Dear Author:

Enclosed is proof for the portions of your paper, currently in production, that contains mathematics and tables. The text portions of your paper are being proofread by the printer.

THIS PROOF MUST BE MAILED BACK TO THE PRINTER WITHIN 48 HOURS OF RECEIPT.

1. Please check this material for corrections and typographic errors only. If there are errors, use a red pencil to show what should be changed.

2. No additions or changes (other than the correction of errors) can be made. If extensive changes or alterations are made on the proof, we may be obliged to delay publication and to bill you for the costs involved in making the Author's Alterations.

Your cooperation in assuring the prompt publication of your paper is appreciated.

RETURN PROOF AND MANUSCRIPT TO:

EDWARDS BROTHERS, INC.
2500 South State Street
Ann Arbor, Michigan  48104

Attention:  Mrs. Mary Ichesco

Sincerely,

Richard R. Torrens
Editor of Technical Publications

Encl.
Handling of proof copy received August 23, 1974 for the paper on "Colorado River Flow Management."

Corrections made on the proof received August 23, 1974.

Greek Ψ for velocity in list of notation and text to be changed to a lower case v to agree with the usage on figure 2. "Wave profile".

Formula 8 (their number). They omitted the integrand \( e^{-u^2} \) du.

Table 1. They omitted the minus sign in \( e^{-\frac{x^2}{4\alpha t}} \) and \( \frac{e^{-\frac{x^2}{4\alpha t}}}{\sqrt{T\alpha t}} \) in columns 11 and 12 (Their numbering).

Formula 15, (Their numbering). The quantity \( \sqrt{T\alpha t} \) is to be raised to the same level with \( S \) so that the denominator reads \( 2S\sqrt{T\alpha t} \).

The corrected proof copy was returned to them on August 24, 1974. The sheets of the editing copy received with the proof copy were also returned.

Robert E. Glover

1936 South Lincoln Street
Denver, Colorado
August 24, 1974.
Mr. Chester A. Nelson
U.S. Department of the Interior
Bureau of Reclamation
Engineering and Research Center
P.O. Box 25007 - Bld'g 67
Denver Federal Center
Denver, Colorado - 80225

Dear Mr. Nelson:

The edited photocopy of the paper on Colorado River Flow Management, by Glover, Nelson and Sanders arrived last Saturday. The ASCE letter of transmittal signed by Torrens is also enclosed. I have a photocopy of this for my file. They sent a copy of the "Author's Guide..." which I am also sending along.

Their usage such as A = area in the list of notation does not win my enthusiasm. I planned to ask them to restore the original wording, but finally conceded for the reason that to object might delay publication another three years.

For the same reason I have agreed to let them use their "...scientific and engineering terms..." as listed on pages 40, 41 and 42 of the "Author's Guide" even though they are obviously inferior to the comprehensive system we used. I have to laugh a little when I note that the Editor had to use our system to designate metric equivalents, as on pages 6, 22 and other places. I think he will have to concede also in table 1 where he has no approved units for the quantities at the bottom of the page.

The twenty-five complimentary copies will be enough for my needs and I am asking them to set aside the same number for you. If you want more you can order them later.

Your thought on these corrections would have been of much help to me but since you will be away from Denver until next Monday and I will be away for a week starting next Sunday with his deadline on August 16, 1974, I have returned the corrected copy to the ASCE. A photocopy of the corrected photocopy is enclosed.

Sincerely yours,

Robert E. Glover
Mr. Richard R. Torrens
Editor of Technical Publications
345 East 47th Street
New York, N.Y. — 10017

Dear Mr. Torrens:

Please refer to your letter of July 31, 1974 forwarding to me an edited photocopy of the proposed paper on "Colorado River Flow Management" by R.E. Glover, C.A. Nelson and J.I. Sanders. This copy is returned herewith with corrections made in red, as requested. I think you will agree that the requested corrections should be made.

The 25 complimentary reprints should fill the needs of the Authors.

Because Mr. Nelson will be out of the City all this week and I will be away all of next week, beginning Sunday, I have again acted for both of us.

Sincerely yours,

[Signature]

Robert E. Glover

cc:
C.A. Nelson
With Photocopy of the corrected photocopy and original of the letter of July 31, 1974.
July 31, 1974

File: 16-2-4.G

Mr. Robert E. Glover
1936 South Lincoln Street
Denver, Colorado  80210

Dear Mr. Glover:

Subject:  "Colorado River-Flow Management"

Enclosed is an edited photocopy of your paper for your approval before publication in the November Journal of the Hydraulics Division. Please examine it carefully, make any necessary corrections directly thereon in red, and return it without recopying. Also enclosed is a copy of the "Authors' Guide to the Publications of ASCE" which outlines ASCE's requirements for Proceedings manuscripts and explains the purpose of our editing.

Also enclosed are photocopies of the redrawn figures. Please make any corrections thereon and return for the necessary additional drafting work.

You will receive from the printer proof of the portions of your paper containing mathematics and tables. This should be checked and returned directly to the printer.

Please note that our editing is intended to improve the presentation. To include any corrections in your paper, it is necessary that we be notified before August 16, 1974. Your approval will be assumed if no communication is received from you prior to this date.

Approximately six weeks after publication of the Journal you will receive 25 complimentary reprints of your paper. Prepaid orders can be placed at a cost of $75 for the first 100 copies and $15 for each additional hundred. Return the enclosed order card with your remittance. If a purchase order is required, it must accompany the order card.

Very truly yours,

Richard R. Torrens
Editor of Technical Publications

RRT: cp
Enc.
1936 South Lincoln Street
Denver, Colorado - 80210
June 28, 1974

Mr. Richard E. Torrens
Editor of Technical Publications
American Society of Civil Engineers
345 East 47th Street
New York, N.Y. - 10017

Dear Mr. Torrens:

Since Mr. Sanders is now deceased and Mr. Nelson has had to go
to the hospital I have been asked to reply to your letter, of June 13,
1974 to Mr. Nelson, on the subject: "Colorado Riverflow Management"

A sheet on "Key words" is with the material left with me. The
original keywords have been replaced with descriptors gleaned from
the E.J.C. Thesaurus.

The Reviewer's comments have been followed in revising the text.
His comments were especially helpful for the material on pages 23
and 24. My changes are shown in green.

A sheet carrying the caption "Necessary adjustments" asks that
titles to all figures should be listed in numerical order on a separate
sheet. This has been done. The item "Figures should be drawn in
black ink...." is unclear. The original drawings, in black ink,
were transmitted with the original letter of August 11, 1971. At
least this is the recollection of Mr. Nelson, who is now convalescing
at home. He says these drawings are not in his file. The size
should be satisfactory to meet the ASCE requirements. If you can not
find the originals please let us know and we will have them redrawn.
Metric equivalents have been included in the text.

Two tabulations, originally arranged as Figures, have been
changed to tables. A summary (Abstract) of less than 40 words is
included with the corrected single copy of the paper which is trans-
mittted herewith.

The Reviewer's comments, the sheet captioned "Necessary adjust-
ments" and the sheet concerning "Keywords" are returned herewith.

If anything has been missed please advise.

Sincerely yours,

Robert E. Glover

cc: C.A. Nelson
with corrected copy
of the paper.
Mr. Chester A. Nelson  
U. S. Dept. of the Interior  
Bureau of Reclamation  
Engineering & Research Center  
P. O. Box 25007 - Bldg. 67  
Denver Federal Center  
Denver, Colorado  80225

Dear Mr. Nelson:

Subject:  "Colorado Riverflow Management"
   By R. E. Glover, C. A. Nelson, and J. I. Sanders

   It is a pleasure to advise you that your paper has been tentatively approved for publication in the Journal of the Hydraulics Division. Prior to processing it further, however, we ask that you consider the suggestions offered by the reviewer in the enclosed abstract of his comments and in the enclosed copy of the paper (which should be returned).

   On the accompanying sheet entitled "Necessary Adjustments" we have indicated those items that should be modified prior to editing and publication. The enclosed "Authors' Guide to the Publications of ASCE" explains how these changes should be made and the reasons for our requests.

   You can make the necessary adjustments directly on the enclosed manuscript. If the changes to a particular page are such that it will be difficult for a nontechnical typist to follow, would you please retype that page?

   If you cannot complete the necessary work by July 5, 1974 please advise us as to the additional time needed.

Sincerely,

Richard R. Torrens  
Editor of Technical Publications

RRT:mcn  
Encl.
NECESSARY ADJUSTMENTS

General Instructions

The title should have a maximum length of 50 characters, including spaces.

Centered and side headings should be used to divide paper into logical parts.

A set of Conclusions should be provided to bring the paper to a logical close.

Figures

Titles to all figures should be listed in numerical order on a separate sheet, and each figure should be clearly identified.

Figures should be drawn in black ink, at a size that, with a 50% reduction, will have a published width in the Journals of from 3 in. to 4-1/2 in. The lettering must be legible at the reduced size.

Extensive notes and descriptions should be removed from the figures.

Photographs should be submitted as glossy prints.

A copy of a recent paper is enclosed to show the quality of figures we would like to receive.

Tables should be typed on one side of 8-1/2 in. by 11 in. paper; an original and one duplicate are required.

Mathematics

All mathematics must be typewritten. Equations should be completely typed; handwritten symbols cannot be accepted.

Mathematical symbols (such as Greek letters) must be properly identified for the printer.

The letter symbols should be defined where they first appear and should be listed alphabetically in an Appendix-Notation.

All centered equations should be numbered in sequence.

Metrication

Dual units (U.S. Customary followed by SI) are to be given as described on page 43 of the enclosed Authors' Guide. When U.S. Customary Units are used in text and table and figure captions they are to be followed by the equivalent numerical value in SI units placed in parentheses. In headings of table columns the statement of SI equivalents of U.S. Customary units should appear in parentheses following the U.S. Units. In the body of the table the value of equivalent SI units should be placed in parentheses below the U.S. numerical quantity; the same is true in the body of figures. Alternatively, SI conversion factors can be placed as a footnote in tables or as a note in parentheses in figure captions.

References and Employment Footnotes

Separate footnotes should be used for the present-employment description of the authors.

All bibliographic references should be listed in an Appendix at the end of the paper. The list should be alphabetical, by last name of senior author, and numbered. Where reference is made in text to an author's work, the appropriate number should appear in parentheses.

Abstracts

A 40-word abstract, for use in CIVIL ENGINEERING, is required.

To advance the information-retrieval program described on page 21 of the "Authors' Guide," we need an informative abstract with a maximum length of 175 words and a list of key words.
Titles of figures

Figure 1. Colorado River Flow Management.
Figure 2. Wave Profile.
Figure 3. Effect on Downstream Stations of a Unit Flow Increase at Parker Dam.
Figure 4. Prediction Made 3-25-66 For Taylor's Ferry.

EG June 28, 1974
Titles of figures

Figure 1. Colorado River Flow Management.
Figure 2. Wave Profile.
Figure 3. Effect on Downstream Stations of a Unit Flow Increase at Parker Dam.
Figure 4. Prediction Made 3-25-66 For Taylor's Ferry.
Title/Author: "Colorado Riverflow Management" by Glover/Nelson/Sanders

Date to Division: 6/13/73 Division: HY (Flood Control) ind. sub.

Note to Reviewer: Please keep all of your comments anonymous, to permit direct copying.

☐ This paper should be published because:

☐ This paper should be declined because:

(use additional sheets if necessary)

☐ Prior to publication the author should consider making the following adjustments:

If the author were to make the following adjustments, the paper might be found acceptable when re-reviewed:

make first few Tables agree with pp. 13-14 in Author's guides, column headings, column numbers.

(use additional sheets if necessary)

☐ The author should be provided with a copy of the review comments, which have been written in an anonymous form.

☐ Prior to publication the paper should be returned to the reviewers to be certain that the author has made all of the requested changes.

The attached sheets may be used by reviewers as a guide to their review and may be used to make comments about specific items in the paper.
SUGGESTIONS on the following items will help the author and the editors in the presentation of the paper. (Use additional sheets to amplify the remarks.)

20. Needs to include references to the following works:

21. Should correct the following inaccuracies:

   Fig 2/p.20

22. Would be better organized if the author would use col headings in discussing table (from p.23)

23. Should include the following key-words

24. Would be improved if the information-retrieval abstract were modified as follows

25. Should have the following material eliminated or condensed

26. Would be improved if the following material were added or elaborated upon

27. Is more accurately entitled

NOTE: Please note that reviewers’ names are held in strictest confidence.
KEYWORDS

ASCE, and many of the member societies of EJC, asks that authors furnish keywords with their papers. When properly selected, these keywords characterize all of the information in the paper and greatly facilitate its retrieval. Another purpose of the keywords is to provide selective dissemination of information to individuals by matching of keywords of reader interest profiles with the keyword indexes of the paper. Keyword indexes can be used in automated information retrieval systems as well as in an individual's manual filing system.

A randomly chosen set of general terms is not a useful keyword index. Rather, each keyword should be carefully chosen to represent a significant item of information in a paper or report.

Suggestions for selecting keywords are:

1. Use the EJC Thesaurus to select the keywords (descriptors).
2. A way of approaching the task of selecting keywords is: If you were looking for your paper in a subject index, which entries would you consult?
3. Consider the title as a prime source for keywords.
4. Next consider the abstract. An informative abstract will generally contain most of the concepts that should be indexed.
5. Then scan the complete paper to ensure that all concepts are covered by index terms.
6. The keywords selected for any paper need not be unrelated nor of equal specificity. Redundant and both broad and narrow terms can be used to cover all of the concepts in a document. A paper of average length should be indexed with from 15 to 30 terms. A good objective for "deep indexing" is about 20 terms, although many users will find that about five terms are sufficient for retrieval of the document in their own systems.
7. After the indexing terms have been selected, they should be reviewed and the most important, specific terms should be underlined. This serves to weight the terms and is useful in retrieval, in matching for selective dissemination of information, and in the preparation of cumulative indexing papers published in the various journals.
8. If your paper contains a concept for which the thesaurus does not have an approved term, you may suggest your own keyword.
9. Other terms such as geographical locations, project names, and names of persons or procedures which may be too specific to qualify as concept-oriented descriptors may be used as keywords.
Manager, Technical Publications
American Society of Civil Engineers
345 East 47th Street
New York, NY 10017

Dear Sir:

The paper "Colorado Riverflow Management" is herewith submitted with an abstract for consideration for publication in the Journal of the Hydraulics Division. The abstract and paper have been prepared according to the information in Volume 96, No. IR4, December 1970, inside back cover.

As requested in your letter dated September 15, 1971, we have reduced the length of this paper. It is felt that any further reduction in length would seriously lower the quality of the paper. Additional information will be furnished as requested.

Very truly yours,

[Signature]

C. A. Nelson

Enclosures

Copy to: Mr. Robert E. Glover
1936 South Lincoln Street
Denver, CO 80210
COLORADO RIVERFLOW MANAGEMENT

KEY WORDS: Rivers; Unsteady flow; Transient flow; Flood routing.

ABSTRACT: A method of computing transient flow changes in a river is described. The method is applied to the 147 mile (237 Km) reach of the Colorado River between Parker and Imperial dams.

REG June 18, 1976
COLORADO RIVERFLOW MANAGEMENT

KEY WORDS: river hydraulics; stream flow; flow transients; propagation of flow changes.

ABSTRACT: A method of computing transient flow changes in a flowing stream is described. The results of more than five years of experience in the use of this method for estimating flows at the gaging stations of the 147 mile Parker-Imperial reach of the Colorado River are summarized. An account is also made of digital computer applications which enable tabulations of the expected flows at the gaging stations for each hour of the ensuing 24 hours to be made and transmitted to the Boulder City Office of USBR in time to be available at the beginning of work each day. Tabulations covering 72 hours are prepared for weekend operations.

Note: This is the list of key words and the abstract sent originally. In the revision of June 28, 1974 shown in green in the copy, the key words and abstract dated June 28, 1974 were substituted.
COLORADO RIVERFLOW MANAGEMENT

by

Robert E. Glover, 1/ F. ASCE
Chester A. Nelson, 2/ M. ASCE
John I. Sanders 3/

Introduction

The Bureau of Reclamation of the Department of the Interior administers water deliveries from the Colorado River of the Southwest with its system of powerplants and storage reservoirs. This paper describes the needs and the procedures developed for improving the monitoring of transient flow changes of the Colorado River from Parker Dam to Imperial Dam. This reach of the river is shown on Figure 1.

1. Need for Close Control of the River

   a. Authority and Regulations. All operations of the Colorado River below Hoover Dam are made in accordance with the Colorado

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1/ General Engineer, U.S. Bureau of Reclamation, Retired.
2/ Research Structural Engineer, U.S. Bureau of Reclamation, Denver, Colorado.
3/ Hydraulic Engineer, U.S. Bureau of Reclamation, Deceased.
River Compact, the Boulder Canyon Project Act and its associated power and water delivery contracts, the Mexican Water Treaty, and the Decree of the Supreme Court of the United States in Arizona v. California, dated March 9, 1964. The requirements stipulated by these regulatory documents include operation for flood control, irrigation and domestic water release, and releases for power generation.

b. Water Conservation Necessary. Each year the spring runoff from melting of the winter's snow accumulation on the west slope of the Rocky Mountains is stored in Colorado River reservoirs. This is a part of the flood control operations of the river. However, there have been very few releases strictly for flood control during the past several years because of the drought period in the 1950's and a recurrence of below-normal runoff again during part of the last decade. The construction of Glen Canyon Dam on the Colorado River has vastly decreased the probability of having to make flood control releases that would waste water to the Gulf of California in the near future.

Early in the 1960's it became apparent to those responsible for operating the Lower Colorado River that, in order to prevent waste of water to the Gulf of California and inadvertent overdelivery, it would be necessary to improve the
system for monitoring the riverflows below Parker Dam. As a result of this realization and of studies made of available equipment, a new system of telemetering and data recording was installed to monitor flow changes in the river as it progressed downstream from Parker Dam to Imperial Dam, a distance of 147 (237 km) miles.

c. Parker Dam Last Major Storage. Parker Dam, located near Parker, Arizona, is the most downstream major storage reservoir on the Colorado River and is the point at which releases are made to meet daily water orders for all downstream users. From a point 16 miles below Parker Dam, the river traverses a wide valley floor for 85 miles to the lower end of Cibola Valley. The river channel is then confined by low desert mountains for 46 miles before reaching Imperial Dam which is essentially the terminus of all normal riverflows. Much of the river in the wide valley is subject to a variety of water and channel control problems such as storm inflow, ground-water movement, and shifting sediment.

d. Limited Storage at Imperial Dam. The storage in the forebay of Imperial Dam is limited to approximately 1,000 acre-feet. (123,500,000 m$^3$) This is not very significant for meeting immediate increased demands by water users below that point. This storage also is
decreasing due to sediment and phreatophyte growth above the dam. The need for additional storage near Imperial Dam became more acute as the years passed. Changing weather conditions placed critical demands on riverflows arriving at Imperial Dam that could not be delivered from Parker Dam storage in sufficient time to meet the needs of the water users.

e. Senator Wash Facility. The need for this additional storage near Imperial Dam resulted in the construction of Senator Wash Dam on Senator Wash a short distance upstream from Imperial Dam. Senator Wash is an offstream site and the constructed facilities include capabilities for pumping to storage when a surplus of water arrives and generation of power when releases are made back to the river above Imperial Dam to meet unforeseen demands that are in excess of available river water. This facility was designed for use only in case of emergencies that could not be foreseen and supplied normally by regulation of releases at Parker Dam. It has greatly improved the availability of water in these emergency situations as well as providing space for storage of unpredictable flood inflow.

The pumping and release capacities at the plant at about 80 percent of maximum head are about 1,200 cfs as compared to flows in the river that range from 2,000 to 10,000 cfs. The active capacity of the reservoir is about 12,000 acre-feet. The reservoir
active storage is normally maintained at about 7,000 acre-feet so that about 5,000 acre-feet of space are available to receive excess water or about 7,000 acre-feet of water is available to meet shortages in the river. Since the Senator Wash facility and the procedure for estimating and monitoring transient flow changes have been in operation, overdeliveries and wastes to the Gulf of California have been negligible.

2. Management of Releases

a. Water Orders. Requests for delivery of water are made to the Supervisor at Imperial Dam on Wednesday of each week. All water users below Parker Dam are required to submit to the Supervisor at Imperial Dam their daily estimates of water requirements for the following week (Monday through Sunday). These water requirements are combined by the Supervisor's office with estimates of transit losses. Routing computations are then made to determine the required Parker releases to meet the demands along the river and at Imperial Dam. These water orders include those as far away as Coachella and Imperial Valleys, which may require as long as 5 or 6 days for delivery of water from Parker Dam to the farthest user in Coachella Valley.
b. Estimates of Daily Releases. The estimates of mean daily releases at Parker Dam are the result of a combination of the Mexican Treaty obligation, diversions at Imperial Dam, the estimated losses in the reach from Imperial Dam to Parker Dam, diversions at Palo Verde Dam, Headgate Rock Dam, return flows, and pumping diversions at several points along the river. These diversion requirements and estimates of losses combined with a routing computation produce estimates of mean daily releases at Parker Dam to meet the downstream requirements as ordered for the week being scheduled. Estimates of daily requirements for the first and second advanced weeks are also included in the routings for estimates of Parker Dam mean daily releases beyond the scheduled week.


c. Peaking Power Production at Parker Dam. The capability for obtaining peaking power has been utilized at Parker Dam since its construction. Because of the variation in the system loads during the day, hourly releases are scheduled to meet the portion of these varying requirements supplied by the Parker Dam Powerplant. The installation at Parker Dam includes four turbines and generators. These are utilized to the maximum advantage to meet peaking power requirements. The hourly rates of release during the day may vary from 2,000 cfs to approximately 20,000 cfs. This
extreme variation in an approximately cyclical pattern causes added difficulty in routing flows and monitoring the river-flow as it progresses downriver toward Imperial Dam. This pattern of release not only complicates the routing procedure but disturbs the unstable river channel which is particularly troublesome at gaging stations where a channel control is required for a stage versus flow relationship. The variation of Parker releases produces continual shifting of the control at these stations and has been one of the major causes of constantly changing conditions where some of the flow monitoring equipment has been installed.

d. Transit Time, Parker to Imperial. The time necessary for water to flow from Parker Dam to Imperial Dam is approximately 72 hours. This time may vary with the condition of pondage above Headgate Rock and Palo Verde Dams and with rising or falling stages. There is some variation in travel time from winter to summer flow conditions. The time during high summer flows may be as short as 60 hours. These travel times represent average velocities of from 2 to 2-1/2 miles per hour or roughly 3 to 3-1/2 feet per second. This travel time of from 2-1/2 to 3 days means that water users at the lower end of the reach must order water 3 days in advance in order to expect delivery of the required flows when needed.
Water users also must anticipate any changes in their requirements from day to day and keep the Supervisor at Imperial Dam informed of these required changes so that corresponding changes can be made in Parker Dam releases to prevent shortages of requirements or overdeliveries at Imperial Dam. The Decree in Arizona v. California issued by the Supreme Court dated March 9, 1964, requires an accounting for water ordered but not taken as well as other pertinent records.

3. **Major Diversion Structures**

There are four major diversion structures on the Colorado River downstream from Parker Dam: Headgate Rock Dam, Palo Verde Dam, Imperial Dam, and Morelos Dam, in downstream order.

a. **Headgate Rock Dam.** Headgate Rock Dam, located 14 miles (22.5 km) downstream from Parker Dam, diverts water on the Arizona side of the river to the Colorado River Indian Irrigation Project. The dam is a gated structure consisting of ten 34-foot 10-inch (10.62 m) radial gates in the river proper that maintain the water surface high enough to cause pondage upriver about 10 miles (16.1 km) within about 4 miles of Parker Dam. The radial gates operate automatically to maintain the normal operating water surface level but can be converted to manual control when necessary to lower.
b. Palo Verde Dam. Palo Verde Diversion Dam is located 59 miles downstream from Parker Dam and diverts water to the Palo Verde Irrigation District on the California side of the river. It has three 50-foot radial gates in the river operated automatically to maintain a normal operating level in the forebay sufficiently high to permit diversion of water into the Palo Verde Canal. The dam is operated by the Palo Verde Irrigation District. Storage of water in the forebay above Palo Verde Dam is small compared to that above Headgate Rock Dam and does not impose a problem in the operation of the river. However, backwater above the dam extends for several miles and does affect the travel time in that reach to some extent.

c. Imperial Dam. Imperial Dam is 147 miles downstream from Parker Dam. It was constructed for diversion of water into the All-American Canal and the Gila Gravity Main Canal. The All-American Canal diverts water to the Reservation and Valley Divisions of the Yuma Project and to Imperial and Coachella Valleys. The Gila Gravity Main Canal diverts water east of the river to the North and South Gila Valleys, to the Wellton-Mohawk Irrigation and Drainage District, and to the Yuma Mesa areas. All the water arriving at Imperial Dam is accounted for by the Supervisor at Imperial Dam and any water passing
Imperial Dam through the sluiceways or otherwise released to the river below Imperial Dam is normally scheduled for delivery to Mexico. Very infrequent flood releases from upstream reservoirs exceed Mexico's requirements. None have been made in recent years.

Senator Wash facility, although not considered a diversion structure, can divert water from the river immediately above Imperial Dam for storage in Senator Wash Reservoir. This is done only when flow arriving exceeds the requirements of the diverters at Imperial Dam. Imperial Dam is the nerve center for detailed operations of the Colorado River below Parker Dam. The Supervisor at Imperial Dam is responsible for correct delivery and operational accounting for all water released at Parker Dam and delivered to diverters along the river and at Imperial Dam. He also is responsible for the delivery of water to Mexico. Techniques discussed later in this paper were developed to assist in the monitoring of the movement of water from Parker Dam to Imperial Dam.

d. Morelos Dam. Morelos Dam, constructed and operated by Mexico, is located on the Colorado River 26 miles below Imperial Dam and about 1 mile south of the Northerly International Boundary of the United States and Mexico. It serves
to divert water into the Alamo Canal for delivery to Mexican water users. Water can be released to the river at Imperial Dam, at Yuma Main Canal Wasteway, and at Pilot Knob Powerplant for diversion at Morelos Dam to meet the United States obligation for delivery of water to Mexico.

4. **Return Flow and Losses**

a. **Drainage Return Flow.** Surface drainage return flow is largely confined to a few major drainage channels in the valley area between Parker and Imperial Dams. This drainage return is mainly the result of irrigation applications and is fairly constant and therefore comparatively easy to account for. Gaging stations at or near the outlets of major drainage channels are operated by the Geological Survey and data are regularly reported by field hydrographers and available to river operating personnel.

b. **Spillage Return Flow.** Spillage return flow is a greater operating problem along the river than drainage return flow, since it cannot always be predicted or reported in time for normal operating adjustments in Parker releases. Spills therefore require special action by the Imperial Dam office to adjust riverflow quantities to counter their effect.
c. Ground-Water Gains and Losses. Inflow to the river and losses from the river as underground flow is another matter. Constant surveillance at gaging stations along the river is necessary to estimate the movement of water to and from the river channel by way of underground flow. Reaches of the river between gaging stations perform differently with respect to these gains and losses. Analysis of these gains and losses over a period of years has afforded some knowledge of the performance in the various reaches. However, continuous monitoring of the riverflow at the established gaging stations is necessary to provide more and continued information on underground water movement.

5. Inflow from Storms

a. Unpredictable Thunderstorm Inflow. Thunderstorm activity along and adjacent to the river between Parker and Imperial Dams sometimes causes operating difficulties. These storms occur in sparsely settled or unsettled drainage areas adjacent to the river which are in some cases many miles from the river but cause comparatively large inflows to the river. These inflows usually reach the river before there is knowledge of their occurrence. Many of these thunderstorms occur during the night, and the following morning there is sometimes little evidence of the occurrence of a
storm of sufficient magnitude to produce runoff. Effect upon riverflow, however, is reflected in the first downstream gaging station and usually can be recognized from the analysis of the record of flow at that station. Adjustments in Parker releases are made on the basis of the amount of inflow indicated at these major river gaging stations.

b. Predictable Major Storm Inflow. Where a major storm inflow can be anticipated from Weather Bureau forecasts and precipitation records, adjustments to riverflow quantities can be made sufficiently in advance to, at least partially, counteract the effects of any inflow from major storms. Predictable major storm inflow is usually of such magnitude as to reduce irrigation diversions. Releases for flood control due to major storms are much less frequent than the adjustment of river operation because of reduced irrigation requirements. Operation changes due to flood control regulations would ordinarily involve the operation of Lake Havasu due to inflow from the Bill Williams River and other contributing drainage areas or the reduction of releases at Parker due to inflow from major storms on the Gila River.
6. Gaging Stations in the Parker-Imperial Reach

The following described gaging stations on the river between Parker Dam and Imperial Dam were selected on the basis of their locations with respect to diversions, drainage returns, and adaptability to channel control for maintaining as nearly as possible relatively constant stage-flow relationships. The five gaging stations above Imperial were selected early in the 1960's for installation of special monitoring equipment because they would provide information about the transient flow changes between Parker and Imperial Dam as well as information on side inflow. (Refer to Figure 1 for the relative locations of these gaging stations.)

a. Headgate Rock. A station in the river about 2 miles downstream from Headgate Rock Dam was selected for monitoring the river below Headgate Rock Dam to reflect possible malfunction of the automatic gates as well as providing information on the transient flow changes of water released at Parker Dam. This station was particularly desirable because of the cyclical releases for power production at Parker Dam. It provides the first indication of the continuation of the cyclical flows as they progress downstream.

b. Water Wheel. The Water Wheel gaging station is 40 miles downstream from Parker Dam and provides riverflow information
on the river above the backwater from the Palo Verde Diversion Dam. It also reflects some of the drainage return flow from the Colorado River Indian Irrigation Project and its major spillage return flow. This gaging station is located at the point in the river where relatively stable rock controls the flow in the rated section of the channel and it is the most stable section of the river in the entire reach between Parker Dam and Imperial Dam for maintaining a stage-flow relationship.

c. Palo Verde. A special installation was made at Palo Verde Diversion Dam in cooperation with the Geological Survey and Palo Verde Irrigation District to monitor gate openings and differential heads on both the river and canal gates. Analog computers are used at the Dam to determine the flow through the river gates and through the canal gates to provide data at this installation for operational purposes. This installation affords an opportunity to study the attenuation of cyclical flow as well as to provide information on flood inflow between Water Wheel and Palo Verde Diversion Dam. Records of flow at Palo Verde Diversion Dam are published in the Geological Survey "Water Supply Papers."
d. Taylor Ferry. Taylor Ferry gage is 27 miles downstream from Palo Verde Diversion Dam and is located at a point in the river where changes due to shifting sediment and channel erosion are relatively minor. Dredging in the river below Taylor Ferry gage has caused some degradation of the control section. This gage was installed originally as a backup for the Cibola gage to provide Imperial Dam with information on the progress of the Parker mean daily releases as well as monitoring the diurnal flow changes as they progressed down the river.

e. Cibola. The Cibola gage is 24 miles below Taylor Ferry and is at the lower end of the Cibola Valley. Records of riverflow here include all surface returns from the Palo Verde and Cibola Valleys. This is a Geological Survey station and records at this point on the river are published in the "Water Supply Papers." The Imperial Dam office monitors this station to determine the mean daily flow that arrives at the lower end of Cibola Valley in the Colorado River approximately one day earlier than scheduled to arrive at Imperial Dam.

f. Imperial Dam. The flow reaching Imperial Dam is computed as the sum of the water diverted to the All-American Canal, the
water diverted to the Gila Gravity Main Canal, and flow released to the river immediately below Imperial Dam. These computations are made at the office of the Supervisor at Imperial Dam. The flow reaching Imperial Dam is adjusted by releases or pumping at the Senator Wash facility to determine the riverflow exclusive of Senator Wash operation. Flows computed in this manner are compared with routed releases from Parker Dam to check loss estimates.

7. **Communications and Data Transmittal**

a. FM Radio System. The Bureau of Reclamation maintains an FM radio system on the Lower Colorado River for voice communication and data transmittal. Voice communication is used for water scheduling traffic between Imperial Dam, the Parker-Davis Project Office in Phoenix, Arizona, and the Regional Office in Boulder City, Nevada. Streamflow data stations and mobile units are used for hydrographic data transmittal from gaging stations and other field locations as needed. The monitoring of the gage at Cibola began early in the 1950's with a time programmed Stevens Telemark. Because of the need for backup data in case of equipment failure, a similar installation was later made at Taylor Ferry. Because of the impending need to monitor riverflows more closely in the 1960 decade, more sophisticated equipment was installed at Cibola and Taylor
Ferry and similar equipment was added at the other upstream stations for additional monitoring of transient flow changes.

b. Gage Data Reporting Techniques. The stations between Parker and Imperial Dams were equipped with solid-state radios and encoders to transmit flow data to the base station. The base station was installed at Boulder City for interrogating all stations over the radio network. The base station included installation of strip-chart recorders to receive and chart each of the gage reports. The equipment was designed and programmed so that once each hour on the half-hour all stations were automatically called, in order, to give a report. Except for Palo Verde Diversion Dam, potentiometers were installed in conjunction with Stevens Type A recorders or Stevens Telemarks to convert shaft positions to a voltage in proportion to a river stage or flow. The encoders converted these voltages to frequencies for transmittal by radio. At Palo Verde Diversion Dam, potentiometers were installed and driven by Selsyn motors which fed gate shaft positions and differential stages data to the analog computers which in turn fed flow data to the encoder which transmitted tones to Boulder City the frequencies of which were in proportion to flows. Since the radio system was used for voice communications and continuous monitoring at the stations was not feasible because of battery drain, the receiving recorders
were programed to record data from each station for only about 19 seconds of each hour throughout the day. This resulted in a step-type hydrograph which provided an effective tool for analysis of flow variation.

c. Predicted Flows. Flows arriving at these stations were analyzed with respect to Parker releases and diversions along the river. However, it became necessary to route the cyclical changes from Parker power releases downstream through each reach of the river in order to have a more suitable basis for comparison of indicated flows arriving at each station and flows expected to arrive at those stations. In order to make an analysis, a laborious procedure of routing was devised using a channel storage curve developed for each reach. These predicted flows were plotted on the recorder charts of each station in advance of and for comparison with the plotting of the actual recorded flows. In 1965, it became apparent that it would be necessary to computerize these predictions in order to keep them plotted sufficiently ahead of the actual recorded flows for analysis of the transient flow changes.

8. Analytical Basis for Flow Computations

a. Basis of Computations. This development is based upon the following assumptions:
(1) The flow is controlled by friction.

(2) For changes varying by small amounts from the mean flow condition, flow changes can be assumed to be proportional to the slope changes.

(3) The factor of proportionality $K$ can be evaluated from the stage-flow curves.

(4) An increase of flow produces a step increase in stage which moves downstream at the mean velocity of flow $v$.

Conditions near the step are shown on Figure 2.

b. Condition of Continuity. If $f$ represents the change of flow associated with the step then

$$f = -K \frac{\partial n}{\partial x} \quad .... \quad (1)$$

where $n$ represents the increase of depth and $x$ the distance measured from the step in the downstream direction.

The continuity condition is:

$$\frac{\partial f}{\partial x} \ dx \ dt = -T \frac{\partial n}{\partial t} \ dx \ dt$$

where $T$ represents the top width. This relation can be put into the form:

$$\frac{\partial f}{\partial x} = -T \frac{\partial n}{\partial t} \quad .... \quad (2)$$
c. Differential Equation of the Transient Flow. Elimination of $f$ between (1) and (2) yields the differential equation:

$$K \frac{\partial^2 \eta}{\partial x^2} = T \frac{\partial \eta}{\partial t}$$

\[\text{\ldots (3)}\]

or if

$$\alpha = \frac{K}{T}$$

\[\text{\ldots (4)}\]

The relation takes the form

$$\alpha \frac{\partial^2 \eta}{\partial x^2} = \frac{\partial \eta}{\partial t}$$

\[\text{\ldots (5)}\]

d. Solution of the Differential Equation. A solution satisfying the conditions that

$$\eta \to \eta_0 \text{ as } x \to 0$$

\[\text{\ldots (6)}\]

is for $x > 0$:

$$\eta = \eta_0 \left[ 1 - \frac{2}{\sqrt{\pi}} \int_0^{x} e^{-u^2} \, du \right]$$

\[\text{\ldots (7)}\]

e. Example. As an example of the use of this formula treat the stream as being represented by the Taylors Ferry section at a flow of 10000 ft$^3$ per sec. Hydraulic considerations give the flow as

$$F = Av = AC \sqrt{rS}$$

\[\text{\ldots (8)}\]

where $A$ represents the area of the cross section (ft)$^2$

$v$ the mean velocity of flow

$r$ the hydraulic radius

$S$ the slope maintaining the flow $F$

$C$ the Chezy coefficient
The change of flow as a function of the slope only is given by

\[
\frac{\partial F}{\partial S} = \frac{AC\sqrt{S}}{2\sqrt{S}} = \frac{AC\sqrt{S}}{2S} = \frac{F}{2S}
\]

then

\[
K = \frac{F}{2S}
\]

and

\[
\alpha = \frac{K}{T} = \frac{F}{2ST}
\]

The wave front gradient is obtained from Equation (7) by differentiation with respect to \( x \). Then,

\[
\frac{\partial \eta}{\partial x} = -\frac{x^2}{4at} \frac{\eta_e}{\sqrt{\pi at}}
\]

Application will be made to the reach of the Colorado River between Parker and Imperial Dams. The distance between these points is 147.0 miles or 776160 feet. The section at Taylors Ferry will be taken as representative of the river hydraulics.

From stage-flow tables for this section

- \( F = 10000 \text{ ft}^3/\text{sec} \)
- \( A = 3284 \text{ ft}^2 \)
- \( v = 3.045 \text{ ft/sec} \)
- \( S = 0.00020268 \) (dimensionless)

\( 283.2 \text{ m}^3/\text{sec} \)

\( 305.0 \text{ m}^2 \)

\( 0.9281 \text{ m/sec} \)
The top width here is 354 feet but maps and photographs indicate that the top width is generally wider than this. A width of 800 feet will be used to obtain a realistic representation of the surface storage. This latter width is used in the evaluation of \( \alpha \). Then
\[
\alpha = \frac{10000}{(2)(0.00020208)(800)} = \frac{10000}{0.32428} = 30840 \text{ ft}^2/\text{sec} \quad (2865. \text{ m}^2/\text{sec})
\]
With a velocity of 3.045 ft/sec the travel time between Parker and Imperial is
\[
\frac{776160}{3.045} = 254896 \text{ seconds or } 2.950 \text{ days}.
\]
The computation is made in the manner shown in Table 1. The first three columns are self-explanatory. Since the point of flow change is assumed to be carried along at the mean stream velocities, the \( x \) distance is obtained by subtracting the distance moved by the point of flow change from the distance from Parker to Imperial. The next two columns are self-explanatory. The next column of figures is obtained from Formula 7. Values of \( \bar{\eta} \), for \( x \) positive, are obtained by multiplying the ratio \( \frac{\eta}{\eta_0} \) by \( \eta_0 \). When \( x \) becomes negative \( \bar{\eta} \) should be interpreted as
\[
\bar{\eta} = 2\eta_0 - \eta. \quad \text{The value to be used for } \eta_0 \text{ is one-half the increase in depth due to the increased flow. This depth is 0.4105 feet. It is computed on the supposition that, for small changes, the velocity of flow remains unchanged and the top width is 800 feet. The flow due to the increased depth is obtained by multiplying the increased area of the cross section by the mean stream velocity. The next three columns are used for figures needed for estimating the increased gradient due to the slope of the wave}
### Table 1

**Base flow: 10,000 ft³/sec**  
**Top width: 800 feet**  
**Flow at Imperial due to 1,000 ft³/sec increase at Parker**  

<table>
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<tr>
<th>Time (days)</th>
<th>Time (seconds)</th>
<th>( vt ) (feet)</th>
<th>( x )</th>
<th>( \sqrt{4at} )</th>
<th>( \frac{x}{\sqrt{4at}} )</th>
<th>( \frac{\eta}{\eta_0} ) for ( x &gt; 0 )</th>
<th>Flow due to increase in depth ( \eta = 2\eta_0 - \eta ) for ( x &gt; 0 )</th>
<th>( \frac{\eta}{\eta_0} ) (ft³/sec)</th>
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</table>

\[
vt = (800)(3.045) = 2,436 \text{ ft}^2/\text{sec}.
\]

\[
2\eta_0 = \frac{1,000}{(800)(3.045)} = 0.4105 \text{ feet}.
\]

\[
\eta_0 = 0.20525 \text{ feet}.
\]

\[
\frac{F_{\eta_0}}{2S} = \frac{(10,000)(0.20525)}{(2)(0.00020268)} = 5,063,400 \text{ ft}^4/\text{sec}.
\]

*Base flow 10,000 cubic feet per second. Top width 800 feet.*
front as given by Formula (11). The flow due to the slope of the wave front is obtained by use of (9) and (11) from the relation

$$f_1 = \frac{x^2}{4at}$$

This implies that the flow is proportional to the gradient for small changes. The total flow increase is the sum of the flow due to increased in slope, depth and to increase in slope.

9. Computerized Computation Procedures

a. Computer Program. The computer program predicts the hourly flows for the six stations of the Colorado River from Parker Dam to Imperial Dam during a desired time span. In addition to the flows, it is also possible to produce two types of graphical output: influence graphs and prediction graphs.

b. Equipment. The program was originally written for the Honeywell H-800 computer and was revised to also run on the CDC-1604 B computer. A Benson-Lehner Model J electroplotter is utilized to produce the graphical output. The data are transmitted via telephone, using a Xerox Telescopier II.

c. Computations. To calculate the downstream flows, the program uses a table of influence coefficients previously
computed, which gives the effect of a unit flow for 1 hour at one station on the succeeding station’s flows for the next 48 hours. The program thus has at least 48 hours of flow history prior to the start of a prediction. By adding the effect of each of the past 48 hours’ flows at a station (N-1), and subtracting or adding a given flow constant, the flow at the following station (N) for a specific hour may be computed.

The program generates 240 influence factors which determine the wave shape and travel speed down the river, in accordance with river constants and desired percentage changes in wave magnitude and phase. Recalculating these influence factors will produce the changes in the traveling wave. These changes are made whenever a consistent prediction error is noted in either the peak-to-valley distance or the phase position of the wave profile in a reach.

Some details of experience with computations for the Parker-Imperial reach of the Colorado River may be of interest. For some reason flows in the Parker-Headgate Rock reach do not follow the pattern described by Formula (7). In this reach the propagation of Parker changes proceed as though carried by a bore wave in which inertia factors play a dominant role.
The time required to traverse this 14.34-mile reach is just 1 hour. A computation of the velocity of the bore wave is

\[ V_w = \sqrt{gD_m} + v = \sqrt{(32.2)(10)} + 3.0 = 20.9 \text{ ft/sec or } 15.7 \text{ mi/hr} \]

where \( V_w \) represents the velocity of the bore wave with respect to an origin moving with the stream and \( v \) the flow velocity. The quantities of \( g \) and \( D_m \) represent the acceleration of gravity and the mean depth, respectively. This is a reasonably close check. If there were no friction the Parker releases would be propagated without change of shape. There is, of course, some friction and gage readings do show some rounding of the abrupt profiles of the Parker releases. In the actual computations the Parker releases are treated as coming from Headgate Rock but with an hour delay with respect to the time of release at Parker. No reason is known that would explain why the performance of the river is different in the Parker-Headgate Rock reach than it is in the remaining reaches between Headgate Rock and Imperial.

d. Input Information. The complete set of input data is checked for errors by the computer program and if any errors are found, indications are printed and the computer computations are terminated.

e. Output Information. A complete set of output data consisting of flow predictions, daily totals, and averages is pro-
duced (see Figure 3), while a condensed form of output more suitable for teletype transmission is also produced (see Figure 4). For the condensed form the time is given on a 12-hour clock, station names are abbreviated, and flow values are given for odd hours only, rounded to the nearest 1 cfs, and 2 days per page are printed.

One of the sets of plots which can be produced is S-shaped curves giving the influence of a step increment in flow on downstream stations for various stations on the river. These curves are calculated, using the influence factors described above (see Figure 5).

The second set of plots which can be produced is curves giving the historic and predicted flows at the various stations on the river. These curves are calculated using the maximum and minimum prediction and history values (see Figure 6).

The scales for these two types of plots can be varied, thereby producing the desired graphs most easily used.

f. Arrangements for Making the Computations. The daily procedures used in processing and transmitting data between the Bureau of Reclamation offices in Denver, Colorado, and Boulder City, Nevada, are as follows:
1. Parker Dam releases, together with the flow changes at all stations, are transmitted via the teletypewriter from Boulder City, Nevada, to Denver, Colorado. (Data for phase shifts and magnitude changes for any station may also be included.)

2. The data are then processed on the H-800 computer.

3. The output data are immediately transmitted via the teletypewriter from Denver, Colorado, to Boulder City, Nevada.

4. If any plots are requested, they are produced at a later time and then sent via teletypewriter or regular mail to Boulder City, Nevada.

10. Summary

The 1953-1956 drought period in the Colorado River Basin and subsequent below normal runoff from the basin, together with the decree of the Supreme Court of the United States, dated March 9, 1964, made it necessary to improve the old system of monitoring transient flow changes of the Colorado River from Parker Dam to Imperial Dam, a distance of 147 miles. Lake Havasu above Parker Dam is the most downstream holdover storage for the delivery of domestic and irrigation water to United States and Mexican users along and at the terminus of the 147-mile reach of river.
Using the actual and scheduled releases at Parker Dam, there was a need to develop a method for predicting normal transient changes in riverflow as the flow traversed the reach. The recorded flow at monitored stations was to be compared with these predicted flows to determine storm inflow, gate malfunctions, changes in loss and return flow rates, and unscheduled changes in diversions.

The continual cyclic change in the transient flows throughout the reach due to generation of power to meet peak demands necessitated the development of a procedure that could be adapted to a computer, in order to make predicted flows available on a usable time schedule.

11. Conclusions

a. A method of estimating transient flow changes in a natural stream has been developed.

b. Daily comparisons made over a 5-year period on the 147-mile reach between the Parker and Imperial Dams on the Colorado River have demonstrated its usefulness for tightening control of the river for the purpose of preventing waste of water.

c. The constant $\alpha$ has been found to remain stable over a wide range of mean flows.
Figure 2  WAVE PROFILE
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<th>PARKER DAM</th>
<th>HEADGATE ROCK</th>
<th>WATER WHEEL</th>
<th>PALO VERDE</th>
<th>TAYLORS FERRY</th>
<th>CIBOLA</th>
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**TOTAL:** 252570 230280 219823 198890 192323 200257 193904

**AVERAGE:** 10524 9595 9159 8287 8013 8344 8079

Figure 3
### Table 3:
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**WED TOTAL:** 213780 215432 185762 188077 198874 186671

**WED AVE:** 8907 8976 7740 7837 8286 7778

---

Figure 4
INFLUENCE COEFFICIENT GRAPH

DAYS

PERCENT OF FLOW INCREMENT RECEIVED

0 1 2 3 4 5

0 10 20 30 40 50 60 70 80 90 100 110 120

HOURS

WATER WHEEL
HEADGATE ROCK
Palo Verde
Taylors Ferry
Cibola
Imperial

EFFECT ON DOWNSTREAM STATIONS OF A UNIT FLOW INCREASE AT PARKER DAM

Figure 3
PREDICTION MADE 3-25-66 FOR TAYLORS FERRY
The following symbols are used in this paper.

\[ A = \text{represents the area of the stream cross section}; \]
\[ B_1, B_2 = \text{disposable constants}; \]
\[ C = \text{the Chezy coefficient}; \]
\[ D_m = \frac{A}{T} = \text{the mean depth of the stream}; \]
\[ f = \text{the change of flow associated with a step increase of stage} \]
\[ \text{(see Equation 1)}; \]
\[ F = \text{the flow of the stream}; \]
\[ g = \text{the acceleration of gravity}; \]
\[ K = \text{a factor of proportionality connecting the flow change} f \]
\[ \text{with the gradient associated with a step increase of stage}; \]
\[ m = \text{a quantity used in determining a value for} \ \alpha \ \text{from the} \]
\[ \text{observed performance of the stream (see Equation 13)}; \]
\[ r = \text{a hydraulic radius}; \]
\[ s = \text{the stream gradient (see Figure 1)}; \]
\[ t = \text{time}; \]
\[ T = \text{top width of the stream}; \]
\[ v = \text{the mean velocity of flow of the stream}; \]
\[ V_w = \text{the velocity of a bore wave}; \]
\( x \) — distance measured downstream from the step;

\( \alpha = K/T = F/2ST \) — a constant specifying the transient performance of the stream;

\( \eta \) — the increase of stage produced at the distance \( x \) and the time \( t \) by a step increase in stage of initial amount \( 2\eta_0 \);

\( \eta_0 \) — one half the initial stage increase associated with an abrupt change of flow;

\( \bar{\eta} \) — \( \eta \) for \( x > 0 \) (see Figure 2);

\( \tilde{\eta} \) — \( 2\eta_0 - \eta \) for \( x \leq 0 \) (see Figure 2);

\( \pi = 3.14159^+ \); and

\( e = 2.71828^+ \).
September 15, 1971

File: 16-2-1.HY

Mr. Chester A. Nelson
United States Department of the Interior
Bureau of Reclamation
Engineering and Research Center
Building 67, Denver Federal Center
Denver, Colorado 80225

Dear Mr. Nelson:

Subject: "Colorado Riverflow Management
by R.E. Glover, C.A. Nelson & J.I. Sanders

We have received the above-cited paper for review for possible publication by ASCE.

As you will note from page 6 of the enclosed "Authors' Guide to the Publications of ASCE" the length limit for ASCE papers is 10,000 word-equivalents. We estimate that your paper consists of 9,000 word equivalents of text, 700 of tables, and 2,600 of figures. You are, therefore, urged to reduce your paper to the length limit. If you find that shortening the paper does not bring it under the limit, we would like you to resubmit it with an explanation of the reasons why further shortening is not feasible and why you feel the length limit should be waived in this particular case.

It is, of course, hoped that you can accomplish the reduction down to the established limit. If you do so, and if the reviewers find the paper acceptable, publication in the Journal will follow as soon thereafter as practical. If you cannot bring the paper to the length limit, the process of obtaining length-limit waivers will consume a period of time that both of us may find unacceptable.

Three copies of the paper are being returned herewith.

Your cooperation and understanding will be appreciated.

Sincerely,

Paul A. Parisi
Manager, Technical Publications

PAP:cp
Enc.
United States Department of the Interior  
BUREAU OF RECLAMATION  
ENGINEERING AND RESEARCH CENTER  
BUILDING 67, DENVER FEDERAL CENTER  
DENVER, COLORADO 80225

AUG 17 1971

IN REPLY
REFER TO: 1510

Manager, Technical Publications  
American Society of Civil Engineers  
345 East 47th Street  
New York, New York 10017

Dear Sir:

The paper "Colorado Riverflow Management" is herewith submitted with an abstract for consideration for publication in the Journal of the Hydraulics Division. The abstract and paper have been prepared according to the information in Volume 96, No. IR4, December 1970, inside back cover. Additional information will be furnished as requested.

Very truly yours,

[Signature]

Chester A. Nelson

Enclosures

Blind to: Mr. Robert E. Glover  
1936 South Lincoln Street  
Denver, Colorado 80210

Mr. John I. Sanders  
1317 Denver Street  
Boulder City, Nevada 89005
Mr. B.P. Bellport, Director
Office of Design and Construction
Bureau of Reclamation
Denver Federal Center
Denver, Colorado - 80225

Attention:
Mr. C.A. Nelson
Room 239H-Bldg 56
Mail Code 1512-C

Dear Mr. Bellport:

A list of notation, with an original and two copies, is forwarded herewith as was agreed with Mr. Nelson when I visited his Office yesterday. This list is to supplement the proposed paper on Colorado River Flow Management by Messrs Glover, Nelson and Sanders.

A review of the final draft reveals two minor errors. Figure 1 of page 2 lacks a notation that it is Figure 1. On page 28 there is a double bar over the $\eta$ symbol. There should be one bar only.

Sincerely yours,

Robert E. Glover
COLORADO RIVERFLOW MANAGEMENT

KEY WORDS: river hydraulics; stream flow; flow transients; propagation of flow changes.

ABSTRACT: A method of computing transient flow changes in a flowing stream is described. The results of more than five years of experience in the use of this method for estimating flows at the gaging stations of the 147 mile Parker-Imperial reach of the Colorado River are summarized. An account is also made of digital computer applications which enable tabulations of the expected flows at the gaging stations for each hour of the ensuing 24 hours to be made and transmitted to the Boulder City Office of the U.S.B.R. in time to be available at the beginning of work each day. Tabulations covering 72 hours are prepared for weekend operations.
Appendix - Notation.

The symbols used have the following significance.
Consistent units are used.

\[ A \] represents the area of the stream cross section,
\[ B_1 \text{ and } B_2 \] disposable constants,
\[ C \] the Chezy coefficient,
\[ D_m = \frac{A}{T} \] the mean depth of the stream,
\[ f \] the change of flow associated with a step increase of stage,
(See equation 1),
\[ F \] the flow of the stream,
\[ g \] the acceleration of gravity,
\[ K \] a factor of proportionality connecting the flow change \( f \) with the gradient associated with a step increase of stage,
\[ m \] a quantity used in determining a value for \( \alpha \) from the observed performance of the stream (see equation 13),
\[ r \] an hydraulic radius,
\[ s \] the stream gradient (see figure 1),
\[ t \] time,
\[ T \] top width of the stream,
\[ v \] the mean velocity of flow of the stream,
\[ V_{w} \] the velocity of a bore wave,
\[ x \] distance measured downstream from the step,
\[ \alpha = \frac{K}{T} \text{ or } \alpha = \frac{F}{2 g T} \] a constant specifying the transient performance of the stream,
\[ \eta \] the increase of stage produced at the distance \( x \) and the time \( t \) by a step increase in stage of initial amount \( 2 \eta_0 \)
\( \eta_0 \), one half the initial stage increase associated with an abrupt change of flow,

\[
\bar{\eta}, \text{ (see figure 2) } \quad \bar{\eta} = \eta \quad \text{for } \chi > 0
\]

\[
= 2 \eta_0 - \eta \quad \text{for } \chi < 0.
\]

\( \pi = 3.14159^+ \)

\( e = 2.71828^+ \)
COLORADO RIVERFLOW MANAGEMENT

by

Robert E. Glover, 1/ F. ASCE
Chester A. Nelson, 2/ M. ASCE
John I. Sanders 3/

Introduction

The Bureau of Reclamation of the Department of the Interior administers water deliveries from the Colorado River of the Southwest with its system of powerplants and storage reservoirs. This paper describes the needs and the procedures developed for improving the monitoring of transient flow changes of the Colorado River from Parker Dam to Imperial Dam. This reach of the river is shown on Figure 1.

1. Need for Close Control of the River

   a. Authority and Regulations. All operations of the Colorado River below Hoover Dam are made in accordance with the Colorado River Compact, the Boulder Canyon Project Act and its associated power and water delivery contracts, the Mexican Water Treaty, and the Decree of the Supreme Court of the United States

1/ General Engineer, U.S. Bureau of Reclamation, Retired.
2/ Research Structural Engineer, U.S. Bureau of Reclamation, Denver, Colorado.
3/ Hydraulic Engineer, U.S. Bureau of Reclamation, Retired.
in Arizona v. California, dated March 9, 1964. The requirements stipulated by these regulatory documents include operation for flood control, irrigation and domestic water release, and releases for power generation.

b. Water Conservation Necessary. Each year the spring runoff from melting of the winter's snow accumulation on the west slope of the Rocky Mountains is stored in Colorado River reservoirs. This is a part of the flood control operations of the river. However, there have been very few releases strictly for flood control during the past several years because of the drought period in the 1950's and a recurrence of below-normal runoff again during part of the last decade. The construction of Glen Canyon Dam on the Colorado River has vastly decreased the probability of having to make flood control releases that would waste water to the Gulf of California in the near future.

Early in the 1960's it became apparent to those responsible for operating the Lower Colorado River that, in order to prevent waste of water to the Gulf of California and inadvertent overdelivery to Mexico, it would be necessary to improve the system for monitoring the riverflows below Parker Dam. As a result of this realization and of studies made of available equipment, a new system of telemetering and data recording was installed to monitor flow changes in the river.
as it progressed downstream from Parker Dam to Imperial Dam, a distance of 147 miles.

c. *Parker Dam Last Major Storage.* Parker Dam, located near Parker, Arizona, is the most downstream major storage reservoir on the Colorado River and is the point at which releases are made to meet daily water orders for all downstream users. From a point 16 miles below Parker Dam, the river traverses a wide valley floor for 85 miles to the lower end of Cibola Valley. The river channel is then confined by low desert mountains for 46 miles before reaching Imperial Dam which is essentially the terminus of all normal riverflows. Much of the river in the wide valley is subject to a variety of water and channel control problems such as storm inflow, ground-water movement and shifting sediment.

d. *Limited Storage at Imperial Dam.* The storage in the forebay of Imperial Dam is limited to approximately 1000 acre-feet. This is not very significant for meeting immediate increased demands by water users below that point. This storage also is decreasing due to sediment and phreatophyte growth above the dam. The need for additional storage near Imperial Dam became more acute as the years passed. Changing weather conditions placed critical demands on riverflows arriving at Imperial Dam that could not be delivered from Parker Dam storage in sufficient time to meet the needs of the water users.
e. **Senator Wash Facility.** The need for this additional storage near Imperial Dam resulted in the construction of Senator Wash Dam on Senator Wash a short distance upstream from Imperial Dam. Senator Wash is an offstream site and the constructed facilities include capabilities for pumping to storage when a surplus of water arrives and generation of power when releases are made back to the river above Imperial Dam to meet unforeseen demands that are in excess of available river water. This facility was designed for use only in case of emergencies that could not be foreseen and supplied normally by regulation of releases at Parker Dam. It has greatly improved the availability of water in these emergency situations as well as providing space for storage of unpredictable flood inflow.

The pumping and release capacities at the plant at about 80 percent of maximum head are about 1200 cfs as compared to flows in the river that range from 2000 to 10000 cfs. The active capacity of the reservoir is about 12000 acre-feet. The reservoir active storage is normally maintained at about 7000 acre-feet so that about 5000 acre-feet of space is available to receive excess water or about 7000 acre-feet of water is available to meet shortages in the river. Since the Senator Wash facility and the procedure for estimating and monitoring transient flow changes have been in operation, overdeliveries to Mexico and wastes to the Gulf of California have been negligible.
2. Management of Releases

a. Water Orders. Requests for delivery of water are made to the Supervisor at Imperial Dam on Wednesday of each week. All water users below Parker Dam are required to submit to the Supervisor at Imperial Dam their daily estimates of water requirements for the following week (Monday through Sunday). These water requirements are combined by the Supervisor's office with estimates of transit losses. Routing computations are then made to determine the required Parker releases to meet the demands along the river and at Imperial Dam. These water orders include those as far away as Coachella and Imperial Valleys, which may require as long as 5 or 6 days for delivery of water from Parker Dam to the farthest user in Coachella Valley.

b. Estimates of Daily Releases. The estimates of mean daily releases at Parker Dam are the result of a combination of the Mexican Treaty obligation, diversions at Imperial Dam, the estimated losses in the reach from Imperial Dam to Parker Dam, diversions at Palo Verde Dam, Headgate Rock Dam, return flows, and pumping diversions at several points along the river. These diversion requirements and estimates of losses combined with a routing computation produce estimates of mean daily releases at Parker Dam to meet the downstream requirements as ordered for the week being scheduled. Estimates of daily requirements for
the first and second advanced weeks are also included in the routings for estimates of Parker Dam mean daily releases beyond the scheduled week.

c. *Peaking Power Production at Parker Dam.* The capability for obtaining peaking power has been utilized at Parker Dam since its construction. Because of the variation in the system loads during the day, hourly releases are scheduled to meet the portion of these varying requirements supplied by the Parker Dam Powerplant. The installation at Parker Dam includes four (4) turbines and generators. These are utilized to the maximum advantage to meet peaking power requirements. The hourly rates of release during the day may vary from 2000 cfs to approximately 20000 cfs. This extreme variation in an approximately cyclical pattern causes added difficulty in routing flows and monitoring the riverflow as it progresses downriver toward Imperial Dam. This pattern of release not only complicates the routing procedure but disturbs the unstable river channel which is particularly troublesome at gaging stations where a channel control is required for a stage versus flow relationship. The variation of Parker releases produces continual shifting of the control at these stations and has been one of the major causes of constantly changing conditions where some of the flow monitoring equipment has been installed.
d. **Transit Time, Parker to Imperial.** The time necessary for water to flow from Parker Dam to Imperial Dam is approximately 72 hours. This time may vary with the condition of pondage above Headgate Rock and Palo Verde Dams and with rising or falling stages. There is some variation in travel time from winter to summer flow conditions. The time during high summer flows may be as short as 60 hours. These travel times represent average velocities of from 2 to 2-1/2 miles per hour or roughly 3 to 3-1/2 feet per second. This travel time of from 2-1/2 to 3 days means that water users at the lower end of the reach must order water 3 days in advance in order to expect delivery of the required flows when needed. Water users also must anticipate any changes in their requirements from day to day and keep the Supervisor at Imperial Dam informed of these required changes so that corresponding changes can be made in Parker Dam releases to prevent shortages of requirements or overdeliveries at Imperial Dam. The Decree in Arizona v. California issued by the Supreme Court dated March 9, 1964, requires an accounting for water ordered but not taken as well as other pertinent records.

3. **Major Diversion Structures**

There are four major diversion structures on the Colorado River downstream from Parker Dam; Headgate Rock Dam, Palo Verde Dam, Imperial Dam, and Morelos Dam, in downstream order.
a. *Headgate Rock Dam.* Headgate Rock Dam, located 14 miles
downstream from Parker Dam, diverts water on the Arizona side
of the river to the Colorado River Indian Irrigation Project.
The dam is a gated structure consisting of ten 34-foot 10-inch
radial gates in the river proper that maintain the water sur-
face high enough to cause pondage upriver about 10 miles to
within about 4 miles of Parker Dam. The radial gates operate
automatically to maintain the normal operating water surface
level but can be converted to manual control when necessary to
lower the reservoir above the dam for emergencies or mainte-
nance service. The reservoir is usually drained annually for
maintenance work on the project canal and along the river
reach above the dam. This operation imposes a problem for
water control since the reservoir contains about 6000 acre-
feet of water. Special procedures are followed during this
period which utilize the storage released from the lake
above the dam to meet downstream requirements. Parker Dam
releases are reduced by a flow sufficient to drain the reser-
voir at the desired rate. When the lake is refilled, the
reverse procedure is followed.

b. *Palo Verde Dam.* Palo Verde Diversion Dam is located
59 miles downstream from Parker Dam and diverts water to the
Palo Verde Irrigation District on the California side of the
river. It has three 50-foot radial gates in the river operated automatically to maintain a normal operating level in the forebay sufficiently high to permit diversion of water into the Palo Verde Canal. The dam is operated by the Palo Verde Irrigation District. Storage of water in the forebay above Palo Verde Dam is small compared to that above Headgate Rock Dam and does not impose a problem in the operation of the river. However, backwater above the dam extends for several miles and does affect the travel time in that reach to some extent. One or more of the river gates are opened periodically to sluice trash and sediment from the forebay. This does not present a problem when accomplished in a short period of time. However, when the river gates are opened to drain the forebay and the upper end of the canal for maintenance work and the gates remain open for more than a 24-hour period, consideration is given to the volume of water drained and Parker releases may be reduced accordingly. As in the case of the operation of Headgate Rock Dam during similar periods, the reverse procedure is followed when the forebay and the canal are refilled.

c. **Imperial Dam.** Imperial Dam is 147 miles downstream from Parker Dam. It was constructed for diversion of water into the All-American Canal and the Gila Gravity Main Canal. The All-American Canal diverts water to the Reservation and Valley Divisions of the Yuma Project and to Imperial and Coachella
Valleys. The Gila Gravity Main Canal diverts water east of the river to the North and South Gila Valleys, to the Welton-Mohawk Irrigation and Drainage District, and to the Yuma Mesa areas. All the water arriving at Imperial Dam is accounted for by the Supervisor at Imperial Dam and any water passing Imperial Dam through the sluiceways or otherwise released to the river below Imperial Dam is normally scheduled for delivery to Mexico. Very infrequent flood releases from upstream reservoirs exceed Mexico's requirements. None have been made in recent years.

Senator Wash facility, although not considered a diversion structure, can divert water from the river immediately above Imperial Dam for storage in Senator Wash Reservoir. This is done only when flow arriving exceeds the requirements of the diverters at Imperial Dam. Imperial Dam is the nerve center for detailed operations of the Colorado River below Parker Dam. The Supervisor at Imperial Dam is responsible for correct delivery and operational accounting for all water released at Parker Dam and delivered to diverters along the river and at Imperial Dam. He also is responsible for the delivery of water to Mexico. Techniques discussed later in this paper were developed to assist in the monitoring of the movement of water from Parker Dam to Imperial Dam.
d. Morelos Dam. Morelos Dam, constructed and operated by Mexico, is located on the Colorado River 26 miles below Imperial Dam and about 1 mile south of the Northerly International Boundary of the United States and Mexico. It serves to divert water into the Alamo Canal for delivery to Mexican water users. Water can be released to the river at Imperial Dam, at Yuma Main Canal Wasteway, and at Pilot Knob Powerplant for diversion at Morelos Dam to meet the United States obligation for delivery of water to Mexico.

4. Return Flow and Losses

a. Drainage Return Flow. Surface drainage return flow is largely confined to a few major drainage channels in the valley area between Parker and Imperial Dams. This drainage return is mainly the result of irrigation applications and is fairly constant and therefore comparatively easy to account for. Gaging stations at or near the outlets of major drainage channels are operated by the Geological Survey and data are regularly reported by field hydrographers and available to river operating personnel.

b. Spillage Return Flow. Spillage return flow is a greater operating problem along the river than drainage return flow, since it cannot always be predicted or reported in time for
normal operating adjustments in Parker releases. Spills therefore require special action by the Imperial Dam office to adjust riverflow quantities to counter their effect. Small operating spillage return flows can normally be ironed out in the river within a 24-hour period. The Colorado River Indian Irrigation Project normally maintains a constant level in its main canal by use of the automatic spillway at the end of the main canal which spills water directly into the river a few miles above Palo Verde Dam. This spillage return reflects the variations of diversions from the canal and is dependent upon the needs of the farmers for irrigation water. Occasionally spillage return into the river is caused by malfunction of an automatic gate at one of the diversion dams. This can usually be detected at a monitored gaging station downstream if not previously reported by a gate tender and adjustments can be made to Parker releases to compensate for the change in scheduled flows resulting from the malfunction.

c. Ground-Water Gains and Losses. Inflow to the river and losses from the river as underground flow is another matter. Constant surveillance at gaging stations along the river is necessary to estimate the movement of water to and from the river channel by way of underground flow. Reaches of the river between gaging stations perform differently with respect to these gains and losses. Analysis of these gains and losses over a period of years has afforded some knowledge of the
performance in the various reaches. However, continuous monitoring of the riverflow at the established gaging stations is necessary to provide more and continued information on underground water movement.

5. Inflow from Storms

a. Unpredictable Thunderstorm Inflow. Thunderstorm activity along and adjacent to the river between Parker and Imperial Dams sometimes causes operating difficulties. These storms occur in sparsely settled or unsettled drainage areas adjacent to the river which are in some cases many miles from the river but cause comparatively large inflows to the river. These inflows usually reach the river before there is knowledge of their occurrence. Many of these thunderstorms occur during the night, and the following morning there is sometimes little evidence of the occurrence of a storm of sufficient magnitude to produce runoff. Effect upon riverflow, however, is reflected in the first downstream gaging station and usually can be recognized from the analysis of the record of flow at that station. Adjustments in Parker releases are made on the basis of the amount of inflow indicated at these major river gaging stations.

b. Predictable Major Storm Inflow. Where a major storm inflow can be anticipated from Weather Bureau forecasts and precipitation records, adjustments to riverflow quantities
can be made sufficiently in advance to, at least partially, counteract the effects of any inflow from major storms. Predictable major storm inflow is usually of such magnitude as to reduce irrigation diversions. Releases for flood control due to major storms are much less frequent than the adjustment of river operation because of reduced irrigation requirements. Operation changes due to flood control regulations would ordinarily involve the operation of Lake Havasu due to inflow from the Bill Williams River and other contributing drainage areas or the reduction of releases at Parker due to inflow from major storms on the Gila River.

6. Gaging Stations in the Parker-Imperial Reach

The following described gaging stations on the river between Parker Dam and Imperial Dam were selected on the basis of their locations with respect to diversions, drainage returns, and adaptability to channel control for maintaining as nearly as possible relatively constant stage-flow relationships. The five gaging stations above Imperial were selected early in the 1960's for installation of special monitoring equipment because they would provide information about the transient flow changes between Parker and Imperial Dam as well as information on side inflow. (Refer to Figure 1 for the relative locations of these gaging stations.)
a. Headgate Rock. A station in the river about 2 miles downstream from Headgate Rock Dam was selected for monitoring the river below Headgate Rock Dam to reflect possible malfunction of the automatic gates as well as providing information on the transient flow changes of water released at Parker Dam. This station was particularly desirable because of the cyclical releases for power production at Parker Dam. It provides the first indication of the continuation of the cyclical flows as they progress downstream. Of particular importance is the record of the effect of storage above Headgate Rock Dam and the automatic gates on the cyclical flow.

b. Water Wheel. The Water Wheel gaging station is 40 miles downstream from Parker Dam and provides riverflow information on the river above the backwater from the Palo Verde Diversion Dam. It also reflects some of the drainage return flow from the Colorado River Indian Irrigation Project and its major spillage return flow. This gaging station is located at the point in the river where relatively stable rock controls the flow in the rated section of the channel and it is the most stable section of the river in the entire reach between Parker Dam and Imperial Dam for maintaining a stage-flow relationship. There is usually little inflow from storm runoff between Parker Dam and the Water Wheel Gage. This provides a good opportunity to identify and analyze flow travel time and attenuation of Parker Dam cyclical releases down to this
point on the river. Below Water Wheel Gage, however, major side inflow and the effect of valley seepage has made the analysis of records somewhat more difficult.

c. Palo Verde. A special installation was made at Palo Verde Diversion Dam in cooperation with the Geological Survey and Palo Verde Irrigation District to monitor gate openings and differential heads on both the river and canal gates. Analog computers are used at the Dam to determine the flow through the river gates and through the canal gates to provide data at this installation for operational purposes. This installation affords an opportunity to further study the attenuation of cyclical flow as well as to provide information on flood inflow between Water Wheel and Palo Verde Diversion Dam. Records of flow at Palo Verde Diversion Dam are published in the Geological Survey "Water Supply Papers."

d. Taylor Ferry. Taylor Ferry Gage is 27 miles downstream from Palo Verde Diversion Dam and is located a point in the river where there are relatively minor changes due to shifting sediment and channel erosion. Dredging in the river below Taylor Ferry Gage has caused some degradation of the control section. This gage was installed originally as a backup for the Cibola gage to provide Imperial Dam with information on the progress of the Parker mean daily
releases as well as monitoring the diurnal flow changes as they progressed down the river.

e. *Cibola.* The Cibola Gage is 24 miles below Taylor Ferry and is at the lower end of the Cibola Valley. Records of riverflow here include all surface returns from the Palo Verde and Cibola Valleys. This is a Geological Survey station and records at this point on the river are published in the "Water Supply Papers." Imperial Dam monitors this station to determine the mean daily flow that arrives at the lower end of Cibola Valley in the Colorado River approximately one (1) day earlier than scheduled to arrive at Imperial Dam.

f. *Imperial Dam.* The flow reaching Imperial Dam is computed as the sum of the water diverted to the All-American Canal, the water diverted to the Gila Gravity Main Canal, and flow released to the river immediately below Imperial Dam. These computations are made at the office of the Supervisor at Imperial Dam. The flow reaching Imperial Dam is adjusted by releases or pumping at the Senator Wash facility to determine the riverflow exclusive of Senator Wash operation. Flows computed in this manner are compared with routed releases from Parker Dam to check loss estimates.
Communications and Data Transmittal

a. **FM Radio System.** The Bureau of Reclamation maintains an FM radio system on the Lower Colorado River for voice communication and data transmittal. Voice communication is used for water scheduling traffic between Imperial Dam, the Parker-Davis Project Office in Phoenix, Arizona, and the Regional Office in Boulder City, Nevada. Streamflow data stations and mobile units are used for hydrographic data transmittal from gaging stations and other field locations as needed. The monitoring of the gage at Cibola began early in the 1950's with a time programed Stevens Telemark. Because of the need for backup data in case of equipment failure, a similar installation was later made at Taylor Ferry. Because of the impending need to monitor riverflows more closely in the 1960 decade, more sophisticated equipment was installed at Cibola and Taylor Ferry and similar equipment was added at the other upstream stations for additional monitoring of transient flow changes.

b. **Gage Data Reporting Techniques.** The stations between Parker and Imperial Dams were equipped with solid-state radios and encoders to transmit flow data to the base station. The base station was installed at Boulder City for interrogating all stations over the radio network. The base station included installation of strip-chart recorders to receive and chart each of the gage reports. The equipment was designed and programed
so that once each hour on the half-hour all stations were automatically called, in order, to give a report. Except for Palo Verde Diversion Dam, potentiometers were installed in conjunction with Stevens Type A recorders or Stevens Telemarks to convert shaft positions to a voltage in proportion to a river stage or flow. The encoders converted these voltages to frequencies for transmittal by radio. At Palo Verde Diversion Dam, potentiometers were installed and driven by Selsyn motors which fed gate shaft positions and differential stages data to the analog computers which in turn fed flow data to the encoder which transmitted tones to Boulder City the frequencies of which were in proportion to flows. Since the radio system was used for voice communications and continuous monitoring at the stations was not feasible because of battery drain, the receiving recorders were programmed to record data from each station for only about 19 seconds of each hour throughout the day. This resulted in a step-type hydrograph which provided an effective tool for analysis of flow variation.

c. Predicted Flows. Flows arriving at these stations were analyzed with respect to Parker releases and diversions along the river. However, it became necessary to route the cyclical changes from Parker power releases downstream through each reach of the river in order to have a more suitable basis for comparison of indicated flows arriving at each station and flows expected to arrive at those stations. In order to
make an analysis, a laborious procedure of routing was devised using a channel storage curve developed for each reach. These predicted flows were plotted on the recorder charts of each station in advance of and for comparison with the plotting of the actual recorded flows. In 1965, it became apparent that it would be necessary to computerize these predictions in order to keep them plotted sufficiently ahead of the actual recorded flows for analysis of the transient flows changes.

8. **Analytical Basis for Flow Computations**

a. **Basis of Computations.** This development is based upon the following assumptions:

(1) The flow is controlled by friction.

(2) For changes varying by small amounts from the mean flow condition, flow changes can be assumed to be proportional to the slope changes.

(3) The factor of proportionality \( K \) can be evaluated from the stage-flow curves.

(4) An increase of flow produces a step increase in stage which moves downstream at the mean velocity of flow \( v \). Conditions near the step are shown on Figure 2.
Figure 2  WAVE PROFILE
b. **Condition of Continuity.** If \( f \) represents the change of flow associated with the step then

\[
f = -K \frac{\partial n}{\partial x}
\]

\( \ldots \quad (1) \)

where \( n \) represents the increase of depth and \( x \) the distance measured from the step in the downstream direction.

The continuity condition is:

\[
\frac{\partial f}{\partial x} \, dx \, dt = -T \frac{\partial n}{\partial t} \, dx \, dt
\]

where \( T \) represents the top width. This relation can be put into the form:

\[
\frac{\partial f}{\partial x} = -T \frac{\partial n}{\partial t}
\]

\( \ldots \quad (2) \)

c. **Differential Equation of the Transient Flow.** Elimination of \( f \) between (1) and (2) yields the differential equation:

\[
K \frac{\partial^2 n}{\partial x^2} = T \frac{\partial n}{\partial t}
\]

\( \ldots \quad (3) \)

or if

\[
\alpha = \frac{K}{T}
\]

\( \ldots \quad (4) \)

The relation takes the form

\[
\alpha \frac{\partial^2 n}{\partial x^2} = \frac{\partial n}{\partial t}
\]

\( \ldots \quad (5) \)

d. **Solution of the Differential Equation.** A solution satisfying the conditions that

\[n \to n_0 \text{ as } x \to 0\]

\( \ldots \quad (6) \)
is for $x > 0$:

$$n = n_0 \left[ 1 - \frac{2}{\sqrt{\pi}} \int_0^\frac{x}{\sqrt{4at}} e^{-u^2} \, du \right]$$ .... (7)

e. Example. As an example of the use of this formula treat the stream as being represented by the Taylors Ferry Section at a flow of 10000 $ft^3$ per sec. Hydraulic considerations give the flow as

$$F = Av = AC \sqrt{rS}$$ .... (8)

where $A$ represents the area of the cross section $(ft)^2$

$v$ the mean velocity of flow

$r$ the hydraulic radius

$S$ the slope maintaining the flow $F$

$C$ the Chezy coefficient

The change of flow as a function of the slope only is given by

$$\frac{\partial F}{\partial S} = \frac{AC\sqrt{r}}{2\sqrt{S}} = \frac{AC\sqrt{rS}}{2S} = \frac{F}{2S}$$

then

$$K = \frac{F}{2S}$$ .... (9)

and

$$\alpha = \frac{K}{T} = \frac{F}{2ST}$$ .... (10)

The wave front gradient is obtained from Equation (7) by differentiation with respect to $x$. Then,
\[
\frac{\Delta n}{\Delta x} = -\frac{x^2}{4at} \frac{\eta_0 e}{\sqrt{4at}}
\]

..... (11)

Application will be made to the reach of the Colorado River between Parker and Imperial Dams. The distance between these points is 147.0 miles or 776160 feet. The section at Taylors Ferry will be taken as representative of the river hydraulics.

From stage-flow tables for this section

\[ F = 10000 \text{ ft}^3/\text{sec} \]
\[ A = 3284 \text{ ft}^2 \]
\[ v = 3.045 \text{ ft/sec} \]
\[ S = 0.00020268 \text{ (dimensionless)} \]

The top width here is 354 feet but maps and photographs indicate that the top width is generally wider than this. A width of 800 feet will be used to obtain a realistic representation of the surface storage. This latter width is used in the evaluation of \( a \). Then

\[ a = \frac{10000}{(2)(0.00020268)(800)} = \frac{10000}{0.32428} = 30840 \text{ ft}^2/\text{sec} \]

With a velocity of 3.045 ft/sec the travel time between Parker and Imperial is

\[ \frac{776160}{3.045} = 254896 \text{ seconds or 2.950 days.} \]

The computation is made in the manner shown in Table 1. The first three columns are self-explanatory. Since the point of flow change is assumed to be carried along at the mean steam velocities the \( x \) distance is obtained by subtracting
Table 1

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Time (seconds)</th>
<th>( vt )</th>
<th>( x )</th>
<th>( \frac{x}{\sqrt{4at}} )</th>
<th>( \frac{x}{\sqrt{4at}} )</th>
<th>( \eta_1 )</th>
<th>( \eta_0 )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>776,160</td>
<td>0</td>
<td>0</td>
<td>( \infty )</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>43,200</td>
<td>131,544</td>
<td>644,616</td>
<td>73,000</td>
<td>8.830</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>86,400</td>
<td>263,088</td>
<td>513,072</td>
<td>103,200</td>
<td>4.972</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.5</td>
<td>129,600</td>
<td>394,632</td>
<td>381,528</td>
<td>126,400</td>
<td>3.018</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>2.0</td>
<td>172,800</td>
<td>526,176</td>
<td>249,984</td>
<td>146,000</td>
<td>1.712</td>
<td>0.01547</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>216,000</td>
<td>657,720</td>
<td>118,440</td>
<td>163,200</td>
<td>0.727</td>
<td>0.0389</td>
<td>0</td>
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<tr>
<td>3.0</td>
<td>259,200</td>
<td>789,264</td>
<td>-13,104</td>
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<td>302,400</td>
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<td>193,100</td>
<td>-0.749</td>
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<td>-1.337</td>
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<td>5.0</td>
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<td>230,800</td>
<td>-2.334</td>
<td>0.00096</td>
<td>0</td>
</tr>
</tbody>
</table>

\[ \bar{\eta} = \eta \quad \text{for } x > 0 \]

\[ \text{Flow due to increase in depth (ft}^3/\text{sec)} \]

\[ \frac{x^2}{4at} \quad \frac{x^2}{e^{4at}} \quad \frac{x^2}{\sqrt{\pi ax}} \]

\[ \text{Flow due to slope of wave front (ft}^3/\text{sec)} \]

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>( 2\eta_0 - \eta ) for ( x &lt; 0 ) (ft(^3)/sec)</th>
<th>( \frac{x^2}{4at} )</th>
<th>( e^{4at} )</th>
<th>( \frac{x^2}{\sqrt{\pi ax}} )</th>
<th>Flow due to wave front (ft(^3)/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>(10)(^6)x 0</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>16.53</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.5</td>
<td>0</td>
<td>0</td>
<td>9.14</td>
<td>0.00011</td>
<td>0.000098 0</td>
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<tr>
<td>2.0</td>
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<td>7.75</td>
<td>2.931</td>
<td>0.05334</td>
<td>0.4122 2.08</td>
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<td>2.5</td>
<td>0.06237</td>
<td>151.93</td>
<td>0.528</td>
<td>0.58978</td>
<td>4.0827 20.67</td>
</tr>
<tr>
<td>3.0</td>
<td>0.22212</td>
<td>541.08</td>
<td>.005</td>
<td>0.99501</td>
<td>6.2732 31.76</td>
</tr>
<tr>
<td>3.5</td>
<td>0.35108</td>
<td>855.23</td>
<td>.561</td>
<td>0.57064</td>
<td>3.3563 16.89</td>
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<tr>
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<td>970.65</td>
<td>1.798</td>
<td>1.16563</td>
<td>0.9072 4.59</td>
</tr>
<tr>
<td>4.5</td>
<td>0.40876</td>
<td>995.74</td>
<td>3.467</td>
<td>.03121</td>
<td>.1608 0.81</td>
</tr>
<tr>
<td>5.0</td>
<td>0.41030</td>
<td>999.49</td>
<td>5.448</td>
<td>.00430</td>
<td>.0210 0.11</td>
</tr>
</tbody>
</table>

\[ vT = (800)(3.045) = 2,436 \]

\[ 2\eta_0 = \frac{1,000}{(800)(3.045)} = 0.4105 \]

\[ \eta_0 = 0.20525 \text{ (ft)} \]

\[ \frac{\eta_0}{2S} = \frac{(10,000)(0.20525)}{(2)(0.00020268)} = 5,063,400 \]
<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Total flow increase (ft³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>1.5</td>
<td>0</td>
</tr>
<tr>
<td>2.0</td>
<td>9.83</td>
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<td>2.5</td>
<td>172.60</td>
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<tr>
<td>3.5</td>
<td>872.12</td>
</tr>
<tr>
<td>4.0</td>
<td>975.24</td>
</tr>
<tr>
<td>4.5</td>
<td>996.55</td>
</tr>
<tr>
<td>5.0</td>
<td>999.60</td>
</tr>
</tbody>
</table>
the distance moved by the point of flow change from the distance from Parker to Imperial. The next two columns are self-explanatory. The next column of figures is obtained from Formula 7. Values of \( \bar{\eta} \), for \( x \) positive, are obtained by multiplying the ratio \( \frac{n}{n_0} \) by \( n_0 \). When \( x \) becomes negative \( \bar{\eta} \) should be interpreted as \( \bar{\eta} = 2n_0 - \eta \). The value to be used for \( n_0 \) is one-half the increase in depth due to the increased flow. This depth is 0.4105 feet. It is computed on the supposition that, for small changes, the velocity of flow remains unchanged and the top width is 800 feet. The flow due to the increased depth is obtained by multiplying the increased area of the cross section by the mean stream velocity. The next three columns are used for figures needed for estimating the increased gradient due to the slope of the wave front as given by Formula (11). The flow due to the slope of the wave front is obtained by use of (9) and (11) from the relation

\[
f_1 = \frac{F n_0 e^{-\frac{x^2}{4a t}}}{2S \sqrt{vat}}
\]

\[\ldots\] \( (12) \)

This implies that the flow is proportional to the gradient for small changes. The total flow increase is the sum of the flow due to increased depth and the flow due to increased slope.
9. Check of the Value of the Constant $a$

a. **Basis of Computations.** Suppose a period can be found when the gages show a nearly sinusoidal flow variation during each day. If a solution of Equation (5) could be found to represent such a situation it would offer the possibility for determination of the constant $a$ directly from the performance of the river. Consider the solution

$$n = A e^{-am^2t} \sin(mx) + B e^{-am^2t} \cos(mx) \quad \ldots \quad (13)$$

where $A$ and $B$ are disposable constants. This solution represents a surface profile which is of sinusoidal shape for $-\infty < x < \infty$ when $t = 0$. This represents no actual condition but it may reasonably be supposed that the flattening of an actual undulatory profile may follow closely the pattern of one of the undulations as described by Formula (13).

b. **Application to the Study Reach.** To illustrate the use of this relation consider the situation on the river during the days of June 4, 5, and 6 of 1970. These are the last 3 days of a 6-day period during which the Parker releases were similar and ranged from about 4000 to 18000 ft$^3$/sec during each day. Flows read from gage charts for the Water Wheel and Cibola stations are as shown below.
<table>
<thead>
<tr>
<th>Day</th>
<th>Water Wheel</th>
<th>Cibola</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Max</td>
<td>Min</td>
</tr>
<tr>
<td>6-4-70</td>
<td>13000</td>
<td>5600</td>
</tr>
<tr>
<td>6-5-70</td>
<td>12600</td>
<td>5200</td>
</tr>
<tr>
<td>6-6-70</td>
<td>14500</td>
<td>6500</td>
</tr>
<tr>
<td></td>
<td>Total</td>
<td></td>
</tr>
<tr>
<td>Average</td>
<td>7600</td>
<td></td>
</tr>
</tbody>
</table>

Average amplitude $3800 \text{ ft}^3/\text{sec}$ $767 \text{ ft}^3/\text{sec}$

With the approximation being used the $\eta$ amplitudes should be proportional to the flow amplitudes. On this basis

$$\frac{\eta_2}{\eta_1} = \frac{767}{3800} = 0.201$$

From tables

$$e^{-1.604} = 0.201 \text{ and } \alpha m^2 t = 1.604$$

with a mean velocity of 3.045 seconds and a day of 86400 seconds

$$m = \frac{2\pi}{(86400)(3.045)} = 23.882(10)^{-6}.$$  

The distance between the Water Wheel and Cibola stations is about 366190 feet or 69.354 miles. Then the time $t$ required for an undulation to go from Water Wheel to Cibola would be

$$t = \frac{366190}{3.045} = 120259 \text{ seconds}$$

then

$$\omega = \frac{1.604}{m^2 t} = 23385 \text{ ft}^2/\text{sec}.$$
This compares with the 30840 ft$^2$/sec as derived previously from hydraulic considerations. The two values are probably as close as might reasonably be expected since determination of the first is influenced by day-to-day flow variations and determination of the second is subject to errors due to difficulties inherent in assignment of average values of hydraulic properties to a natural stream. Either would be good enough for an initial value. Experience would soon show what modifications might be desirable. Experience in the Parker-Imperial reach shows the constant $\alpha$ to be quite stable. A reason may be found in Formula (10) where it can be seen that reduction of the flow $F$ in a natural stream could be expected to bring with it a reduction in the top width $T$. At any rate, the value $\alpha = 30840$ ft$^2$/sec proves to be useful for both high-flow summer conditions and low-flow winter conditions.

10. **Computerized Computation Procedures**

a. **Computer Program.** The computer program predicts the hourly flows for the six stations of the Colorado River from Parker Dam to Imperial Dam during a desired time span. In addition to the flows, it is also possible to produce two types of graphical output: influence graphs and prediction graphs.
b. Equipment. The program was originally written for the Honeywell H-800 computer and was revised to also run on the CDC-1604 B computer. A Benson-Lehner Model J electroplotter is utilized to produce the graphical output. The data is transmitted via telephone, using a Xerox Telecopier II.

c. Computations. To calculate the downstream flows, the program uses a table of influence coefficients previously computed, which gives the effect of a unit flow for 1 hour at one station on the succeeding station's flows for the next 48 hours. The program thus has at least 48 hours of flow history prior to the start of a prediction. By adding the effect of each of the past 48 hours' flows at a station (N-1), and subtracting or adding a given flow constant, the flow at the following station (N) for a specific hour may be computed.

The program generates 240 influence factors which determine the wave shape and travel speed down the river, in accordance with river constants and desired percentage changes in wave magnitude and phase. Recalculating these influence factors will produce the changes in the traveling wave. These changes are made whenever a consistent prediction error is noted in either the peak-to-valley distance or the phase position of the wave profile in a reach.
Some details of experience with computations for the Parker-Imperial reach of the Colorado River may be of interest. For some reason flows in the Parker-Headgate Rock reach do not follow the pattern described by Formula (7). In this reach the propagation of Parker changes proceed as though carried by a bore wave in which inertia factors play a dominant role. The time required to traverse this 14.34-mile reach is just 1 hour. A computation of the velocity of the bore wave is

$$V_w = \sqrt{gD_m} + v = \sqrt{(32.2)(10)} + 3.0 = 20.9 \text{ ft/sec or } 15.7 \text{ mi/hr}$$

where $V_w$ represents the velocity of the bore wave with respect to an origin moving with the stream and $v$ the flow velocity. The quantities of $g$ and $D_m$ represent the acceleration of gravity and the mean depth respectively. This is a reasonably close check. If there were no friction the Parker releases would be propagated without change of shape. There is, of course, some friction and gage readings do show some rounding of the abrupt profiles of the Parker releases. In the actual computations the Parker releases are treated as coming from Headgate Rock but with an hour delay with respect to the time of release at Parker. No reason is known that would explain why the performance of the river is different in the Parker-Headgate Rock reach than it is in the remaining reaches between Headgate Rock and Imperial.

d. *Input Information.* The complete set of input data is checked for errors by the computer program and if any errors
are found, indications are printed and the computer computations are terminated.

e. **Output Information.** A complete set of output data consisting of flow predictions, daily totals and averages is produced (see Figure 3), while a condensed form of output more suitable for teletype transmission is also produced (see Figure 4). For the condensed form the time is given on a 12-hour clock, station names are abbreviated, and flow values are given for odd hours only, rounded to the nearest 1 cfs, and 2 days per page are printed.

One of the sets of plots which can be produced is S-shaped curves giving the influence of a step increment in flow on downstream stations for various stations on the river. These curves are calculated, using the influence factors described above (see Figure 5).

The second set of plots which can be produced is curves giving the historic and predicted flows at the various stations on the river. These curves are calculated using the maximum and minimum prediction and history values (see Figure 6).

The scales for these two types of plots can be varied, thereby producing the desired graphs most easily used.
**LOWER COLORADO RIVER FLOW PREDICTION 06/17/66**

- FOR THURSDAY, MARCH 31, 1966 -

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**TOTAL:** 252570 230280 219823 198890 192323 200257 193904

**AVERAGE:** 10524.9595.9159.8287.8013.8344.8679.

*Figure 3*
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**TUES TOTAL:** 196000  201118  177929  186258  191262  79338
**TUES AVE:**  8167  8380  7414  7761  7969  3306

| WED 1 AM | 8500 | 9820 | 6200 | 8470 | 7900 | 6190 |
| 3    | 3930 | 11030 | 6650 | 8300 | 8080 | 6690 |
| 5    | 3930 | 11550 | 7370 | 7970 | 8320 | 7140 |
| 7    | 3930 | 10870 | 8260 | 7590 | 8550 | 7520 |
| 9    | 8430 | 9350 | 9090 | 7260 | 8690 | 7810 |
| 11 PM | 8430 | 7770 | 9520 | 7070 | 8710 | 8020 |
| 1    PM | 8430 | 6670 | 9350 | 7060 | 8630 | 8150 |
| 3    | 13350 | 6480 | 8660 | 7260 | 8460 | 8210 |
| 5    | 13350 | 6920 | 7730 | 7620 | 8240 | 8240 |
| 7    | 13350 | 7660 | 6930 | 8080 | 8040 | 8250 |
| 9    | 13350 | 8870 | 6490 | 8500 | 7910 | 8260 |
| 11 PM | 12750 | 10280 | 6470 | 8770 | 7880 | 8290 |

**WED TOTAL:** 213780  215432  185762  188077  198874  186671
**WED AVE:**  8907  8976  7740  7837  8286  7778

Figure 4
INFLUENCE COEFFICIENT GRAPH

DAYS

PERCENT OF FLOW INCREMENT RECEIVED

HOURS

EFFECT ON DOWNSTREAM STATIONS OF A UNIT FLOW INCREASE AT PARKER DAM

Figure 5
PREDICTION GRAPH

Recorded flow

Predicted flow

CFS

0 6 12 18 24 6 12 18 24 6 12 18 24

SAT 3-24-66  SUN 3-25-66  MON 3-26-66  TUES 3-27-66

PREDICTION MADE 3-25-66 FOR TAYLORS FERRY

Figure 6
f. Arrangements for Making the Computations. The daily procedures used in processing and transmitting data between the Bureau of Reclamation offices in Denver, Colorado, and Boulder City, Nevada, are as follows:

1. Parker Dam releases, together with the flow changes at all stations, are transmitted via the teletypewriter from Boulder City, Nevada, to Denver, Colorado. (Data for phase shifts and magnitude changes for any station may also be included.)

2. The data is then processed on the H-800 computer.

3. The output data is immediately transmitted via the teletypewriter from Denver, Colorado, to Boulder City, Nevada.

4. If any plots are requested, they are produced at a later time and then sent via teletypewriter or regular mail to Boulder City, Nevada.

11. Summary

The 1953-1956 drought period in the Colorado River Basin and subsequent below normal runoff from the basin, together with the Decree of the Supreme Court of the United States, dated March 9, 1964, made
it necessary to improve the old system of monitoring transient flow changes of the Colorado River from Parker Dam to Imperial Dam, a distance of 147 miles. Lake Havasu above Parker Dam is the most downstream holdover storage for the delivery of domestic and irrigation water to United States and Mexican users along and at the terminus of the 147-mile reach of river.

Using the actual and scheduled releases at Parker Dam, there was a need to develop a method for predicting normal transient changes in riverflow as the flow traversed the reach. The recorded flow at monitored stations was to be compared with these predicted flows to determine storm inflow, gate malfunctions, changes in loss and return flow rates, and unscheduled changes in diversions.

The continual cyclic change in the transient flows throughout the reach due to generation of power to meet peak demands necessitated the development of a procedure that could be adapted to a computer, in order to make predicted flows available on a usable time schedule.

12. Conclusions

a. A method of estimating transient flow changes in a natural stream has been developed.

b. Daily comparisons made over a 5-year period on the 147-mile reach between the Parker and Imperial Dams on the
Colorado River have demonstrated its usefulness for tightening control of the river for the purpose of preventing waste of water.

c. The constant $a$ has been found to remain stable over a wide range of mean flows.

13. Acknowledgments

The authors express their appreciation to the Director of Design and Construction, EGR Center, Denver, Colorado, and to the Regional Director, Region 3, Boulder City, Nevada, for their permission to prepare this paper and for the cooperation of their respective offices in making available the necessary information.

Special recognition is due Mr. Charles M. Smith, Chief, Water Scheduling Branch, and Mr. Gordon B. Freeny, Supervisory Hydraulic Engineer, for their review of the paper.

Messrs. Carl F. Mayrose, Darwin Russell, Richard L. Sampson, and Richard J. Scanlon, also assisted in collection of data and preparation of the figures and charts and the authors express their thanks to them.
Propagation of Changes of Flow

This development will be based upon the following assumptions:

(1) The flow is controlled by friction.

(2) For changes varying by small amounts from the mean flow condition, flow changes can be assumed to be proportional to the slope changes.

(3) The factor of proportionality $K$ can be evaluated from the stage-flow curves.

(4) An increase of flow produces a step increase in stage which moves downstream at the mean velocity of flow $v$. Conditions near the step are shown on Figure 2.

If $f$ represents the change of flow associated with the step then

$$ f = -K \frac{\partial h}{\partial x} $$

... (1)

where $h$ represents the increase of depth and $x$ the distance measured from the step in the downstream direction.

The continuity condition is:

$$ \frac{\partial f}{\partial x} \, dx \, dt = -T \frac{\partial h}{\partial t} \, dx \, dt $$

where $T$ represents the top width. This relation can be put into the form:

$$ \frac{\partial f}{\partial x} = -T \frac{\partial h}{\partial t} $$

... (2)

Elimination of $f$ between (1) and (2) yields the differential equation:

$$ K \frac{\partial^2 h}{\partial x^2} = T \frac{\partial h}{\partial t} $$

... (3)
or if
\[ \alpha = \frac{\nu}{T} \] .... (4)

The relation takes the form
\[ \alpha \frac{\partial^2 \eta}{\partial x^2} = \frac{\partial \eta}{\partial t} \] .... (5)

A solution satisfying the conditions that
\[ \eta \rightarrow \eta_o \text{ as } x \rightarrow 0 \] .... (6)

is,
\[ \eta = \eta_o \left[ 1 - \frac{2}{\pi} \int_0^\infty e^{-\nu u^2} du \right] \] .... (7)

Example

As an example of the use of this formula, treat the stream as being represented by the Taylors Ferry Section at a flow of 10,000 ft³ per sec. Hydraulic considerations give the flow as
\[ F = Av = AC \sqrt{rS} \] .... (8)

where \( A \) represents the area of the cross section (ft)²
\( v \) the mean velocity of flow
\( r \) the hydraulic radius
\( S \) the slope maintaining the flow \( F \)
\( C \) the Chezy coefficient

The change of flow as a function of the slope only is given by
\[ \frac{\partial F}{\partial S} = \frac{AC \sqrt{r}}{2 \sqrt{S}} = \frac{AC \sqrt{rS}}{2S} = \frac{F}{2S} \]

then
\[ K = \frac{F}{2S} \] .... (9)
and

\[ \alpha = \frac{K}{T} = \frac{F}{2St} \quad \ldots \quad (10) \]

The wave front gradient is obtained from Equation (7) by differentiation with respect to \( x \). Then,

\[ \frac{\partial \eta}{\partial x} = -\frac{\eta_0 e^{-\frac{4\pi \alpha t}{\pi \alpha t}}}{\sqrt{\pi \alpha t}} \quad \ldots \quad (11) \]

Application will be made to the reach of the Colorado River between Parker and Imperial Dams. The distance between these points is 147.0 miles or 776.160 feet. The section at Taylors Ferry will be taken as representative of the river hydraulics. From stage-flow tables for this section

\[
\begin{align*}
F &= 10,000 \text{ ft}^3/\text{sec} \\
A &= 3,284 \text{ ft}^2 \\
v &= 3.045 \text{ ft/sec} \\
S &= 0.00020268 \text{ (dimensionless)}
\end{align*}
\]

The top width here is 354 feet but maps and photographs indicate that the top width is generally wider than this. A width of 800 feet will be used to obtain a realistic representation of the surface storage. This latter width is used in the evaluation of \( \alpha \). Then

\[ \alpha = \frac{10,000}{(2)(0.00020268)(800)} = \frac{10,000}{0.32428} = 30,840 \text{ ft}^2/\text{sec} \]

With a velocity of 3.045 ft/sec the travel time between Parker and Imperial is

\[ \frac{776.160}{3.045} = 254.896, \text{ seconds or 2.950 days.} \]

The computation is made in the manner shown in Table 1.
The first three columns are self-explanatory. Since the point of flow change is assumed to be carried along at the mean stream velocity the x distance is obtained by subtracting the distance moved by the point of flow change from the distance from Parker to Imperial. The next two columns are self-explanatory. The next column of figures is obtained from Formula 7. Values of \( \bar{n} \), for \( x \) positive, are obtained by multiplying the ratio \( \frac{n}{n_0} \) by \( n_0 \). When \( x \) becomes negative, \( \bar{n} \) should be interpreted as \( \bar{n} = 2n_0 - n \). The value to be used for \( n_0 \) is one-half the increase in depth due to the increased flow. This depth is 0.4105 feet. It is computed on the supposition that, for small changes, the velocity of flow remains unchanged and the top width is 800 feet. The flow due to the increased depth is obtained by multiplying the increased area of the cross section by the mean stream velocity. The next three columns are used for figures needed for estimating the increased gradient due to the slope of the wave front as given by Formula (11). The flow due to the slope of the wave front is obtained by use of (9) and (11) from the relation

\[
\bar{f}_i = \frac{F \eta_0}{2S} \frac{e^{-\frac{x^2}{4 \alpha t}}}{\sqrt{\pi \alpha t}}
\]

This implies that the flow is proportional to the gradient for small changes. The total flow increase is the sum of the flow due to increased depth and the flow due to increased slope.
Table 1
Base flow 10,000 (ft³/sec)  Top width 800 feet

Flow at Imperial due to 1,000 ft³/sec increase at Parker

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<th>(\frac{x}{\sqrt{4\alpha t}})</th>
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<td>5.448</td>
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<td>0.0210</td>
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</table>

\(vt = (800)(3.045) = 2,436\)

\(2\eta_0 = \frac{1,000}{(800)(3.045)} = 0.4105\) \(\eta_0 = 0.20525\) (ft)

\(\frac{F_{\eta_0}}{2S} = \frac{(10,000)(0.20525)}{(2)(0.0002688)} = 5,063,400\)
<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Total flow increase (ft³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
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<td>1.0</td>
<td>0</td>
</tr>
<tr>
<td>1.5</td>
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<td>9.83</td>
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<td>996.55</td>
</tr>
<tr>
<td>5.0</td>
<td>999.60</td>
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</table>
Check of the value of the constant $\alpha$

Suppose a period can be found when the gages show a nearly sinusoidal flow variation during each day. If a solution of Equation (5) could be found to represent such a situation it would offer the possibility for determination of the constant directly from the performance of the river. Consider the solution

$$\eta = Ae^{-\alpha m^2 t} \sin(mx) + Be^{-\alpha m^2 t} \cos(mx)$$

where $A$ and $B$ are disposable constants. This solution represents a surface profile which is of sinusoidal shape for $-\infty < x < \infty$ when $t = 0$. This represents no actual condition but it may reasonably be supposed that the flattening of an actual undulatory profile may follow closely the pattern of one of the undulations as described by Formula (13).

To illustrate the use of this relation consider the situation on the river during the days of June 4, 5 and 6 of 1970. These are the last three days of a six day period during which the Parker releases were similar and ranged from about 4000 to 18,000 ft$^3$/sec during each day. Flows read from gage charts for the Water Wheel and Cibola stations are as shown below.

<table>
<thead>
<tr>
<th>Day</th>
<th>Max</th>
<th>Min</th>
<th>Difference</th>
<th>Max</th>
<th>Min</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td>6-4-70</td>
<td>13000</td>
<td>5600</td>
<td>7400</td>
<td>9000</td>
<td>7000</td>
<td>2000</td>
</tr>
<tr>
<td>6-5-70</td>
<td>12600</td>
<td>5200</td>
<td>7400</td>
<td>8500</td>
<td>7400</td>
<td>1100</td>
</tr>
<tr>
<td>6-6-70</td>
<td>14500</td>
<td>6500</td>
<td>8000</td>
<td>8500</td>
<td>7000</td>
<td>1500</td>
</tr>
<tr>
<td>Total</td>
<td>42500</td>
<td>22800</td>
<td>2000</td>
<td>16500</td>
<td>46000</td>
<td>1500</td>
</tr>
<tr>
<td>Average</td>
<td>7600</td>
<td></td>
<td></td>
<td>1533</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Average amplitude</td>
<td>3800 ft$^3$/sec</td>
<td></td>
<td></td>
<td>767 ft$^3$/sec</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
With the approximation being used the \( \eta \) amplitudes should be proportional to the flow amplitudes. On this basis

\[
\frac{\eta_2}{\eta_1} = \frac{767}{3800} = 0.201
\]

From tables

\[
e^{-1.604} = 0.201 \quad \text{and} \quad \alpha \cdot m^2 t = 1.604
\]

with a mean velocity of \( 3.045 \) seconds and a day of 86400 seconds

\[
m = \frac{2\pi}{(86400)(3.045)} = 23.882 \times 10^{-6}
\]

The distance between the Water Wheel and Cibola stations is about 366,190 feet or 69.354 miles. Then the time \( t \) required for an undulation to go from Water Wheel to Cibola would be

\[
t = \frac{366,190}{3.045} = 120259 \text{ seconds}
\]

then

\[
\alpha = \frac{1.604}{m^2 t} = 23.385 \text{ ft}^2/\text{sec}
\]

This compares with the 30840 ft\(^2\)/sec as derived previously from hydraulic considerations. The two values are probably as close as might reasonably be expected since determination of the first is influenced by day to day flow variations and determination of the second is subject to errors due to difficulties inherent in assignment of average values of hydraulic properties to a natural stream. Either would be good enough for an initial value. Experience would soon show what modifications might be desirable. Experience in the Parker-Imperial reach shows the constant \( \alpha \) to be quite stable. A reason may be found in Formula (10) where it can be seen that reduction of the flow \( F \) in a natural stream could be expected to bring with it a reduction in the top width \( T \). At any rate, the value
\( \alpha = 30840 \text{ ft}^2/\text{sec} \) proves to be useful for both high flow summer conditions and low flow winter conditions.
Fig. 2. Wave profile.
1317 Denver Street
Boulder City, Nevada 89005
February 12, 1971

Mr. Robert E. Glover
1936 South Lincoln Street
Denver, Colorado 80210

Dear Bob:

We have made several minor changes in addition to those suggested in your letter of January 18. Reviews, typing and other events have caused the delay in my getting the revisions back to you and Chet.

Enclosed is a revised draft of my narrative and a first draft of my suggested summary, conclusions (yours) and my part of the acknowledgements. Do not hesitate to change my summary and acknowledgements as you see the need.

There are a couple of symbol and comma omissions in your draft, Bob, that you have probably found by now; page 3, sixth line from bottom of page; middle of next to the last page, should the (.) in 23,385 be (,) 23,385?

Sincerely,

[Signature]
John I. Sanders

JIS: s
Encl.

Copy to Chet Nelson
Memorandum

TO : Chief, Hydrology Branch
FROM : R. E. Glover
DATE: August 16, 1965

SUBJECT: Propagation of flow changes down the Colorado River

Purposes

Demands for irrigation water come into the Imperial Office on Wednesday of each week. Since it requires about 3 days for water released at Parker Dam to reach Imperial Dam, it is necessary to program the mean daily releases at Parker so that the water will be available at Imperial Dam when it is needed. This is done now by using factors derived from experience. It is suspected that these factors may be somewhat inaccurate because the constant demands on the river leave no opportunity for definitive experiments. There is good reason to suspect also that these factors may change somewhat with change of stage. A means of computing these factors from the stream properties would provide some new information on the characteristics of the response of the river to flow changes. It is the purpose of this memorandum to describe a formula for estimating these changes.

Propagation of Changes of Flow

This development will be based upon the following assumptions:

1. The flow is controlled by friction.

2. For changes varying by small amounts from the mean flow condition, flow changes can be assumed to be proportional to the slope changes.

3. The factor of proportionality $K$ can be evaluated from the stage-flow curves.

4. An increase of flow produces a step increase in stage which moves downstream at the mean velocity of flow $v$.

Conditions near the step are shown on Figure 1.

If $f$ represents the change of flow associated with the step then

$$f = - K \frac{\partial n}{\partial x}$$

... (1)
where $\eta$ represents the increase of depth and $x$ the distance measured from the step in the downstream direction.

The continuity condition is

$$\frac{\partial f}{\partial x} \ dx \ \frac{dt}{dt} = - \ dx \ \frac{\partial \eta}{\partial t} \ dt \ T$$

where $T$ represents the top width. This relation can be put into the form.

$$\frac{\partial f}{\partial x} = - T \ \frac{\partial \eta}{\partial t} \quad \ldots \ (2)$$

Elimination of $f$ between (1) and (2) yields the differential equation.

$$K \ \frac{\partial^2 \eta}{\partial x^2} = T \ \frac{\partial \eta}{\partial t} \quad \ldots \ (3)$$

or if

$$\alpha = \frac{K}{T} \quad \ldots \ (4)$$

The relation takes the form

$$\alpha \ \frac{\partial^2 \eta}{\partial x^2} = \frac{\partial \eta}{\partial t} \quad \ldots \ (5)$$

A solution satisfying the conditions that

$$\eta + \eta_0 \ \text{as} \ x \to 0 \quad \ldots \ (6)$$

is, for $x > 0$:
\[ \eta = \eta_0 \left[ 1 - \frac{2}{\sqrt{\pi}} \int_0^{\frac{x}{\sqrt{4\alpha t}}} e^{-u^2} \, du \right] \]  

... (7)

**Example**

As an example of the use of this formula we can treat the stream as being represented by the Taylors Ferry Section at a flow of 10,000 ft³ per sec. Hydraulic considerations give the flow as

\[ F = Av = AC \sqrt{r} \, S \]  

... (8)

where \( A \) represents the area of the cross section \((ft)^2\)

\( v \) the mean velocity of flow

\( r \) the hydraulic radius

\( S \) the slope maintaining the flow \( F \)

\( C \) the Chezy coefficient

The change of flow as a function of the slope only is given by

\[ \frac{\partial F}{\partial S} = \frac{AC\sqrt{r}}{2\sqrt{S}} = \frac{AC\sqrt{r}S}{2S} = \frac{F}{2S} \]

then

\[ K = \frac{F}{2S} \]

... (9)

and

\[ \alpha = \frac{K}{T} = \frac{F}{2ST} \]

... (10)

The wave front gradient is obtained from Equation (7) by differentiation with respect to \( x \). Then,

\[ \frac{\partial \eta}{\partial x} = -\frac{\eta_0 e^{-\frac{x^2}{4\alpha t}}}{\sqrt{\pi \alpha t}} \]

... (11)
Application will be made to the reach of the Colorado River between Parker and Imperial Dams. The distance between these points is 147.0 miles or 776,160 feet. The section at Taylors Ferry will be taken as representative of the river hydraulics. From stage-flow tables for this section

\[
F = 10,000 \, \text{ft}^3/\text{sec} \\
A = 3,284 \, \text{ft}^2 \\
v = 3.045 \, \text{ft/sec} \\
S = 0.000020268 \, \text{(dimensionless)}
\]

The top width here is $35\frac{1}{4}$ feet but maps and photographs indicate that the top width is generally wider than this. A width of 800 feet will be used to obtain a realistic representation of the surface storage. This latter width is used in the evaluation of $\alpha$. Then

\[
\alpha = \frac{10,000}{(2)(0.000020268)(800)} = \frac{10,000}{0.32428} = 30,840 \, \text{ft}^2/\text{sec}
\]

With a velocity of 3.045 ft/sec the travel time between Parker and Imperial is

\[
\frac{776,160}{3.045} = 254,896 \, \text{seconds or 2.950 days.}
\]

The computation is made in the manner shown in Table 1.

The first three columns are self-explanatory. Since the point of flow change is assumed to be carried along at the mean stream velocity the $x$ distance is obtained by subtracting the distance moved by the point of flow change from the distance from Parker to Imperial. The next two columns are self-explanatory. The next column of figures is obtained from the relation

\[
\frac{\eta_1}{\eta_0} = \left[1 - \frac{2}{\sqrt{x}} \int_0^{\sqrt{4\alpha t}} e^{-u^2} \, du \right] 
\]

\[
\ldots \ldots (12)
\]
Value of \( \bar{\eta} \), for \( x \) positive, are obtained by multiplying the ratio \( \eta_1/\eta_0 \) by \( \eta_0 \). When \( x \) becomes negative \( \bar{\eta} \) should be interpreted as \( \bar{\eta} = 2\eta_0 - \eta_1 \). The value to be used for \( \eta_0 \) is one half the increase in depth due to the increased flow. This depth is 0.4105 feet. It is computed on the supposition that, for small changes, the velocity of flow remains unchanged and the top width is 800 feet. The flow due to the increased depth is obtained by multiplying the increased area of the cross section by the mean stream velocity. The next three columns are used for figures needed for estimating the increased gradient due to the slope of the wave front as given by Formula (11). The flow due to the slope of the wave front is obtained by use of (9) and (11) from the relation

\[
f_1 = \frac{F \eta_0}{2S} \cdot \frac{x^2}{4\pi \alpha t}
\]

... (13)

This implies that the flow is proportional to the gradient for small changes. The total flow increase is the sum of the flow due to increased depth and the flow due to increased slope. The factor of increase applies to flows at the end of the day. A similar factor for the mean daily flow is given in the next to the last column. Increments are shown in the last column. They are estimated by using Simpson's rule methods and the total flow increase values.

**Comments**

The wave propagation pattern obtained from the formulas given herein agrees well with the performance of the actual stream. Because both the stream cross sections and top widths vary widely along the course of the stream it will be difficult to fix accurately the proper values of the constants to be used on the basis of the hydraulics of the stream. It may be better to adjust them by trial to represent the actual performance of the stream.
Fig. 1. Wave profile
Fig. 7
Flow increase at Imperial
due to a release at Parker
(Base flow 10,000 c.f.s.)

Mean for the day...
Increase for the day.
COLORADO RIVERFLOW MANAGEMENT

Introduction

1. Need for Close Control of the River.
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   b. Water Conservation Necessary.
   c. Parker Dam last Major Storage.
   d. Limited Storage at Imperial Dam.
   e. Senator Wash Facility.

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   c. Peaking Power Production at Parker Dam.
   d. Transit Time - Parker Dam to Imperial Dam.

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   b. Palo Verde Dam.
   c. Imperial Dam.
   d. Morelos Dam.

4. Return Flow and Losses
   b. Spillage Return Flow.
   c. Ground Water Gains and Losses
5. Inflow from Storms.
   a. Unpredictable Thunderstorm Inflow.
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7. Communications and Data Transmittal.
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   b. Gage Data Reporting Techniques.
   c. Predicted Flows.
COLORADO RIVER FLOW MANAGEMENT

Introduction

The Bureau of Reclamation of the Department of the Interior administers water deliveries from the Colorado River of the Southwest with its system of powerplants and storage reservoirs. This paper describes the needs and the procedures developed for improving the monitoring of transient flow changes of the Colorado River from Parker Dam to Imperial Dam. This reach of the river is shown on Figure 1.

1. Need for Close Control of the River

   a. Authority and Regulations. All operations of the Colorado River below Hoover Dam are made in accordance with the Colorado River Compact, the Boulder Canyon Project Act and its associated power and water delivery contracts, the Mexican Water Treaty, and the Decree of the Supreme Court of the United States in Arizona v. California, dated March 9, 1964. The requirements stipulated by these regulatory documents include operation for flood control, irrigation and domestic water release, and releases for power generation.

   b. Water Conservation Necessary. Each year the spring runoff from melting of the winter's snow accumulation on the west slope
of the Rocky Mountains is stored in Colorado River reservoirs. This is a part of the flood control operations of the river. However, there have been very few releases strictly for flood control during the past several years because of the drought period in the 1950's and a recurrence of below-normal runoff again during part of the last decade. The construction of Glen Canyon Dam on the Colorado River has vastly decreased the probability of having to make flood control releases that would waste water to the Gulf of California in the near future.

Early in the 1960's it became apparent to those responsible for operating the Lower Colorado River that, in order to prevent waste of water to the Gulf of California and inadvertant overdelivery to Mexico, it would be necessary to improve the system for monitoring the riverflows below Parker Dam. As a result of this realization and of studies made of available equipment, a new system of telemetering and data recording was installed to monitor flow changes in the river as it progressed downstream from Parker Dam to Imperial Dam, a distance of 147 miles.

c. Parker Dam Last Major Storage. Parker Dam, located near Parker, Arizona, is the most downstream major storage reservoir on
the Colorado River and is the point at which releases are made
to meet daily water orders for all downstream users. From a
point 16 miles below Parker Dam, the river traverses a wide valley
floor for 85 miles to the lower end of Cibola Valley. The river
channel is then confined by low desert mountains for 46 miles
before reaching Imperial Dam which is essentially the terminus of all
normal riverflows. Much of the river in the wide valley is subject
to a variety of water and channel control problems such as storm
inflow, ground water movement and shifting sediment.

d. Limited Storage at Imperial Dam. The storage in the forebay of
Imperial Dam is limited to approximately 1,000 acre-feet. This is
not very significant as far as meeting immediate increased demands
by water users below that point. This storage also is decreasing due
to sediment and phreatophyte growth above the dam. The need for
additional storage near Imperial Dam became more acute as the years
passed. Changing weather conditions placed critical demands on
riverflows arriving at Imperial Dam that could not be delivered
from Parker Dam storage in sufficient time to meet the needs of the
water users.

e. Senator Wash Facility. The need for this additional storage
near Imperial Dam resulted in the construction of Senator Wash Dam
on Senator Wash a short distance upstream from Imperial Dam. Senator Wash is an offstream site and the constructed facilities include capabilities for pumping to storage when a surplus of water arrives and generation of power when releases are made back to the river above Imperial Dam to meet unforeseen demands that are in excess of available river water. This facility was designed for use only in case of emergencies that could not be foreseen and supplied normally by regulation of releases at Parker Dam. It has greatly improved the availability of water in these emergency situations as well as providing space for storage of unpredictable flood inflow.

The pumping and release capacities at the plant at about 80 percent of maximum head are about 1,200 cfs as compared to flows in the river that range from 2,000 to 10,000 cfs. The active capacity of the reservoir is about 12,000 acre-feet. The reservoir active storage is normally maintained at about 7,000 acre-feet so that about 5,000 acre-feet of space is available to receive excess water or about 7,000 acre-feet of water is available to meet shortages in the river. Since the Senator Wash facility and the procedure for estimating and monitoring transient flow changes have been in operation, over-deliveries to Mexico and wastes to the Gulf of California have been negligible.

a. Water Orders. Requests for delivery of water are made to the Supervisor at Imperial Dam on Wednesday of each week. All water users below Parker Dam are required to submit to the Supervisor at Imperial Dam their daily estimates of water requirements for the following week (Monday through Sunday). These water requirements are combined by the Supervisor's office with estimates of transit losses. Routing computations are then made to determine the required
Parker releases to meet the demands along the river and at Imperial Dam. These water orders include those as far away as Coachella and Imperial Valleys, which may require as long as 5 or 6 days for delivery of water from Parker Dam to the farthest user in Coachella Valley.

b. Estimates of Daily Releases. The estimates of mean daily releases at Parker Dam are the result of a combination of the Mexican Treaty obligation, diversions at Imperial Dam, the estimated losses in the reach from Imperial Dam to Parker Dam, diversions return flows, at Palo Verde Dam, Headgate Rock Dam, and pumping diversions at several points along the river. These diversion requirements and estimates of losses combined with routing computation produce estimates of mean daily releases at Parker Dam to meet the downstream requirements as ordered for the week being scheduled. Estimates of daily requirements for the first and second advanced weeks are also included in the routings for estimates of Parker Dam mean daily releases beyond the scheduled week.

c. Peaking Power Production at Parker Dam. The capability for obtaining peaking power has been utilized at Parker Dam since its/
Because of the variation in the system loads during the day, hourly releases are scheduled to meet the portion of these varying requirements supplied by the Parker Dam powerplant. The installation at Parker Dam includes four (4) turbines and generators. These are utilized to the maximum advantage to meet peaking power requirements. The hourly rates of release during the day may vary from 2,000 cfs to approximately 20,000 cfs. This extreme variation in an approximately cyclical pattern causes added difficulty in routing flows and monitoring the riverflow as it progresses downriver toward Imperial Dam. This pattern of release not only complicates the routing procedure but disturbs the unstable river channel which is particularly troublesome at gaging stations where a channel control is required for a stage versus flow relationship. The variation of Parker releases produces continual shifting of the control at these stations and has been one of the major causes of constantly changing conditions where some of the flow monitoring equipment has been installed.

d. Transit Time, Parker to Imperial. The time necessary for water to flow from Parker Dam to Imperial Dam is approximately 72 hours. This time may vary with the condition of pondage above
Headgate Rock and Palo Verde Dams and with rising or falling stages. There is some variation in travel time from winter to summer flow conditions. The time during high summer flows may be as short as 60 hours. These travel times represent average velocities of from 2 to 2-1/2 miles per hour or roughly 3 to 3-1/2 feet per second. This travel time of from 2-1/2 to 3 days means that water users at the lower end of the reach must order water 3 days in advance in order to expect delivery of the required flows when needed. Water users also must anticipate any changes in their requirements from day to day and keep the Supervisor at Imperial Dam informed of these required changes so that corresponding changes can be made in Parker Dam releases to prevent shortages of requirements or overdeliveries at Imperial Dam. The Decree in Arizona v. California issued by the Supreme Court dated March 9, 1964, requires an accounting for water ordered but not taken as well as other pertinent records.

3. **Major Diversion Structures.**

There are four major diversion structures on the Colorado River downstream from Parker Dam; Headgate Rock Dam, Palo Verde Dam, Imperial Dam, and Morelos Dam, in downstream order.
a. **Headgate Rock Dam.** Headgate Rock Dam, located 14 miles downstream from Parker Dam, diverts water on the Arizona side of the river to the Colorado River Indian Irrigation Project. The dam is a gated structure consisting of ten radial gates in the river proper that maintain the water surface high enough to cause pondage upriver about 10 miles to within about 4 miles of Parker Dam. The radial gates operate automatically to maintain the normal operating water surface level but can be converted to manual control when necessary to lower the reservoir above the dam for emergencies or maintenance service. The reservoir is usually drained annually for maintenance work on the project canal and along the river reach above the dam. This operation imposes a problem for water control since the reservoir contains about 6,000 acre-feet of water. Special procedures are followed during this period which utilize the storage released from the lake above the dam to meet downstream requirements. Parker Dam releases are reduced by a flow sufficient to drain the reservoir at the desired rate. When the lake is refilled, the reverse procedure is followed.

b. **Palo Verde Dam.** Palo Verde Diversion Dam is located 59 miles downstream from Parker Dam and diverts water to the Palo Verde Irrigation District on the California side of the river. It has
three 50-foot radial gates in the river operated automatically to maintain a normal operating level in the forebay sufficiently high to permit diversion of water into the Palo Verde Canal. The dam is operated by the Palo Verde Irrigation District. Storage of water in the forebay above Palo Verde Dam is small compared to that above Headgate Rock Dam and does not impose a problem in the operation of the river. However, backwater above the dam extends for several miles and does affect the travel time in that reach to some extent. One or more of the river gates are opened periodically to sluice trash and sediment from the forebay. This does not present a problem when accomplished in a short period of time. However, when the river gates are opened to drain the forebay and the upper end of the canal for maintenance work and the gates remain open for more than a 24-hour period, consideration is given to the volume of water drained and Parker releases may be reduced accordingly. As in the case of the operation of Headgate Rock Dam during similar periods, the reverse procedure is followed when the forebay and the canal are refilled.

c. Imperial Dam. Imperial Dam is 147 miles downstream from Parker Dam. It was constructed for diversion of water into the
All-American Canal and the Gila Gravity Main Canal. The All-American Canal diverts water to the Reservation and Valley Divisions of the Yuma Project and to Imperial and Coachella Valleys. The Gila Gravity Main Canal diverts water east of the river to the North and South Gila Valleys, to the Wellton-Mohawk Irrigation and Drainage District, and to the Yuma Mesa areas. All the water arriving at Imperial Dam is accounted for by the Supervisor at Imperial Dam and any water passing Imperial Dam through the sluiceways or otherwise released to the river below Imperial Dam is normally scheduled for delivery to Mexico. Very infrequent flood releases from upstream reservoirs exceed Mexico's requirements. None have been made in recent years.

Senator Wash facility, although not considered a diversion structure, can divert water from the river immediately above Imperial Dam for storage in Senator Wash Reservoir. This is done only when flow arriving exceeds the requirements of the diverters at Imperial Dam. Imperial Dam is the nerve center for detailed operations of the Colorado River below Parker Dam. The Supervisor at Imperial Dam is responsible for correct delivery and operational accounting for all water released at Parker Dam and delivered to
diverters along the river and at Imperial Dam. He also is responsible for the delivery of water to Mexico. Techniques discussed later in this paper were developed to assist in the monitoring of the movement of water from Parker Dam to Imperial Dam.

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a. Drainage Return Flow. Surface drainage return flow is largely confined to a few major drainage channels in the valley area between Parker and Imperial Dams. This drainage return is mainly the result of irrigation applications and is fairly constant and therefore comparatively easy to account for. Gaging stations
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hydrographers and available to river operating personnel.

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operating problem along the river than drainage return flow, since
it cannot always be predicted or reported in time for normal
operating adjustments in Parker releases. Spills therefore require
special action by the Imperial Dam office to adjust riverflow
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return flows can normally be ironed out in the river within a
24-hour period. The Colorado River Indian Irrigation Project
normally maintains a constant level in its main canal by use of the
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directly into the river a few miles above Palo Verde Dam. This
spillage return reflects the variations of diversions from the
canal and is dependent upon the needs of the farmers for irrigation
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This can usually be detected at a monitored gaging station downstream
if not previously reported by a gate tender and adjustments can be
made to Parker releases to compensate for the change in scheduled
flows resulting from the malfunction.
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The following-described gaging stations on the river between Parker Dam and Imperial Dam were selected on the basis of their locations with respect to diversions, drainage returns, and adaptability to channel control for maintaining as nearly as possible relatively constant stage-flow relationships. The five gaging stations above Imperial were selected early in the 1960's for installation of special monitoring equipment because they would provide information about the transient flow changes between Parker and Imperial Dam as well as information on side inflow. (Refer to Figure 1 for the relative locations of these gaging stations.)

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downstream from Parker Dam and provides riverflow information on
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the river above the backwater from the Palo Verde/Dam. It also
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d. Taylor Ferry. Taylor Ferry gage is 27 miles downstream from Palo Verde Diversion Dam and is located at a point in the river where there are relatively minor changes due to shifting sediment and channel erosion. Dredging in the river below Taylor Ferry gage has caused some degradation of the control section. This gage was installed originally as a backup for the Cibola gage to provide Imperial Dam with information on the progress of the Parker mean daily releases as well as monitoring the diurnal flow changes as they progressed down the river.

e. Cibola. The Cibola gage is 24 miles below Taylor Ferry and is at the lower end of the Cibola Valley. Records of riverflow here include all surface returns from the Palo Verde and Cibola Valleys. This is a Geological Survey station and records at this point on the river are published in the "Water Supply Papers." Imperial Dam
monitors this station to determine the mean daily flow that arrives at the lower end of Cibola Valley in the Colorado River approximately one (1) day earlier than scheduled to arrive at Imperial Dam.

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7. **Communications and Data Transmittal**

a. **F. M. Radio System.** The Bureau of Reclamation maintains an F. M. radio system on the Lower Colorado River for voice communication and data transmittal. Voice communication is used for water
scheduling traffic between Imperial Dam, the Parker-Davis Project Office in Phoenix, Arizona, and the Regional Office in Boulder City, Nevada. Stream flow data stations and mobile units are used for hydrographic data transmittal from gaging stations and other field locations as needed. The monitoring of the gage at Cibola began early in the 1950's with a time programmed Stevens Telemark. Because of the need for backup data in case of equipment failure, a similar installation was later made at Taylor Ferry. Because of the impending need to monitor riverflows more closely in the 1960 decade, more sophisticated equipment was installed at Cibola and Taylor Ferry and similar equipment was added at the other upstream stations for additional monitoring of transient flow changes.

b. Gage Data Reporting Techniques. The stations between Parker and Imperial Dams were equipped with solid-state radios and encoders to transmit flow data to the base station. The base station was installed at Boulder City for interrogating all stations over the radio network. The base station included installation of strip-chart recorders to receive and chart each of the gage reports. The equipment was designed and programmed so that once each hour on the half-hour all stations were automatically called, in order, to give
a report. Except for Palo Verde Diversion Dam, potentiometers were installed in conjunction with Stevens type A recorders or Stevens Telemarks to convert shaft positions to a voltage in proportion to a river stage or flow. The encoders converted these voltages to frequencies for transmittal by radio. At Palo Verde Diversion Dam, potentiometers were installed and driven by Selsyn motors which fed gate shaft positions and differential stages data to the analog computers which in turn fed flow data to the encoder which transmitted tones to Boulder City the frequencies of which were in proportion to flows. Since the radio system was used for voice communications and continuous monitoring at the stations was not feasible because of battery drain, the receiving recorders were programmed to record data from each station for only about 19 seconds of each hour throughout the day. This resulted in a step type hydrograph which provided an effective tool for analysis of flow variation. Flows arriving at these stations were analyzed with respect to Parker releases and diversions along the river. However, it became necessary to route the cyclical changes from Parker power releases downstream through each reach of the river in order to have a more suitable basis for comparison of indicated flows arriving at each station and flows expected to arrive at
those stations. In order to make an analysis, a laborious procedure of routing was devised using a channel storage curve developed for each reach. These predicted flows were plotted on the recorder charts of each station in advance of and for comparison with the plotting of the actual recorded flows. In 1965, it became apparent that it would be necessary to computerize these predictions in order to keep them plotted sufficiently ahead of the actual recorded flows for analysis of the transient flows changes.
Summary:

The 1953-1956 drought period in the Colorado River Basin and subsequent below normal runoff from the basin, together with the Decree of the Supreme Court of the United States, dated March 9, 1954, made it necessary to improve the old system of monitoring transient flow changes of the Colorado River from Parker Dam to Imperial Dam, a distance of 147 miles. Lake Havasu above Parker Dam is the most downstream hold-over storage for the delivery of domestic and irrigation water to United States and Mexican users along and at the terminus of the 147 mile reach of river.

Using the actual and scheduled releases at Parker Dam, the need to develop a method for predicting normal transient changes in river flow as the flow traversed the reach. The recorded flow at monitored stations was to be compared with these predicted flows to determine storm inflow, gate malfunctions, changes in loss and return flow rates, and unscheduled changes in diversions.

The continual cyclic change in the transient flows throughout the reach due to generation of power to meet peak demands necessitated the development of a procedure that could be adapted to a computer, in order to make predicted flows available on a usable time schedule.

Conclusions

( ) A method of estimating transient flow changes in a natural stream has been developed.
Daily comparisons made over a five year period on the 147 mile reach between the Parker and Imperial Dams on the Colorado River have demonstrated its usefulness for tightening control of the river for the purpose of preventing waste of water.

The constant $\alpha$ has been found to remain stable over a wide range of mean flows.

Acknowledgements:

The author's express their appreciation to the Director of Design and Construction, E&R Center, Denver, Colorado, and to the Regional Director Region 3, Boulder City, Nevada, for their permission to prepare this paper and for the cooperation of their respective offices in making available the necessary information.

Special recognition is due Mr. Charles M. Smith, Chief, Water Scheduling Branch, and Mr. Gordon B. Freeny, Supervisory Hydraulic Engineer, for their review of the portion of the paper, prepared in Boulder City.

Mr. Carl F. Mayrose

Mr. Darwin Russell

Mr. Richard L. Sampson

Mr. Richard J. Scanlon

Also assisted in collection of data and preparation of maps and charts and the authors express their thanks to them.
Mr. John I. Sanders  
1317 Denver Street  
Boulder City, Nevada - 89005

Dear John:

The comments mentioned during the telephone conversation this morning between you and Nelson and me are as follows:

First sentence - your Page 1 - suggested wording......
Interior administrators water deliveries from the Colorado River...

Following the second sentence - in lieu of"Figure 1" This reach of the river is shown on Figure 1.

At end of item e, your Page 4, possibly two sentences could be added with profit to the reader. One sentence would give the storage and pump capacities at Senator Wash. The second would relate the Senator Wash capacities and the storage capacity behind Imperial to the flow of the river and the daily total flow. These sentences would give the reader some quantitative ideas about the necessity for close control of the river.

Page 5 item b - After Headgate Rock Dam, suggested, insert: "return flows".

Page 5 item c - Suggested wording "The possibilities for obtaining peaking power" have been utilized........

Sincerely yours,

[Signature]

Robert E. Glover

P.S. These suggestions are being sent because you indicated in your last letter that you were not entirely satisfied with the wording. If these suggestions help - fine. If not - file in Circular file.
Dec., 17, 1970

Dear Bob —

I'm a little later with this than I expected. The steno pool got real busy and delayed it a few days.

I don't feel too good about parts of the wording, and there's nothing sacred about any of it, so feel free to revise it to fit your ideas.

Happy Holidays

John
COLORADO RIVERFLOW MANAGEMENT

Introduction

1. Need for Close Control of the River.
   a. Authority and Regulations.
   b. Water Conservation Necessary.
   c. Parker Dam last Major Storage.
   d. Limited Storage at Imperial Dam.
   e. Senator Wash Facility.

   a. Water Orders.
   c. Peaking Power Production at Parker Dam.
   d. Transit Time - Parker Dam to Imperial Dam.

   a. Headgate Rock Dam.
   b. Palo Verde Dam.
   c. Imperial Dam.
   d. Morelos Dam.

   b. Spillage Return Flow.
5. Inflow from Storms.
   a. Unpredictable Thunderstorm Inflow.
   b. Predictable Major Storm Inflow.

6. Gaging Stations in the Parker Dam - Imperial Dam Reach.
   a. Headgate Rock Dam.
   b. Water Wheel.
   c. Palo Verde Dam.
   d. Taylor Ferry.
   e. Cibola
   f. Imperial Dam.

7. Communications and Data Transmittal.
   a. F.M. Radio System.
   b. Gage Data Reporting Techniques.
   c. Predicted Flows.
COLORADO RIVER FLOW MANAGEMENT

Introduction

The Bureau of Reclamation of the Department of the Interior operates water deliveries from the Colorado River of the Southwest with its system of powerplants and storage reservoirs. This paper describes the procedures developed for monitoring the transient flow changes of the Colorado River from Parker Dam to Imperial Dam. (Figure 1.)

1. Need for Close Control of the River.

a. Authority and Regulations. All operations of the Colorado River below Hoover Dam are made in accordance with the Colorado River Compact, the Boulder Canyon Project Act and its associated power and water delivery contracts, the Mexican Water Treaty, and the Decree of the Supreme Court of the United States in Arizona v. California, dated March 9, 1964. The requirements stipulated by these regulatory documents include operation for flood control, irrigation and domestic water release, and releases for power generation.

b. Water Conservation Necessary. Each year the spring runoff from melting of the winter's snow accumulation on the west slope
of the Rocky Mountains is stored in Colorado River reservoirs. This is a part of the flood control operations of the river. However, there have been very few releases strictly for flood control during the past several years because of the drought period in the 1950's and a recurrence of below-normal runoff again during part of the last decade. The construction of Glen Canyon Dam on the Colorado River has vastly decreased the probability of having to make flood control releases that would waste water to the Gulf of California in the near future.

Early in the 1960's it became apparent to those responsible for operating the Lower Colorado River that, in order to prevent waste of water to the Gulf of California and inadvertant overdelivery to Mexico, it would be necessary to improve the system for monitoring the riverflows below Parker Dam. As a result of this realization and of studies made of available equipment, a new system of telemetering and data recording was installed to monitor flow changes in the river as it progressed downstream from Parker Dam to Imperial Dam, a distance of 147 miles.

c. Parker Dam Last Major Storage. Parker Dam, located near Parker, Arizona, is the most downstream major storage reservoir on
the Colorado River and is the point at which releases are made to meet daily water orders for all downstream users. From a point 16 miles below Parker Dam, the river traverses a wide valley floor for 85 miles to the lower end of Cibola Valley. The river channel is then confined by low desert mountains for 46 miles before reaching Imperial Dam which is essentially the terminus of all normal riverflows. Much of the river in the wide valley is subject to a variety of water and channel control problems such as storm inflow, ground water movement and shifting sediment.

d. Limited Storage at Imperial Dam. The storage in the forebay of Imperial Dam is limited to approximately 1,000 acre-feet. This is not very significant as far as meeting immediate increased demands by water users below that point. This storage also is decreasing due to sediment and phreatophyte growth above the dam. The need for additional storage near Imperial Dam became more acute as the years passed. Changing weather conditions placed critical demands on riverflows arriving at Imperial Dam that could not be delivered from Parker Dam storage in sufficient time to meet the needs of the water users.

e. Senator Wash Facility. The need for this additional storage near Imperial Dam resulted in the construction of Senator Wash Dam
on Senator Wash a short distance upstream from Imperial Dam.
Senator Wash is an offstream site and the constructed facilities
include capabilities for pumping to storage when a surplus of water
arrives and generation of power when releases are made back to the
river above Imperial Dam to meet unforeseen demands that are in excess
of available river water. This facility was designed for use only
in case of emergencies that could not be foreseen and supplied
normally by regulation of releases at Parker Dam. It has greatly
enhanced the availability of water in these emergency situations
as well as providing space for storage of unpredictable flood inflow.
Since the Senator Wash facility has been in operation over-deliveries
to Mexico and wastes to the Gulf of California have been negligible.
The pumps have a capacity of


a. Water Orders. Requests for delivery of water are made to the
Supervisor at Imperial Dam on Wednesday of each week. All water
users below Parker Dam are required to submit to the Supervisor at
Imperial Dam their daily estimates of water requirements for the
following week (Monday through Sunday). These water requirements
are combined by the Supervisor's office with estimates of transit
losses. Routing computations are then made to determine the required
Parker releases to meet the demands along the river and at Imperial Dam. These water orders include those as far away as Coachella and Imperial Valleys, which may require as long as 5 or 6 days for delivery of water from Parker Dam to the farthest user in Coachella Valley.

b. Estimates of Daily Releases. The estimates of mean daily releases at Parker Dam are the result of a combination of the Mexican Treaty obligation, diversions at Imperial Dam, the estimated losses in the reach from Imperial Dam to Parker Dam, diversions at Palo Verde Dam, Headgate Rock Dam, and pumping diversions at several points along the river. These diversion requirements and estimates of losses combined with routing computation produce estimates of mean daily releases at Parker Dam to meet the downstream requirements as ordered for the week being scheduled. Estimates of daily requirements for the first and second advanced weeks are also included in the routings for estimates of Parker Dam mean daily releases beyond the scheduled week.

c. Peaking Power Production at Parker Dam. The advantages of hydropower have been utilized at Parker Dam since its construction.
Because of the variation in the system loads during the day, hourly releases are scheduled to meet the portion of these varying requirements supplied by the Parker Dam powerplant. The installation at Parker Dam includes four (4) turbines and generators. These are utilized to the maximum advantage to meet peaking power requirements. The hourly rates of release during the day may vary from 2,000 cfs to approximately 20,000 cfs. This extreme variation in an approximately cyclical pattern causes added difficulty in routing flows and monitoring the riverflow as it progresses downriver toward Imperial Dam. This pattern of release not only complicates the routing procedure but disturbs the unstable river channel which is particularly troublesome at gaging stations where a channel control is required for a stage versus flow relationship. The variation of Parker releases produces continual shifting of the control at these stations and has been one of the major causes of constantly changing conditions where some of the flow monitoring equipment has been installed.

d. Transit Time, Parker to Imperial. The time necessary for water to flow from Parker Dam to Imperial Dam is approximately 72 hours. This time may vary with the condition of pondage above
Headgate Rock and Palo Verde Dams and with rising or falling stages. There is some variation in travel time from winter to summer flow conditions. The time during high summer flows may be as short as 60 hours. These travel times represent average velocities of from 2 to 2-1/2 miles per hour or roughly 3 to 3-1/2 feet per second. This travel time of from 2-1/2 to 3 days means that water users at the lower end of the reach must order water 3 days in advance in order to expect delivery of the required flows when needed. Water users also must anticipate any changes in their requirements from day to day and keep the Supervisor at Imperial Dam informed of these required changes so that corresponding changes can be made in Parker Dam releases to prevent shortages of requirements or overdeliveries at Imperial Dam. The Decree in Arizona v. California issued by the Supreme Court dated March 9, 1964, requires an accounting for water ordered but not taken as well as other pertinent records.


There are four major diversion structures on the Colorado River downstream from Parker Dam, Headgate Rock Dam, Palo Verde Dam, Imperial Dam, and Morelos Dam, in downstream order.
a. **Headgate Rock Dam.** Headgate Rock Dam, located 14 miles downstream from Parker Dam, diverts water on the Arizona side of the river to the Colorado River Indian Irrigation Project. The dam is a gated structure consisting of ten (10) radial gates in the river proper that maintain the water surface high enough to cause pondage upriver about 10 miles to within about 4 miles of Parker Dam. The radial gates operate automatically to maintain the normal operating water surface level but can be converted to manual control when necessary to lower the reservoir above the dam for emergencies or maintenance service. The reservoir is usually drained annually for maintenance work on the project canal and along the river reach above the dam. This operation imposes a problem for water control since the reservoir contains about 6,000 acre-feet of water. Special procedures are followed during this period which utilize the storage released from the lake above the dam to meet downstream requirements. Parker Dam releases are reduced by a flow sufficient to drain the reservoir at the desired rate. When the lake is refilled, the reverse procedure is followed.

b. **Palo Verde Dam.** Palo Verde Diversion Dam is located 59 miles downstream from Parker Dam and diverts water to the Palo Verde Irrigation District on the California side of the river. It has
three 50-foot radial gates in the river operated automatically to maintain a normal operating level in the forebay sufficiently high to permit diversion of water into the Palo Verde Canal. The dam is operated by the Palo Verde Irrigation District. Storage of water in the forebay above Palo Verde Dam is small compared to that above Headgate Rock Dam and does not impose a problem in the operation of the river. However, backwater above the dam extends for several miles and does affect the travel time in that reach to some extent. One or more of the river gates are opened periodically to sluice trash and sediment from the forebay. This does not present a problem when accomplished in a short period of time. However, when the river gates are opened to drain the forebay and the upper end of the canal for maintenance work and the gates remain open for more than a 24-hour period, consideration is given to the volume of water drained and Parker releases may be reduced accordingly. As in the case of the operation of Headgate Rock Dam during similar periods, the reverse procedure is followed when the forebay and the canal are refilled.

c. Imperial Dam. Imperial Dam is 147 miles downstream from Parker Dam. It was constructed for diversion of water into the
All-American Canal and the Gila Gravity Main Canal. The All-American Canal diverts water to the Reservation and Valley Divisions of the Yuma Project and to Imperial and Coachella Valleys. The Gila Gravity Main Canal diverts water east of the river to the North and South Gila Valleys, to the Wellton-Mohawk Irrigation and Drainage District, and to the Yuma Mesa areas. All the water arriving at Imperial Dam is accounted for by the Supervisor at Imperial Dam and any water passing Imperial Dam through the sluiceways or otherwise released to the river below Imperial Dam is normally scheduled for delivery to Mexico. Very infrequent flood releases from upstream reservoirs exceed Mexico's requirements. None have been made in recent years.

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d. Morelos Dam. Morelos Dam is located on the Colorado River 26 miles below Imperial Dam and about 1 mile south of the Northerly International Boundary of the United States and Mexico. It serves to divert water into the Alamo Canal for delivery to Mexican water users. Water can be released to the river at Imperial Dam, at Yuma Main Canal Wasteway, and at Pilot Knob Powerplant for diversion at Morelos Dam to meet the United States obligation for delivery of water to Mexico.


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a. **Headgate Rock.** A station in the river about 2 miles downstream from Headgate Rock Dam was selected for monitoring the river below Headgate Rock Dam to reflect possible malfunction of the automatic gates as well as providing information on the transient flow changes of water released at Parker Dam. This station was particularly desirable because of the cyclical releases for power production at Parker Dam. It provides the first indication of the continuation of the cyclical flows as they progress downstream. Of particular importance is the record of the effect of storage above Headgate Rock Dam and the automatic gates on the cyclical flow.
b. Water Wheel. The Water Wheel gaging station is 40 miles downstream from Parker Dam and provides riverflow information on the river above the backwater from the Palo Verde Dam. It also reflects some of the drainage return flow from the Colorado River Indian Irrigation Project and its major spillage return flow. This gaging station is located at the point in the river where relatively stable rock controls the flow in the rated section of the channel and it is the most stable section of the river in the entire reach between Parker Dam and Imperial Dam for maintaining a stage-flow relationship. There is usually little inflow from storm runoff between Parker Dam and the Water Wheel Gage. This provides a good opportunity to identify and analyze flow travel time and attenuation of Parker Dam cyclical releases down to this point on the river. Below Water Wheel Gage, however, major side inflow and the effect of valley seepage has made the analysis of records somewhat more difficult.

c. Palo Verde. A special installation was made at Palo Verde Diversion Dam in cooperation with the Geological Survey and Palo Verde Irrigation District to monitor gate openings and differential heads on both the river and canal gates. Analog computers are used at the Dam to determine the flow through the river gates and through the canal gates to provide data at this installation for operational purposes. This installation affords an opportunity to further
study the attenuation of cyclical flow as well as to provide information on flood inflow between Water Wheel and Palo Verde Diversion Dam. Records of flow at Palo Verde Diversion Dam are published in the Geological Survey "Water Supply Papers."

d. Taylor Ferry. Taylor Ferry gage is 27 miles downstream from Palo Verde Diversion Dam and is located at a point in the river where there are relatively minor changes due to shifting sediment and channel erosion. Dredging in the river below Taylor Ferry gage has caused some degradation of the control section. This gage was installed originally as a backup for the Cibola gage to provide Imperial Dam with information on the progress of the Parker mean daily releases as well as monitoring the diurnal flow changes as they progressed down the river.

e. Cibola. The Cibola gage is 24 miles below Taylor Ferry and is at the lower end of the Cibola Valley. Records of riverflow here include all surface returns from the Palo Verde and Cibola Valleys. This is a Geological Survey station and records at this point on the river are published in the "Water Supply Papers." Imperial Dam
monitors this station to determine the mean daily flow that arrives at the lower end of Cibola Valley in the Colorado River approximately one (1) day earlier than scheduled to arrive at Imperial Dam.

f. *Imperial.* The flow arriving at Imperial Dam is computed as the sum of the water diverted to the All-American Canal, the water diverted to the Gila Gravity Main Canal, and flow released to the river immediately below Imperial Dam. These computations are made at the office of the Supervisor at Imperial Dam. Flows computed in this manner are recorded and used along with losses and diversions upstream to compare with releases at Parker Dam three (3) days earlier. The flow arriving at Imperial Dam also includes releases or pumping respectively at the Senator Wash facility and this pumping or the release is taken into consideration when comparing flow arriving at Imperial Dam as a result of Parker releases, losses, and diversions upstream.

7. **Communications and Data Transmittal**

a. *F. M. Radio System.* The Bureau of Reclamation maintains an F. M. radio system on the Lower Colorado River for voice communication and data transmittal. Voice communication is used for water
scheduling traffic between Imperial Dam, the Parker-Davis Project Office in Phoenix, Arizona, and the Regional Office in Boulder City, Nevada. Stream flow data stations and mobile units are used for hydrographic data transmittal from gaging stations and other field locations as needed. The monitoring of the gage at Cibola began early in the 1950's with a time programmed Stevens Telemark. Because of the need for backup data in case of equipment failure, a similar installation was later made at Taylor Ferry. Because of the impending need to monitor riverflows more closely in the 1960 decade, more sophisticated equipment was installed at Cibola and Taylor Ferry and similar equipment was added at the other upstream stations for additional monitoring of transit flow changes.

b. *Gage Data Reporting Techniques.* The stations between Parker and Imperial Dams were equipped with solid-state radios and encoders to transmit flow data to the base station. The base station was installed at Boulder City for interrogating all stations over the radio network. The base station included installation of strip-chart recorders to receive and chart each of the gage reports. The equipment was designed and programmed so that once each hour on the half-hour all stations were automatically called, in order, to give
a report. Except for Palo Verde Diversion Dam, potentiometers were installed in conjunction with Stevens type A recorders or Stevens telemarks to convert shaft positions to a voltage in proportion to a river stage or flow. The encoders converted these voltages to frequencies for transmittal by radio. At Palo Verde Diversion Dam, potentiometers were installed and driven by Selsyn motors which fed gate shaft positions and differential stages data to the analog computers which in turn fed flow data to the encoder which transmitted tones to Boulder City the frequencies of which were in proportion to flows. Since the radio system was used for voice communications and continuous monitoring at the stations was not feasible because of battery drain, the receiving recorders were programmed to record data from each station for only about 19 seconds of each hour throughout the day. This resulted in a step type hydrograph which was not objectionable for analysis of flow variation. Flows arriving at these stations were analyzed with respect to Parker releases and diversions along the river. However, it became necessary to route the cyclical changes from Parker power releases downstream through each reach of the river in order to have a more suitable basis for comparison of indicated flows arriving at each station and flows expected to arrive at
those stations. In order to make an analysis, a laborious procedure of routing was devised using a channel storage curve developed for each reach. These predicted flows were plotted on the recorder charts of each station in advance of and for comparison with the plotting of the actual recorded flows. In 1965, it became apparent that it would be necessary to computerize these predictions in order to keep them plotted sufficiently ahead of the actual recorded flows for analysis of the transient flows changes. This procedure was developed by Mr. Robert E. Glover and programmed on the computer in the Denver Office of the Bureau under the supervision of Mr. Chester A. Nelson. A development of this procedure and its further implementation is described in the material and the paragraphs that follow.
Propagation of Changes of Flow

This development will be based upon the following assumptions:

(1) The flow is controlled by friction.

(2) For changes varying by small amounts from the mean flow condition, flow changes can be assumed to be proportional to the slope changes.

(3) The factor of proportionality $K$ can be evaluated from the stage-flow curves.

(4) An increase of flow produces a step increase in stage which moves downstream at the mean velocity of flow $v$.

Conditions near the step are shown on Figure 1.\footnote{2}

If $f$ represents the change of flow associated with the step then

$$f = -K \frac{\partial n}{\partial x} \quad \ldots \quad (1)$$

where $n$ represents the increase of depth and $x$ the distance measured from the step in the downstream direction.

The continuity condition is:

$$\frac{\partial f}{\partial x} \frac{dx}{dt} = -T \frac{\partial n}{\partial t} \frac{dx}{dt}$$

where $T$ represents the top width. This relation can be put into the form:

$$\frac{\partial f}{\partial x} = -T \frac{\partial n}{\partial t} \quad \ldots \quad (2)$$

Elimination of $f$ between (1) and (2) yields the differential equation:

$$K \frac{\partial^2 n}{\partial x^2} = T \frac{\partial n}{\partial t} \quad \ldots \quad (3)$$
or if

\[ \alpha = \frac{K}{T} \]  

The relation takes the form

\[ \alpha \frac{\partial^2 \eta}{\partial x^2} = \frac{\partial \eta}{\partial t} \]  

A solution satisfying the conditions that

\[ \eta \rightarrow \eta_0 \text{ as } x \rightarrow 0 \]  

is,

\[ \eta = \eta_0 \left[ 1 - \frac{2}{\pi} \int_{0}^{x} e^{-u^2} du \right] \]  

**Example**

As an example of the use of this formula treat the stream as being represented by the Taylors Ferry Section at a flow of 10,000 ft\(^3\) per sec. Hydraulic considerations give the flow as

\[ F = A v = AC \sqrt{r S} \]  

where \( A \) represents the area of the cross section (ft\(^2\)), \( v \) the mean velocity of flow, \( r \) the hydraulic radius, \( S \) the slope maintaining the flow \( F \), and \( C \) the Chezy coefficient.

The change of flow as a function of the slope only is given by

\[ \frac{\partial F}{\partial S} = \frac{AC \sqrt{r}}{2 \sqrt{S}} = \frac{AC \sqrt{r S}}{2S} = \frac{F}{2S} \]

then

\[ K = \frac{F}{2S} \]  

(9)
and

\[ \alpha = \frac{K}{T} = \frac{F}{2ST} \quad \ldots \quad (10) \]

The wave front gradient is obtained from Equation (7) by differentiation with respect to \( x \). Then,

\[ \frac{\partial n}{\partial x} = -\frac{\eta e}{4\alpha t} \quad \sqrt{\pi \alpha t} \quad \ldots \quad (11) \]

Application will be made to the reach of the Colorado River between Parker and Imperial Dams. The distance between these points is 147.0 miles or 776,160 feet. The section at Taylors Ferry will be taken as representative of the river hydraulics. From stage-flow tables for this section

\[ F = 10,000 \text{ ft}^3/\text{sec} \]

\[ A = 3,284 \text{ ft}^2 \]

\[ v = 3.045 \text{ ft/sec} \]

\[ S = 0.00020268 \text{ (dimensionless)} \]

The top width here is 354 feet but maps and photographs indicate that the top width is generally wider than this. A width of 800 feet will be used to obtain a realistic representation of the surface storage. This latter width is used in the evaluation of \( \alpha \). Then

\[ \alpha = \frac{10,000}{(2)(0.00020268)(800)} = \frac{10,000}{0.32428} = 30,840 \text{ ft}^2/\text{sec} \]

With a velocity of 3.045 ft/sec the travel time between Parker and Imperial is

\[ \frac{776,160}{3.045} = 254,896 \text{ seconds or 2.950 days.} \]

The computation is made in the manner shown in Table 1.
The first three columns are self-explanatory. Since the point of flow change is assumed to be carried along at the mean stream velocity the $x$ distance is obtained by subtracting the distance moved by the point of flow change from the distance from Parker to Imperial. The next two columns are self-explanatory. The next column of figures is obtained from Formula 7. Values of $\bar{\eta}$, for $x$ positive, are obtained by multiplying the ratio $\frac{\eta}{\eta_0}$ by $\eta_0$. When $x$ becomes negative, $\bar{\eta}$ should be interpreted as $\bar{\eta} = 2\eta_0 - \eta$. The value to be used for $\eta_0$ is one-half the increase in depth due to the increased flow. This depth is 0.4105 feet. It is computed on the supposition that, for small changes, the velocity of flow remains unchanged and the top width is 800 feet. The flow due to the increased depth is obtained by multiplying the increased area of the cross section by the mean stream velocity. The next three columns are used for figures needed for estimating the increased gradient due to the slope of the wave front as given by Formula (11). The flow due to the slope of the wave front is obtained by use of (9) and (11) from the relation

$$f_i = F \frac{\eta_0}{2S} \frac{e}{4\alpha c} \quad \frac{\chi^2}{\pi \alpha t}$$

This implies that the flow is proportional to the gradient for small changes. The total flow increase is the sum of the flow due to increased depth and the flow due to increased slope.
Table 1

Base flow 10,000 (ft³/sec)  Top width 800 feet

Flow at Imperial due to 1,000 ft³/sec increase at Parker

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Time (seconds)</th>
<th>vt</th>
<th>x</th>
<th>(\sqrt{4\alpha t})</th>
<th>(\frac{x}{\sqrt{4\alpha t}})</th>
<th>(\eta_1)</th>
<th>(\eta_0)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>776,160</td>
<td>0</td>
<td>(\infty)</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>43,200</td>
<td>131,544</td>
<td>644,616</td>
<td>73,000</td>
<td>4.972</td>
<td>0.01547</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>86,400</td>
<td>263,088</td>
<td>513,072</td>
<td>103,200</td>
<td>3.018</td>
<td>0.30389</td>
<td>0</td>
</tr>
<tr>
<td>1.5</td>
<td>129,600</td>
<td>394,632</td>
<td>381,528</td>
<td>126,400</td>
<td>0.727</td>
<td>0.91777</td>
<td>0</td>
</tr>
<tr>
<td>2.0</td>
<td>172,800</td>
<td>526,176</td>
<td>249,984</td>
<td>146,000</td>
<td>0.073</td>
<td>2.8949</td>
<td>0</td>
</tr>
<tr>
<td>2.5</td>
<td>216,000</td>
<td>657,720</td>
<td>118,440</td>
<td>163,200</td>
<td>0.749</td>
<td>0.05865</td>
<td>0</td>
</tr>
<tr>
<td>3.0</td>
<td>259,200</td>
<td>789,264</td>
<td>13,104</td>
<td>178,800</td>
<td>-1.337</td>
<td>0.00846</td>
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<td>3.5</td>
<td>302,400</td>
<td>920,808</td>
<td>-144,648</td>
<td>193,100</td>
<td>-1.862</td>
<td>0.00096</td>
<td>0</td>
</tr>
<tr>
<td>4.0</td>
<td>345,600</td>
<td>1,052,352</td>
<td>-276,192</td>
<td>206,500</td>
<td>-2.334</td>
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<td>0</td>
</tr>
<tr>
<td>4.5</td>
<td>388,800</td>
<td>1,185,896</td>
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<td>219,000</td>
<td>-2.8949</td>
<td>0.00096</td>
<td>0</td>
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<tr>
<td>5.0</td>
<td>432,000</td>
<td>1,315,440</td>
<td>-539,280</td>
<td>230,800</td>
<td>-3.456</td>
<td>0.00096</td>
<td>0</td>
</tr>
</tbody>
</table>

\(\bar{\eta} = \eta\) for \(x > 0\)  Flow due to increase in depth (ft³/sec)

\(x^2\)  \(\frac{x^2}{4\alpha t}\)  \(\frac{-x^2}{e^{4\alpha t}}\)  Flow due to slope of wave front (ft³/sec)

<table>
<thead>
<tr>
<th>Time (days)</th>
<th>for (x &lt; 0)</th>
<th>in depth</th>
<th>(\frac{x^2}{4\alpha t})</th>
<th>(\frac{-x^2}{e^{4\alpha t}})</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>(10)^2 x 0</td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.5</td>
<td>0</td>
<td>0</td>
<td>9.14</td>
<td>0.00011</td>
</tr>
<tr>
<td>2.0</td>
<td>0.00318</td>
<td>7.75</td>
<td>2.931</td>
<td>0.03334</td>
</tr>
<tr>
<td>2.5</td>
<td>0.06237</td>
<td>151.93</td>
<td>0.528</td>
<td>0.5978</td>
</tr>
<tr>
<td>3.0</td>
<td>0.22212</td>
<td>541.08</td>
<td>0.005</td>
<td>0.57064</td>
</tr>
<tr>
<td>3.5</td>
<td>0.55108</td>
<td>855.23</td>
<td>0.561</td>
<td>0.57064</td>
</tr>
<tr>
<td>4.0</td>
<td>0.39846</td>
<td>970.65</td>
<td>1.798</td>
<td>0.7072</td>
</tr>
<tr>
<td>4.5</td>
<td>0.40876</td>
<td>995.74</td>
<td>3.467</td>
<td>0.3121</td>
</tr>
<tr>
<td>5.0</td>
<td>0.41030</td>
<td>999.49</td>
<td>5.448</td>
<td>0.0430</td>
</tr>
</tbody>
</table>

\[vt = (800)(3.045) = 2436\]

\[2\eta_0 = \frac{1,000}{(800)(3.045)} = 0.4105\]

\[\eta_0 = 0.20525 \text{ (ft)}\]

\[\frac{F_n_o}{2S} = \frac{(10,000)(0.20525)}{(2)(0.00020268)} = 5,063,400\]
<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Total flow increase (ft³/sec)</th>
<th>Factor of increase (dimensionless)</th>
<th>Main increase for the day (ft³/sec)</th>
<th>Decrease of rate of rise (ft³/sec)</th>
<th>Final rate of rise (ft³/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>0.5</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
</tr>
<tr>
<td>1.0</td>
<td>0</td>
<td>1.64</td>
<td>1.64</td>
<td>0</td>
<td>1.64</td>
</tr>
<tr>
<td>1.5</td>
<td>9.83</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0</td>
<td>172.60</td>
<td>0</td>
<td>272.60</td>
<td>210.54</td>
<td>512.10</td>
</tr>
<tr>
<td>2.5</td>
<td>572.84</td>
<td>0.59</td>
<td>839.42</td>
<td>627.34</td>
<td></td>
</tr>
<tr>
<td>3.5</td>
<td>872.12</td>
<td>0.91</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.0</td>
<td>975.24</td>
<td>0.91</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4.5</td>
<td>966.55</td>
<td>0.91</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.0</td>
<td>999.60</td>
<td>1.00</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
Check of the value of the constant $\alpha$

Suppose a period can be found when the gages show a nearly sinusoidal flow variation during each day. If a solution of Equation (5) could be found to represent such a situation it would offer the possibility for determination of the constant $\alpha$ directly from the performance of the river. Consider the solution

$$\eta = Ae^{-\alpha m^2 t} \sin(mx) + Be^{-\alpha m^2 t} \cos(mx) \quad \ldots \quad (13)$$

Where $A$ and $B$ are disposable constants. This solution represents a surface profile which is of sinusoidal shape for $-\infty < x < \infty$ when $t = 0$. This represents no actual condition but it may reasonably be supposed that the flattening of an actual undulatory profile may follow closely the pattern of one of the undulations as described by Formula (13).

To illustrate the use of this relation consider the situation on the river during the days of June 4, 5 and 6 of 1970. These are the last three days of a six day period during which the Parker releases were similar and ranged from about 4000 to 18,000 ft³/sec during each day. Flows read from gage charts for the Water Wheel and Cibola stations are as shown below.

<table>
<thead>
<tr>
<th></th>
<th>Water Wheel</th>
<th></th>
<th>Cibola</th>
</tr>
</thead>
<tbody>
<tr>
<td>Day</td>
<td>Max</td>
<td>Min</td>
<td>Difference</td>
</tr>
<tr>
<td>6-4-70</td>
<td>13000</td>
<td>5600</td>
<td>7400</td>
</tr>
<tr>
<td>6-5-70</td>
<td>12600</td>
<td>5200</td>
<td>7400</td>
</tr>
<tr>
<td>6-6-70</td>
<td>14500</td>
<td>6500</td>
<td>8000</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>22800</strong></td>
<td><strong>22500</strong></td>
<td><strong>4600</strong></td>
</tr>
<tr>
<td><strong>Average</strong></td>
<td>7600</td>
<td></td>
<td>1533</td>
</tr>
<tr>
<td><strong>Average amplitude</strong></td>
<td>3800 ft³/sec</td>
<td></td>
<td>767 ft³/sec</td>
</tr>
</tbody>
</table>
With the approximation being used the \( \eta \) amplitudes should be proportional to the flow amplitudes. On this basis

\[
\frac{\eta_a}{\eta_i} = \frac{767}{3800} = 0.201
\]

From tables

\[ e^{-1.604} = 0.201 \quad \text{and} \quad \alpha m^2 t = 1.604 \]

with a mean velocity of 3.045 seconds and a day of 86400 seconds

\[ m = \frac{2 \pi}{(86400)(3.045)} = 23.882(10)^{-6} \]

The distance between the Water Wheel and Cibola stations is about 366,190 feet or 69.354 miles. Then the time \( t \) required for an undulation to go from Water Wheel to Cibola would be

\[ t = \frac{366.190}{3.045} = 120259 \text{ seconds} \]

then

\[ \frac{\alpha}{m^2 t} = \frac{1.604}{23.385} = 23.385 \text{ ft}^2/\text{sec} \]

This compares with the 30840 ft\(^2\)/sec as derived previously from hydraulic considerations. The two values are probably as close as might reasonably be expected since determination of the first is influenced by day to day flow variations and determination of the second is subject to errors due to difficulties inherent in assignment of average values of hydraulic properties to a natural stream. Either would be good enough for an initial value. Experience would soon show what modifications might be desirable. Experience in the Parker-Imperial reach shows the constant \( \alpha \) to be quite stable. A reason may be found in Formula (10) where it can be seen that reduction of the flow \( F \) in a natural stream could be expected to bring with it a reduction in the top width \( T \). At any rate, the value
\( \alpha = 30840 \text{ ft}^2/\text{sec} \) proves to be useful for both high flow summer conditions and low flow winter conditions.

A comparison of estimated and observed maximum and minimum flows for the day of June 6, 1970 is shown in Table 2.

The sequence Min-Max-Min. The reverse indicates the sequence observed during the day at the station. The values are as read from recorder charts.

(Table 2)
Conclusions

( ) A method of estimating transient flow changes in a natural stream has been developed.

( ) Daily comparisons made over a five year period on the 147 mile reach between the Parker and Imperial Dams on the Colorado River have demonstrated its usefulness for tightening control of the river for the purpose of preventing waste of water.

( ) The constant $\infty$ has been found to remain stable over a wide range of mean flows.
Fig 1. Wave profile.
The computer program predicts the hourly flows for the six stations of the Colorado River from Parker Dam to Imperial Dam during a desired time span. In addition to the flows, it is also possible to produce two types of graphical output: influence graphs and prediction graphs.

The program was originally written for the Honeywell H-800 computer and was revised to also run on the CDC-1604 B computer. A Benson-Lehner Model J electro-plotter is utilized to produce the graphical output. The data is transmitted via telephone, using a Xerox Teletypewriter II.

To calculate the downstream flows, the program uses a table of influence coefficients previously computed, which gives the effect of a unit flow for 1 hour at one station on the succeeding station's flows for the next 48 hours. The program thus has at least 48 hours of flow history prior to the start of a prediction. By adding the effect of each of the past 48 hours' flows at a station \((N-1)\), and subtracting or adding a given correction constant, the flow at the following station \((N)\) for a specific hour may be computed.

The program generates 240 influence factors which determine the wave shape and travel speed down the river, in accordance with river constants and desired percentage changes in wave magnitude and phase. Recalculating these influence factors will produce the changes in the traveling wave. These changes are made whenever a consistent prediction error is noted in either the peak-to-valley distance or the phase of the wave in a reach.
The complete set of input data is checked for errors by the computer program and if any errors are found, indications are printed and the computer computations are terminated.

A complete set of output data consisting of flow predictions, daily totals, and averages is produced (see Figure 1), while a condensed form of output more suitable for teletype transmission is also produced (see Figure 2). For the condensed form the time is given on a 12-hour clock, station names are abbreviated, and flow values are given for odd hours only, rounded to the nearest lfs, and 2 days per page are printed.

One of the sets of plots which can be produced is S-shaped curves giving the influence of a step increment in flow on downstream stations for various stations on the river. These curves are calculated, using the influence factors described above (see Figure 3).

The second set of plots which can be produced is curves giving the historic and predicted flows at the various stations on the river. These curves are calculated using the maximum and minimum prediction and history values (see Figure 4).

The scales for these two types of plots can be varied, thereby producing the desired graphs most easily used.

The daily procedures used in processing and transmitting data between the Bureau of Reclamation offices in Denver, Colorado, and Boulder City,
Nevada, are as follows:

1. Parker Dam releases, together with the flow changes at all stations, are transmitted via the teletypewriter from Boulder City, Nevada, to Denver, Colorado. (Data for phase shifts and magnitude changes for any station may also be included.)

2. The data is then processed on the H-800 computer.

3. The output data is immediately transmitted via the teletypewriter from Denver, Colorado, to Boulder City, Nevada.

4. If any plots are requested, they are produced at a later time and then sent via teletypewriter or regular mail to Boulder City, Nevada.
## Lower Colorado River Flow Prediction 06/17/66

For Thursday, March 31, 1966

<table>
<thead>
<tr>
<th>HOUR</th>
<th>Parker Dam</th>
<th>Headgate Rock</th>
<th>Water Wheel</th>
<th>Palo Verde</th>
<th>Taylor's Ferry</th>
<th>Cibola</th>
<th>Imperial Dam</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4650</td>
<td>8260</td>
<td>11473</td>
<td>6769</td>
<td>8403</td>
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|      | 3    | 3930   | 11030  | 6650   | 8300   | 8080   | 6690   |
|      | 5    | 3930   | 11550  | 7370   | 7970   | 8320   | 7140   |
|      | 7    | 3930   | 10870  | 8260   | 7590   | 8550   | 7520   |
|      | 9    | 8430   | 9350   | 9090   | 7260   | 8690   | 7810   |
|      | 11   | 8430   | 7770   | 9520   | 7070   | 8710   | 8020   |
|      | PM   | 8430   | 6670   | 9350   | 7060   | 8630   | 8150   |
|      |      | 13350  | 6480   | 8660   | 7260   | 8460   | 8210   |
|      |      | 13350  | 6920   | 7730   | 7620   | 8240   | 8240   |
|      |      | 13350  | 7660   | 6930   | 8080   | 8040   | 8250   |
|      |      | 13350  | 8870   | 6490   | 8500   | 7910   | 8260   |
|      |      | 12750  | 10280  | 6470   | 8770   | 7880   | 8290   |
| WED TOTAL: | 213780 | 215432 | 185762 | 188077 | 198874 | 186671 |
| WED AVE :  | 8907   | 8976   | 7740   | 7837   | 8286   | 7778   |
Figure 3

Effect on downstream stations of a unit flow increase at Parker Dam
Figure 4

Prediction Graph
[Half Normal Size]

Prediction made 3/25/66 for Taylors Ferry
Machine computation

The solution of Formulas (7) expresses the changes due to a release initiated at time zero and maintained thereafter. The data on which computations must be based express the total flow passing a station at a given time and it is desirable to adapt the computing procedure to use values of this kind. To make this adaptation a table like Table 1 is first made but is computed out for time intervals of one hour. The total flow increases of the last column are then differenced by subtracting from each value the one immediately preceding it. The values so obtained represent the flow at the lower station due to a flow originating at the upper station at time zero and maintained for one hour only. The flows so obtained rise from zero to a maximum and then decrease to zero. In what follows it will be supposed that the sequence has been computed for a flow increase of one cubic foot per second maintained for one hour. The factors so obtained will be referred to hereafter as the unit flow function.

Because stage and flow increases are assumed to be proportional, the computation is made in terms of flow rather than in terms of stage, as expressed in Formula (7). Since provision for including the effects of diversions, return flows and losses must be made, the computations are made for the reaches between gaging stations. The flows at each gaging station, coming from upstream, are first computed and to these are added the flow changes originating at the station. These combined flows then become the basis for the computations of the flows at the next station downstream and so the computations proceed down the river.
To compute the flow at a downstream station the computer must sum the effects of hourly flows at the upstream station over a period long enough so that the factors obtained from the unit flow function have run out. Also, if the flow values for the upstream station are arranged to run forward in time then the factors from the unit flow function must be arranged in reverse order. To fix ideas assume that the computation is to be made for the tenth hour at the downstream station. The flow originating at the ninth hour will be the first which could contribute to flow at the downstream station and the factor to use from the unit flow function to estimate its effect is the factor for 1 hour. Likewise the flow originating at the eighth hour produces a flow at the downstream station at the tenth hour which can be estimated by multiplying the flow originating at the upstream station at the eighth hour by the factor for two hours coming from the unit flow function. The products of flows and unit flow function factors must be accumulated until the unit flow function factors again become zero. For computation of Colorado River flows it is necessary to arrange for the machine to retain a record of flows at the several stations for two days previous to the day for which the computation is being made. The unit flow function factors are stored in the machine's memory. The number of these factors will depend on the character of the stream and the distance between stations. There are less than one hundred of these factors when the computation is carried from station to station in the Parker-Imperial reach of the Colorado River. Hand computation of these flows would, admittedly, be a tedious
procedure but the digital computer makes short work of it. A
days computation for the Colorado River stations consumes about
seconds of computer time.

Some details of experience with computations for the Parker-
Imperial reach of the Colorado River may be of interest. For
some reason flows in the Parker-Headgate Rock reach do not
follow the pattern described by Formula (7). In this reach the
propagation of Parker changes proceed as though carried by a
bore wave, in which inertia factors play a dominant role. The
time required to traverse this 14.34 mile reach is just one
hour. A computation of the velocity of the bore wave is

\[ u_\text{w} = \sqrt{g D_m} - u = \sqrt{32.2 (10)} - 3.0 = 20.9 \text{ ft/sec or } 15.7 \text{ mi/hr} \]

Where \( u_\text{m} \) represents the velocity of the bore wave with respect
to an origin moving with the stream and \( u \) the flow velocity.
The quantities \( g \) and \( D_m \) represent the acceleration of
gravity and the mean depth respectively. This is a reasonably
close check. If there were no friction the Parker releases
would be propagated without change of shape. There is, of course,
some friction and gage readings do show some rounding of the
abrupt profiles of the Parker releases. In the actual computa-
tions the Parker releases are treated as coming from Headgate
Rock but with an hour delay with respect to the time of release
at Parker. No reason is known that would explain why the perform-
ance of the river is different in the Parker-Head Rock reach than
it is in the remaining reaches between Headgate Rock and Imperial.
In the fall when flows in the river are decreasing a period is sometimes observed when the river responds sluggishly to the Parker releases. The sluggish period is terminated by a rush of water into Imperial, after which, the river again answers the releases. Silting is believed to be the cause.
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* Gaging station is about 2 miles below the diversion dam.
Mr. B.F. Bellport, Director
Office of Design and Construction
U.S. Bureau of Reclamation
Building 67 Denver Federal Center
Denver, Colorado - 80225

1936 South Lincoln Street
Denver, Colorado - 80210
December 15, 1970

Attention of
Mr. Chester A. Nelson

Dear Mr. Bellport:

Please refer to your letter of November 19, 1970 and previous correspondence.

A draft of My paragraphs to be included in the paper being prepared jointly by Messrs. Sanders, Nelson and Glover are forwarded herewith. Some items for the "Conclusions" are included.

Sincerely yours,

[Signature]
Robert E. Glover
1936 South Lincoln Street  
Denver, Colorado - 80210  
December 15, 1970

Dear John:

A draft of my treatment of items (7) and (8) is forwarded herewith for your file.

Some items for the conclusions are included.  
I have a copy for Mr. Nelson and will get it to him soon.

Sincerely yours,

[Signature]

Robert E. Glover
1936 South Lincoln Street
Denver, Colorado - 80210
December 14, 1970

Dear John:

Many thanks for the material sent with your letter of December 8, 1970. They will permit me to finish my items 7 and 8 which I expect to complete today.

By using the records for Water Wheel and Cibola I can get a diffusion constant $\alpha = 23385 \text{ ft}^2/\text{sec}$. This is determined from the performance of the river. The one now being used $\alpha = 30840 \text{ ft}^2/\text{sec}$ was determined from the average steam hydraulics. Considering the uncertainties this is a good check.

We will need some conclusions. I will write up some summarizing the results of items 7 and 8.

Sincerely yours,

Robert E. Glover
Bob -

There are no spillways or x-sections above Headgate Rock Dam. Looking at the profile I would guess 10' as about the average depth Headgate to Parker Dam.

You can get the normal operating levels and Tailwater elevations from the enclosed Profile.

I'm sending charts showing the average 6-hour flows at Imperial - we found that hourly flows and maximum & minimum instantaneous flows had little meaning because of the number of stations involved in computing the flows arriving at Imperial Dam.

Note that differences in predicted and actual flows on the charts are a combination of losses and errors in everything. We considered the errors were relatively constant on an hour to hour basis. Sudden differences, if not malfunctions which could be quickly identified, were water changes.
The Palo Verde charts show the greatest deviation of predicted and actual flows. Leaks on the trash racks and sediment build-up are the probable cause of much of these differences. All these things must be taken into consideration when analyzing the flows. If you think this period has too much difference at the stations, we could search for a better period. I'm not sure we can find one.

Let me know what you think after you have looked over the charts.

Sincerely,

John

P.S. I'm also enclosing a draft of revised outline as you suggested.
Colorado River Flow Management

(1) Need for Close Control of the River.
   (b) Parker Dam last major storage.
   (c) Limited storage at Imperial Dam.
   (d) Senator Wash facility.

(2) Management of Releases.
   (a) Requests come into Imperial on Wednesday.
   (b) Estimate of daily release.
   (c) Peaking Power Production at Parker Dam.
   (d) Transit time—Parker to Imperial.

(3) Major Diversion Structures
   (a) Headgate Rock Dam.
   (b) Palo Verde Dam.
   (c) Imperial Dam.
   (d) Automatic gate operations.

(4) Return Flow
   (a) Drainage return flow.
   (b) Spillage return flow.

(5) Inflow from Storms
   (a) Unpredictable thunderstorm inflow.
   (b) Predictable major storm inflow.
(6) Gaging Stations in the Parker-Imperial Reach.
   (a) Headgate Rock.
   (b) Water Wheel.
   (c) Palo Verde.
   (d) Taylors Ferry
   (e) Cibola.
   (f) Imperial.

(7) Communications and Data Transmittal.
   (a) F.M. radio system.
   (b) Gage data reporting techniques.
Blue = Scheduled Releases
Green = Actual
Parker Dam Hourly Releases
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WATER WHEEL

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6-7-70
Taylor Ferry

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6 - 4 - 70

Telemark Data

Actual Release Data

Est. Release Data
Telephone conversation with John Sanders.

Dec 7 1970

(1)  Memo

Memo of August 16, 1965 on Propogation of flow changes
down the Colorado River.

(2)  Data for examples and checks

(a) Distances between gaging stations

(b) Diffusion constant being used and variations from
    summer to winter

(c) Data for three consecutive days for flow at
    Parker
    Headgate Rock
    Water Wheel
    Palos Verde
    Taylors Ferry
    Cibola
    Imperial

These should be in the form of a plot covering 24 hours
to show the computed flow and the observed flow. They
are needed for preparation of a full page chart for
the paper to show agreement of the observed and com-
puted values and to give a check on the diffusion con-
stant based upon a sinusoidal variation at Parker.
Mean flow should be obtainable.

(d) Tailwater and crest elevations headgate Rock and Palo-
    Verde dams

(3) Manuscript should be of the form prescribed under "Basic
    requirements for manuscripts" inside the front cover of the
    Journal of the Irrigation and Drainage Division ASCE

(4) Mean depth, Parker to Headgate Rock
Original copy will be reproduced by Xerox to provide additional copies and preserve accuracy.

Nelson will type up final draft so that format is consistent throughout.

Plans are to complete the text before the 18th of the month.

Paper proposed by Dave Click should be complementary rather than competitive. A third paper could cover the river control from Glen Canyon to Parker.

The following correspondence went to Sanders:

Oct 8, 1970
Oct 2, 1967, John Sanders to Dave (6 pages)
Oct 15, 1967 (10-5-67) Note CMS to Paul - Attached note Paul to Smutty
Oct 26, 1967, Click to ASCI Session Program Committee Chairman.
May 14, 1968, Koolzer to Click.
Oct 2, 1967, Johnny to Smutty.

Nelson's mail code 1502c.
702-385-6011

Chief Nelson
MM 1423

Call John Sanders

Gordon From C No.
2:00 our time
1:00 Boulder City

Perfor to

H & R 14.34 miles
WW 40.61 miles - 1500 ft.
P.V. 58.63"
T. F. 85.96"
Oib- 109.68"
Imp 147.0"
Mr. John I. Sanders
1317 Denver Street
Boulder City, Nevada - 89005

November 25, 1970

Dear Mr. Sanders:

The plan I had in mind when I wrote you on November 12, 1970 has run into some difficulties because of a need to get additional data from Mr. Nelson. The enclosed outline is therefore being sent to provide something for a start.

Please consider it something to break the ice. I hope you and Mr. Nelson will revise it extensively. My present thought is that you might do items 1 to 6 inclusive, I would take 7 and 8 and Mr. Nelson items 9 to 11 inclusive. We would collaborate on item 12.

With best wishes for a happy Thanksgiving holiday.

Sincerely yours,

Robert E. Glover

Enclosure
Mr. B. P. Bellport, Director  
Design and Construction  
U.S. Bureau of Reclamation  
Building 67, Denver Federal Center  
Denver, Colorado - 80225  

Attention:  
Mr. Chester A. Nelson  

Dear Mr. Bellport:  

Please refer to your letter of November 19, 1970 concerning preparation of a paper dealing with computation of a transient flow changes in the Colorado River.  

A tentative outline for this paper is transmitted herewith. Before I can complete the items of (7) and (8) it will be necessary for me to get together with Mr. Nelson and get from him certain data such as distances between gaging stations, flow variations at Parker and the like.

Sincerely yours,

[Signature]

Robert E. Glover  

Copy to:  
John I. Sanders
Colorado River Flow Management

(1) Need for close control of the River.
   (a) Proximity of the border
   (b) Limited storage at Imperial Dam
   (c) Senator Wash facility

(2) Management of Releases.
   (a) Requests come into Imperial on Thursday
   (b) Estimates of daily release
   (c) Demands of Power Production at Parker
   (d) Transit time—Parker to Imperial

(3) Headgate Rock and Palo Verde dams.
   (a) Diversions for irrigation
   (b) Drainage return flow
   (c) Automatic gate operations
   (d) Gate malfunctions

(4) Washes.
   (a) Inflow from local thunderstorms

(5) Short Wave radio network.
   (a) Communications
   (b) Gage reporting

(6) Gaging stations in the Parker—Imperial reach.
   (a) Head Gate Rock
   (b) Water Wheel
   (c) Palo Verde
   (d) Taylors Ferry
   (e) Cibola
   (f) Imperial
(7) Computation of flows at the gaging stations.
   (a) Basis of computations
   (b) Condition of continuity
   (c) Flow controlled by gradient
   (d) Differential equation of the transient flow
   (e) Solution of the differential equation
   (f) Flow changes due to a flow at Headgate Rock lasting for one hour

(8) Check of the diffusivity factor.
   (a) Approximately sinusoidal releases variations at Parker
   (b) Solution of the differential equation
   (c) Evaluation of the diffusivity factor

(9) Arrangements for making the computations.
   (a) Computations made at Denver by use of the digital computer
   (b) Information concerning changes
   (c) Transmission of the data to Boulder City
   (d) Relaying of the data to Imperial Dam
   (e) Continuous monitoring of the River at Imperial

(10) Details of the computation procedure.
    (a) Distances between gaging stations
    (b) Program for computation of the effect of an hourly flow
    (c) Computation made from each gaging station to the next to permit of accounting for changes
    (d) Revision of the diffusion constant for the conditions of decreased flow during the winter
    (e) Temporary difficulties

(11) Comparison of computed and observed flows.
    (a) Head Gate Rock
    (b) Water Wheel
    (c) Palo Verde
    (d) Taylors Ferry
    (e) Cibola
    (f) Imperial
(12) Summary and conclusions.

(a) Computed flows are closely realized
(b) Diffusion factor remains within close limits for both summer and winter conditions
(c) The use of previously calculated flows has permitted a much earlier detection of trouble and has been an effective basis for monitoring the flow of the river for prevention of waste of water
Mr. Robert E. Glover  
1936 South Lincoln Street  
Denver, Colorado 80210

Dear Bob:

As promised in my letter of October 28, I am now writing to you regarding your request for approval of a paper on the history of the development of the trial load method for design of dams.

After careful consideration, we would prefer, for the following reasons, not to sponsor such a paper. The method and its history have been thoroughly documented in a number of technical papers and Bureau publications, including the paper "Arch Dams: Trial Load Studies for Hungry Horse Dam," which you coauthored with M. D. Copen, and the special article "Trial Load Method of Analyzing Arch Dams" by Ivan E. Houk, published in the second edition of the Bureau's Dams and Control Works. In addition, our designers have improved the method of analysis to the extent that the trial concept is no longer utilized.

As you may perhaps know, a recent paper we submitted to the ICOLD on the evolution of arch dams design and construction aroused considerable criticism, particularly regarding our trial load method of analysis. For this reason, we would prefer not to generate further controversy and criticism which might stem from your proposed paper.

As you now have been informed by copy of his letter of November 4, Mr. John Sanders has agreed to participate in the preparation of the second paper you have proposed – computation of transient flow changes in the Colorado River. The Regional Director at Boulder City has also informed me that he is agreeable to Mr. Sanders' participation in the writing of the paper. By copy of this letter, I am asking Mr. Chester A. Nelson of the Division of General Research to contact you so that you may begin plans for the writing of the paper with Mr. Sanders.

Sincerely yours,

[Signature]

B. P. Bellport  
Director  
Design and Construction
Mr. R. E. Glover  
1936 South Lincoln Street  
Denver, Colorado - 80210  

Dear Mr. Glover:

I certainly would have no objection to your preparation of a preliminary draft. My review of notes and memos that I brought with me here hasn't produced anything that would be of much assistance.

In 1967 Dave Chopp, Sub-District Office Chief, for U.S.G.S., then stationed in Yuma, Arizona, asked me to co-author a paper with him for ASCE publication. The paper was to have covered specifically the cooperative efforts of U.S.B.R. and U.S.G.S. in installing the Palo Verde Diversion Dam Recorders and analog computers. It was not accepted at that time and we have not made another effort to get such a paper accepted. I thought you should know about it, although it wouldn't cover subject you have proposed. In any case I am enclosing my copies of the correspondence for you to see. I would like to have them back.

We will return to Boulder City the latter part of this week. I will then be able to research any material necessary for review of your preliminary draft. Please request any assistance you may need at Boulder City.

Sincerely,

John J. Sanders
Mr. John I. Sanders
Route #2 Box 97
Ignacio, Colorado - 81137

Dear Mr. Sanders:

It was a pleasure to hear from you and to receive assurance, by copy of your letter to Mr. Bellport, that you would be interested in collaborating on a paper on the Colorado River developments. A letter of October 28th from Mr. Bellport states that they will write me again as soon as they hear from the Regional Director's Office.

My present thought is that I could write up a preliminary draft of a paper and get copies so you and Mr. Nelson. This would provide something to work on. If you have a better idea as to how we could get started I would be much interested to hear about it.

Sincerely yours,

Robert E. Glover
1317 Denver Street
Boulder City, Nevada - 89005
November 4, 1970
(Phone: 702-293-1758)

Mr. E. F. Bellport, Director
Office of Design and Construction
Bureau of Reclamation
Denver Federal Center - Bldg. 67
Denver, Colorado - 80225
Attention: 1421

Dear Mr. Bellport:

I will be happy to assist in the preparation of the paper proposed by Mr. Robert E. Glover, as suggested in your letter dated October 28, 1970 to the Regional Director, Boulder City, Nevada.

By a copy of this letter I am informing Mr. Glover that I will write to him or contact him otherwise as soon as I have reviewed the information available here. In the meantime any suggestions that he or Mr. Nelson have will be welcome.

The Regional Director in Boulder City has indicated that pertinent information in his office can be made available.

Sincerely yours,

John I. Sanders

Copy to: Mr. Robert E. Glover, 1936 South Lincoln Street, Denver, Colorado - 80210
Regional Director, Boulder City, Nevada - 89005
Attention: 3-460/462.1

Dear Mr. Glover - I've been wondering what you were doing. I'm pleased to know you're still going strong.

I'll be at address below for a couple of weeks. I'll be reviewing some of my old notes & memos that I'll take with me. Best regards -

(Phone 303-883-2338)
Mr. Robert E. Glover  
1936 South Lincoln Street  
Denver, Colorado  80210

Dear Bob:

I have your letter of October 12, concerning your suggestions for two technical papers: History of the development of the Trial Load Method; and computation of transient flow changes in the Colorado River between Parker and Imperial Dams.

I shall soon write to you again regarding our views on the first paper. In the meantime, I am writing to the Regional Director at Boulder City informing him of your suggestion for the second paper and recommending that you, John I. Sanders, former Region 3 engineer, who recently retired, and Chester A. Nelson of my office coauthor the paper. You and Messrs. Sanders and Nelson have each contributed significantly to the development of the river flow computation technique and your combined writings should comprise an interesting and informative paper. We suggest that the paper be offered for publication in the Journal of the Hydraulics Division of the American Society of Civil Engineers.

I shall inform you promptly when I have received the Regional Director's response.

Sincerely yours,

[Signature]

B. P. Bellport  
Director  
Design and Construction
Mr. B.P. Bellport, Director
Office of Design and Construction
Bureau of Reclamation
Denver Federal Center - Bldg. 67
Denver, Colorado - 80225

October 12, 1970

Dear Mr. Bellport:

Last week I discussed with Mr. Arthur the desirability of preparing two papers for publication. These would be:

2. Computation of transient flow changes in the Colorado River between Parker and Imperial Dams.

So far as I am aware, no history of the development of the Trial Load Method has been attempted. My service with the Bureau of Reclamation dates back to 1917 when it was the United States Reclamation Service. I have contributed to these developments and can speak from personal knowledge about much of it. The people whose memory goes back even as far as the Gibson Dam era are becoming scarce.

As you know, the flow of the Colorado River at each of the gaging stations between the Parker and Imperial dams is computed each night for each hour of the ensuing 24 hours. This is done to conserve water by permitting a tighter control of the river. Now, when gage readings come in over the short wave radio which indicate flows differing from the computed values, action can be immediately taken to find out what is wrong and correct it. Before the computed flows were available such prompt remedial action was not possible.

The method of computing these transient flows represents an advancement in the art of Hydraulics. I would like to see these developments reported in the technical press. Co-authors from your Office and from Boulder City would be desirable. My part would be the mathematical development.

To prepare these papers I would need your permission to consult Bureau sources and personnel.

Your approval for the preparation of these papers is requested.

Sincerely yours,

Robert E. Glover
Memorandum

TO: Chief, Hydrology Branch

FROM: R. E. Glover

DATE: August 16, 1965

Denver, Colorado

SUBJECT: Propagation of flow changes down the Colorado River

Propagating

Demands for irrigation water come into the Imperial Office on Wednesday of each week. Since it requires about 3 days for water released at Parker Dam to reach Imperial Dam, it is necessary to program the mean daily releases at Parker so that the water will be available at Imperial Dam when it is needed. This is done now by using factors derived from experience. It is suspected that these factors may be somewhat inaccurate because the constant demands on the river leave no opportunity for definitive experiments. There is good reason to suspect also that these factors may change somewhat with change of stage. A means of computing these factors from the stream properties would provide some new information on the characteristics of the response of the river to flow changes. It is the purpose of this memorandum to describe a formula for estimating these changes.

Propagation of Changes of Flow

This development will be based upon the following assumptions:

1. The flow is controlled by friction.

2. For changes varying by small amounts from the mean flow condition, flow changes can be assumed to be proportional to the slope changes.

3. The factor of proportionality $K$ can be evaluated from the stage-flow curves.

4. An increase of flow produces a step increase in stage which moves downstream at the mean velocity of flow $v$.

Conditions near the step are shown on Figure 1.

If $f$ represents the change of flow associated with the step then

$$f = - K \frac{\partial \eta}{\partial x} \quad \ldots \ (1)$$
where $\eta$ represents the increase of depth and $x$ the distance measured from the step in the downstream direction.

The continuity condition is

$$\frac{\partial f}{\partial x} \ dx \ dt = - \ dx \ \frac{\partial \eta}{\partial t} \ dt \ T$$

where $T$ represents the top width. This relation can be put into the form.

$$\frac{\partial f}{\partial x} = - \ T \ \frac{\partial \eta}{\partial t} \quad \ldots \ (2)$$

Elimination of $f$ between (1) and (2) yields the differential equation.

$$K \ \frac{\partial^2 \eta}{\partial x^2} = T \ \frac{\partial \eta}{\partial t} \quad \ldots \ (3)$$

or if

$$\alpha = \frac{K}{T} \quad \ldots \ (4)$$

The relation takes the form

$$\alpha \ \frac{\partial^2 \eta}{\partial x^2} = \frac{\partial \eta}{\partial t} \quad \ldots \ (5)$$

A solution satisfying the conditions that

$$\eta + \eta_0 \ \text{as} \ x \to 0 \quad \ldots \ (6)$$

is, for $x > 0$:
\[ \eta = \eta_0 \left[ 1 - \frac{x}{\sqrt{4\pi ct}} \int_0^\infty e^{-u^2} du \right] \] ... (7)

**Example**

As an example of the use of this formula we can treat the stream as being represented by the Taylors Ferry Section at a flow of 10,000 \( \text{ft}^3 \) per sec. Hydraulic considerations give the flow as

\[ F = Av = AC \sqrt{rS} \] ... (8)

where \( A \) represents the area of the cross section \( (\text{ft})^2 \)
\( v \) the mean velocity of flow
\( r \) the hydraulic radius.
\( S \) the slope maintaining the flow \( F \)
\( C \) the Chezy coefficient.

The change of flow as a function of the slope only is given by

\[ \frac{\partial F}{\partial S} = \frac{ACr}{2\sqrt{S}} = \frac{ACrS}{2S} = \frac{F}{2S} \]

then

\[ K = \frac{F}{2S} \] ... (9)

and

\[ \alpha = \frac{K}{T} = \frac{F}{2ST} \] ... (10)

The wave front gradient is obtained from Equation (7) by differentiation with respect to \( x \). Then,

\[ \frac{\partial \eta}{\partial x} = -\frac{\eta_0 e^{-x^2/4ct}}{\sqrt{\pi ct}} \] ... (11)
Application will be made to the reach of the Colorado River between Parker and Imperial Dams. The distance between these points is 147.0 miles or 776,160 feet. The section at Taylors Ferry will be taken as representative of the river hydraulics. From stage-flow tables for this section

\[
F = 10,000 \text{ ft}^3/\text{sec}
\]
\[
A = 3,284 \text{ ft}^2
\]
\[
v = 3.045 \text{ ft/sec}
\]
\[
S = 0.00020268 \text{ (dimensionless)}
\]

The top width here is 354 feet but maps and photographs indicate that the top width is generally wider than this. A width of 800 feet will be used to obtain a realistic representation of the surface storage. This latter width is used in the evaluation of \( \alpha \). Then

\[
\alpha = \frac{10,000}{(2)(0.00020268)(800)} = \frac{10,000}{0.32428} = 30,840 \text{ ft}^2/\text{sec}
\]

With a velocity of 3.045 ft/sec the travel time between Parker and Imperial is

\[
\frac{776.160}{3.045} = 254,896 \text{ seconds or 2.950 days.}
\]

The computation is made in the manner shown in Table 1.

The first three columns are self-explanatory. Since the point of flow change is assumed to be carried along at the mean stream velocity the \( x \) distance is obtained by subtracting the distance moved by the point of flow change from the distance from Parker to Imperial. The next two columns are self-explanatory. The next column of figures is obtained from the relation

\[
\frac{\eta_1}{\eta_0} = \left[ 1 - \frac{2}{\sqrt{x}} \int_0^{\sqrt{4\alpha t}} e^{-u^2} \, du \right]
\]

\[\text{Omit} \quad \ldots \quad (12)\]
Values of $\eta$, for $x$ positive, are obtained by multiplying the ratio $\eta_1/\eta_0$ by $\eta_0$. When $x$ becomes negative $\eta$ should be interpreted as $\eta = 2\eta_0 - \eta_1$. The value to be used for $\eta_0$ is one half the increase in depth due to the increased flow. This depth is 0.4105 feet. It is computed on the supposition that, for small changes, the velocity of flow remains unchanged and the top width is 800 feet. The flow due to the increased depth is obtained by multiplying the increased area of the cross section by the mean stream velocity. The next three columns are used for figures needed for estimating the increased gradient due to the slope of the wave front as given by Formula (11). The flow due to the slope of the wave front is obtained by use of (9) and (11) from the relation

$$f_1 = \frac{F \eta_2}{2S} \cdot \frac{x^2}{4\alpha t} \cdot \frac{1}{\sqrt{\pi \alpha t}}$$

This implies that the flow is proportional to the gradient for small changes. The total flow increase is the sum of the flow due to increased depth and the flow due to increased slope. The factor of increase applies to flows at the end of the day. A similar factor for the mean daily flow is given in the next to the last column. Increments are shown in the last column. They are estimated by using Simpson's rule methods and the total flow increase values.

**Comments**

The wave propagation pattern obtained from the formulas given herein agrees well with the performance of the actual stream. Because both the stream cross sections and top widths vary widely along the course of the stream it will be difficult to fix accurately the proper values of the constants to be used on the basis of the hydraulics of the stream. It may be better to adjust them by trial to represent the actual performance of the stream.
## Table 1

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\( \bar{\eta} = \eta \) for \( x > 0 \)

<table>
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<tr>
<th>Time (days)</th>
<th>Flow due to increase in depth (( \text{ft}^2/\text{sec} ))</th>
<th>( \frac{x^2}{4acT} )</th>
<th>( e^x )</th>
<th>( \frac{x^2}{4acT} e^x )</th>
<th>Flow due to slope of wave front (( \text{ft}^2/\text{sec} ))</th>
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\( vT = (800)(3.045) = 2436 \)

\( v = \sqrt{1064} = 103240 \)

\( \eta_0 = \frac{1.000}{(800)(3.045)} = 0.4105 \)

\( \eta_0 = 0.20525 \) (\( \text{ft} \))

\( \frac{1}{25} = \frac{(10,000)(0.20525)}{(2)(0.00000266)} = 5,000,400 \)

\( \frac{1}{25} = 111.838 \)
<table>
<thead>
<tr>
<th>Time (days)</th>
<th>Total flow increase (ft³/sec)</th>
<th>Factor of increase (dimensionless)</th>
<th>Mean increase for the day (ft³/sec)</th>
<th>Increment of mean increase for the day (ft³/sec)</th>
<th>Factor of incremental mean increase (dimensionless)</th>
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