3	53.89	55.84	55.72	57.94	100.0	103.7	103.5	107.5
Larimer Co. Canal No. 2	2.5	1.7	78.51	78.67	77.75	82.99	100.0	100.2	99.0	105.7
Totals			964.56	972.68	985.38	1018.69	100.0	100.8	102.2	105.6

and 14, derived from the data from which Figs. 30, 31, 32 and 33 were constructed, the discharges indicated by the several methods are compared on a percentage basis, taking the multiple point determination as 100 per cent. This procedure is also applied in Table 15, which summarizes the determinations of flow in ten flumes, rating flumes and canals, including the four previously stated. The channels are all comparatively shallow, ranging from 1.0 to 5.7 feet in mean depth of water, with mean velocities of from 1.3 to 4.3 feet per second. It will be observed from the relative percentages of the aggregate discharge, given at the bottom of Table 15, that the multiple point method gives the least value and that the vertical integration, the 0.2 and 0.8 depth, and the 0.6 depth methods give higher values by percentages of 0.8, 2.2, and 5.6 respectively. These data, and general observations, apparently warrant the following statements:

1. The multiple point method, whereby the velocity is obtained by holding the meter successively at points relatively close together in a vertical plane, gives the closest determination of the actual mean velocity. Where accuracy is more essential than time, and where the condition of flow will not change during the time required to make the rating, the multiple point method should be used. The greater the number of points taken, the more accurate will be the determination.

2. The vertical integration method, in which the mean velocity at each station in the cross-section is determined by moving the meter slowly upward and downward, is particularly applicable where reasonable accuracy is desired in a comparatively short time. The water should not be too deep nor too swift to permit of the meter being moved with a uniform, slow motion in a vertical line. It is very essential that a sufficient number of complete trips be made from top to bottom, and as previously stated, that the motion be slow and unvarying. Under suitable conditions this method gives results next in accuracy to the multiple point, but the accuracy depends largely upon the skill of the operator. Some hydrographers allow a little longer time when near the top and the bottom, but since these points are the most unreliable in the section, such practice is questionable, for it has a tendency to give them too much weight in the average result.

3. The 0.2 and 0.8 method, which consists in taking separate velocity measurements at two-tenths and at eight-tenths of the depth of water at each station, and using their average as the mean velocity for the vertical section, is third in point of accuracy. This method is more rapid than either of the previous ones, but

should not be used in very narrow or very shallow channels.

4. The 0.6 method assumes the average velocity from top to bottom to be represented by the velocity at six-tenths of the depth from the surface. Actual experience has shown that this is not true in all cases, for the coefficient to be applied to velocities at this depth ranges in deep channels from 0.94 to 1.04, and in shallow channels from 0.97 to 1.04*. In some cases the mean velocity does occur at exactly 0.6 of the depth, but the error may be at least as great as six percent.

With either the 0.2 and 0.8, or the 0.6 method, approximate results will probably be obtained, the degree of accuracy varying with the cross-sectional factor and the roughness of the material. However, for large streams, and a swift current, they are to be preferred to the multiple point or vertical integration methods.

Obviously many sections of canals are unsuited to current meter work and quite unreliable results would be obtained by whatever method used. Very often when the water is not clear, what appears to the eye to be a good place for taking a rating has some hidden obstruction which exerts an influence on the lower portion of the section. This would be detected by the multiple point method, somewhat compensated for in the vertical integration method, but might cause serious error by the 0.2 and 0.8, or 0.6 method.

Due regard should always be paid to securing a favorable section for rating. It should be free from debris, moss and other aquatic growth, and the channel should be straight, or fairly so, both above and below for a distance varying with the quantity of water and general conditions. For accuracy of measurement it is essential that the water flow in as nearly straight lines as possible. Just as it is necessary to choose a suitable section for rating a channel, so is it necessary to suit the method to the conditions which obtain therein. In an ideal section each of the four methods will give reasonably accurate results, but a perfect section is rarely encountered. The value of a current meter measurement often depends as much upon the hydrographer's judgment in choosing the method and place of rating, as upon his kill in manipulating the instrument. The novice should use the more simple methods.

Acknowledgment.—To the managers and engineers of the canal systems upon which these experiments were conducted, acknowledgments are here made for the assistance rendered in permitting access to original records, and otherwise facilitating the work.

*"River Discharge." Hoyt & Grover, p. 50.