SIXTEENTH BIENNIAL REPORT

of the

STATE ENGINEER

to the

GOVERNOR OF NORTH DAKOTA

1933-1934

ROBT. E. KENNEDY
State Engineer

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The Honorable
Thomas H. Moodie,
Governor of North Dakota.

Sir:

The Sixteenth Biennial Report of this Department submitted herewith according to statutory provisions is devoted exclusively to phases of water conservation uppermost in the public mind at present.

Other problems such as flood control and navigation will have their day in court later.

Respectfully submitted,

ROBT. E. KENNEDY,
State Engineer.

Bismarck, North Dakota,
January 15, 1935.

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A STATE-WIDE SURFACE-WATER CONSERVATION PLAN

A state-wide water conservation and use plan is not the inspired fabrication of some fertile imagination but rather a policy arising from a background of studies and surveys ready for execution when the opportunity presents itself.

A "Conserve Water, Dam it" program was instituted by this Department in 1932, consisting of investigation, surveys and publicity. It has borne fruit in the present state-wide construction of small dams.

Federal Agencies Now Building Small Dams

Four of the numerous federal agencies which have come into the state in the last biennium in response to the serious emergencies created by drought and unemployment have undertaken some phase of water conservation in which this Department has been directly interested.

The Civilian Conservation Corps under Mr. A. D. McKinnon, former Highway Commissioner, has made the largest showing. Since July 1933 this organization has completed 242 projects most of which have been small dams involving 115,607 acre-feet at a cost of $930,000 for the two year period. The purpose of the C.C.C. is to provide healthy employment for worth while boys from needy families. The dam sites must be located within a cruising radius of a city of approved water supply and with a sanitary camp site.

So anxious were Forestry Service officials to put the largest possible part of the relief dollar into the boys' pockets that at first they would not sanction even the purchase of cement for the dams. Now they will not approve reinforced concrete. Nor will they approve any dams to be used for irrigation.

The Federal Emergency Relief Administration, Mr. E. A. Willson, Director, Mr. Harris Robinson, State Engineer, has undertaken the construction of about 125 dams. They are built entirely for unemployment relief and only where such labor is available in sufficient amount.

The Biological Survey has entered the state for the purpose of creating nesting and breeding areas for wild fowl. They are reported to be planning the construction of dams on the James and the Mouse Rivers. One is to be located above Minot near a site proposed for flood protection to the city. According to reports this is to be in only a limited sense a flood control dam.

The Public Works Administration, H. C. Knudsen, State Engineer, has undertaken among other things to promote conservation in the James and Sheyenne Rivers and Devils Lake by means of the Missouri River Diversion. The Corps of Army Engineers acting as field agent for the P.W.A. are again making surveys of the James River this winter (1934-35).

In addition to these federal agencies the State Game and Fish Commission has built a number of dams. No coordinating agency has supervised any of these undertakings on a state-wide plan.

Most of these dams have no legal status. They are extra-legal at least and apparently illegal in many instances. Many were built in such a hurry that no engineering study was possible. Some have already gone out. Some have been repaired. Local residents in at least one instance became dissatisfied and removed it entirely.

The immediate need is to provide some permanent agency with a cooperative fund which will give the state a voice in the location of these small dams to the end that other benefits as well as unemployment relief may be obtained. Moreover some such an agency will have to assume responsibility for the repair and maintenance of many of the more important structures. Otherwise most of this program is merely temporary unemployment relief the results of which will be more or less lost when the emergency has passed.

Prepare for the Next Drought Now

North Dakota has been called the greatest "next year" state. We are persistently jubilant over the rosy hued prospects of a return of those rainy and lucrative years. However this Department makes bold to assert that such a wishful vision is largely an unfounded fantasy and not based upon the most reasonable interpretation of facts.

It was noted in the last biennial report that the mean annual temperatures in this region have shown a definite rise during the last 30 years. The U. S. Weather Bureau published a study in the September, 1933, Monthly Weather Review which tended to indicate this to be a world wide phenomena. This does not indicate a world wide drought. Rainfall is not geared to mean annual temperatures in any hard and fast manner. But we do know that seasonal precipitation tends to reflect seasonal temperature conditions. Hot summers accompany droughts and cold winters usually have more snow somewhere in the region.

It does not seem reasonable to assume that Dame Nature is going to leap off this high temperature plateau in a wild and precipitous manner. More than likely she intends to make the descent in a gentle and dignified procedure as befits the old lady, if indeed she is yet ready to come down at all.

It would appear, therefore, from our present tho limited vantage point that the climate of the western part of the state is going to reflect this higher temperature, semi-arid condition for some time to come with a limited and insufficient rainfall and an occasional group of real wet years thrown in as a temporary respite.

A Drought Resisting Program

Now is an opportune time to face the facts and institute a permanent drought resistant program of water conservation. The methods proposed are in no sense novel or untried. They consist of more extensive terracing or contour diking, of flood irrigation from small dams wherever there is a watershed of 100 square miles more or less, and of pumping for regular irrigation from channel reservoirs in the larger streams. These are all now successfully practiced by the more enterprising farmers. The main difficulty with small private dam construction in the past has been lack of engineering suggestions. Western North Dakota is dotted liberally with the wrecks of small dams built by farmers who accurately appraised their need but were ill-advised or unadvised as to their construction.

The present drought has emphasized the fact that our present wheat area population in Western Dakota is too dense when both prices and rainfall fail. Agronomists claim much of it should never have been broken up in the first place. However the population is here and largely rooted down. Some of it will voluntarily migrate. Some in more favored locations may prefer to turn to stock raising. They should be encouraged to enlist in an effort to make the larger creek valleys as drought resistant as possible, with both stock water and flood irrigation dams. The costs are relatively small. The benefits have been and would be large. A systematic survey of such opportunities is recommended.

New Versus Old Irrigation Projects

Any program to restore a drought stricken farmer to productivity whether in a humid region or an irrigated region adds a theoretical but negligible amount to the agricultural surplus. Objection to new irrigation projects have been raised because old projects need settlers.

The one federal irrigation project in this region, the Lower Yellowstone Project, has the advantage of being a going concern of known merit. Like most federal projects it has such a large debt structure that annual repayment of construction charges constitute a tax in practical perpetuity. However the charges do not seem to be an impossible burden if one may judge from the prosperous appearance. of the project.

A complete cancellation such as is furtively hoped by many would simply increase the present landowners' capital valuation. Not much would be passed on to the new settler. If the arguments advanced are the various intangible social values such as the pioneering hardships, the creation of new communities, the increased economic stability of the social fabric, and so on, then new and undeveloped projects have equal claim to free money.

New irrigation projects are justified only where the dry land can be obtained by the government at a sufficiently low price to make the final cost competitive with the older projects. Once the title passes out of federal control the law of the jungle returns. If completely self-liquidating projects are not possible the same percent of self-liquidation should be required of all projects both new and old.

New Irrigation Possibilities

All the population that wanted to move from the drought stricken region could be located on the Missouri River bottoms and irrigation accomplished by means of pumping from a water supply of ample and perennial amount. The potential opportunities are enormous. Numerous small pumping projects are now springing up along the river. A large and permanent project would have to meet the problem of unstable river banks.

It is also the contention of this Department that pumping from channel reservoirs in the three principle rivers, the Knife, the Heart and the Cannonball, has not been fully investigated. There is a much larger minimum flow than is usually thought. The Cannonball River while dry in many places in the summer of 1934 yet had enough flow to keep water in the few channel reservoirs on the stream. Successful pumping for irrigation was done from one at Mott, North Dakota.

A pretentious irrigation and flood control project has been proposed on the Heart River about 80 river miles above Mandan, North Dakota. As a flood control project it has little merit because it is too far away. Moreover the irrigation is expensive. Twelve thousand acres in a few patches along the river would cost $111.00 an acre according to a federal report in 1926.

The idea of pumping to the lowest benches with the reservoir for supplemental supply only should be investigated. It may prove decidedly cheaper.

The Knife River project at Hazen, North Dakota, involves 5000 acres at about $190.00 an acre. That also is out of the picture for any reasonable degree of self-liquidation because the principle products will be garden truck and winter feed for the local market. Pumping from small channel reservoirs will irrigate less area but it probably would prove to be much cheaper. Electric power is also available here. The larger project would be still available if irrigation on the lower benches proved successful.

A project on the north branch of Grand River in Bowman County involves 7400 acres at $180.00 an acre. This is in a region where winter stock feed is much needed. A restudy of the water supply is being made.

Irrigation on the Mouse River by artificial flooding is described elsewhere in this report.

Stream Development for Other Purposes

Streams in various parts of the state have been relied upon for sewage dilution and for city park and recreational purposes. Reduced run-off has forced the construction of disposal plants in several cities. In some instances the run-off has been too small to even properly dilute the disposal plant effluent.

Channel storage of the smaller annual flows is an angle not now considered to any extent. Small dams built for unemployment relief could be located in reaches above these cities and designed for the express purpose of more evenly distributing the flow thru the season.

Stream Measurements Necessary

It is further contended that stream measurements are vital to the intelligent use of our water resources. The popular notion that there is no water anyway just now so why go thru the motions of measuring it is entirely erroneous. The minimum flow is exactly the limiting function of many engineering works. It is larger than the layman realizes because he sees the river dry perhaps but does not see the flood that slipped thru after a big rain for instance. The stream gager lives near the station and catches a record of every flow.

SPILLWAY HYDRAULICS FOR SMALL DAMS IN NORTH DAKOTA

BY ROBT. E. KENNEDY
State Engineer

This bulletin is prepared primarily to encourage among North Dakota engineers the use of computed channel discharges in the absence of actual stream measurements in the design of spillways for small dams.

Contents and Conclusions

The essential contents and conclusions are as follows:

(1) The **Kutter formula** and the **Manning formula,** which is preferred, require three field data from which to compute the flow of water in an open channel; the cross sectional area, the probable slope of the water surface, and the type of channel roughness. (See page 12.)

(2) **An illustration** is submitted where each formula is applied to the data from high water marks of an actual flood. The computed values are probably within about 25% of correct and hence sufficiently accurate for the purpose. Computations of a dry channel without high water marks use the average bottom grade for the slope of the water surface. (See page 14.)

(3) **Spillways submerged** take a broad-crested weir coefficient which decreases with an increase of submergence above about 60%. When the volume of discharge is available from a computed rating curve for the channel below the dam then the dimensions of the spillway are readily computed. The **size of the spillway** should be governed by the velocity of the water thru it when its maximum capacity is reached. This is a rough measure of the limiting velocity over the embankment at some higher stage. In the illustration submitted the velocity thru the spillway notch is six feet per second at its maximum capacity. The velocity is 6.5 feet per second over the embankment when it is overtopped to a depth of 0.56 feet. When the stage rises to three feet over the embankment the velocity over it is 5.4 feet per second. (See page 16.)

(4) **Secondary spillways** if properly designed and **natural spillways** if investigated for capacity may be relied upon to take care of that region of discharge between the capacity of the main spillway and that of the channel below. The maximum amount of water shunted around the structure protected with neither of these devices is governed by the safe velocity over the edge of an earth bank. This is roughly computed to exist for water depths of six inches or less in ordinary soils. (See page 20.)

(5) **The probable degree of accuracy** is not so important as the assurance that the computations lie well on the safe side. A water surface slope 50% too steep and a channel roughness yielding results 20% too large gives a computed discharge of 50% too large. If a free-flowing weir coefficient of 3.7 is used by mistake instead of a submergence coefficient of 2.33 with the same assumed discharge the crest length is shortened 37%. Under submerged conditions the discharge is reduced 22% and the overtopping velocity is increased 33%. (See page 22.)

(6) **Freeboard** added to a spillway which will be eventually overtopped in any event is an additional hazard rather than a factor of safety because the overtopping velocity is increased. The factor of safety lies in the computation of the channel capacities. An illustration is submitted which shows a velocity of 5.2 feet per second thru a 4x75 foot spillway, a velocity of 7.6 feet per second thru a 6x75 foot spillway and a velocity of 13.1 feet per second thru a 9x75 foot spillway. (See page 23.)

(7) **Chute drops** and **baffle walls** for channel spillways located at one side of an earth dam are investigated and illustrated by computations and sketch designs. The channel dimensions are readily designed when the quantity of discharge and the limiting velocity are available. The length of the jump zone on the apron of the chute determines its length. The length of the water creep under the structures must be made suitable to the material in the foundation. Structures may be placed on sandy soils if properly designed for this condition. (See page 24.)

(8) **Comments and criticisms** from other engineers. (See page 29.)

Small Dams Defined

Small dams have been defined by Mr. Tschudy* as earth dams not over 20 feet and masonry dams not over 10 feet in height. Some such arbitrary division is necessary. But most small dams in North Dakota create reservoirs mainly in the channel of the stream with perhaps a shallow overflow on the adjacent flood plain. Hydraulically it is suggested that small dams could be more accurately defined as low structures which will be subjected to more or less complete submergence during floods of channel full size or thereabout. Such structures are considered small in North Dakota because they are usually short.

Handbooks and Bulletins Must be Obtained

No hydraulic computations are possible without suitable handbooks. One very convenient handbook is King's "Handbook of

* "Instructions for the Design and Construction of Small Dams for Emergency Conservation Work in North Dakota." L. C. Tschudy and John G. Sutton, Mimeograph Bulletin, U.S.D.A. Bureau of Agricultural Engineering. Mr. Tschudy is supervising engineer for the Forestry Service in charge of small dam construction for this region.

Hydraulics" Second Edition, sold by McGraw Hill Book Company, New York, for $4.00. Another is "Hydraulic and Excavation Tables," Bureau of Reclamation, Washington, D. C., and sold for $1.50.

Three technical papers of pertinent value have been issued by the Bureau of Agricultural Engineering, Department of Agriculture, Washington, D. C. "Brief Instructions for the Design and Construction of Small Dams for Emergency Conservation Work in North Dakota" by Tschudy and Sutton is available only in mimeograph as yet (October 1934). "Flow of Water in Drainage Channels" by C. E. Ramser, Technical Bulletin No. 129, is sold for 40 cents by the Superintendent of Documents, Washington, D. C. It illustrates various conditions of channel roughness by pictures and computations. Another bulletin by Ramsey is in mimeograph form. It is entitled "Brief Instructions on Methods of Gully Control." It illustrates a variety of small gully dams.

An excellent study of the effect of submergence upon weir coefficients as illustrated by flood flows over railroad embankments was made by Yarnell and Nagler and published in "Public Roads", April 1930, entitled "Flow of Flood Water over Railway and Highway Embankments." This is included in another bulletin now being prepared by the Bureau of Agricultural Engineering but not yet available for distribution.

(1) Computation of Channel Capacity

The two formulas most extensively used are the Kutter and the Manning formulas. Each will be used in the following illustration but preference is given to the Manning formula. The necessary data are threefold; the average cross sectional area of the stretch selected, the probable slope of the water surface thru the stretch, and an estimate of the channel roughness.

Cross Sections

These formulas are based upon the assumption that the velocity of the water thru the cross section is reasonably uniform thruout. This is never exactly true. Velocities are highest on top and lowest where the water drags along the bottom. If there are shallow areas there may be no velocity at all thru them. For that reason it is preferable where possible to select cross sections fairly trapezoidal in shape and as similar as possible. In case of a very wide and shallow flood plain the coefficient of roughness should be raised to compensate for this probable range in velocities.

In the alluvial streams of North Dakota the cross section does not change radically from point to point. Relatively uniform stretches will usually be found near the point where the information is needed. Each cross section should be staked on the bank for later reference. They should be at least three in number, preferably more. They may be a few hundred feet apart. If the grade of the stream channel is very flat the sections may be 1000 feet or more apart. Vertical distances should be to the nearest tenth and horizontal distances should be in five foot intervals and at breaks in the slope.

These cross sections are plotted on cross section paper on a scale of 5 or 10 feet to the inch in each direction. The area and wetted perimeter are ascertained for any three or four water depths. The hydraulic radius is computed by dividing the area by the wetted perimeter. The area and hydraulic radius are then each plotted as abscissas with the three or four water surface elevations as the ordinate. Curves drawn thru these plotted points give these functions at any intermediate point.

Probable Slope of the Water Surface

High water marks along the banks are preferable if made by the same flood but they are seldom available at the right time and place. A small low water flow is sometimes found but the stage may be changing unknown to the engineer. If these water surface elevations are to be used the stretch should be relatively free of debris and should be long enough to minimize the effect of fluctuations upon the slope measurement.

More often the coulee is dry. The slope of the bottom grade of the channel is the only indication of the probable water surface slope of high channel stages. This is illustrated in King's Handbook, page 434, figure 138, 1929 edition. He says "at flood stages the water surface is essentially a plane having a slope equal to the average grade of the stream bed."

The average grade of the stream bed must be obtained by means of numerous elevations. But these must not be taken along the extreme bottom if it is too meandering but along the probable thread of the stream when at its higher stages. These elevations when plotted on cross section paper will reveal an average slope. A long reach will of course yield a more accurate average unless a definite break in the grade is detected.

Lines paralleling this bottom grade are drawn for the various water depths desired. Each line will give a particular water depth at each of the cross sections above described. This in turn reveals the corresponding area and hydraulic radius. The average area and radius for all the sections at each water depth is used in the formula to compute the discharge.

The discharge of several water depths thru the stretch may be plotted with the water depths and a rough rating or capacity curve obtained.

Observations of Channel Roughness

It is desirable during the survey to note the conditions of the channel which indicate the coefficient of roughness, n, in Kutter's or Manning's formula. A snap shot picture of the channel makes an excellent record for subsequent study.

Conditions causing various values of n are illustrated by pictures in Ramser's "Flow of Water in Drainage Channels." Table 89, 255, of King's Handbook gives a description of various channel conditions and their corresponding values of n by Horton.

The character and texture of the soil has a relation to the computed velocities. Table 91, page 282, "Permissible Canal Velocities after Aging", in the handbook gives the designer a basis upon which to judge whether or not his computed velocities for high stages agree with actual conditions. Does the channel, for instance, show signs of erosion during bank-full stages?

Notations

A .... Area of cross section in square feet.
c.f.s. .... cubic feet per second.
C .... Coefficient in Chezy formula or coefficient in weir formula.
D .... Depth of water above weir crest on upstream side thereof = H in King's Handbook, page 81.
D_1 ...... Depth of water entering zone of hydraulic jump.
D_2 ...... Depth of water in zone of hydraulic jump.
d .... Depth of water above weir crest on downstream side thereof = D in King's Handbook, page 96.
d_c .... Depth of water, d, causing critical velocity. See Figure 2.
g .... Acceleration due to gravity, 32.16 feet.
H, h, .. Hydraulic head. H = H_a in King's Handbook, page 332.
h_c .... Head due to critical velocity.
h_{Va} ..... Head due to velocity of approach.
L .... Length of crest, or length of hydraulic jump zone.
n .... Coefficient of roughness in Kutter's or Manning's formula.
Q .... Quantity of discharge in cu. ft. per sec.
r .... Hydraulic radius = $\frac{A}{W.p.}$
s .... Slope of water surface.
V .... Velocity in feet per second = $\sqrt{2\,gh}$
V_1 ...... Velocity of water entering zone of hydraulic jump.
V_a ...... Velocity of approach.
V_c ...... Critical velocity.
W.p. .... Wetted perimeter.
W.S. .... Water surface.

(2) Illustration of Channel Computations

An earth dam was proposed in a deep coulee tributary to the Missouri River near Mandan, North Dakota. During the survey of the site a large flood occurred. No stream flow measurements were possible but high water marks were left in the form of a well defined line of leaves, twigs, and fine debris. Local inhabitants insisted it was the largest flood they had ever seen. If so then information as to its approximate size was desirable.

Three cross sections were obtained thru a fairly straight length of channel containing some weeds and brush. These were plotted and the area and hydraulic radius derived for each section.

Station Feet	Area Square feet	Radius Feet	Elevation W.S. feet
0	139.0	1.74	105.20
110	157.7	1.66	104.73
180	136.3	1.95	104.38
Average	144.3	1.78	

(a) Solution by the Widely Used Kutter's Formula for C in the Chezy Formula $V = C \sqrt{r s}$

Referring to King's Handbook page 255 for a value for n the conditions of this channel are most accurately described under the heading "natural stream channels; (2) some weeds and stones." Since brush in leaf will offer more obstruction than stones the n of 0.040 is selected.

Average slope of water surface, s, from the data is $\frac{0.82}{180} =$ .0046 feet per foot. Average radius, r, is 1.78 feet.

C from Table 93, page 288 for slopes .001 or thereabout, is interpolated in the column headed .040 between values for r of 1.5 and 2.0. For r of 1.78 it amounts to 38.7.

$$\text{Then } V = 38.7 \sqrt{1.78 \times .0046} = 3.50 \text{ feet per second}$$
$$Q = 3.50 \times 144.3 = 505 \text{ c.f.s.}$$

(b) Solution by the Manning Formula

Referring to Table 99 page 297, the tabulated value for r equal to 1.78 and s equal .0046 is interpolated between s = .0045 and .0050 and r = 1.7 and 1.8. It amounts to 0.1480. Dividing this by n equals 0.040 for velocity, V, as instructed at head of table; V = 3.70 feet per second. Q = 3.70 x 144.3 = 534 c.f.s. The difference is in the formulas.

If it is desired to use a higher value of n than given in Table 93 then Table 99 may again be used. Try n = .045, for instance. Then the velocity would equal $\frac{0.1480}{.045} =$ 3.29 feet per second and the discharge Q is 475 c.f.s.

It was observed that the soil was a sandy loam and that some erosion developed during the four or five hours of flood flow. From Table 91, page 282, it may be seen that a flood on sandy loam with velocities around 3½ feet per second might cause some erosion.

The flood therefore probably contained from 450 to 550 c.f.s. Closer accuracy is unnecessary. It was a large flood, but not the one for which to design the spillway. Larger floods in terms of run-off per square mile of drainage area have occurred on other tributaries to the Missouri River in North Dakota. They will occur here in time.

The dam was subsequently built. The location is just above a bridge on a branch line of a railroad. The spillway was designed for 1000 c.f.s. The drainage area is seven square miles.

Capacity of Dry Channels

The capacity of a dry channel is computed exactly as in the above illustration with the exception that the average slope of the stream bed is used for the slope of the water surface in lieu of observed high water marks and several water depths are computed instead of one. The smaller value of n gives the larger computed discharge.

It is not to be assumed that all channel computations will be as simple as this. One situation occasionally met is a highway fill and culvert across the valley floor a short distance below the dam site. This will control the elevation of the water just below the dam. To compute its elevation the capacity of the channel below the fill must first be computed. The fill is then computed as a broad-crested weir and the culvert as a short tube or submerged orifice as the case may be. Appropriate formula and coefficient may be obtained from King's Handbook. This is a situation similar to an overtopped dam and spillway which is illustrated on page 18. If the highway fill is not overtopped the culvert handles all the flood and the computations are simplified.

Another topographic situation occasionally found is a stream channel in the form of a series of wide lake beds connected by short narrow channels. Here the channels between the ponds usually control the elevation of the water surface in the ponds. The slope of the water surface thru the pool is usually so flat as to be negligible. But in case of doubt a few elevations along the thread of the high water flow will indicate what the slope probably will be.

(3) Effect of Submergence Upon Weir Coefficient

Masonry dams are in effect broad-crested weirs. The one proposed by Mr. Tschudy to be built by boulders with only enough concrete to hold them in place has a rounded crest approximately equivalent to a 2.5 feet radius, a 3/4:1 batter on the downstream side and a 2:1 earth fill on the upstream side. This is roughly similar to Figures 64 to 68, page 161, King's Handbook. The coefficients, Table 62, for a four to five foot head range from 3.53 to 3.83 and average about 3.7. This is for free-flowing conditions.

Submergence has little effect until $\frac{d}{D}$ is about 60%. Above that the coefficient diminishes rapidly as $\frac{d}{D}$ approaches 100%. Figure 1 shows the six curves given in the Yarnell and Nagler article superimposed upon each other. See also Figure 2. It will be noticed that altho the free-flowing coefficients are widely divergent they all begin curving downward at about $\frac{d}{D}$ = 60% and approach a point of coincidence for a coefficient of one and a submergence of about 99%. In the absence of more accurate information this rate of decrease for the Tschudy structure is assumed in this paper to parallel that for a single track railroad embankment whose free-flowing coefficient is 3.66. Mr. Tschudy prefers to use 3.0 which he feels will cover average conditions. The following coefficients for various percents of submergence are taken from the curve and are to be used in the formula Q = CLH $^{3/2}$.

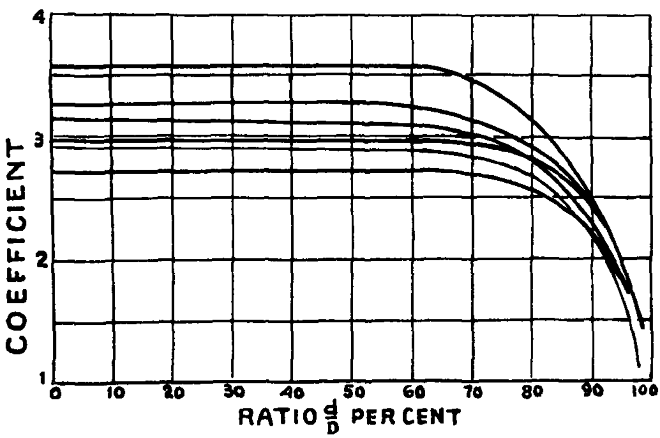

FIGURE I. RELATION BETWEEN COEFFICIENT AND PERCENT SUBMERGENCE OF RAILWAY EMBANKMENTS

These curves superimposed upon each other are taken from an article in Public Roads, April 1930 by Yarnell and Nagler, "Flow of Flood Water Over Railway and Highway Embankments."

Submergence	Coefficient
0 to 60%	3.7
70%	3.45
80%	3.15
90%	2.55
95%	2.00

Illustration

A river channel 85 feet wide and about 12 feet deep has the following capacities for a stretch downstream from the dam site as taken from the rough rating curve not shown.

Water Depth Feet	Capacity C.f.s	
5	250	
8	550	
10	900	
11	1,150	
12	1,500	
13	2,500	(out of bank)
14	4,500	(estimate)
16	10,000	(estimate)

The depth agreed upon is eight feet to the lip of the spillway.

Dam Designed for Overtopping

It is the contention of this paper that the velocity of the water thru the spillway notch at its maximum capacity is a rough upper limit of the velocity which will develop when higher stages overtop the embankment. Initial stages of overflow will probably yield less velocity because of wider top width, greater friction, and smaller hydraulic radius. The velocity should become reduced during later stages because of increasing submergence and reduced velocity of approach. The embankment may be especially prepared for overflow by covering it with gravel which according to Table 91, King's Handbook, is safe for a velocity of six feet per second in a canal carrying muddy water. Conditions are assumed to be sufficiently similar to set six feet per second as a roughly limiting velocity over an embankment, at least for illustration.

It is assumed for illustration that a dam about 500 feet long extends entirely across the flood plain. All floods must either go thru the spillway or over the top. See Figure 2 for illustration of submergence and notations.

To ascertain the proper size of the spillway notch a "cut and try" procedure is necessary. The downstream water depth, d, is first assumed to be five feet above the spillway lip. The down stream channel depth is then 13 feet for which the computed discharge, Q, is 2500 c.f.s. It is readily seen that if this amount is to pass thru an area of five feet in depth, d, with a velocity of six feet per second the length of the area, or crest length, L, is 83 feet. It remains to compute the value for D, the upstream height of the notch, from the weir formula for that discharge and crest length.

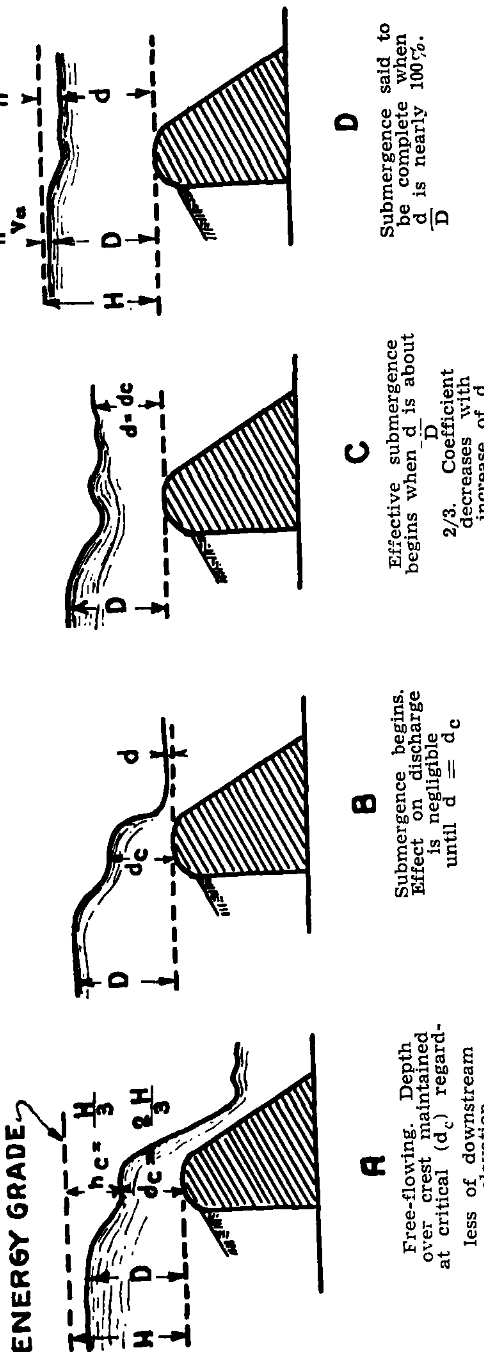

FIGURE 2. SUBMERGENCE OF BROAD-CRESTED WEIRS

Assume a submergence of 92%. If $\frac{d}{D}$ = 92%, D = 5.44 feet. The velocity of approach may be computed from the cross sectional area of the pond as taken from the contour map. It is here assumed to be two feet per second. Then from the formula

$$V = \sqrt{2gh}$$

the velocity head is found to be 0.06 feet. Then H = 5.50 feet. C for a submergence of 92% is equal to 2.33. The weir formula is $Q = 2.33\ LH^{3/2}$.

Substituting the values indicated the computed values are found to agree very closely with the assumed. The spillway notch is 5.4 x 83 feet.

It is now desirable to know what velocity will develop over the embankment with a 14 foot channel stage for instance and an estimated discharge of 4500 c.f.s. The degree of submergence will not materially change during this particular increase in stage because of the greatly increased weir length from 83 to 500 feet. The embankment is now a broad-crested weir with a length of 417 feet and an elevation of 13.44 feet above the river bottom. The free flowing weir coefficient is assumed to be 3.0. Submergence effect is given on page 25.

At the spillway d = 6.0 feet and if $\frac{d}{D}$ = 92%, D = 6.52 feet. H = 6.58 feet if the velocity of approach is two feet per second. C = 2.33. Then $Q = 2.33 \times 83 \times (6.58)^{3/2} = 3265$ c.f.s. Velocity is 6.6 feet per second.

Over the embankment the d is 14.00 — 13.44 = 0.56 feet. D = 6.52 — 5.44 = 1.08 feet. Velocity of approach is probably unchanged. So H = 1.14 feet. $\frac{d}{D}$ = 52%. C is therefore for free-flowing conditions and equals 3.0. L = 417 feet. $Q = 3.0 \times 417 \times (1.14)^{3/2} = 1520$ c.f.s. The combined discharge is 4785 c.f.s. which is sufficiently close to the desired 4500 c.f.s. for the purpose of illustration. The velocity over the embankment is 6.5 feet per second.

When 10,000 c.f.s. passes three feet over the embankment with a channel depth of 16 feet it is found that 4100 c.f.s. passes thru the spillway notch and 5720 c.f.s. flows over the embankment. Velocity is 5.4 feet per second thru each section.

(4) Secondary Spillways

This 5.4 x 83 foot spillway notch requires an embankment built to an elevation of 13.4 feet above the stream bed the banks of which are 12 feet high. It may appear impractical to build such a low embankment for the sole purpose of making all floods less than 2500 c.f.s. go thru the spillway. Since larger floods go over it in any event it may be omitted and these floods allowed to go around the end of the structure.

A critical situation develops when this water becomes deep enough to erode the edge of the bank before the channel fills up sufficiently to prevent it. This can be easily and cheaply remedied by the construction of a secondary or bypass spillway of earth which will spread out this excess amount in a thin sheet with a predetermined velocity.

With a capacity curve for the channel the necessary data are all available. The fall over the bank will be about the same as the fall in the water surface thru the spillway notch. The amount to be provided for is the difference between the spillway and the channel capacity.

For purposes of illustration the length of the spillway crest is shortened with an eye on the amount of water that will be forced around the structure. D is the distance from the crest to the top of the bank or 4.0 feet. $\frac{d}{D}$ is assumed for trial to be 60%. Then d = 2.4 feet. Let H = D this time. C = 3.7. The water depth below the dam is 8 plus 2.4 or 10.4 feet. This corresponds to a discharge of about 1000 c.f.s. The formula is $Q = 3.7 \, LH^{3/2}$.

Substituting and solving for L the crest length,

$$L = \frac{1000}{3.7 \times 8.0} = 34 \text{ feet}$$

This spillway notch of 4 x 34 feet forces all water in excess of 1000 c.f.s. around the structure until 1500 c.f.s. is reached and the channel below becomes full. The maximum amount thus bypassed is 500 c.f.s. It must fall 1.6 feet over the edge.

The bypass spillway is merely a wide, flat, shallow canal in earth dug around the structure upon such a grade that the velocity will not exceed three feet per second or whatever is suitable for the material in the site. The coefficient of roughness, n, may be taken conveniently at .034 for illustration. The water depth is about equal to the hydraulic radius and will be 1.6 feet when full. The necessary slope is taken from table 100, King's Handbook, page 303. This amounts to 0.24%. The approximate length would be 1.6 feet divided by .0024 or 670 feet. The width is equal to 500 divided by 3 x 1.6 or about 100 feet. It involves about 4000 cubic yards in level cut.

This construction is proposed to care for the critical period of flood stage when the discharge is between the spillway capacity and channel capacity below.

If this is too much earth construction try a submergence of $\frac{d}{D}$ = 75%. Then d = 3.0 feet. Q = 1150 c.f.s. C = 3.3, L = 44 feet.

The bypass canal must now carry 350 c.f.s. at one foot depth and hydraulic radius. This requires a slope of 0.45% for a three-foot per second velocity. The width is 117 feet and the length 220 feet.

Natural Spillways

Frequently the banks are higher at the water's edge than elsewhere. The excess waters collect in what is usually called a natural spillway which is a shallow depression or old high water channel nearby. They follow this to a junction with the main channel farther downstream.

Natural spillways have proven disastrous in many instances because their capacity was usually over-rated. The procedure here is to ascertain their capacity with a few cross sections remembering that the side of safety lies in too small a computed discharge rather than too large. The spillway for the dam must be designed to cooperate with this amount.

The elevation of the water in the main channel at the junction with this natural spillway channel should also be computed to indicate what drop-off, if any, will occur.

Limiting Velocity Over Earth Bank

Erosion of a cut bank of earth occurs at two places. Water depth develops velocity over the edge. Quantity and falling distance develop erosion where the water hits the earth below. The combination is destructive. If it pours over into a pool of water then velocity over the top is the limiting factor. If three feet per second is the limiting velocity over the top the critical head, h_c, corresponding to that velocity is 0.14 feet and the total head, H, is 0.42 feet, theoretically speaking.

Figure 2A gives a rough illustration of the situation. In this case there is no enlarged cross sectional area upstream so velocity of approach may be high. The friction factor is also large because the breadth of the weir is indefinite. The location of the critical depth is at some distance upstream and away from the edge. Riffles are apt to develop between that point and the edge. Velocity will average close to the critical. In general it may be said that the water depth over a plot of ground around the end of a structure and adjacent to the edge of an earth bank with free fall below should not be allowed to exceed six inches if erosion of the edge is to be avoided.

(5) Probable Degree of Accuracy

Computations for the capacity of a natural water course need not be carried to great refinement for the purpose of these studies. The exact amount is not so necessary as the assurance that the amount assumed is well on the safe side whichever way that may lie. Thus if for an assumed water surface elevation a computed discharge is certainly too large then for an actual discharge of that amount the actual water surface elevation will exceed the assumed and certainly will not be less.

In the water slope computations any percent of error in estimating the amount makes a little less than one-half that percent in the computed velocity. In the assumption of n it was shown that a change from .045 to .040 increased the computed discharge 12% for that particular situation. If the change had been from .050 to .040 the increase would have amounted to 25%.

The area and hydraulic radius computations will probably be fairly accurate. If the slope assumption were 50% too large that would make the computed amount 25% more than the correct amount or the correct amount would be 80% of the computed. If the n assumptions increased the correct amount by 20% the correct amount would then be 83% of the computed and the 80% would be reduced to 67%. Then the correct amount would be two-thirds of the computed or the computed would be 50% too large.

Uncertainty will exist as to the proper weir coefficient to use and the effect upon the results of an erroneous selection. In the illustration on page 18 where a coefficient of 2.33 for a 92% submerged weir was used to obtain a crest length of 83 feet a free-flowing weir coefficient of 3.7 might have been used in the absence of information regarding submergence. That would shorten the length 37% or to 52 feet for the assumed discharge of 2500 c.f.s. But it would have been submerged whether so designed or not. The capacity would not be 2500 c.f.s. but something much less. It is desirable to ascertain the overtopping velocity for such a spillway notch, namely 5.4 x 52 feet. D = 5.44 feet, H = 5.50 feet assuming the same two feet per second velocity of approach. It is necessary to try for a per cent of submergence which will yield a value for d and Q from the weir formula.

A few trials reveal that if $\frac{d}{D}$ is taken at 85% the d is 4.62 feet, and Q from the rating curve is about 1950 c.f.s. The corresponding H is 5.57 feet which is close enough. The velocity thru the notch just as overtopping begins is $\frac{1950}{4.62 \times 52} = 8.1$ feet per second. Then an increase in the weir coefficient of 59% produces an overtopping velocity of 33% over the assumed limit.

(6) Additional Freeboard Increases Overtopping Velocity

Not only does shortening the crest length increase the overtopping velocity but increasing the depth of the notch has the same effect when or if the structure is overtopped. For this illustration assume that the spillway is designed to fit the profile. A notch of 75 foot crest length and four foot depth would fit nicely into a channel 85 feet wide on top and 12 feet deep.

The abutments are first assumed to be just four feet high. Then D = 4.0 feet and d is found by trial to be about 3.7 feet. The capacity is about 1400 c.f.s. Amounts in excess of that which pass around the structure will have only about 0.3 feet to fall over the edge. Even tho it collected in a notch of that depth erosional velocity would not develop. Velocity thru the spillway is 5.2 feet per second.

Next assume the abutments to be raised to an elevation of six feet above the spillway lip and an embankment built across the flood plain to an elevation of 14 feet above the stream bed. D = 6.0 feet, d is found by trial to be 5.25 feet. Q = 3000 c.f.s. Velocity thru the notch is 7.6 feet per second.

Add three feet more. The spillway notch is now 9.0 x 75 feet. D = 9.0 feet, d is about 6.8 feet. Q = 6700 c.f.s. and velocity thru the notch is 13.1 feet per second.

(7) Chute-drop for Earth Dam

Earth dams are desirable where feasible because they usually put more of the relief dollar into local labor than does a masonry spillway. A suitable arrangement for a spillway is to make an excavated canal of one of the borrow pits and thereby pass the stream around one end of the dam. One type of structure commonly installed at the end of the canal is the chute-drop a sketch design for which is shown in Figure 3.

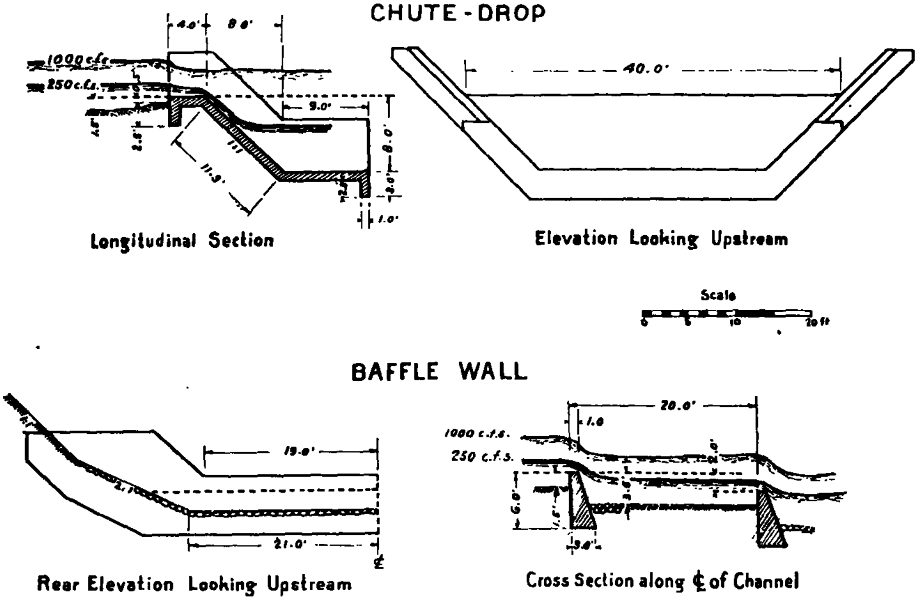

FIGURE 3. SKETCH DESIGN OF CHUTE-DROP AND BAFFLE-WALL SPILLWAY

This chute is in effect a broad-crested weir with a four foot top thickness, a 1:1 downstream slope and a varying but very flat upstream face. A coefficient for such a structure may be estimated from Tables in King's Handbook. From Table 51, page 164, it may be noted that increasing the breadth of a vertical walled weir from nine inches to four feet decreased the coefficient under a four foot head from 3.32 to 2.79 or 84%. From Table 58, page 167, it is noted that an 8-inch weir with 2:1 sloping up and downstream sides has a coefficient of 3.62 under a four-foot head. If broadening this crest to four feet also reduced it 84% the resulting coefficient would be 3.04. In lieu of more accurate information a coefficient of 3.0 is suggested.

The effect of submergence of a railroad embankment having a free flowing coefficient of 3.0 is as follows:

Submergence	Coefficient
0 to 50%	3.00
60%	2.96
75%	2.88
90%	2.50
95%	1.82

The throat opening should be designed for submerged conditions if they are likely to occur. A bypass channel may also be indicated by topographical conditions.

Dimensions of the Excavated Canal

The velocity in the earth section should not be allowed to exceed those given in Table 91 of King's Handbook. Even this is trespassing slightly on the factor of safety because new canals are more sensitive to erosion than aged canals. A conservative figure suggested for general conditions is three feet per second.

If it is desired to pass 1000 c.f.s. thru an earth canal at such a velocity the cross section would be $\frac{1000}{3}$ or 333 square feet. This may be arranged in whatever dimensions are most suitable to construction requirements. A bottom width of 50 feet, a water depth of 5.5 feet and side slopes of 2:1 yields the necessary cross sectional area. If the spillway notch previously designed for a safe overtopping velocity is 4 x 40 feet, the earth section is 10 feet wider and 1.5 feet deeper. The transition should flare at a rate of about 4:1 from each side which would make it $\frac{10}{2}$ x 4 or 20 feet long.

Transition head losses are ignored.

Hydraulic Jump Below Chute

The hydraulic jump is more apt to occur during less-than-submergence flows. If the actual pool depth is greater than that required for the jump it will occur upon the apron close to the structure. Assume a discharge of 250 c.f.s. This corresponds to a five foot water depth. The water surface is three feet below the crest. Using the free-flowing coefficient in the formula with a length, L, of 40 feet,

$$Q = 3\ LH^{3/2}$$

$$H^{3/2} = \frac{250}{3 \times 40} = 2.08 \text{ feet. } H = 1.63 \text{ feet.}$$

That is, the water would be 1.6 feet deep on the upstream side of the crest of the chute-drop. Near the edge it would be flowing at the critical depth of 2/3 of 1.6 feet or 1.1 feet. The critical velocity would be

$$V_c = \frac{250}{1.1 \times 40} = 5.7 \text{ feet per second, this for use later.}$$

The hydraulic jump will occur where this high velocity stream strikes the downstream pool provided it is deep enough.

The water coming from the canal above must drop 1.6 feet to the crest of the chute plus 3 feet to the water surface in the pool or a total of 4.6 feet of head, h. Neglecting friction and turbulance this would produce a velocity of $V_1 = \sqrt{2gh}$ or 17.2 feet per second. Since the chute width is still 40 feet the depth of the stream striking the pool, $D_1 = \frac{250}{40 \times 17.2} = 0.36$ feet.

Entering Table 110, page 350, King's Handbook, with $V_1 = 17.2$ feet per second and $D_1 = 0.36$ feet, the necessary depth, D_2, in the pool below if it be a rectangular channel is found to be only about 2.4 feet. A trapezoidal channel of 40 feet bottom width will give practically the same depth. A pool depth of five feet is ample. In any event the pool depth should exceed D_2.

V-shaped Chutes

Some saving in material will result if the sides down the chute are brought in on a 1:1 slope as shown in the sketch, Figure 3. Then at a five feet water depth in the pool the width of the chute would be 34 feet and at the bottom of the pool it would be 24 feet. Jump conditions are practically unchanged.

Length of Apron*

A formula in "Civil Engineering", May, 1934, page 262 for length of hydraulic jump is useful in computing the length of the apron. It is

$$L = \frac{9.0\ V_1\ D_2}{2V_1 - 1.5\ V_c}$$

Substituting these quantities as computed above

$$L = \frac{9.0 \times 17.2 \times 2.4}{34.4 - 8.6} = 14.4 \text{ feet.}$$

There are five feet of pool on the submerged part of the chute. Then an apron nine feet long will meet requirements. A wedge of concrete at the outer edge of the apron will further aid in confining the jump to the concrete section.

*See Page 32.

Length of Water Creep

Each structure should be inspected for possible piping thru the earth underneath it or around the ends. A paper by E. W. Lane in the September, 1934, Proceedings of the American Society of Civil Engineers offers excellent suggestions in this matter.

In general water has more of a tendency to creep along the smooth hard surface of a structure than to pursue the shorter path directly thru the earth. It also creeps under a horizontal surface more readily than along a vertical surface. Mr. Lane suggests that the horizontal contact distance be divided by three before being added to vertical distances to obtain the total length of contact between the structure and the earth. This is called the weighted-creep distance. Sloping surfaces of 1:1 or steeper are classed as vertical.

The shortest path directly thru the soil is the proper length to be considered when its ratio to the pressure head is 80% or less of that computed for the weighted creep.

The theoretical head which pushes the water thru this underground path is the greatest difference possible between the upper and lower water surface elevations. For a masonry dam or a chute-drop such as is shown in Figure 3 is occurs when the pool is just level with the crest and no water is going over. The head on the sub-soil path would then be 8.0 feet. The path by weighted-creep computation would be composed of a horizontal distance of 13/3 feet plus a vertical of 20.8 feet or 25.1 feet. The ratio of the weighted-creep path to the pressure head would be $\frac{25.1}{8.0}$ or 3.1.

The shortest path thru the soil includes 5.5 feet of vertical distance and amounts to 28.5 feet as scaled from the drawing. This gives a ratio of $\frac{28.5}{8.0}$ or 3.5 which is larger than the weighted-creep rather than smaller so it may be ignored.

One qualification should be made. If there is any probability of the last three feet of the contact surface being eroded away that would reduce the weighted-creep path to 22.1 feet and the ratio to 2.8. From the table the structure would appear safe for almost any clay soil but not for sandy or silty soils.

Location of Dams on Sandy Foundation

With this information it is possible to locate dams at sites otherwise suitable except for sandy foundations. The path of percolation can be lengthened to meet the requirements of the material in the site.

TABLE 1—WEIGHTED-CREEP RATIOS FOR VARIOUS SOILS*

Material	Safe weighted-creep ratio
Very fine sand or silt	8.5
Fine sand	7.0
Medium sand	6.0
Coarse sand	5.0
Fine gravel	4.0
Medium gravel	3.5
Coarse gravel, including cobbles	3.0
Boulders with some cobbles and gravel	2.5
Soft clay	3.0
Medium clay	2.0
Hard clay	1.8
Very hard clay, or hardpan	1.6

Recommendations are made in the paper regarding weep holes and drains and several other interesting conditions. It is worthy of study by anyone entrusted with the design of dams either large or small.

Baffle-Wall Spillways

Mr. Tschudy has suggested and constructed a few spillway structures which for want of a better name might be called baffle-wall spillways. They are not included in his report previously referred to.

They involve probably the minimum amount of "foreign" material. They consist of a series of vertical concrete walls at intervals of 20 feet or so down a long excavated channel. Each one is two feet lower than the one above, although any other difference in elevation may be used. See Figure 3.

Assume it is desired to pass, say, 1000 c.f.s. over these weirs with an H of four feet. The drop over each weir is two feet. This is a 50% submergence. The free-flowing coefficient is applicable.

From Table 51, King's Handbook, it appears that if they are one foot thick on the top the proper coefficient is 3.3. The formula would be

$$Q = 3.3\ LH^{3/2}, \text{ then } L = 38 \text{ feet.}$$

The notch would probably be made trapazoidal as shown rather than rectangular.

The intervening earth channel section for a three foot per second velocity would have the dimensions previously described namely a water depth of 5.5 feet, a bottom width of 50 feet, and

* From Lane, E. W.; Security from Under-seepage Masonry Dams on Earth Foundations; Proceedings American Society of Civil Engineers, Sept. 1934, p. 951.

side slopes of 2:1. Each weir crest would then extend 1.5 feet above the bottom of the upstream channel and 3.5 feet above the downstream channel floor. The 38 foot crest would extend to within six feet of the side walls of the earth excavation. It is desirable to remove any tailwater turbulence well away from the earth side walls. Paving is shown on the downstream side of each wall as an added protection.

Other Types of Structures

Description of other types of structures may be found in the literature on the subject. These hydraulic principles will be found applicable to most of them.

(8) Comments and Criticisms

Mr. M. R. Lewis, Irrigation Engineer, Oregon State College, Corvallis, Oregon.*

> "Your discussion of the method of determining the capacity of an open channel seems to be correct. However, personally, I am very skeptical of the value of these results in the case of the very flat coulees of the Dakotas. On those coulees, the determination of the slope by means of elevations taken in the dry bed of the coulee over a distance of a few hundred feet seems, to me, a very doubtful procedure. With a slope of one foot per mile which, I understand, is not uncommon, an error of judgment in determining the correct points to use on a 1000-foot stretch resulting in an elevation one inch off at each end might mean an error of almost 100 per cent in the slope.
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> I suppose that it is desirable to excavate the channel above the spillway large enough so there will be no erosion. However, I am inclined to think that any erosion which might take place would not be dangerous. So long as the channel is large enough to take care of the required run-off, I should be inclined, where the earth excavated must be wasted, to let nature do some excavating on its own. In such cases the side of the spillway toward the dam should be protected from erosion and the wing-walls at the upper end of the spillway structure should extend far enough into the bank to be sure that they will not be cut out by erosion.
>
> \- - - - - - - - -
>
> In considering the water shed necessary to maintain a constant level in a reservoir, would it not be better to consider the area of water surface and the area of the soil surface in contact with the water of the reservoir instead of considering the volume of water in the reservoir. Losses from the reservoir are due to evaporation and seepage, and both of these are more closely related to surface area than to volume. I suppose that the sanitary conditions in the recreational pools during periods when there is no flow through the pools are related more closely to the volume of water per unit of use than they are to the surface area of the pool. To that extent, the question of volume is an important one. Otherwise, I am inclined to think that a deep reservoir of small area would be better than a shallow reservoir of larger storage capacity.

*Received Nov. 18, 1934

I think that your paper is a real contribution in the field of construction of small dams and I trust that you have it printed soon. When you do so, I would very much appreciate having several copies, if they can be spared."

Mr. Waldo E. Smith, Assistant Professor Civil Engineering, North Dakota State College, Fargo, North Dakota.*

"I congratulate you for producing a paper which I believe is a very definite step forward in the matter of handling problems of this nature which seem to be especially troublesome to the engineers that have been working with them. Your paper, I believe, should do much to clarify their difficulties. It is put in definite form for the first time, I believe, the principles underlying such cases and it really covers the problem with a considerable degree of thoroughness."

Mr. H. M. Hill, Engineer, U. S. Engineer Office, St. Paul, Minnesota.**

"Computation of flow should be supplemented by appropriate flood flow control, if any available, and if drainage area is known—in any event, allow liberal percentage in addition."

Mr. G. O. Guesmer, Associate Engineer, U. S. Engineer Office, St. Paul, Minnesota**

a. "Under 'Probable Slope of Water Surface' it might be noted that the high water slope is practically that of the low water slope with the sharp breaks and fluctuations of low water ironed out into a smooth line.

b. 'Observations of Channel Roughness.' Several pictures from different angles and positions generally tell the story better than one picture, less likely to have a picture failure spoil the story.

c. With reference to coefficients for submergence it might be noted that model tests give close coefficients which may be applied to prototype (Addison's Applied Hydraulics—John Wiley & Sons). It might be possible to build some small models and get answers to the problem of submergence coefficients in that way."

Mr. Chas. L. Batchelder, District Engineer, U.S.G.S., St. Paul, Minnesota, made certain corrections in the run-off data.

Mr. J. R. Van Dyke, Engineer in Bismarck Office, F.E.R.A., pointed out features that might be confusing to one not previously familiar with the procedure.

Mr. Arvid Backlund, Engineer in Bismarck Office, F.E.R.A., made the interesting remark, "You would design a dam like you would a bridge."

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*Received Nov. 15, 1934
**Received Jan. 12, 1935

Mr. L. C. Tschudy, Supervising Engineer, Emergency Conservation Work, Milwaukee, Wisconsin.***

"I have read your Biennial Report while in Bismarck to the Governor of North Dakota, covering water conservation. Although I have not had sufficient time to analyze it as I would like, I have given it as close study as was possible in the limited time.

One of the main reasons for not using reinforced concrete is because there is an abundance of so called 'nigger head' rock in North Dakota which lends itself to economical rubble masonry construction. Another reason is the scarcity of available gravel pits, which makes rubble masonry fit more into our program. Another reason is that two and one-half sacks of cement is required per cubic yard of rubble masonry and at the same time no purchase of gravel, steel or form lumber is necessary. It has been felt that with these arguments for rubble masonry as compared to the material costs of reinforced concrete, the former is the most economical for permanent structures in North Dakota. Irrigation has not been authorized under the E.C.W. Water Conservation Program and for this reason we cannot construct irrigation projects.

The report states that many of these dams or their location have no legal status. Would it not be a good point to make recommendations to pass legislation to protect the State from legal action for these dams constructed by U. S. Forest Service?

Regarding the 'channel flow capacities' to be used as maximum spillway capacities, I think this can be used to advantage. The man directing a survey party might however get certain channel measurements slightly in error that might result in considerable error in maximum flow. With an overflow dam such as rubble masonry or timber crib, complete submergence should not endanger the structure; but it is necessary to have material around the wing walls such as gravel or rip rap that will not be moved by the velocities of water passing over it when submerged. I have a feeling many of the camp men would have some difficulty in figuring the hydraulics of maximum channel capacities. In the example you work out my curve for fast run-off is in excess of your designed flow by about 400 S.F. if I remember correctly. On page No. 2 of our bulletin, point No. 3, this was intended to be used as maximum flow to check the curves, although the bulletin does not explain this in detail.

On non-overflow earth dams, I feel we must design for maximum run-off. This can and is observed by the freeboard. In my experience with soil erosion, I would feel an earth dam would be in danger, if it were submerged.

I think your point of designing for maximum channel capacities in over-flow dams is an excellent way to check our curves and it should be useful in designs."

***Received Feb'y. 12, 1935

Reply to Comments

Mr. Lewis questions the accuracy of the method to determine the probable slope of the water surface from a dry coulee bed. He mentions an error of 100%. That would make the answer 50% too large, which, if on the safe side, is not too much of a factor of safety. If on the other side, which is unlikely, there is still a chance of balancing it with the selection of a liberal coefficient of roughness, n.

The regions of flat gradient stream beds in North Dakota are in the old Lake Agassiz area of the Red River, the Mouse River channel and the James and Sheyenne Rivers. A study was made of the Mouse River channel near Towner, North Dakota, in 1934, and at Minot, North Dakota, in 1927. Discharge measurements were available, in each instance. The slope of a ten mile stretch near Towner was .00003 for both low water surface and bottom grade. Five cross sections were taken. The area curves were very similar. Ceofficient of n for low stage was about .025 which seems rather small. The channel was a smooth clay mud without much debris. The Minot studies were for flood stages. The channel was obstructed by trees and brush in its upper section. Coefficient of n was .045 to .050. This appears reasonable.

After a conference with Mr. Tschudy he wrote his letter and I wrote my answer in the Foreword.

I appreciate the courtesy these men have extended to me to take the time and interest from their own work necessary to read and criticize this paper.

N. B. Length of Apron (Alternate Formula)

A last minute suggestion for the length of the apron of a small dam has been obtained from reading a paper in the February issue of the Proceedings of the Am. Soc. C. E. by Bakhmeteff and Matzke entitled "The Hydraulic Jump in Terms of Dynamic Similarity" and the discussions in following issues. A much simpler and in general a more accurate formula than the one quoted, unless varying coefficients are applied to it, is this one

$$L = 5\ D_2$$

RUN-OFF DATA FOR NORTH DAKOTA STREAMS

In computing the capacity of a channel it is usually desirable to know the probable amount the watershed can yield in order to know roughly how much a small dam will be submerged when it occurs. The absolute maximum is a large and indefinite figure beyond the scope of these studies. The feasible maximum is also very indefinite and depends largely upon the individual appraisal of the particular conditions involved.

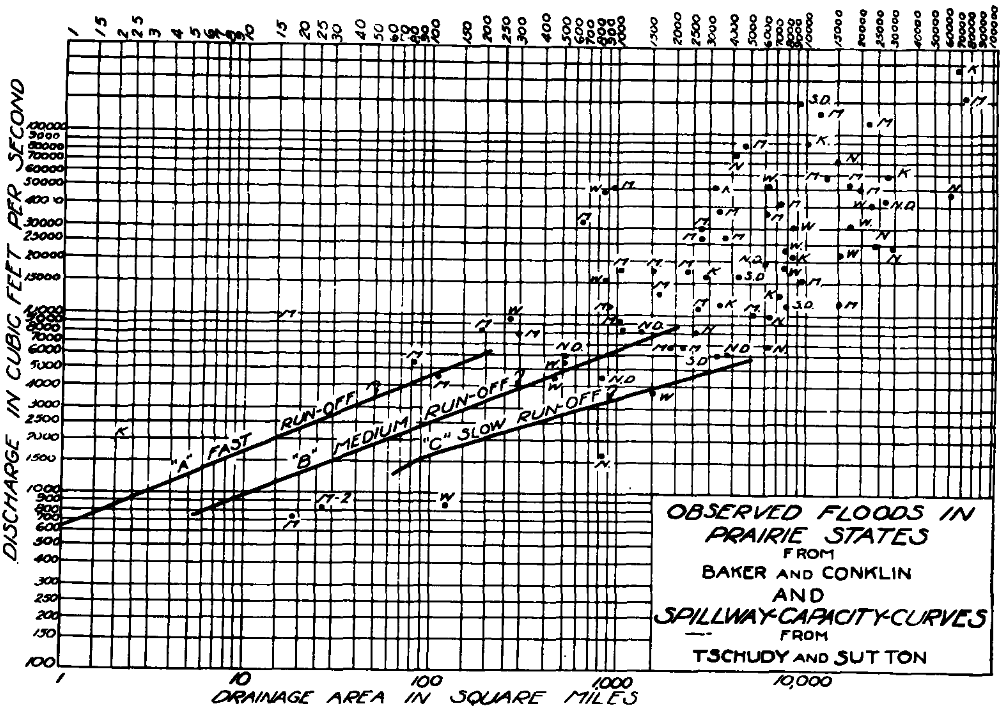

FIGURE 4.

Various charts and formulas have been suggested which are more or less inconsistent. Figure 4 shows the high rates of run-off recorded on the principal streams in Montana, Wyoming, North and South Dakota, Nebraska and Kansas. It was prepared by Mr. M. R. Lewis of the Bureau of Agricultural Engineering who is now consultant for the Federal Emergency Relief Administration of North Dakota.

Average Annual Water Supply

Information is frequently needed as to the probable average annual amount of water available at some reservoir site where no stream flow records exist. In such a situation a common practice applicable to North Dakota streams is to assume that the average annual run-off as recorded at a gaging station on some other part of the watershed or on an adjacent watershed tends over a period of time to become a uniform contribution from each section of land. It can therefore be applied directly to the watershed in question. This is of course only a rough approximation depending upon whatever similarity there may be in rainfall, topography, soil texture, soil cover, and other factors too numerous to mention. It is a short cut method at best and mainly justified by the fact that no other means are immediately so convenient and available.

Ratio of Storage Capacity to Watershed

Table II, gives the run-off data for the principal streams in North Dakota.

The size of the reservoir feasible for a given watershed depends entirely upon the purpose for which the water is to be used. If the dam is to create a stock water pond or to store whatever water is available regardless of amount the limiting consideration is its cost per acre-foot. The Civilian Conservation Corps dams built under the President's Emergency Conservation Work are intended not to exceed $150.00 an acre-foot.

A reservoir holding two or three times the mean annual run-off will have large fluctuation of its water surface.

On the other hand if the dam is to create a recreational pool where boating, swimming, and fishing are desired as permanent attractions, a fairly stable water surface elevation is desirable. Otherwise bath houses, boat houses, and summer cottages built at the water's edge during wet years will be an unsightly and inconvenient distance away when the dry hot years return. Also fish planted in deep water will die when the pool becomes shallow and stagnant.

A recreational reservoir should be so small in relation to the mean annual run-off that even small summer flows during dry years will be enough to keep the water surface up to near the crest of the dam and to flush out the pool occasionally for sanitary reasons.

As a general rule applicable to ponds not extensively spring-fed a reasonably permanent water surface elevation can be expected only when the storage capacity amounts to about one percent or less of the average annual run-off at the damsite.

Minimum Water Depth

An ice thickness of 30 inches such as occurs at Bismarck, North Dakota, on the average would cut a seven foot water depth down to 4½ feet. The volume of water remaining under such an ice cover would have to be large to sustain many fish for the four winter months. For a fish pond seven feet is frequently suggested as a minimum depth.

TABLE II—RUN-OFF DATA FOR NORTH DAKOTA RIVERS

The data for this table are taken largely from the summary of North Dakota stream records published in the Twelfth Biennial Report of the State Engineer, 1924-1926, and prepared by E. F. Chandler, University of North Dakota, Grand Forks, North Dakota. He filled in many of the missing winter months which will account for some discrepancies between this record and that of the U.S.G.S.

Conversion factors:

$$\text{Ac-ft. per sq. mi.} = \frac{\text{Total annual acre-feet}}{\text{Watershed in square miles}}$$

$$\text{Inches run-off} = \text{Ac-ft. per sq. mi.} \times 0.01875$$

$$\text{Ave. annual cu. ft. per sec.} = \frac{\text{Total annual acre-feet}}{724}$$

TABLE II—RUN-OFF DATA NORTH DAKOTA RIVERS (Cont'd)

River	Cannonball	Grand	Heart
Gaging Station	Timmer, N. Dak.	Haley, N. Dak.	Richardton, N. Dak.
Watershed	3,650 square miles	500 square miles	1,250 square miles
Years of Record	1904-8; '12-18; '22-34 (25 yrs.)	1908-1920 (13 yrs.)	1903-23 (21 yrs.)
Average Annual Precipitation	15.7 inches	14.8 inches	15.7 inches
Mean Annual Flow	114,500 acre-feet 31.4 ac.ft. per sq. mile 0.54 inches 158 cu.ft. per second	16,400 acre-feet 32.9 ac.ft. per sq.mile 0.62 inches 23 cu.ft. per sec.	61,100 acre-feet 48.9 ac.ft. per sq.mile 0.92 inches 84 cu.ft. per sec.
Minimum Annual Flow	Year 1934 2,765 acre-feet 0.8 ac.ft. per sq.mile 0.01 inches 4 cu.ft. per second	Year 1911 860 acre-feet 1.7 ac.ft. per sq.mi. 0.03 inches 1 cu.ft. per sec.	Year 1921 10,700 acre-feet 8.6 ac.ft. per sq.mi. 0.16 inches 15 cu. ft. per sec.
Maximum Annual Flow	Year 1923 243,000 acre-feet 66.6 ac.ft. per sq.mile 1.25 inches 336 cu.ft. per second	Year 1920 34,600 acre-feet 69.2 ac.ft. per sq.mile 1.30 inches 48 cu.ft. per second	Year 1912 140,800 acre-feet 112.6 ac.ft. per sq.mile 2.11 inches 194 cu.ft. per second
Maximum Recorded Flow	June 10, 1932 8,600 cu.ft. per second 2.4 c.f.s. per sq.mi.	June 21, 1914 5,650 cu.ft. per second 11.3 c.f.s. per sq.mi.	June 7, 1906 8,000 cu.ft. per second 6.4 c.f.s. per sq.mi.
Use	Sewage dilution Irrigating small projects Recreation Stock water	Stock water Large potential irrigation project	Sewage dilution Recreation Irrigating small projects Large potential irrigation project

TABLE II—RUN-OFF DATA NORTH DAKOTA RIVERS (Cont'd)

River	Heart	James	Knife
Gaging Station	Mandan, N. Dak.	Jamestown, N. Dak.	Broncho, N. Dak.
Watershed	3,320 square miles	3,140 square miles	1,200 square miles
Years of Record	1924-25; '28-32 (7 yrs.)	1929-33 (5 yrs.)	1904-19; '22-25 (20 yrs.)
Average Annual Precipitation	15.8 inches	17.9 inches	15.7 inches
Mean Annual Flow	83,300 acre-feet 25.1 ac.ft. per sq.mi. 0.47 inches 115 cu.ft. per sec.	17,200 acre-feet 5.5 ac.ft. per sq.mi. 0.10 inches 24 cu.ft. per sec.	66,200 acre-feet 55.2 ac.ft. per sq.mi. 1.04 inches 91 cu.ft. per sec.
Minimum Annual Flow	Year 1931 31,700 acre-feet 9.6 ac.ft. per sq.mi. 0.18 inches 44 cu.ft. per sec.	Year 1931 3,570 acre-feet 1.1 ac.ft. per sq.mi. 0.02 inches 5 cu.ft. per sec.	Year 1905 14,700 acre-feet 12.2 ac.ft. per sq.mi. 0.23 inches 20 cu.ft. per sec.
Maximum Annual Flow	Year 1929 132,000 acre-feet 39.8 ac.ft. per sq.mi. 0.74 inches 182 cu.ft. per sec.	Year 1930 37,700 acre-feet 12.0 ac.ft. per sq.mi. 0.22 inches 52 cu.ft. per sec.	Year 1912 145,700 acre-feet 121.4 ac.ft. per sq.mi. 2.28 inches 200 cu.ft. per sec.
Maximum Recorded Flow	Feb. 28, 1932 3,400 cu.ft. per sec. 1.0 c.f.s. per sq.mi.	Mar. 14, 1929 1,100 cu.ft. per sec. 0.4 c.f.s. per sq.mi.	June 26, 1914 7,700 cu.ft. per sec. 6.4 c.f.s. per sq.mi.
Use	Sewage dilution Recreation Irrigating small projects Stock water	Sewage dilution Recreation Stock water Migratory game refuges	Stock water Irrigating small projects Recreation

TABLE II—RUN-OFF DATA NORTH DAKOTA RIVERS (Cont'd)

River	Knife	Little Missouri	Little Muddy
Gaging Station	Hazen, N. Dak.	Medora, N. Dak.	Williston, N. Dak.
Watershed	2,180 sq. mi.	6,190 sq. mi.	1,100 sq. mi.
Years of Record	1929-32 (4 yrs.)	1903-8; '23-25; '29-32 (13 yrs.)	1904-8; '33 (6 yrs.)
Average Annual Precipitation	15.9 inches	15.4 inches	15.7 inches
Mean Annual Flow	58,800 acre-feet 27.0 ac.ft. per sq.mi. 0.51 inches 81 cu.ft. per sec.	422,500 acre-feet 68.3 ac.ft. per sq.mi. 1.28 inches 584 cu.ft. per sec.	39,600 acre-feet 36.0 ac.ft. per sq.mi. 0.67 inches 55 cu.ft. per sec.
Minimum Annual Flow	Year 1931 26,800 acre-feet 12.3 ac.ft. per sq.mi. 0.23 inches 37 cu.ft. per sec.	Year 1931 79,800 acre-feet 12.9 ac.ft. per sq.mi. 0.24 inches 109 cu.ft. per sec.	Year 1905 11,000 acre-feet 10.0 ac.ft. per sq.mi. 0.19 inches 15 cu. ft. per sec.
Maximum Annual Flow	Year 1930 130,120 acre-feet 59.8 ac.ft. per sq. mi. 1.12 inches 180 cu.ft. per sec.	Year 1929 958,000 acre-feet 154.7 ac.ft. per sq.mi. 2.90 inches 1,322 cu.ft. per sec.	Year 1907 53,800 acre-feet 48.9 ac.ft. per sq.mi. 0.92 inches 74 cu.ft. per sec.
Maximum Recorded Flow	Feb. 21, 1930 3,070 cu.ft. per sec. 1.4 c.f.s. per square mile	June 7, 1929 38,700 cu.ft. per sec. 6.3 c.f.s. per sq.mi.	April 11, 1904 2,990 cu.ft. per sec. 2.7 c.f.s. per sq.mi.
Use	Industrial use Sewage dilution Recreation Stock water Large potential irrigation project	Irrigating small projects Stock water Recreation Sewage dilution Large potential irrigation project	Stock water Recreation Irrigating small projects

TABLE II—RUN-OFF DATA NORTH DAKOTA RIVERS (Cont'd)

River	Missouri	Missouri	Mouse
Gaging Station	Bismarck, N. Dak.	Williston, N. Dak.	Minot, N. Dak.
Watershed	186,400 square miles	164,530 square miles	10,270 square miles
Years of Record	1905; '28-34 (8 yrs.)	1905-06; '29-34 (8 yrs.)	1903-33 (31 yrs.)
Average Annual Precipitation	14.8 inches	14.8 inches	16.8 inches
Mean Annual Flow	21,000,000 acre-feet 113.0 ac.ft. per sq.mi. 2.11 inches 29,000 cu.ft. per sec.*	13,460,000 acre-feet 81.8 ac.ft. per sq.mi. 1.53 inches 18,600 cu.ft. per sec.	119,200 acre-feet 11.6 ac.ft. per sq.mi. 0.22 inches 165 cu.ft. per sec.
Minimum Annual Flow	Year 1931 9,420,000 acre-feet 50.6 ac.ft. per sq.mi. 0.95 inches 13,000 cu.ft. per sec.	Year 1931 9,620,000 ac.ft. 58.4 ac.ft. per sq.mi. 1.10 inches 13,300 cu.ft. per sec.	Year 1931 940 ac.ft. 0.1 ac.ft. per sq.mi. .002 inches 1 cu.ft. per sec.
Maximum Annual Flow	Year 1929 18,600,000 acre-feet 99.8 ac.ft. per sq.mi. 1.87 inches 25,700 cu.ft. per sec.	Year 1929 16,900,000 acre-feet 102.7 ac.ft. per sq.mi. 1.92 inches 23,150 cu.ft. per sec.	Year 1904 678,100 acre-feet 66.0 ac.ft. per sq.mi. 1.24 inches 936 cu.ft. per sec.
Maximum Recorded Flow	March 3, 1928** 201,000 cu.ft. per sec. 1.1 c.f.s. per sq.mi.	April 4, 1930 231,000 cu.ft. per sec. 1.4 c.f.s. per sq.mi.	April 20, 1904 12,000 cu.ft. per sec. 1.2 c.f.s. per sq.mi.
Use	Industrial use Sewage dilution Irrigating small projects stock water Navigation and power Large potential irrigation projects		Industrial use Sewage dilution Irrigation Migratory game refuges Stock water

*Based upon rough extension of early rating curve with gage heights from 1893 to 1927 and discharge records 1928-34

**Maximum estimated discharge equals 310,000 c.f.s. June 19, 1908

TABLE II—RUN-OFF DATA NORTH DAKOTA RIVERS (Cont'd)

River	Pembina	Red	Red
Gaging Station	Neche, N. Dak.	Fargo, N. Dak.	Grand Forks, N. Dak.
Watershed	2,960 square miles	6,420 square miles	25,480 square miles
Years of Record	1903-15; '19-33 (28 yrs.)	1901-34 (34 yrs.)	1882-34 (53 yrs.)
Average Annual Precipitation	18.4 inches	22.6 inches	21.3 inches
Mean Annual Flow	106,400 acre-feet 36.0 ac.ft. per sq.mi. 0.67 inches 147 cu.ft. per sec.	336,000 acre-feet 52.4 ac.ft. per sq.mi. 0.98 inches 464 cu.ft. per sec.	1,690,000 acre-feet 66.3 ac.ft. per sq.mi. 1.24 inches 2,315 cu. ft. per sec.
Minimum Annual Flow	Year 1931 42,000 acre-feet 14.2 ac.ft. per sq.mi. 0.27 inches 57 cu.ft. per sec.	Year 1934 12,660 acre-feet 2.0 ac.ft. per sq.mi. 0.04 inches 18 cu.ft. per sec.	Year 1934 177,190 acre-feet 7.0 ac.ft. per sq.mi. 1.30 inches 245 cu.ft. per sec.
Maximum Annual Flow	Year 1904 481,600 acre-feet 162.5 ac.ft. per sq.mi. 3.05 inches 664 cu.ft. per sec.	Year 1916 1,368,000 acre-feet 213.0 ac.ft. per sq.mi. 4.00 inches 1,890 cu.ft. per sec.	Year 1897 4,066,000 acre-feet 159.5 ac.ft. per sq.mi. 3.00 inches 5,620 cu.ft. per sec.
Maximum Recorded Flow	May 2, 1904 3,870 cu.ft. per sec. 1.3 c.f.s. per sq.mi.	July 11, 1916 7,740 cu.ft. per sec. 1.2 c.f.s. per sq.mi.	April 10, 1897 43,000 cu.ft. per sec. 1.7 c.f.s. per sq.mi.
Use	Municipal supply Sewage dilution Recreation Stock water		Municipal supply Sewage dilution Industrial use Recreation Stock water

TABLE II—RUN-OFF DATA NORTH DAKOTA RIVERS (Cont'd)

River	Sheyenne	Sheyenne
Gaging Station	Sheyenne, N. Dak.	West Fargo, N. Dak.
Watershed	1,300 square miles	5,420 square miles
Years of Record	1930-33 (4 yrs.)	1902-7; '19; '30-33 (11 yrs.)
Average Annual Precipitation	16.3 inches	19.1 inches
Mean Annual Flow	12,900 acre-feet 10 ac.ft. per sq.mi. 0.19 inches 18 cu.ft. per sec.	123,000 acre-feet 22.7 ac.ft. per sq.mi. 0.43 inches 170 cu.ft. per sec.
Minimum Annual Flow	Year 1931 1,860 acre-feet 1.4 ac.ft. per sq.mi. 0.03 inches 3 cu.ft. per sec.	Year 1931 46,000 acre-feet 8.5 ac.ft. per sq.mi. 0.16 inches 64 cu.ft. per sec.
Maximum Annual Flow	Year 1930 32,100 acre-feet 25 ac.ft. per sq.mi. 0.46 inches 44 cu.ft. per sec.	Year 1904 233,000 acre-feet 43.0 ac.ft. per sq.mi. 0.81 inches 322 cu.ft. per sec.
Maximum Recorded Flow	February 24, 1930 990 cu.ft. per sec. 0.8 c.f.s. per sq. mi.	April 10, 1902 2,030 cu.ft. per sec. 0.4 c.f.s. per sq.mi.
Use	Sewage dilution Recreation Subsoil supply for municipal wells Stock water	Municipal supply Sewage dilution Industrial power Recreation Stock water

FLOOD IRRIGATION ALONG THE MOUSE RIVER

By WALDO E. SMITH

Asst. Prof. Civil Engineering

North Dakota State College

Fargo, North Dakota

General Statement

The idea of controlling and artificially securing the flooding of the hay meadows along the Mouse River in McHenry and Bottineau Counties has long been agitated by residents of the region. They have observed that in those years that the spring floods due to melting snows or to rains were large enough to inundate the meadows of the flood plain of the river, they uniformly obtained good yields of hay, regardless of lack of subsequent rainfall. Topographically speaking the situation is quite ideal; there is a broad, flat flood plain along a river whose gradient is unusually flat, making it possible to flood artificially extensive areas by means of properly located low dikes. Geologically, the site is quite satisfactory inasmuch as the underlying materials and the surface soils are high in their clay content, thus providing suitable footing conditions for the small dams required, as well as excellent materials from which to form dikes. Soil of this sort, too, will retain any moisture that it receives much longer than sandy or gravelly soils. Agriculturally, then, the advantage can be seen for flooding the land for a perennial and inexpensive crop such as hay.

To facilitate such projects and prescribe procedure, the Regular Session of the Legislature in 1919 enacted a law (8320 a1 to a24, 1925 Supplement to the Compiled laws of North Dakota). No specific project had been undertaken under this law, however, until in 1933. Early in that year some of the farmers and land owners of the vicinity of Towner requested the State Engineer to study the possibilities of artificial control of the floods of the Mouse River for the purpose of flooding regularly the hay meadows once each year. With few field studies, but utilizing available maps and other data on this region, the study was made; and a preliminary report was issued under date of April 10, 1933, on the project known as the Eaton Flood Irrigation project. Land owners involved petitioned the Board of County Commissioners for the establishment of the McHenry County Board of Flood Irrigation and for the establishment of a flood irrigation district, as provided by law. The Board appointed consists of L. G. Hardie, Chairman; R. M. Lunday, J. E. Westford, A. G. Anderson, and J. C. Eaton, Secretary. The State Engineer was appointed the engineer for the project and by funds made available for the payment of salaries through the Civil Works Administration and the Federal Emergency Relief Administration, the surveys for this project were carried on during the winter of 1933-34. Before these surveys were complete, petitions for two other flood irrigation projects downstream were received by the Board, and the Board, in turn, designated the State Engineer as the engineer for these projects. The field parties proceeded with the surveys for these as the Eaton Flood irrigation project surveys were completed. These two projects are respectively designated as the Westford Flood Irrigation Project and the Hardie Flood Irrigation project, and will be discussed in turn later. A brief preliminary report was prepared on the Westford project under date of December 13, 1933, but none has been prepared on the Hardie project.

The Eaton Flood Irrigation Project

During the spring and summer months designs, detailed plans, specifications and other work pertinent to the Eaton Flood Irrigation project was done. The employment of men for the office work, including the writer, was financed by funds made available by the F.E.R.A. by an arrangement similar to that for the expense of the field work. The Board applied to the Federal Emergency Administration of Public Works for a loan and grant to construct the project. The necessary engineering work, including the preparation of the general layout sheet of the project, the designs for the main diversion dam and estimate of cost on the basis of completed designs, were submitted to the State Office of the Federal Emergency Administration of Public Works to supplement and complete the application for funds made by the Board.

The detailed plans, specifications, and the second report which gives data on the benefitted district, and other work necessary and prescribed by law were completed by mid-summer, and a hearing was held by the Board in Towner on August 6, 1934, for the purpose of hearing objections to the project and the assessing of damages, as required by law. The second hearing as required by law was held two weeks later, to hear objections to the damage assessed. As engineer for the project, the State Engineer attended the first of these. It appears that the legal requirements of the State Law have been fulfilled and that advertisement for bids on the work can be published if and as soon as the loan and grant for the project are approved.

The project (See Figure 5) has its extreme upper (southern) end in Section 23, T. 155 N., R. 77 West, near the point on the Mouse River designated by the United States Geological Survey as Mile 235. The main dam is located in Section 6, T. 155 N., R. 76 W. The project extends downstream, having its extreme north end in Section 19, T. 157 N., R. 75 W., and Section 24, T. 157 N., R. 76 W., near Mile 267 on the river. The overall air-line length is thus about 14 miles. In addition to the main diversion dam which will make it possible to raise the water about 12 feet, there are

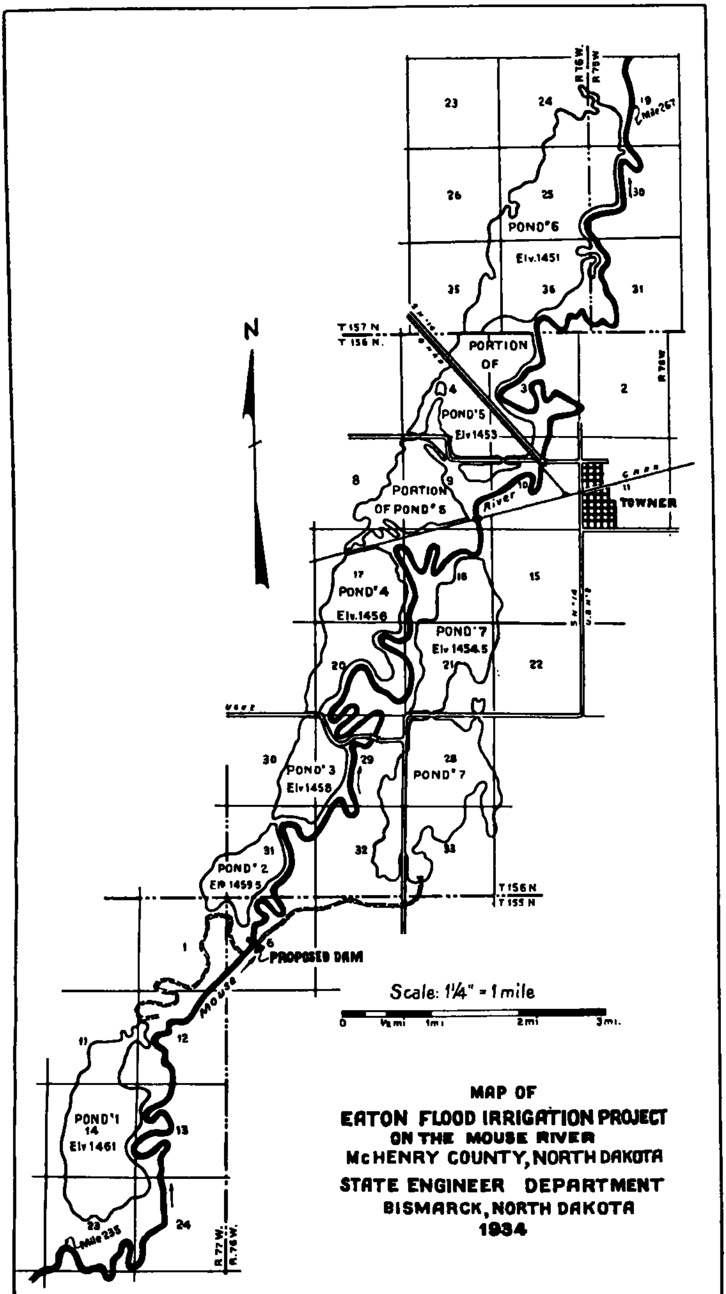
N
T 157 N
T 156 N.
PORTION OF
POND'5
Elv 1453
POND'6
Elv.1451
PORTION OF POND'5
TOWNER
River
POND'4
Elv.1456
POND'7
Elv 1454.5
POND'7
POND'3
Elv 1458
POND'2
Elv 1459.5
T 156 N
T 155 N
PROPOSED DAM
Mouse
POND'1
Elv 1461
Scale: 1¼" = 1 mile
0 ½ mi 1 mi 2 mi 3 mi.
MAP OF
EATON FLOOD IRRIGATION PROJECT
ON THE MOUSE RIVER
McHENRY COUNTY, NORTH DAKOTA
STATE ENGINEER DEPARTMENT
BISMARCK, NORTH DAKOTA
1934

FIGURE 5.

to be 12 timber head gates and waste gates to facilitate movement of the water into and out of the seven ponds involved. In addition, there are about 60,000 cubic yards of excavation in the necessary dikes and ditches. The total construction and engineering cost under present market conditions will be fifty thousand dollars, in round numbers. Direct benefits will occur on about eight thousand seven hundred acres of land.

Of the seven ponds referred to, one is on the east bank of the river, the other six on the west bank. The levels of the ponds have been determined to conform to the configuration of the ground, so that the level of the particular pond is not in general above the level of the natural levee that the normal but not annual inundations have built up close to the river. Thus, on the west bank of the river, each pond, in turn, is from one and a half to three feet below the one preceding. The plan of operation will be briefly as follows. As the spring flood comes on, the gates of the main dam will be closed, causing the water to rise, overflow the lands upstream from the dam immediately adjacent to the river, and also flow through the diversion outlets provided for flooding the rest of the ponds. At the outset, all waste gates will be closed, and all head gates opened. When the pond situated at the lowest elevation is filled, the head gate of that pond will be closed. Then when the next pond is filled, its head gate in turn will be closed, and so on, until all of the ponds are filled, which process will normally require about five days. Then the gates of the main dam will be opened, permitting the normal flow of the river to pass. The various ponds may be drained directly into the river as desired. It is anticipated that the normal period during which the pond will be retained will be from three to six weeks, but this can probably be best determined by trial.

Inasmuch as the pond levels are not above the level of the occasional, but not annual flood, no farm yard site and no farm buildings will be submerged by the artificial flooding, on this project or either of the other two.

The Westford Flood Irrigation Project

The field studies for this project were carried substantially to the same degree of completion as those of the Eaton project. On the basis of the preliminary report, an application for a P.W.A. loan and grant was made by the Board. However, the Eaton project was given priority in the matter of office and design work, and for this reason, the required documents to support this application were not available until late in the summer of 1934. At that time, they were filed with the State Office of the P.W.A. in Devils Lake, as required.

Detailed designs, plans and specifications for this project have not been made as yet. Inasmuch as the work involved in those is considerable, it seems desirable to see what steps will definitely be taken with respect to the Eaton project before further work is done on this. As most economically worked out, this project should extend into Bottineau County, but inasmuch as the State Law requires administration county by county, the county line has been found to be an obstacle. If the project is constructed along the lines now suggested, in all probability, this addition in Bottineau County, known as the Moen addition, will not be included. With the Moen addition, the total engineering and construction cost is estimated to be about $78,500 benefitting 11,950 acres of land. Without this addition, the cost is to be $74,300 and the benefitted area, 9410 acres. The complete project (with the Moen addition) extends from section 24 and 26, T. 159, N., R. 77 W. to Sections 20 and 30, T. 160 N., R. 78 West, an extreme length of about 12 miles (from mile 306 to mile 332 on the river). It also contemplates the construction necessary to create seven ponds at different elevations with the same general arrangement as that of the Eaton project. The plan of operation will also be that of the Eaton project, in regard to the filling and draining of ponds. In the case of this project it appears that further field and office studies may effect some saving in the cost.

The Hardie Flood Irrigation Project

Though the field studies for the Hardie project have ostensibly been carried to completion, the office studies have not been begun. The Board has taken no steps toward its financing by filing application for P.W.A. funds. The only step they have taken is that of designating the State Engineer as the engineer for the project, and requesting that he proceed with studies as rapidly as possible. Inasmuch as the engineers who have been engaged on this work with F.E.R.A. funds are all now otherwise engaged, the prospects of early office studies of this project are remote.

The general plan, only, has been formulated, no estimate of cost and no estimate of benefitted acreage are as yet available. The project extends from Section 18, T. 157 N., R. 75 W., to Section 26, T. 159 N., R. 77 West (River miles 269 to 307), an extreme length of about twelve miles, filling quite completely the gap between the Eaton and the Westford projects. It involves the work necessary to create eight ponds, all but one of which will be on the west side of the river. The general arrangement of ponds and the plan of operation will be similar to those outlined in connection with the Eaton project.

Agricultural Products Proposed

The desire of the present promoters is to obtain an increased yield of the native prairie grass. Various claims are made for the probable amount of increase. Doubling of the yield is most frequently suggested as probable.

The average yield now obtained in McHenry County according to the State Commissioner of Agriculture is about a ton per acre. The price ranges from $3.00 to $10.00 a ton.

The following is the hay production in McHenry County:

Year	Tons	Acres	Tons per acre	Average Annual Precipitation Inches
1931	30,191	60,260	0.5	11.7
1930	45,984	55,520	0.8	12.9
1929	47,469	51,649	0.9	12.6
1928	25,002	22,689	1.1	17.8
1927	19,007	20,147	0.9	20.8

THE LOWER YELLOWSTONE RECLAMATION PROJECT

Interest in irrigation has been greatly enhanced by the present drought in western North Dakota. Attention is being focused upon the advantages of irrigation over dry-farming. The lower Yellowstone Reclamation Project is a federal irrigation project of 59,000 acres located on both sides of the North Dakota-Montana state boundary. About 20,000 acres are located in North Dakota. It has been operated by the local water users since 1932.

Crop Yield Statistics

Table III, furnished by Mr. Axel Persson, Project Manager, Sidney, Montana gives the reported crop yield and value for irrigable lands for the year 1933. These are the official returns reported to the authorities for the purpose of estimating the annual repayment of construction costs. In 1925 a contract was entered into with the government whereby the landowners were to repay construction costs on the basis of 5% of the average annual gross income per acre during the preceding 10-year period.

Local real estate agents and others interested in immigration to the project were consulted October 17, and 18, 1934. They were free to intimate that because of this contract the returns would tend to be conservative. Two of the men were therefore asked to give their estimate of the average crop yields under irrigation.

One man, not a landowner, made the following statement:

Alfalfa ....3 tons per acre
Sugar beets ....12-13 tons per acre
Oats ....75 bu. per acre
Wheat ....35 bu. per acre
Potatoes ....300 bu. per acre

Another man, a landowner and real estate agent, offered the following figures:

Alfalfa	2 tons per acre
Sugar beets	14 tons per acre (17% sugar)
Oats	50 bu. per acre
Wheat	35 bu. per acre
Potatoes	150 bu. per acre

The 1933 return may or may not be up to the average. But from Table III it may be noted:

	Irrigated	Dry-farmed
Alfalfa	1.3 tons per acre	0.7 tons per acre
Sugar beets	13.04 tons per acre	
Oats	32.9 bu. per acre	11.0 bu. per acre
Wheat	19.5 bu. per acre	6.7 bu. per acre
Potatoes	139.7 bu. per acre	62.5 bu. per acre

Greater Crop Yields Cost More Money

The cost of water is not the only difference between irrigated and dry-farmed crops. The 1930 U. S. census of Irrigation in North Dakota states that preparing the land for irrigation cost $14.88 an acre for about 22,000 acres reported. Part of this would be as necessary for dry-farming as for irrigation. But the expense for leveling and grading rough areas, building laterals and turn-out structures is chargeable only to irrigation.

Local government taxes are higher. An average dry-farm acre in western North Dakota is taxed about 15 cents. On the North Dakota side of the Yellowstone project an average acre is charged about 60 cents to pay for schools and sheriffs and the other accessories of civilization.

Basic costs are therefore approximately as follows:

Construction costs	$1.50 an acre per year
Operation and maintenance	1.25 an acre per year
Local government	.60 an acre per year
	$3.35 an acre per year

Construction costs were not collected in 1931 or since by reason of a moratorium declared by Congress. They are not cancelled, however, but merely postponed.

High priced crops are more costly to raise. Take beets for instance. The 1933 crop return indicates that sugar beets yielded an average of $72.76 an acre. To that should be added $5.00 an acre for beet tops a by-product usually available for feeding. An acre is considered as valuable as a ton of hay. That makes a total gross return of $77.76 an acre with seed furnished by the sugar company which makes it look like a gold mine.

It is lucrative even after costs have been deducted. But costs are heavy. Fertilizer costs $3.00 an acre. Beet help for thinning and topping costs from $20 to $25 an acre unless the farmer's own family is willing to get down and crawl through the rows on hands and knees. Then there is irrigating, spraying, cultivating, and moving the crop to the factory. Just what these amount to is not immediately available. But a good margin of profit is usually the net result even under adverse conditions. It is a profitable crop for those regions fortunate enough to be permitted to grow it in competition with the Cuban, Hawaiian, and Philippine industry. Truck garden crops, however, are almost as lucrative, especially if a canning factory can be induced to locate on the project. They also require less water.

The Lucrative Crops Require More Water

Mr. J. C. Ellickson, Ass't. Director, Rural Rehabilitation, read the manuscript of this report and then prepared the following to show that the most lucrative crops are those in the main possible only by irrigation or greatly benefitted by irrigation.

IRRIGATED CROP VALUES—1933

32,362 acres irrigated, all crops, averaged a gross return of	$30.62
17,857 acres (55.2% of total area) in alfalfa, beans, sugar beets, garden, and potatoes averaged a gross return of	51.44
14,505 acres (44.8% of total area) in small grain, hay, and pasture averaged a gross return per acre of	4.98
The 2,979 acres of dry-farmed crop within the project averaged a gross return per acre of	2.48

Settlers Encouraged

There has been a continual, though relatively small, influx of new settlers. Many begin as renters. The renter is given 80% of the crop. The owner pays the taxes.

The dry-farmed area has been gradually reduced. In 1927, 18,300 acres were dry-farmed yielding crops worth $14.77 an acre. In 1933 only 3,000 acres were dry-farmed and yielded only $2.48 an acre. Not all but much of the difference in area has been sub-divided and colonized in the interval. The present population is about 2,600 people on the 610 farms and 4,100 people in the six towns, making a total of about 6,700 people. This popula-the six towns, making a total of about 6,700 people. There is immediate room for about 125 additional families on the Montana side and 75 families on the North Dakota area.

Project Costs

The project cost to June 30, 1934, $4,543,462.67. The government has cancelled $382,254.00. The settlers have paid $290,497.13 to that date. Balance due is $3,870,711.53. No payments due the government on that date were unpaid.

Twelve hundred irrigated acres on the North Dakota side of the project were inspected October 17, 1934. These were owned by eight land owners. Three pieces totaling 276 acres had no taxes against them. The balance had an average of $2.75 an acre unpaid taxes due at the time of inquiry, October 19, 1934.

Operation and Maintenance

The project was started in 1905. Water was available in 1909. 17,500 acres were irrigated in 1912. It has about 37,000 acres now under water. Over 3,000 structures are required to control and distribute the water. Repair and replacement is more or less continuous. The most important structure is a wooden framed dam 700 feet long in the Yellowstone River at the head of the project. It cost about $200,000. It now requires annual maintenance. The headgate is a large concrete structure in fair condition but with a dilapidated wooden superstructure of walk and hoisting platform.

There are 71 miles of main canal and about 250 miles of laterals. Ditch cleaning is a continuous process. As long as nothing serious happens to the major structures, the present rate of $1.25 an acre would appear to be sufficient to operate and maintain the project.

Local taxes will increase because, among other reasons, the school population among the immigrants will increase faster than the adult tax-paying population.

It will probably rise to $1.00 an acre as it has on some other projects.

TABLE III—CROP YIELD REPORT

Irrigated Crop

Lower Yellowstone Project — Montana—North Dakota — Year of 1933

CROP	AREA (ACRES)	Unit of Yield	YIELDS Total	YIELDS Average Per A.	VALUES Per Unit of Yield	VALUES Total	VALUES Per Acre
Alfalfa	4,692	ton	6,035	1.3	$4.00	$ 24,140	$ 5.14
Barley	2,686	bu.	60,102	22.4	.20	12,020	4.48
Beans	1,027	bu.	20,264	19.7	.90	18,238	17.76
Beets, Sugar	11,544	ton	150,518	13.04	5.58	839,890	72.76
Cane	123	ton	276	2.2	2.00	552	4.49
Clover Hay, Sweet	586	ton	646	1.1	2.00	1,292	2.20
Corn, Indian	1,091	ton	3,091	2.8	2.00	6,182	5.67
Flax	287	bu.	1,438	5.0	1.40	2,013	7.01
Garden	309	acre	0	.0	.00	20,345	65.84
Hay	2,755	ton	3,065	1.1	2.00	6,130	2.23
Millet Seed	22	bu.	106	4.8	.50	53	2.41
Oats	2,689	bu.	88,322	32.9	.15	13,248	4.93
Pasture, Tame	796	acre	0	.0	3.00	2,388	3.00
Pasture, Wild	1,099	acre	0	.0	1.00	1,099	1.00
Potatoes	285	bu.	39,816	139.7	.40	15,926	55.88
Wheat	2,252	bu.	43,930	19.5	.60	26,358	11.70
Speltz	98	bu.	2,544	26.0	.15	382	3.90
Miscellaneous	21	acre	0	.0	.00	535	25.48
Total Cropped	32,362		Total and Average			$999,954	$29.50

AREAS	ACRES	FARMS
Total Irrigable Acreage Classes 1 to 4, District Proper	46,279	610
Total Irrigable Area Farms Reported	40,734	495
Total Irrigated Area Farms Reported	33,902	495
Grand Total Irrigated: Total Cropped Area Farms Reported	36,881	554

TABLE III—CROP YIELD REPORT (Cont'd)

Dry-Farmed Crop

Lower Yellowstone Project | Montana—North Dakota | Year of 1933

CROP	AREA (ACRES)	Unit of Yield	YIELDS Total	YIELDS Average Per A.	VALUES Per Unit of Yield	VALUES Total	VALUES Per Acre
Alfalfa	91	ton	63	0.7	$4.00	$ 252	$2.77
Barley	425	bu.	5,536	13.0	.20	1,107	2.60
Beans	18	bu.	0	.0	.90	0	.00
Corn, Indian	134	bu.	1,808	13.5	.20	362	2.70
Corn, Fodder	432	ton	365	0.8	2.00	730	1.69
Flax	61	bu.	127	2.1	1.40	178	2.92
Garden	14	acre	0	.0	.00	885	63.29
Hay	435	ton	290	0.7	2.00	580	1.33
Millet Seed	13	bu.	33	2.5	.50	17	1.31
Oats	254	bu.	2,793	11.0	.15	419	1.65
Pasture, Wild	423	acre	0	.0	1.00	423	1.00
Potatoes	6	bu.	375	62.5	.40	150	25.00
Wheat	539	bu.	3,593	6.7	.60	2,156	4.00
Speltz	134	bu.	836	6.2	.15	125	.93
Total Cropped	2,979		Total and Average			$7,384	$2.48

THE FORT PECK PROJECT

General Statement

Any project no matter where located that purports to so regulate the flow of the Missouri River thru North Dakota that 30,000 c.f.s. will be maintained at Yankton, South Dakota, thruout the navigation season deserves mention in the report of the State Engineer Department. The following data have been taken largely from an article in "Civil Engineering" for September 1934, by Theodore Wyman, Jr., Captain, Corps of Engineers, U. S. Army, Kansas City, Missouri.

Location: 20 miles southeast of Glasgow, Montana, above the mouth of the Milk River.

Drainage area: 57,700 square miles.

Years of record used: 1890-1933.

Average annual precipitation: 15 inches.

Mean annual flow	8,253,000 acre-feet 143 acre-feet per square mile 2.68 inches 11,300 cu. ft. per second
Minimum annual flow 1931	3,900,000 acre-feet 68 acre-feet per square mile 1.27 inches 5,300 cu. ft. per second
Maximum annual flow 1907	12,540,000 acre-feet 217 acre-feet per square mile 4.08 inches 17,200 cu. ft. per second
Maximum flow	June 1908 154,000 cu. ft. per second estimated 2.7 cu. ft. per second per sq. mile

Reservoir

Top area at spillway lip	245,000 acres
Capacity to spillway lip	19,500,000 acre-feet
Average depth of lake	80 feet
Approximate length	185 miles
Greatest width	17 miles
Silt accumulation	225,000 acre-feet per 100 years.

Dam

Material: Hydraulic filled earth with steel sheet piling cut-off.

Top length:	Main dam	8,800 feet
	Low levee	11,000 feet
	Total	19,800 feet (3¾ miles)

Maximum height	247 feet
Free board	35 feet
Height to spillway lip	212 feet
Top width	100 feet
Bottom width	2,700 feet
Upstream face	Rock paving
Estimated cost	$86,000,000

Excavation and Materials in Fort Peck Dam

Stripping	4,100,000 cu.yd.
Earth-fill in the dam section	92,000,000 " "
Earth-fill in the dike section	6,500,000 " "
Rock and gravel fill in the upstream toe of the dam	861,000 " "
Rock and gravel fill in the downstream toe of the dam	2,745,000 " "
Rock for facing the surface of the dam	870,800 " "
Rock in the parapet wall	201,190 " "
Gravel for facing the surface of the dam	497,200 " "
Portland cement concrete in the parapet wall	246,500 " "
Steel sheet piling	1,042,100 sq.ft.
Excavation incidental to the placing of steel sheet piling	110,700 cu.yd.

Purpose

"When the Fort Peck Dam is completed, it may be used for the following purposes:

1. Storage in the reservoir of a major part of the annual "June rise" of the Upper Missouri Basin caused by the melting snows of the mountainous region and the heavy run-off from rainfall, which normally occurs during the months of April, May, June, and July.

2. Operation of the reservoir so as to maintain a minimum flow of 30,000 cu.ft. per sec. at Yankton, S. D., at all times during the navigation season (March 20 to November 15), thus maintaining a 9-ft. navigation channel from Sioux City, Iowa, to the mouth of the river, a distance of approximately 800 miles.

3. Operation of the top 8 ft. of the reservoir for complete flood control of the upper river and, incidentally, for lower flood heights in the Lower Missouri Valley.

4. The generation of power to be used for worthy irrigation pumping projects located in the upper basin and for other purposes.

Incidental benefits will be great, as the reduction of flood heights will make possible the reclamation of thousands of acres of very fertile, contiguous lands and throughout the unimproved river will materially reduce bank erosion, which during a normal year is as great as 47 acres of land per mile of river. The maintenance of a minimum discharge of 30,000 cu.ft. per sec. at Yankton will materially benefit navigation conditions on the Mississippi River below the mouth of the Missouri, especially during the low-water months of September, October, November, and December."

Operation of Reservoir

The operation of this reservoir so as to put 30,000 c.f.s. at Yankton, South Dakota, 1150 river miles away during the navigation season will be an interesting undertaking. While it is not primarily a flood control project yet any floods that can be anticipated will save just that much reservoir water. Flood stages on the principle rivers in the immediate vicinity such as the Milk, the Yellowstone and the Little Missouri can be anticipated quite accurately from the daily gage heights at the various gaging stations. The flow from the reservoir can then be regulated to put approximately the desired amount in the river at Williston or Elbowoods, North Dakota, provided there is sufficient water in the reservoir. In that respect the effect will be similar to that of a dam at Garrison, North Dakota. But floods in tributaries farther down the river will be more difficult to anticipate as the distance from this region increases.

River Stages for 30,000 c.f.s.

Williston, North Dakota: The gage at the Lewis and Clark bridge will read about 6.6 feet for a discharge of 30,000 c.f.s. The average stage for August 1934 was about 0.5 feet. That is a rise of six feet.

Bismarck, North Dakota: The Weather Bureau gage will read about 7.5 feet for a flow of 30,000 c.f.s. The average August 1934 stage was about 2.5 feet. That means an increase in depth of five feet.

If there was at least a foot of depth at even the shallowest parts during August, which seems a reasonable assumption, then the Ft. Peck Reservoir will create a six foot channel for another 1000 miles of river above Sioux City, provided of course it has the water. Boats of the Benton Packet Co., Bismarck, draw only three feet of water, but they could not operate this last summer.

TABLE IV—LAKE LEVELS

Devils Lake			Turtle Mountain Lakes					
			Lake Metigoshe			Lake Upsilon		
	Sea Level Elev. Feet	Diff. Feet		Sea Level Elev. Feet	Diff. Feet		Sea Level Elev. Feet	Diff. Feet
Oct. 1, 1931	1410.81		Sept. 25, 1931 ..	2135.18		Oct. 25, 1931 ..	2103.47	
		0.57			0.89			0.40
Oct. 1, 1932	1410.24		Sept. 30, 1932 ..	2134.29		Sept. 30, 1932 ..	2103.07	
		2.87			1.69			0.83*
Aug. 7, 1934	1407.37		Oct. 7, 1934 ..	2132.60		Oct. 21, 1934 ..	2102.24	
Ave. bottom	1401.00		Ave. bottom	2121.3		Ave. bottom	2091.1	

*These two Turtle Mountain lakes are twenty miles apart and under such generally similar topographic and meteorological conditions that evaporation should be practically the same. But larger losses occur from Lake Metigoshe. These may occur from the bottom rather than from the top and be in part responsible for neighboring lakes and for springs in Oak Creek. Lake Upsilon received an additional supply of water by pumping from Wakopa Creek for several weeks during the spring of 1933. For description of equipment see p. 24, 15th Biennial Report.

TABLE V—LOSSES FROM DEVILS LAKE COMPARED TO SHEYENNE RIVER RUN-OFF

DEVILS LAKE

	Elev. Ft. Sea level	Diff.in Elev.Ft.	Area Sq.Mi.	Volume Ac-Feet	Run-off Sheyenne, N. D. Ac-Feet	Rainfall Maddock, N. D. Inches
June 27, 1924	1416.8		46.5			
		1.4		40,100		20.55
Sept. 3, 1925	1415.4		43.0			
		1.1		28,600		14.09
Sept. 7, 1926	1414.3		38.5			
		0.1		2,400		18.80
Sept. 1, 1927	1414.2		38.0			
		0.8		18,000		10.30
June 19, 1928	1413.4		32.5			
		0.9		16,000		19.17
Aug. 10, 1929	1412.5		23.5			
		0.9		12,100	32,100	16.73
Oct. 1, 1930	1411.6		18.5			
		0.8		8,800	1,860	16.83
Oct. 1, 1931	1410.8		16.0			
		0.6		5,900	4,820	16.34
Oct. 1, 1932	1410.2		14.5			
		1.2		10,600	12,880	12.61
Oct. 1, 1933	1409.0		13.0	(53,400)	(51,660)	
		1.6		12,500	Discontinued	8.19
Aug. 7, 1934	1407.4		11.5			
		6.4		32,900		
Bottom	1401.0		5.0			

The question has been raised as to whether or not there was enough annual flow in the Sheyenne River in the vicinity of Devils Lake to offset the annual losses from the lake. It is about 12 miles north of the river.

The above table shows that the flow in the Sheyenne River for the four years ending October 1, 1933 roughly equaled the losses from the lake for 5¼ years prior to that date.

This particular quantity would have been insufficient for the higher lake stages shown in the table.

FINANCIAL STATEMENT

Status of budget at the end of the biennium, June 30, 1933:

	Present Budget	Total Expenditures	Balance
Salary, State Engineer	$ 6,000.00	$ 6,000.00	$ ....
Clerkhire, stenographic	2,700.00	2,586.55	113.45
Postage	100.00	100.00	....
Office Supplies	400.00	400.00	....
Furniture and Fixtures	100.00	24.50	75.50
Printing	300.00	236.67	63.33
Miscellaneous	500.00	458.52	41.48
Travel Expense	3,000.00	2,999.16	.84
Field Assistants	3,000.00	3,000.00	....
Hydrographic Survey	6,000.00	4,977.22	1,022.78
Missouri River Commission	5,000.00	5,000.00	....
Lake Conservation	7,000.00	6,977.45	22.55
Flood Irrigation	551.09	443.17	107.92
General Prior	1,196.25	963.57	232.68
Fire Contingent Fund	1,636.60	1,587.70	48.90
Total	$37,483.94	$35,754.51	$1,729.43
Present Prior	1,317.38	1,052.96	264.42

Distribution of expenditures for the last quarter of the biennium period ending June 30, 1933.

Principal Features	This Quarter	Biennium To Date
Miscellaneous Examinations and Surveys	$ 142.17	$ 2,187.47
Irrigation	60.20	928.58
Hydrometry	2,002.04	8,883.86
Flood Control	63.45	555.70
River and Lake Improvement	901.81	21,696.12
Topographic Surveys		
Meteorology		
Subtotal	$3,169.67	$34,251.73
Office Operation	66.91	541.10
15th Biennial Report	320.62	——
New Equipment less depreciation	—5.30	9,180.87
Total withdrawn from appropriation	$3,551.90	$43,973.70
*Old Equipment		7,166.23
		$36,807.47

*Equipment and Field Notes and Township Plats from previous Biennium.

Total appropriation		$37,483.94
Expenditures to date		36,807.47
Balance to date		$ 676.47
Previous prior	$232.68	
Present prior	264.42	497.10
Lake Conservation	$ 22.55	
Flood Irrigation	107.92	
Fire Contingent	48.90	
Hold over balance		$179.37

Status of budget on June 30, 1934.

	Present Budget	Total Expenditures	Balance
Salary, State Engineer	$ 3,840.00	$ 1,920.00	$ 1,920.00
Clerkhire, Stenographic	500.00	51.30	448.70
Postage	100.00	50.75	49.25
Office Supplies	200.00	109.21	90.79
Furniture and Fixtures	100.00	73.25	26.75
Printing	200.00	4.67	195.33
Miscellaneous	400.00	218.69	181.31
Travel Expense	1,500.00	661.79	838.21
Lake Conservation	22.55	....	22.55
Flood Irrigation	107.92	25.00	82.92
General Prior	1,317.38	1,077.96	239.42
Fire Contingent Fund	48.90	48.90	....
Total	$ 8,336.75	$ 4,241.52	$ 4,095.23

Distribution of State Engineer's time from July 1st, 1933, to June 30th, 1934, is as follows:

Flood Control	$ 21.75
Irrigation	848.00
River and Lake Improvement	696.10
Correspondence	158.47
16th Biennial Report	62.15
Vacation	118.00
Sick Leave	15.53
Total	$1,920.00