Blake

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

HYDRAULIC MODEL STUDIES OF SUPERIOR-COURTLAND DIVERSION DAM, HEADWORKS AND SLUICEWAY STRUCTURES--PROGRESS REPORT NO. I ON GENERAL STUDIES OF HEADWORKS AND SLUICEWAY STRUCTURES

Hydraulic Laboratory Report No. Hyd.-275

RESEARCH AND GEOLOGY DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

MARCH 22, 1950

FOREWORD

Hydraulic model studies reported herein were conducted in
the Hydraulic Laboratory, Bureau of Reclamation, during the period
August 1948 to June 1949.

The Superior-Courtland Diversion Dam is a part of the
Bostwick Division, Kansas River District, Missouri River Basin
Project.

The designs and studies were made in cooperation with the
Diversion Dam Section, Canals Division, Branch of Design and Construc-
tion, Bureau of Reclamation. Messrs. A. W. Kidder, H. E. White, and
M. E. Day of the Canals Division visited the laboratory on numerous
occasions and made many helpful suggestions.

The studies were made by O. S. Hanson under the direct
supervision of E. J. Carlson and C. W. Thomas. Mr. E. W. Lane,
Consulting Hydraulic Engineer, provided advice and guidance throughout
the testing program.

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Branch of Design and Construction
Research and Geology Division
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Laboratory Report No. 275
Hydraulic Laboratory
Compiled by: O. S. Hanson
Reviewed by E. J. Carlson
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Subject: Hydraulic model studies of Superior-Courtland Diversion Dam,
          headworks and sluiceway structures--Progress Report No. 1 on
          general studies of headworks and sluiceway structures

SUMMARY

        The primary purpose of these model studies was to find the
headworks design that would pass the highest percentage of the bed load
of the stream through the sluiceway. The various designs tried were
compared on the basis of the ratio of the concentration of the sand in
the water passing through the sluiceway to that passing through the
headworks $(\frac{C_s}{C_h})$.

        The various designs tested are shown in Figures 5, 6, and 7.
The original design for the Courtland headworks and sluiceway gave a
ratio of $\frac{C_s}{C_h} = 0.68$ for the standard prototype discharge used: 400 cfs
through the headworks and 200 cfs through the sluiceway. Changes No. 1
to 3 were unsuccessful and showed no improvement over the original de-
sign. This was partly due to the greater turbulence created in front of
the headworks. This turbulence caused a larger percentage of the bed
load to be picked up and carried through the headworks as suspended load.
Change No. 4 using a divide wall between the overflow weir and the
sluiceway proved to be the most favorable, resulting in a ratio of
$\frac{C_s}{C_h} = 6.63$. Further tests with this type of wall involving changes in
alignment of the bank of the pool excavation, length of wall, and wall
location failed to show any further improvement in its desilting
characteristics. Figure 5 shows the recommended design.

        Additional tests were made incorporating a vortex tube across
the face of the headworks and a narrower sluice gate. These tests
indicated an even greater improvement in sediment distribution, but
since it was impractical to incorporate these changes in the Superior-
Courtland design, further testing of these schemes was postponed until
a later date.

Tests on the Superior Canal headworks were limited to designs similar to that proven best for the Courtland Canal headworks. These designs are shown on Figure 7 with the recommended design shown on Figure 5.

INTRODUCTION

The problem of control and removal of coarse sediment carried into canals by water diverted from heavily sediment laden streams has recently become a larger and larger item in the operation and maintenance costs of many of the Bureau of Reclamation projects; and with the increasing demand for water resulting in greater diversions from these streams, the importance of the problem of excluding the sediment from the canals will continue to increase.

On some of the larger projects elaborate desilting works have been built, such as those on the All-American Canal. On the smaller projects, however, the cost of such structures cannot be justified, and simpler and cheaper means must be devised. The use of sluiceways to carry the sediment past the diversion weirs by wasting part of the water has been used in many instances. Some of these structures have proven satisfactory, but many have failed to exclude the coarse sediment from the canal system, and frequent dredging of the canals has been necessary.

As authorized by letter from Assistant Director, Region 7, dated July 12, 1948, a model study of diversion structures to test headwork and sluiceway design for the control of sediment was begun by the Hydraulic Laboratory in August 1948. Since the Superior-Courtland Diversion Dam on the Republican River was the first of several diversion dams to be built in the Kansas and Lower Platte River Basins, it was decided to use this design as the starting point in the model studies. A 1:15 undistorted scale model of half the diversion weir and the Courtland Canal headworks and sluiceway was built.

Since the design of this structure was already complete and construction was underway before model testing could be finished, the scope of these studies was limited. Only minor changes and additions could be incorporated in the designs. Tests are now being made on the Republic Diversion Dam headworks and sluiceways, and general studies are underway, results of which will be incorporated in the design of the Scandia diversion and the diversions built in the Columbia, Middle Loup, and Grand Island Divisions.

DESCRIPTION OF MODEL

Since it was necessary to obtain good movement of the sand used in the bed with relatively small discharges, as large a model as practicable was deemed necessary to obtain satisfactory results. Space in the Hydraulic Laboratory approximately 30 by 70 feet was available. By utilizing this entire area, it was found that a 1:15 undistorted scale could be used which would include an area sufficient to cover one-half of the diversion weir, the Courtland Canal headworks and sluiceway, and approximately 400 feet of the upstream and downstream river channel. The general layout of the model is shown in Figure 4.

Although the model was built to an undistorted scale, it was realized that in order to get sufficient movement of sand through the model either the discharge or slope scale would have to be increased. To simplify operation and computations, the discharge was kept at the proper scale and sand was added at a constant rate allowing the model to build up whatever slope was necessary to reach an equilibrium condition.

At the time of a prior sediment load experiment, sand samples from practically all local sources were given a size analysis. The most satisfactory of these sands for experimental purposes was obtained from a loosely cemented sandstone ground in a hammer mill giving a sand with a median diameter of approximately 0.2 mm with 90 percent between the 40- and 100-mesh Tyler Standard screens (0.43 mm to 0.15 mm). This sand was used in the previous tests and found to move satisfactorily under relatively low discharges, and because of its uniformity in size no difficulty was encountered due to sorting under the action of the water. Other materials were considered, but the fine uniform sand had the best characteristics and was easily available so it was used.

Figure 8 shows photomicrographs of the model sand and washed Republican River sand. Size comparison can be made from the 1 mm rectangular grid shown on the photographs.

Water was supplied to the model by a portable pump mounted over the supply channel. Flow into the model was measured with a venturi orifice meter and controlled by means of a valve. Division of flow through the sluiceway and headworks was controlled by gate settings, and a V notch weir was placed in the end of the return channel from the headworks to measure the amount of water diverted through the headworks.

Sand was added at 5-minute intervals by filling a 3- by 1-1/2-inch aluminum channel 12 feet long and dumping it on a broad-crested weir. The sand was then washed into the model by the water flowing over the weir.

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Samples of the water flowing through the sluiceway and headworks were taken at regular intervals by passing a collecting trough, Figure 9A, through the falling nappe. These samples were collected in tanks, Figure 9B, calibrated to read the amount of water in liters. The sand settled into glass funnels mounted at the bottom of the tanks. These funnels were graduated in grams of dry sand so the amount of sand could be read directly and the concentration computed without any further conversion of the data.

METHOD OF OPERATION

In order to reduce to a minimum the number of variables affecting the sediment discharge, it was necessary to choose a standard water discharge at which to operate the model during the tests. This discharge did not necessarily represent an exact condition in the prototype. It was felt that the design which appeared to operate best using the standard flow would probably be the best for practically all other flow conditions.

The proposed plan of operation for the project showing river discharges and canal requirements was obtained from the Hydrology Division, Branch of Project Planning. From a study of these data it was decided to use a total flow of 600 cfs divided 400 cfs through the headworks and 200 cfs through the sluiceway as the standard discharge. Normal water-surface elevation of 1639.0 feet was maintained in the reservoir for all general tests.

Results of sediment investigations in the Kansas River Basin, November 1, 1942, to September 30, 1946, by the Corps of Engineers, Department of the Army, showed the Republican River near Bloomington, Nebraska, to carry a bed load of approximately 0.165 percent of the water discharge by weight. At the standard discharge of 600 cfs this would require a rate of sand feed into the model of 0.0713 pounds per second. The channel used as a feeding trough had a capacity of approximately 25 pounds. The rate of sediment feed used was one channel full each 5 minutes. This gave a concentration just slightly higher than the prototype concentration which proved very satisfactory.

On the preliminary test runs it was found that the concentrations passing through the headworks and sluiceway varied with time due to the intermittent addition of the sand load. To correct for this fluctuation samples of the headworks and sluiceway water were taken simultaneously at a constant interval following the addition of sand. From these samples the concentration of sand passing through the headworks and sluiceway in parts per million was calculated.

TEST RUNS ON COURTLAND HEADWORKS

Original Design

An initial test run was made with the sluiceway and headworks arranged as shown on Drawing No. 271-D-29 (Figure 3) with training walls omitted. The model was operated at the standard discharge of 600 cfs; 200 cfs through the sluiceway; and 400 cfs through the headworks. Figure 10A shows the sand bed upstream from the headworks immediately before the start of this run.

Samples were taken of both the sluiceway and the headworks discharge at 30-minute intervals. After only a few hours of operation rather heavy concentrations of sand were coming through Headgates 4 and 5 and the area in front of the headworks started filling from the upstream end. The majority of flow through the model was concentrated in a channel along the right bank, as shown in Figure 11.

As the test was continued, the area in front of the headworks continued to fill and the concentrations in the headworks discharge increased. Very little sand was drawn through the sluiceway, however. After approximately 20 hours of operation the entire area in front of the headworks had filled with the exception of a small triangular area immediately upstream from the sluiceway. This area was then filled while the model was shut down. The model was run for an additional 5 hours, during which the concentration in the sluiceway began to increase. Averaging the concentrations shown by the samples after an equilibrium condition had apparently been reached, Figure 10B, showed a ratio of concentration in the sluiceway to the concentration in the headworks of 0.682.

The discharges through the sluiceway and the headworks were then reversed giving a canal discharge of 200 cfs and a sluiceway discharge of 400 cfs. This run was continued for 14 hours and 30 minutes, at which time the concentrations showed by the samples seemed to have stabilized. The headworks and sluiceway gates were then reset to their original positions and an additional run of 5 hours was made. This run gave the ratio $\frac{C_s}{C_h}$ equal to 1.33 for $\frac{Q_s}{Q_h} = 2$ and 0.314 for $\frac{Q_s}{Q_h} = 0.50$.

With the bed left as it was after the completion of the second run a system of intermittent sluicing was tried. The sluiceway and headworks gates were set to the standard discharge and the model operated at these settings for 55 minutes. The sluiceway gate was then fully opened for 5 minutes. This procedure was repeated each hour for a total run of 20 hours and 30 minutes.

When the sluice gate was full open the level of the pool dropped considerably, causing the discharge through the headwork gates to drop practically to zero. After the sluicing period the sluice gate was completely closed until the pool had filled to its normal elevation of 1639.0 feet, after which the gates were reset to the 200- and 400-cfs discharges. Regular samples were taken at 20-minute intervals between sluicing and occasional samples were taken during the sluicing period.

During sluicing heavy scour occurred in front of the headworks with the riprap floor, at elevation 1632.0 feet, being exposed over most of its area. A pronounced channel was scoured upstream through the pool deposit, Figures 12A and 12B.

Samples taken between sluicing periods showed $\frac{C_s}{C_h} = 0.713$ and the samples taken during the sluicing periods gave $\frac{C_s}{C_h} = 4.269$. The combined ratio was 3.770. When the sand was removed from the tail box it was measured and showed that 89 percent of the total sand moved had passed through the sluiceway using only 36 percent of the water.

Although this system of operation appeared to offer a great deal of promise as far as efficient removal of the sand was concerned, it was felt that the fluctuation of the canal water level due to the varying discharge through the headworks during the sluicing periods would cause sloughing of the canal banks and was, therefore, not a satisfactory means of operation on a project having unlined canals. No further study was made of this system of operation.

Upstream Guide Walls

A guide wall 55 feet long and 12 feet 6 inches high was then installed as shown in Change No. 1, Figure 6. The bed was set at elevation 1632.0 feet between the guide wall and the headworks. The remainder of the bed in the model was left as it was at the end of the previous run. The pier between the sluiceway and the overflow weir was also cut back 8.0 feet to eliminate the large draw-down it caused when the sluice gate was full open.

The model was then operated under the same conditions as for Run No. 1. Samples were taken at regular intervals.

This arrangement proved to be less satisfactory than the original design giving a ratio of $\frac{C_s}{C_h} = 0.216$. The unsatisfactory sediment distribution seemed to be caused by the increased turbulence in the flow around the end of the guide wall causing a larger percentage of the load to be thrown into suspension and drawn out the headworks.

Photographs, Figures 13A and 13B, were taken during and at the end of this run. They show the heavy scour that occurred upstream from and around the end of the guide wall and the heavy deposit between the guide wall and the headworks.

In order to speed up the testing program no complete runs were made on the next several changes. Several curved shapes were tried on the end of the guide wall shown as Change No. 1. The space between the sluiceway and the guide wall was varied and submerged vanes, shown as dashed lines on Change No. 1, Figure 6, were tried. None of these changes showed any appreciable improvement.

A new guide wall, Change No. 2, Figure 6, was then tried. This wall was built to operate submerged allowing the water to be diverted to flow over the top. Various wall heights were tried. It became apparent that when the elevation of the top of the wall was lowered sufficiently to allow the full flow of the headworks to pass over it, the sand bed upstream from the wall built up to an elevation sufficient to allow the bed load sand to also pass over the wall. A horizontal lip extending upstream from the face of this guide wall was tried in an attempt to keep the sand from flowing over the top. This lip showed a slight improvement over the other arrangement but the improvement was insufficient to warrant further tests. A vertical guide wall was also tried in place of the sloping bank. Figure 14A shows the model at the completion of the test with this arrangement.

Change No. 3, Figure 6, was the last of the upstream guide walls to be tried. The curved wall extending from the upstream edge of the headworks had its top above water and replaced the sloping bank of the pool excavation. The straight wall across the face of the headworks was a submerged weir with a 1-inch lip extending outwards from its face. The wall extended to the upstream face of the sluice gate.

When this wall was built high enough to prevent the sand from passing over the top, it caused too great a loss in head and the necessary discharge could not be obtained through the headworks. When the wall was lowered sufficiently to pass the required amount of water, the sand bed built up to the point where the sand passed over the top and there was no improvement over the original design.

Downstream Divide Walls

With the failure of the upstream guide walls to show any improvements over the original design, attention was next directed toward the use of a divide wall between the sluiceway and the overflow weir. It was felt that such a wall would induce a curved flow past the headworks with the headworks gates on the outside of the curve.

The first such wall tried is shown as Change No. 4, Figure 6. On preliminary runs this arrangement showed a marked improvement over the original design so a complete run at the standard water and sediment discharges was made with samples being taken at regular intervals. During the early part of the run, the area in front of the headworks began filling with sand. During this time, however, the samples showed approximately equal concentrations in both the sluiceway and headworks. After a few hours of operation, the area in front of the headworks had filled to the level of the headworks sill, and a roller across the face of the sill began to form. This roller immediately began scouring a large hole in front of the headworks and carried the majority of the sand past the headworks to the sluiceway.

During the remainder of the run this roller continued, reestablishing itself each time the model was started up. The roller carried a large part of the bed load sediment at right angles to the face of the headworks but was not strong enough to carry it the full distance to the sluiceway. A rather large sand bar was built up across the entrance to the sluiceway. The samples taken during this run showed a ratio of $\frac{C_s}{C_h} = 6.629$ even though there was a heavy concentration through Headgate No. 1. Figure 14B shows the sand bed in front of the headworks at the end of this run. The deep scour in front of the headworks and the bar built up across the entrance to the sluiceway is plainly visible.

After completion of the test with Change No. 4, the model was operated at a number of discharge combinations with sand added at irregular intervals. With a total of 600 cfs flowing through the model, the sluice gate was gradually closed and the headgates opened until the division of the water was 540 cfs diverted through the headworks and 60 cfs through the sluiceway. Flow conditions remained approximately the same with this new division of the discharge. The roller, however, became weaker and the bar across the front of the sluiceway built up resulting in a slightly higher concentration passing through Headgates 1 and 2. Upon returning to the 400-200 cfs division of flow the bar and roller returned to their original condition. With a higher total discharge through the model, conditions were the same with the height of the bar across the sluiceway controlled by the discharge through the sluice gate.

With the results of the previous tests indicating that the divide wall as used in Change No. 4 was the most satisfactory approach to the solution of the problem, attention was turned to the alignment of the sloping bank of the pool excavation. In all test runs made up to this point the alignment of the bank of the pool excavation was left as originally designed.

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The first revision of this alignment is shown as Change No. 5, Figure 6. The sloping bank was extended straight out from the headworks and the divide wall shortened to provide sufficient area to pass the required flow. This arrangement proved unsatisfactory. The reverse curve in the flow pattern resulted in very heavy concentrations passing through Headgates 4 and 5. The roller noticed in the previous run was apparent only in front of Headgates 1 and 2 and even at this location it was very weak.

The excavation bank was then swung back, Change No. 5A, cutting off the point which was causing the detrimental reverse curve in Change No. 5. This new alignment appeared better but still was not as satisfactory as Change No. 4. Sand distribution appeared to be about equal between the sluiceway and the headworks. No roller formed and the sand bed in front of the headworks built up to the level of the headworks sill.

The model was operated for a short time with Headgates 4 and 5 completely closed and the total diverted flow passing through Headgates 1, 2, and 3. This system of operation improved the sediment distribution. However, concentrating the flow too much by keeping some gates closed appeared to increase the turbulence in the flow through the headworks resulting in a higher concentration of sand being carried through as suspended load. Figure 15A shows the condition of the bed after these tests.

The guide wall was then extended to 67 feet 6 inches, Change No. 6, Figure 6. The flow conditions for this arrangement appeared to be approximately the same as that for Change No. 4. The roller action, however, was further upstream than previously noted and heavy concentrations passed through both Headgates 1 and 2. The bar across the face of the sluiceway also formed slightly further upstream, being located between Headgates 1 and 2. The restricted opening between the bank and divide wall caused an appreciable loss in head for the 600 cfs flow. Therefore, the wall was shortened to 52 feet 6 inches. The velocity of the water in front of the headworks was much lower and there was no indication of the roller. There was, however, a definite movement of sand across the face of the headworks toward the sluiceway. This set up showed enough promise to warrant a complete run. This run was made with the standard water and sand discharges and all headworks gates opened equally. When the model had reached equilibrium conditions the $\frac{C_s}{C_h}$ ratio equaled 5.129. Figure 15B shows the condition of the sand bed at the completion of this run.

After 20 hours' run with standard settings Headgates 4 and 5 were completely closed, Headgates 1 and 2 fully opened, and Headgate 3 used for regulation. This set-up resulted in a less favorable sand

distribution than had occurred in the first portion of the run. Head-
gates 1, 2, 3, and 4 were then opened uniformly and Headgate 5 re-
mained closed. Some improvement was noted but the conditions were
still not as favorable as with all gates opened equally.

From observations made on runs to this point it seemed that
the strongest roller formed and the most favorable distribution of the
sand load occurred when the flow past the headworks was at a fairly
high velocity. Thus, it appeared necessary to narrow the channel
between the divide wall and headworks as much as possible. To deter-
mine the minimum width of opening that could be used and still divert
sufficient water to meet canal and sluicing requirements, a tail box
was built below the headworks structure to maintain proper tail-water
conditions. It was found that a 37.5-foot opening would pass 1,000
cfs--750 cfs diverted and 250 cfs for sluicing--with the pool held at
elevation 1639.5 feet. With this wall arrangement, a strong roller
formed and sand distribution was favorable. Therefore, 37.5 feet was
chosen as the minimum distance between the divide wall and riprap
embankment.

The arrangement shown as Change No. 7, Figure 6, was then
installed. Included in this set-up was a vortex tube extending across
the face of the headworks immediately upstream from the sill. As
originally installed, the tube ended at the left side of Headgate 1.
With this arrangement the vortex inside the tube was very weak and
after only a short period of operation a bar built up across the lower
end of the tube blocking it completely. A closed conduit was then
installed on the end of the vortex tube which discharged under the
sluice gate. With this conduit in place the vortex tube kept itself
clean. Occasionally, a bar would build across it but in a very short
time the tube would clean itself out and again operate satisfactorily.
A complete run at standard water and sand discharges was made and showed
a $\frac{C_s}{C_h}$ ratio of 7.5. The roller action was also present ahead of the
vortex tube. Figure 16A shows the condition of the bed at the com-
pletion of the run. Note the absence of the bar extending into Head-
gate 1 which was present in the majority of the runs including the
downstream divide wall.

The run was then continued with the sluice gate blocked
off to give an effective width of 10 feet. This arrangement was even
more satisfactory, giving a $\frac{C_s}{C_h}$ of 10.5. Figure 16B shows the bed at
the completion of this run.

The vortex tube was then removed. The location of the
divide wall and riprap bank was left unchanged. A run was made using
the usual settings with both a 20- and 10-foot sluice gate width. The

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action was similar to that in the previous run but the sand distribution was not as satisfactory. The $\frac{C_s}{C_h}$ ratio for the 20-foot sluice gate was 2.92 and for the 10-foot gate 5.83. Figures 17A and 17B show the condition of the sand bed at the end of the runs with the 20- and 10-foot gates, respectively.

These runs indicated that the vortex tube and narrow sluice gate improved the sand distribution considerably. Due to the necessity of passing floating debris and other design considerations, these two features could not, however, be incorporated in the Superior-Courtland design. Further studies along these lines were therefore postponed until a later date.

Recommended Design

From the results of these tests the arrangement of the divide wall and excavation embankment shown on Figure 5 was recommended as the most favorable design for the Courtland diversion. Two further test runs were made on this design. One using only 150 cfs for sluicing which gave a $\frac{C_s}{C_h}$ ratio of 1.52 and the second in which the sluicing water was cut to 90 cfs with the total flow of 600 cfs remaining the same. This final run gave a $\frac{C_s}{C_h}$ of 0.94. Figure 18 shows the condition of the bed at the end of these two runs.

TEST RUNS ON SUPERIOR HEADWORKS

The model of the Courtland headworks was then modified to represent the Superior headworks by blocking off four of the five headgates and changing the alignment of the excavation embankment. The Courtland headworks tests had shown the desirability of the divide wall and necessity for as narrow a passage between this wall and the headworks as possible so tests on this structure were limited to variations in the location of the riprap bank. Due to the necessity of passing floating debris the width of the passage in this headworks was limited to a minimum of 20 feet, the same width as the sluiceway. Since this width was more than enough to pass the small amount of water diverted at this headworks, the sluiceway width was the controlling factor.

The model was first run as originally designed and gave a $\frac{C_s}{C_h}$ of 0.014. A plan of this design is shown on Figure 7. Figure 19A shows the bed at the end of this run.

A divide wall, Change No. 1, Figure 7, was then installed. When this arrangement was tested, practically the entire flow in the

model was in a channel down the face of the excavation bank. Very little flow occurred over the remainder of the model. This condition can be seen in Figure 19B.

This design showed some improvement over the original design, giving a $\dfrac{C_s}{C_h}$ of 0.18. When the model was run at higher discharges, however, there was a pronounced wave formed where the main flow struck the headworks wing wall. To eliminate this condition the excavation bank was extended straight into the headwork wing wall, Change No. 2, Figure 7. This arrangement improved the condition in front of the wing wall and gave a $\dfrac{C_s}{C_h}$ equal to 0.64. Figures 20A and 20B show the bed at the end of the run with Change No. 1 and during the run with Change No. 2 in place. The recommended design for the Superior headworks, Change No. 2, is shown in Figure 5. No further tests were run on this design.

ADDITIONAL STUDIES RECOMMENDED

The designs as recommended in this report represent a decided improvement over the original designs. It is felt, however, that with further investigation of several possibilities indicated in these studies additional improvement can be made.

The most promising of these possible improvements is the use of the vortex tube in connection with canal headworks. At the present time no definite information is available on the proper size, location, and shape of the tube for the most favorable operation. Necessary velocities over the tube, head on the tube outlet, length of the tube, and size of material handled also need to be determined. Further studies to ascertain these factors would undoubtedly result in a very large improvement in headwork designs for use on a sediment-carrying stream.

The size and location of the sluiceway was also indicated as a governing factor in the sediment distribution. Narrowing of the sluiceway concentrates the sluicing water, thus producing higher velocities and greater scouring action. The necessity of maintaining a channel through the upstream pool deposits during periods of no diversion will require a sluiceway of a certain capacity. The proper width to best satisfy both these requirements is another feature requiring further study. The feasibility of setting the sluiceway sill at an elevation lower than that of the normal riverbed and utilizing the scouring action of a contraction works should also be investigated.

The most favorable position for the headworks structure in relation to the sluiceway should be determined. The angle between the

headworks and sluiceway, position of headworks relative to sluiceway gate, and elevation of headwork sill above sluiceway sill are factors requiring further study.

Some of this additional information has probably been determined by other investigators and will require only a library research. Most of these problems will, however, require further laboratory studies. A library search of published literature relative to design of headworks and sluiceway structures is being carried on by Mr. E. W. Lane, Consulting Hydraulic Engineer, and will be covered in a separate report. It is recommended that the additional laboratory work required be accomplished as soon as funds and personnel are available.

OPERATING INSTRUCTIONS

The following operating instructions, based on the hydraulic model studies, are recommended as a guide to operating personnel in order to obtain the best results from the operation of this structure from a sediment control standpoint. Actual observations on the completed prototype may indicate some modifications in these procedures. In order to best determine these changes, records of the amount of sediment deposited in the canal, sediment load in the river upstream from the diversion, and operating procedure followed should be kept for the first several years the project is in operation.

Any testing or calibration of the headworks gates should be accomplished as soon as possible after closure of the diversion dam. These tests will probably cause a wide fluctuation in the pool elevation and canal discharge, and if these tests can be run before the pool area has filled with sediment the quantity of sediment drawn into the canal will be smaller than that which will occur if the tests are made after the pool area has become filled with sediment.

Intermittent sluicing, periodically opening the sluiceway gate full open gives the most favorable sediment distribution as indicated by the model studies. Whenever irrigation and canal conditions permit, this type of sluicing operation should be used. When the sluice gate is opened the headworks gates should be closed and the entire flow of the river allowed to flow through the sluice gate until the pool elevation has dropped to a minimum. The sluicing period should be alternated between the Courtland and Superior headworks, with only one side being sluiced at a time.

In all tests on the model the ratio between the canal and sluiceway discharges were kept constant for both headworks. It is felt, however, that in actual operation the available sluicing water
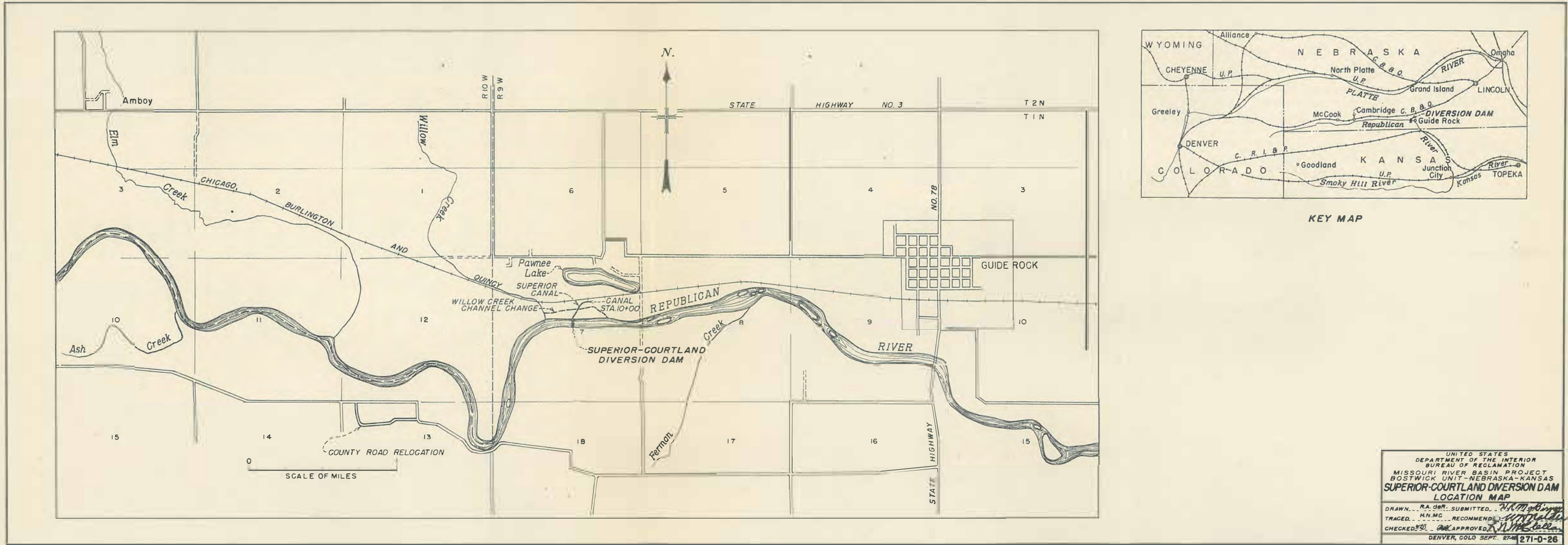
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should be apportioned between the two sluiceways by checking the amount of sediment being carried into the canals rather than by the water discharges. Although the Superior Canal draws approximately one-fifth the discharge of the Courtland Canal, it will probably require a greater proportion of the available sluicing water. Additional improvement in the sediment distribution can undoubtedly be obtained by varying this apportionment with changing conditions in the river flow and sediment deposition.

Two of the main periods during which care should be used in the settings of the sluice gates are during the recession of flood flows and the nonirrigation seasons. During either of these periods, it is possible that one of the channels to the headworks may become blocked by sediment deposits. It is very likely that these channels can be kept open by proper division of sluicing water between the sluice gates. It may be necessary at times to use the entire available flow of sluicing water in one sluiceway to maintain the channel.

Another factor governing the formation of the sediment deposits behind the diversion works is the water-surface elevation in the pool. The lower this elevation can be carried the lower the sediment deposit near the headworks and sluiceways will be. It would be desirable to set the headworks and sluice gate so as to maintain a pool elevation just sufficient to obtain the proper canal discharge.

The quantity of water diverted should be held as low as possible and still satisfy irrigation demands. Any surplus water diverted and returned to the river through wasteways will tend to aggravate the sediment problem by carrying additional sediment into the canals. The majority of this sediment will be deposited in the upper reaches of the canal and any sluicing action caused by flow through the wasteways will not offset this additional deposition.

FIGURE I

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FIGURE I

KEY MAP

WYOMING   Alliance   N E B R A S K A   Omaha
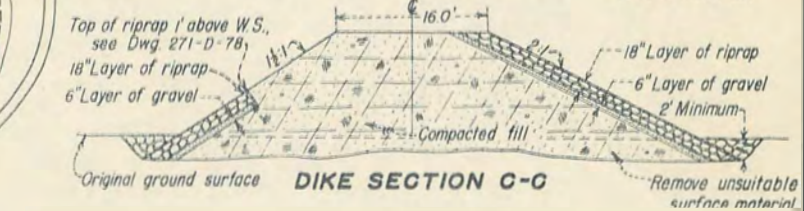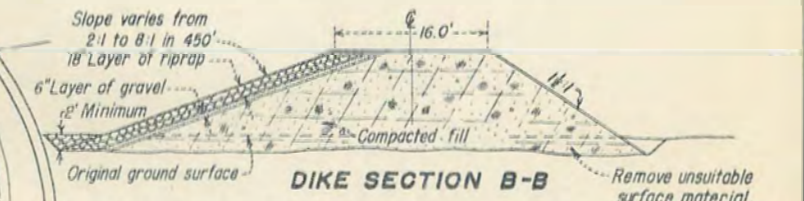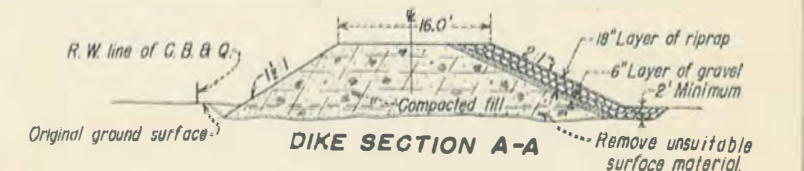   C.B.&Q.   RIVER
CHEYENNE   U.P.   North Platte   U.P.   Grand Island   LINCOLN
                        PLATTE   U.P.
Greeley   C. B.& Q.   DIVERSION DAM
                        McCook   Cambridge   Guide Rock
DENVER   C. R.I.&P.   Republican   River
COLORADO   ° Goodland   U.P.   Junction   Kansas   TOPEKA
                Smoky Hill River   City   River
                                                K A N S A S

Amboy

STATE   HIGHWAY   NO. 3   T 2 N
                                        T I N

R IO W
R 9 W

Elm   Creek   CHICAGO,   2   1   6   5   4   3
        Creek   BURLINGTON   Willow
3   Creek

AND   QUINCY
                Pawnee
                Lake
        SUPERIOR   CANAL
WILLOW CREEK   SUPERIOR CANAL   CANAL   REPUBLICAN   GUIDE ROCK
CHANNEL CHANGE   STA.10+00
Ash   Creek   11   12   7   8   Creek   9   10
10   SUPERIOR-COURTLAND
                                DIVERSION DAM   RIVER   NO. 7B

15   14   13   18   Fermon.   17   16   15
        COUNTY ROAD RELOCATION
0
SCALE OF MILES   STATE HIGHWAY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
BOSTWICK UNIT-NEBRASKA-KANSAS
SUPERIOR-COURTLAND DIVERSION DAM
LOCATION MAP
DRAWN. R.A.deR. SUBMITTED.
TRACED. H.N.MC RECOMMEND.
CHECKED. APPROVED.
DENVER, COLO SEPT.   271-O-26

FIGURE 2

LOGS OF DRILL HOLES
(STATIONS ARE ALONG AXIS OF DAM)

DIKE SECTION A-A

DIKE SECTION B-B

DIKE SECTION C-C

TYPICAL SECTION D-D

DIKE SECTION E-E

DIKE SECTION F-F

NOTES

The Government does not guarantee any interpretation of these records or the correctness of any information relative to geological conditions. Bidders and the contractor must assume all responsibility for deductions and conclusions as to the nature of the rock and other materials to be excavated.
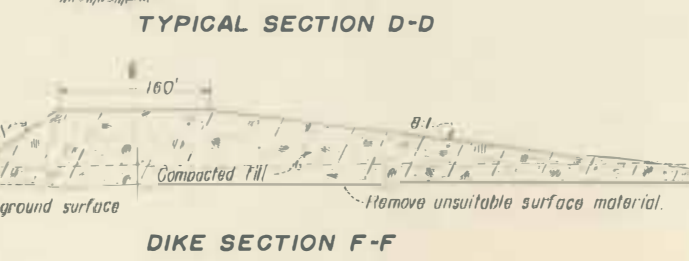
PAWNEE LAKE

REPUBLICAN RIVER

AXIS OF DAM   AZ 8°08'30"

SCALE IN FEET
100 0   500   1000

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
BOSTWICK UNIT - NEBRASKA - KANSAS
SUPERIOR-COURTLAND DIVERSION DAM
VICINITY MAP - DRILL HOLE LOGS

DRAWN R.A.de R.	SUBMITTED ...W.R.McBirney
TRACED P.M.F.	RECOMMENDED ...W.N.Kalder
CHECKED	APPROVED ...L.N.McClellan CHIEF ENGINEER

DENVER, COLORADO, SEPT. 5, 1946   271-D-28

FIGURE 3

PLAN

Natural ground surface
El. 1642 ±
24" Riprap on a 6" layer of gravel
North downstream wing-wall
Superior Canal
El. 1635.0
El. 1620.0
Excavate to El. 1632.0
North stilling pool
El. 1615.0
El. 1648.0
El. 1648.0
Axis of dam Sta 24+08.7
Superior Canal Sta. 1+46.8
North sluiceway
North training-wall
Superior Canal headworks
North upstream wing-wall
Excavate to El. 1632.0
18" Riprap on a 6" layer of gravel
-3% Grade
2:1
100'R.
El. 1642.0
Sta. 24+70
Axis of overflow weir and ₵ cableway
420'-0"

REPUBLICAN RIVER

24" Riprap on a 6" layer of gravel
Excavate to El. 1632.0
South downstream wing-wall
South stilling pool
El. 1620.0
South sluiceway
South stilling pool
Courtland Canal headworks
South upstream wing-wall
South training-wall
18" Riprap on a 6" layer of gravel
El. 1642.0
Sta. 28+90
Earth transition
₵ Courtland Canal
Canal Sta. 1+87.3- End earth transition. End of these specifications. El. 1630.07
-6% Grade
Axis of dam Sta. 29+96.6 = Courtland Canal Sta. 0+87.3
El. 1648.0
El. 1653.0
50'R.
+3% Grade
Warped slope
100'R.
100'R.
100'

SCALE OF FEET
40   0   40   80

SECTION A-A

422'-6" c.c. Towers
Cable
W.S. El. 1648.0 Q = 42,000 c.f.s.
El. 1653.0
El. 1632.0
Weir crest El. 1639.55
Firm shale
El. 1653.0
Cutoff

SECTION B-B
SECTION F-F SIMILAR

30'-0" Courtland Canal headworks
20'-0" Superior Canal headworks
14'-0"   7'-6"   27'-0"
El. 1653.0
El. 1648.0
El. 1641.55
Compacted backfill
El. 1643.22
10.0'
El. 1630.07 Courtland Canal
El. 1634.55 Superior Canal
W.S. El. 1638.57 Courtland Canal
W.S. El. 1638.88 Superior Canal
Q = 751 Courtland Canal
Q = 139 Superior Canal
Pile cap
El. 1639.55
Present ground line
18" Riprap
El. 1635.55
El. 1632.0
Backfill
Compacted backfill
10'x 6' Radial gate
Undisturbed earth or compacted backfill
Filter
6" Layer of gravel
Firm shale
8" 12" Dry rock paving
20' Steel sheet-piling
6" Layer of gravel

SCALE OF FEET
10   0   10   20

SECTION C-C - SOUTH SLUICEWAY
(NORTH SLUICEWAY OPPOSITE HAND)

47'-2"
35'-0"
El. 1653.0
El. 1639.55
Present ground line
18" Riprap
El. 1632.0
20'x 12' Radial gate
El. 1635.0
6" Layer of gravel
Backfill
Max. El. 1619.0
Max. El. 1611.0
Firm shale
El. 1615.0
El. 1620.0
24" Riprap
6" Layer of gravel

SECTION D-D

El. 1653.0
57'-8"
Compacted backfill
El. 1648.0
Compacted backfill
Present ground surface
El. 1641.55
El. 1635.55
El. 1630.05

SECTION E-E

El. 1648.0
Present ground surface
Compacted backfill
El. 1635.0
24" Riprap
El. 1620.0
El. 1615.0
6" Layer of gravel
Firm shale

SECTION G-G

El. 1653.0
18" Riprap
6" Layer of gravel
Backfill
Firm shale
El. 1648.0
Present ground line
Compacted backfill

SECTION H-H

24" Riprap
Present ground line
El. 1648.0
Compacted backfill
Backfill
Firm shale
24"

SECTION OF OVERFLOW WEIR

Axis of dam
Estimated Max. T.W. El. 1645.2
14'-8"   45'-3"   30'-6"
Present ground line
Estimated Min. T.W. El. 1635.4
El. 1639.55
Excavate to El. 1632
Backfill
El. 1622.00
El. 1623.55
Line of firm shale
Firm shale
Min. 8'-0"
Min. 5'-0"
18"
24"
6" Layer of gravel

FIGURE 4

SECTION A-A

SECTION B-B.

SECTION C-C

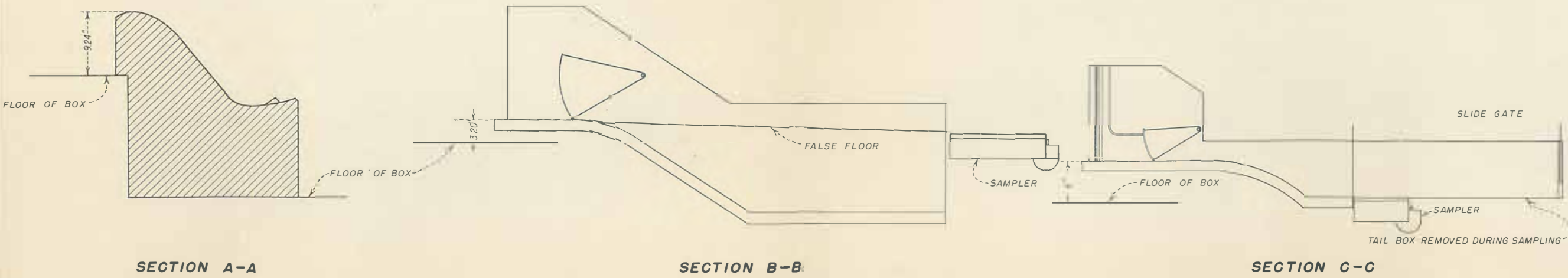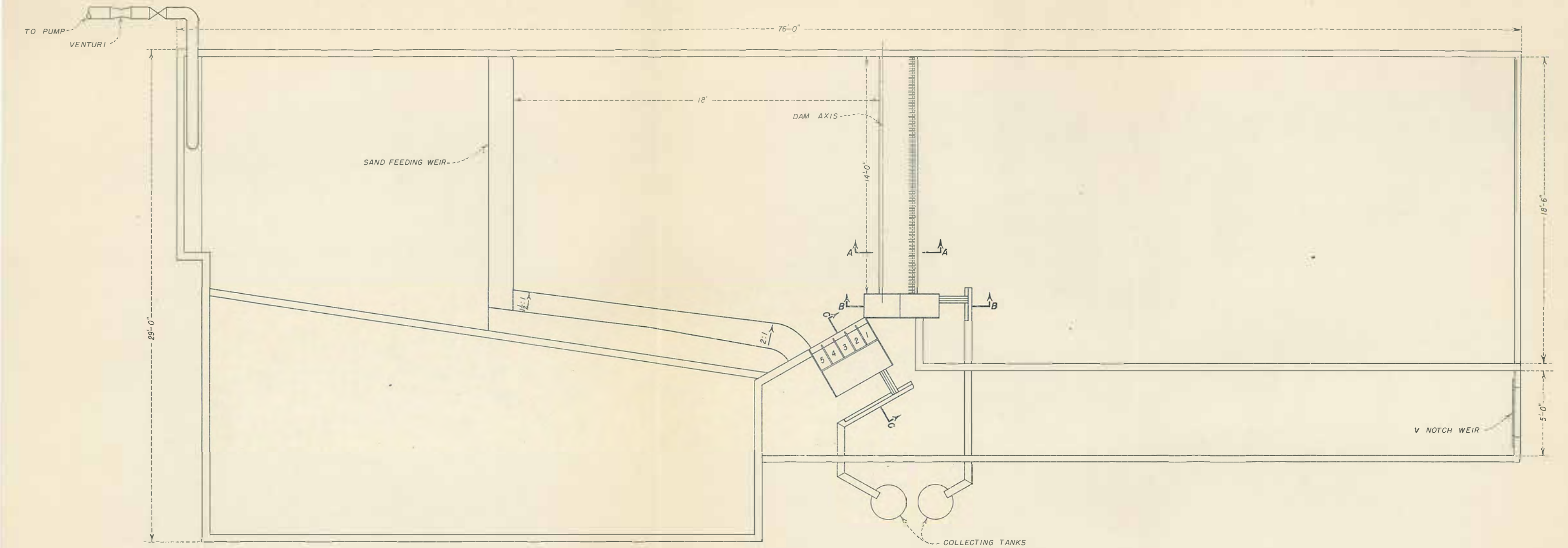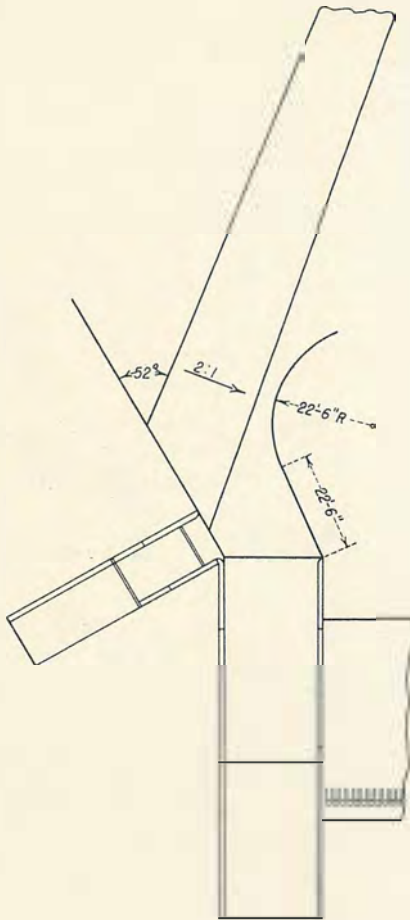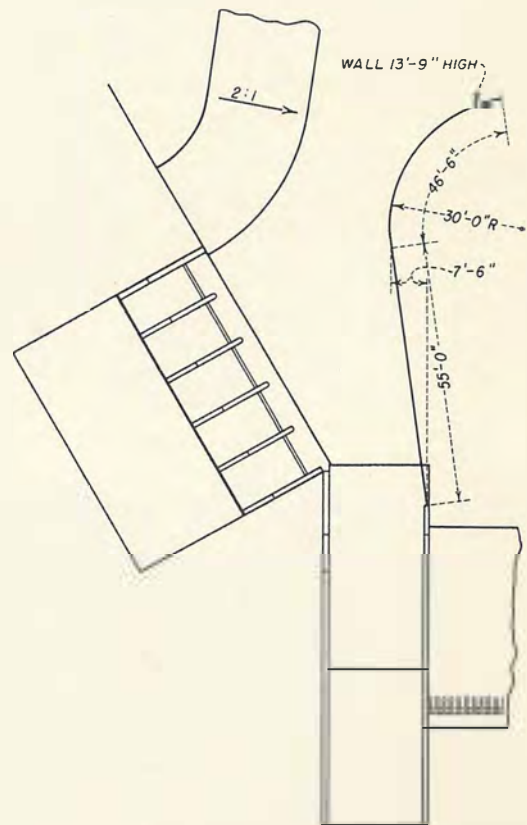FIGURE 5

SUPERIOR HEADWORKS

COURTLAND HEADWORKS

WALL 13'-9" HIGH

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL STUDIES
SUPERIOR—COURTLAND DIVERSION DAM
RECOMMENDED DESIGNS

DRAWN... F.E.S.   SUBMITTED
TRACED.. E.J.H. — P.V.   RECOMMENDED
CHECKED..   APPROVED

DENVER, COLORADO—JAN.19,1950
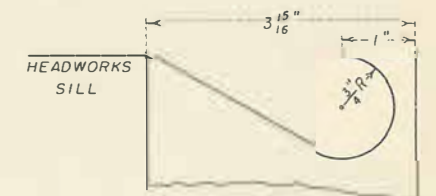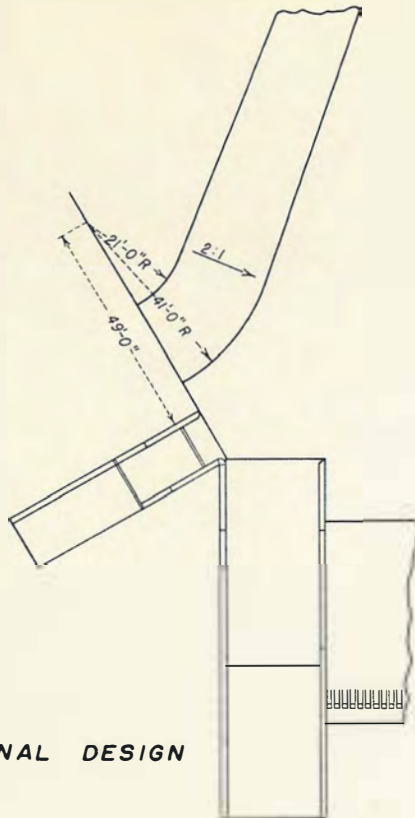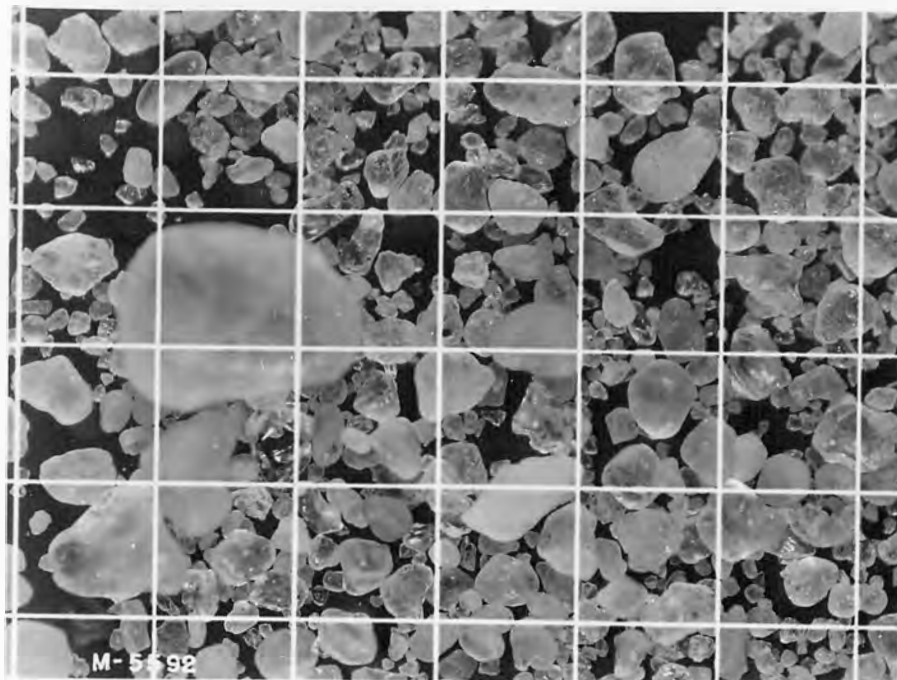
FIGURE 6

ORIGINAL DESIGN

CHANGE NO. 1

CHANGE NO. 2

CHANGE NO. 3

CHANGE NO. 4

CHANGE NO. 5

CHANGE NO. 6

CHANGE NO. 7

VORTEX - TUBE
MODEL SCALE
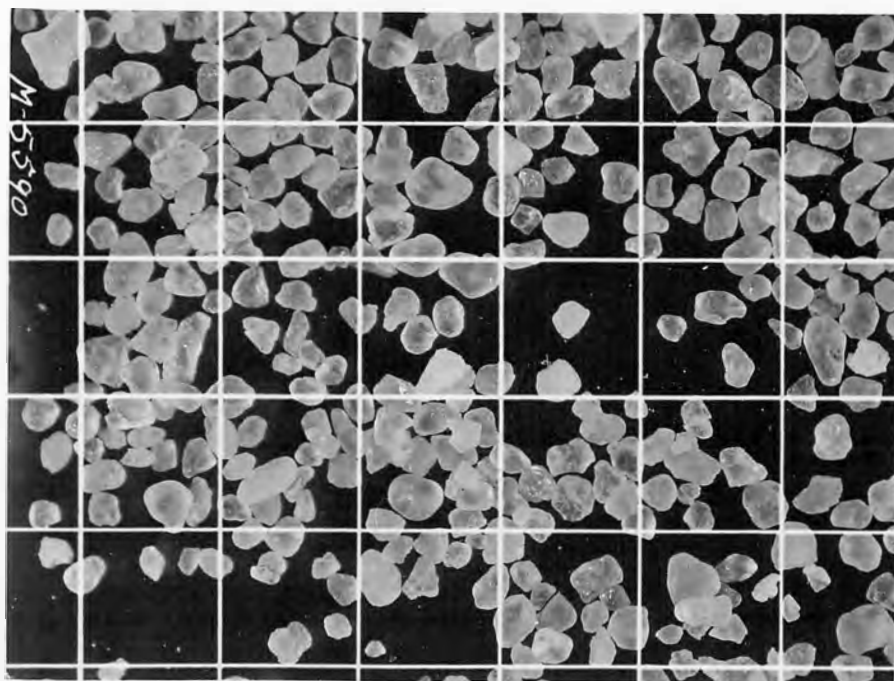
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
HYDRAULIC MODEL STUDIES
SUPERIOR-COURTLAND DIVERSION DAM
DESIGNS TESTED—COURTLAND HEADWORKS

DENVER, COLORADO—JULY 19, 1949

FIGURE 7

ORIGINAL DESIGN

CHANGE NO. 2

CHANGE NO. 1

Figure 8

A. Republican River sand

B. Model Sand

PHOTOMICROGRAPHS OF MODEL AND PROTOTYPE SANDS
GRID SPACING 1 mm.

Figure 9

A. Headworks Collecting Trough

B. Measuring Tanks

SAMPLING APPARATUS

Figure 10

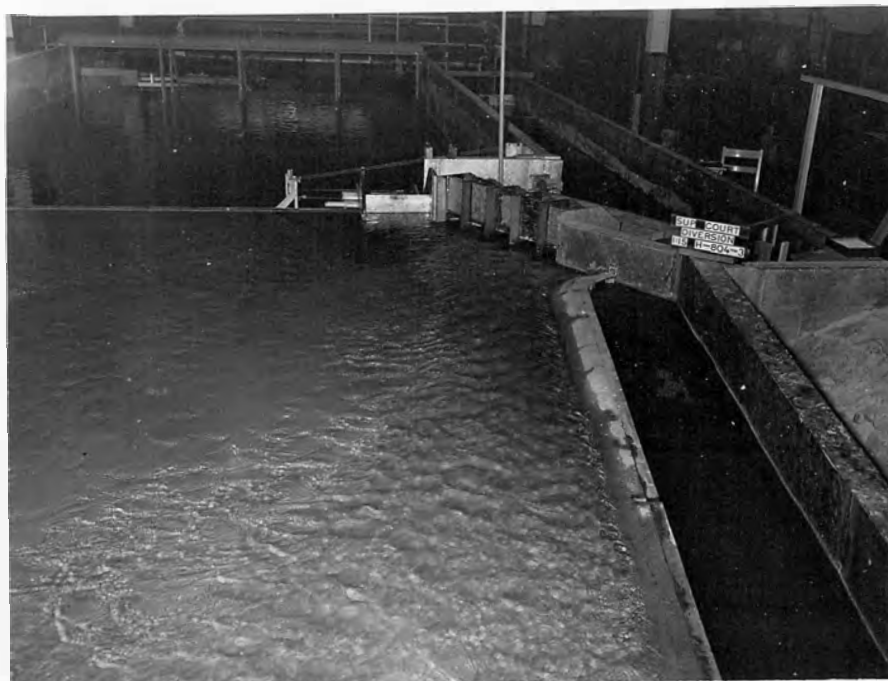A. Before Test Run Number 1

B. After Test Run Number 1

COURTLAND HEADWORKS
ORIGINAL DESIGN

Figure 11

A.   Channel Formed During Run Number 1

B.   Channel Formed During Run Number 1

COURTLAND HEADWORKS
ORIGINAL DESIGN

Figure 12

A.   Closeup in front of headworks

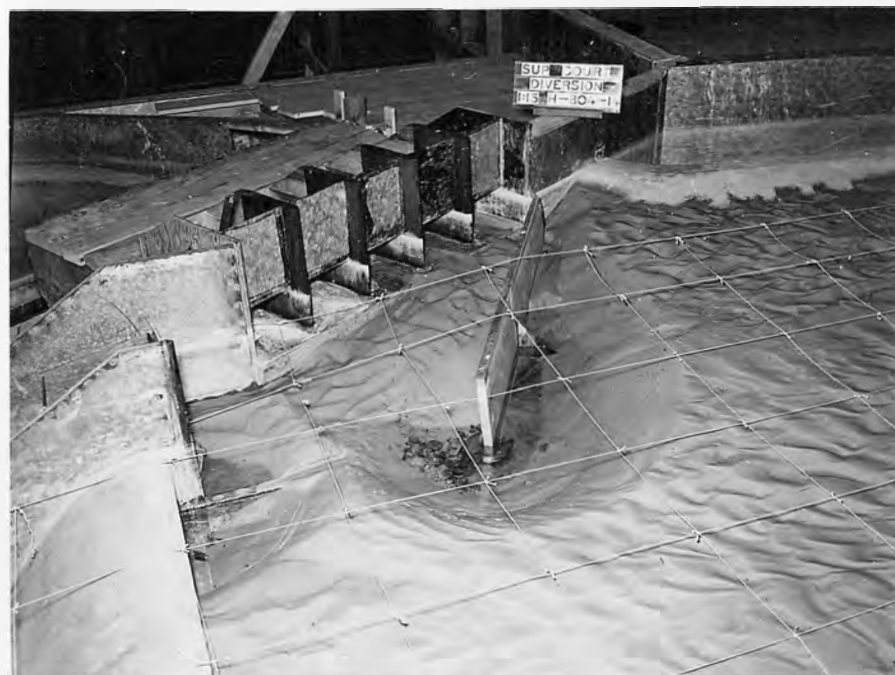B.   General shot of bed

Channel scoured during intermittent sluicing
String grid 15 ft. interval prototype

COURTLAND HEADWORKS
ORIGINAL DESIGN

Figure 13

A. General view of bed after 3 hours run

B. Closeup of bed after 21 hours run

COURTLAND HEADWORKS
CHANGE NUMBER 1

Figure 14

A. Closeup of bed after a short run
Change Number 2

B. Closeup of bed after 15 hours run
Change Number 4

COURTLAND HEADWORKS

Figure 15

A.   Closeup of bed after 6 hours run
Change Number 5A

B.   Closeup of bed after 20 hours run
Change Number 6

COURTLAND HEADWORKS

Figure 16

A.   Using 20 ft. Sluicegate

B.   Using 10 ft. Sluicegate

COURTLAND HEADWORKS
CHANGE NUMBER 7
VORTEX TUBE INSTALLED

Figure 17

A. Using 20 ft. Sluicegate

B. Using 10 ft. Sluicegate

COURTLAND HEADWORKS
CHANGE NUMBER 7
VORTEX TUBE REMOVED

A. Using 25% of Total Discharge
for Sluicing

B. Using 15% of Total Discharge
for Sluicing

COURTLAND HEADWORKS
RECOMMENDED DESIGN

Figure 18

Figure 19

A. Original Design

B. Change Number 1--Total Discharge 120 Cfs

SUPERIOR HEADWORKS

Figure 20

A. End of test run
Change Number 1

B. During test run
Change Number 2--Recommended Design

SUPERIOR HEADWORKS