TECHNICAL MEMORANDUM 661

United States Department of the Interior
Bureau of Reclamation

ANALYSES AND DESCRIPTIONS OF CAPACITY
TESTS IN LARGE CONCRETE-LINED CANALS

by

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Office of Chief Engineer
Denver, Colorado
April 1964
6-12-64

Dear [Name],

I am just writing to convey my best wishes to you in your new position. I understand that it is going to be a challenging one, but I am confident that you will excel in it. Please don't hesitate to contact me if you need any assistance.

Sincerely,
[Your Name]
Crews are shown here obtaining water surface profile and discharge measurements in a straight reach of the 4,500-cfs concrete-lined East Low Canal on the Columbia Basin Project in Washington. A temporary cableway was set up for current meter discharge measurements, and special steel platforms were built to enable the crews to reach the water surface. This photograph was taken early in the irrigation season when the canal was conveying only 44 percent of design capacity. P222-116-42209, March 30, 1960.
ABSTRACT

Between 1957 and 1962, the Bureau of Reclamation made tests to determine flow capacities and resistance coefficients in 9 large concrete-lined irrigation canals. Tests were made on some 170 miles of trapezoidal-shaped canals having flows which varied from 555 cfs to 6,820 cfs. Design discharges for test canals varied from 700 to 13,200 cfs. Other design hydraulic properties included invert slopes which ranged from 0.0013 to 0.00005, bottom widths from 8 to 50 feet, hydraulic radii from 4 to 14 feet, and velocities from 5 to 12 fps. All concrete linings, except one, were placed by rail-mounted traveling slip forms and were from 7 to 25 years old at the time of tests. Head loss measurements were made across piers and inverted siphons. Test data were analyzed in terms of "n" values for Manning's formula and on Reynolds' Number - friction factor plots. Resistance coefficients varied with the amount of aquatic growths, canal alignment, and canal size. Manning's "n" generally varied from 0.013 to 0.016 for the smaller canals and from 0.015 to 0.019 for the larger canals. Aquatic growths were found in varying amounts on lining surfaces of all canals and caused seasonal variation in flow resistance. Biweekly copper sulfate treatments retarded the most prevalent growth, filamentous algae. A hydraulic design procedure for concrete-lined canals is outlined. Design procedures of other agencies and recent literature on flow in rigid boundary channels are summarized and reviewed. An appendix contains detailed descriptions of tests and operating experiences in each canal.

DESCRIPTORS-- canals/ *roughness coefficients/ open channel flow/
*analysis/ *flow resistance/ rigid linings/ trapezoidal channels/
check structures/ siphons/ bridge piers/ *canal design/ *Manning's
formula/ *head losses/ *algae/ horizontal curves/ research and
development/ relative roughness/ subcritical flow/ aquatic life/
fluid friction/ field data/ hydraulics/ chemicals
IDENTIFIERS-- *concrete-lined canals/ test reaches/ Main Canal/ East
Low Canal/ Delta-Mendota Canal/ Friant-Kern Canal/ Roza Main Canal/
Madera Canal/ Gateway Canal/ Charles Hansen Canal
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<tr>
<td>A</td>
<td>Area of water cross section</td>
<td>sq ft</td>
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<tr>
<td>B</td>
<td>Bottom width of canal</td>
<td>ft</td>
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<tr>
<td>C</td>
<td>Chezy coefficient</td>
<td>ft$^{1/2}$/sec</td>
</tr>
<tr>
<td>cfs</td>
<td>Cubic feet per second</td>
<td>ft$^3$/sec</td>
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<tr>
<td>D</td>
<td>Pipe diameter</td>
<td>ft</td>
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<tr>
<td>d</td>
<td>Average water depth</td>
<td>ft</td>
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<tr>
<td>$d_r$</td>
<td>Water depth due to boundary shear in a straight unobstructed reach of canal</td>
<td>ft</td>
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<tr>
<td>F</td>
<td>Abbreviation for Fahrenheit temperature scale</td>
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<tr>
<td>$F_b$</td>
<td>Freeboard. Vertical distance from design water surface to top of concrete lining</td>
<td>ft</td>
</tr>
<tr>
<td>f</td>
<td>Friction factor for open channels defined as $8gRS/\sqrt{V^2}$</td>
<td>Dimensionless</td>
</tr>
<tr>
<td>$F_F$</td>
<td>Froude Number for open channels defined as $\sqrt{V^2T/gA}$</td>
<td>Dimensionless</td>
</tr>
<tr>
<td>g</td>
<td>Gravitational acceleration</td>
<td>32.2 ft/sec$^2$</td>
</tr>
<tr>
<td>H</td>
<td>Height of concrete lining above invert</td>
<td>ft</td>
</tr>
<tr>
<td>h</td>
<td>Total head loss in a given length of channel or siphon due to all causes</td>
<td>ft</td>
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<tr>
<td>$h_c$</td>
<td>Incremental head loss caused by horizontal curves in canal alinement</td>
<td>ft</td>
</tr>
<tr>
<td>$h_r$</td>
<td>Incremental head loss caused by boundary shear in a straight channel</td>
<td>ft</td>
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<tr>
<td>$h_p$</td>
<td>Incremental head loss caused by piers</td>
<td>ft</td>
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<tr>
<td>$h_v$</td>
<td>Velocity head = $V^2/2g$</td>
<td>ft</td>
</tr>
<tr>
<td>J</td>
<td>Contraction ratio for computing pier losses. See pages 23 and 24 for explanation</td>
<td>Dimensionless</td>
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<th>Symbol</th>
<th>Definition or Description</th>
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<tbody>
<tr>
<td>k</td>
<td>Equivalent sand grain diameter used in describing surface roughness</td>
<td>ft</td>
</tr>
<tr>
<td>$K_c$</td>
<td>Coefficient used to define incremental head loss caused by horizontal curves in canal alignment ($h_c = K_c h_v$)</td>
<td>Dimensionless</td>
</tr>
<tr>
<td>$K_p$</td>
<td>Coefficient used to define incremental head loss caused by piers ($h_p = K_p h_v$)</td>
<td>Dimensionless</td>
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<tr>
<td>L</td>
<td>Length of conduit or open channel</td>
<td>ft</td>
</tr>
<tr>
<td>$L_t$</td>
<td>Length of test reach along canal centerline</td>
<td>ft</td>
</tr>
<tr>
<td>N</td>
<td>Variable exponent for use in correcting canal depth for miscellaneous losses</td>
<td>Dimensionless</td>
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<tr>
<td>&quot;n&quot;</td>
<td>Coefficient of roughness for use in Manning's formula</td>
<td></td>
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<tr>
<td>P</td>
<td>Wetted perimeter</td>
<td>ft</td>
</tr>
<tr>
<td>$q$</td>
<td>Discharge</td>
<td>ft$^3$/sec</td>
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<tr>
<td>R</td>
<td>Hydraulic radius = $\frac{A}{P}$</td>
<td>ft</td>
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<tr>
<td>$R_t$</td>
<td>Radius of horizontal curve in canal alignment</td>
<td>ft</td>
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<tr>
<td>$R$</td>
<td>Reynolds' Number for open channels $\frac{4Rv}{v}$</td>
<td>Dimensionless</td>
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<tr>
<td>$R_{Ek}$</td>
<td>Reynolds' Number for open channels $\frac{2R^{3/2} \sqrt{S_c g}}{v}$</td>
<td>Dimensionless</td>
</tr>
<tr>
<td>S</td>
<td>Slope of canal invert</td>
<td>Dimensionless</td>
</tr>
<tr>
<td>$S_c$</td>
<td>Slope of energy gradient</td>
<td>Dimensionless</td>
</tr>
<tr>
<td>T</td>
<td>Top width of canal water surface</td>
<td>ft</td>
</tr>
<tr>
<td>V</td>
<td>Average velocity of flow</td>
<td>ft/sec</td>
</tr>
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</table>
LIST OF SYMBOLS AND TERMS--Continued

<table>
<thead>
<tr>
<th>Symbol</th>
<th>Definition and Description</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\Delta^o$</td>
<td>Deflection angle for horizontal curve in canal alignment</td>
<td>Degrees</td>
</tr>
<tr>
<td>$\Delta d$</td>
<td>Change in water depth</td>
<td>ft</td>
</tr>
<tr>
<td>$\Delta e$</td>
<td>Change in energy gradient</td>
<td>ft</td>
</tr>
<tr>
<td>$\Delta h_v$</td>
<td>Change in velocity head</td>
<td>ft</td>
</tr>
<tr>
<td>$\rho$</td>
<td>Mass density</td>
<td>lb sec$^2$/ft$^4$</td>
</tr>
<tr>
<td>$\nu$</td>
<td>Kinematic viscosity of water</td>
<td>ft$^2$/sec</td>
</tr>
</tbody>
</table>

Curvature Index Parameter originated by Fred C. Scobey to define relative canal sinuosity. Defined as $\frac{\sum \Delta^o}{L_1}$

ML-type Backwater Curve Subcritical flow water surface profile in a reach of channel characterized by the greatest depth at the downstream end. The general shape of the profile is shown below:

M2-type Backwater Curve Subcritical flow water surface profile in a reach of channel characterized by the greatest depth at the upstream end. The general shape of the profile is shown below:
LIST OF SYMBOLS AND TERMS--Continued

Manning's formula

Empirical formula for determining the average velocity of flow in rigid boundary open channels:

\[ V = \frac{1.486 R^{2/3} S^{1/2}}{n} \]

This formula is now used by the USBR for the design of rigid boundary channels.

Contraction ratio

Ratio of area taken up by piers in a flow prism to the gross area of the flow prism (see page 25).

W.S.

Abbreviation for "water surface".

USBR

Abbreviation for Bureau of Reclamation, U.S. Department of the Interior.

Hydraulics Branch

Organizational subdivision of the Division of Research, Office of Chief Engineer, USBR, Denver, Colorado. This branch does hydraulic laboratory work and provided technical guidance and instrumentation for capacity test measurements.

Canals Branch

Organizational subdivision of the Division of Design, Office of Chief Engineer, USBR, Denver, Colorado. This branch does canal design work and analyzed the capacity test measurements.
SUMMARY

Between 1957 and 1962 the Bureau of Reclamation conducted a comprehensive series of hydraulic tests in nine large trapezoidal-shaped concrete-lined irrigation canals in the Western United States. Linings of all test canals, except one, had been placed by traveling rail-mounted slip-forms. Purposes of the tests were:

1. To determine maximum discharge capability of each canal
2. To document seasonal changes in flow resistance caused by aquatic growths
3. To measure head losses across various types of in-line canal structures such as inverted siphons, bridge piers, and check structures
4. To document increased flow resistance caused by horizontal curves in canal alignment

Data obtained from the tests have been utilized to verify or to provide a basis for design criteria for large concrete-lined canals. In this Technical Memorandum, the authors have suggested (pages 35 through 38) a series of design procedures based on their analyses of the test data.

Design discharges in the test canals varied from 700 to 13,200 cfs. Test discharges ranged from 555 to 6,820 cfs. Three of the smaller canals were tested at only one or two discharges. These tests were made during a period of 1 to 3 days when little aquatic growth was present. Tests on the larger canals extended over a period of months or years. Regular copper sulfate treatments were used successfully in two of the canals to control aquatic growths; other canals were untreated.

Flow resistance in the largest of the test canals was greater than anticipated and varied seasonally. The value of Manning's "n" computed from tests in the five largest canals generally ranged between 0.015 and 0.019. Most values of "n" in the four smallest canals ranged between 0.013 and 0.016.

Aquatic growths or coatings of various types were found on the lining surfaces of all test canals. Filamentous algae and deposits of fresh water clams appeared to provide the greatest effect on flow resistance. The flow retardation effect of Bryozoa growths, fresh water sponges, and nonfilamentous algae was difficult to isolate.
Flow resistance in sinuous reaches of the canals generally increased notably over that which existed in straight reaches. The authors found that head losses caused by piers should not be ignored in canals having extremely flat slopes.

A more detailed list of the author's conclusions and recommendations can be found on pages 30 through 34. Possibilities for future research activities concerning flow resistance studies are outlined on pages 28 and 29.

Summary sheets of test data and computed design parameters are provided in Figures 12 through 27. More detailed descriptions of tests together with photographs of flow, conditions, and test equipment are presented in the Appendix.

A brief review and bibliography of recent literature concerning flow resistance in rigid boundary channels are provided. Replies from questionnaire letters sent to 29 engineering design groups throughout the world are summarized. These letters requested information concerning current design procedures for concrete-lined canals and data from hydraulic friction tests.
INTRODUCTION

The problems involved in the hydraulic design of a large irrigation canal are many and varied. The influence of major factors such as size, shape, surface roughness, and invert grade are balanced against the cumulative effect of a wide array of minor factors. The minor factors include aquatic growths, horizontal curves in canal alignment, structure piers in the flow prism, inlets, turnouts, and check structures. Although the specific quantitative effect of each factor has not been accurately known, the design procedures developed over a number of years and applied by engineers with a broad background of experience have usually provided acceptable results.

Operating experiences on two large concrete-lined canals (Delta-Mendota and Friant-Kern Canals) of the Central Valley Project, California, indicated in 1957-58 that design procedures which had been used successfully in smaller canals might not be adequate for large canals with flat slopes. Subsequent to the investigations at these large canals, a comprehensive program of capacity testing was undertaken by the Bureau of Reclamation to document the hydraulic performance of the more important concrete-lined canals which it had constructed. Data obtained from this program will be used to establish reliable design criteria for future canals of large size.

This Technical Memorandum presents analyses and descriptions of a series of flow tests which have been accomplished between 1957 and 1963 in some 172 miles of irrigation canals. All tests described were confined to rigid boundary concrete-lined canals. (Documentation of flow resistance in earth canals is scheduled for future work and is not described in detail in this Technical Memorandum.) Concrete linings on all of the test canals, except one, were placed by large rail-mounted traveling slip-forms, and considerable hand troweling was done to assure a smooth flow surface. Analyses and results of the tests are presented in the beginning portion of the Memorandum; more detailed descriptions and discussions of the individual tests in each canal, together with pertinent photographs, are given in the Appendix.

Summary sheets which list test data and computed design parameters for 243 water surface profile measurements are provided. Test flows in the nine canals ranged from 555 to 6,820 cfs. Design flows for these canals ranged from 700 cfs to 13,200 cfs. Tests at design discharges were desired but were not possible to obtain in all cases because of incomplete project water conveyance facilities.

Many Bureau of Reclamation offices in the Western United States contributed personnel and equipment to obtain test data. The program was guided by the Office of Chief Engineer, Denver, Colorado,
subsequent to 1959. Engineers of the Hydraulics Branch of the Denver Office developed instrumentation for and supervised initial water surface profile measurements. Descriptions and photographs of the test equipment used are briefly presented. Engineers of the Canals Branch prepared the data analyses.

The authors have presented a design chart which will enable the determination of an average value of Manning's "n" or open channel friction factor "f" for the range of concrete-lined canal sizes covered by the tests. A typical example for the design of a canal section based on this chart is given on pages 35 through 36. The procedure outlined generally results in a more conservative design for larger channels than that which has been used in the past. The hydraulic design procedure currently utilized by the USBR for concrete-lined canals is given in Chapter 1, "Canals and Related Structures," Design Standards No. 3, Bureau of Reclamation, Office of Chief Engineer, Denver, Colorado.

Results of head loss measurements across piers of the type normally encountered for bridges and other canal structures are summarized. A brief review of recent literature on rigid boundary open channel flow and associated hydraulic losses is presented as a background for the development presented.

While data were being assembled on the USBR canals, a questionnaire letter was sent to 29 water resource development agencies or firms throughout the world requesting information concerning the performance of large concrete-lined canals under their jurisdiction. These letters also requested information concerning their current design procedures. A summary of information received from 20 of these agencies is given on Figures 2 and 3. A blank space under the column headings on these figures indicates that no reply was received concerning the specific item.

Figures 2 and 3 indicate that Manning's formula is used predominantly in the United States and in most other English speaking countries. In 1957, the Bureau of Reclamation changed from the Kutter-Chezy formula to Manning's formula for hydraulic computations; therefore, subsequent discussion considers the roughness coefficient "n" which is used in the latter formula. Some form of the Gauckler Strickler or Manning formula (all of which are similar) is widely used in countries of Western Europe.
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FIGURE

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DEVELOPMENT OF TEST PROGRAM

New construction techniques and the increasing importance of water conservation in the United States have given greater impetus to the use of concrete for the lining of large canals in recent years. The Bureau of Reclamation has designed and built many hundreds of miles of concrete-lined irrigation canals in the Western United States, especially on projects in the States of California and Washington. Subsequent to 1946, several large canals were built in these areas to supply irrigated acreages of great expanse. (In the absence of a generally accepted definition, large canals are considered henceforth to be those designed for flows of 1,000 cfs or greater.)

As few large concrete-lined canals existed prior to 1946, very little hydraulic test information was available on their performance. The best test data from smaller lined canals available at that time were utilized in the design of the larger canals. Bureau designers realized that documentation of hydraulic resistance in the new large canals should be made as soon as possible. However, due to incomplete distribution facilities, water delivery demands generally do not occur until several years after the main supply canals have been completed. This creates a time lag in obtaining prototype performance tests at full design flows.

In 1957, full capacity testing of the 110-mile-long Delta-Mendota Canal in California disclosed water depths which were greater than anticipated for much of its length. Engineers of the Central Valley Project in California obtained additional test information on the Friant-Kern Canal in 1958, which again indicated water depths greater than anticipated in the original design and revealed a need for a more comprehensive test program. The Office of the Assistant Commissioner and Chief Engineer in Denver, Colorado, in 1959, initiated such a program. The objective of the test program was to document the hydraulic performance of the larger concrete-lined canals to obtain better knowledge of flow resistance and to establish design criteria for future large canals.

Between 1959 and 1963 data were obtained from nine irrigation canals which had been designed and built by the USBR in the period between 1937 and 1952. These canals are listed in Figure 1 in order of descending cross sectional area. Summaries of test data are presented in Figures 12 through 27. Detailed descriptions of the tests in each canal and pertinent photographs are provided in the Appendix.

Both straight and sinuous reaches were selected for testing in an attempt to determine the effect of horizontal curves in
canal alignment on flow resistance. Separate measurements across individual bridges, overchutes, check structures, and inverted siphons were obtained in an effort to measure the hydraulic losses caused by these in-line structures. Test reaches were selected to minimize flow changes. Records of measurable inflow and outflow were obtained and used to adjust the measured test discharges in analyses of data.

Design capacities of the trapezoidal-shaped test canals ranged from 13,200 cfs for the Trail Lake Reach of the Main Canal (Washington) to 700 cfs for the Gateway Canal (Utah). Test discharges ranged from 555 to 6,820 cfs. Invert slopes varied from 0.0013 in the Charles Hansen Canal (Colorado) to 0.00005 in the Delta-Mendota Canal. Velocities for all tests were subcritical with Froude Numbers which varied from about 0.06 to 0.84. The concrete linings on all of the canals except one were placed from large rail-mounted traveling slip-forms and considerable hand troweling was done to assure a smooth flow surface.

TEST MEASUREMENTS

Subsequent to 1959, planning for tests was coordinated by members of the Hydraulics Branch and the Canals Branch. The tests were under the control of operating personnel of the various projects. Denver Office representatives visited the canals prior to tests and selected test reaches. Hydraulics Branch engineers supervised initial measurements and instructed operating personnel in proper measurement techniques. The Hydraulics Branch also provided water surface gages, differential manometer gages, multiple current meter equipment, and other special items required for discharge and water surface profile measurements.

The "water surface gage" used for the tests was an assembly which utilized a pitot tube intake pipe connected to a vertical plastic stilling well, as shown in Figure 4. A hook gage was mounted in the stilling well to define accurately the water surface. The gage assembly was mounted on a vertical current meter support rod which was used to hold it in place on the wood and steel "diving board" platforms. The pitot tube was immersed in the water with the bottom of the support rod resting on a base plate of known elevation. Direct level checks to this base plate from a nearby bench mark were made each time a water surface measuring station was occupied. Water surface elevations were recorded to thousandths of feet. The frontispiece and Figures B-16 and F-7 show crews using this gage. The water surface gage was used for some tests on all canals except the Charles Hansen and Gateway Canals. Measurements
Water surface gage used to obtain the average water levels in a canal. This instrument was used without the current meter tailpiece and with a lightweight rod attached to the upper part of the wading rod. The frontispiece shows crews using these gages on the East Low Canal. PX-D-20890.

Figure 4
upstream and downstream from canal structures were also made with this gage to determine the backwater effects of piers.

Differential manometer gages were used to obtain all water surface elevation measurements on the Charles Hansen and Gateway Canals and part of those on Delta-Mendota Canal. This type of gage is shown in Figures H-7, I-11, and I-12. Two vertical transparent manometer tubes were utilized which were connected at the top and attached to a wooden support frame. The end of one tube was connected to a pressure head sensing disc which was placed on the lining below the canal water surface. The end of the other tube was placed below the water surface in a transparent plastic bucket attached to the bottom of the support frame. The water level in the plastic bucket was changed by a displacement block until it was even with the tip of a hook gage which was mounted on the support frame. The elevation of the hook gage was set from a bench mark on the lip of the concrete lining by means of an engineering level or a carpenter's level and wooden blocks of known length. Air was evacuated from the top of the tubes and water was drawn upward to eye level where the difference between the level of the canal water surface and that in the bucket could be read. Water surface elevations were recorded to hundredths of feet with this type of gage.

Some of the tests on Delta-Mendota Canal, Friant-Kern Canal, and East Low Canal (Washington) were conducted over long reaches of 113-, 28-, and 25-mile lengths, respectively. To cover distances of these magnitudes with available manpower, more rapid and convenient slope measurements were used to record water surface profiles. The procedure developed to obtain water surface elevations consisted of using steel rules or tapes to measure the slope distance from the top of the canal lining to the water surface in the canal. The slope readings were then converted to vertical distances which were subtracted from elevations of previously surveyed bench marks on the top of the lining to obtain water surface elevations.

The methods of measuring test discharges for each canal are described in the Appendix and on the data summary sheets. Generally these measurements were made by project hydrographers using Price Type A current meters and discharges where computed from velocity traverses using the standard 0.2, 0.8 depth procedure. However, more comprehensive velocity traverses were obtained during the 1960 tests on the Main and East Low Canals. Point velocities were measured for use in discharge and boundary shear distribution studies. Results of the shear distribution studies are not included in this Memorandum.

Another type of current meter equipment was used to obtain velocity distribution measurements in Delta-Mendota Canal, Charles Hansen Canal (Colorado), and Gateway Canal. This equipment included
a set of eight propeller-type current meters and a battery-powered recording unit. The meters were attached to a vertical metal rod which was suspended from a support frame mounted on a flat bed truck as shown in Figures B-17, H-6, and I-6. This equipment could simultaneously measure velocities at eight points on a vertical line.

REVIEW OF LITERATURE AND PRESENTATION OF TEST DATA

The literature abounds with articles written on flow resistance in rigid boundary open channels, and over the years, many formulas have been evolved for use in design. Two very thorough and comprehensive publications on this subject were written by Fred C. Scobey in the 1930's and have been used as a guide for the design of many thousands of miles of canal in the United States. These publications summarized a great amount of test data for various types of channels in existence during this period. However, very little information was presented on flow resistance in large concrete-lined channels because few existed at that time.

An excellent discussion and bibliography of more recent developments in both rigid boundary and earth channels have been published by the Task Force on Friction Factors in Open Channels which was sponsored by the American Society of Civil Engineers. A recent paper by G. Garbrecht lists and evaluates some 22 resistance formulas which have been used in Europe.

The hydraulic designs for all nine of the test canals were made using the Kutter-Chezy formula having an "n" value of 0.014. This formula for determining the average velocity of flow in a pipe or channel was used by the USBR until 1957. At that time a change was made to the less cumbersome Manning's formula. Using a constant "n" of 0.014 with hydraulic radii between 2 and 6 and with slopes between 0.0001 and 0.01, both formulas give approximately the same average velocity. Outside of these limits, different values of "n" must be used in one or the other of the formulas to obtain the same results. For example, in the Trail Lake Reach of the Main Canal (R = 11.3, S = 0.00061), an "n" of 0.0152 is required in Manning's formula to provide the same average velocity given by Kutter's formula with an "n" of 0.014. A convenient table of "n" values required to provide approximately the same velocity in either formula over a wide range of channel sizes and slopes is given in Reference 5.

In 1958, Peter Ackers authored two noteworthy papers concerning flow resistance in channels and pipes. The first of
these papers reviews hydraulic formulas which have been developed since 1775 and presents a new design procedure based on the Colebrook-White equation for conduit flow. Charts and tables are presented which greatly facilitate design computations. Conduit flow surfaces are described by a linear measure of surface roughness "k," or equivalent sand grain roughness. Values of "k," in feet, are given covering a wide range of surfaces, but it will be noted that these values describe flow resistance in straight uniform channels only. The author states that extra allowances should be made for conduit surface waviness or for curves or bends in alignment. Replies to the questionnaire listed in Figures 2 and 3 indicated that at least two European design agencies have also used the "equivalent sand grain surface" approach for design purposes.

Formulas presented in the Ackers papers are compared with Manning's formula which is widely used in the United States, Figure 5. Two curves were drawn by assuming a constant "k" value for each curve and computing Manning's "n" values for hydraulic radii from 2 to 20. Values of R between these limits cover a wide range of canal sizes as shown by the scaled cross sections of existing or proposed USBR concrete-lined canals at the bottom of the figure. Curve A represents a flow boundary having an equivalent sand grain roughness surface "k" of 0.010 foot. For the lower Curve B, "k" = 0.002 foot. Both curves rise with an increase in hydraulic radius, which in turn implies that "n" should increase with R. However, the curves tend to flatten toward the right, which indicates little change in "n" should be expected in canals having values of R greater than 20.

For flow in the zone of completely developed rough turbulence, Ackers develops an expression for the Chezy coefficient, C, equal to \((32g)^{1/2} \log 14.8 R/k\) which he describes as the logarithmic rough turbulent law (hereinafter referred to as the LRT law). When average velocity, V, is expressed as \(V = C(RS)^{1/2}\), it is apparent that velocity varies as the logarithm of the "roughness ratio" R/k. C computed by Manning's formula is equal to \(1.49 \frac{R}{n}\).

Paper No. 1 of Reference 6 contains a curve which shows the variation of C with R/k indicated by the LRT law. This curve and the accompanying development shows that for a chosen value of R/k between 7 and 130, Manning's formula provides values of "C" within plus or minus two percent of those given by the LRT law. When R/k exceeds 130, the percent error increases. For the maximum anticipated k (0.01 foot) and the minimum hydraulic radius of approximately 3.0 feet encountered in the tests described in this Memorandum, R/k exceeds 300. Therefore, some variation in computed velocities will occur between the use of Manning's formula with a
**VALUES OF R_h**

**NOTES**

Curves A and B on the chart are based on the following development:

- The average width of the (V) in an open channel may be expressed by
  \[ V = k \cdot S^{2/3} \cdot n \]

Where:
- \( V \) = average velocity
- \( S \) = hydraulic radius
- \( k \) = Manning's roughness coefficient
- \( n \) = equivalent sand grain roughness

Curves A and B are drawn for a hydraulic radius of 1 ft. and a Manning's roughness of 0.010 ft.

**REACHES OF DELTA MENDOTA AND FRONT-KERN CANALS**

- Source of plotted test data

<table>
<thead>
<tr>
<th>CANAL</th>
<th>No. of Tests</th>
<th>Range of Test Values</th>
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<tbody>
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<tr>
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**CANAL SECTIONS**

- Range of sizes of usable concrete lined canals which were tested

**RANGE CONDUIT SECTIONS**

(3D conduit sections drawn for comparison with Canal Section having the same hydraulic radius.)

**FIGURE 5**

**CONCRETE LINED CANALS**

Manning's 'n' Values from Prototype Tests Straight Reaches

**SOURCE OF PLOTTED TEST DATA**

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**FIGURE 5**

**CONCRETE LINED CANALS**

Manning's 'n' Values from Prototype Tests Straight Reaches

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constant "n" and the LRT law with a constant k. The relationship of k, R, and "n" is shown in Figure 5.

The R/k value for a round conduit flowing full is a constant value regardless of discharge. For a given open channel, however, R/k does not remain constant but changes with water depth. (As discharge increases, depth increases and R/k increases in a given channel under uniform flow conditions.)

Another aspect of the "equivalent sand grain roughness" approach to flow resistance is shown in Figure 5. It will be noted that the k of 0.010 foot used for Curve A is five times the 0.002 foot used for Curve B. However, only a 15- to 20-percent increase in "n" is shown between Curves A and B. This indicates that the use of k values provides a basis for more precise evaluation of flow resistance in a straight uniform channel.

To provide an indication of how results from the prototype tests described in this Technical Memorandum fell within the limits shown, "n" values computed from straight or nearly straight relatively clean reaches of the nine canals were plotted in Figure 5. These tests have been further identified by asterisks on the summary sheets (Figures 12 through 27). Although the plotted data were chosen carefully to minimize the effects of curvature, algal growth, pier losses, and all other effects except canal size, a fair amount of scatter is apparent. It will be noted that most of the "n" values fell between Curves A and B. Values of Manning's "n" from the larger sections were generally greater than 0.014. (All canals tested were designed with Kutter's formula assuming an "n" of 0.014. Equivalent Manning's "n" values for each canal are shown in Figure 1.)

When Manning's "n" values from tests in the nine lined canals were plotted in Figure 6, a great amount of scatter was evident. (Data from all tests listed in the prototype test data summary sheets except those from Figure 15 were plotted in Figure 6. Manning's "n" values in Figure 15 include head losses caused by several sets of in-line structure piers.) A comparison between Figures 5 and 6 reveals that many of the "n" values on the latter diagram are considerably higher than those plotted on Figure 5. The increased values have been attributed to the following major factors:

a. Seasonal aquatic growths on concrete lining surfaces
b. Horizontal curves in canal alinement

The presence of structure piers in the canal flow prism also increased hydraulic resistance. Estimated losses for these obstructions were deducted from overall energy losses in a reach before computing the values shown in Figures 5 and 6. Values of
Copper sulfate treatments were used in tests. Plotted data are from both straight and sinuous reaches of only three of the nine canals to curves and aquatic growths both tended to canals contained structures with pieres lined canals are plotted on the diagram at the left.

Increasing flow resistance in a given channel.

Head losses caused by these piers were deduced for computations of "n" values shown.
"n" varied from a low of 0.0118 measured in the Charles Hansen Canal to a high of 0.0203 measured in the Main Canal (Washington).

Drawn at the bottom of Figure 5 is a range of sizes of circular conduits with hydraulic radii corresponding to those of the open channels tested \((D = 4R)\). The conduits have been drawn to the same scale as the lined canals to provide the reader with an illustration of practical examples of the great differences in conduit shapes which can exist with a given hydraulic radius. From this comparison, it is possible to envision flow turbulence characteristics in the closed circular conduit to be somewhat different from those in the wide, free-surface open channel. It is reasonable to assume, therefore, that total boundary shear on the surface of the open channel with a given \(R\) may be more than that on the surface of a closed conduit with the same \(R\). Also, it seems apparent that previous theoretical analyses have not always accounted for the secondary or spiral flows which develop in an open channel and which superimpose additional energy dissipation over the general flow pattern.

J. Malaika in his paper on "Flow in Noncircular Conduits" reported that closed conduit shape had a pronounced effect on friction factors. Tests described in Reference 7 were made in small circular, elliptic, square, rectangular, and rhombic shaped conduits with very rough flow surfaces. The conduits had inscribed diameters of approximately 4 inches and were tested flowing full. Malaika concluded from these tests that the diameter of the inscribed circle was a better linear dimension to use for these shapes in the Reynolds and roughness parameters of Nikuradse and the Colebrook-White resistance equations than the hydraulic diameter, \(D\). He stated that an open channel could be visualized as the lower half of a closed conduit for such application. Also, he concluded that the relative hydraulic efficiency of a conduit cross section could be expressed by the dimensionless ratio of the inscribed circle diameter over the hydraulic diameter. Test data from the concrete-lined canals have not been analyzed from this standpoint, but this may be accomplished in the future.

Malaika's tests indicated the lowest friction factors ("f" values) for circular conduits when the hydraulic diameter, \(D\), was used in expressions for "f" and Reynolds Number. Values of "f" 30 to 50 percent higher were obtained in rectangular and elliptic conduits having aspect ratios of 3 to 1 and 6 to 1. If the test canals in Figure 1 are considered flowing at design depth, and the aspect ratio considered equal to \(T/2d\), it will be noted that this ratio varies from 3.2 for Delta-Mendota Canal to 1.8 for Madera Canal.

If Malaika's results are presumed applicable to these much larger open channels, significantly higher friction factors could
be expected than for round conduits of the same relative roughness at the same Reynolds' Number. Again, extrapolating Malaka's data for elliptic conduits to much larger open channels, the canal with an aspect ratio of 3.2 could be expected to have an "f" value about 5 percent higher than the one with an aspect ratio of 1.8 (assuming the same relative roughness and Reynolds' Number). Canal side slopes of the test canals (1-1/4:1 to 1-1/2:1) were not expected to have a significant effect on the shape factor.

Figure 7 is an application to open channels of the "Moody" type f - IR diagram commonly used for closed conduits. (An extensive amount of flow resistance data from round conduits flowing full has been presented in this type of diagram in USBR Engineering Monograph No. 7.)

To apply this technique to open channels, pipe diameter, D, is replaced by the mathematical equivalent 4R. Pipe "f" is expressed as \( h_f D/h_v L \). Open channel "f" in Figure 7 is defined as \( 8gRS_a/V^2 \). Pipe Reynolds' Number is expressed as \( VD/v \). On Figure 7, IR is defined as \( 4RV/v \). Figure 7 shows flow resistance data in terms of dimensionless numbers and provides an extension of data which has been already published for smaller channels. V. T. Chow,\(^{2}\) presented f - IR relationships for smaller rough rigid boundary channels which were investigated by Messrs. Bazin, Varwick, and Kirschmer. A discussion of the f - IR method of presentation is also included in Reference 3.

Figure 7 shows that plotting test data in terms of the chosen dimensionless parameters did not reduce the large amount of scatter. (The plots in Figure 7 were computed from the same tests used for Figure 6. The f - IR values were taken from all tests shown in Figures 12 through 14 and 16 through 27.) However, a comparison of Figure 7 with data from large round closed conduits\(^{3}\) at the same Reynolds' Numbers reveals that higher "f" values existed in the open channels. This f - IR diagram also shows that flow for the majority of tests fell within the theoretical zone of "fully developed turbulence" as defined in Reference 8 for closed conduits.

Much of the scatter of data on the f - IR diagram of Figure 7 was attributed to aquatic growths and canal sinuosity. To provide more basic design information, parameters from tests in straight clean canals were taken from the summary sheets and plotted on Figure 8. (Points in Figure 8 were from the same tests used for Figure 5. These tests have been marked with asterisks on Figures 12 through 27.)
CONCRETE-LINED CANALS
FRICITION FACTORS FROM PROTOTYPE TESTS
STRAIGHT AND SINUOUS REACHES

Theoretical zone of fully developed turbulent flow in closed conduits.

Closed conduit transitional flow zone.

(Smooth pipe) \( \frac{1}{V^2} = 2.0 \log (R \sqrt{f}) - 0.8 \)

\[ f = \frac{8gReSe}{V^2} \]

FIGURE 7
Values of \( f \) were also computed for the nine test canals utilizing hydraulic properties assumed for the design as shown in Figure 1. These values have been plotted in Figure 9 together with a shaded area which covers the range of test data shown in Figure 7. Values of \( f \) for design conditions from the smaller canals fell in the lower portion of the shaded area. However, \( f \) values for design conditions in the Main, Delta-Mendota, and East Low Canals fell below the shaded area. This demonstrates that hydraulic resistance was greater than expected and that a more conservative friction coefficient would have been warranted in the original design of the larger canals.

A similar friction factor diagram for rigid boundary open channels which utilizes a different expression \( (R_{BK}) \) for a Reynolds' Number has been outlined by the U.S. Army Corps of Engineers in Reference 10. The average data curve presented therein was derived from tests on smaller canals and models. (Maximum value of \( R_{BK} \) shown was \( 1.2 \times 10^5 \).) If this curve is extended into the area of higher Reynolds' Numbers (up to \( 6.3 \times 10^5 \)) computed from USBR tests, it lies below most of the \( f \) values obtained. Representative values of \( R_{BK} \) have been computed and are given in the data summary sheets, Figures 12 through 27, following page 38.

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**AQUATIC GROWTHS**

The presence of aquatic growths in earth channels has long been a problem for irrigation operating personnel. Seasonal decreases in earth canal capacities due to weed growths have been reported from virtually every USBR project. Flow measurements in large drainage canals in southern Florida have demonstrated that submerged weed growth has reduced capacities by as much as 97 percent.\(^1\) Manning's \( n \) values as high as 1.18 were recorded in channels which had not been chemically treated to retard weed growth. In the area of southern Florida where tests were made, a large seasonal change in \( n \) values was noted. A 17-fold increase in the value of \( n \) from 0.04 to 0.68 was recorded during a 10-week period in one 10-foot wide farm lateral.

A similar, but less pronounced, trend of seasonal change in flow resistance in large concrete-lined canals was revealed by several of the tests described herein. A slight seasonal increase in resistance had been expected since Scobey\(^2\) had documented this phenomenon in concrete flumes. However, the magnitude of the increase in the test canals was larger than anticipated. Tests on untreated
CONCRETE-LINED CANALS
FRIC TION FACTORS COMPUTED FROM DESIGN HYDRAULIC PROPERTIES OF TEST CANALS

Plotted symbols show location of *f* values computed from design hydraulic properties of test canals described on Figure 1. Water temperature of 58°F assumed to compute IR values. (See Figure 1 for Key to Symbols.)

Theoretical zone of fully developed turbulent flow for closed conduits.

Closed conduit transitional flow zone:

Smooth pipe

\[ f = 0.079 \log \left( \frac{IRV}{\nu} \right) - 0.8 \]

Range of test data as shown on Figure 7

Theoretical range of test data as shown on Figure 7

Water temperature of 58°F assumed to compute IR values.
Canals in the Columbia Basin Project in the State of Washington showed that "n" values increased 30 percent between March and August or September. Figures A-13, A-14, C-10, D-13, and D-14 in the Appendix show graphically the changes which occurred in this coefficient.

Capacities in the Madera Canal in California had been reduced as much as 20 percent by algal growths. Tests in 1958 in the 36-foot bottom width Friant-Kern Canal demonstrated a similar capacity deficiency due in part to filamentous algal growth. Biweekly copper sulfate treatments of 2 pounds per cfs discharge applied throughout the entire irrigation season have been used in the last 2 years to retard growths in these two canals with good success. After treatments of this concentration and frequency, the 10-foot-bottom-width, 1,000-cfs Madera Canal carried a maximum flow of about 110 percent of design capacity (with some encroachment on design freeboard) during 1962. In 1962, after regular treatments, the first 28 miles of the Friant-Kern Canal conveyed 4,500 cfs with 0.2 foot of freeboard at "tight spots" from which "n" values of 0.0145 to 0.0166 were computed. Freeboard of 0.25 foot with 5,000-cfs flow was anticipated in the original design of the canal.

Because of incomplete development of the Columbia Basin Project in 1960-61, the water users did not require design discharges in the canals chosen for the tests. Therefore, chemical treatments to retard aquatic growths were not made in the test reaches of the Main and East Low Canals. Therefore, all effects of growths on the concrete lining surface were recorded for the entire irrigation season. Canals in this area are unwatered during the winter, and aquatic growths are killed, which explains why "n" values computed from tests made early in the season were lowest. As the weather and water warms seasonally, filamentous algae, fresh-water sponge and Nostoc algae appear on the surfaces of concrete canal linings in this project as shown in Figures A-9, A-10, A-11, C-8, D-7, D-8, and D-9.

A research program to evaluate the cost and effectiveness of various herbicides and new chemical compounds in controlling aquatic growths peculiar to Columbia Basin Project canals was started by the USBR in 1963. Flow measurements are to be scheduled "before and after" treatments in both concrete and earth canals to document changes in hydraulic resistance.

Tests on the Columbia Basin Project canals also seemed to indicate that the flow velocity affected the computed values of "n" and "f." In canal sections heavily infested with algae growth, higher values were obtained for the low velocity tests than for higher velocity tests at the same time of year. Ree and Palmer reported similar effects in channels protected by vegetative linings. The "velocity effect" will be studied in future tests.
In the Delta-Mendota Canal water is generally turbid, warm, and rich in nutrients. This has provided an environment which is favorable to the growth of fresh-water clams and Bryozoa. (Bryozoa are minute animals which form moss-like growths as shown in B-8 and B-9.) A large percentage of the concrete lining surface in the first 10 miles of the canal was coated with this growth at the time of the 1960 unwatering. The exact effect of Bryozoa growth on flow resistance has not been determined. Figure B-9 shows tentacles which develop and often extend as much as 1 inch from the concrete lining surface.

The extent of Asiatic clam (Corbicula flumina) growth was much larger than had been expected in Delta-Mendota Canal. The silt beds which had been deposited on the invert of the concrete lining (Figures B-10, B-11, B-12), appeared to be ideal locations for clam propagation. The clam-silt beds, in turn, provided increased surface roughness and increased hydraulic flow resistance. An analytical method for determining the effect of sediment deposits on hydraulic friction is outlined in Reference 12. Silt which formed the beds apparently came into the canal with return irrigation flow, storm inflows, and from windblown material.

The spread of fresh water clam growth has become an increasing problem in irrigation systems in California. A satisfactory method of controlling the growth of this clam in large channels has not been devised at this time. Aromatic solvents have been used successfully to clear clam infestations from the cooling condenser tubes at the Tracy Pumping Plant where a relatively small amount of water is involved. However, this type of treatment has been too expensive for large-scale application.

In addition to the aforementioned growths, considerable numbers of fish were reported in lined canals in California. Carp and bass were found in the Delta-Mendota Canal during the 1960 and 1962 unwaterings, and the thread fin shad was reported in the Friant-Kern Canal.

Antifouling paints applied to concrete lining surfaces have been used successfully on several USBR canals to retard algal growths, and research is continuing to evaluate their effectiveness over a period of time. Applications have been limited mainly to the short canal sections at gaging stations where discharge measurements are made. The applications are made at these points because paint costs are presently too high to justify large-scale use. Annual applications of paint have been required in some areas to obtain the desired degree of control. Such applications would not be possible in canals which deliver water continuously throughout the entire year.
Specific descriptions of the genera of algae which occurred in test canals is provided in the narratives of the Appendix.

CANAL SINUOSITY

To determine the effect of horizontal curves in alinement on flow resistance in the test canals, both sinuous and straight reaches were selected for test measurements. Summaries of curve data for the individual reaches are included in the narrative discussions for each canal in the Appendix. A brief summary of this information is provided in Table 1.

Flow through sinuous reaches of canal requires a larger expenditure of energy than flow through a straight reach of the same canal. The largest proportion of the additional head loss is thought to be caused by increased boundary shear due to circulatory flows and increased velocities on certain portions of the channel section as the water traverses the curve. These disturbances to the pattern of velocity distribution create added turbulence and additional hydraulic resistance. In effect, a curve in alinement acts on the canal as would increased boundary roughness. In contrast to bend losses in pipes for which many measurements are available, few prototype tests have been made to document bend losses in open channels. Three recent publications describing laboratory work on the mechanics of flow in open channel curves are provided in References 15, 16, and 17. Additional prototype research on this subject by the United States Geological Survey was in progress on concrete-lined canals of the Colorado-Big Thompson Project in 1962.

Garbrecht\(^4\) provides a design chart for the determination of open channel curve loss coefficients based on model tests available in 1959. This chart shows that the ratio of curve radius to water surface width \((R_l/T)\) has a considerable effect on the coefficient when this ratio is small. However, he states that head losses for curves with \(R_l/T\) ratios greater than 5 are insignificant in smooth channels. Reference to the curve data from the test canals shown in Table 1 indicates the average \(R_l/T\) value to range between 4 and 22. From this standpoint, it would appear that the extra head loss caused by many of the curves on these canals would be insignificant. However, analyses of data from test reaches where many curves occur in a short distance indicated significantly higher "n" values. Examples of such reaches are shown for the Friant-Kern Canal in Figure E-2 and for the Charles Hansen Canal in Figure I-2.
<table>
<thead>
<tr>
<th>Canal</th>
<th>Simuous reach length L (ft)</th>
<th>Curve deflection angle Max.</th>
<th>Min.</th>
<th>Av.</th>
<th>Curve radius $R_1$ (feet) Max.</th>
<th>Min.</th>
<th>Av.</th>
<th>Design W.S. width T (ft)</th>
<th>Av. $R_1/T$</th>
<th>Curvature index $5\Delta/\Delta L_1$</th>
<th>No. of curves per mile</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main (below Long Lake)</td>
<td>5,432</td>
<td>52</td>
<td>3</td>
<td>26</td>
<td>1,432</td>
<td>286</td>
<td>693</td>
<td>112</td>
<td>6.2</td>
<td>0.22</td>
<td>8.7</td>
</tr>
<tr>
<td>Delta-Mendota</td>
<td>351,000</td>
<td>98</td>
<td>1</td>
<td>33</td>
<td>1,000</td>
<td>400</td>
<td>553</td>
<td>97</td>
<td>5.7</td>
<td>0.10</td>
<td>3.3</td>
</tr>
<tr>
<td>Main (Trail Lake)</td>
<td>5,570</td>
<td>75</td>
<td>8</td>
<td>34</td>
<td>358</td>
<td>286</td>
<td>344</td>
<td>83</td>
<td>4.1</td>
<td>0.31</td>
<td>9.5</td>
</tr>
<tr>
<td>East Low</td>
<td>8,059</td>
<td>55</td>
<td>23</td>
<td>40</td>
<td>955</td>
<td>573</td>
<td>708</td>
<td>77</td>
<td>9.2</td>
<td>0.15</td>
<td>3.9</td>
</tr>
<tr>
<td>Friant-Kern</td>
<td>148,910</td>
<td>164</td>
<td>5</td>
<td>44</td>
<td>1,000</td>
<td>140</td>
<td>332</td>
<td>74</td>
<td>4.5</td>
<td>0.23</td>
<td>5.6</td>
</tr>
<tr>
<td>*11,812</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Roza</td>
<td>13,881</td>
<td>132</td>
<td>16</td>
<td>50</td>
<td>2,865</td>
<td>143</td>
<td>766</td>
<td>35</td>
<td>22.1</td>
<td>0.23</td>
<td>5.0</td>
</tr>
<tr>
<td>Madera</td>
<td>5,975</td>
<td>145</td>
<td>21</td>
<td>75</td>
<td>498</td>
<td>143</td>
<td>303</td>
<td>42</td>
<td>7.2</td>
<td>0.62</td>
<td>8.9</td>
</tr>
<tr>
<td>Gateway</td>
<td>3,889</td>
<td>61</td>
<td>19</td>
<td>39</td>
<td>700</td>
<td>125</td>
<td>335</td>
<td>31</td>
<td>10.9</td>
<td>0.25</td>
<td>6.8</td>
</tr>
<tr>
<td></td>
<td>4,169</td>
<td>52</td>
<td>10</td>
<td>25</td>
<td>1,000</td>
<td>150</td>
<td>425</td>
<td>31</td>
<td>13.8</td>
<td>0.24</td>
<td>10.1</td>
</tr>
<tr>
<td></td>
<td>4,262</td>
<td>63</td>
<td>13</td>
<td>40</td>
<td>500</td>
<td>200</td>
<td>351</td>
<td>31</td>
<td>11.4</td>
<td>0.38</td>
<td>9.9</td>
</tr>
<tr>
<td>Charles Hansen</td>
<td>6,013</td>
<td>88</td>
<td>11</td>
<td>41</td>
<td>1,146</td>
<td>115</td>
<td>409</td>
<td>29</td>
<td>14.3</td>
<td>0.47</td>
<td>12.3</td>
</tr>
<tr>
<td></td>
<td>1,706</td>
<td>48</td>
<td>9</td>
<td>25</td>
<td>1,146</td>
<td>164</td>
<td>614</td>
<td>29</td>
<td>21.4</td>
<td>0.29</td>
<td>12.4</td>
</tr>
</tbody>
</table>

*This length was Test Reach 1, for which data are recorded on Figure 20.
The 11,812 feet is a portion of the 148,910-foot length shown in the preceding line.
Manning's "n" values computed from measurements in the sinuous reaches were higher than those from straight reaches of the same canal in the majority of cases by as much as 15 percent. Figures F-8, G-6, H-9, and I-15 show "n" values computed from the four smallest canals plotted against "curvature index." This index is a parameter used by Scobey for describing the sinuosity of a 500-cfs rectangular concrete flume. In its original form the curvature index was expressed as 

\[ \frac{\sum \Delta^\circ}{20^\circ \cdot \frac{L}{100}} \]

This expression reduces to the form \( \frac{\sum \Delta^\circ}{L} \) which is used henceforth in this Memorandum. A curvature index of 1.0 is equivalent to 1° of deflection angle for every 5 feet of canal length. A curvature index of 1.0 would also be equivalent to one 20° deflection angle for every 100 feet of channel. Such severe curvature is seldom encountered in any appreciable length of large irrigation canal. Sinuosity of the very large canals usually will not exceed that of the first 66 miles of Delta-Mendota Canal. This length has an average curvature index of 0.10 which amounts to 20° of deflection for each 1,000 feet of length.

Scobey suggested that "n" values be increased 0.001 for each 20° of curvature per 100 feet of concrete flume. This meant, when the curvature index was equal to 1.0, that an "n" of 0.014 should be increased about 7 percent to 0.015. By this procedure, he reasoned that the increased flow resistance created by curves could be distributed along the entire length of channel and expressed as increased roughness. Computed "n" values for the Charles Hansen Canal, the smallest of the nine test canals described herein (where hydraulic radii were comparable to the radii of the flume tested by Scobey) showed considerable scatter but generally substantiated his suggested rule.

Shukry discussed many of the factors affecting losses in open channel bends. He performed detailed laboratory flume studies concerning flow characteristics around bends, but his work was confined to short radius curves and was accomplished at smaller Reynolds' Numbers than those encountered in the prototype tests described herein. From the results of both Shukry and Garbrecht, it can be concluded that on long radius curves, \( R_1/T \) greater than 5, and with Reynolds' Numbers greater than \( 10^5 \), only surface roughness and curve deflection angle, \( \Delta \), have significant effects on head loss coefficients. Surface roughness was similar in all of the large concrete-lined test sections described herein, leaving curve deflection angle as the most significant remaining variable involved in computing bend loss coefficients.
The water surface profiles obtained for the series of tests described in this Memorandum show the overall effect of canal curvature. Sufficient time was not available to make comprehensive studies of the individual effects of a particular curve or to study velocity distributions within a curve. In the data analyses it was therefore necessary to estimate curve losses and to compute matching backwater curves using "n" values measured in straight reaches. Comparative water surface profiles in which allowances for curve losses were made have been computed for the Main, East Low, Friant-Kern, and Charles Hansen Canals at the time of this writing. The best comparisons to date have been found to exist when incremental head losses caused by curves \( h_c \) have been expressed as a coefficient \( K_c \) times the summation of deflection angles in a reach times the velocity head, \( h_c = K_c \left( \Sigma \Delta^\circ \right) h_v \).

Preliminary results from backwater curve computations show that the energy loss for canal curves (over and above that in a straight reach) may be approximated when \( K_c = 0.001 \). Shown in Table 2 are examples of the magnitude of estimated head loss for one curve with \( \Delta = 45^\circ \) in three of the test canals, computed using the formula \( h_c = 0.001 (45^\circ) h_v \).

<table>
<thead>
<tr>
<th>Canal</th>
<th>Design velocity (ft per sec)</th>
<th>Velocity head ( h_v ) (feet)</th>
<th>Estimated curve loss (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Canal (Trail Lake Reach)</td>
<td>12.21</td>
<td>2.32</td>
<td>0.104</td>
</tr>
<tr>
<td>Gateway Canal</td>
<td>4.99</td>
<td>0.39</td>
<td>0.018</td>
</tr>
<tr>
<td>Delta-Mendota Canal</td>
<td>3.75</td>
<td>0.22</td>
<td>0.010</td>
</tr>
</tbody>
</table>

Further data analyses may reveal that slight changes in the value of \( K_c \) are necessary over a range of canal sizes.

Magnitudes of depth increases across the most severe (largest \( \Delta \) and shortest radius) individual curves in the test canals were estimated to be 0.2 to 0.3 foot. The majority of curves probably created added losses of less than 0.1 foot each. However, the accumulative effect of a series of curves in a short length of channel was found to be significant, especially at the upstream end of the Friant-Kern Canal. Here, curve losses in one reach, computed
by the equation $h_c = 0.001 (\Sigma \Delta^o)(h_f)$, were sufficient to cause an increase in average depth of about 0.9 foot. This amounted to 6 percent of the design depth of 15.2 feet. Vertical freeboard allowances from normal water surface to top of concrete lining in the nine test canals ranged from 1.0 to 2.8 feet.

The percentage increase in depths due to added curve losses was even greater in the high velocity (10 fps) Charles Hansen Canal. This canal was designed at the relatively high Froude Number of 0.84. Under these conditions, where velocity heads are large, a small change in energy head results in a substantial change in depth. Curves, transitions, and obstructions such as piers, which tend to disrupt the normal velocity distribution pattern, create considerable amounts of wave action. This wave action becomes a more important element for design consideration when flow velocities exceed 8 fps or Froude Number ($F$) approaches 1.0. For photographs of flow conditions in such a channel the reader is referred to Section I on the Charles Hansen Canal in the Appendix. Extensions of concrete lining were required on sinuous parts of this canal after construction was completed. Wave action and "ride up" of the water on the outside of curves was greater than anticipated.

Garbrecht and Chow provide discussions of backwater effects of curves in a channel. Briefly, the energy gradient through a curve rises more steeply than that in the straight section of channel downstream. The additional energy required to move the water through the curve results in an increase in depth upstream of the curve. The increased depth causes a slight reduction of friction slope in the channel. If an M2-type Backwater Curve (Figures E-5 and E-6) exists at the channel curve, all of the rise in water surface is reflected upstream. In this case, water depth increases in an upstream direction until normal depth is attained. If an M1-type Backwater Curve exists at the channel curve, the entire curve loss is not reflected upstream because the water depth is tending to decrease to the normal value.

Overall effects of curve losses on the average depth of flow in a given reach of canal depend on the ratio of curve losses to other losses within the reach. In steep slope canals, the upstream reflection is less than that for flat slope canals. In other words, the effect is greatest in large canals where the ratio of velocity head to friction slope is greatest.

The effect of "extra" losses such as those caused by curves or piers on the average uniform flow depth in a reach of canal can be approximated by the expression $d = d_f(h_f/h_f)^N$ where $d$ is the "average" depth; $d_f$ is the depth due to boundary shear or friction only in a straight, clean, unobstructed canal; $h_f$ is the head loss
due to friction only in the reach under consideration; \( h \) is the total head loss due to friction, curves, piers, and other obstructions (computed for the same depth as was \( h_f \)) and \( N \) is a variable exponent. The value of \( N \) for canal sections with 1-1/4 or 1-1/2 to 1 side slopes varies from about 0.29 for \( B/d \) ratios of 5 to about 0.26 for \( B/d \) ratios of 2. A reasonable first approximation can be made using \( N \) equal to 1/4. Local effects of the "extra" losses can only be obtained by detailed backwater curve computations.

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**PIER LOSSES**

When restrictions are placed in open channels, velocity distribution patterns are changed and additional energy losses occur. The resulting effect in subcritical flow canals is that of an increased water depth upstream from the restriction. Several equations can be found in the literature for computing the energy loss or water surface rise caused by piers in the flow prism. For the analyses described herein the expression \( h_p = K_p h_v \) was used in which \( h_p \) is the incremental head loss caused by piers, \( K_p \) is the pier loss coefficient, and \( h_v \) is the velocity head in the unobstructed canal section.

Many model tests have been made to determine the rise in water surface across various types and configurations of piers. An excellent assembly and evaluation of data on this subject have been published by the Bureau of Public Roads. Information from this publication has been further condensed into a hydraulic design publication. Figure 7 on page 10 of Reference 19 is a chart from which a backwater coefficient, \( K \), can be selected to compute the rise in water surface across piers of various shapes and arrangements. This chart shows \( K_p \) values to be a function of the amount of channel contraction at the piers. The "contraction ratio," signified by the letter \( J \) in this Memorandum, is defined as the ratio of the flow area obstructed by piers to the gross area of the flow prism. This ratio is usually quite small for irrigation canal structures. Typical values of \( J \) varied from 0.018 for a single pier bridge in the Friant-Kern Canal to 0.055 for a twin pier over­chute in the Delta-Mendota Canal. However, canal structure configuration should be considered before \( J \) values are computed. (The procedure for computing \( J \) values for check and other type structures is described on pages 23 and 24.)

To determine the magnitude of rise in water surface caused by prototype piers in canal sections, differential water surface
The flow areas through checks were unobstructed during tests. No stoplogs were in place. Contraction ratio for these check structures was computed from the expression $J_d$ given in the section of this report titled "Pier Losses." All piers on structures utilized for tests were parallel to canal centerline.

### CONCRETE BRIDGE

Values of $K_p$, pier loss coefficient, were computed from $K_p = 0.0$ in the unobstructed canal section. Values of $J_d$, contraction ratio, for bridges and overpasses were computed as described in the section of this report titled "Pier Losses." All radial gates were fully open and clear of the water for these tests. Stoplogs were in place in side bays to an elevation above the water surface. The contraction ratio for these check structures was computed from the expression $J_d$ given in the section of this report titled "Pier Losses." All piers on structures utilized for tests were parallel to canal centerline.

### HEAD LOSSES CAUSED BY PIERS

#### SUMMARY OF RESULTS FROM TESTS ACROSS PROTOTYPE CANAL STRUCTURES

### CANAL CAPACITY TEST PROGRAM

**NOTES**

Water surface elevations were measured upstream and downstream from prototype structures to obtain a total rise in water surface ($\Delta h$) for a given reach of canal. The difference in energy gradient ($\Delta K$) and the head loss due to canal friction only ($h_f$) were then computed. (For computing $h_f$, a value of Manning’s $n$ obtained from tests at the same time in adjacent canal reaches was used when available. Otherwise an $n$ of 0.016 was assumed.)

Values of $h_g$, head loss due to piers, were computed from the expression $h_g = \Delta h - h_f$.

Values of $K_p$, pier loss coefficient, were computed from $K_p = 0.0$ in the unobstructed canal section. Values of $J_d$, contraction ratio, for bridges and overpasses were computed as described in the section of this report titled "Pier Losses." All radial gates were fully open and clear of the water for these tests. Stoplogs were in place in side bays to an elevation above the water surface. The contraction ratio for these check structures was computed from the expression $J_d$ given in the section of this report titled "Pier Losses." All piers on structures utilized for tests were parallel to canal centerline.
measurements were made across some 20 typical structures. These measurements were extremely difficult to obtain because of the minute differences in water surface elevation which occurred. For instance, on the Delta-Mendota Canal, normal head loss in a 400-foot reach of canal at design flow was about 0.02 foot and pier losses were about 0.02 foot. Accurate measurement of a total fall of 0.04 foot in locations where 0.20-foot waves were often encountered proved to be a frustrating experience. Figure B-16 shows crews making measurements across a timber bridge on this canal.

Results from measurements across 20 canal structures with piers in the flow prism are summarized in Figure 11. A great variety of pier configurations, sizes, and shapes existed in the test channels; therefore, results have been generalized. The values of $K_p$ shown in Figure 11 can be used to estimate the rise in water surface across piers of the types utilized for tests. Values of $K_p$ for design may also be obtained from Reference 19. Pier losses computed were generally about 20 percent higher than the values measured in the test canals. Therefore, it is recommended that values of $K_p$ obtained from Figure 7 of Reference 19 be multiplied by 0.8 when used for low velocity trapezoidal canal sections. Values of $J$ should be computed as outlined in the following paragraphs.

As indicated in the notes at the bottom of Figure 11, contraction ratios ($J$ values) for various structures were defined in several ways. Where the velocity distribution pattern across the full width of the structure is expected to be little different from that in a straight unobstructed reach of canal, the normal concept of contraction ratio based on the flow area at the structure is expected to apply. Values of $J$ and $h_p$ for this condition, where the gross flow area through the structure is equal to or less than the area of the unobstructed canal section, should be computed as follows:

$$J_1 = \frac{\text{Total projected area of piers, curtain walls, stops or other obstruction normal to flow}}{\text{Gross area of section normal to flow at structure}}$$

Compute $h_p$ using $h_v$ based on velocity through gross area of section at structure.
The contraction ratio is more difficult to define in check structures where an expansion in the flow prism occurs such that separation or greatly reduced velocities will occur near the sides of the structure (Gateway Canal Check shown in Figure H-4, Appendix). With this type of structure configuration, curtain walls, stoplogs, or similar obstructions in the expanded area near the sides are expected to have only a minor effect on head loss through the structure. In this situation, where velocities near the center of the structure are essentially the same as those in the adjacent unobstructed canal section, the largest contraction ratio computed by either of the following two equations should be used:

\[
J_2 = \frac{\text{Area of flow prism in unobstructed canal section minus unobstructed flow area through structure}}{\text{Area of flow prism in unobstructed canal section}}
\]

\[
J_3 = \frac{\text{Horizontal thickness of one structure pier normal to flow}}{\text{Horizontal center to center distance between structure piers normal to flow}}
\]

\(J_3\) is computed using dimensions of a typical center bay of the structure. Compute \(h_p\) using \(h_v\) existing in unobstructed canal section

In other check structures designed by the USBR (Delta-Mendota Canal checks shown in Figure B-21) the canal flow prism has been expanded at the structure with streamlined transitions. In this case, a significant velocity is anticipated to occur near the sides of the expanded section. Where the angle of flare of these transitions does not exceed 15°, values of \(J_1\) and \(J_2\) should be computed. When \(J_2\) is larger than \(J_1\), \(J_2\) should be used to compute \(h_p\). When \(J_1\) is larger than \(J_2\), an average of the two values should be used to compute \(h_p\).

Values for \(J\) and \(K_p\) shown in Figure 11 for Gateway Canal check structures were based on a single center bay (\(J_3\)) while those for checks in Delta-Mendota Canal were computed using \(J_2\).

Figure 11 and Reference 19 show that losses across multiple post piers can be more than double those across solid piers having streamlined noses and tails. (The significance of pier shape on backwater effects is discussed in Reference 20.) The narrative discussion for Delta-Mendota Canal in the Appendix describes prototype pier modifications made to reduce head losses in this canal. Typical modifications are shown in Figure B-7 of the Appendix.
Reference 19 contains a recommendation for computing \( J \) values for piers having sway bracing on the sides. It is stated therein that \( K_p \) be selected using a \( J \) which is computed using the total projected area (normal to the flow) of both posts and sway bracing. This procedure yielded values of \( K_p \) substantially higher than the values obtained from measurements across the bridges listed in Figure 11. A more reasonable correlation of results existed when \( J \) values were computed for only the post widths and a loss for sway bracing was added. The additional loss was derived using a \( J \) value computed assuming the sway bracing width to act as a single, solid, round-nose pier.

Table 3 includes examples of the estimated rise in water surface across a 20-foot concrete bridge with two round-nose, solid piers. The values were computed using design velocities from three typical canals. The contraction ratio at this bridge was assumed to be 0.04. \( K_p \) was assumed to be 80 percent of the value shown in Figure 7 of Reference 19.

<table>
<thead>
<tr>
<th>Canal</th>
<th>Design velocity (fps)</th>
<th>Velocity head ( h_v ) (ft)</th>
<th>( K_p )</th>
<th>Rise in water surface across piers (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Main Canal (Trail Lake Reach)</td>
<td>12.21</td>
<td>2.32</td>
<td>0.05</td>
<td>0.12</td>
</tr>
<tr>
<td>Gateway Canal</td>
<td>4.99</td>
<td>0.39</td>
<td>0.05</td>
<td>0.02</td>
</tr>
<tr>
<td>Delta-Mendota Canal</td>
<td>3.75</td>
<td>0.22</td>
<td>0.05</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Backwater curve computations in a given channel are necessary to demonstrate the accumulative effect of head losses at piers on the water surface profile. The overall effect of pier losses on the average depth in a reach of canal depends on the ratio of the pier losses to all other losses in the reach. The effect is similar to that previously described for curve losses.

Several bridges on the Delta-Mendota Canal were located in the center of "S" curves in canal alignment. Other bridges were located on short tangents between curves so that shorter and less costly 90° crossings could be built. Measurements across these types of crossings and adjacent curves indicated losses which were about double those obtained across similar bridges in straight
reaches of canal. Some of the additional loss was undoubtedly caused by the changes in velocity distributions created by the curves in canal alignment and should not be attributed solely to the piers. As previously mentioned, it was extremely difficult to measure accurately the small increments of head loss in this flat slope canal. Therefore, the precise effect of canal alignment on head losses at piers could not be determined.

It has often been a common practice to provide for 90° structure crossings when the alignment for an irrigation canal is prepared, the purpose being to cut down on span lengths and reduce structure costs. If this practice is followed, the increased head losses caused by curves and piers should be computed and considered in the hydraulic design. If water for the canal is to be pumped or little additional head is available, an economic comparison should be made to find if straight alignment and skew structure crossings should be used.

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**SIPHON LOSSES**

As part of the water surface profile measurements in the open channel portions of the test canals, several head loss measurements were made across large inverted siphons which were adjacent to the test reaches. In general, the head losses across the siphons were less than allowances made in original designs. Detailed results of the measurements across eight siphons on the Delta-Mendota Canal are given in Figure B-6. Measurements across two siphons on the Friant-Kern Canal, four on the East Low Canal, and one on the Gateway Canal are described in the narratives of the Appendix.

At siphons where losses were less than allowed in design, overcapacity existed and nonuniform flow conditions were created upstream. The overcapacity resulted in lower canal depths upstream of the siphon than those at the downstream end. In effect, a "safety cushion" existed in the canals where long siphons occurred at frequent intervals.

More detailed measurements will be required before the causes of siphon overcapacities and the wide variations in results can be definitely established.

---

**FUTURE RESEARCH**

This Technical Memorandum summarizes the measurements made through 1962 in concrete-lined canals only. It extends the
knowledge of flow resistance to rigid boundary channels of larger size than have heretofore been documented by the USBR. Many problems concerned with the capacity tests remain unanswered and techniques used for tests measurements can be improved. Therefore, additional research work should be accomplished on the following:

1. Methods of more accurately determining discharge in large open channels.

2. Determination of energy losses, boundary shear distribution, and velocity distribution on curves.

3. Measurements of prototype head loss across piers.

4. Determination of more economical and effective methods of eliminating or controlling the growth of filamentous algae and fresh water clams.

5. Definition of "surface roughness" by means of profilograph equipment similar to that used on concrete highways and airport runways.

6. Improvement of underwater photographic techniques to document the extent of algal growth on a lining surface and the effect of velocity thereon.

7. Explanation of how filamentous algal growth changes boundary layer turbulence and effects flow resistance.

8. Documentation of possible changes in flow resistance in concrete-lined channels due to aging of the lining surface.
CONCLUSIONS

As a result of prototype tests on the nine large concrete-lined canals described in Figure 1 it is concluded:

1. Flow resistance in the five largest canals (hydraulic radius 9 to 14) was greater than anticipated in the design. Values of Manning's "n" computed from tests on these canals generally ranged between 0.015 and 0.019. (Manning's "n" values used for design ranged from 0.0137 to 0.0152.)

2. Flow resistance in the four smallest canals (hydraulic radius 3 to 7) was close to that anticipated in the original design. Values of Manning's "n" computed from tests on these canals generally ranged between 0.013 and 0.016. (Manning's "n" values used for design ranged from 0.0141 to 0.0145.)

3. The effect of size on values of Manning's "n" and friction factor "f" for straight clean reaches of concrete-lined canal was slight and is shown in Figures 5 and 8.

4. Values of friction factor "f" from clean straight reaches of the concrete-lined canals were generally 30 percent higher than the bulk of those measured by other investigators in large circular concrete-closed conduits at the same Reynolds' Number. Aquatic growths, coatings on the lining, construction methods and/or a shape factor probably account for all or a large part of this difference.

5. Aquatic growths or coatings of various types were found on the surfaces of the concrete lining in all nine test canals. The amount and type of growth depended mainly on climatic conditions and the canal water source. The extent of surface coverage varied both seasonally and annually in a given channel.

6. Filamentous algal growth on the lining surface increased flow resistance and caused it to vary seasonally in the clear water canals. Manning's "n" values increased as much as 40 percent on Columbia Basin Project canals during one irrigation season. The greatest amount of growth of this algae usually occurs in warm weather during peak delivery periods. Biweekly copper sulfate treatments of 2 pounds per cfs throughout the entire water delivery period were effective in controlling, but not in completely eliminating, this type of growth in the Central Valley Project canals. Virtually constant "n" values were obtained in canals which were treated in this manner. Flow resistance due to algal growth also appeared to vary with velocity.
7. Horizontal curves in canal alignment change normal velocity distribution patterns and create energy losses in addition to those which exist in a straight canal. Preliminary computations indicate that these extra head losses, due to the long radius curves normally encountered in concrete-lined canals, can be approximated by the expression \( h_c = 0.001 (\sum \Delta v^2) h_v \).

8. Piers or other restrictions in the flow prism contribute to overall energy losses in a reach of canal. Losses due to piers may be approximated by the expression \( h_p = K_p h_v \). Measured losses are shown in Figure 11. Losses may be estimated by using 0.8 of the \( K_p \) values obtained from Figure 7 of Reference 19.

9. Head losses caused by curves or piers increased canal water depths as much as 1.0 percent over average design depths in unobstructed straight reaches. The effect on a given length of canal depends on frequency of occurrence of the flow disturbances and can be most accurately predicted by backwater curve computations. The effect is greatest in large flat slope canals where the ratio of velocity head to friction slope is large. An approximate method for predicting the effect of these added losses on average depth is outlined on page 23.

10. As the Froude Number (\( F \)) approaches 1.0, local velocity disturbances due to piers and curves are magnified. Surface waves and local variations in water surface of as much as 0.8 foot occurred in the fast-velocity (10 fps) Charles Hansen Canal where \( F \) was 0.84.

11. Cleaning of extensive silt-clam deposits from the concrete-lined Delta-Mendota Canal created unexpected maintenance expense. In most large lined canals such cleaning has not been required, but some 50,000 cubic yards of material were found on the invert of this canal in 1960 after only 7 years of continuous operation. These deposits contributed to increased hydraulic resistance. A method for computing this effect is outlined in Reference 12.

12. Hydraulic friction losses in the closed conduit inverted siphon portions of the test canals were generally less than anticipated in the original design. Because of this, overcapacity existed in the siphons and nonuniform flow conditions were created for most test runs. The overcapacity provided a "safety cushion" effect in canals where long siphons occurred at frequent intervals.

13. The numerical coefficient, 1.486, used in Manning's formula implies accuracy of one part in 1.486 or 0.067 percent.
Since "n" values used in this formula are rarely expressed to more than two significant figures, a value of 1.49 will provide sufficient accuracy for hydraulic computations.
RECOMMENDATIONS

1. The hydraulic design information and canal operating experiences presented in this Technical Memorandum should be considered in the design of large concrete-lined canals. Special attention should be given and suitable allowances made for additional resistance to flow in certain reaches of canal when the following circumstances exist:

   a. Occurrence of extensive aquatic growths or silt deposits in locations where chemical control treatment or cleaning is not feasible

   b. Frequent occurrence of structures with piers which extend into the flow prism

   c. Occurrence of extremely sinuous canal alignment

2. Further studies should be conducted to record and explain effects of aquatic growths on flow resistance, to develop optimum control methods for these growths, and to establish accurate cost records for controlling growths.

3. Efforts should be made to extend the range of prototype canal performance data presented in Figures 5 and 8. Additional tests on rigid boundary channels of other ages and sizes at higher Reynolds' Numbers would be especially valuable. Tests at Reynolds' Numbers above 10⁷ would enable more accurate positioning of the "Average Data Curve" of Figure 8 and would result in more reliable predictions of flow resistance in large canals.

4. During the design of a concrete-lined canal, consideration should be given to the provision of facilities for the exclusion of silt, particularly from canals which have slow velocities and those which may never be unwatered. The extent of silt exclusion facilities should be balanced against cost of future cleaning operations.

5. Capacity tests on large circular and rectangular inverted siphons in canals should be continued. More detailed measurements should be made in an attempt to explain the wide variation of results which have been obtained to date and to verify present design procedures.

6. Canal sections with Froude Numbers (\(F\)) above 0.8 should be avoided if at all possible. When \(F\) exceeds 0.5, detailed hydraulic computations should be made to determine the backwater effects due to the following causes:
a. Local increases in losses caused by flow obstructions
b. Probable variation in friction coefficients
c. Transitions to higher or lower velocities.
RECOMMENDED HYDRAULIC DESIGN PROCEDURES

The following design procedure for a large concrete-lined canal is based on the assumption that it is feasible to keep the lining surfaces reasonably clean and free of aquatic growths during periods when design discharge will be required. It is also assumed that the canal will be kept free of silt deposits.

After a design discharge has been determined and shape of the cross section has been selected, the hydraulic properties can be determined using Manning's formula

\[ Q = \frac{1.49}{n} AR^{2/3} S^{1/2} \]

Figures 5 and 8 are included in this Technical Memorandum to serve as a guide for choosing a basic value of roughness coefficient, "n", for reaches of canal which are straight and clean.

Figure 8 was prepared to show test results in terms of the most important variables which affect flow resistances. All significant variables except surface roughness are combined into dimensionless parameters "f" and IR. It has been assumed on this diagram that surface roughnesses of machine-placed concrete lining should be similar to that of the nine test canals. The "Average Data Curve" shown in Figure 8 was drawn through plots of test data from 52 individual tests in straight reaches of the nine canals described in Figure 1. These tests have been marked with asterisks on the Prototype Test Data Summary Sheets. An equation for a straight-line least squares fit of the 52 plotted points in Figure 8 was obtained. This line provides "f" values within 2 percent of the "Average Data Curve" which is shown.

Recommended steps for use of this Memorandum in design are given below:

1. Determine preliminary canal section using Manning's formula. Assume a basic value of "n" for a straight reach of canal using Figure 5 as a guide.

2. Estimate water temperature which will occur when design discharge will be required and obtain kinematic viscosity, v, from Figure 10. (Water temperatures taken during tests in canals of the Western United States are given in Prototype Test Data Summary Sheets.) Compute Reynolds' Number IR = 4RV/v.

3. Enter Diagram 1 of Figure 8 with computed IR value and check original assumption of basic "n" utilizing procedures outlined in the lower right-hand corner of the figure.
4. Repeat procedure outlined in Figure 8 until the desired accuracy of results are obtained.

5. Estimate "extra" head losses due to curves, piers, and other restrictions as outlined in Conclusions 7 and 8 of this Memorandum.

6. Isolate reaches where "extra" losses are exceptionally high. Adjust hydraulic properties of section to accommodate these losses. Backwater curves should be prepared to provide the most accurate prediction of the effect of these losses. For long reaches with uniformly distributed losses, depth will approach approximations made as outlined on page 23. The fact that backwater effects often extend for miles upstream in flat slope canals should not be overlooked in setting a freeboard allowance.

7. Compute Froude Number, \( F = \sqrt{\frac{V^2}{gA}} \). Check local backwater effects in high velocity canals when \( F \) is greater than 0.5.

Following are three other helpful "rules of thumb" which were derived from test analyses:

1. If it is not feasible to control aquatic growths on lining surfaces during periods of peak deliveries and velocities are less than 10 fps, the basic "n" chosen in Step 1 should be increased. The type of aquatic growth which will be expected to occur should be investigated and reference made to Test Data Summary Sheets and Figure 6 in choosing a higher "n."

2. For sinuous reaches of intermediate size canals (hydraulic radii between 3 and 5) increased flow resistance due to curves can be approximated by a factor which varies with "curvature index," a parameter equal to \( \frac{\Sigma \Delta \theta}{L_1} \). The product of \( \left( \frac{\Sigma \Delta \theta}{L_1} \right)(0.001) \) is added to the basic straight reach "n" obtained from Figure 8 to determine the approximate increased value. Previous design procedures which neglected effects of curvature have been satisfactory for canals with hydraulic radii less than 3.0.

3. Freeboard allowances for concrete-lined canals, based on judgment and operating experience, have been established and are provided in various design manuals. These values should be checked for adequacy whenever they are used for a particular canal. Based on analyses of capacity test data, the minimum freeboard allowance from design water surface (computed with a basic "n") to the top of the concrete lining should be sufficient

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to contain design discharge at the theoretical depth computed using basic "n" plus 0.003. (This procedure can be approximated by the expression freeboard = \frac{\text{design water depth}}{10}.) Additional freeboard allowance should be provided in reaches where frequent alignment curves or piers are present, especially when adjustments in invert grade have not been made for these losses.

An example problem for the design of a large concrete-lined canal follows:

Given: 48-foot-wide trapezoidal-shaped cross section
1-1/2:1 side slopes
Design discharge (Q) = 4,600 cfs
Invert slope = 0.00005
Water temperature at time of year when design discharge is required = 76 ° F
Kinematic viscosity, \( v \), from Figure 10 = 0.98 x 10^{-5} \text{ ft}^2/\text{sec}
Manning's formula to be used for hydraulic design

Estimate "basic straight reach n" to be 0.0150 from Figure 5

Compute the following hydraulic properties:

\[ d = 17.30 \text{ ft}, \quad A = 1,279 \text{ ft}^2, \quad R = 11.59 \text{ ft}, \quad V = 3.60 \text{ fps} \]

Compute \( IR = 1.70 \times 10^7 \)

Enter Figure 8 and check original assumption for "n".

As Figure 8 shows "n" for \( R = 11.59 \) to be 0.0154, the original assumption was within 2.7 percent of the value shown by the "Average Data Curve." Adjustment of the canal section for an "n" of 0.0154 provides the following hydraulic properties:

\[ d = 17.54 \text{ ft}, \quad A = 1,303 \text{ ft}^2, \quad R = 11.72 \text{ ft}, \quad V = 3.53 \text{ fps}, \]

\[ h_v = 0.194 \text{ ft}, \quad IR = 1.69 \times 10^7 \]

The effect of bridge piers and canal alignment on water depth in this canal can then be estimated. The following conditions are assumed to exist and an example for use of the equation on page 21 is presented.

Given: Canal with sufficient length for uniform flow conditions to be established
An average of two bridges per mile
Average $K_p$ per bridge = 0.10
Canal alignment sinuosity amounts to 20° of curve deflection angle per 1,000 feet of canal length

Find: Total head loss and water depth in 1,000 feet length of canal in reach where uniform flow conditions prevail

Solution: Head loss due to boundary shear (friction) in a straight reach of canal flowing at 17.54-foot depth

$$h_f = 1,000 \ (0.00005) = 0.0500 \text{ ft}$$

Head loss due to bridge piers

$$h_p = K_p h_v = (2)(0.10)(0.194)(1,000/5,280) = 0.0073 \text{ ft}$$

Head loss due to curves

$$h_o = K_o h_v = (0.001)(5 \Delta^\circ = 20)(0.194) = 0.0039 \text{ ft}$$

Total head loss in 1,000-foot length of canal

$$0.0612$$

Using the equation given on page 21, $d = d_f (h/h_f)^N$, the canal water depth which will exist in an infinitely long reach with the assumed conditions will be $d = 17.54(0.0612/0.0500)^{0.28} = (17.54)(1.064) = 18.66 \text{ ft}$.

It is emphasized that water will reach this depth only if sufficient length of canal is available. In canals built on extremely flat slopes (i.e., the example slope of 0.00005), this length may amount to many miles.

The effect of water temperature on flow resistance can also be obtained from Figure 8. If design discharge occurred at 56°F, $v = 1.28 \times 10^{-8}$ ft²/sec and $R = 1.29 \times 10^7$. Figure 8 shows "n" for this condition to be 0.0157. This indicates a 2.0 percent increase in "n" for a 20° drop in water temperature.
The Figures 12 through 27 are Prototype Test Data Summary Sheets. The information presented in these sheets provides brief descriptions of 243 tests made in the 9 concrete-lined canals shown in Figure 1. Each test has been assigned a reference number which is shown in the first column. Other listed information includes properties of each test reach and canal section, method of discharge measurement and water temperatures which existed during tests. Computed values of Manning's "n", friction factor "f", and Reynolds' Number \( R \), are shown for each test. Representative values of Reynolds' Number \( R_{BK} \), Froude Number \( F \), and Chezy C are shown. Pertinent comments concerning tests are shown in the Remarks column.

Analyses of data are continuing and it is possible that minor changes may be made in values of "n", f, \( R \), \( R_{BK} \), \( F \), and C. These changes will not be of sufficient significance to affect conclusions set forth in this Technical Memorandum.

In the opinion of the writers, values of "n" and "f" shown are within ± 2 percent of actual values for all canals except the Delta-Mendota Canal. Representative test data from this canal are shown in Figures 14 and 15. Accuracy of computed values of "n" and "f" from this canal is probably ± 4 percent due to the many variables which influenced flows. These conditions are described in the Delta-Mendota Canal narrative of the Appendix.

Two of the main factors which influence the accuracy of results from flow tests in canals are the method of discharge measurement and the ability to hold steady flow. Extreme care was taken to measure canal discharges with the best available current meter equipment, and canal water stages were held as constant as possible during the test periods. The desired degree of refinement for both of these factors was not always obtained. Therefore, when more than one current meter gaging was made during a test, an average value was used for computing test data. Similarly, a few slight adjustments to measured water surface elevations were made when water surface recorder charts indicated rising or falling stages of a significant amount. These adjustments are described in the Remarks column of the Prototype Data Summary Sheets.

Canal cross sections and invert grades were surveyed in portions of all test canals. These measurements are described for each canal in the Appendix. In general, areas of surveyed cross sections were within ± 1 percent of the design area. Little variation in cross sections was noted during inspections of the unwatered
canals although a slight waviness of the invert was observed in several locations. This characteristic appears to be a random occurrence inherent in machine-placed linings.

Average hydraulic radius was used to compute \( n, f, R, R_K, \) and \( C \). No corrections were made for changes in velocity distributions between measuring stations; i.e., the velocity distribution coefficient was assumed to be 1.0. Computations were made by slide rule and carried to three places.

Backwater curves existed for most tests. Where the computed energy gradient \( (S_e) \) was less than 95 percent of the canal invert slope \( (S) \), an M1-type Backwater Curve is indicated on the summary sheets. Where \( S_e \) was greater than 1.05\( (S) \), an M2-type Backwater Curve is indicated.

The USBR is continuing additional capacity tests in not only concrete-lined canals but also in earth channels and closed conduits. This information will be published and made available for distribution as sufficient new information is collected.
### MAIN CANAL - COLUMBIA BASIN PROJECT

**Prototype Test Data**

#### REMARKS

Water surface stage changes shown by recorder charts were not considered to be significant. Pit tests, attempted at each point, were not made due to the high stage. Further observations were made to determine that observed data agreed with those obtained without corrections for all test. In fact, little error was on minor contaminants was made based on water surface allowed to estimated levels of the time of discharge measurements.

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<table>
<thead>
<tr>
<th>Test Reach Description</th>
<th>Test Measurements</th>
<th>Geometric Properties of Test Section</th>
<th>Water Surface</th>
<th>Hydrologic Properties of Test Section</th>
<th>Water Quality</th>
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*Data from tests marked with asterisks were used on Figures 2 and 3.*
### Test Reach Description

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<th>Transit or Elevation</th>
<th>Water Stage</th>
<th>Hydraulics Properties of Test Section</th>
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### Test Measurements

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<th>Horizontal Curve Data</th>
<th>Transit or Elevation</th>
<th>Water Stage</th>
<th>Hydraulics Properties of Test Section</th>
<th>Remarks</th>
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<tr>
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### Hydraulic Properties of Test Section

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<td>(ft.)</td>
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Water surface stage changes shown by recorder charts were not considered to be significant for tests except those on 9-19-60 and 9-20-60. On these days a foaming stage existed. Computations on this sheet were based on observed water surface elevations without corrections for all tests, except for tests referenced 244 and 254. A second computation was made for these two tests based on water surfaces adjusted to estimated levels at the time of discharge measurements.

See preceding Sheet (Figure 13) for other notes which apply to this reach.
### Table 1: Hydraulic Properties of Test Sections

<table>
<thead>
<tr>
<th>Test Section</th>
<th>Length (ft)</th>
<th>Radius (ft)</th>
<th>Hydraulic Properties</th>
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<tbody>
<tr>
<td>Test 1</td>
<td>30</td>
<td>300</td>
<td>0.59</td>
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<td>Test 2</td>
<td>30</td>
<td>300</td>
<td>0.59</td>
</tr>
<tr>
<td>Test 3</td>
<td>30</td>
<td>300</td>
<td>0.59</td>
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</table>

### Table 2: Test Measurements

| Test | Head Loss (ft) | Flow Rate (cfs) | | Head Loss (ft) | Flow Rate (cfs) |
|------|----------------|-----------------| | Head Loss (ft) | Flow Rate (cfs) |
| Test 1 | 1 | 10 | | Test 1 | 1 | 10 |
| Test 2 | 2 | 20 | | Test 2 | 2 | 20 |
| Test 3 | 3 | 30 | | Test 3 | 3 | 30 |

### Table 3: Measurements of Test Sections

| Test | Length (ft) | Radius (ft) | | Length (ft) | Radius (ft) |
|------|------------|------------| | Length (ft) | Radius (ft) |
| Test 1 | 30 | 300 | | Test 1 | 30 | 300 |
| Test 2 | 30 | 300 | | Test 2 | 30 | 300 |
| Test 3 | 30 | 300 | | Test 3 | 30 | 300 |

### Table 4: Typical head loss in this reach due to piers = \( \Delta \text{K} \) per mile

<table>
<thead>
<tr>
<th>Piers</th>
<th>Head Loss (ft)</th>
<th>Flow Rate (cfs)</th>
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<tbody>
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<td>4</td>
<td>2.5</td>
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</tbody>
</table>

### Table 5: Reynolds' Friction Numbers

| Test 1 | 1 | 10 |
| Test 2 | 1 | 10 |
| Test 3 | 1 | 10 |

### Table 6: Data from tests marked with asterisks were used for Figures 5 and 8

<table>
<thead>
<tr>
<th>Test</th>
<th>Head Loss (ft)</th>
<th>Flow Rate (cfs)</th>
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<tbody>
<tr>
<td>Test 1</td>
<td>1</td>
<td>10</td>
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<td>20</td>
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<tr>
<td>Test 3</td>
<td>3</td>
<td>30</td>
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### Figure 14

- Test 1:
  - Concrete bridge, 2 piers
  - Data from tests marked with asterisks were used for Figures 5 and 8.
- Test 2:
  - Concrete bridge, 2 piers
  - Data from tests marked with asterisks were used for Figures 5 and 8.
- Test 3:
  - Concrete bridge, 2 piers
  - Data from tests marked with asterisks were used for Figures 5 and 8.

The water surface elevation shown for Test 2 is an average of two readings.
<table>
<thead>
<tr>
<th>Test Section</th>
<th>Hydraulics of Test Section</th>
<th>Reynolds' Formula</th>
<th>Chezy's Formula</th>
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<tr>
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<td>Description</td>
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**Remarks:**
- See profile sheet for General remarks which apply to these sections.
- For more detailed listing of additional tables and charts, see Appendix C.
- See Figure 15 for additional information.
## Test Reach Description

### Properties of Design Section

- **Surface Traction**: 0.68
- **Side Slop**: 1:3.0
- **Vertical Relief**: 0.5m
- **Transverse Grooves**: Provided in the center of the center line of the test reach.
- **Hydraulic Design**: Based on Kutter’s formulas.
- **S - Curve Rate**: 3.3% with 3.2% curves per mile.

### Summary Sheet

<table>
<thead>
<tr>
<th>Test Measurements</th>
<th>Interim Properties of Test Section</th>
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### Remarks

For Figures 5 and 8.

*Data from tests marked with crosses were used for Figure 5 and 8.*
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<th>TEST DESCRIPTION</th>
<th>TEST MEASUREMENTS</th>
<th>PHYSICAL PROPERTIES OF TEST SECTION</th>
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**Water surface measurement at Point 2 made on 7-26-65 opposite to & in error, and was therefore omitted.**
<table>
<thead>
<tr>
<th>Station (ft)</th>
<th>Elevation (ft)</th>
<th>Flow (cfs)</th>
<th>Temperature (°F)</th>
<th>Specific Conductance (us/cm)</th>
<th>Water Temperature (°F)</th>
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<tr>
<td>0.0</td>
<td>80.5</td>
<td>0.3</td>
<td>65</td>
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<td>104</td>
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<td>68</td>
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<td>70</td>
<td>110</td>
<td>75</td>
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</table>

*Note: Measurements were taken at 1-hour intervals.*

The following tables and graphs provide detailed data on the flow, temperature, and specific conductance of the water at various stations. The data was collected using high-precision equipment and validated through multiple rounds of testing. The graphs illustrate the trend over time, showing a consistent increase in flow and temperature, with a slight decrease in specific conductance. The data is further supported by detailed field observations and laboratory analyses.
<table>
<thead>
<tr>
<th>Test Reach Description</th>
<th>Test Measurements</th>
<th>Roughness of Riverbed</th>
<th>Water Surface</th>
<th>Reynolds' Froude Number</th>
<th>Remarks</th>
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</tbody>
</table>

*Data from tests marked with asterisks were used for Figures 3 and 6.
### Test Reach Description

- Test Reach: Central Valley Project
- Location: California
- Research (Reoch) Section

#### Test Measurements

- Hydraulic Properties
- Design was determined between stationing
- Contraction section at toe of embankment

#### Test Section Design

- Water surface elevations for 1958 tests at Points A and B were recorded on this sheet to the nearest hundredth foot.
- Water surface elevations shown are average values for three sets of measurements on each day.

#### Test Section Data

<table>
<thead>
<tr>
<th>Date</th>
<th>Flow Rate</th>
<th>Discharge</th>
<th>Velocity</th>
<th>Depth</th>
<th>Width</th>
<th>Slope</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/1/58</td>
<td>1000</td>
<td>1200</td>
<td>2000</td>
<td>3000</td>
<td>4000</td>
<td>5000</td>
</tr>
<tr>
<td>2/1/58</td>
<td>1500</td>
<td>1800</td>
<td>2200</td>
<td>3000</td>
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<td>5000</td>
</tr>
<tr>
<td>3/1/58</td>
<td>2000</td>
<td>2400</td>
<td>2800</td>
<td>3200</td>
<td>4400</td>
<td>5600</td>
</tr>
</tbody>
</table>

The data includes discharge measurements and velocity calculations for various flow rates. The discharge is measured in thousand cubic feet per second (cfs), velocity in feet per second (fps), depth in feet (ft), width in feet (ft), and slope in percent (%).
<table>
<thead>
<tr>
<th>Test Number</th>
<th>Method of Replacement</th>
<th>Average Water Surface Elevation (ft.)</th>
<th>Uniform Water Surface Elevation Assumed</th>
<th>Water Surface Elevation in Meters</th>
<th>Water Surface Elevation in Feet</th>
</tr>
</thead>
<tbody>
<tr>
<td>Test 4208</td>
<td>G ECC 1000</td>
<td>1.45</td>
<td>1.15</td>
<td>1.14</td>
<td>1.45</td>
</tr>
<tr>
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<td>1.15</td>
<td>1.14</td>
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<tr>
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<td>1.15</td>
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<td>1.15</td>
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<td>1.45</td>
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</tbody>
</table>

Note: Water surface elevations at points C and D for Test 4208 were corrected from measured values of 1.45 ft. to 1.15 ft. Uniform water surface was assumed between points C and D for Tests 4209 to 4213.

*Data from tests marked with asterisks were used on Figures 5 and 8.

**HORIZONTAL CURVE DATA**

Test Measurements and Hydraulic Properties of Test Section

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Method of Replacement</th>
<th>Average Water Surface Elevation (ft.)</th>
<th>Uniform Water Surface Elevation Assumed</th>
<th>Water Surface Elevation in Meters</th>
<th>Water Surface Elevation in Feet</th>
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</thead>
<tbody>
<tr>
<td>Test 4208</td>
<td>G ECC 1000</td>
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<td>1.15</td>
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</tbody>
</table>

Note: Water surface elevations at points C and D for Test 4208 were corrected from measured values of 1.45 ft. to 1.15 ft. Uniform water surface was assumed between points C and D for Tests 4209 to 4213.

*Data from tests marked with asterisks were used on Figures 5 and 8.*
**SUMMARY SHEET**

**PROTOTYPE TEST DATA**

<table>
<thead>
<tr>
<th>TEST DESCRIPTION</th>
<th>TEST MEASUREMENTS</th>
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<tbody>
<tr>
<td><strong>HYDRAULIC PROPERTIES OF TEST SECTION ROUGHNESS</strong></td>
<td><strong>REYNOLDS' FLOW CHART</strong></td>
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</table>

Water surface graphs used in these tests were developed by the indifference method and utilized a pilot tube and back gauge sliding wall principle. The figure to clear and closed water surface conditions at both sides of the reach at peak discharging station. The water surface shown in this chart is in stage of minimum flow. Measurements were made on each side of the center of each station.

**FIGURE 22**

No structures with parts outside limits were marked to show the effect of structures to major and minor flows. All structures were considered to be located within reach of the main flow. Water levels fluctuated slightly during test periods. Measured lines and water surface elevations were used for computations without adjustments. Corrections of lines and water levels for friction factors were estimated to have an insignificant effect less than 2% on computed values of Q, F, and A. Water surface elevations used to report measurements that were used to compute Q, F, and A.
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<th>DATE LANE</th>
<th>TEST BEAM DESCRIPTION</th>
<th>APPEARENCE OF CURVE</th>
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<th>PEAKS OF CURVE</th>
<th>CHANGES IN TESTING</th>
<th>METHOD OF MEASUREMENT</th>
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(See preceding sheet [Figure 31] for Remarks and details of Cu 60 treatments.)
<table>
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<th>Test Reach Description</th>
<th>Test Measurements</th>
<th>Hydraulic Properties of Test Section</th>
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<td>Total Energy Loss [Joules]</td>
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</tbody>
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**Test Reach Description**
- Straight reach
- Curve
- Discharge
- Roughness

**Test Measurements**
- Volume in Section
- Flow Depth
- Flow Velocity
- Angle of Approach
- Angle of Departure
- Roughness
- Discharge
- Total Energy Loss

**Hydraulic Properties of Test Section**
- Volume in Section: [cubic meters]
- Flow Depth: [meters]
- Flow Velocity: [m/s]
- Angle of Approach: [degrees]
- Angle of Departure: [degrees]
- Roughness: [cm]
- Discharge: [cubic meters per second]
- Total Energy Loss: [Joules]
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<th>Remarks</th>
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**Table:**

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<tbody>
<tr>
<td>78 +00</td>
<td>0.1240 ft</td>
<td>Flow data collected at 0.1240 ft to complete 0.001 ft.</td>
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**Diagram:**

- Curve 1: Monitoring Line Flow
- Curve 2: Flow data collected at 0.1240 ft to complete 0.001 ft.
### Horizontal Curve Data

<table>
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<th>Deflection Angle (°)</th>
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### Water Surface

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### Hydraulic Properties of Test Section

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<tbody>
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### Test Measurements

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<td></td>
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</tr>
</tbody>
</table>

### Remarks

- Data from tests marked with asterisks were used in figures 5 and 8.

**Figure 27**

- The greater downstream depths were caused by a triangular shaped dam, 58 inches high, on the crest of the canal at the dam end. See Figure 1-5.
ACKNOWLEDGMENT

The capacity tests described in this Technical Memorandum have required the efforts of many individuals throughout the Bureau of Reclamation. The authors are indebted to many engineers and irrigation operations personnel for their cooperation, comments, and criticisms. Notable among these are D. J. Hebert of the Regional Office in Sacramento, California, and P. W. Terrell, W. E. Schneider, J. W. Morris, W. B. McMorney, and C. W. Thomas of the Office of Chief Engineer, Denver.

The program was outlined and data analyses performed in the Denver Office under the direction of H. K. Brickey, Canals Branch, and A. J. Peterka, Hydraulics Branch. J. C. Schuster, R. E. Dexter, and J. M. Bergmann of the Hydraulics Branch directed prototype field tests. Denver Office Engineers F. F. McIntire, J. T. Netzer, J. R. Kamicar, and J. D. Darles, who are presently in the Canals Branch, and D. P. LaGatta and P. C. Knodel, who are presently in the Soils Engineering Branch, assisted in the data analyses. D. J. Hebert was instrumental in organizing the first capacity tests in the Central Valley Project canals in 1957. He directed subsequent testing in those canals and analyzed much of the original data.
REFERENCES


44


14. Control of Attached Algae on Concrete-lined Irrigation Structures with Copper-Bearing Antifouling Paint, Chemical Engineering Laboratory Report No. SI-23, Office of Chief Engineer, USBR, Denver, Colorado, 1959


20. Reh, Uwe, Simplified Application of Rehbock's Formula for the Backwater Caused by Bridge Piers, Translation No. 59-5, U.S. Army Waterways Experiment Station, Vicksburg, Mississippi, 1959

APPENDIX

DESCRIPTIONS OF CANAL CAPACITY TESTS AND DISCUSSIONS OF RESULTS
Section A

MAIN CANAL

(Long Lake Reaches)
Columbia Basin Project, Washington
MAIN CANAL
(Long Lake Reaches)
Columbia Basin Project, Washington

A series of 14 water surface profiles was documented and other hydraulic performance data were obtained from a 4.5-mile section of the Main Canal immediately downstream from Long Lake during the 1960 irrigation season. The information was obtained between March and October at flows which varied from 865 to 6,215 cfs. Water depths were greater than anticipated for all tests except the first two in March.

This 4.5-mile section of the concrete-lined Main Canal was designed for a flow of 9,700 cfs and supplies water to the West Canal and East Low Canal as shown in Figure A-1. It was the largest lined section to be documented in the current series of tests. Tests at design discharge were desired but could not be obtained because flows were limited by the capacity of existing upstream conveyance facilities. Some of the test flows in 1960 exceeded irrigation demands at the stage of project development which existed at that time. Excess water, up to a maximum of 1,700 cfs during the test of September 19, was discharged through downstream wasteways.

Comprehensive water surface profile measurements were obtained in two reaches of the 4.5-mile section. A 5,432-foot sinuous reach, hereinafter called Reach 1, and a 5,614 straight reach, Reach 2, were selected as typical locations for the tests. Figures A-1 through A-4 show location, alinement, profile, and section details concerning these reaches. Two 20-foot timber deck highway bridges and a Bailey bridge were located between the reaches. The two timber bridges were supported by square post concrete piers which extended into the flow prism. Head loss measurements were obtained across the two timber bridges (Figures A-5 and A-6) and current meter discharge measurements were made from the clear span Bailey bridge.

Concrete lining for this canal was placed under Specifications No. 2324 in 1949. A description of construction activities can be found in the "Long Lake Dam and Main Canal, Technical Record of Design and Construction," pages 123 to 140, which was published by the Bureau of Reclamation in 1955. A large rail-mounted slip-form was used to line the entire canal prism in one pass. Finishing of the concrete lining was accomplished by 4- by 12-foot floats or ironer plates which were semirigidly attached behind the skinplate of the lining machine. The Technical Record states "the surface of the concrete was left with a satisfactory finish by the lining machine and very little troweling was
Information on this Figure obtained from Drawing 222-116-5453.

NOTE

MAIN CANAL — COLUMBIA BASIN PROJECT
STA 752+965 TO 1001+29
LONG LAKE REACHES
ALIGNMENT

FIGURE A-2
Note: Construct embankments as shown on profiles.

SECTION NO. 5
LINED, ROCK AND EARTH

Lining Detail
Rock Excavation

3 Long grooves @13'-6" crs

5 Long grooves @10'-0" crs

Drainage and outlet as directed.
Trench filled with gravel as directed.

Earth or rock spall fill as directed.

Slope as directed in rock

Trench filled with gravel as directed.

22'-0" Min.

Approx 1 Ft

6'-Min.

Original ground surface

Finish on core banks and berms

Approx 1 Ft of cover

Original ground surface

Note: Construct embankments as shown on profiles.

HYDRAULIC PROPERTIES

<table>
<thead>
<tr>
<th>Section</th>
<th>A</th>
<th>V</th>
<th>D</th>
<th>r</th>
<th>c</th>
<th>e</th>
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<td>168'</td>
<td>10</td>
<td>9700</td>
<td>13.48</td>
<td>0.14</td>
<td>0.001</td>
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</table>

NOTES

Reinforced or unreinforced lining to be constructed where directed.
Cap all bars 34 diameters, unless otherwise shown.
Provide transverse grooves at 12'-6" centers.

See Dwg. 222-D-12743 for details of safety ladders

UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT-WASHINGTON

MAIN CANAL - STA. 752+96.5 TO STA. 110+29.0

CANAL LINING

FIGURE A-4
required. * * * the major finishing job consisted principally of groove repairing and cutting."

Tolerances for placing the concrete lining were provided in the specifications. Transverse and longitudinal contraction grooves were located at approximate 12-foot intervals and were filled with asphalt mastic. Figures A-7, A-8, and A-9, taken in March 1960, show surface texture and joints. Actual measurements of mastic filler in grooves within the flow prism showed the mastic to be generally even with or below the concrete surface. Maximum measured depression below the surface was 5/16 inch.

Concrete lining surface irregularities were measured at selected locations within the test reaches from lines normal and parallel to the canal centerline. Measurements on planes parallel to the centerline were made from a 36-foot-long taut nylon string. Measurements from lines normal to the centerline were made from taut strings attached at the top and bottom of each slope and at each edge of the invert. Measurements were made at irregular intervals of 2 to 3 feet. The average measured deviation from a plane surface was 0.03 foot and the maximum deviation was 0.15 foot.

Reaches 1 and 2 contained no in-line canal structures. Flow depths in the canal were controlled by radial gates at the check structure and the East Low Canal turnout structure near the bifurcation point. This point is 1.7 miles below the end of Reach 2 as shown in Figure A-2. Tests were made at approximate 1,000-cfs flow increments and under various operating conditions. For the 14 tests, five M1-type backwater curves and nine M2 curves were documented. Nearly uniform flow conditions existed for several of these tests, as evidenced by the very small changes in water depth from end to end of the test reaches. Figure A-3 shows plots of nine typical water surface profiles.

Kutter's formula with a roughness coefficient "n" of 0.014 was used for the design of this canal. No extra allowances were provided for head losses across bridge piers, horizontal curvature in canal alignment, or for increased flow resistance due to aquatic growths. Equivalent Manning's "n" is 0.0146. (Discharge computed using "n" = 0.014, Manning's formula, and design depth is 10,130 cfs, 4 percent greater than the design discharge.) A vertical freeboard allowance of 2.77 feet was provided from the design water surface to the top of the concrete lining.

In addition to the two bridges, 20 horizontal curves in canal alignment exist between Points A and E. These curves are described in Table A-1.
View of straight Reach No. 2 of the Main Canal during the unwatered period before the start of the 1960 irrigation season. The canal was cleaned of rocks and debris before the capacity tests. P222-116-42181, March 17, 1960.

Figure A-7
Closeup view showing surface texture of a side of the concrete lining and typical contraction groove on Main Canal at approximate Station 981+00. Measurements generally showed mastic filler to protrude above the lining surface above the waterline. Below the waterline the filler was generally below the concrete surface, by a maximum measured amount of 5/16 inch. P222-116-42180, March 17, 1960.

Figure A-8
A. Closeup view showing typical concrete surface texture, contraction groove condition, and dried fresh water sponge (tentatively, spongilla Fragilis) growth. Photograph was taken in Reach 2 prior to 1960 capacity tests. P222-116-42188, March 17, 1960.

B. Typical concrete lining surface texture and contraction groove condition in sinuous Reach 1 of Main Canal. The lining was 11 years old in the spring of 1960 when this photograph was taken. P222-116-42189, March 17, 1960.

Figure A-9
Elevations for these tests were based on the standard datum used during construction of the canal. Relative elevations at measuring stations were determined by level circuits and checked with a static pool test. Averaged adjusted level elevations were used for the data analyses.

Description of Tests

As this canal is unwatered during the winter months, it was possible to secure survey information and photographs of the empty canal prior to and following the series of tests. When the canal was inspected prior to the tests a few small rocks were noted on the invert, but these and all other debris-type material were removed before water was turned into the canal. Figures A-7 through A-12 are photographs of the unwatered canal prior to and after test measurements.

Surveyed invert profiles were plotted in Figure A-3. In Reach 1, 8 of the 15 surveyed points were above design grade, 1 was on grade, and 6 were below grade. Deviations from design grade averaged about 0.04 foot in this reach. In Reach 2, all of the 13 surveyed points were above grade by an average of 0.07 foot. Maximum deviations from grade in the two reaches were +0.14 and -0.07 foot.

Cross sections at the seven water surface measuring points, A through G, were plotted from survey data. These plots indicated the measured cross section to be within plus or minus 1/2 percent of the design areas at test flows. No unusual humps or distressed areas were observed in the concrete lining in test reaches. Cross sectional areas for data analyses were computed from the design section shown in Figure A-4 using an average invert elevation determined from 1960 surveys.

Discharge measurements of test flows were made from the Bailey bridge located between Reaches 1 and 2 at Station 871+00. Comprehensive velocity traverses were obtained with a Price Type A current meter at 2- to 3-foot intervals across the section. Because of the large number of readings to be taken, the time required to make these measurements varied from 3 hours for the 865-cfs run on March 28 to 9-1/2 hours for the 5,855-cfs run on September 19. The Hydraulics Branch used data from the vertical velocity profiles to compute discharges and to analyze the theoretical boundary shear distributions. A computer program was written to facilitate these analyses. The results of this study will be utilized by Hydraulics Branch personnel in a continuing study of the boundary shear distribution mechanics around the perimeter of various open channel shapes.
An electronic computer program was also used to compute test discharges from the comprehensive current meter traverses. The program provided for the integration of discharges from approximate 2-foot-square areas over the entire canal cross section. The section was broken down into a grid, and average velocities for the elemental areas were computed from the vertical velocity profiles. Discharges obtained by the computers were used for data analyses. Computer discharges were less than those obtained from the standard 0.2, 0.8 depth method of computation for all 14 tests by an average of 1.4 percent. The maximum deviation of -3.6 percent occurred on the March 28 run.

Water samples to determine sediment concentrations were obtained at the Bailey bridge, Station 871+00, during the first five test runs, utilizing a depth integrating sediment sampler. It was thought that sediment might be a factor which influenced aquatic weed growth or flow resistance. These samples were analyzed in the Denver laboratories and sediment concentrations were found to be 0.2 to 0.6 ppm by weight. Since the concentrations appeared to be insignificantly small, the project was notified to discontinue this sampling unless system operations changed in a manner which might introduce appreciable sediment.

Continuous recordings of water stages in the canal were obtained at two locations. Prints from recorders located at Station 13+79, upstream from Point A, and 1100+08, downstream from Point G, showed a continuous trace of the water surface for the entire 1960 irrigation season. During the 14 tests the most notable stage change of about 0.03 foot per hour occurred on September 19 and 20. Allowance was made for this change on these 2 days in data analyses summarized in Figures 12 and 13.

No turnouts were located in Reach 1 and there was only one 2-cfs turnout in Reach 2. As no record of water deliveries from the turnout was submitted with test data, it was assumed that the turnout gate was closed during the tests. No intermediate inflows into the test reaches occurred. An asphalt-surfaced deer escape was located near the midpoint of Reach 1, but it did not encroach appreciably on the flow prism and was not considered to affect significantly the test data analyses.

Gages developed by the Hydraulics Branch were used to measure water surface elevations. These gages utilize a pitot tube and hook gage stilling well principle and are described in the main body of this Technical Memorandum. At least five readings of water surface elevation were obtained on each side of the canal at each measuring point during each test.
Seepage and evaporation losses in this wide canal were estimated to be 1 cfs per mile. Discharge measurements were made between Reaches 1 and 2. Allowance for losses was made by rounding discharges up to the next higher 5 cfs in Reach 1 and down to the next lower 5 cfs in Reach 2. This adjustment amounted to less than 0.6 percent of the lowest test flow.

Time required to obtain all of the water surface elevations for one test in the 4.5 miles of canal varied from 3 to 6 hours. The time required was reduced as crews became more familiar with the gages. Also, the checked flow runs took less time because less difficulty was encountered in moving up and down the canal side slope. Measurements in Reach 1 were completed in 1 to 3.5 hours, those in Reach 2 were completed in 0.7 to 1.5 hours. It was noted that weather conditions changed rapidly on some days. For example, on June 13, weather was recorded as calm at the start of measurements, but the wind increased enough to form waves which overtopped water surface gage stilling wells before measurements were completed.

Photographs taken in the unwatered canal in March (Figures A-7, A-8, and A-9) showed relatively clean concrete lining surfaces in the test reaches except for remains of fresh water sponge growth (tentatively, spongilla Fragilis). Photographs taken in November (Figures A-10 through A-12) show considerable evidence of this growth on the lower part of the canal section. This growth has covered up to one-third of the bottom and sides of smaller canals in other parts of this project. Fresh water sponge growth has also been noted in the upper half of the barrel of the 25-foot-diameter portion of the Soap Lake Siphon. In addition, considerable amounts of Nostoc (an algae) and filamentous algae were reported during the tests, the greatest concentration occurring near the bifurcation point at the end of the canal.

Crews who obtained water surface elevation measurements in Reach 2 during October reported that they had difficulty getting down to measurement platforms because the lower portion of the lining was covered with very slippery algae. These crews also experienced considerable difficulty with pondweed and horsetail moss which tangled with water surface gages and current meters in August and September. Unfortunately, techniques and equipment were not available on this project in 1960 to examine lining surfaces beneath the water. Therefore, detailed observations of aquatic growth were not made of test reaches during these tests.

The flow required to supply project demands in 1960 did not tax the capacity of the canal; therefore, the water was not chemically treated to retard aquatic growths, and test results
A. Gemmule patterns of fresh-water sponge growth in the bottom of the Main Canal approximately 200 feet upstream from bifurcation works at Mile 6.6. These 1-foot diameter patterns have a small amount of sponge skeleton covered with filamentous algae overlying the gemmules or egg-like reproductive parts. Photograph taken at end of 1960 irrigation season. P222-116-42727, November 16, 1960.

B. Closeup of the algae, Nostoc, in its wintering stage on the right side of the concrete lining of the Main Canal. This photograph was taken immediately upstream from the bifurcation works at Mile 6.6. P222-116-42731, November 16, 1960.

Figure A-10
A. Fresh-water sponge on the left bank of the Main Canal, immediately upstream from the measuring well at the bifurcation works. This photograph shows patterns of infestation which include numerous small colonies and groups of colonies completely covering an area. These patterns result chiefly from deposits of gemmules, egg- or seed-like reproductive bodies, and a small amount of the spongy skeleton. P222-116-42728, November 16, 1960.

B. Fresh-water sponge on left bank of concrete lining immediately upstream from the bifurcation works. The scale shown is 6 inches long. Average thickness of sponge at this site was 1 inch. P222-116-42725, November 14, 1960.
Main Canal at bifurcation works, looking upstream. Stilling well at right is at Station 1100+08 and is equipped with a continuous recorder to record water stages in the canal. Canal bottom width is 50 feet and lining height is 23.50 feet.
P222-116-42729, November 16, 1960

Figure A-12
reflect all effects of such growth on hydraulic resistance. Future testing in this canal is planned to determine the effectiveness of various chemicals in retarding underwater growths peculiar to this project.

Head loss measurements across two 20-foot timber bridges in Reach D-E were made on August 15 and 22 and on September 19 and 20. These bridges are supported by concrete bents, each bent consisting of two 18-inch square concrete posts. Four of the bents for the bridge at Station 831+66 were in the flow prism and three were in the flow prism at Station 942+53. Water surface elevation measurements were made 100 feet upstream and 200 feet downstream from the centerline of the bridges.

Analyses of Test Data

Figure A-3 is a plot of 9 of the 14 water surface profiles which were recorded at approximate 1,000-cfs flow increments, starting at 865 cfs on March 28, increasing to 6,215 cfs on June 27, and decreasing to 955 cfs on October 28. Five M1 and nine M2 curves were obtained in the 4.5-mile reach between Points A and G.

Figures 12 and 13 summarize in more detail data concerning the test reaches, measurements, and computed results. Tests numbered 1 through 28 show results for overall test Reaches A-D (Reach 1) and E-G (Reach 2). Results were also computed but not shown in this Technical Memorandum for shorter segments, i.e., A-B, B-C, C-D, etc. Values of Manning's "n" and friction factor "f" were computed for these shorter segments and found to vary less than 10 percent from corresponding values obtained for the overall reaches. The average values for all shorter segments within an overall reach were about the same as the overall values.

The seasonal change of flow resistance in terms of Manning's "n" is shown in Figures A-13 and A-14. Values of "n" for the low velocity checked flow runs of April 18, May 2 and 9, and October 3 were omitted from these figures. On these dates the measured energy loss in Reaches 1 and 2 varied from 0.085 to 0.343 foot. An error in the measurement of water surface of 0.01 foot under these circumstances would result in an error in the computed friction slope of 3 to 12 percent. Weather conditions recorded on these four days appeared favorable for good measurements; nevertheless, "n" values were consistently high. The remaining 10 tests provided M2-type backwater curves or conditions which were very close to uniform flow. When only these 10 tests are considered, hydraulic resistance coefficients increased about
Note: This reach of canal was not chemically treated to retard aquatic growths.

**Figure A-13**

**Main Canal**

5432 Ft. Sinuous Reach 1

Seasonal Change in Manning's "n"

1960 Tests
Note: This reach of canal was not chemically treated to retard aquatic growths.
30 percent (0.014 to 0.019) from March through September and decreased approximately 10 percent (0.019 to 0.017) from September through October.

This series of tests appears to indicate an interesting trend in that velocity of the water may have a significant effect on the resistance coefficient where algal growth occurs on rigid boundary channels. An examination of test results for straight Reach 2 indicates that, after seasonal variations in algal growth are considered, "n" values were lowest when velocities were highest. Results from sinuous Reach 1 show the same general trend but are less consistent.

Horizontal curves in Reach 1 apparently increased the hydraulic resistance slightly over that which existed in straight Reach 2. Values of "n" in Reach 1 were generally (but not always) higher than those in Reach 2. The average "n" for 10 tests in Reach 1 was 0.0171; in Reach 2, 0.0170. Horizontal curve information is given in Table A-1.

Figure 11 summarizes head loss measurements made across the two 20-foot timber bridges in Reach D-E. The maximum measured rise in canal water surface between a point 200 feet downstream from the bridge and a point 100 feet upstream was 0.10 foot and the minimum was 0.03 foot, for flows of 4,545 to 5,855 cfs. The head loss due to canal boundary shear only was computed for the 300-foot reach, based on measured friction coefficients, and varied from 0.02 to 0.04 foot. Therefore, the head loss attributable to piers only varied from 0.01 to 0.06 foot. Values of $K_p$ computed for use in the equation $h_v = K_p h_v$ varied from 0.03 to 0.19. The average values for $K_p$, as shown in Figure 11, were 0.15 for the four-bent bridge and 0.10 for the three-bent bridge.

Summary and Discussion of Test Results

Hydraulic flow resistance in this large open channel varied seasonally, and was generally greater than anticipated. The major cause of the increase in resistance was apparently aquatic growths on the concrete lining which reached their peak coverage in August. Project personnel reported that filamentous and Nostoc algae and fresh water sponge growth occurred in varying degrees on the lining during the entire irrigation season. Computed Manning's "n" values increased about 30 percent (0.014 to 0.019) between March and September, and decreased about 10 percent (0.019 to 0.017) between September and November.
<table>
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<th>Reach description</th>
<th>Description of horizontal curves</th>
<th>Av. Manning's &quot;n&quot; for 10 tests</th>
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<td>From</td>
<td>To</td>
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<td>A</td>
<td>B</td>
<td>1,650</td>
</tr>
<tr>
<td>B</td>
<td>C</td>
<td>2,177</td>
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<tr>
<td>C</td>
<td>D</td>
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<td>E</td>
<td>G</td>
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</table>

Reach 1: No. 1
Reach 2: No. 2
Data from the 10 test runs which exhibited M2 backwater curves or conditions close to uniform flow (depth change less than 0.7 foot and maximum 6.7 percent depth change in 4 miles of canal) are considered most applicable to design conditions. Considering only these tests, the average value of Manning's "n" was 0.0171 for the sinuous Reach 1 and 0.0170 for the straight Reach 2. A minimum value of 0.0133 was obtained in Reach 1 for the March 28, 865-cfs run. A maximum value of 0.019 was obtained in Reach 2 for the August 22, 4,545-cfs run.

No satisfactory explanation was found for the higher "n" values computed from tests made under checked conditions and at low velocities. The higher resistance coefficients for these circumstances may be caused by the greater tendency of the filamentous algae and fresh water sponge growth to extend itself into the flow prism and to change boundary layer characteristics. A slight increase in the value of Manning's "n" at lower Reynolds' Numbers is shown by the upward slope of the "Average Data Curve" of Figure 8. This sloping line indicates that in a channel where depth is constant, "n" increases with a decrease in velocity. The increase in "n" computed from the Long Lake Reaches was, however, much greater than that indicated by the "Average Data Curve" in Figure 8.

The backwater effects of two sets of square post bridge piers located between test Reaches 1 and 2 were measured. The rise in water surface caused by these piers varied from 0.01 to 0.06 foot when canal flow was approximately 60 percent of design capacity. Based on four measurements, the average computed value of the pier loss coefficient, $K_p$, was 0.15 for the bridge at Station 831+66. This bridge was supported by four bents which extended into the flow prism, each consisting of two 18-inch square posts. The corresponding value for $K_p$ was 0.10 for the bridge at Station 942+53. This bridge was supported by three similar bents.

Figures 12 and 13 provide more detailed information concerning prototype flow tests made during 1960 in this canal. These figures also show computed values of hydraulic properties and design parameters.
Section B

DELTA-MENDOTA CANAL

Central Valley Project, California
DELTA-MENDOTA CANAL
Central Valley Project, California

Water surface profiles and other hydraulic performance data were obtained from parts of this 117-mile canal in the 4-year period between 1957 and 1961. Tests in 1957 revealed water depths in concrete-lined reaches of the canal to be greater than anticipated and prompted the initiation of additional testing in this and other large Bureau of Reclamation canals.

Figures B-1 through B-4 show condensed versions of the canal alignment, hydraulic properties, and a portion of the invert profile. Figure B-5 shows a section typical of the 95-mile length which is lined with concrete. The 4-inch-thick concrete lining was built under seven different contracts during a 4-year period which began in 1946. As the canal was dedicated in 1951, the lining was 6 to 12 years old at the time of tests which are described in the following pages.

The 48-foot bottom width canal was lined with concrete from Mile 3.5 to Mile 98.6 using large rail-mounted traveling slip-forms. The entire section was paved in one pass of this machine. The lining was unreinforced for most of its length. Specifications requirements prior to 1948 stated "** the finish of the concrete lining shall be equivalent ** to that obtainable by effective use of a long-handled steel trowel." Specifications issued after 1948 provided specific dimensional tolerances for placing and finishing the concrete lining. As many as 26 concrete finishers working from the slip-form, and from a jumbo which followed, were required to secure the specified surface smoothness. Other details concerning this canal are given in Figures 14 and 15 of this Technical Memorandum and in the "Delta-Mendota Canal, Technical Record of Design and Construction," published by the USBR in 1959.

Comprehensive test measurements were confined to the first 70 miles of this canal. Below this point, land subsidence had occurred and the canal invert grade was considerably altered from the design slope. Figures B-3 and B-4 show 1961 water surface profiles together with survey data obtained as a check on as-built invert and top of lining elevations. Invert survey data have been obtained to Mile 70 but were not plotted past Mile 34.4. The reach of canal between the end of the Tracy Pumping Plant discharge lines (Mile 3.5) and San Luis Wasteway (Mile 70.0) contained the following canal structures which have piers in the flow prism:
### DELTA-MENDOTA CANAL
### Design Hydraulic Properties

<table>
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<th>CONCRETE LINED SECTION</th>
<th>A</th>
<th>V</th>
<th>Q</th>
<th>R</th>
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<th>s</th>
<th>d</th>
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<th>d</th>
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**FIGURE B-2**
Water was periodically above top of concrete lining during tests of July 16, 1961. Mannings' 'n' values. Top of concrete lining during July 1961 6-pump tests in Desian show by arrows.


Top of concrete lining (Design location). Water surface fluctuated between these two lines during dry/wet periods. Fluctuation was due to tidal changes in the pumping plant intake works in turn. Fluctuation was due to constant speed pumps.

Design freeboard to top of concrete lining = 1.00 ft. Design water surface invert location (Slope = 0.00005).

Calculated concrete lining with piers in the flow prism in 1961; All aforelisted structures were not shown due to a lack of space.

DELTA-MENDOTA CANAL
WATER SURFACE PROFILES -- MILES 3.5 TO 34.4
6-PUMP FLOW TEST -- JULY 1961

FIGURE B-3


Silt deposits containing fresh water clams were present at various locations on the invert during tests. The deposits were located by underwater survey in January, 1961.

In 1961 the reach of canal shown contained the following on-line structures with piers in the flow prism:
- 21 Timber bridges (3 or 4 piers each)
- 14 Concrete bridges (2 piers each)
- 4 Overheads (3 piers each)
- 2 Pipe crossings (1 or 2 piers each)
- 1 Check structure.

All aforementioned structures were not shown on this drawing due to a lack of space.

---

**Figure B-4**

**Delta - Mendota Canal**

**Water Surface Profile -- Miles 34.4 to 70.0**

6-Pump Flow Test -- July 1961
varied flow. Such conditions do not make for simple analyses of test data.

Figure B-6 lists and describes the single round barrel and nine multiple barrel rectangular inverted in-line siphons between the head end of the canal and Mile 70 and shows results of head loss measurements across these siphons.

Description of Tests

In the period between canal completion in 1951 and 1957, water demands increased rapidly and it soon became obvious that demands would exceed the design capacity of the canal. It was therefore decided to operate the canal at its maximum capacity to find if additional flow could be accommodated and the water marketing program expanded. The first capacity test was held in February 1957 at a time when irrigation deliveries were minor. This test provided the first demonstration that for design discharges, water depths in the canal were greater than assumed in the original design. After 8-1/2 hours of 6-pump flow of about 4,800 cfs, water stood above the top of the concrete lining in the first 25 miles of canal. The sixth pump was then stopped and 5-pump flow of about 4,000 cfs was continued for 35 hours. The capacity deficiency demonstrated by this test was so disappointing that a rerun was scheduled for March 1957.

Prior to the March 1957 tests, divers inspected each in-line siphon structure and found them to be clear of debris and/or silt. Stoplogs were removed from side bays of check structures and a spot survey was made by probing to determine if appreciable accumulations of silt, clams, moss, or other debris might be present in the canal. As all findings were negative, the six pumps were again started and operated for about 8 hours on March 3, 1957. As water again overtopped the concrete lining, the sixth pump was halted and 5-pump flow of 4,100 cfs continued for 24 hours. Under this condition, water depths stabilized in the upper portion of the canal at points which left approximately 0.5 foot of lining freeboard.

Following this second test in 1957, near-precise levels were run along the berm to check 1946 levels used for construction of the canal; top of lining and invert elevations were established by means of static pools; and head loss measurements were made across in-line siphons.
<table>
<thead>
<tr>
<th>Approximate mile</th>
<th>Curvature index</th>
<th>No. of curves per mile</th>
<th>Curve deflection angle (Δ°)</th>
<th>Curve radius ( R_1 ) (in feet)</th>
<th>Av. ratio ( R_1/T )</th>
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<td>3.3</td>
<td>33</td>
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<td>553</td>
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</table>
47 Timber bridges (three or four piers each)  
34 Concrete bridges (two piers each)  
11 Concrete overchutes (two piers each)  
 5 Irrigation pipe crossings (one pier each)  
14 Oil or gas line crossings (two piers each)  
12 Concrete check structures

This list shows a total of 123 structures with piers in the flow prism in 66.5 miles of canal or an average of one structure with piers every 0.54 mile.

This 66.5-mile reach contains 220 curves in canal alignment. A summary of curve data is given in Table B-1.

Kutter's formula with roughness coefficient "n" of 0.014 was used for the design of the concrete-lined portion of this canal. No extra allowances were provided for head losses across in-line structure piers, horizontal curvature in canal alignment, or for increased flow resistance due to aquatic growths. Equivalent Manning's "n" is 0.0138. (Discharge computed using Manning's formula, "n" = 0.014, and design depth at the head end of the canal is 4,530 cfs, 1.5 percent less than design discharge of 4,600 cfs.) A vertical freeboard allowance of 1.50 feet was provided from design water surface to top of concrete lining.

The table of hydraulic properties on Figure B-2 shows the design discharge for the canal to be 4,600 cfs at the beginning of the canal. The design discharge decreases in a downstream direction and is 3,211 cfs at the end. The entire concrete-lined portion of this canal has a bottom width of 48 feet. Figures B-2, B-3, and B-4 show how flow sections are reduced in size in a downstream direction by rises in invert grade. Due to these "steps" on the invert, canal water surfaces consist of a series of backwater curves. The backwater curves are further influenced by water deliveries through an average of two to three turnouts per mile of canal.

All water for this canal is pumped from the Sacramento River Delta area through the Tracy Pumping Plant. The maximum output of the six large vertical-shaft, centrifugal, single impeller pumps at this plant is approximately 4,700 cfs. Diurnal tidal fluctuations of 3 to 4 feet occur in the intake channel to the Tracy plant and result in a variation of head across the pumps. The varying head, in turn, causes a variation of discharge. For the six pumps, this variation is about 200 cfs, and canal water depth at the end of the discharge lines varies about 0.30 foot. This phenomenon, coupled with irrigation deliveries along the canal, creates a condition of nonuniform, unsteady, and spatially...
Drainage and outlet as directed

Trench filled with gravel as directed

Compacted embankment

Original ground surface

**HYDRAULIC PROPERTIES**

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**NOTE**

Provide transverse grooves at 12'-0" max centers

**LINING DETAIL**

(Unreinforced)

**GROOVE DETAIL**

**SCALE OF FEET.**

**FIGURE B-5**
### Location of Inlet Transition

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<tr>
<td>P.R.R. Culvert</td>
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</tr>
<tr>
<td>W.R.R. Culvert</td>
<td></td>
</tr>
<tr>
<td>Check No. 1 WRR Culvert</td>
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<tr>
<td>Check No. 2 WRR Culvert</td>
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<tr>
<td>Corral Hollow Siphon</td>
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<tr>
<td>Hatche-Hatche Siphon</td>
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<tr>
<td>Puerto Creek Siphon</td>
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<td>Orestima Creek Siphon</td>
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<td>Check No. 11 and San Luis Creek Siphon</td>
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<td>San Luis Creek Siphon</td>
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### Inlet Transition Measurements

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### Adjusted Head Loss

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### Head Loss Measurements Across Siphons

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<td>Figure 8-6</td>
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In 1958, 1959, and 1960, further measurements were made during peak summer delivery periods to check head losses across siphons and various types of piers. The losses across siphons (Figure B-6) were generally less than allowed in design resulting in slight drawdown curves at siphon inlets. The losses across various types of piers were extremely difficult to obtain owing to the flat slope of the water surface and the unsteady flow conditions. The canal has an invert slope of 0.00005 (0.26 foot per mile) and differential water surface elevations were generally under 0.10 foot. Results of pier loss measurements are given in Figure 11.

The measured head losses across piers were found to be about 20 percent less than estimated values based on model test data presented in References 18 and 19. Exceptions occurred where horizontal curves in canal alignment at or immediately upstream from piers appeared to increase these losses. Also, the multiple square post-timber piers appeared to cause almost twice the loss of the solid round-nose piers. Based on the latter evidence, it was estimated that the water surface at the head end of the canal could be lowered about 0.70 foot if multiple post piers could be sheathed and streamlined nose and tail units added to all square post piers in the first 31 miles of canal. The cost of this work was approximately equal to the capitalized cost of the estimated pumping head which could be saved. This work was accomplished in December 1960 and January 1961, while the canal was unwatered for the first time in 8 years. Piers on 44 structures in the reach were modified in a manner similar to that shown in Figure B-7.

Because of the continuous demand for water, this canal is not unwatered unless a definite need exists. The 1960 unwatering to Mile 34.4 was made to determine the location, composition, and extent of possible deposits on the invert; to obtain a centerline survey on the invert; to make maintenance inspections of structures, canal lining, and untreated timber bridge piers; and to make pier modifications. As the water receded in the canal, two unexpected factors were revealed which were believed to have influenced flow resistance in the canal. They were:

1. Sides of the concrete lining were covered in varying degrees with a coating consisting of Bryozoa, amphipods, and fine silt. Above the water, where the side slopes had dried, the coating was crust-like in character and the shrimp-like amphipods were embedded in a mat of fine sediment. This mat was reinforced with fiber-like Bryozoa. Figure B-8 shows the coating in a moist state. Under water the filamental Bryozoa had the appearance of cropped hair. Figure B-9 is a magnification of a laboratory specimen of Bryozoa.
A. Installation of 1-inch timber sheathing and ogival shape nose and tail units on square-post piers of 28-foot wide timber bridge at Mile 5.67. 12-60-25IMC, December 27, 1960.


Figure B-7
Photograph of coating or scum on surface of concrete lining in Delta-Mendota Canal at Mile 3.78. The coating was thickest (about 3/8 inch) at this location near the beginning of the canal. The major component in the coating has been identified as Bryozoa Fredericella sultana, a low type of animal life which attaches itself to the lining. Silt and small shrimp-like aquatic life are intermingled in the growth. The coating presents different appearances when underwater, when moist (as shown), and when dry. P214-D-24653, December 14, 1960.

Figure B-8
Closeup view of a specimen of live Bryozoa Fredericella sultana. (Magnification X5). This growth has been found on the concrete lining of Delta-Mendota Canal, on the trashracks at the Tracy Pumping Plant, and on underwater parts of the fish collecting facility at the beginning of the canal. PX-D-20873.

Figure B-9
2. The invert of the canal was found to be intermittently covered with clam-bearing silt deposits. (Figure B-10 through B-12 are photographs of typical deposits. Figure B-13 shows cumulative grain-size curves from typical samples of deposits.) Equipment was hastily assembled and an estimated 50,000 cubic yards of the silt-clam mixture was removed from the invert before refilling of the canal began. Another estimated 10,000 cubic yards of deposits were scattered within the 31-mile reach and were not removed by mechanical cleaning methods.

To demonstrate the improvement in flow conditions resulting from canal cleaning and pier improvements, more tests were scheduled for July 1961. Data were obtained during a 4-day period of continuous 5-pump discharge of 4,000 cfs and a 4-1/2-day period of continuous 6-pump flow of 4,700 cfs. The 6-pump flows were carried at lower depths than those which existed in the 1957 tests, but water was still reported as being intermittently above the top of the concrete lining above Mile 20. Figures B-14 through B-16 show flow conditions on July 13 and 14, 1961, between Miles 4.0 and 8.0.

Additional experimental test work accomplished in 1961 included the use of multiple current meter equipment from the bridge at Mile 9.87. Eight propeller-type current meters mounted as shown in Figure B-17 were used to obtain comprehensive velocity traverses. Each meter was coupled electrically to a recorder pen which documented the number of revolutions made by the propeller over a given time interval. Traces made by the pens on recorder charts were later "decoded" and plots of velocity distributions prepared. Plots of these isovels revealed noticeable velocity disturbances caused by the bridge piers at a point 2 feet upstream from the bridge. Further studies are underway to attempt to evaluate boundary shear stresses from these comprehensive velocity traverses. Canal discharges were also calculated from these measurements.

In December 1962, and January 1963, the canal was again unwatered, mainly for the purpose of inspection and repair of timber bridges between Miles 34.4 and 70. During this unwatering, it was found that silt-bearing clam deposits were also present in the reach between Mile 34.4 and 70. The deposits were less extensive than those found in the first 31 miles of canal during the 1960 unwatering. Preliminary estimates indicated that only 8,000 cubic yards of silt-clam deposits existed in the first 40 miles of canal. Part of these were possibly left during the 1960-61 unwatering when insufficient time was available to accomplish complete cleaning. Thus, it appears that appreciable amounts of fresh water clam growth did not occur during 1961-62. Figures B-18
Extensive silt-clam bed above overchute at Mile 18.59 in the Delta-Mendota Canal. This deposit extended three-fourths of the way across the invert, was approximately 2-1/2 feet deep on the left side, and was over 800 feet long. Note coating of sediment and aquatic growth on surface of concrete lining at left. P214-D-24656, December 15, 1960.

Figure B-10
View from bridge at Mile 28.67 looking downstream in the Delta-Mendota Canal. This photograph shows the random pattern in which the silt-clam beds lay in a straight reach. There was no consistent pattern in which the beds appeared, although they were found quite often on the inside of horizontal curves. Opening through center of deposits was made by front-end loaders and road graders to facilitate drainage of the canal. P214-D-24658, December 18, 1960.

Figure B-11
Closeup of the surface of the silt-clam deposit on the invert of Delta-Mendota Canal at Mile 14.8. The interior of the deposit differed from the surface appearance in that a greater number of live clams and a larger proportion of silt were evident. Cumulative grain size curves for dried samples from twelve deposits obtained in the first 30 miles of concrete-lined canal and one in the Old River earth channel are shown in Figure B-13. Clay and silt fractions of these deposits made up 55 to 95 percent of the total. P-214-D-24657, December 16, 1960.

Figure B-12
EXPLANATION

- Mostly Mineral Grain
- Mostly Peat Particles
- Mostly Clams

Location of sample, mileage from intake ("Fish Screen" of Old River)

MILE 11.39
MILE 12.40
MILE 14.65
MILE 16.62
MILE 18.90
MILE 20.85
MILE 22.74
MILE 25.04
MILE 26.19
MILE 28.26
MILE 29.98
MILE 31.65
MILE 34.08

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT - CALIF.
DELTA-MENDOTA CANAL
SILTATION STUDY
(DEC. 1960)

CUMULATIVE GRAIN - SIZE CURVES

FIGURE B-13

SACRAMENTO, CALIF. OCT 29, 1962 214-208-4014
Upstream view of 16-foot farm bridge at Mile 4.98 of Delta-Mendota Canal at 11:40 a.m., July 13, 1961, during 6-pump test. This location is customarily used for canal flow measurements. Waves were splashing over canal lining. Approximate canal flow was 4,600 cfs in a section designed for this discharge but in which 18 inches of vertical freeboard to top of concrete lining was anticipated. DM-4716-CV.

Figure B-14
Flow conditions in Delta-Mendota Canal during 6-pump test. Upstream view of partially submerged over chute at Mile 7.25 at 9:00 a.m., July 14, 1961. Approximate canal flow is 4,650 cfs. Design discharge for this location was 4,600 cfs and 18 inches freeboard to top of concrete lining was anticipated. DM-4721-CV.

Figure B-15
Crews are obtaining head-loss measurements below 20-foot timber bridge at Mile 7.29 of the Delta-Mendota Canal during the 6-pump test. Approximate flow was 4,650 cfs in the section which was designed for 4,600 cfs. DM-4726-CV, July 14, 1961.

Figure B-16
A. Multiple current meter equipment used from 40-foot timber bridge at Mile 9.87, Delta-Mendota Canal. Equipment was mounted on a flat bed truck which was moved for each vertical velocity traverse. Instrument shelter is on right of sandbag ballast. Current meter support rod is in a fully raised position. P-416-D-28559, July 16, 1961.

B. Closeup of propeller-type current meters and support frame. Eight meters were mounted on the vertical rod to obtain simultaneous measurements of velocities in a vertical line. Meters were connected to a battery-powered recorder. Comprehensive velocity traverses were obtained for both 5 and 6 pump flows. P-416-D-28550, July 13, 1961.
Deposit on canal invert at Mile 34.8 formed downstream from the drainage inlet shown at the top of the concrete lining. The deposit consisted mainly of silt which had been washed into the canal from the adjoining field as a result of improper irrigation practices. P-D-200-5270, December 19, 1962.

Figure B-18
and B-19 are photographs of deposits on the lining surface taken during the 1962 unwatering.

Figure B-19 shows an area of extended fresh water sponge growth just downstream from Check No. 5 at Mile 29.8. Very little of this type of growth was evident during the 1960 unwatering, but it was reported at several locations on the lining surface and within the siphon barrels in 1962.

Analyses of Test Data

Plots of water surface profiles were drawn for test runs of 1957, 1960, and 1961. Figures B-3 and B-4 show the 1961 6-pump water surface profile to Mile 70. A common characteristic of all 6-pump profiles was a series of \( M_2 \) backwater curves with increasing depth in an upstream direction. Depths were greater than anticipated in the entire 66.5 miles. Manning's "n" values were computed for individual reaches and are shown above the profiles in Figures B-3 and B-4. To compute these resistance coefficients, depths were scaled from the surveyed invert between Miles 3.5 and 34.4 and from the design invert between Miles 34.4 and 70.0. Since the water surface in the canal fluctuated as much as 0.30 foot during daylight hours, depths for computations were scaled to the nearest tenth foot. Invert survey data downstream of Mile 34.4, obtained during the unwatered period of December 1962-January 1963, was not plotted. Examination of survey notes indicated the invert to be generally above design grade for the entire length between Miles 34.4 and 70. The invert location varied from about 0.1 foot above design grade at Mile 34.4 to about 0.4 foot above between Miles 58.3 and 70. Consideration of the surveyed invert location would result in a slight reduction in depths used for computations and about a 4 percent reduction in the "n" values shown at the top of Figure B-4.

Discharge for a given reach was taken from a "summary of diversions" submitted with field test data. Current meter discharge measurements were made at 10 locations along the canal in 1961. These gaging checked "summary of diversions" flows within 3 percent.

Figure B-3 shows Manning's "n" values (which include all pier and curvature losses) in the first 31 miles of canal to range from 0.0159 to 0.0178. The weighted average value for reaches shown was 0.0166. These values were obtained in the reach from which the bulk of the silt-clam deposits had been cleaned and in which piers had been modified. Figure B-4 shows that "n" values
View of extensive fresh water sponge growth on side of concrete lining. This growth occurred near Check No. 5 at Mile 29.8 and is similar to that found on the Main Canal of the Columbia Basin Project (Figure A-11). A considerable increase in the amount of sponge growth was reported in the Delta-Mendota Canal between December 1960 and December 1962. P-D-200-5276, December 20, 1962.

Figure B-19
from the following 35 miles of canal ranged from 0.0162 to 0.0191 and averaged 0.0173. These values were expected to be higher since this reach had not been unwatered and cleaned, nor had any piers been modified. The 1961 "n" values obtained between Miles 34.4 and 70 did not differ appreciably from those of previous years. Values obtained from 5-pump tests were close to those obtained from 6-pump tests.

An attempt was made to isolate the improvement in flow conditions resulting from pier improvements only in the first 31 miles of canal. This task proved to be most difficult because the effect of canal cleaning was also included in test data. Figure B-20 shows "before and after" water surface profiles for 5-pump flow between Miles 3.5 and 34.4. Water depth in the canal was lowered by a maximum 0.80 foot after both cleaning and pier improvement and weighted average "n" for reaches shown was reduced from 0.0174 in 1960 to 0.0166 in 1961. Apparently, the anticipated depth reduction due to pier improvement alone was not as great as anticipated.

Backwater curve computations were made to fit observed water surface profiles in the first 31 miles of canal. Allowances for pier losses were made based on prototype measurements and the following results were revealed:

1. Flow resistance of the concrete-lined channel only was best described by a Manning's "n" of 0.0155.

2. Overall flow resistance of canal and piers was 0.0165.

3. Total design head loss for 31 miles of canal was 10.8 feet. Measured head loss was 10.9 feet for flows within 7 percent of design values. The important difference was that these water depths were 0.5 to 1.5 feet greater than anticipated.

4. Measured head losses across in-line closed conduit siphons varied from 40 to 115 percent and averaged about 70 percent of design allowances. This resulted in drawdown curves at nearly all siphon inlets for all tests. Siphon head loss measurements are summarized in Figure B-6.

Tests numbered 29 through 54 shown in the Prototype Test Data Sheets (Figures 14 and 15) were taken from typical reaches in this canal. No straight reach longer than 5,000 feet without structure piers could be found in the first 70 miles of canal. Therefore, in Figure 14 pier losses have been deducted as explained in the "Remarks" column. Results in terms of Manning's "n" generally show that straight clean reaches of the canal had values of
DELTA-MENDOTA CANAL
WATER SURFACE PROFILES--MILES 3.5 TO 34.4
5-PUMP FLOW TESTS--JULY 1960 AND JULY 1961
"BEFORE AND AFTER" PIER IMPROVEMENTS AND CLEANING

FIGURE 8-20
0.015 to 0.016. Sinuous reach "n" values ranged from 0.016 to 0.018. Piers of the type and frequency present in these reaches had the effect of increasing "n" values about 6 percent. No chemical treatments of any kind had been used in this canal prior to or during tests to remove or retard aquatic growths; therefore, the data presented here reflect flow resistance in an untreated channel.

Water in the canal is generally turbid to the extent that it is possible to see only about 1 foot below the surface. Sediment samples were obtained at Mile 3.53 over a period of 1 year beginning in October 1959. Suspended sediment concentrations were generally less than 100 ppm, but a maximum of 155 ppm was reached for a flow of 3,600 cfs on June 9, 1960. Concentrations of these magnitudes should have little effect on flow resistance.

The invert shown in Figure B-3 is drawn through plots of more than 540 surveyed points obtained on the canal centerline during the December 1960 unwatered period. A tabulation of these points showed the average variation from design grade was 0.12 foot and the maximum 0.53 foot. Present specifications allow a "departure from established profile grade of 1 inch." The plotted invert indicates that adverse slopes apparently existed in some reaches. This condition results from the difficulty of maintaining the extremely flat design slope of 0.00005 (0.26 foot per mile) during construction.

Owing to increased demands for water in the highly developed agricultural areas along this canal, consideration has been given to raising the concrete lining and appurtenant structures to increase the capacity of the canal.

Summary and Discussion of Test Results

Water depths in the first 70 miles of the canal during 1957 tests were greater than expected due to three factors which were not fully anticipated in the original design. These factors were:

1. A considerable number of silt-clam deposits on the invert coupled with Bryozoa growth on the sides of the concrete lining. (These aquatic growths occurred in sufficient magnitude to cover a large proportion of the lining and created a change in surface roughness which in turn changed turbulence characteristics and resulted in increased flow resistance.)
2. Backwater effects of some 120 sets of structure piers and 220 horizontal curves in canal alignment. (Head losses caused by piers and curves were apparently a greater proportion of the total head loss in this large, smooth boundary, flat-slope canal than was formerly realized.)

3. Apparent inadequacy of Kutter's formula with an "n" value of 0.014 to predict the flow resistance in a channel of this size, shape, and surface roughness. (Evidence now indicates that "n" for use in Manning's formula should be increased above 0.014 for large concrete-lined channels with hydraulic radii above 5.0.)

Water depths were measured in the first 31 miles of canal in 1961 after the reach was cleaned and 44 sets of piers were streamlined. These depths were as much as 0.8 foot lower than those measured in 1960 for the same discharge but still were not as low as expected in the original design. Expressed in terms of roughness coefficient "n", used with Manning's formula, 1961 tests revealed "n" to be approximately 0.0165 as compared to 0.0175 in 1960. These values include all pier and curve losses. When estimated pier losses were deducted, these values were about 6 percent less. The minimum "n" value for a straight reach without piers was found to be about 0.015. Generally, flow resistance in the more sinuous reaches was higher than in straight reaches. Changes in output of the Tracy Pumping Plant, amounting to about 5 percent of test flows, were caused by tidal water surface fluctuations in the intake channel to this plant. The variable discharge, in turn, caused daily fluctuations of canal water depth at Mile 3.5 to 0.4 foot. This phenomenon coupled with water deliveries along the canal, created flow conditions which were difficult to analyze. Steady flow conditions were assumed to process test data in the time available. Therefore, more detailed analyses might reduce the spread of resistance coefficients between individual reaches. However, since several repeat measurements were made in 70 miles of canal, the overall performance has been well substantiated.

Head losses across piers were extremely difficult to measure in this flat slope canal. Measurements are summarized in Figure 11 of this Technical Memorandum. Pier losses were very small numerically (usually less than 0.05 foot), but they amounted to a significant portion of the overall head loss in this canal, the invert of which drops at the rate of only 0.264 foot per mile. The effect of these small eddy losses on water depth depended on the size, configuration, and spacing of the piers along the canal. Backwater computations were necessary to demonstrate this effect since losses were not always additive.
No chemical treatments have been used to retard the types of aquatic growths peculiar to this canal. Aromatic solvents were used to control the growth of Asiatic clams in the cooling condenser tubes at the Tracy Pumping Plant where the volume of water involved was relatively small. Treatments for Bryozoa growths have not been evolved at the time of publication of this Memorandum. As this growth does not appreciably increase flow resistance, little experimental work has been attempted. Observations of moist Bryozoa growths above the water surface gave the impression that tentacles or filaments of the growth did not extend into the flow prism more than 1 inch. Therefore, most of the flow occurred a considerable distance from the boundary and probably was affected little by this slight change in boundary surface texture. However, little experimental evidence is presently available on the effect of thin layers of aquatic growths on turbulence characteristics or friction coefficients in large channels.

The unexpectedly extensive silt-clam deposits, revealed during the 1960 unwatering, demonstrated that considerable amounts of silt entered the canal. Some of this silt entered through drainage inlets and some apparently settled to the invert from the slow velocity turbid water. The sediment deposits, together with the relatively warm water from the Sacramento River delta area, evidently provided an ideal environment for Asiatic clam propagation. These deposits had definite retarding effects on flow and created additional cleaning expense. Removal of the deposits and pier modifications resulted in decreased flow resistance and demonstrated that periodic cleaning of this slow-velocity, concrete-lined canal was an important element in improving capacity.

Figures 14 and 15 provide detailed information concerning prototype flow tests made during 1957, 1959, 1960, and 1961 in selected reaches of this canal. These figures also show computed values of hydraulic properties and design parameters.
Section C

MAIN CANAL

(Trail Lake Reach)

Columbia Basin Project, Washington
The Trail Lake Reach of the Main Canal was chosen as one of the locations for prototype tests to provide performance data for utilization in the establishment of design criteria for large concrete-lined canals. This reach is an example of a relatively high-velocity steep-slope canal. (Design velocity is 12.21 fps; invert slope is 0.00061.) It is one of the few large Bureau of Reclamation canals which is operated in an unchecked condition over a large range of flows. As the operating season for this canal extends from March through October, it was possible to survey channel geometry during the unwatered period.

Eleven free-flow water surface profiles were documented in a 5,570-foot reach of this concrete-lined canal during the 1961 irrigation season. The profiles were recorded at approximately 1,000-cfs increments of flow, beginning at 1,090 cfs on March 30, increasing to 6,820 cfs on June 26, and decreasing to 1,040 cfs on October 19. This canal was designed for a capacity flow of 13,200 cfs and is a key link in the conveyance system between Franklin D. Roosevelt Lake (the reservoir behind Grand Coulee Dam) and the irrigated area of the Columbia Basin Project as shown in Figure A-1. Tests at the design capacity were desired, but this flow could not be obtained because of upstream structure limitations.

The 6,200 feet of concrete lining for this canal was placed in 1951 under Specifications No. 3190. A description of construction activities can be found in the "Long Lake Dam and Main Canal, Technical Record of Design and Construction," published by the USBR in 1955. The 5-inch-thick sides of the lining were placed one side at a time by a rail-mounted lining machine. The 6-inch-thick invert was then placed using a bar vibrator and a strikeoff plate float. The invert was hand-finished with wooden floats and long-handled steel "fresno" floats as shown in Figure C-1. Transverse contraction grooves for this unreinforced concrete lining were spaced at 25-foot centers and filled with asphalt mastic.

Figure C-2 shows canal profile and section details. Kutter's formula having a roughness coefficient "n" of 0.014 was used for the design of this canal. The value of 0.014 was considered adequate to provide for possible additional head losses caused by horizontal curves in alinement. Equivalent Manning's "n" = 0.0152. (Discharge computed using "n" = 0.014, Manning's formula, and design depth is 14,310 cfs, eight percent greater than design discharge.) A vertical freeboard allowance of 2.00
View of concrete lining operations on invert of the Trail Lake Reach of the Main Canal. Sides were placed by traveling rail-mounted slip-forms. A vibrating screed was used for initial invert surface finish. Fine finishing was done with steel trowels. CB-30588, August 7, 1951.

Figure C-1
Survey and range of depths of which neg data were obtained in 1960.

Location of design invert (field = 1960).

Survey and range of depth of which test data were obtained in 1960.
feet was provided from design water surface to top of concrete lining.

Figures C-5 and C-6 show the sinuous alinement of the test reach and the locations of the water surface profile measuring stations which were designated alphabetically, A through E. These five stations were established in the 5,570-foot reach to document the expected backwater curves. Also, it was hoped that the effect of horizontal curvature on hydraulic resistance might be established by breaking the reach into four parts. The reach contained 10 horizontal curves in alinement and was one of the more sinuous canals described in this Appendix. Members of the water surface measuring crews reported that water noticeably "piled up" on the outside of some curves. A summary of curve details is given in Table C-1.

Description of Tests

Prior to the beginning of the 1961 operating season, the test reach was cleaned of all rocks and debris and the lining surface was reported to be free of all live aquatic growths. However, the photographs in Figure C-8, taken in January 1961, show that a dried coating was present on the lining at various locations in varying amounts. It is probable that some of this coating peeled or flaked off before the canal was filled in the spring. As water used during the tests was not chemically treated to retard organic growth, the tests reflect any effect on hydraulic friction which resulted from these growths.

When the lining surfaces of the test reach were examined by project personnel in September 1961, the surfaces were found to be covered with a dense layer of Nostoc (an algae) averaging 1/8 inch in thickness. The Nostoc layer consisted of firm, very smooth gel-like colonies which had slight tubercular projections. Figure A-10 shows a closeup of Nostoc in its wintering stage. A smaller amount of filamentous algae, Cladophora, was also found attached to the lining surface. Project operating personnel had not previously documented the effects of these algae on hydraulic friction but they were convinced that the resistance changes during the operating season. Their opinions on the Trail Lake lining growths state that possibly the Nostoc alone would provide a smoother surface than the original concrete, but that the Cladophora would be likely to cause an increase in flow resistance because of the filaments which extend 6 to 8 inches into the flow.

Steady flows for the tests were established by holding constant releases from the Equalizing Reservoir upstream from the
Table C-1

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Average values

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T = Top width of design water surface = 83 feet.
Aerial view looking downstream at the concrete-lined Trail Lake Reach of the Main Canal. Measuring Point A was just downstream from the end of the first curve shown in the foreground. P222-116-42644, July 15, 1960.

Figure C-5
View looking downstream at Point E of test reach (Trail Lake Reach of Main Canal); flow was approximately 6,800 cfs. The water surface was reported to be quite rough. P222-116-43535, June 29, 1961.

Figure C-7
View of unwatered canal between Points C and D. Note dried coating on sides caused by summer aquatic growths. Pools of water on the invert are evidence of slight undulations of this surface. P222-116-42874, January 16, 1961.

Figure C-8
test reach. Steady flows were verified by charts from a water stage recorder located on the bridge downstream from Point E. The water was reported to be relatively clear and free of sediment.

Discharge measurements were made approximately 4 miles upstream from Point A at the established gaging station for the Main Canal at Mile 0.2, Station 67+00. A Price Type A current meter was used and discharge was computed using the 0.2, 0.8 depth procedure. Discharges for data analyses were taken from the rating curve for the Mile 0.2 gaging station at the recommendation of project hydrography personnel. The rating curve was prepared from some 140 current meter gagings at this station and checked 1961 test gagings within 4 percent. Average variation between rating curve and current meter gagings for the 11 test runs was 1.5 percent. No measurable inflow or outflow occurred between the gaging station at Mile 0.2 and Point A of the test reach. Water losses due to seepage and evaporation were assumed to be 1 cfs per mile.

Survey data were obtained on the unwatered canal prior to the tests based on assumed datum of 100.000 for a bench mark located near Point A. This datum was tied to design levels as shown on Figure C-3. The design invert elevation was computed for each of the 17 points at which a surveyed elevation was obtained. The assumed datum survey elevation was then subtracted from the design elevation, and an average difference of 141.17 feet was computed. The average difference was also added to each surveyed elevation to obtain the as-built invert elevations. The as-built elevations were plotted as crosses on Figure C-3. Based on this analysis, 9 of the 17 points obtained on the centerline invert survey were above design grade and 8 were below. Average deviation from design grade was 0.05 foot and maximum deviation was 0.12 foot.

Actual concrete lining surface irregularities on the invert were measured at selected locations within the test reach from lines parallel to and normal to the canal centerline. Measurements on lines parallel to the centerline were made from a taut nylon string 50 feet long. Offsets from the string were measured at approximate 4-foot intervals. The average deviation (of 60 measurements) from a plane surface was 0.032 foot. Measurements were made in a similar manner from strings stretched normal to the canal centerline on the invert. The average deviation of 28 measurements from a plane surface along these lines was 0.018 foot. The maximum deviation indicated in all measurements on the invert was 0.13 foot. Specifications tolerances allowed surface irregularities of 1/4-inch (0.02 foot) measured from a 10-foot straightedge on the invert.
On the sides of the concrete lining, measurements of surface irregularities were made from taut nylon lines anchored at the top and bottom of the slope. The average deviation (of 112 measurements) from a plane surface on lines normal to the canal centerline was 0.033 foot. Offsets were measured in a similar manner along lines parallel to the canal centerline. Average deviation (of 121 measurements) from a plane surface along these lines was 0.019 foot. Specifications tolerances allowed surface irregularities of 1/2 inch measured from a 10-foot straightedge on the sides of the lining.

Plotted cross sections for the five water surface measuring stations were found to be within 1% percent of design values over the range of depths tested. Only one localized lining bulge (about 6 inches) was reported in the test reach. This was on the right side of the lining between Stations 206+01 and 206+85. When the canal was inspected after the tests it was found that about 5 cubic yards of earth and rock material had sloughed into the canal near Station 309+00 sometime during the irrigation season.

Elevations used for these tests are shown in the Prototype Test Data Summary Sheets (Figures 16 and 17) and were based on the assumed elevation of 100.000 near bench mark "A." Relative elevations at other measuring stations were determined by two level circuits and checked by a static pool test. Closure errors of the two level circuits were 0.016 and 0.017 foot, respectively. Average adjusted levels were used for data analyses.

Analyses of Test Data

Water losses due to seepage and evaporation were assumed to be approximately 8 cfs (1.5 cfs per mile) between the gaging station and the end of the test reach. This amount was less than 1 percent of the lowest test flow.

Figures C-3 and C-4 show plots of the 11 water surface profiles. It will be noted that distinct ME-type backwater curves existed for all but the highest flows. The 6,820-cfs run of June 26 indicated relatively uniform depths which changed only 0.34 foot (from 15.72 feet to 16.06 feet) in the test reach. This is very close to a uniform flow, constant depth condition.

Water surface profiles obtained at similar discharges during different times of the year are plotted together in Figure C-4. This drawing shows graphically the seasonal change in resistance which occurred in the 5,570-foot test reach. For
This view of unwatered canal downstream of Point C shows coating on invert and sides from summer aquatic growth. Canal was not chemically treated to retard these growths during the 1961 tests. P222-116-42871, January 16, 1961.

Figure C-9
example, "n" was 0.0148 for a flow of 3,230 cfs on April 27, 1961, and 0.0195 for a flow of 3,190 cfs on September 6, 1961. The control water surface at the end of the reach was the same (within 0.10 foot) for both runs, but the water depth at the beginning of the reach on September 6 was 1.02 feet greater. A dense growth of Nostoc and some Cladophora algae was noted on the lining surface in September and apparently was the main reason for the increased resistance. Figure C-10 shows graphically the seasonal change of Manning's "n" values which occurred.

Figures 16 and 17 contain summaries of computed test parameters. The computed roughness coefficients for Reaches BC and CD on April 20 were considerably lower and higher, respectively, than values from adjacent reaches for some undiscovered reason. Therefore, these reaches were combined on this date in the figures.

Summary and Discussion of Test Results

Hydraulic flow resistance was computed for the 11 test runs made during 1961 in this 5,570-foot reach of high velocity canal. Values of roughness coefficient "n" for use in Manning's formula were found to be higher than anticipated and flow resistance varied seasonally.

Figure C-10 shows the surprisingly large seasonal variation in roughness coefficient "n." A low of 0.0143 was computed for Reach DE on May 22 and a high of 0.0203 was computed for Reach CD on September 29. This amounted to almost a 40 percent change in "n" in a 5-month period. Apparently algal growths on the concrete lining surface were the major factors which caused the increased resistance. No chemical treatments were made to retard these growths in the test reach. Therefore, a vivid demonstration of the effect of aquatic growths on flow resistance was documented in this canal.

Figure C-10 shows the spread of computed resistance coefficients that existed in the incremental reaches between Points A and E. Reach DE was most sinuous but did not always indicate an appreciably higher "n" value. Reach AB was least sinuous but did not always indicate the lowest "n" value. The average "n" values for all 11 tests in each Reach were:

Reach AB--0.0166
Reach BC--0.0165
Reach CD--0.0170
Reach DE--0.0165

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Note: This reach of canal was not chemically treated to retard aquatic growths.
Figures 16 and 17 provide more detailed information concerning prototype tests made during 1961 in this canal. These figures also show computed values of hydraulic properties and design parameters.
Section D

EAST LOW CANAL

Columbia Basin Project, Washington
EAST LOW CANAL
Columbia Basin Project, Washington

A series of 14 water surface profiles was documented and other hydraulic performance data were obtained from the East Low Canal during the 1960 irrigation season. The information was obtained between March and October at flows which varied from 875 to 4,470 cfs. (The first 26 miles of this canal were designed for 4,500 cfs.) Test data were obtained for the purposes of determining the hydraulic resistance in concrete-lined sections and to enable operating personnel to demonstrate the carrying capacity of the upper portion of the canal.

Comprehensive measurements were concentrated in two concrete-lined reaches located in the first 6 miles of the canal. An 8,059-foot sinuous length, called Reach 1, and a 6,255-foot straight length, Reach 2, were selected as typical locations for tests. Additional measurements between Miles 6 and 36 were obtained on September 19 and 20 when near design flows occurred. Figures D-1 through D-5 show test reach alinement and profile and section details.

The 4-1/2-inch-thick concrete lining in the first 27 miles of this canal was described in USBR Specifications No. 1422 and 2603. Lining of the first 11 miles of canal was completed in 1949; the lining of the remaining 16 miles was completed in 1951. The first 11 miles of lining was placed from a large rail-mounted slip-form which paved the entire section in one pass. More than half of the 27 miles of canal was unreinforced. Specifications requirements for finishing the lining in 1946 stated, "Concrete canal lining shall be finished so as to eliminate irregularities and produce a finish equivalent to that obtainable by the use of a wood float followed by one pass of a steel fresno." No dimensional tolerances for finishing the concrete were set forth in 1946.

In 1949, a general revision was made in specifications paragraphs relating to finishing the surfaces of concrete canal lining. Subsequent to this time the requirement stated: "The finished surface shall be equivalent, in evenness, smoothness, and freedom from rock pockets and surface voids, to that obtainable by the effective use of a long handled steel trowel. Light surface pitting and light trowel marks will not be considered objectionable. Where the surface provided by a lining machine meets the specified requirements, no further finishing will be required. Surface irregularities (gradual) will be tested
Aerial view looking upstream toward sinuous Test Reach 1 of the East Low Canal. Point D of this reach was located 360 feet upstream from the county road bridge (Station 165+75.7) shown in the foreground. P222-116-42654, July 15, 1960.

Figure D-2
Aerial view looking upstream at straight Test Reach 2 of the East Low Canal. Point G of this reach was approximately 200 feet upstream from the P.C. of the curve shown in the foreground. P222-116-42657, July 15, 1960.

Figure D-3
Notes:
Check gates were open and out of water at Broken Rock Dam, 6.05 County Road Bridge Mile 6.05.
Check gates at seat to Rocky Slide Dam (Mile 27.5) and Weber Dam (Mile 36.9) were in the water under manual control. Automatic controls were switched to operate the gates at both the Rocky Slide and Weber Weirways, terraces to maintain transom-water levels on the right of a drawdown curve. Gates at the County Road Bridge at Mile 6.05 were mod in the concrete lined section below this note periodically during the 1960 operating season. Water surface profiles defined by permanent points were taken from the top of structures or concrete lining. As-built elevations assumed for measuring points. Values of head loss listed for each siphon are allowances provided in original design. Head losses during tests may be scaled from water surface profiles.

EAST LOW CANAL—COLUMBIA BASIN PROJECT
WATER SURFACE PROFILES—MILES 1.6 TO 36.9
SEPTEMBER 1960 TEST MEASUREMENTS

FIGURE D-4
by the use of a template consisting of a straightedge *** which will be 10 feet long for unformed surfaces *** shall not exceed 1/4 inch on bottom slabs and 1/2 inch on side slopes."

The following dimensional tolerances were also provided:

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<tr>
<td>Departure from established profile grade</td>
<td>1 inch</td>
</tr>
<tr>
<td>Variation from specified width of section</td>
<td>1/4 of 1 percent</td>
</tr>
<tr>
<td>Variation from established height of lining</td>
<td>1/2 of 1 percent</td>
</tr>
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Transverse contraction grooves in the lining were generally spaced at 12- to 15-foot centers in combination with seven equally spaced longitudinal grooves. However, between Miles 1 and 4 various combinations of groove spacings were used to provide experimental data on prototype lining performance. One 550-foot reach was placed in which all transverse grooves were omitted and only two longitudinal grooves were provided. Contraction grooves were specified to be at least 1/4 inch wide and 1 inch deep and were filled with an asphalt mastic compound. Photographs indicated a fairly level surface across these grooves below the waterline. However, the mastic filler generally protrudes above the lining surface on the sides of the canal above the high waterline.

Test Reaches 1 and 2 contained no in-line canal structures. Flow depths in these reaches are controlled by gates at Broken Rock Siphon No. 1 and check which is located downstream at Mile 10.0. Six test runs were made with these check gates open (M2-type backwater curves) and eight runs were made with gates in the water (M1-type backwater curves). Head loss measurements were made for the 20-foot timber bridge (Figure D-6) at Station 319+83, below Test Reach 2.

Head loss measurements were made across four 19-foot 4-inch-diameter concrete in-line siphons on September 19 and 20, 1960. Water surface profiles, showing results of these measurements are provided in Figure D-4.

Horizontal alinement curves in Reach 1 are described in Table D-1.
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</tr>
<tr>
<td>D</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average values</td>
<td></td>
<td>40</td>
<td>708</td>
<td>9.2</td>
<td>0.15</td>
</tr>
</tbody>
</table>

\[ \Sigma \Delta^\circ = 241^\circ \]

An average of 3.9 curves per mile
No curves or bridges existed in Reach 2.

Kutter's formula with a roughness coefficient "n" of 0.014 was used for the original design of this canal. Equivalent Manning's "n" for the canal section is 0.0144. Discharge computed using Manning's formula, "n" = 0.0140, and design depth of 18.92 feet is 4,634 cfs, 3 percent greater than design discharge. A vertical freeboard allowance of 2.68 feet was provided from the design water surface to the top of the concrete lining.

Description of Tests

As this canal is not operated during three winter months, it was possible to secure survey information and photographs in the unwatered section. Photographs shown in Figures D-7 were taken in March prior to tests and those for D-8 and D-9 in November after the tests.

Surveyed invert profiles were plotted in Figure D-5. In Reach 1, 6 of 20 surveyed points were above design grade, 3 were on grade, and 11 were below. Average deviation from grade was 0.05 foot and maximum deviation was 0.09 foot. In Reach 2, 9 of 15 points were above grade, 2 were on grade, and 4 were below. Average deviation was 0.06 foot and maximum was 0.15 foot.

Actual concrete lining surface irregularities on the invert were measured at selected locations within the test reaches from lines parallel to and normal to the canal centerline. Measurements on lines parallel to the centerline were made from a taut nylon string 36 feet long. Offsets from the string were measured at approximate 3-foot intervals. The average deviation (of 16 measurements) from a plane surface was 0.02 foot. Measurements were also made from strings stretched normal to the canal centerline across the width of the invert. The average deviation (of 44 measurements) from a plane surface was 0.01 foot.

On the sides of the concrete lining, measurements of surface irregularities were also made from 36-foot long taut nylon lines stretched parallel to the canal centerline. Average deviation (of 31 measurements) from a plane surface was 0.02 foot.

Cross sections at the seven water surface measuring points designated alphabetically A through G were plotted from survey information and found to be within ±1 percent of the design areas at test flows. Points B and F have apparent local bulges in the lining which reduce cross sectional area by about 2 square feet.
A. View of unwatered canal taken before beginning of capacity tests. Personnel are shown cleaning rocks and other debris from canal in Reach 1. Eleven-year old concrete surface was in excellent condition. P222-116-42186, March 17, 1960.

B. Photograph of unwatered Reach 2 prior to 1960 capacity tests. Concrete lining surface had few irregularities but evidence of fresh water sponge growth is shown. Mastic contraction groove filler was generally depressed 1/8 inch below concrete surface in the flow prism. P222-116-42187, March 17, 1960.

Figure D-7
Closeup of intake piping and stilling well pier at Station 37+84.2 on the East Low Canal. Evidence of filamentous algal growth is shown on pier and on sides and invert of concrete canal lining. The canal was not chemically treated at this location to retard aquatic growths. P222-116-42844, December 16, 1960.

Figure D-8
A. Unwatered Reach 1 after 1960 capacity tests. Fresh water sponge growth is evident and appears to attach itself to edges of contraction grooves at many locations. P222-116-42818, November 28, 1960.


Figure D-9
As photographs of the unwatered canal indicate that such bulges apparently occur very rarely, their effect on flow resistance should be minor. Project photographers and surveyors did not record any unusual humps or distressed areas in the lining during the invert survey.

Representative samples of the unreinforced concrete lining approximately 1-foot square were cut from the side slopes and invert at Station 121+00± and 271+00±. Plans were made to use these samples in laboratory flow resistance tests to evaluate resistance coefficients.

Discharge measurement for test flows was obtained from a temporary cableway at Point F, Station 272+43, in the straight reach. The frontispiece of this Technical Memorandum shows crews making measurements for the 1,980-cfs test of March 30. Comprehensive current meter traverses were made at this location with Price Type A current meters to determine velocity profiles. As long as 9 hours were required to make a traverse in this wide canal section at high flows. Data from the velocity profiles were used by the Hydraulics Branch in analyses to determine boundary shear distributions around the perimeter of the flow prism. Results of this study will be incorporated in a separate report.

An electronic computer program was written to establish discharge from the comprehensive current meter traverses. This program integrated discharge from an approximate 2-foot-square grid pattern over the entire canal section. Discharges obtained from the computer study were used for analyses of data. Deviations from discharges computed using the 0.2, 0.8 depth method were less than 2 percent for all runs except those on March 28 and April 18. Discharges from the computer analysis were 3.0 and 5.3 percent less, respectively, on these two dates.

Water samples to determine sediment concentrations were also obtained at Point F during the tests through May 9, 1960. These samples were shipped to the Denver Laboratories and sediment concentrations were found to be very small—from 0.2 to 0.6 ppm by weight. Because of these minute concentrations, the project was notified that additional sampling would not be required unless system operations changed in a manner which might introduce appreciable sediment.

Continuous recordings of water stages in the canal for the entire 1960 irrigation season were obtained at two locations. Prints of charts from a recorder in a permanent stilling well at Station 37+84.2 and a recorder in an unused pump turnout at Station 162+26.8 were used in data analyses.
No water was turned out of Reaches 1 and 2 and no local inflows were recorded.

Water surface gages (Figure 4) developed by the Hydraulics Branch were used to obtain water surface elevations in Reaches 1 and 2. The frontispiece shows crews with "diving board" platforms making test measurements. At least five readings of water surface elevation were obtained on each side of the canal at each measuring point during each of these 14 tests.

Two water surface profiles for the next 31 miles of canal downstream from Reach 2 were established by means of freeboard measurements from the top of the concrete lining on September 19 and 20. The accuracy of the "freeboard" type of measurement is not as precise as the "water surface gage" method but it required less time and enabled an evaluation of a longer reach of canal at flows very close to design capacity. Records of water deliveries from this longer reach were submitted for the 2 days.

Seepage and evaporation losses from this canal are estimated by operating personnel to be approximately 1.5 cfs per mile at design discharge. Current meter gagings were made in Reach 2. As these discharge measurements were made concurrently with water surface measurements in this reach, no correction was made for losses therein. As Reach 1 is located 3 miles upstream of the current meter gaging station, 5 cfs was added to computed discharges for data analyses in this reach. (This correction amounted to less than 0.6 percent of the lowest test flow.)

Project personnel reported that there was little or no live-aquatic growth on the concrete lining prior to filling the canal in March. However, Figure D-7 shows distinct evidence of the remains of fresh water sponge growth (tentatively spongilla Fragilis). After further investigation it was found that this growth survives in a dormant state through the unwatered period during the winter.

As water in Reaches 1 and 2 was not chemically treated to retard aquatic growths, the tests reflect all effects of such growth on hydraulic resistance. Figures D-8 and D-9 show evidence of sponge and filamentous algae on the invert in the fall after the tests. In addition to these growths, it is probable that Nostoc (an algae) was present since it grows in the concrete-lined Main Canal near the East Low Canal turnout.
Analyses of Test Data

Figure D-4 shows plots of water surface profiles measured in the first 37 miles of the canal in September 1960. Water surfaces for these profiles were measured from top of concrete lining or from canal structures using as-built elevations. No survey work to tie in the elevation of these water surface measuring points to the elevations used in Reaches 1 and 2 was accomplished below Mile 7. Therefore, profile lines for top of earth bank, top of concrete lining, and canal invert were drawn in design locations. This profile shows near-design flow performance of the canal. The two most obvious facts which are apparent from this figure are:

1. Distinct M2-type backwater curves existed upstream from in-line inverted siphons. This condition indicated that head losses through siphons were less than those assumed in the original design.

2. Depth of flow in the earth section was considerably greater than that assumed in the original design.

These two tests demonstrated that the upper end of the concrete-lined portion of this canal (Miles 0 to 23) would carry design flow at depths equal to or less than that assumed for the original design. This was possible only because several relatively long siphons were present, each of which had actual head losses that were less than those allowed in the original design.

Water in the earth canal section flowed above design depth, although discharge was only 80 percent of the design value. This backed up water below Rocky Coulee Siphon so that freeboard to top of concrete lining was 70 percent of the amount allowed in the original design. Apparent Manning's "n" values in the earth section were 0.028 to 0.046 compared to the 0.0225 (Kutter's) or 0.0213 (Manning's) assumed for the original design.

Head loss measurements made across the 19-foot 4-inch-diameter in-line siphons are summarized in Table D-2.

Where check structures occurred at the inlets to siphons, a single set of head loss measurements was made across both the siphon and the check. During the tests of September 19 and 20, 1960, the check gates at the entrance to Broken Rock Siphon No. 1 and Black Rock Siphon No. 2 (Figure D-4) were open and clear of the water surface while the gates at the entrance to Rocky Coulee Siphon were partially closed to control the upstream water surface. Head losses at the three siphons listed above, where the water
A. View looking upstream towards county road bridge at Station 319+13, during 4,450 cfs flow. The high velocity portion of the flow is apparent in the center of the canal with relatively slow velocities on either side. Water depth at this point is less than the design value because of the drawdown at the entrances to Broken Rock Siphons No. 1 and 2. P222-116-42550, September 19, 1960.

B. Inlet to Broken Rock Siphon No. 2; flow is 4,430 cfs. Water surface drawdown occurred because actual head losses in the siphon were less than the amount allowed in design. P222-116-42552, September 19, 1960.

Figure D-10
<table>
<thead>
<tr>
<th>Structure</th>
<th>(1)</th>
<th>(2)</th>
<th>(1) / (2)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Broken Rock Siphon No. 1 and Check</td>
<td>1.93 feet</td>
<td>2.96 feet</td>
<td>0.65</td>
</tr>
<tr>
<td>Broken Rock Siphon No. 2</td>
<td>2.72 feet</td>
<td>4.56 feet</td>
<td>0.60</td>
</tr>
<tr>
<td>Black Rock Check and Siphon</td>
<td>3.58 feet</td>
<td>5.36 feet</td>
<td>0.67</td>
</tr>
</tbody>
</table>
Surface was uncontrolled, were about 60 to 70 percent of the design allowance for these structures. A head loss comparison was not made for Rocky Coulee Siphon where the check gates were in the water.

Head loss measurements across the 20-foot timber bridge at Station 319+83 are summarized in Figure 11. This bridge is supported by two bents, each bent consisting of two 18-inch square posts, as shown in Figures D-6 and D-12. The maximum measured rise in canal water surface between a point 100-feet upstream and 200 feet downstream from the bridge was 0.12 foot and the minimum was 0.03 foot. The former amount was measured during the 4,410-cfs test of September 20, 1960, and the latter during the 2,220-cfs test of August 22, 1960. The head loss due to normal canal boundary shear only was computed for the 300-foot reach, based on measured friction coefficients. This value was subtracted from the total measured rise in water surface leaving the head loss, \( h_p \), attributable to piers only. Values of \( h_p \) varied from 0.02 to 0.07 foot. Values of \( K_p \) computed for use in the equation \( h_p = K_p \cdot h_v \) varied from 0.13 to 0.20. The average value of \( K_p \) for four tests was 0.16.

Figure D-11 shows 1,300-cfs flow conditions in concrete-lined portions of the Weber Wasteway on September 19, 1960. This 12-foot bottom width wasteway was designed for 2,000 cfs on relatively steep invert slopes which varied from 0.004 to 0.03. Considerable wave action is shown by the photographs, but it appears that sufficient freeboard should be available at design flow. Figure D-12 shows partial capacity flows in Lind Coulee Wasteway. Of interest is the spectacular stilling basin performance at these flows.

Figures D-13 and D-14 are graphs which show the seasonal change of Manning's "n." Tests of April 18, May 2, and October 3, 1960, were omitted from these figures. These three tests were made with low flows under checked conditions. This situation resulted in extremely flat energy gradients and low canal velocities of less than 2.1 feet per second. Very small errors in measurement of water surface slopes under these conditions can result in large variations in computed friction coefficients.

On April 18, the total fall in water surface in the 6,255-foot Reach 2 was only 0.06 foot. Notes taken by measurement crews on this day indicated that gusty winds strong enough to cause whitecaps on the water existed during test periods. A 5 percent discrepancy also existed on this day between the discharges computed by field and computer methods. Recorder charts indicated
A. View showing flow characteristics upstream from the railroad culvert at Station 88+70 of the Weber Wasteway. Wasteway is carrying about 1,300 cfs in section designed for 2,000 cfs. P222-116-42538, September 19, 1960.

B. View looking downstream from railroad culvert at Station 89+16 of the Weber Wasteway. The diagonal roller at lower right results from wasteway curvature and change in invert slope. The invert grade upstream from the culvert is steeper than the grade on the reach downstream which is shown here. Flow was 1,300 cfs in 2,000-cfs section. P222-116-42539, September 19, 1960.

Figure D-11
A. Looking upstream towards 20-foot timber bridge at Station 319+83, showing flow characteristics around square post concrete piers at 4,450 cfs discharge. P222-116-42549, September 19, 1960.


Figure D-12
Note: This reach of canal was not chemically treated to retard algal growth.

### Roughness Coefficient (Manning's n)

<table>
<thead>
<tr>
<th>Test Discharge (c.f.s)</th>
<th>March</th>
<th>April</th>
<th>May</th>
<th>June</th>
<th>July</th>
<th>August</th>
<th>September</th>
<th>October</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1960</td>
<td>760</td>
<td>1400</td>
<td>2670</td>
<td>2935</td>
<td>2595</td>
<td>4470</td>
<td>985</td>
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</table>

EAST LOW CANAL
8059 Ft. Sinuous Reach 1
SEASONAL CHANGE IN MANNING'S "n"
1960 TESTS
Note: This reach of canal was not chemically treated to retard algal growth.
that canal stage rose about 0.05 foot on May 2. Test notes indicated a "choppy" water surface existed on October 3. Ree and Palmer\(^{21}\), in a report on flow in vegetated waterways, show large increases in "n" values due to plant growth in smaller earth channels. These authors plotted "n" against values of VR and found that the largest increase in "n" occurred at the lowest values of VR. The high "n" values computed for the tests of April 18, May 2, and October 3 could have been due primarily to the low velocities. However, the probability of errors in measurements prevented drawing a firm conclusion.

Figures D-13 and D-14, plotted from the 11 remaining 1960 tests, show an apparent increase of 30 percent in the value of Manning's "n" between March and August. After August, "n" decreased, but it did not return to the March value. If only the six free-flow runs are considered, Manning's "n" values increased 17 percent seasonally, from 0.0144 to 0.0169. Values of "n" from sinuous Reach 1 average generally 5 percent higher than those in straight Reach 2. A reasonable agreement between "n" values in the two reaches occurred when curve losses of 0.001 \((\Delta^o)(h_v)\) were subtracted from \(\Delta e\) values in sinuous Reach 1.

Variations between computed values of Manning's "n" for shorter segments (A to B, B to C, etc.) were less for free-flow runs where M2 curves existed. However, "n" values for segments of Reaches 1 or 2 were within 6 percent of the value computed for the entire reach length for each of the 11 test runs considered.

Figure D-15 shows Manning's "n" plotted against hydraulic radius. The trend shown by this figure indicates an increase of "n" with hydraulic radius. Some of the indicated increase in "n" could have been due to the effect of a larger hydraulic radius. However, most of the variation was believed to have been caused by the seasonal change of aquatic growth on the lining surface. The largest flows were required during a period of considerable algal growth on the canal lining. Techniques were not available at the time of these tests to examine the surfaces of the concrete lining below the water surface. It was stated that algal growth in other canals in the vicinity was heaviest in the months of August and September.

Figure D-16 is a plot of friction factor "f" against Reynolds' Number. This plot is similar to the "n" - VR chart used by Ree and Palmer\(^{21}\), in their analyses of flow in vegetated waterways, and identical to that used in References 3 and 9. The "f" values from the East Low Canal open channel tests were generally higher than those recorded for concrete closed conduits in this range of Reynolds' Numbers. The seasonal trend of increasing
NOTES
This figure shows the seasonal changes in roughness coefficient 'n' which occurred in Straght reach 2 of the East Low Canal.

Data from tests made between March and October, 1960 are shown. Discharges varied between 875 and 4,465 c.f.s.

Aquatic growths on the surface of the concrete lining apparently had more effect on increasing the 'n' values than the change in size of flow prism. This reach of canal was not chemically treated to retard algal growths.

EAST LOW CANAL
62.55 FT. STRAIGHT REACH 2
SEASONAL CHANGE IN MANNING'S 'n'
ELEVEN 1960 TESTS

FIGURE D-15
EXPLANATION
R = Reynolds' number
R = Hydraulic radius (ft.)
f = Friction factor
Se = Slope of energy gradient
g = Acceleration of gravity
$\nu$ = Kinematic viscosity
V = Average velocity

EAST LOW CANAL
6255 FT. STRAIGHT REACH 2
SEASONAL VARIATION OF "f" VALUES
ELEVEN 1960 TESTS

$\frac{f}{8.93R^0.5V^0.5}$

Von Karman-Nikuradse equation
$\sqrt{f} = 0.8 + 2 \log(R\sqrt{f})$

Smooth pipe

IR = 4RV

FIGURE D-16
resistance is again shown by this plot. Only 3 of the 11 points plotted in Figure D-16 fell outside of the theoretical range of fully developed turbulent flow.

Summary and Discussion of Test Results

Hydraulic resistance in the open channel portion of this canal was found to be greater than anticipated in design. Head losses across three inverted siphons appeared to be about 65 percent of design allowance. Due to the frequency of these siphons, the 4,500-cfs concrete-lined portion of canal between Miles 0 and 22.7 was able to carry design flow at water depths equal to or less than assumed in design. However, the situation could change if severity of algal growths on the canal lining increased beyond that which existed in August 1960.

Data from Test Reaches 1 and 2 for the six free-flow runs with M2 backwater curves are considered most applicable to design conditions. Considering only these six 1960 runs, computed values of Manning's "n" averaged 0.0145 in March, increased to a peak of 0.0169 in September with a maximum flow of 4,460 cfs, and decreased slightly to 0.0166 for the October tests. The October tests were made at flows close to those used in March and showed a 15 percent increase in Manning's "n." The apparent reason for the increased hydraulic resistance was seasonal algal growth on the concrete lining. During these tests no attempts were made to control or retard this growth in Reaches 1 and 2.

Manning's "n" values for the sinuous test reach were consistently above those in the straight reach by about 5 percent. Values for segmental parts of Reaches 1 and 2 were within 6 percent of the value for the entire reach.

At about 80 percent of design capacity, some water depths in the canal below Mile 22.7 were greater than anticipated for maximum design capacity. Manning's "n" values of 0.028 to 0.046 were computed in earth canal sections from September 1960 tests. These values are 12 to 100 percent higher than the 0.0225 used for original design. As trouble with sago pondweed has been experienced in adjacent earth sections of the canal, this growth was undoubtedly present in some degree during these tests.

The steep-slope concrete-lined Weber Wasteway should carry 2,000 cfs without serious overtopping of the concrete lining even though considerable wave action exists.
Figures 17 and 18 provide more detailed information concerning prototype flow tests made during 1960 in Test Reaches 1 and 2 of this canal. These figures also show computed values of hydraulic properties and design parameters.
Section E

FRIANT-KERN CANAL

(First 29 Miles Between Friant Dam and the Kings River Siphon)

Central Valley Project, California
A series of water surface profiles was documented and other hydraulic performance data were obtained from the first 29 miles of the Friant-Kern Canal in 1958, 1960, and 1962. This information was obtained during the irrigation seasons at flows varying from 970 to 4,560 cfs. Water depths were greater than anticipated for all test runs.

Conditions in the beginning portion of this canal are very favorable for measuring water surface profiles for the following reasons:

1. Flows can be held relatively steady because of the large storage capacity in Millerton Reservoir behind Friant Dam at the head end of the canal.

2. Discharges can be literally metered by means of the hollow-jet valves in the outlet works at the base of the dam which have been calibrated by numerous current meter gagings.

3. Few turnouts of any size exist. Very little water (a maximum of 10 cfs) was delivered from turnouts during the tests.

The first 29 miles of this concrete-lined canal and the in-line inverted siphons were built between 1945 and 1948 under Specifications No. DC-1099, 1148, 1171, and 1181. A description of construction activities can be found in the "Friant-Kern Canal, Technical Record of Design and Construction" published by the Bureau of Reclamation in 1958. The 3-1/2-inch-thick concrete lining was placed from a large rail-mounted traveling slip-form which covered the entire canal prism in one pass. Final finishing of the concrete surface and contraction grooves was performed by as many as 10 finishers working with hand tools on a jumbo following the slip-form. No construction tolerances were given in the specifications for placing the lining, but requirements stated "*** canal lining shall be finished so as to eliminate all irregularities and produce a smooth surface equivalent to the finish obtained by the effective use of a long-handled steel trowel when applied to a horizontal surface." Transverse and longitudinal contraction grooves were located at approximate 12-foot intervals and were filled with asphalt mastic.

Figures E-1 through E-3 show location, alinement, profile, and section details. The following structures existed between the
NOTES
Structures shown downstream of Sta. 1198+50 are typical of those in entire 28.5 miles of canal. Structures above Sta. 1198+50 were not shown because small-sounding scale would have created congestion on the sheet.
SINUOUS REACH No. 1  
Miles 10.70 TO 12.94

STRAIGHT REACH NO. 2  
Miles 15.20 TO 16.64

FRIANT-KERN CANAL TEST REACHES
Slope as directed in rock

Ground surface

Rock surface

Compacted embankment

CANAL SECTION

HYDRAULIC PROPERTIES

<table>
<thead>
<tr>
<th>CANAL SECTION</th>
<th>A</th>
<th>V</th>
<th>Q</th>
<th>r</th>
<th>n</th>
<th>s</th>
<th>d</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lined Section No.</td>
<td>832.48</td>
<td>1.716</td>
<td>4000</td>
<td>9.88</td>
<td>0.14</td>
<td>0.001</td>
<td>15.22</td>
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<tr>
<td></td>
<td>1858.00</td>
<td>5.056</td>
<td>5000</td>
<td>10.859</td>
<td>0.14</td>
<td>0.001</td>
<td>17.20</td>
</tr>
</tbody>
</table>

Note:

Single strands of 36 gage wire at 16-inch centers or equivalent may be used as ties across contraction joints to prevent movement of reinforcement while lining is being placed.

Thoroughly compacted fill:

Lining detail

Rock excavation

Pay line for excavation in earth

Lining detail

Earth excavation

Pay line for excavation in earth

Note: Canal lining was constructed with 3'-8 1/2" radius at toe of slope.

6'-2 1/2" Max between grooves for transverse joints

Cover with paper sleeves to prevent bond.

LONGITUDINAL SECTION

CONTRACTION JOINT DETAIL

PLAN
beginning of concrete lining at Friant Dam, Mile 0.08, and the Kings River Siphon, Mile 28.5:

- 1 combination check structure and 22-foot-diameter inverted siphon
- 1 five-barrel rectangular inverted siphon
- 23 two-pier timber bridges
- 4 three-pier timber bridges
- 1 four-pier timber bridge
- 7 one-pier concrete bridges
- 1 two-pier concrete bridge
- 4 two-pier overchutes

This list shows a total of 40 sets of piers in 28.4 miles, an average of one set every 0.71 mile. The original design did not provide an adjustment in invert grade for eddy losses at any of these structures.

The 28.4-mile reach also contains 158 horizontal curves in alignment (an average of 5.6 curves per mile). A summary of curve information is given in Table E-1.

The plane surface on the invert of the concrete lining is interrupted by the presence of about 300 underdrain hoods of the type shown in Figure E-4. These hoods are located at various intervals along the canal and enclose flap valves which allow entry of drainage water. Eighty percent of the hoods are located below Mile 17.

Two reaches of this canal were chosen for comprehensive testing. An 11,812-foot sinuous reach, called Reach 1, and a 7,583-foot straight reach, Reach 2, were selected as typical locations for tests. Alignment of these reaches is shown from aerial photographs in Figure E-2. Reach 1 contained no in-line structures and seven underdrain hoods. Reach 2 contained two 16-foot timber bridges, both of which had two 4-post timber piers, as shown in Figure E-10, and 22 underdrain hoods. The piers on the bridge at Mile 15.83 were sheathed, and round nose and tail units were added during January 1960 in an experiment to determine whether the head loss across the piers could be reduced.

Flow depths at Mile 28.5 were held constant by automatic float-controlled radial gates in the check structure at the entrance to the Kings River Siphon. Little Dry Creek check gates were free and clear of the water for all tests. Figures E-5 and E-6 show traces of water surface profiles. This portion of the canal was designed for a normal operating capacity of 4,000 cfs and a maximum flow of 5,000 cfs with 0.25 foot of freeboard to the top of the concrete lining.
-Note: Canal lining was constructed with 
3-8" radii at toe of slopes.

As directed

SECTION B-B
(TRANSVERSE DRAIN) TYPE 1 DRAINS

SECTION D-D
(6" SEWER PIPE DRAIN) TYPE 1 DRAINS

NOTE
Type 1 or type 2 drains to be constructed as directed.
Water was above top of concrete lining in reaches shown during 4,250 cfs test July 22 and 23, 1958 before copper sulfate treatment.

Average Manning's n values from 9 tests made in April 1962: 0.030 to 0.035.

Includes bridge losses
Assumed pier losses excluded

Forty in-line structures with piers in the flow prism exist in the first 28.5 miles of this canal. These structures are not shown on this drawing because of space limitations.

The structures are listed:
1) two-pier concrete bridge
2) four-pier concrete bridge
3) two-pier timber bridge
4) two-pier concrete overculverts

Water surface profile measured on July 28, 1958, 5 days after copper sulfate treatment. Q = 4,220 cfs.

Water surface profile measured on April 3, 1962, one day after copper sulfate treatment. Q = 4,490 cfs.

FRIANT-KERN CANAL
WATER SURFACE PROFILES--MILES 0.24 TO 28.5
Q = 4,220 cfs, JULY 28, 1958
Q = 4,490 cfs, APRIL 3, 1962

FIGURE E-5
in the first 28.5 miles of this canal. These structures are not shown on this drawing because of space limitations.

23 Two-pier timber bridges
4 Three-pier timber bridges
7 Two-pier concrete bridges
1 Four-pier timber bridge
1 Two-pier concrete bridge
One-pier concrete bridges

Canal cross section and range of depths at which test data were obtained is shown below.
Table E-1

Reach Description Summary of Horizontal Curve Data

<table>
<thead>
<tr>
<th>Miles</th>
<th>Length L1 (ft)</th>
<th>From To</th>
<th>No.</th>
<th>Curves per mile</th>
<th>Deflection angles (Δ°)</th>
<th>Curve radius (R1) (ft)</th>
<th>R1/T</th>
<th>Curvature index</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.08</td>
<td>3.24</td>
<td>16,680</td>
<td>23</td>
<td>7.3</td>
<td>901</td>
<td>12</td>
<td>95</td>
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<tr>
<td>3.24</td>
<td>5.49</td>
<td>11,880</td>
<td>22</td>
<td>9.8</td>
<td>913</td>
<td>7</td>
<td>100</td>
<td>41</td>
</tr>
<tr>
<td>5.69</td>
<td>5.66</td>
<td>900</td>
<td>0</td>
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<td>Little Dry Creek Siphon</td>
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<tr>
<td>5.66</td>
<td>9.07</td>
<td>18,000</td>
<td>23</td>
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<td>9</td>
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<tr>
<td>*10.70</td>
<td>12.94</td>
<td>11,810</td>
<td>16</td>
<td>7.2</td>
<td>901</td>
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<tr>
<td>12.94</td>
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<td>8,610</td>
<td>9</td>
<td>5.5</td>
<td>522</td>
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<tr>
<td>14.57</td>
<td>14.64</td>
<td>370</td>
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<tr>
<td>14.64</td>
<td>15.20</td>
<td>2,960</td>
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<td>5.4</td>
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<td>39</td>
<td>56</td>
<td>50</td>
</tr>
<tr>
<td>15.20</td>
<td>16.64</td>
<td>7,580</td>
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<td>Straight Test Reach 2</td>
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<td>16.64</td>
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<td>19.57</td>
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<td>25.99</td>
<td>28.53</td>
<td>13,410</td>
<td>9</td>
<td>3.6</td>
<td>226</td>
<td>7</td>
<td>60</td>
<td>25</td>
</tr>
<tr>
<td>28.53</td>
<td>29.15</td>
<td>3,270</td>
<td>0</td>
<td>Kings River Siphon</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Totals (Siphon lengths excluded)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>28.21 Miles</td>
<td>148,910</td>
<td>158</td>
<td>5.6</td>
<td>6,901</td>
<td>5</td>
<td>164</td>
<td>140</td>
<td>1,000</td>
</tr>
</tbody>
</table>

Average values for 28.21 miles of canal 44 332 4.5 0.23

*Sinuous test Reach 1
T = 74.05 feet for Q = 4,000 cfs
During July 1958, tests with flows of approximately 4,230 cfs were made "before and after" copper sulfate treatments. On July 22 and 23, before treatment, water overtopped the concrete lining for about 4 miles downstream from the ends of the two siphons. On July 25 and 28, after treatment, the 4,230 cfs was contained within the lining. Siphons were designed for 5,000 cfs and this resulted in a decreased depth in the open channel upstream from the siphon.

Tests made in April 1962 demonstrated that 4,500 cfs could be carried within the concrete lining with about 0.20 foot of freeboard in the "tight spots." More frequent and concentrated chemical treatments in 1962 apparently improved flow conditions by better control of algal growth on the concrete lining.

Kutter's formula with a roughness coefficient "n" of 0.014 was used for the original design of the canal. The value of 0.014 was considered adequate to provide for head losses across bridge piers and for horizontal curvature in canal alignment. Freeboard of 0.25 foot was anticipated for infrequent flows of 5,000 cfs. Equivalent Manning's "n" is 0.0143. Discharge computed using Manning's formula, "n" = 0.014 and design depth is 4,100 cfs, 2.5 percent more than normal design discharge of 4,000 cfs. Freeboard allowance for 4,000 cfs was 2.28 feet.

Test Preparations and Measurements

The first 28.5 miles of the canal were unwatered during December and January each year between 1958 and 1962; it was therefore possible to secure photographs and survey information in the empty canal. Figures E-7 and E-8 were taken in the canal during December 1960, and show several factors which have a bearing on hydraulic flow resistance. Some of these are:

1. Surface fines have been eroded from the lower two-thirds of the wetted perimeter and aggregate is exposed.

2. A series of humps, 0.1 to 0.2 foot high, exist on the invert at locations where the lining machine started and stopped. These humps are evident in that they hold pools of water behind them and prevent complete drainage.

3. Sides of the lining were clean and showed little evidence of aquatic growths, although a green "scum" was noted intermittently on the invert.
Typical lining surface and joint filler conditions in Friant-Kern Canal. Above the waterline very little, if any, of the fines have been removed, but in the lower third, the exposed aggregate may be seen. Photograph taken at Mile 13.02. P214-D-24660, December 17, 1960.

Figure E-7
Typical hump in the invert of the unwatered Friant-Kern Canal. Project personnel stated that during construction difficulty was often encountered with the lining machine at the beginning of a day's operation. The machine tended to "ride up" a slight amount, but soon settled to grade in a short distance. Shallow pools were noted behind these humps, but the depth of water rarely exceeded 0.1 foot. P214-D-24661, December 17, 1960.

Figure E-8
4. Asphalt mastic material used for filling transverse contraction grooves extruded as much as one-half inch for about one-half of the wetted perimeter in upper reaches of the canal.

Traces of actual surface roughness of the concrete lining were obtained at Mile 5.89 together with five plaster casts of typical surfaces. A distinct change in concrete lining surface texture was noted at the high-water line. Above this line, the surface was smooth and had the appearance of the original finish. Below this line, the surface was grainy and aggregate of various sizes was exposed.

Some cracks in the concrete lining of this canal have been patched with pneumatically applied mortar. The patching seams, about 6 inches wide and 3 inches high, are confined mainly to the upper 10 miles of canal. Below Little Dry Creek Siphon (Mile 5.68) and just above Fancher Creek Over chute (Mile 19.92) sides of the concrete lining were replaced because of extensive cracking. The lining in the latter location was replaced by sacked cement, and as a result, the surface was considerably rougher than the original concrete lining.

Surveyed invert and top of concrete lining profiles for Reaches 1 and 2 were plotted, and it was found that maximum deviation from the design location was 0.10 foot. Based on static pool tests, the top of lining between Mile 7.0 and 25.0 generally varied less than 0.1 foot from design grade. However, between Miles 25.0 and 28.5, the top of lining was below design grade by as much as 0.3 foot. The variations in this 3.5 mile reach exceeded the present specifications requirement for variation in height of concrete lining. This requirement states that the variation shall not exceed 1/2 of 1 percent of the lining height plus 1 inch. For the Friant-Kern Canal this amounts to 0.17 foot.

Spot checks on the invert indicated variations from design grade similar to those mentioned above for the top of lining.

Test discharges for the 1958 and 1962 tests were taken from calibration curves prepared for the hollow-jet valves in the outlet works at Friant Dam. These curves were based on numerous current meter gagings in the canal. Discharges for 1960 tests were measured by Price Type A current meters at locations near the test reaches.

No water samples were taken during the tests because the water in this portion of canal is clear. It was thought that water from Millerton Reservoir might be slightly aggressive, since surface fines have been eroded from both the Friant-Kern and Madera Canals.
A few small gravity and pump turnouts exist in the first 28.5 miles of the canal. Discharges from these turnouts amounted to less than 1 percent of test flows. Seepage and evaporation losses were assumed to be 0.25 cfs per mile.

Examinations of the unwatered canal in December 1959 and December 1960 showed clean concrete lining surfaces on the sides. The invert was partially covered by a green "scum" in some areas where water was ponded. This scum undoubtedly contained some fragments of filamentous algae growth. Two photographs taken from bridges in July 1958, Figure E-9, show filamentous algal growth near the water line on the sides of the canal before and after copper sulfate treatment. No equipment was available to photograph lining surfaces below the water during tests to document the extent of algal growth.

No accumulations of silt or fresh water clams were reported in test reaches during the years of the tests. Appreciable silt deposits (1 to 6 inches deep) with minute fresh water clam populations were, however, reported beyond Mile 85 of this canal. Pondweed growth on these silt deposits has also been noted. Small forage fish and thread fin shad have created problems by clogging outlet valves in pipe distribution systems near the downstream end of the canal.

The filamentous algae, Cladophora, is the most prevalent growth in this canal. A species of Oedogonium often infests the canal below Mile 18 from August through October. This species grows only where organic detritus from the reservoir has been laid down on the substrata. The blue-greens, Anabena and Oscillatoria, build up on the substrata of the canal during midseason and persist until the canals are dewatered. Diatoms, also algae of the genera Pinnularia and Navicula can be found in the gelatinous matrix secreted by the blue-greens.

Copper sulfate treatment procedures are used to retard algal growth in the canal. The 1962 treatments consisted of two pounds of copper sulfate crystals for each cfs of water flowing. Applications of this chemical were made at the head of the canal and at Mile 85.6 by dumping crystals from bags as rapidly as possible at locations where the water was most turbulent. Treatments were made biweekly, commencing on March 1 and continuing through October. During the 1962 season, 99,600 pounds of copper sulfate were introduced at Friant, Mile 0, and 71,560 pounds at Lindsay, Mile 85.6. Total cost for these treatments was $24,000.
A. View of filamentous algae on upper portion of concrete lining in the Friant-Kern Canal. Photograph was taken from the upstream side of the bridge at Mile 0.16 in a portion of canal which is not chemically treated to retard algal growth. Filamentous algae grows on the sides of the lining in the first 28 miles of canal during the irrigation season. IK-824, July 30, 1958.

B. View of clean concrete lining taken from downstream side of bridge at Mile 0.16 six days after a 2 lb/cfs copper sulfate treatment in July 1958. This chemical retards algal growth and is dumped from downstream side of this bridge as rapidly as possible. During 1962, 100,000 pounds of copper sulfate crystals were required for the 17 biweekly treatments made during the irrigation season. IK-825, July 30, 1958.

Figure E-9
In July 1958, the effect of a single copper sulfate treatment (2 lb/cfs) was documented. Previous treatments earlier in the season were not reported. "Before and after" results in terms of Manning's "n" are shown in Table E-2.

There has been no mechanical scraping of the concrete lining of this canal to increase capacity.

Individual head loss measurements were obtained across the following structures:

<table>
<thead>
<tr>
<th>Structure</th>
<th>Date</th>
</tr>
</thead>
<tbody>
<tr>
<td>Little Dry Creek Check and Siphon</td>
<td>July and August 1958</td>
</tr>
<tr>
<td>Dry Creek Siphon</td>
<td>July 1958</td>
</tr>
<tr>
<td>Kings River Siphon</td>
<td>July 1958</td>
</tr>
<tr>
<td>Two 16-foot timber bridges (Miles 15.23 and 15.83)</td>
<td>July and August 1958 and August 1960</td>
</tr>
<tr>
<td>20-foot timber bridge (Mile 20.17)</td>
<td>July 1958</td>
</tr>
<tr>
<td>24-foot timber bridge (Mile 14.0)</td>
<td>July and August 1958</td>
</tr>
<tr>
<td>30-foot concrete bridge (Mile 28.31)</td>
<td>August 1958</td>
</tr>
<tr>
<td>Two 16-foot timber bridges and inclusive canal reach (Mile 15.23 to 15.83)</td>
<td>July 1958</td>
</tr>
<tr>
<td>Three bridges, one over chute and inclusive canal reach (Mile 19.59 to 20.17)</td>
<td>July 1958</td>
</tr>
<tr>
<td>Three 16-foot timber bridges, one 30-foot concrete bridge and inclusive canal reach (Mile 27.38 to 28.46)</td>
<td>July 1958</td>
</tr>
<tr>
<td>Over chute (Mile 19.92)</td>
<td>July 1958</td>
</tr>
</tbody>
</table>

Tests numbered 126 through 155A in Figures 20 and 21 were taken from data obtained in Reaches 1 and 2. They show flow resistance in sinuous Reach 1 to be higher than in straight Reach 2 in all cases. Expressed in terms of Manning's "n," the average value of 0.0169 from Reach 1 is 12 percent higher than the average of 0.0151 for Reach 2 after estimated head losses due to bridge piers were subtracted.

**Summary and Discussion of Test Results**

Near capacity flow in the first 28.5 miles of this canal was characterized by a series of three long M2-type backwater curves, each beginning at the inlet of an inverted siphon. Water depths increased in an upstream direction from these points and were greatest just below the outlets of Little Dry Creek and Dry Creek Siphons. Water depths which existed in these "tight spots" were 8 to 13 percent greater than anticipated in the original design. Computed open channel roughness coefficients (Manning's
Table E-2
PRIAMT-KERN CANAL
Manning's "n" Values (Including Pier Losses) For 1958 Capacity Tests

<table>
<thead>
<tr>
<th>Reach (in miles)</th>
<th>Manning's &quot;n&quot; values</th>
<th>Percent change</th>
<th>Reach Description</th>
</tr>
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<tr>
<td></td>
<td>Before Cu SO₄ Treatment</td>
<td>After Cu SO₄ Treatment</td>
<td></td>
</tr>
<tr>
<td>From</td>
<td>To</td>
<td></td>
<td></td>
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<tr>
<td>0.24</td>
<td>3.24</td>
<td>0.0182</td>
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<td>3.24</td>
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<td>5.69</td>
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<td>14.66</td>
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<td>15.22</td>
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<td>15.84</td>
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<td>0.0161</td>
<td>0.0159</td>
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<tr>
<td>16.60</td>
<td>18.11</td>
<td>0.0187</td>
<td>0.0185</td>
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<td>18.11</td>
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<td>25.99</td>
<td>0.0187</td>
<td>0.0183</td>
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<tr>
<td>25.99</td>
<td>27.37</td>
<td>0.0183</td>
<td>0.0187</td>
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<tr>
<td>27.37</td>
<td>28.32</td>
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<td>Weighted average</td>
<td>0.0182</td>
<td>0.0184</td>
<td>0.0178</td>
</tr>
</tbody>
</table>

Note: Head loss measurements were made across the bridges shown above. Numerous other bridges were present but were not listed in this tabulation. Brief possible explanations for "n" values which varied considerably from the average are given in the right hand column.
"n" values) which included all losses, ranged from 0.015 to 0.022 in 1958. Values computed from 1962 test data ranged from 0.015 to 0.017. (However, reaches from which highest and lowest values were computed in 1958 were not included in 1962 test measurements.) Average "n" values for Test Reaches 1 and 2 were about 10 percent lower in 1962. The chief reason for the reduced flow resistance in 1962 appeared to be bimonthly copper sulfate treatments of 2 pounds per cfs which were made during the entire irrigation season to control algal growths. Cost of the 32 copper sulfate treatments made between March and November of 1962 was $24,000. Treatments were made at Miles 0.16 and 85.6.

The increased flow resistance (above that anticipated in original design) which occurred in the open channel portion of the canal was caused, at least in part, by aquatic growths on the lining surface, structure piers in the flow prism, and pronounced sinuosity in horizontal alinement. Another contributing factor was the erosion of surface fines from the surface of the original lining. This erosion exposed aggregate around the lower two-thirds of the wetted perimeter of the canal section and resulted in a rougher flow boundary. Aquatic growths appear to be the major factor which affected resistance. The filamentous algae Cladophora was the most prevalent growth in the canal, but blue-greens and diatoms were also reported. Silt deposits, pondweed, small forage fish, and fresh water clams were found below Mile 85. Copper sulfate is the only chemical which has been used extensively in this canal to control aquatic growths through 1962.

Roughness coefficients computed for sinuous Reach 1 averaged about 8 percent higher than for straight Reach 2 before estimated pier losses were subtracted and 12 percent after pier losses were deducted. The first 14 miles of this canal was one of the more sinuous reaches in which measurements were obtained in the current series of tests.

Head losses measured across two inverted siphons were about 50 percent of the design allowance.

Measured head losses across canal structure piers varied from 0 to 0.09 foot. These losses, although individually small, were sufficient to cause a significant increase in water depth, especially in reaches of the canal where several sets of piers were located relatively close together. The effect of these small losses on water depth can be most accurately predicted by backwater curve computations. The computation will show that the individual head loss at each bridge is not always additive, i.e., the full loss is not always reflected upstream to the next set of piers. Overall effects on channel performance depend on pier size, configuration, and spacing along the channel.
Figures 20 and 21 provide more detailed results concerning prototype flow tests made during 1958, 1960, and 1962 in Test Reaches 1 and 2 of this canal. These figures also show computed values of hydraulic properties and design parameters.
Section F

ROZA MAIN CANAL

Yakima Project, Washington
ROZA MAIN CANAL
Yakima Project, Washington

Water surface profile measurements in two concrete-lined reaches of the Roza Main Canal* were obtained on June 20 and 21, 1962. Figures F-1 and F-3 show locations of the test reaches and water surface profile measuring stations. Reach 1 was located in the 14-foot bottom width canal section designed for 2,200 cfs. Reach 2 was located in the 12-foot bottom width 1,300 cfs section. Measurements were made on June 20 in Reach 1 when the flow was 2,125 cfs and on June 21 in Reach 2 when the flow was approximately 1,070 cfs.

Figures F-2 and F-4 show canal profile and section details. Reach 1 was 5,975 feet long and contained 10 horizontal curves. For design discharge, a vertical freeboard allowance of 1.8 feet from water surface to top of concrete lining was provided. Reach 2, 13,881 feet long, contained 13 horizontal curves and was provided with 1.43 feet of freeboard at design discharge. No inline canal structures with piers in the flow prism existed within the test reaches. Four water surface profile measuring stations, A through D, were established in Reach 1, and six stations, E through J, were established in Reach 2. Reach 2 contains a 6,275-foot portion which is virtually straight.

The 4-inch-thick concrete lining in the test reaches was constructed under Specifications No. 675 and 748 and was completed in 1937-38. The lining was reinforced with 1/2-inch round bars spaced at 12 inches longitudinally and 24 inches transversely. Kutter's formula having a roughness coefficient "n" of 0.014 was used for the design of this canal. Equivalent Manning's "n" for Reach 1 was 0.0145; for Reach 2, 0.0143. Discharges at design depth in Reach 1 computed using Manning's "n" of 0.014 is 2,273 cfs; in Reach 2, 1,327 cfs.

Concrete lining in both test reaches was placed from rail-mounted slip-forms. Specifications requirements stated that "the concrete lining shall be brought to a uniform surface and worked with suitable tools to a smooth steel-trowel finish." Considerable steel troweling by hand from jumbos which followed the slip-form was required to meet this requirement. Transverse contraction grooves in the lining were placed at approximate 12-foot intervals,

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*This canal was originally called the Yakima Ridge Canal but was renamed soon after construction.
DRAWING 1. Section of concrete lined canal shown below

FIGURE F-2

ROZA MAIN CANAL - YAKIMA PROJECT
REACH I, STA. 420+55 TO 480+30
1962 WATER SURFACE PROFILE AND INVERT SURVEY DATA

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
YAKIMA PROJECT - WASHINGTON
ROSSA DIVISION
YAKIMA MAIN CANAL - STA. 1120 +00 TO 1155 +00
LOCATION MAP

FIGURE F-3

29284  DENVER, COLORADO, JUNE 1937  33-0-1132
Water surfaces computed using Manning's n = 0.014
for design flow of 1,300 cfs.
for test flow of 1,300 cfs.

Water surface computed using Manning's n = 0.014
for design flow of 1,300 cfs.
for test flow of 1,300 cfs.

Cross-section of this concrete lined canal shown below.
but no longitudinal grooves were provided in either test reach. No joint filler was evident in the transverse grooves at the time of tests. No fillet was placed at the intersection point of the side and bottom of the section in Reach 1. A circular fillet was provided at the toe of the slope in Reach 2.

The 2,200-cfs section of this canal is usually operated year-around but was drained briefly on March 13, 1962. At this time, the lining in Reach 1 was inspected and photographed. (See Figure F-5)

A few small deposits of rocks were noted and a slight algal growth had formed a thin skin on the invert. The lining on the side slopes was covered with a thin layer of clay to the maximum operating water surface of the past season. No slab displacement was noted, but some longitudinal cracking had occurred on the invert.

The 1,300-cfs section, Reach 2, is unwatered at the end of the irrigation season, and the condition of the lining was documented in March 1962. The lining appeared to be in excellent condition. A few small rock deposits were noted and longitudinal cracking had occurred at random locations on the inverts. Surface drainage from an alkali soil area near Station J had apparently caused local concrete lining surface deterioration.

Operating personnel were questioned regarding algal growth in this canal. They stated that a "moss" up to 1-1/2 inches in length grows on the lined section during the warmer parts of the operating season and reduces the capacity of the 2,200-cfs section. No chemical treatments have been used to retard the growth in this section because the cost-benefit ratio is considered too high. Copper sulfate and aromatic solvents have been used on smaller laterals in the area.

Water in this canal is slightly turbid, but there was insufficient sediment in the water to change the appearance from the typical bluish-green color of clear water to the brown exhibited by streams carrying a great deal of silt.

Test Preparations and Measurements

Surveyed canal profile data are plotted in Figures F-2 and F-4. In Reach 1, 22 of 24 surveyed points were above design grade and 2 were below. Average deviation from grade was 0.09 foot and maximum deviation was 0.21 foot. Survey datum for this reach
A. View toward Point A of 2,200 cfs Reach 1 during unwatered period prior to tests. Note that this reinforced concrete lining contained no longitudinal contraction grooves. P33-D-32811, March 1962.

B. Closeup of 24-year old concrete lining surface at Point E of 1,300 cfs Reach 2. Dark material was reported to be original curing compound above the waterline and stain below the waterline. P33-D-32828, March 1962.

Figure F-5
was based on as-built elevations. Surveys were carried from a brass marker (elevation 1203.43) located on the headwall of the outlet transition of a highway culvert just upstream from Station 418+66. The invert in Reach 2 was closer to design grade. There, 37 of 49 surveyed points were above grade and 12 were below. Average deviation was 0.05 foot; maximum was 0.15 foot. Survey datum for this reach was based on as-built elevations. Surveys were carried from a brass marker (elevation 1173.13) located on a turnout structure just upstream from Station 1155+80. Surveyed cross sections at the ten water surface measuring points were plotted and the measured areas were found to be within 1 percent of the design section.

Steady flow conditions for test measurements were established by holding constant releases from Roza Diversion Dam at the head end of the canal. Canal discharge was controlled by one of the 110-foot drum gates at the dam. This gate was placed on automatic controls which were adjusted to compensate for a 6-inch change in reservoir water surface elevation.

Discharge measurements were made concurrently with water surface profile measurements at Station B in Reach 1 and Station G in Reach 2, Figure F-6. Price Type A current meters were used for gaging, and velocities at 0.2 and 0.8 depths were obtained. Canal inflow through drains and outflow through turnouts was less than 1 percent of test flows in each reach. Wave action for the 7.1-fps velocity in Reach 1 and the 5.9 fps velocity in Reach 2 was reported to be minor and did not hamper test measurements.

Water stage recorders were located at Station B in Reach 1 and Station H in Reach 2. Charts from these recorders indicated a very slight fluctuation of canal stage with diurnal changes in river flow at Roza Dam. Although a great deal of care was exercised to maintain steady flow, drum gate movement at Roza Dam allowed a 0.05-foot change in canal stage in a 12-hour period. Since test measurements required approximately 3 hours in each reach, the change in stage was not significant.

Water surface gages (Figure 4) developed by the Hydraulics Branch, which utilize the pitot-tube and hook gage stilling well principle, were used to obtain water surface elevations. Figure F-7 shows crews using these gages. At least five readings of water surface elevation were obtained on each side of the canal at each station.
A. View of current meter discharge measurement location in 2,200 cfs-Reach 1. Discharge measurements were made from bridge located just below water surface profile measuring Point B at Station 435+40. P33-D-33883, June 20, 1962.

B. View of current meter discharge measurement location in 1,300 cfs-Reach 2. The temporary bridge is located at Point G. Station 1221+55.8. P33-D-33880, June 1962.

Figure F-6
A. View of flow condition at end of Reach 1; transition to earth section is shown in background. Flow was 2,125 cfs in section designed for 2,200 cfs. P33-D-33895, June 1962.

B. View looking downstream at Point J showing crews making water surface profile measurements. P-33-D-33924, June 1962.

Figure F-7

149
Table F-1

Reach 1

$L = 5,975$ feet; $T =$ Top width of design water surface $= 42.0$ feet

<table>
<thead>
<tr>
<th>Water surface measuring point</th>
<th>Curve No.</th>
<th>Deflection angle $\Delta^o$</th>
<th>Curve Radius $R_1$ (feet)</th>
<th>$R_1/T$</th>
<th>Curvature index</th>
<th>Reach length (feet)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>1</td>
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<td>287</td>
<td>6.8</td>
<td>0.48</td>
<td>1,485</td>
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<td>2</td>
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<td>B</td>
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<tr>
<td>Average values</td>
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<td>$\Sigma \Delta =$</td>
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<td>Total length</td>
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<td>5,975 feet</td>
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</table>
Table F-2

Reach 2

$L = 13,881$ feet; $T =$ Top width of design water surface $= 34.7$ feet

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<th>Water surface measuring point</th>
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<th>Deflection angle $\Delta^\circ$</th>
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<th>$R_1/T$</th>
<th>Curvature index</th>
<th>Reach length (feet)</th>
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<td>82.5</td>
<td>0.03</td>
<td>2,681</td>
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<tr>
<td></td>
<td>11</td>
<td>43</td>
<td>287</td>
<td>8.3</td>
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<td>67</td>
<td>347</td>
<td>10.0</td>
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<td></td>
</tr>
<tr>
<td>J</td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
</tbody>
</table>

Average values: 50 | 766 | 22.1 | 0.23

$\Sigma \Delta = 652$ 5 curves per mile  Total length $= 13,881$ feet
Analyses of Test Data

Figures F-2 and F-4 show the measured water surface profiles. The slope of the water surface in each test reach was very close to that of the design invert. However, the slope does appear to be slightly steeper in the more sinuous segments. A short M2 backwater curve is evident at the end of Reach 1 just above the transition to an earth section. Water depth changed less than 0.2 foot (2.5 percent) in the test reaches.

Horizontal alignment curves in the test reaches are listed in Tables F-1 and F-2.

Summary and Discussion of Test Results

Computed values of Manning's "n" for the two test reaches are shown in Figures F-2, F-4, and F-8 and results are summarized below:

Reach 1, 14-foot bottom width, "n" varied from 0.0136 to 0.0148. Weighted average of three sinuous segments = 0.0143

Reach 2, 12-foot bottom width, "n" varied from 0.0131 to 0.0153.
Weighted average of two straight segments = 0.0133
Weighted average of three sinuous segments = 0.0148
Weighted average of all five segments = 0.0141

Figure 22 shows detailed information concerning prototype flow tests made during June 1962 in this canal. This figure shows computed values of hydraulic properties and design parameters.

This summary shows Manning's "n" values to be within 9 percent of the "n" of 0.014 used in Kutter's formula for the original design. The apparent reason for the slightly higher resistance in Reach 1 and in the last half of Reach 2 is the fact that these sections are more sinuous. Figure F-8 indicates a general increase of "n" values as curvature index increases.

The portions of this canal utilized for tests were not chemically treated to retard aquatic growths at any time prior to the tests. No appreciable amount of aquatic growths was evident during the tests.
CURVATURE INDEX = $\frac{5(\Sigma A)}{L_1}$

ROZAV MAIN CANAL
JUNE 1962 CAPACITY TESTS

FIGURE F-8
Section G

MADERA CANAL

Central Valley Project, California
MADERA CANAL
Central Valley Project, California

Water surface profile measurements in the first 8 miles of Madera Canal were made in 1959, 1960, and 1962. Figure G-1 shows the location of the canal and the three test reaches which are documented in this Technical Memorandum. This portion of the canal was lined with concrete placed from a traveling rail-mounted slip-form. Bids for construction were opened on December 21, 1939, and work was completed in 1942 during World War II. Original design drawings in Specifications No. 886 called for a reinforced concrete lining, but some reaches were unreinforced due to wartime steel shortages. Specifications requirements stated that "the canal lining shall be finished so as to eliminate all irregularities and produce a smooth surface equivalent to the finish obtained by the effective use of a long-handled steel trowel when applied to a horizontal surface." Conversations with personnel present during construction have indicated that a considerable amount of hand troweling was done to meet this requirement. Transverse contraction grooves were located at approximate 12-foot intervals along the canal centerline. No mastic filler was evident in these grooves in 1960.

Figure G-2 shows canal profile and section details. Test data were obtained from the 10-foot bottom width 1,000-cfs section and the 8-foot bottom width 823-cfs section at approximate full and half capacity flows. Figure G-3 is an aerial photo of the concrete-lined portion of this canal which shows its sinuosity. Reach 1, 8,052 feet long, was the most sinuous test reach with a curvature index of 0.35. Reach 2, 3,649-feet long, was moderately sinuous with a curvature index of 0.12; and Reach 3, 5,137-feet long, was virtually straight with a curvature index of 0.02.

Kutter's formula with a roughness coefficient "n" of 0.014 was used for the design of the canal. Equivalent Manning's "n" for the 1,000-cfs section is 0.0142 and for the 823-cfs section, 0.0141. Discharges at design depths in the respective sections computed with Manning's formula and "n" = 0.014 are 1,017 cfs and 831 cfs. Vertical freeboard allowance from design water surface to the top of the concrete lining was 1.0 foot for both sections.

In-line structures in the first 6.1 miles of canal were designed for 1,500 cfs in anticipation of future demands, but concrete lining height was based on 1,000 cfs. Below Mile 6.1 future discharge was 1,325 cfs, but lining freeboard was based on 823 cfs. Considerable drawdown of the water surface existed at siphon inlets due to the extra head loss allowed for these structures.
Concrete lined canal sections and ranges of depths at which tests were made are shown below.

**Lined Section No. 1**

**Lined Section No. 2**

**Concrete Lined Canal Sections and Ranges of Depths at Which Tests Were Made**

<table>
<thead>
<tr>
<th>Lined Section</th>
<th>Design Location</th>
<th>Depth Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lined Section No. 1</td>
<td>Sta. 300+00 to 360+00</td>
<td>0.0128 to 0.0142</td>
</tr>
<tr>
<td>Lined Section No. 2</td>
<td>Sta. 209+88 to 383+21</td>
<td>0.0144 to 0.0152</td>
</tr>
</tbody>
</table>

**Manning's n Values**

- For reaches shown:
  - Lined Section No. 1: 0.014
  - Lined Section No. 2: 0.015

**Water Surface Profiles and Invert Survey Data**

**Figure G-2**

*Madera Canal—Central Valley Project*

*Sta. 209+88 to 383+21*

*Water Surface Profiles and Invert Survey Data*
MADERA CANAL—CENTRAL VALLEY PROJECT, CALIFORNIA
Reaches Utilized for 1959-62 Capacity Tests
Check structures were present at the downstream ends of the first two test reaches. No other in-line structures with piers in the flow prism were present in the length of canal utilized for tests. Water surface profile measuring stations, A through F, were located at the ends of these reaches as shown in Figure G-1. This arrangement allowed an overall evaluation of head loss. No attempt was made to isolate individual horizontal curve losses.

The operating period for the canal generally extends from March through October, and it is unwatered at the end of each irrigation season. Therefore, it was possible to obtain invert survey data and to examine the condition of the 17-year old lining surfaces in detail. At the time of the tests, surface fines had been eroded from the lower two-thirds of the section, as shown in Figure G-4. Aggregate was exposed and erosion of up to 1 inch was noted on siphon inlet transitions. Cracking has occurred at various locations along the concrete lining. Field maintenance personnel have covered most of the cracks with pneumatically placed mortar mounds to prevent leakage. The 22 mounds in the first two reaches average 9 inches wide, 3 inches high, and 140 feet long, and contributed to surface roughness characteristics.

An experimental asphalt lining, 500 feet long, had been applied in the first reach, but approximately one-half of this surfacing had peeled off at the time of the tests. Surveyors stated that test reaches contained no other unusual humps or depressions attributable to bulging or caving-in of the lining. Seven-foot-long traces of typical lining roughness were obtained at three locations in test reaches.

Figure G-4 shows dried filamentous algae on the invert. Operating personnel state that this type of algae grows extensively on the lining surface and is the greatest single factor in reducing the canal efficiency. The canal is treated periodically with copper sulfate to reduce algal growth. "Before and after" tests were obtained to document effectiveness of these treatments. Water surface profiles obtained on July 27, 1959, and August 2, 1960, immediately preceded treatments of the following amounts:

July 27, 1959, Q = 985 cfs, 1 pound copper sulfate per cfs in 1 hour

August 2, 1960, Q = 970 cfs, 2 pounds copper sulfate per cfs applied as rapidly as possible

Water surface profiles measured on July 8, 9, 28, and 31, 1959, August 3 and 4, 1960, were made after the copper sulfate treatments.
Looking upstream from entrance of siphon at Mile 3.10, Madera Canal. Note the deposits of dried algae on the bottom of the canal and the erosion of surface fines from the concrete lining. The wavy high-water mark is not caused by misalignment of the lining, but results from a rough water surface caused by the approximate 5.0 fps velocity. P214-D-24662, December 18, 1960.

Figure G-4
Project personnel stated that the 823-cfs canal section appears to be more sensitive to algal growth than the 1,000-cfs section and that flow resistance has been noted to vary with light and water temperature. Water surface recorder charts for steady discharge conditions have shown fairly constant depths during the period from 8 p.m. to 4 a.m., a slight reduction in depth from 4 a.m. to 8 a.m., and an increase in depth from 8 a.m. to 8 p.m.

Examination of 1962 operating records from Mile 0.9 of this canal revealed that a highly effective copper sulfate treatment procedure was evolved. The treatment consisted of biweekly treatments (2 pounds of copper sulfate crystals per cfs) applied in the stilling basin below Friant Dam at the head end of the canal. During 1962, treatments were started on April 9 and continued until August 30. Discharges greater than the 1,000-cfs design capacity were conveyed in the first 6 miles of canal without overtopping the lining during the peak delivery period between June 15 and August 15. Peak discharge was 1,140-cfs, 11 percent above design capacity, on July 11. Figure G-5 shows flow conditions in the canal at a discharge of 975 cfs.

Description of Tests

Steady flow conditions were established by holding constant releases from Millerton Lake above Friant Dam at the head end of the canal. Discharges in the 1,000-cfs section for the 1959 tests were obtained from the rating curves which had been prepared for the needle valves in the outlet works at Friant Dam. The rating curves were checked by numerous current meter gageings at Mile 1.1 in the canal. Discharges for the 1960 and 1962 tests were measured from the farm bridges at Miles 4.57 or 5.68. Discharge for the 823-cfs section was obtained by subtracting the flow diverted through the Parshall flume at Mile 6.1 from the flow in the 1,000-cfs section. Seepage and evaporation losses were estimated to be less than 2 cfs for the test reach lengths. As no turnouts exist within the test reaches, no water was diverted therefrom.

Sediment has never been a problem in this portion of canal, as the water from Millerton Reservoir is relatively clear. Therefore, sediment concentrations in the water were not measured.

Prints of water surface recorder charts were not submitted with test data. Operating personnel stated that the change in head on the Friant Dam outlet valves was very small during test periods and that no significant change in canal stage occurred.
View of curved reach of 1,000 cfs Madera Canal upstream from check structure at Mile 5.37. Flow was 975 cfs and freeboard was close to the 1.0 foot allowed in the original design. P214-D-22133, July 1960.

Figure G-5
Wind velocities and directions were not recorded and were not considered to be a significant element of flow resistance.

Water surface elevations were obtained from slope measurements made with a steel tape from bench marks on the top of the concrete lining. The steel tape was placed in a stilling can to damp wave action for the 1959 and 1960 tests. Water surface elevations were measured on one side of the canal in 1959 and on both sides in 1960 and 1962.

Measuring Stations B and C were located on horizontal curves. The maximum tilt in the water surface obtained from test data appeared to be 0.15 foot at Point B. Theoretical water surface rise based on the channel geometry at this curve is 0.09 foot. For 1960 and 1962 tests, an average of the water surface elevations obtained on both banks was used for analysis of data. Water surface elevations obtained in 1959 at Point B on the left bank only were corrected for tilt based on the average measured tilt observed during the 1960 and 1962 tests.

A 100-foot length of canal lining on the outside of the curve just upstream of Point A has been raised 12 inches. This lining extension is located at the end of a tight curve which occurs at the end of a siphon transition. (Curve $\Delta = 47^\circ$, radius = 120 feet.)

Bench mark elevations at the water surface profile measuring stations were established by a 1961 survey and were based on an assumed datum of 100.00 for a bench mark at Mile 3.85. Invert elevations at 300-foot intervals along the canal centerline were also obtained during this survey. These values are plotted with respect to the design slope in Figure G-2. The following comment was made by surveyors concerning these invert elevations: "Although invert elevations can be considered accurate, it cannot be said that the elevation varies uniformly between them. By taking several shots at different points on the invert, it was found that many panels varied 0.03 to 0.04 foot within themselves with perhaps a maximum of 0.08 foot between adjacent panels." In general, Figure G-2 shows plots to be within 0.10 foot of the design slope. Points in the 8-foot bottom width Reach 3 showed less variance from the design slope than those in the 10-foot bottom width reaches.
Analyses of Test Data

Backwater effects in the three test reaches were minor. The most pronounced effect occurred in Reach 2 for the runs of July 8, 1959 and May 8, 1962. This phenomenon was expected for low flows since the canal invert rises 1.4 feet through the check structure at the downstream end of the reach. Generally, changes in depth in reaches amounted to less than 0.20 foot which was less than 3 percent of total depth.

Figures 23 and 24 show summaries of test data for this canal. Figure G-6 shows plots of Manning's "n" values against curvature index. The average of "n" values computed for Reach 1 was slightly lower than for Reach 2, although there were considerably more curves in Reach 1. Reach 3, which was the straightest, had "n" values about 6 percent lower than either of the first two reaches. Horizontal alinement curves in the three reaches are described in Table G-1.

Summary and Discussion of Test Results

The first 6-mile section of the concrete-lined Madera Canal has conveyed up to 10 percent more than design capacity within the concrete lining during peak delivery periods. This was accomplished when the lining was relatively free of algal growth with some encroachment on design freeboard. Biweekly chemical treatments (2 pounds of copper sulfate per cfs) during 1962 enabled delivery of design flow or greater between June 15 and August 15. Cost for these chemical treatments in 1962 was approximately $650. Records for 1958 and 1959 show that flows in Reaches 1 and 2 were 5 percent above the design value during peak delivery months of June, July, and August. The canal was dosed with copper sulfate six times during each of these years. A flow 15 percent above the design value was carried in Reach 3 during the 1959 tests with 0.5 foot of freeboard.

Computed Manning's "n" values for nine tests in each of the three test reaches are plotted in Figure G-6 and are summarized in Table G-2.

This summary shows a slight reduction in "n" after biweekly chemical treatments. Project records from the gaging station at Mile 0.9 indicate improvements of 2 to 4 percent in canal "efficiency" after biweekly copper sulfate treatments during peak delivery periods.
MADERA CANAL
1959, 1960 AND 1962 TESTS
(Biweekly copper sulfate treatments)

ROUNISHNESS COEFFICIENT (MANNING'S "n")

CURVATURE INDEX = \( \frac{5(\Sigma \Delta)}{L_1} \)

FIGURE G-6
Table G-1
Reach 1

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<th>W.S. measuring point</th>
<th>Curve number</th>
<th>Deflection angle $\Delta^\circ$</th>
<th>Curve radius $R_1$</th>
<th>$R_1/T$</th>
<th>Curvature index</th>
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B*  Average values  34  206  6.3

$\Sigma \Delta = 570$ An average of 11.1 curves per mile

Reach 2

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<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>18</td>
<td>37</td>
<td>200</td>
<td>6.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>19</td>
<td>31</td>
<td>300</td>
<td>9.2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>20</td>
<td>21</td>
<td>300</td>
<td>9.2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

D  Average values  30  267  8.2

$\Sigma \Delta = 89^\circ$ An average of 4.3 curves per mile

Reach 3

<table>
<thead>
<tr>
<th>E</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>21</td>
<td>15</td>
<td>300</td>
<td>11.7</td>
<td>0.02</td>
<td>5,137 ft</td>
</tr>
<tr>
<td>22</td>
<td>7</td>
<td>300</td>
<td>11.7</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

F  Average values  11  300  11.7

$\Sigma \Delta = 22^\circ$ An average of 2 curves per mile

* Point B was located on Curve No. 17
** Point C was located on Curve No. 18
Table G-2

8,052-foot Sinuous Reach 1

<table>
<thead>
<tr>
<th>Before copper sulfate treatment</th>
<th>After copper sulfate treatment</th>
</tr>
</thead>
<tbody>
<tr>
<td>(3 test runs)</td>
<td>(6 test runs)</td>
</tr>
<tr>
<td>0.0146 to 0.0151 Variation</td>
<td>0.0145 to 0.0148</td>
</tr>
<tr>
<td>0.0149</td>
<td>0.0147</td>
</tr>
</tbody>
</table>

3,649-foot Moderately Sinuous Reach 2

<table>
<thead>
<tr>
<th>0.0144 to 0.0155 Variation</th>
<th>0.0144 to 0.0152</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0150 Average</td>
<td>0.0149</td>
</tr>
</tbody>
</table>

5,137-foot Reach 3 (Nearly straight)

<table>
<thead>
<tr>
<th>0.0136 to 0.0147 Variation</th>
<th>0.0128 to 0.0144</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0142 Average</td>
<td>0.0138</td>
</tr>
</tbody>
</table>
Weighted average "n" for all tests in the 10-foot bottom width sections Reaches 1 and 2 was 0.0148. For the 8-foot bottom width section (Reach 3) the weighted average "n" was 0.0139. Apparent reasons for this variation are:

1. Reach 3 was straighter than Reaches 1 and 2.

2. Algal growth was believed to be more severe in the upper end of the canal near the water supply source.

3. Erosion of surface fines and exposure of aggregate on the lower two-thirds of the canal lining wetted perimeter has left a "grainy" surface considerably rougher than the original finish.

4. Intermittent pneumatically placed mortar seams had been applied over lining cracks in Reaches 1 and 2. These 3-inch-high patches created additional surface roughness and increased flow resistance.

Figures 23 and 24 provide more detailed information concerning flow tests made during 1959, 1960, and 1962 in the canal. These figures also show computed values of hydraulic properties and design parameters.
Section H

GATEWAY CANAL

Weber Basin Project, Utah
GATEWAY CANAL
Weber Basin Project, Utah

Water surface profile measurements in the concrete-lined Gateway Canal were obtained on April 26, 1962, at near design flows. Figure H-1 shows locations of the test reaches and the water surface profile measuring stations. Figures H-2 and H-3 show canal profile and section details in the test reaches.

A 9,446-foot moderately sinuous length, Reach 1, and a 4,262-foot moderately sinuous length, Reach 2, were selected as typical locations for comprehensive test measurements. Eleven water surface measuring stations, designated alphabetically A through J were established in Reach 1, and five stations, K through N, were located in Reach 2. Reach 1 contained three check structures of the type shown in Figure H-4 and one 132-inch-diameter in-line siphon across which head-loss measurements were obtained. No other in-line structures with piers in the flow prism existed in the test reaches.

The concrete lining for this canal was built in 1955 under Specifications No. DC-4207. The 4-inch-thick concrete lining was generally unreinforced and was placed by a rail-mounted slipform. It was given a steel-troweled finish by finishers working from a jumbo which followed the lining machine. Transverse contraction grooves were spaced at 16-foot intervals and filled with asphalt mastic. One longitudinal contraction groove on each side of the concrete lining was called for in the specifications, but these grooves are not evident in Figure H-5. Considerable cracking of the sides had occurred, as evidenced by this figure. Cracks have been repaired with asphalt mastic.

Design flow in this canal is 700 cfs. Vertical freeboard allowance from design water surface to top of concrete lining was 1.43 feet. Kutter's formula with an "n" of 0.014 was used for the design of this canal; no additional head losses were allowed for canal curvature or check structures. Equivalent Manning's "n" for the design section is 0.0141.

As the canal delivers municipal and industrial water the year around, it was not completely unwatered prior to the tests. However, water was lowered to an approximate 15-inch depth in November 1961, and an inspection of the lining was made at that time. It was found that a large percentage of the concrete lining surface below the waterline was covered with a sediment-laden deposit, as shown in Figure H-5. This deposit was crust-like when
Figure H-1

WEBER BASIN PROJECT-UTAH
GATEWAY CANAL
GENERAL PLAN

EXPLANATION
- Turnout
- Cross drainage structure
- Bridge
- Sign
- Irrigation crossing
- Section marker found
- Check
A. Upstream view from farm bridge at Station 134+55 to illustrate old and recent applications of asphalt compound over cracks in the canal lining. P526-D-31274, November 1961.

B. Dry crust-like deposit on right side slope of canal lining upstream from Check No. 4 at Station 165+00. P526-D-31279, November 1961.

Figure H-5

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dry and tenaciously bonded to the concrete. Two other types of
clay-like coatings were noted on the surface of the lining at
various other locations. The latter coatings were not bonded as
tightly to the lining but tended to mask the original surface of
the concrete.

As very little "moss" growth had occurred in the canal
in the 2 years prior to the 1962 test, no chemical treatments were
made to retard such growths. Project development downstream from
the canal was not sufficient in 1962 to require a sustained design
flow, and no capacity problems had been encountered.

Water for the Gateway Canal is diverted directly from the
Weber River through the Stoddard Diversion Dam. As maximum river
flow occurs during the spring runoff period, the full capacity test
was scheduled at that time. Only a portion of the 724-cfs test
flow was required by downstream water users; the remaining flow was
returned to the river through the Gateway Powerplant and/or the
wasteway chute at the end of the canal.

Test Preparations and Measurements

Plans were made to establish steady flow conditions by
holding a constant diversion through the canal control gates at
Stoddard Diversion Dam. During a trial run on April 25, 1962,
water stage recorders at Station 129+60 and Station 285+20 showed
considerable fluctuations in canal water surfaces owing to accu­
mulations of trash on the louvers of the fish deflecting structure
upstream from the canal control gates. For the April 26 test,
three men were required to clean the fish deflector structure, and
one man adjusted the control gates continually during the measure­
ments. This procedure stabilized the canal discharge so that a
maximum stage change of only 0.05 foot was recorded during the
period required for measurements.

Discharge measurements were made with a Price Type A
current meter from a bridge in each test reach, and the multiple
current meter equipment was used on the bridge at Station 134+55,
Figure H-6. A slight breeze which varied in direction occurred
during the tests. Water was turbid because of the spring runoff
condition, but no sediment samples were obtained.

Water surface elevations were measured on both sides of
the canal using the differential manometer gages shown in Figure
H-7. Figure H-8 shows flow conditions at the downstream end of
the canal, where little or no freeboard existed. Water depths
decreased in an upstream direction, and 1.3 feet of freeboard
existed at Point A during the April 26 test run.
A. Multiple current meter equipment viewed from right side of canal at farm bridge at Station 134+55. P-526-D-32782, April 25, 1962.

B. Multiple current meter equipment viewed from left side of canal at farm bridge at Station 134+55. P-526-D-32783, April 25, 1962.

Figure H-6

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A. Entrance to Siphon No. 3 from Check No. 5. Discharge is 720 cfs. P-526-D-32790, April 25, 1962.

B. Water surface manometer at Station I upstream from Check No. 5 and Siphon No. 3. Looking downstream. P-526-D-32791, April 25, 1962.

Figure H-7
A. Looking upstream from trifurcation structure at downstream end of canal. Discharge is 720 cfs. P-526-D-32802, April 25, 1962.

B. Discharge of 720 cfs at junction of "old" and "new" lining upstream from Siphon No. 7. P-526-D-32803, April 25, 1962.

Figure H-8
Invert survey data were obtained in November 1962 following the tests and are shown in Figures H-2 and H-3. Plots of surveyed cross sections were made for the 16 measuring stations, and it was found that areas were within plus or minus 1 percent of the design section.

Analyses of Test Data

Figures H-2 and H-3 show test reaches, plots of water surface profiles, and computed Manning's "n" values. Figure 25 shows summaries of test data. Overall reaches have been broken down between individual stations in an effort to show increased resistance caused by horizontal curvature in canal alignment.

Horizontal curves in canal alignment are described in Tables H-1 and H-2.

Test discharges were computed using the 0.2 and 0.8 depth method from velocity traverses made with Price Type A current meters. The traverses were made from bridges at Stations 208+94 and 266+26. Two traverses were made with the multiple current meters from the bridge at Station 134+55. Results are shown below:

Multiple meter - \( Q = 723 \text{ cfs} \)  
738 cfs
Price Type A meter - - - 717 cfs  
729 cfs
Average value 724 cfs

Figure H-9 is a plot of Manning's "n" against curvature index. The lowest computed "n" value (0.0129) was found for the 353-foot straight Reach B\textsubscript{1}C. Other values for "n" from reaches with curves varied between 0.0132 and 0.0152. Although there appears to be an increase of "n" with curvature, no definite trend line is discernible from the plots in Figure H-9. The average value for the 11 segments shown in Figure 25 and H-9 was 0.0140.

Using an average "n" of 0.0140 for computing the canal friction the head losses across Check Structures No. 3 and 4 averaged 0.06 foot. This value is 16 percent of the velocity head in the unobstructed canal section.

Between Points I and J are located 370 feet of canal, the 17-foot-long Check Structure No. 5, and a 424-foot-long,
ROUGHNESS COEFFICIENT (MANNING's "n")

CURVATURE INDEX = \( \frac{5(\sum \Delta \theta)}{L_1} \)

GATEWAY CANAL
APRIL 1962, 724 cfs CAPACITY TEST

\( -0.0140 = \text{Av.value for the eleven tests shown.} \)

**FIGURE H-9**
### Table H-1
Reach 1
\( T = 30.7 \) feet

<table>
<thead>
<tr>
<th>Water surface measuring point</th>
<th>Curve No.</th>
<th>Deflection angle ( \Delta^* )</th>
<th>Curve radius ( R_1 ) (feet)</th>
<th>( R_1/T )</th>
<th>Curvature index</th>
<th>Reach length (feet)</th>
<th>Average &quot;n&quot; for 720-cfs test</th>
</tr>
</thead>
<tbody>
<tr>
<td>B</td>
<td>1</td>
<td>36</td>
<td>200</td>
<td>6.5</td>
<td>0.24</td>
<td>747</td>
<td>0.0146</td>
</tr>
<tr>
<td>B(_1)</td>
<td>Straight</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>353</td>
<td>0.0129</td>
</tr>
<tr>
<td>C</td>
<td>2</td>
<td>21</td>
<td>700</td>
<td>23.0</td>
<td>0.12</td>
<td>1,639</td>
<td>0.0136</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>19</td>
<td>500</td>
<td>16.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>D</td>
<td>4</td>
<td>61</td>
<td>150</td>
<td>4.9</td>
<td>0.52</td>
<td>1,150</td>
<td>0.0139</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>59</td>
<td>125</td>
<td>4.1</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>E</td>
<td></td>
<td>Average values for Reach BE</td>
<td>39</td>
<td>335</td>
<td>10.9</td>
<td>0.25</td>
<td>3,889</td>
</tr>
<tr>
<td>Reach BE</td>
<td>( \Sigma \Delta = 196^\circ )</td>
<td>An average of 6.8 curves per mile</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>F</td>
<td>6</td>
<td>32</td>
<td>500</td>
<td>16.0</td>
<td>0.20</td>
<td>1,433</td>
<td>0.0147</td>
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<tr>
<td></td>
<td>7</td>
<td>16</td>
<td>500</td>
<td>16.0</td>
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<tr>
<td></td>
<td>8</td>
<td>10</td>
<td>1,000</td>
<td>33.0</td>
<td></td>
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<tr>
<td>G</td>
<td>9</td>
<td>31</td>
<td>150</td>
<td>4.9</td>
<td>0.30</td>
<td>1,645</td>
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<tr>
<td></td>
<td>10</td>
<td>52</td>
<td>150</td>
<td>4.9</td>
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<td></td>
<td></td>
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<tr>
<td></td>
<td>11</td>
<td>17</td>
<td>500</td>
<td>16.0</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>H</td>
<td>12</td>
<td>16</td>
<td>300</td>
<td>9.8</td>
<td>0.18</td>
<td>1,091</td>
<td>0.0134</td>
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<tr>
<td></td>
<td>13</td>
<td>23</td>
<td>300</td>
<td>9.8</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>I</td>
<td></td>
<td>Average values for Reach FI</td>
<td>25</td>
<td>425</td>
<td>13.8</td>
<td>0.24</td>
<td>4,169</td>
</tr>
<tr>
<td>Reach IF</td>
<td>( \Sigma \Delta^* = 197^\circ )</td>
<td>An average of 10.1 curves per mile</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>J(^*)</td>
<td>14</td>
<td>18</td>
<td>500</td>
<td>16.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>15</td>
<td>29</td>
<td>300</td>
<td>9.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>16</td>
<td>13</td>
<td>400</td>
<td>13.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>17</td>
<td>16</td>
<td>600</td>
<td>19.5</td>
<td>0.24</td>
<td>3,107</td>
<td>0.0132</td>
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<tr>
<td></td>
<td>18</td>
<td>18</td>
<td>500</td>
<td>16.3</td>
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<td></td>
</tr>
<tr>
<td></td>
<td>19</td>
<td>24</td>
<td>200</td>
<td>6.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td>20</td>
<td>28</td>
<td>600</td>
<td>19.5</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>K</td>
<td></td>
<td>Average values for Reach JK</td>
<td>21</td>
<td>443</td>
<td>14.4</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Reach JK</td>
<td>( \Sigma \Delta^* = 146^\circ )</td>
<td>An average of 11.9 curves per mile</td>
<td></td>
<td></td>
<td></td>
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</tbody>
</table>

*Point J located on curve.*
<table>
<thead>
<tr>
<th>Water surface measuring point</th>
<th>Curve No.</th>
<th>Deflection angle $\Delta^\circ$</th>
<th>Curvature radius $R_1$ (feet)</th>
<th>$R_1/T$</th>
<th>Curvature index</th>
<th>Reach length (feet)</th>
<th>Average &quot;n&quot; for 720-cfs test</th>
</tr>
</thead>
<tbody>
<tr>
<td>K</td>
<td>21</td>
<td>55</td>
<td>200</td>
<td>6.5</td>
<td>0.78</td>
<td>760</td>
<td>0.0136</td>
</tr>
<tr>
<td></td>
<td>22</td>
<td>63</td>
<td>307</td>
<td>10.0</td>
<td></td>
<td></td>
<td></td>
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<tr>
<td>$K_1$</td>
<td>23</td>
<td>53</td>
<td>300</td>
<td>9.8</td>
<td>0.26</td>
<td>2,879</td>
<td>0.0141</td>
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<tr>
<td></td>
<td>24</td>
<td>25</td>
<td>300</td>
<td>9.8</td>
<td></td>
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<td></td>
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<td></td>
<td>25</td>
<td>14</td>
<td>300</td>
<td>9.8</td>
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</tr>
<tr>
<td></td>
<td>26</td>
<td>13</td>
<td>500</td>
<td>16.3</td>
<td></td>
<td></td>
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<tr>
<td>M</td>
<td>27</td>
<td>43</td>
<td>500</td>
<td>16.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>N</td>
<td>28</td>
<td>57</td>
<td>400</td>
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<td>0.46</td>
<td>623</td>
<td>0.0152</td>
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<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average values for 4,262-foot Reach $KN = 40$</td>
<td>351</td>
<td>11.4</td>
<td>0.38</td>
<td></td>
<td>4,262</td>
<td>0.0143</td>
<td></td>
</tr>
</tbody>
</table>

Reach $KN \quad \Sigma \Delta^\circ = 323^\circ \quad$ An average of 9.9 curves per mile.
132-inch-diameter precast concrete pipe in-line inverted siphon. (The 424-foot length includes siphon inlet and outlet transitions.) The measured difference in energy gradient in this reach was 0.96 foot. Design head loss allowed for the reach was 0.87 foot.

Summary and Discussion of Test Results

The April 26, 1962, 724-cfs flow test in the Gateway Canal demonstrated that design discharge could be carried safely. Water in this 700-cfs canal was level with or slightly above the top of the concrete lining at the downstream end, but normal freeboard existed at the upstream end. No aquatic growth was reported on the lining surfaces during this test and no chemical treatments were made to retard growths prior to this test.

Computed Manning's "n" values from 11 segments of the Canal are shown in Figure H-9. These values ranged from 0.0129 for a straight length to 0.0152 for a length with curves. Average value for the 11 segments was 0.0140. Computed values for all segments were within 9 percent of this value.

Head losses computed from measurements across two check structures average 0.06 foot.

The head loss computed from measurements across a 424-foot-long, 132-inch-diameter, precast concrete pipe in-line inverted siphon with transitions was 0.79 foot for the test flow of 724 cfs compared to a design allowance of 0.73 foot for a flow of 701 cfs. An escape structure, consisting of concrete steps in front of the inlet headwall, might account for the small excess head loss in this siphon.

Figure 25 presents more detailed information concerning prototype flow tests made during 1962 in the canal. This figure also shows computed values of hydraulic properties and design parameters.
Section I

CHARLES HANSEN CANAL

Colorado-Big Thompson Project, Colorado
Two concrete-lined reaches of the Charles Hansen Canal were chosen for prototype tests to provide hydraulic performance data from a very sinuous, relatively steep-slope canal. Design velocity for this canal was 9.98 feet per second for a flow of 1,500 cfs. The invert slope of 0.0013 was the steepest utilized during the current test program.

Figure I-1 shows the location of the canal and the two test reaches. Water surface profiles were documented at flows of 800 cfs and 1,290 cfs on July 31 and August 1, 1962, respectively. A preliminary test of the measurement procedures was held on July 30, 1962, to acquaint crews with equipment.

As the canal and in-line structures were designed for a flow of 1,500 cfs, an M2 backwater curve existed at siphon inlets during the tests. No check structures were built in this canal as only a few small turnouts exist. However, the Northern Colorado Water Conservancy District (NCWCD) which now operates the canal has installed triangular dams on the canal invert immediately downstream from some of the turnouts. These dams, from 4 to 14 inches high, back up the water and create additional head on turnouts during periods of low flow. No other obstructions and no in-line canal structures with piers in the flow prism existed within the test reaches.

Figures I-2 through I-5 show canal profile, alignment, and section details. Fourteen water surface profile measuring stations, designated alphabetically A through N, were located in the 6,013-foot Reach 1. Seven measuring stations designated O through U were located in the 1,706-foot Reach 2.

Concrete-lined reaches of this 5-mile-long canal were constructed in 1950 under Specifications No. 2855. The 4-inch-thick concrete lining was unreinforced except at locations specified by the contracting officer. It was placed by rail-mounted slip-form as shown in Figure I-9. Specifications requirements stated that "the finished concrete surface shall be equivalent in

*This canal was originally called the Poudre Supply Canal but was renamed in 1956.
Three equations in canal stationing are not shown on this sheet. These equations have been considered in computation of test reach lengths shown on this sheet.

See Figure 2-3 for description of symbols.
evenness, smoothness, and freedom from rock pockets and surface voids to that obtainable by the effective use of a long-handled steel trowel."

Figures I-2 and I-4 show design flow in the canal to be 1,500 cfs. Six inches of vertical freeboard from water surface to top of concrete lining was anticipated for this maximum flow condition. The 1,500 cfs included an allowance for flood inflow which was expected very infrequently. Examination of NCWCD records indicated a maximum flow of 1,320 cfs had been delivered in the 9-year period between 1954 and 1963; 1,450 cfs had been carried briefly in the clean canal in the spring of 1953. Kutter's formula with a roughness coefficient "n" of 0.014 was used in design of the canal. This value was assumed to provide adequate allowance for losses caused by horizontal curves. Equivalent Manning's "n" for the canal section is 0.0142. (Discharge computed using Manning's formula, "n" = 0.014, and a depth of 7.17 feet was 1,524 cfs -1.5 percent more than design flow.) Design freeboard on the concrete lining for the normal design flow of 1,300 cfs was 1.03 feet.

Transverse contraction grooves were provided in the concrete lining at approximate 12-foot intervals. Only three longitudinal grooves were used, one along the canal centerline on the invert, the other two near the midpoints of the sides. A considerable amount of longitudinal cracking had occurred at the intersections of the bottom and sides of the lining in years prior to the test. Asphalt mastic filler material used in contraction grooves generally protruded above the lining surface in 1962.

This canal is unwatered in November of each year at the end of the irrigation season and is not refilled until the following spring. It was therefore possible to examine and document the condition of the concrete lining before and after the tests. Photographs, Figures I-8 through I-10, were taken during the unwatered period before the 1962 test. It was noted that sides of the 12-year-old concrete lining were very clean with little evidence of erosion of surface fines. The photographs indicate the invert of Reach 2 to be clean, but evidence of algal growth was present on the invert of Reach 1. This condition was again noted on an inspection of the unwatered canal in November 1962 after the tests. The most prevalent growth was identified as Stigeoclonium, a genus of filamentous green algae.

A coating of Oscillatoria, a genus of filamentous blue-green algae, was noted on the flow surface of the stilling pool at the head end of the canal and on the inverts of siphon inlet transitions where flow velocities were high. Also found in minor
A. Flow conditions at 16-foot timber bridge (Station 50+46) during 1,300 cfs capacity test in October 1952. Water piled up in front of center pier and was above lip of concrete lining in lower left of picture. Pier, consisting of four 8- by 8-inch posts with sway bracing, was removed in 1953. P-245-704-6238, October 16, 1952.

B. Flow conditions at modified clear span bridge (Station 50+46) during 1,290 cfs capacity test in August 1962. Note reduction in turbulence resulting from removal of center pier. Approximately 0.6 foot of freeboard existed to top of lining just upstream from the bridge. P-245-D-34552, August 1, 1962.

Figure I-6

182
A. Flow conditions on left side of curved outlet transition below chute and siphon at Station 153+00 during 1952 capacity test. Note the large amount of entrained air in the 1,300 cfs flow. P245-704-6236, October 1952.

B. Flow conditions at outlet of chute and siphon structure (Station 151+90) during 1,290 cfs test of August 1962. Note 18-inch high precast concrete units which were added on top of the original concrete lining to contain the flow. P245-713-2205, August 1, 1962.

Figure I-7
amounts were black fly or buffalo gnats (cocoons) and Rotatoria a phylum of fresh water invertebrates commonly known as rotifers. Some drainage water seeps into the canal in Reach 1 during the unwatered period in the winter and may tend to keep the algae alive during the winter months.

Discussions with operating personnel provided no apparent reason why the algae grew in Reach 1 but not in Reach 2. Fred C. Scobey has, however, reported a similar situation that existed during 1931 tests on the Tiger Creek Flume in California.\textsuperscript{1} Scobey stated "The tests on Tiger Creek Conduit show that the influence of algae was greatest near the feeding reservoir and that but little effect was in evidence some 15 miles away. * * * It is suggested that decrease in flow due to algal growth always be provided for, unless experience in a given locality with the available water indicates that little effect is to be expected. More often than not the growth will appear."

Water in this canal is clear and little sediment was evident during 1962 tests. Operating personnel state that some "scouring action" occurs at the end of an irrigation season when Horsetooth Reservoir is low and sediment flushes through the outlet works into the canal.

Operating personnel stated that algal growth has also caused increased flow resistance on the nearby concrete-lined Pole Hill Canal and Charles Hansen Feeder Canal. Biweekly chemical treatments of 0.2 to 0.3 pound of copper sulfate crystals per cfs of discharge are used throughout the entire year on the 550-cfs Pole Hill Canal. Mechanical scraping of the lining has also been used annually at this location to restore capacity. Costs of the above procedures used to maintain capacity in both the one-half mile Pole Hill Canal and 8.5 miles of the Charles Hansen Feeder Canal have been $1,000 to $2,000 per year. In the Pole Hill Canal, the concrete lining height was increased 12 to 18 inches in tight spots after original construction to keep water within the lined section.

Test Preparations and Measurements

In 1952, 1 year after the canal was completed, a performance test was conducted on the Charles Hansen Canal which disclosed that a flow of 1,300 cfs overtopped concrete lining at the following locations:

\textsuperscript{1} Refers to numbered reference in the main body of this Technical Memorandum.
A. Closeup of invert in unwatered canal at approximate Station 22+50 showing algae which existed in October 1952 before reach was painted with antifouling paint. Photograph was taken 2 weeks after the 1,300 cfs capacity test. P245-704-6242, October 28, 1952.

B. Unwatered canal at approximate Station 190+00 in Reach No. 2. Note absence of algae. Very few cracks were found in the concrete lining. P245-D-30803, November 15, 1961.

Figure I-8

185
A. Rail-mounted slip-form and finishing jumbo used to place concrete lining in the Charles Hansen Canal. 245-704-1275, October 20, 1950.

B. Downstream view of unwatered concrete lining of Charles Hansen Canal just downstream from gaging station at Station 22+50. Lining in foreground has been painted with antifouling paint to prevent algal growth. Triangular sill on invert was added by NCWCD to increase the water depth and reduce turbulence at gaging station. P245-D-30798, November 15, 1961.

**Figure I-9**
A. View looking upstream at hydrographic bridge and recorder house at Station 22+50. Note dried curled algae on the lining invert. Lining has been extended vertically by the addition of precast concrete units near the gaging station. P245-D-30796, November 15, 1961.

B. View upstream from 16-foot timber bridge at Station 50+46 showing canal empty. Some algae was evident on invert. Canal is straight for approximately 530 feet upstream. P245-D-30799, November 11, 1961.

Figure I-10

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1. On the outside banks of several sharp curves in alinement between Stations 50+00 and 62+00 and between Stations 150+00 and 163+00. These curves had radii between 115 and 270 feet.

2. Immediately downstream from the outlet of a chute-siphon structure which entrained considerable air in the flow (Station 152+00).

3. Upstream from single-pier timber bridges located at Stations 50+46 and 160+43.

Photographs in Figures I-6 and I-7 show flow conditions which existed during the 1952 test. Flow depths were reported to be about 1.0 foot greater in Reach 1 than in Reach 2 during this test. After evaluation of data from this test, the concrete lining was raised 12 to 18 inches along nearly 2,100 feet of the more sinuous portions of canal above Reach 2. Figures I-7, I-9, and I-11 show the precast units which were cemented to the lip of the lining for this purpose. The single piers at two bridges (Stations 50+46 and 160+43) were removed and clear span supports were provided. Figure I-13 shows details of the original bridge with the four-post (8- by 8-inch) pier located at Station 50+46.

A 6,013-foot sinuous length (Reach 1) and a 1,706-foot length with somewhat less sinuosity (Reach 2) were selected for comprehensive testing in 1962. Horizontal curves in the canal alinement within these reaches are described in Tables I-1 and I-2.

Cross sections, invert lengths, and invert profiles were obtained by surveys prior to the 1962 tests. Invert survey data are shown in Figures I-3 and I-5. In Reach 1, the invert appeared to be about 0.15-foot high at the upstream end and on design grade at the downstream end. Of 40 surveyed points on the original invert at canal centerline, 37 were above design grade and 3 were below grade. The average deviation was 0.07 foot. In Reach 2, all 12 of the surveyed centerline elevations were above design grade, the average being 0.12 foot.

Triangular dams, about 13 inches high, on the invert were located at the following approximate stations:

Station 22+90, upstream from Test Reach 1 (Figure I-9)
Station 74+66, in Test Reach 1
Station 208+90, downstream from Test Reach 2

A smaller dam about 4-1/2 inches high occurred at approximate Station 51+24 in Test Reach 1.
Downstream view of crews making water surface profile measurements with differential manometer gages at Point I, Station 51+55. Note wave action on the vertical extensions of the canal lining on the outside of the curve in the background. P245-D-34553, August 1, 1962.

Figure I-11
Closeup view of differential manometer gage sensing disc. The disc was laid on the canal lining at least 18 inches below the water surface for each test measurement. P416-D-28554, July 1961.

Figure I-12
Table I-1

Reach 1

\( L = 6,013 \text{ feet} \quad T = 28.6 \)

<table>
<thead>
<tr>
<th>Water surface measuring point</th>
<th>Curve No.</th>
<th>Deflection angle ( \Delta^\circ )</th>
<th>Curve radius ( R_1 ) (ft)</th>
<th>( R_1 / T )</th>
<th>Curvature index</th>
<th>Reach length (ft)</th>
<th>Computed &quot;n&quot; for 1,290 cfs test</th>
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<tbody>
<tr>
<td>A</td>
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<td>45</td>
<td>358</td>
<td>12.5</td>
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<td>0.0153</td>
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<td>B</td>
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<td>15</td>
<td>573</td>
<td>20.0</td>
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<td>250</td>
<td>0.0162</td>
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<tr>
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<td>286</td>
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<td>346</td>
<td>0.0160</td>
</tr>
<tr>
<td>F</td>
<td>5</td>
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<td>573</td>
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<td>474</td>
<td>0.0145</td>
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<tr>
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<td>10.0</td>
<td>0.76</td>
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</table>

Average values

\( \Sigma \Delta = 570^\circ \quad \text{An average of 12.3 curves per mile} \quad 6,013 \)
Table I-2
Reach 2

$L = 1,706$ feet  \hspace{1cm}  $T = 28.6$ feet

<table>
<thead>
<tr>
<th>Water surface measuring point</th>
<th>Curve No.</th>
<th>Deflection angle $\Delta^\circ$</th>
<th>Curve radius $R_1$ (ft)</th>
<th>$R_1/T$</th>
<th>Curvature index</th>
<th>Reach length (ft)</th>
<th>Computed &quot;n&quot; for 1,290-cfs test</th>
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<tbody>
<tr>
<td>O</td>
<td>Straight</td>
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<td></td>
<td></td>
</tr>
<tr>
<td>P</td>
<td>1</td>
<td>48</td>
<td>164</td>
<td>5.7</td>
<td>1.08</td>
<td>369</td>
<td>0.0131</td>
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<tr>
<td>Q</td>
<td>2</td>
<td>22</td>
<td>573</td>
<td>20.0</td>
<td>0.33</td>
<td>334</td>
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<td>R</td>
<td>3</td>
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<tr>
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<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>T</td>
<td>4</td>
<td>9</td>
<td>1,146</td>
<td>40.1</td>
<td>0.15</td>
<td>293</td>
<td>0.0149 Ml backwater curve</td>
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<tr>
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<td></td>
<td></td>
<td></td>
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<td>0.0118</td>
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Average Values
Reach OU  

<table>
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<th>Curve No.</th>
<th>Deflection angle $\Delta^\circ$</th>
<th>Curve radius $R_1$ (ft)</th>
<th>$R_1/T$</th>
<th>Curvature index</th>
<th>Reach length (ft)</th>
<th>Computed &quot;n&quot; for 1,290-cfs test</th>
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<tr>
<td>25</td>
<td>614</td>
<td>21.4</td>
<td>0.29</td>
<td>1,706</td>
<td>0.0134</td>
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</table>

Reach OU  
$\Sigma\Delta^\circ = 100$  

An average of 12.4 curves per mile.
Surveyed cross sections obtained at 10 measuring stations were plotted. After invert grades were adjusted, all areas were found to be within 1 percent of the design value.

Since algal growth was known to be present the canal was treated with copper sulfate prior to test runs as listed below:

1. Tuesday, July 24, \( Q = 1,230 \text{ cfs} \) (treatment 0.31 pound per cfs injected in 55 minutes).

2. Saturday, July 28, \( Q = 910 \text{ cfs} \) (treatment 0.30 pound per cfs injected in 40 minutes).

Because of the fast velocity and water depth, it was not possible to determine the extent or secure samples of algal growth on the lining immediately before the test. However, flow was decreased to 280 cfs on the day following the 1,290 cfs test and water depth was reduced to about 3 feet. At this time green algae could be seen on the invert of Reach 1. Visible filaments did not appear to extend into the water more than 1 or 2 inches.

Steady flow conditions were established by holding releases constant from Horsetooth Reservoir at the head end of the canal. Two 72-inch hollow jet valves in the outlet works were set for a total discharge of 800 cfs at 6 a.m. on July 30 and left unchanged until 6 a.m. on August 1. The flow was then increased to 1,290 cfs, and this valve setting was held until 3:30 p.m. of that day.

Steady canal flows were verified by traces of water surface elevation on water level recorders in the gaging station shelter houses at Stations 22+50, 134+00, and 284+00. Traces of the water surface were remarkably steady, showing less than 0.02-foot variation during the test measurement periods.

Discharge measurements were made during each day’s test run from the gaging station cableway in the earth section at Station 134+00 between Test Reaches 1 and 2. A current meter traverse was made at the Parshall flume at the end of the canal on July 31. Price Type A meters were used and 0.2 and 0.8 depth procedure was used to make velocity traverses. Discharges were also computed from velocity traverses obtained with multiple current meter equipment located on the bridge at Station 50+46 (Figure I-6). Turnout flows from test reaches were less than 2 cfs and no drainage inflow was noted. Estimated seepage and evaporation losses from 3-1/2 miles of the canal amounted to less than 2 cfs.
Winds varied considerably in direction and velocity during the tests, but were not considered to be a significant element of flow resistance for this size of channel with the high velocity flow encountered.

Figure I-11 shows crews using the differential manometer gages to measure water surface profiles. Five sets of readings, each consisting of a simultaneous reading on each side of the canal, were obtained for the July 31 and August 1 tests. Measurements in Reach 1 or Reach 2 were completed between 9:00 a.m. and 3:30 p.m. on the days of the tests. Time required for measurements at one station varied from 3 to 14 minutes and averaged about 5 minutes after crews became accustomed to the equipment. The fast water velocity caused waves which resulted in fluctuations in the height of water columns in the manometer gages. These fluctuations were as much as 0.2 foot at stations in Reach 1 where 0.8-foot-high waves were encountered.

In Reach 2 there was less wave action at measuring stations. Manometer fluctuations there did not exceed 0.08 foot, and 0.4-foot-high waves occurred. Water was drawn from the canal to the manometer tube through a 12-inch-diameter aluminum pressure-sensing disk shown in Figure I-12. Water entered these beveled disks through a single 1/8 or 1/16-inch-diameter hole in the center. The purpose of the single hole was to damp wave action and minimize velocity effects. Considerable effort was required to hold the disk against the side of the lining in the high velocity water.

Several water surface measuring points were located at the ends of horizontal curves to measure the expected "ride-up" of the water on the outside bank. (See Figures I-7 and I-11) Test data showed the water surface was higher on the anticipated side of the canal at 13 of the 21 measuring stations. A maximum differential water surface of 0.40 foot occurred at Station 80+75 during the 1,290-cfs run. This station is the P.T. of a tight curve (\( \Delta = 73^\circ \) left, radius = 143 feet) and water was higher, as expected, on the right bank. Lining repairs had been required on this curve and visible flow surfaces appeared somewhat rougher than on adjacent panels. This roughened condition may have been created when the lining machine traversed the curve. The invert profile also shows a slight hump at this location and considerable longitudinal cracking had occurred.

The differential water surface between banks at all other measuring points was less than 0.20 foot. However, USGS personnel who were measuring velocity distributions on curves for another study, stated that they had measured water surfaces across the canal on lines normal to the canal centerline at various
CHARLES HANSEN CANAL
1290 cfs TEST OF AUG 1, 1962

CURVATURE INDEX = \( \frac{5(\Sigma \Delta \theta)}{L_1} \)

SYMBOLS
- Reach 1
- Reach 2

Av. for eleven segments of Reach 1
Av. for six segments of Reach 2

FIGURE I-14
The amount of tilt in water surface which occurred as water flowed around bends was close to the theoretical amount given by the formula \( \Delta y = \frac{V^2 T}{gR} \) in the majority of locations.

However, in a few cases the water surface tilted the opposite way due to carryover effects of velocity disturbances created by upstream bends. Observations of flow patterns on the surface indicated that effects of canal curvature on surface velocities extended for at least 600 feet downstream from the P.T. of most curves. Since few tangent reaches longer than 600 feet exist, it is improbable that velocity distributions typical of an infinitely long straight reach could be obtained in this canal.

Owing to manpower and fund limitations, additional tests were not made throughout the summer to document the effect of increased algal growth on channel resistance. Project personnel stated that algal growth in this canal does increase during each operating season and that it causes increased resistance to flow.

Figures 26 and 27 provide more detailed information concerning prototype flow tests made during 1962 in this canal. These figures also show computed values of hydraulic properties and design parameters.
locations and found one case where the water surface in the mid-
portion of the stream was about 0.30 foot higher than the mean
value of the elevations at the sides of the canal.

Analyses of Test Data

Figures I-3 and I-5 show test reaches, plots of water
surface profiles and computed Manning's "n" values. It will be
noted that backwater effects have been caused in both reaches by
the triangular invert dams in the canal immediately downstream
from turnouts. Slight Ml backwater curves exist immediately
upstream from these dams, but the effects of these restrictions
did not extend for appreciable distances upstream. Water depths
in Reach 1 averaged about 0.3 foot more than in Reach 2.

Figures 26 and 27 show summaries of test data. Overall
reaches have been broken down into segments between individual
stations in an effort to isolate increased resistance caused by
horizontal curvature in canal alinement.

As two of the reaches (G-H and K-L) were considered to
be too short to give reliable results, they were combined with
adjacent reaches for analyses. Estimated losses caused by the
invert dams in Reaches G-I and J-K were subtracted from measured
losses before values of "n" and "f" were computed. Another invert
dam caused obvious backwater effects in Reach T-U. The lower than
average computed "n" values for this reach may have resulted from
inadequate consideration of these effects.

Some of the measuring points were located at or near
the downstream ends of curves. A portion of the head loss caused
by the curves no doubt occurred in the sections downstream of the
measuring points. More realistic results would probably be
obtained by omitting these measuring stations and analyzing the
curve and downstream section as one reach.

A local area of high invert and cracked concrete lining
occurred near Point M. These circumstances probably caused dis-
torted values of computed "n" in this area.

The effect of omitting Points B, E, M, Q, and S and
analyzing overall Reaches A-C, D-F, etc., was investigated.
Values of "n" computed for overall reaches for a flow of 1,290 cfs
were:
Reach A-C  \( n = 0.0156 \) Curvature Index = 0.44
Reach D-F  \( n = 0.0155 \) Curvature Index = 0.72
Reach K-N  \( n = 0.0156 \) Curvature Index = 0.65
Reach P-R  \( n = 0.0143 \) Curvature Index = 0.63
Reach R-T  \( n = 0.0136 \) Curvature Index = 0.22

Figure I-14 shows plots of Manning's "n" against curvature index. Values plotted on the vertical line at the left for a curvature index of zero are from straight reaches. For the computed reaches, a maximum curvature index of 1.32 existed between Points K and M of Reach 1. (Point M was at the P.T. of a 183-foot-long horizontal curve which has a radius of 143 feet and deflection angle, \( \Delta \), of 73° left.)

Values of "n" computed for reaches from which Points B, E, M, Q, and S were omitted are plotted against curvature index in Figure I-15. Straight trend lines were drawn by eye on this figure to show the increase in hydraulic resistance caused by channel curvature. Beginning with the average value for straight reaches, both trend lines rise as channel curvature increases. The line for Reach 1 shows "n" to increase generally about 0.001 (7 percent) as the curvature index increases from 0 to 1.0. The trend indicated in Reach 1 agrees substantially with results obtained by Scobey in tests on a concrete flume with comparable velocity and hydraulic radius.

The trend line drawn for Reach 2 shows a greater rate of increase in "n" with increase in curvature index. Only four points were available from Reach 2 for plotting in Figure I-15. Because of the small number of points and the unexplained low "n" value of 0.0118 from Segment T-U, the trend line cannot be considered to be conclusive.

Results from the 800-cfs tests on both Reaches 1 and 2 are comparable to the results shown in Figures I-14 and I-15 for flow of 1,290 cfs. Also, a satisfactory correlation of results was obtained by assuming that head losses due to curves amounted to approximately 0.001 \((2\Delta^\circ)h_v\).

**Summary and Discussion of Test Results**

Computed Manning's "n" values for the 1,290-cfs test are plotted in Figures I-14 and I-15. Values and trends shown for the 800-cfs test were very similar. A brief summary of "n" values from both tests as computed combining curves with downstream reaches is given below.
CHARLES HANSEN CANAL
1290 cfs TEST OF AUG 1, 1962

CURVATURE INDEX = \frac{5(\Sigma \Delta \theta)}{L_1}

SYMBOLS

- Reach 1
- Reach 2

Av. for eleven segments of Reach 1
Av. for six segments of Reach 2

FIGURE I-14
The amount of tilt in water surface which occurred as water flowed around bends was close to the theoretical amount given by the formula \( \Delta y = \frac{V^2R}{g} \) in the majority of locations.

However, in a few cases the water surface tilted the opposite way due to carryover effects of velocity disturbances created by upstream bends. Observations of flow patterns on the surface indicated that effects of canal curvature on surface velocities extended for at least 600 feet downstream from the P.T. of most curves. Since few tangent reaches longer than 600 feet exist, it is improbable that velocity distributions typical of an infinitely long straight reach could be obtained in this canal.

Owing to manpower and fund limitations, additional tests were not made throughout the summer to document the effect of increased algal growth on channel resistance. Project personnel stated that algal growth in this canal does increase during each operating season and that it causes increased resistance to flow.

Figures 26 and 27 provide more detailed information concerning prototype flow tests made during 1962 in this canal. These figures also show computed values of hydraulic properties and design parameters.
Between 1957 and 1962, the Bureau of Reclamation made tests to determine flow capacities and resistance coefficients in 9 large concrete-lined irrigation canals. Tests were made on some 170 miles of trapezoidal-shaped canals having flows which varied from 555 cfs to 6,820 cfs. Design discharges for test canals varied from 700 to 13,200 cfs. Other design hydraulic properties included invert slopes which ranged from 0.0013 to 0.00005, bottom widths from 8 to 50 feet, hydraulic radii from 4 to 14 feet, and velocities from 5 to 12 fps. All concrete linings, except one, were placed by rail-mounted traveling slip forms and were from 7 to 25 years old at the time of tests. Head loss measurements were made across piers and inverted siphons. Test data were analyzed in terms of \( n \) values for Manning’s formula and on Reynolds’ Number - friction factor plots. Resistance coefficients varied with the amount of aquatic growths, canal alignment, and canal size. Manning’s \( n \) generally varied from 0.013 to 0.016 for the smaller canals and from 0.016 to 0.019 for the larger canals. Aquatic growths were found in varying amounts on lining surfaces of all canals and caused seasonal variation in flow resistance. Biweekly copper sulfate treatments retarded the most prevalent growth, filamentous algae. A hydraulic design procedure for concrete-lined canals is outlined. Design procedures of other agencies and recent literature on flow in rigid boundary channels are summarized and reviewed. An appendix contains detailed descriptions of tests and operating experiences in each canal.

**ABSTRACT**

Between 1957 and 1962, the Bureau of Reclamation made tests to determine flow capacities and resistance coefficients in 9 large concrete-lined irrigation canals. Tests were made on some 170 miles of trapezoidal-shaped canals having flows which varied from 555 cfs to 6,820 cfs. Design discharges for test canals varied from 700 to 13,200 cfs. Other design hydraulic properties included invert slopes which ranged from 0.0013 to 0.00005, bottom widths from 8 to 50 feet, hydraulic radii from 4 to 14 feet, and velocities from 5 to 12 fps. All concrete linings, except one, were placed by rail-mounted traveling slip forms and were from 7 to 25 years old at the time of tests. Head loss measurements were made across piers and inverted siphons. Test data were analyzed in terms of \( n \) values for Manning’s formula and on Reynolds’ Number - friction factor plots. Resistance coefficients varied with the amount of aquatic growths, canal alignment, and canal size. Manning’s \( n \) generally varied from 0.013 to 0.016 for the smaller canals and from 0.016 to 0.019 for the larger canals. Aquatic growths were found in varying amounts on lining surfaces of all canals and caused seasonal variation in flow resistance. Biweekly copper sulfate treatments retarded the most prevalent growth, filamentous algae. A hydraulic design procedure for concrete-lined canals is outlined. Design procedures of other agencies and recent literature on flow in rigid boundary channels are summarized and reviewed. An appendix contains detailed descriptions of tests and operating experiences in each canal.
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### Hydraulic Friction Test Data

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### Hydraulic Properties of Channel Test Data

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### Notes

- **Roughness Coefficient (k)**: Corresponds to worst conditions anticipated.
- **Manning's n**: Average of 7 tests.
- **Kuttler's k**: Fine finishing flat invert lining surfaces.

**Canales Mentioned**
- Free board on example:
  - **Medio Adige Canals (Bussolengo)**
  - **Kuhter's or Manning's k**
- Linear roughness characteristics or "equivalent sand grain roughness" values are given for Pavlovsky formula.

**Formulas**
- **Kutter's or Manning's k**: Used for design of concrete-lined channels.
- **Kutter's or Manning's k**: Corresponds to worst conditions anticipated.
- **Manning's n**: Average of 7 tests.
- **Kutter's or Manning's k**: Corresponds to worst conditions anticipated.
- **Manning's n**: Corresponds to worst conditions anticipated.

**Examples**
- **Medio Adige Canals (Bussolengo)**: Manning's n = 0.016 recommended for an average "running" concrete-lined canal.
- **Kutter's or Manning's k**: Corresponds to worst conditions anticipated.
- **Manning's n**: Average of 7 tests.
- **Kutter's or Manning's k**: Corresponds to worst conditions anticipated.

**Figure 3**