HYD-195

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

FIELD TRIP TO MEASURE SEEPAGE LOSSES FROM CONCHAS CANAL - TUCUMCARI

Hydraulic Laboratory Report No. 195

ENGINEERING AND GEOLOGICAL
CONTROL AND RESEARCH DIVISION

BRANCH OF DESIGN AND CONSTRUCTION
DENVER, COLORADO

FEBRUARY 20, 1946

PREFACE

The Tucumcari Project was visited February 4-6, 1946, inclusive, to assist in establishing rating stations and procedures to be followed in measuring the amount of water lost from the Conchas Canal by seepage. The problem is unique due to the existence of numerous siphons and tunnels which afford controls for current meter gaging stations.

Much credit is due the project personnel, particularly, Messrs. M. M. Starr, L. F. Carden, E. E. Cerny, and R. K. DeWees for the courtesy and cooperation extended the writer.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
February 20, 1946.

Laboratory Report No. 195
Hydraulic Laboratory
Compiled by: Dale M. Lancaster
Reviewed by: Charles W. Thomas.

Subject: Report of field trip to establish method and procedure to be followed in measuring seepage losses from Conchas Canal - Tucumcari Project, New Mexico.

I. INTRODUCTION

1. Description of project. The Tucumcari Project, in the east central portion of New Mexico, is approximately that portion of the South Canadian River basin below the 4100-foot contour, south of the river and contiguous to the town of Tucumcari in Quay county. The irrigation project includes fifty percent of the gross area or 45,000 acres. The water originates in the reservoir behind Conchas Dam located at the confluence of Conchas Creek and South Canadian River, which controls 7,310 square miles of drainage area. The dam, including the headworks of the Conchas Canal, was built in 1936-1939 by the Corps of Engineers, U. S. Army. One-half of the total reservoir capacity of 600,000 acre-feet is reserved for irrigation, 100,000 acre-feet for dead storage, and the remaining 200,000 acre-feet for flood control. An annual draft of 135,000 acre-feet is expected for irrigation purposes. The irrigable land is connected to the reservoir by the unlined* Conchas Canal having a capacity of 700 second-feet and, after completion, a length of 76 miles. Laterals and sublaterals varying in capacity from 6 to 100 second-feet will represent an additional 180 miles.

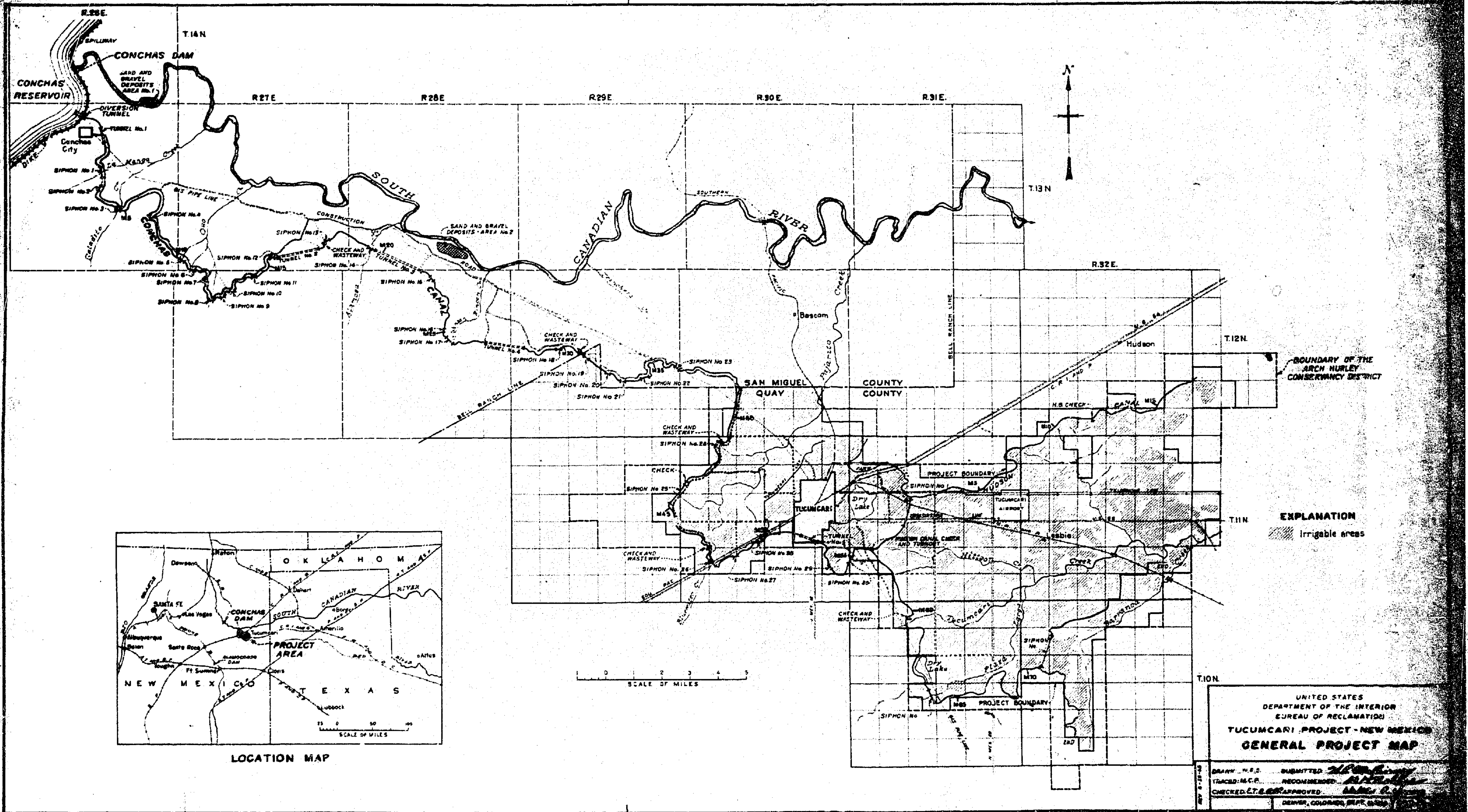
*Approximately 2000 feet of the canal was lined with compacted earth at the time of construction.

The roughness of the country traversed by the first 39 miles of the canal necessitated the construction of 22 siphons, approximately 40 culverts, and 4 tunnels (figure 1). In general the soil is composed of stratified sandy shale and soft sandstone. Occasionally a fairly durable gray sandstone is encountered as well as small amounts of volcanic tuff. The seams in the stratified material are horizontal and frequently show "slickensides."

The irrigable lands of the project overlies sandstones and shales. The soil is dark reddish brown and quite free from alkali although some small areas have alkalinity. The soil is generally classified as being "light brown, granular, noncalcareous topsoil on brown, highly calcareous, heavy subsoil. Depth to lime variable from 4 to 14 inches."[1]

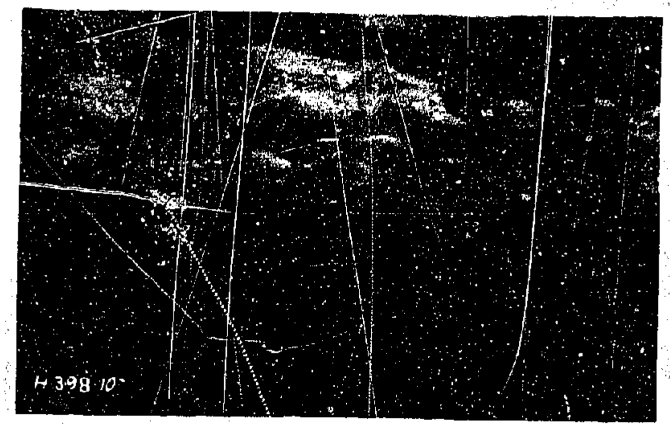
It may be readily seen that the composition of the soil traversed by the canal is susceptible to relatively high seepage, particularly when any seams in the stratified material are opened incidental to construction. Similarily when this soil is used as fill material, voids result due to the irregular gradation of the particles. A cavity under a rock, exemplifying the condition in a filled area, may be seen in figure 2A.

To prevent a continued loss of water by seepage from the Conchas Canal a lining is considered necessary. The type and extent of lining will be determined from a measurement of the quantity of water lost by seepage during the first season of operation. The existing plan provides for the installation of the required lining prior to the second irrigation season.

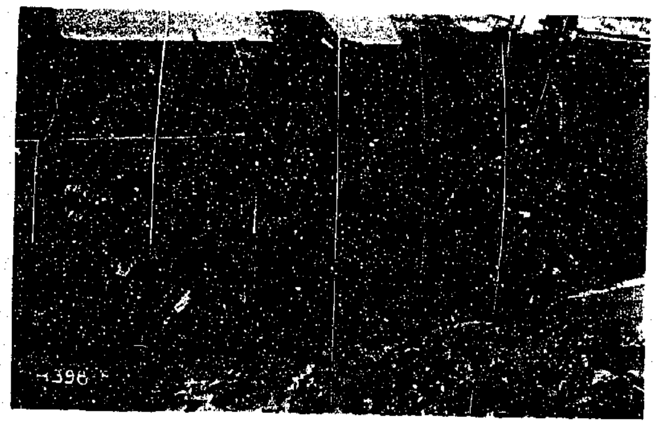


CONCHAS DAM
CONCHAS RESERVOIR
SPILLWAY
DIKE
SAND AND GRAVEL DEPOSITS AREA No. 1
DIVERSION TUNNEL
TUNNEL No. 1
Conchas City
SIPHON No. 1
SIPHON No. 2
SIPHON No. 3
SIPHON No. 4
SIPHON No. 5
SIPHON No. 6
SIPHON No. 7
SIPHON No. 8
SIPHON No. 9
SIPHON No. 10
SIPHON No. 11
SIPHON No. 12
SIPHON No. 13
SIPHON No. 14
SIPHON No. 15
SIPHON No. 16
SIPHON No. 17
SIPHON No. 18
SIPHON No. 19
SIPHON No. 20
SIPHON No. 21
SIPHON No. 22
SIPHON No. 23
SIPHON No. 24
SIPHON No. 25
SIPHON No. 26
SIPHON No. 27
SIPHON No. 28
SIPHON No. 29
SIPHON No. 30
TUNNEL No. 2
TUNNEL No. 3
TUNNEL No. 4
CHECK AND WASTEWAY
SAND AND GRAVEL DEPOSITS AREA No. 2
CONSTRUCTION ROAD
SOUTH CANAL
SOUTH CANADIAN RIVER
SOUTHERN PACIFIC
BELL RANCH LINE
R.26E.
R.27E.
R.28E.
R.29E.
R.30E.
R.31E.
R.32E.
T.14N.
T.13N.
T.12N.
T.11N.
T.10N.
Bascom
Hudson
SAN MIGUEL COUNTY
QUAY COUNTY
BOUNDARY OF THE ARCH HURLEY CONSERVANCY DISTRICT
C.R.I. AND P.
HUDSON CANAL
PROJECT BOUNDARY
TUCUMCARI
TUCUMCARI AIRPORT
Dry Lake
Revuelto Creek
Tucumcari
Plaza Larga
Barrancas
Leebie
END
EXPLANATION
Irrigable areas
SCALE OF MILES
N
LOCATION MAP
OKLAHOMA
NEW MEXICO
TEXAS
Raton
Dawson
SANTA FE
Las Vegas
CONCHAS DAM
Tucumcari
Amarillo
Borger
Dalhart
CANADIAN RIVER
PROJECT AREA
Albuquerque
Belen
Santa Rosa
ALAMOGORDO DAM
Ft. Sumner
Clovis
Lubbock
Altus
RED RIVER
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TUCUMCARI PROJECT - NEW MEXICO
GENERAL PROJECT MAP
DRAWN
TRACED
CHECKED
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO

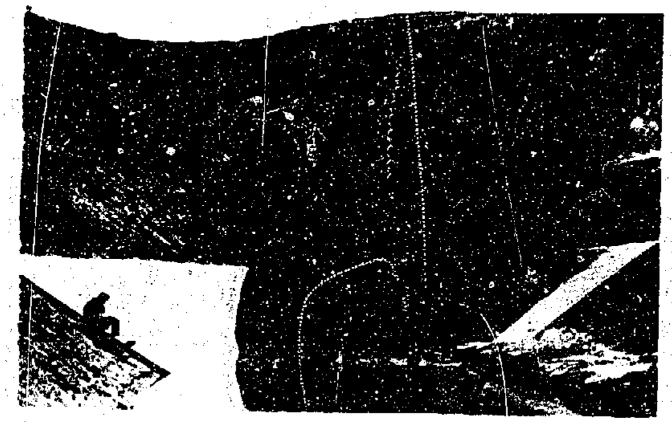
A - Cavity under rock in canal bank near siphon No. 17



B - Looking downstream at gaging station No. 1



C - Silt at gaging station No. 1



D - Looking downstream at tunnel No. 1

CONCHAS CANAL - TUCUMCARI PROJECT

PURPOSE OF INVESTIGATION

Scope of study. The study was undertaken to determine the amount of loss by seepage, locate the critical area contributing to the loss, and define the effectiveness of any lining installed. Rating curves for future operation of the canal will also be prepared. An indication of the probability of the irrigable lands becoming water-logged may be seen from the proposed observations.

Summary and recommendations. It is recommended that the required work be performed on the canal to insure proper hydraulic characteristics of the gaging stations. If time prevents accomplishment of the necessary grading and installation of riprap in the vicinity of gaging station 1, the current meter measurements should be taken a short distance downstream in the transition of tunnel No. 1. The unlined canal section upstream from any transition utilized as a gaging station should be brought to proper grade to prevent disturbance of the flow. Since these areas cannot be stabilized with riprap prior to the irrigation season, the canal should be graded for a short distance as proposed by the acting construction engineer.

MEASURING SEEPAGE LOSS

Methods of measuring seepage losses. There are seven generally known methods of measuring the amount of water lost from a canal by seepage, namely, (1) current meter measurements of inflow and outflow in reaches of the canal between established rating stations, (2) use of weirs to measure inflow and outflow in reaches of a canal, (3) determining the drop in water surface during a given time in short reaches of the canal isolated by tappoons, (4) measurements with a laboratory seepage meter at various points in the bed of the canal while water is flowing, (5) constant or variable head permeameter measurements at various points in the canal under the condition of no flow, (6) laboratory tests on undisturbed cohesive soils or remolded granular soils from representative points in the periphery of the canal, and (7) by computing the flow from the slope of the ground water surface observed in test wells adjacent to the canal while flowing.

Current meter measurements. To determine the loss in any particular reach of the canal by utilising a current meter, it is necessary to establish gaging stations at each end and measure the discharge. Neglecting evaporation, the difference between the two quantities of flow will represent the loss by seepage. If the observations are taken in a short period of time the effect of evaporation may be omitted.

The main disadvantage of this method is that the accuracy is sufficient only for a considerable length of the canal producing an appreciable differential discharge. The measured losses represent an average throughout the entire reach being considered, while, actually the permeability of the soil may vary throughout the length under observation.

Weir measurements. The procedure for using weirs to determine the differential flow at the extremities of a reach in a canal is similar to the current meter method. The weir provides a high degree of accuracy but the installation requires considerable time and expense. The backwater produced may be sufficient, in some cases, to overflow the banks thereby limiting the capacity of the canal.

Drop of water surface in a segregated canal section. The loss may be quite accurately determined by segregating a section of the canal with tappoons and observing the drop in water level during a given period of time. The quantity lost can then be computed, but this method is only applicable during the season when irrigation water is not required. Time does not permit utilization of this procedure on the problem at hand.

Measurements with a laboratory seepage meter. The laboratory seepage meter consists of a cylinder covered with a cone having a valve at its apex, an upper cylinder, a flexible bag, and a spring balance. To operate, the lower cylinder is forced into the canal bed in which water is flowing, the valve at the top of the cone being open allows entrapped air to escape and be replaced by water. The bag is then filled and connected by tubing to the lower cylinder and vented to the atmosphere to facilitate refilling the bag. The flexible bag is then placed in the upper cylinder and any water seeping from the lower cylinder through the canal bed is replaced by flow from the bag which must be submerged during the test. The loss of weight of the bag as determined by the spring balance is equal to the weight of water lost through the area of the canal bed confined by the lower cylinder in a given time. By using Darcy's law, as explained later, the seepage loss or soil permeability may be computed.

The main disadvantage to this method is that the obtained permeability is applicable to only one point in the section, however, the average of a large number of tests will minimize this error.

Constant or variable head permeameter, or laboratory measurements. The determination of the permeability of a soil by the constant head permeameter method involves adding a known quantity of water to maintain a constant head for a given time in a cylinder imbedded in the bottom of the canal. Again Darcy's law is used to compute the seepage losses, but is applicable to only the small area under the cylinder. Since this apparatus is not ordinarily used with water flowing in the canal, a correction should be applied to compensate for the radial flow which cannot exist if the canal is carrying water. However, no method is known to determine the amount of correction. An additional disadvantage exists in that the flow length through the soil ordinarily cannot be determined.

The variable head permeameter method possesses the same disadvantages but is more applicable to soils of very low permeability. It is similar to the previous system except the measured drop of water level in the cylinder is observed for a given time.

By Darcy's law the volume of flow through the soil in time, dt, is $K(\frac{h_f}{l})$ Adt which is equal to the decrease in volume of water in the tube above the cylinder, therefore:[2] $- adh = K(\frac{h_f}{l})Adt$

where, h_f = loss of hydraulic head in the flow length, l.

dh = loss of hydraulic head in time, dt

K = coefficient of permeability

l = flow length of water through soil

A = area of cross-section of cylinder driven into soil

a = area of cross-section of small tube above cylinder

By integrating between the limits of h_1 and h_2, and t_1 and t_2 and converting natural logarithm to base 10:

$$K = \frac{2.3 . a\ l}{A(t_2\ t_1)} \log._{10} \frac{h_1}{h_2}.$$

The laboratory tests for determining permeability on undisturbed cohesive soils are conducted in a manner similar to the constant or variable head permeameter methods. This procedure has an additional disadvantage to those listed for the constant or variable head methods in that the undisturbed soil sample is difficult to obtain.

10. Determination of loss by observing slope of ground water. The slope of the ground water caused by seepage from a canal may be obtained by observing the water surface in test wells drilled on a line normal to the flow of the canal and at varying distances from the canal bank. From Bernoulli's theorem,

$$\frac{V_1^2}{2g} + \frac{p1}{w} + Z_1 = \frac{V_2^2}{2g} + \frac{p2}{w} + Z_2 + h_f$$

where, $\frac{V_1^2}{2\ g}$ = velocity head at point 1.

$\frac{V_2^2}{2\ g}$ = velocity head at point 2.

$\frac{p_1}{w}$ = pressure head at point 1.

$\frac{p_2}{w}$ = pressure head at point 2.

Z_1 = position head at point 1.

Z_2 = position head at point 2.

h_f = hydraulic head lost due to friction in flow between points 1 and 2.

Since the velocity of water flowing in the soil is very small, the velocity head may be neglected. Hence, by solving Bernoulli's equation for the head lost in friction,

$$h_f = \left(\frac{p_1}{w} + Z_1\right) - \left(\frac{p_2}{w} + Z_2\right)$$

From Darcy's law $V = K \frac{h_f}{l}$ . . . . (1)

where, V = velocity of flow in saturated soil.

K = coefficient of permeability of the soil.

h_f = hydraulic head lost between any two points.

l = length of soil through which flow occurs.

Substituting the value of h_f into equation (1).

$$V = K\left[\frac{\left(\frac{p_1}{w} + Z_1\right) - \left(\frac{p_2}{w} + Z_2\right)}{l}\right]$$

and since $Q = A\,V$,

$$Q = A\,K\left[\frac{\left(\frac{p_1}{w} + Z_1\right) - \left(\frac{p_2}{w} + Z_2\right)}{l}\right]$$

Each item in this equation may be easily measured except the value of K. Various methods exist for determining K but all are quite involved and of questionable exactness. The amount of time necessary to use this method makes its application undesirable for the problem at hand.

After considering the advantages and disadvantages of the various methods for ascertaining the seepage loss, the current meter procedure appears to be the best for the existing problem. This decision is also influenced by the large number of structures on the Conchas Canal which will provide gaging stations thus enabling immediate inception of the program.

THE INVESTIGATION

Field inspection. To assist in establishing current meter measuring procedures and gaging stations, the author visited the Tucumcari Project February 4-6, inclusive. The canal had been operated at various discharges since June and at the time of inspection the quantity varied from approximately 35 second-feet with one valve of the headworks 0.1 foot open to 60 second-feet with a valve opening of 0.2 feet. As far as could be determined visibly, no silt was entering the canal from the reservoir, however, the small flows eroded silt from the canal bottom as evidenced by the red color of the water. The small amount of silt in the canal apparently had been deposited under conditions of higher discharge when the velocity near the canal bottom was smaller.

Existing gaging stations. Two permanent gaging stations have been provided in the canal, one being near the headworks at station 0 + 60, (figure 2B) and the other at station 2075 + 04. The details of construction are shown on figure 3. At gaging station 1 the concrete floor is covered with silt and moss to a depth of approximately 12 inches making it impossible to accurately measure

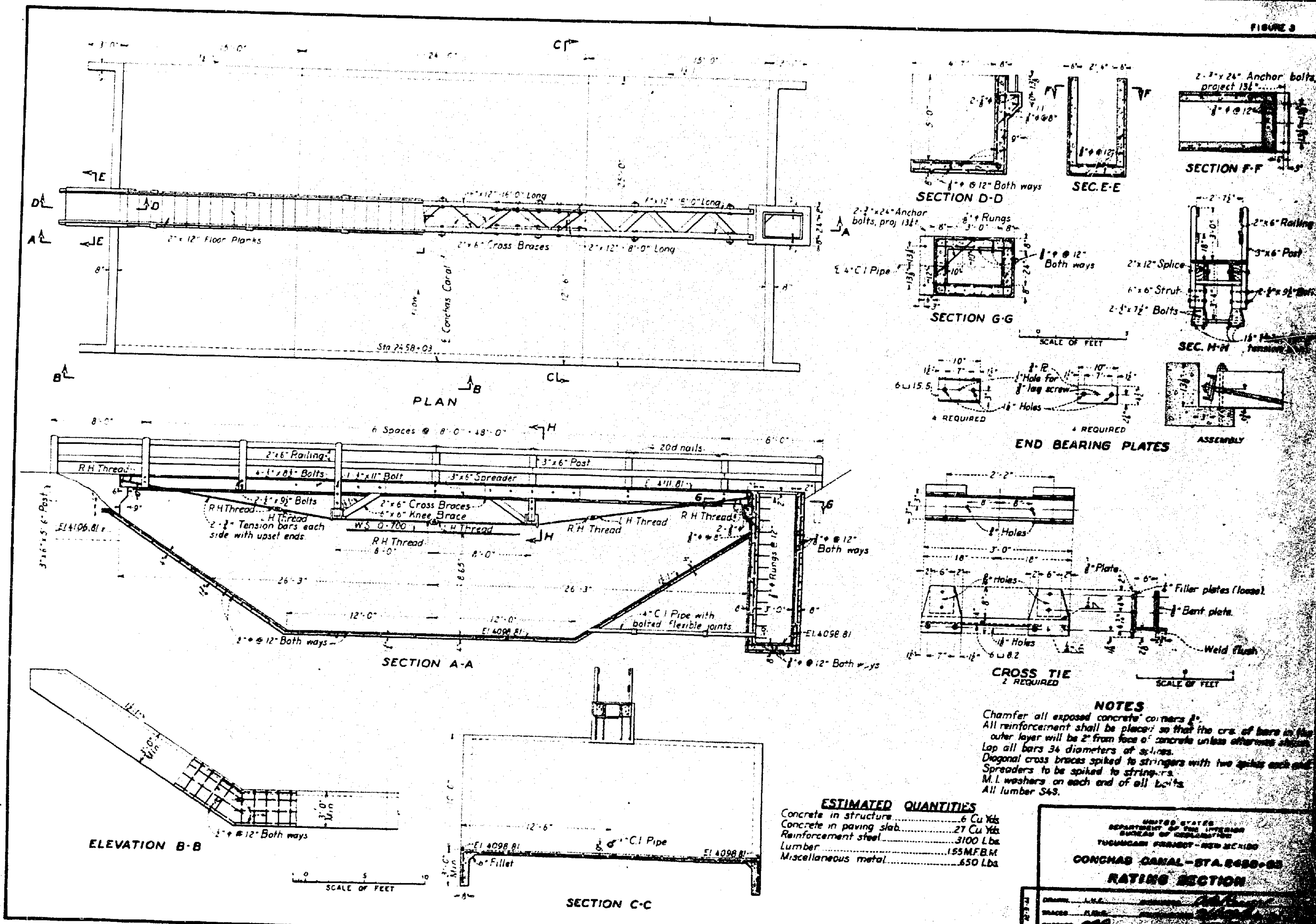


FIGURE 3
PLAN
SECTION A-A
ELEVATION B-B
SECTION C-C
SECTION D-D
SEC. E-E
SECTION F-F
SECTION G-G
SEC. H-H
END BEARING PLATES
CROSS TIE
2 REQUIRED
ASSEMBLY
4 REQUIRED
SCALE OF FEET
2"x 12" Floor Planks
2"x 6" Cross Braces
2"x 12" - 8'-0" Long
2"x 12" - 16'-0" Long
Sta 24 58 + 03
℄ Conchas Canal
FLOW
6 Spaces @ 8'-0" = 48'-0"
2"x 6" Railing
3"x 6" Post
3"x 6" Spreader
20d nails
R H Thread
L H Thread
2 - 3/4" Tension bars each side with upset ends
W.S. Q-700
El 4106.81
El 4098.81
4" C.I. Pipe with bolted flexible joints
Both ways
Rungs
2-3/4"x 24" Anchor bolts, project 13 1/2"
2"x 12" Splice
6"x 6" Strut
Filler plates (loose)
Bent plate
Weld flush
Holes
Plate
Fillet
C.I. Pipe
NOTES
Chamfer all exposed concrete corners 3/4".
All reinforcement shall be placed so that the crs. of bars in the outer layer will be 2" from face of concrete unless otherwise shown.
Lap all bars 34 diameters at splices.
Diagonal cross braces spiked to stringers with two spikes each end.
Spreaders to be spiked to stringers.
M.I. washers on each end of all bolts.
All lumber S4S.
ESTIMATED QUANTITIES
Concrete in structure .... 6 Cu Yds.
Concrete in paving slab .... 27 Cu Yds.
Reinforcement steel .... 3100 Lbs.
Lumber .... 1.55 M.F.B.M.
Miscellaneous metal .... 650 Lbs.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TUCUMCARI PROJECT - NEW MEXICO
CONCHAS CANAL - STA. 2458+03
RATING SECTION

the depth. The accuracy of the automatic recorder is questionable since the inlet to the float well is submerged in silt. Figure 2C shows the deposit on the concrete lining of the gaging station. The alignment of the canal has changed due to erosion by storm water flowing down the banks. The result is an unequal velocity distribution across the section. Another factor contributing to inaccuracies of the measurements is the pool of water which stands in the gaging section under the condition of no flow.

To accurately measure the water at station 1 will necessitate regrading the entire canal from the stilling pool of the headworks to the transition of tunnel No. 1. To insure stability, the riprap downstream from the stilling pool should be extended to the gaging station. For a distance of 25 feet upstream from the station the riprap should be hand placed. The importance of this station justifies the expense required to perform the necessary work as it is the basis for measurement of water into the entire project. A copy of the rating data is furnished the Corps of Engineers for their use in controlling the reservoir. If the work necessary to place this station in proper condition cannot be accomplished this season, the current meter observations should be taken a short distance downstream at the transition entrance to tunnel No. 1 shown on figure 2D. However, the deposit on the right side of the transition should be removed and the unlined section made flush with the concrete.

The Bureau is obligated to maintain a full canal for irrigation purposes, beginning April 1, by use of the check and wasteway upstream from siphon No. 26. The backwater from this check will probably extend to siphon No. 20 making the velocities at gaging station 2 of insufficient magnitude to accurately measure with a current meter. Accordingly, this station will serve no purpose in the present study.

<u>Gaging stations at transitions</u>. It is believed the concrete transitions to the numerous siphons and tunnels afford the desired hydraulic conditions for current meter measurements in that the section

flow depth. The accuracy of the automatic recorder is questionable since the inlet to the float well is submerged in silt. Figure 2C shows the deposit on the concrete lining of the gaging station. The alignment of the canal has changed due to erosion by storm water flowing down the banks. The result is an unequal velocity distribution across the section. Another factor contributing to inaccuracies of the measurements is the pool of water which stands in the gaging station under the condition of no flow.

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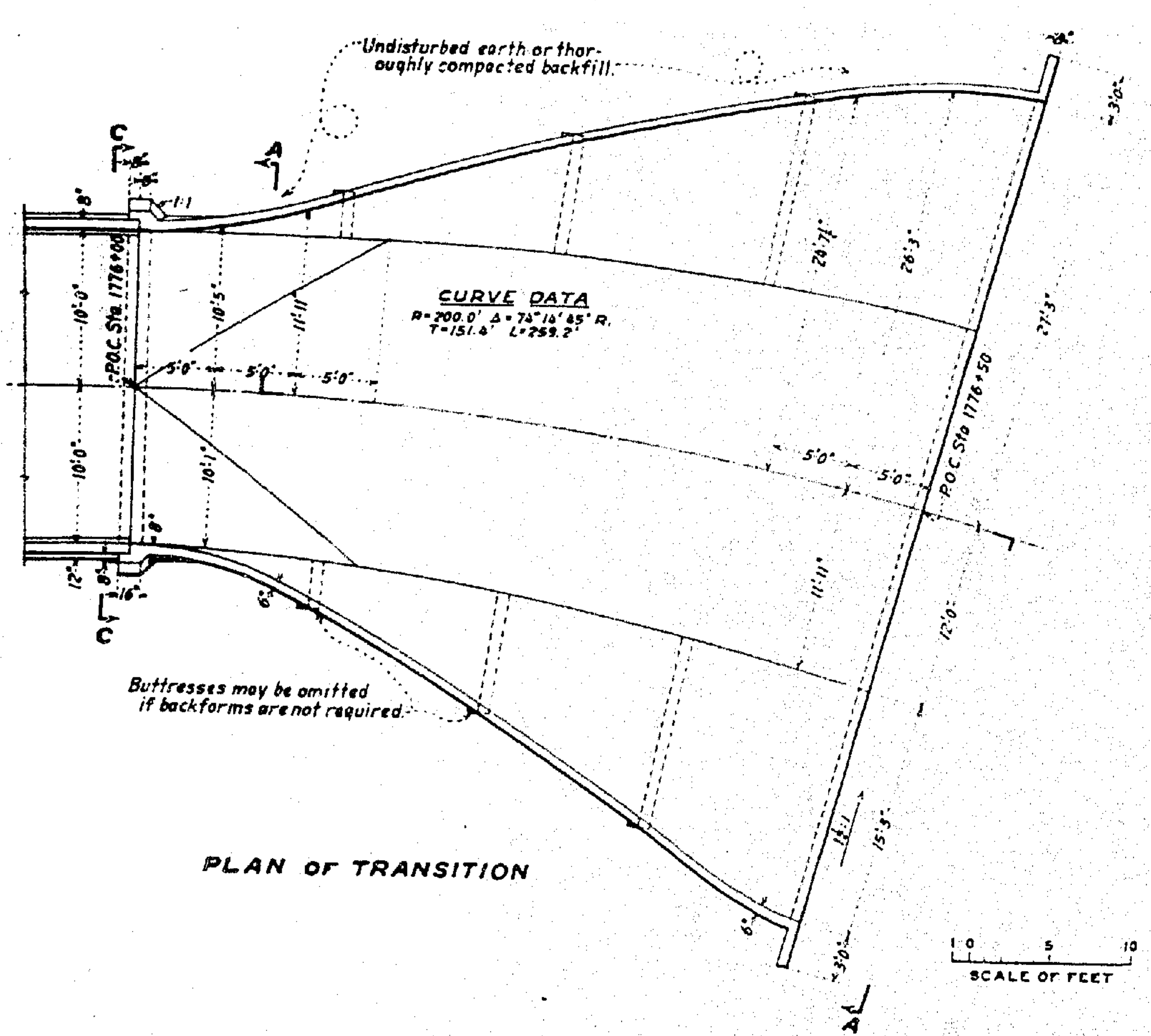
Gaging stations at transitions. It is believed the concrete transitions to the numerous siphons and tunnels afford the desired hydraulic conditions for current meter measurements in that the section is converg[illegible]h fixed boundaries and a control exists immediately downstream [illegible]nsure constantly increasing gage heights for increasing disch[illegible]. Figure 4 shows the design of a typical transition.

[illegible]dition of canal upstream from transitions. At the upstream end of nearly every concrete transition, erosion has resulted in a hole being formed in the bottom and sides of the canal. The largest of these holes is approximately six inches deep and extending several feet upstream from the edge of the concrete. It is understood that the original specifications provided riprap for these areas but this portion of the construction was omitted. As a result of the holes, a disturbance in the flow occurs which in some instances continues through the entire length of the transition. Current meter measurements in this disturbed flow would unquestionably be in error. It is strongly recommended that the canal section be protected with riprap for a distance of 50 feet upstream from any transition containing a gaging station. The 25 feet of riprap nearest the concrete should be hand placed to insure a smooth approach to the station.

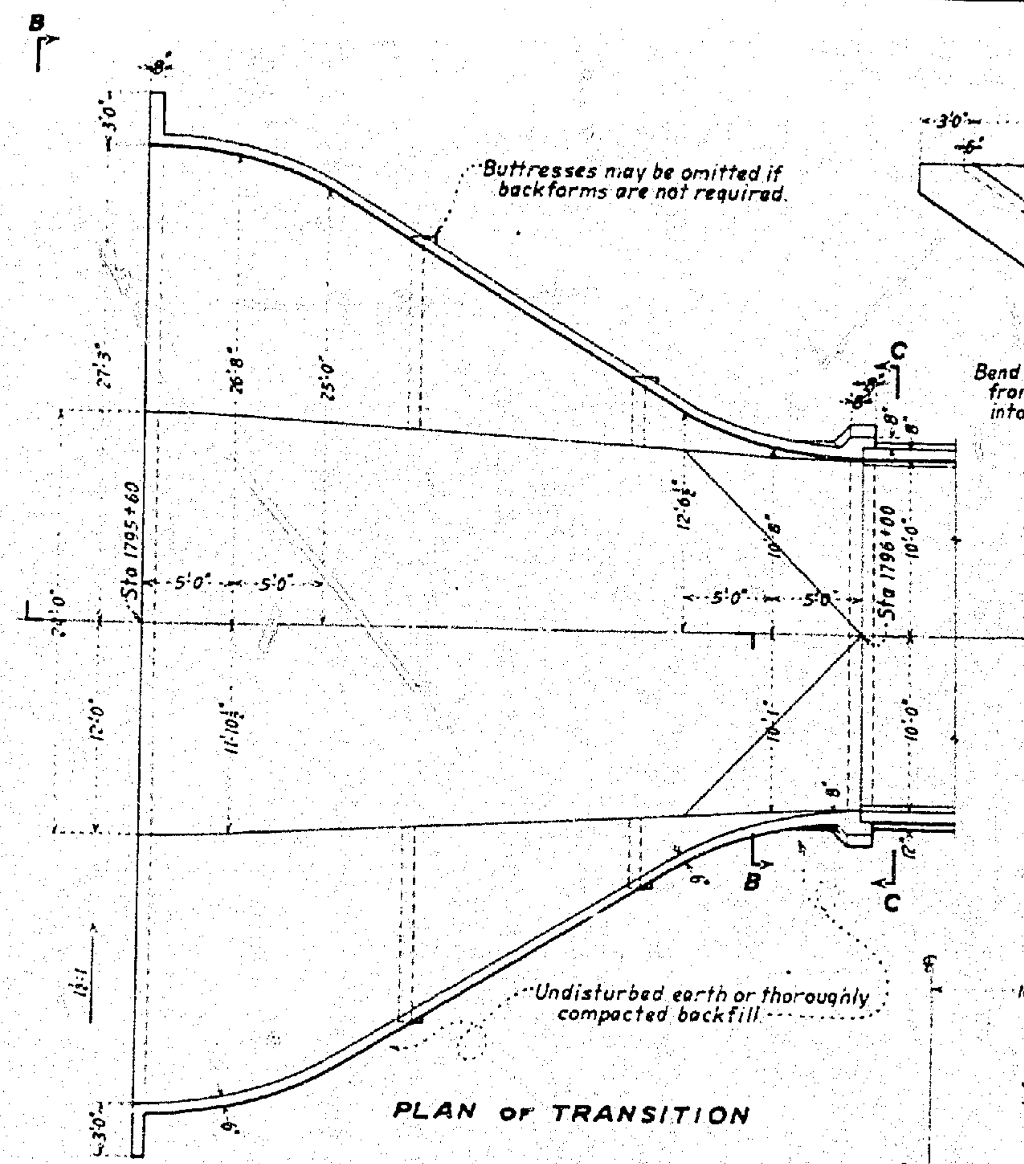
Observed leakage from the Conchas Canal. The only visible leakage from the Conchas Canal during the field visit, occurred between siphon No. 15 and tunnel No. 4, the greater part being near siphon No. 17 (Siphon No. 16 was not constructed). The flow from the area of greater leakage becomes concentrated in a natural channel a short distance from the canal making it possible to accurately determine the discharge by the installation of a weir. The project personnel have designed a weir box and it is understood installation will commence as soon as personnel are released from more urgent work.

A clay lining 15 inches thick is being placed through the critical area by project personnel. This lining will only extend four feet high on the canal banks to accommodate the maximum flow during the approaching season. Insufficient data exists, however, to determine the efficiency of the lining.

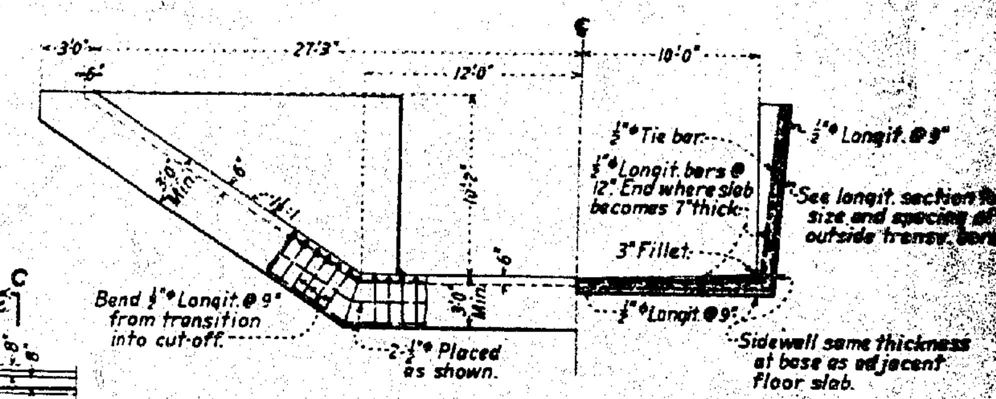
FIGURE 4



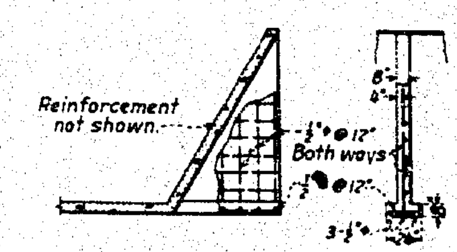
PLAN OF TRANSITION



PLAN OF TRANSITION



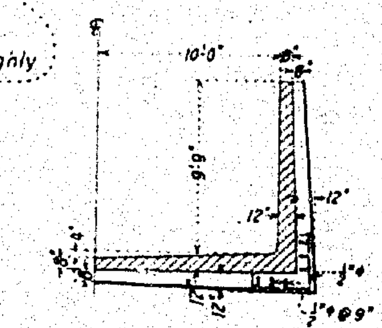
SECTION A-A
(SECTION B-B SIMILAR)



DETAIL OF BUTTRESS

HYDRAULIC PROPERTIES

SECTION	A	V	Q	r	n	s
Canal	319.83	2.19	700	5.80	.0225	.0001
Flume	168.72	4.15	700	4.65	.014	.0002



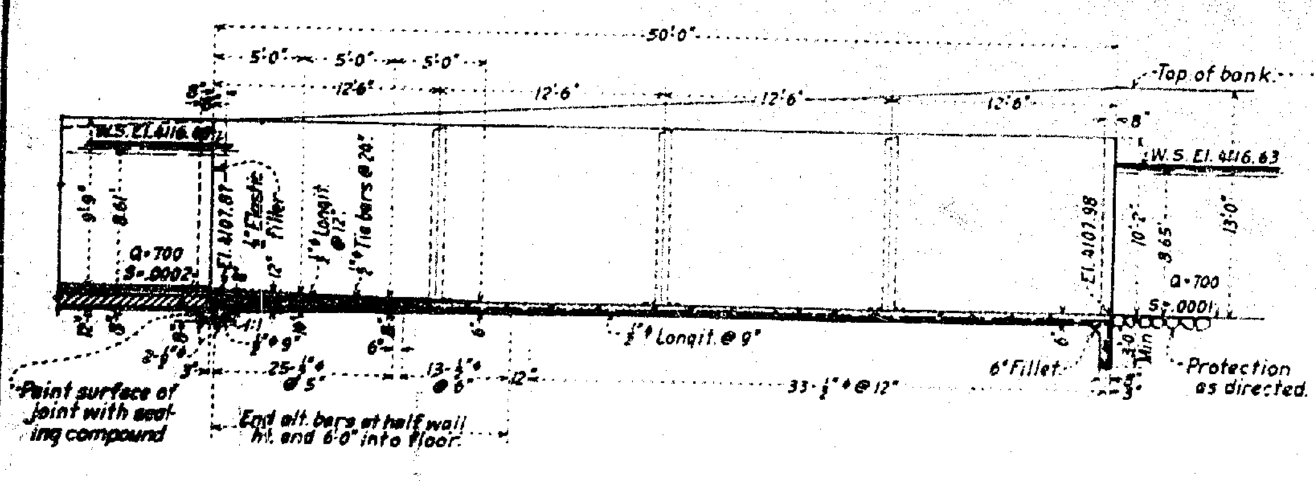
HALF-SECTION C-C

ESTIMATED QUANTITIES

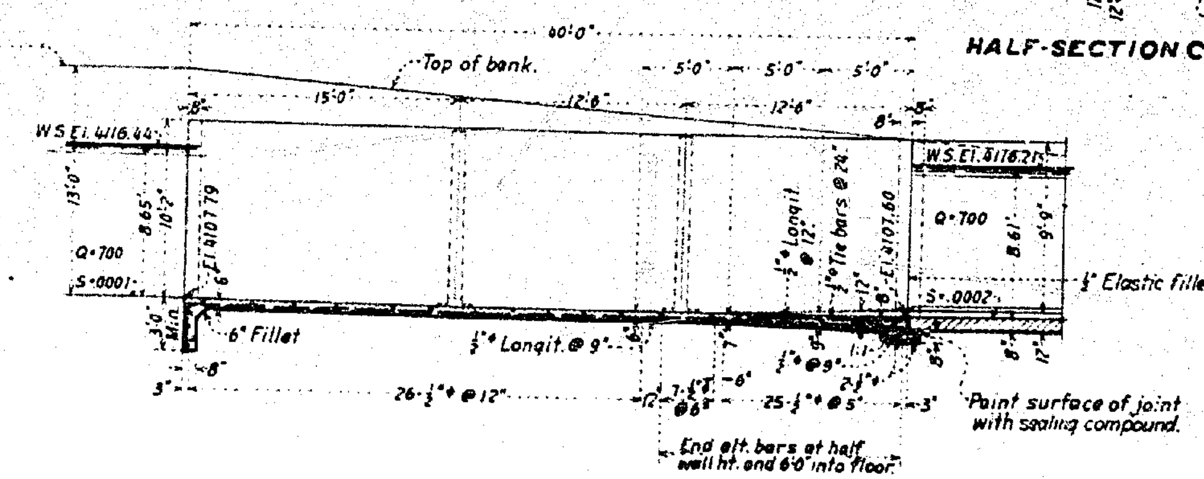
Concrete .... 110 Cu. Yds.
Reinforcement steel .... 10,200 Lbs.

NOTES

All reinforcement shall be placed so that the centers of bars in the outer layers will be 2" from face of concrete unless otherwise shown.
Lap all bars 34 diameters at splices.
Thickness of concrete to vary uniformly between dimensions shown.
Base of entire structure to be placed on undisturbed natural foundation or thoroughly compacted fill.
For transition on curve transverse dimensions are radial, and longitudinal dimensions are on canal center line.
For details of bench flume sections, see Dwg. 25?-D-[illegible].



LONGITUDINAL SECTION



LONGITUDINAL SECTION

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
TUCUMCARI PROJECT – NEW MEXICO
CONCHAS CANAL – STA. 1776+00 AND STA. 1796+00
TRANSITIONS

DRAWN: C.H.N. SUBMITTED: [signature]
TRACED: L.M.R. RECOMMENDED: [signature]
CHECKED: C.R.X. APPROVED: [signature]

DENVER, COLORADO, DEC. 14, [illegible] | [illegible]

Figure 2A indicates the amount of rock in the canal bank near siphon No. 17 and exemplifies the cavities resulting from rocks in the fill material.

<u>Location of gaging stations to measure seepage loss</u>. The minimum number of gaging stations and their location considered necessary to determine the discharge from which seepage loss may be computed is:

Gaging station 1*	Station 0 + 60
Tunnel No. 2	Station 873 + 80
Siphon No. 14	Station 1238 + 80
Siphon No. 15	Station 1357 + 95
Siphon No. 17	Station 1518 + 20
Tunnel No. 4	Station 1603 + 75

(One at farthest point downstream at which velocity may be accurately measured, probably siphon No. 21 or 22.)

After a determination is made of the amount of water lost, it may be desirable to establish additional stations to localize the principal areas contributing to the leakage. Only by experimentation will it be possible to define the most critical sections.

Observation wells are desirable in the vicinity of siphon No. 17 to establish the ground water level and study its effect on seepage. It is suggested the wells be drilled on the low side and on lines perpendicular to the center line of the canal. Three lines of wells are proposed with one set 150 feet upstream from the end of the clay lining presently being placed, a second row 300 feet downstream from the first one, and a third row at some convenient point farther downstream. To obtain a representative indication of the ground water table, the wells in each line should be 25, 75, and 200 feet from the centerline of the canal.

*Tunnel No. 1 at station 49 + 50 should be used under the condition discussed in section 12.

Existing wells in the irrigated lands are now being observed to determine the water surface elevation. Although the wells are constantly being pumped, the data will be especially helpful in determining the trend of the ground water table over a period of considerable time.

17. Factors contributing to successful measurements. Several factors in addition to those previously mentioned should receive proper consideration to insure accuracy in current meter measurements[3]. Overemphasis cannot be placed upon the factors contributing to the proper characteristics of a gaging station. A station should be approached by a considerable length of straight canal to insure uniform velocity distribution. The measuring section should be free of silt and moss to enable accurate determination of the depth. Care must be exercised to keep the bucketwheel of the meter in the same plane of the transition to avoid error due to the change in velocity of the water entering the transition. The chosen station should be marked to permit future measurements at exactly the same section. A rigid staff gage near the station with readings taken before and after each set of measurements and more frequently if the water surface varies appreciably, will insure correct depth of flow. Due to limitations of the current meter, velocities less than 0.5 foot per second cannot be accurately measured and if such velocities occur through more than 15 percent of the cross section the data should be discarded. The one-point (0.6 depth) method should be used for depths less than two feet while the two-point (0.2 and 0.8 depth) method is more accurate for measuring velocities in greater depths. The verticals chosen for velocity measurements are dictated by good practice to be spaced one foot from the side and at every break in slope of the canal bottom except no spacing is to exceed two feet.

After a change in the valve opening of the headworks, sufficient time must elapse prior to measurements to permit steady flow. Upon completion of observations pertaining to any particular reach of the canal, an inspection of the air bleeders for the siphons will reveal if any discharge of water has occurred to destroy the accuracy of the data. Correction for any discharge from a wasteway may be considered by measuring the flow in the waste channel or observations in the canal each side of the structure.

Another factor which should be investigated results from the cattle water tanks being supplied from the main canal by a 1-1/2 - 2 inch pipe. An inspection of one of these units revealed a constant quantity of water was being drawn from the canal by permitting the tank to overflow, while at another installation a float-operated valve has been provided to maintain a constant elevation in the tank and prevent waste. The quantity being consumed by a unit controlled by an automatic float valve is negligible while the amount drawn from the canal with no control on the supply line may be appreciable. It is recommended that the number of continuously operating units be determined and the total discharge ascertained by measuring the flow through a single unit with an ordinary water meter.

Current meter equipment. The current meter equipment existing on the project is inadequate since the only rating table is for the meter suspended 4.9" above a 30# Columbus weight. Accordingly, readings taken with the wading rod or 15# weight are in error.[4] Observations previously made with suspensions other than the one for which a table exist could be corrected by having the meter rerated. However, the data obtained does not appear sufficient to justify the additional work. Additional equipment including a Morgan hand reel and weight pins will simplify the mechanics of operation.

Automatic recorder. The two permanent gaging stations are equipped with automatic recorders. An error will result through use of these gages unless the pipe connecting the float well with the canal is kept free of silt. To obtain maximum accuracy a correction must be made for the lag of the float between the rising and falling stage and the effect of the line shift[5].

Correction due to rising and falling stage:

Let X = depth of floatation of float with counterpoise in air.

r and f denote rising and falling stage.

F = force required to operate instrument.

w = unit weight of water.

A = area of float.

$X_r - X_f = \frac{2F}{wA}$ or if the float is circular

$$X_r - X_f = \frac{8F}{w \pi D^2}$$

In practice the indicator may be set to give true height for a rising stage, then $\frac{8F}{w \pi D^2}$ can be subtracted from the reading at the same gage height on the falling stage to obtain true height. For example, let

F = 1 ounce.

D = 8 inches.

then $X_r - X_f = \frac{\frac{1}{16}}{\left(\frac{8}{12}\right)^2 (3.1416) 62.5} = 0.0057$ ft

which is the correction factor.

Correction due to line shift:

Let u = weight of line per foot.

h_o = elevation of liquid or gage height before shift of line from one side to the other.

h_1 = elevation of liquid or gage height after shift from one side to the other.

$$d\,x = -\frac{2\,u}{w\,A}\;(h_1 - h_o).$$

For example,

Assume the line has a weight of 0.0065 lbs/ft.

$D = 8"$

$h_1 - h_o = 10'$

Then $$dx = -\frac{2\,(0.0065)\qquad(10)}{62.5\;\frac{(8)^2}{(12)}\;\frac{(3.1416)}{4}} = -\,0.006'$$ which

is the amount of correction due to line shift.

Submergence of counterpoise:

If the counterpoise becomes submerged, a correction must be made from the equation,

$$X' - X = \frac{C}{S_c\;w\;A} \quad \text{where,}$$

X' = Depth of floatation of float with counterpoise submerged.

S_c = Specific gravity of counterpoise.

C = Wt. of counterpoise.

Many manufacturers of stage recorders will furnish, upon request, tables showing the corrections for their instruments.

<u>Computation of discharge from current meter data</u>. The discharge obtained by a current meter is ordinarily computed from the

equation $Q = b_1 \frac{(d_o + d_1)}{2} \frac{(V_o + V_1)}{2} + b_2 \frac{(d_1 + d_2)}{2} \frac{(V_1 + V_2)}{2} + \cdots$

$$bn \frac{(d_{n-1} + d_n)}{2} \frac{(V_{n-1} + V_n)}{2}$$

Where b_1, b_2, b_3 —— b_n are the spaces between the verticals, d_o, d_1, d_2, —— d_n are the depths at the respective measuring points, and V_o, V_1, V_2 —— V_n the velocities at the corresponding measuring points.

The discharge may also be computed graphically by plotting, from a common reference, the mean velocity and soundings. The profile of the section will be obtained from the plotted soundings, while the velocity curve will give the "transverse" velocity. The mean velocity and mean depth is scaled from the plot for each partial area. The product of the corresponding velocity, depth and width will give the discharge through the area being considered. The summation of the partial discharges will give the total quantity. For convenience, the soundings are plotted below the water surface as a reference line while the velocities are plotted above the same line. This method has the advantage of obviating erroneous readings.

Analysis of discharge measurements. By making an analysis of discharge measurements, irregularities may appear which would otherwise be unnoticed. One such study requires plotting the area against the gage height to obtain an area curve. This curve may be expressed in the form of an equation which for the trapezoidal section of the Conchas Canal will be:

$$A = BD + 1/2\, D^2(\tan\theta + \tan\phi)$$

where D = depth

B = bottom width

θ and ϕ are the angles the side slopes make with the vertical.

A = area of cross-section

By differentiating,

$$\frac{d\ A}{d\ D} = B + (\tan\theta + \tan\phi)\ D$$ which is

the slope of the area curve.

Again differentiating and multiplying by A.

$$\frac{d^2 A}{d\ D^2}\ (A) = 2\ (\tan\theta + \tan\phi)\ A.$$

If the value of $\frac{d^2 A}{d\ D^2}$ (A) is positive, the curve is concave toward the A axis. This value will always be positive for the studies under consideration.

To find the slope of the area curve at any point,

Let, w = width at any stage.

A = Corresponding area of the stream,

D = corresponding water depth above lowest point of bed.

For any small change, d D, in the depth, D, the area will be changed by an amount, $dA = w.dD$ since the width and depth may be taken as constant for the differential rise. The slope of the area curve may then be expressed by $w = \frac{d\ A}{d\ D}$. Therefore, for any stage, the width of the stream is equivalent to the slope of the area curve at that stage. A knowledge of the slope of the curve will be helpful in its construction. However, a method for laying off the tangent so that the scale may be taken into account is necessary. A satisfactory procedure is to let d D = 1 foot, hence, w = d A. Next lay off 1 foot from the plotted point along the gage axis to the scale of gage heights and at the end of this distance, lay off w along the area axis to the scale of areas. The line joining the end of the w line and the plotted point will have the desired slope. For a canal with flat bottom, the curve at the origin will make an angle with the gage axis the slope of which will be equal to the bottom width.

By determining the slope in this manner for the various plotted points the curve may be drawn more accurately and correction made to any erroneous readings.

CONCLUSIONS

Limitation of present study. The present study can only include the portion of the canal upstream from approximately siphon No. 21, or the limit established by the backwater. Since the project is not complete, the canal will flow only at 50 percent capacity or less. Accordingly, the seepage loss will be materially less than anticipated at maximum discharge. With proper precautions, sufficient data can be obtained to accomplish the purpose of the study and establish a criteria of the anticipated conditions upon completion of the project.

Project personnel were fully aware that the present condition of the canal at the gaging station and entrance to the transition sections is unsatisfactory. The situation exists because of the limited maintenance crews and time available. In the opinion of the writer, all possible corrective measures are being taken but time does not permit completion of the scheduled program prior to the approaching irrigation season.

Continuation of program. It is of particular importance that sufficient data be obtained to determine the effectiveness of any lining by determination of loss both before and after any measures are taken to decrease the seepage. To establish the economy of any lining requires the maintenance of accurate cost data. These data should include cost of flow determinations as well as installation of lining by Government forces or contract.

The program should be continued until the project is completed permitting the canal to operate under full head. If the enlarged canal lining program presently being considered by the Bureau is approved, it is recommended that the determination of the seepage losses on the Tucumcari Project be given a broader aspect.

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