United States Department of the Interior
BUREAU OF RECLAMATION

LOWER COST CANAL LININGS

A progress report on the development of lower-cost linings for irrigation canals

Denver, Colorado, June 1948

FOREWORD

Because of the increasing urgency of conserving water lost from unlined irrigation canals, the Lower-Cost Canal Lining Program was inaugurated by the Bureau of Reclamation in June 1946 as a concerted effort to develop lower-cost linings. An interim report, issued in 1946, outlined the broad scope of the program and presented information available at that time on types of linings and their service records. In the past two years, the program has been advanced by laboratory and office research, surveys of installations, seepage determinations, field experiments, and equipment development.

This report records the progress made in developing lower-cost linings. In many phases of the program, accomplishment has been limited, and the many problems that remain unsolved emphasize the complexity of the undertaking. Although prepared primarily for use within the Bureau of Reclamation, this progress report may be of interest to others concerned with irrigation, as the problem of canal seepage losses is common and important to other agencies, water users' organizations, and individuals. Copies of this report, therefore, are made available for selective distribution outside the Bureau.

The report was prepared in the office of the Chief Engineer by T. V. Woodford, M. C. Lipp, and H. M. Sult, under the direction of R. F. Blanks, H. R. McBirney, and H. S. Meissner. Information on the laboratory investigations and tests was supplied by the concrete, bituminous, earth materials, and hydraulic laboratories. Data on the condition of existing linings and much of the information on experimental installations were reported by project offices. Many of the field trial installations involving new materials and techniques were made possible through the cooperation of a number of equipment manufacturers, the Asphalt Institute and affiliated organizations, and the Portland Cement Association.

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INTRODUCTION

Thousands of miles of irrigation canals in the western states fail to deliver a large proportion of the water turned into them at their intakes because of leakage from unlined sections. This problem was recognized as early as 1907 by Dr. Elwood Mead, later appointed Commissioner of Reclamation, who wrote: (51)* "The water which sinks into the soil from ditches and reservoirs is one of the chief sources of waste in irrigation. In gravelly soils, or where ditches cross gypsum strata, the losses sometimes amount to more than half the total flow. The water which escapes is often worse than wasted. It collects in the lower lands, fills the soil, drowns the roots of trees and plants and brings alkali to the surface," Photograph No. 1.

Six years later, E. G. Hopson, a Bureau of Reclamation consultant, writing in A.S.C.E. Transactions (34) stated "...many of the water supplies which appeared to be inexhaustible a few years ago are being rapidly fully appropriated; so that reasons for economy and waste prevention are becoming more and more cogent. It is the writer's belief that, as time goes on, it may even be found necessary for legislation to require canal systems to be lined or otherwise protected from seepage loss, not only in the interest of investor and water user, but as a reasonable measure of conservation where water supplies are limited. As an engineering and business policy, it is well in the front rank and should be considered by all who are building new works or operating and extending those already constructed."

Since these early days in the history of western irrigation, when the need and urgency for a program of water conservation were realized by only a few farsighted engineers, the situation has become much more critical. The water in our streams and rivers, which is utilized for irrigation, power, transportation, and many other productive uses, has come to be recognized as one of the greatest and most valuable of our natural resources. Today, many of our western streams are fully appropriated and diverted, yet there remain many thousands of acres of dry, fertile land along these streams which could be irrigated if sufficient water were available. Furthermore, the demand continues for additional irrigated farm land to keep pace with the rapid industrial and population growth of the nation. The development and successful operation of new projects, as well as the further expansion of existing projects, depend, in a large measure, upon the most efficient and beneficial use being made of the water presently diverted for irrigation.

It has been estimated that one-third of all the water diverted from western streams for irrigation is lost in transit to the farm land and it is known that in a few individual cases, this loss in transit is as great as 60 percent. Of the 14,600,000 acre-feet of water diverted for use on 36 Bureau of Reclamation projects during 1946, approximately 37 percent was lost in transit. More than half of the transit loss, or 23 percent of the total water diverted, was attributed to seepage from canals and laterals. The remaining 14 percent was lost through waste. Evaporation is recognized as a factor in transit losses but it is generally agreed to be negligible, and no attempt was made to segregate it in the above figures. Assuming a water duty of 3 acre-feet per acre, the 23 percent of water lost through seepage would adequately irrigate an additional acreage of more than 1,000,000 acres. The losses through waste may be reduced and controlled by more efficient operation of the irrigation systems. It is also common knowledge that in some cases more water is applied to irrigated land than is necessary or conducive to maximum production. This again is a practice which can be corrected through improved farming methods and careful operation. Seepage losses, on the contrary, can be reduced only by providing a relatively impervious conduit for the water. In open channels this can be accomplished either by constructing a lining or by special treatment of the canal section.

Linings of various types and materials have been installed in numerous canals in the past where conditions made lining imperative or where the lining could be justified economically. However, of 125,000 miles of canals and laterals constructed in 17 western states (1942 Census Report) less than 5,000 miles, or 4 percent, are lined. The chief reason for this lack of canal lining, in the face of an urgent and obvious need for conservation of irrigation water, is the prohibitive cost of dependable linings. Therefore, the solution of this problem depends upon the development of canal linings or methods of treatment which can be provided at a sufficiently low cost to be economically feasible for use on the majority of irrigation projects.

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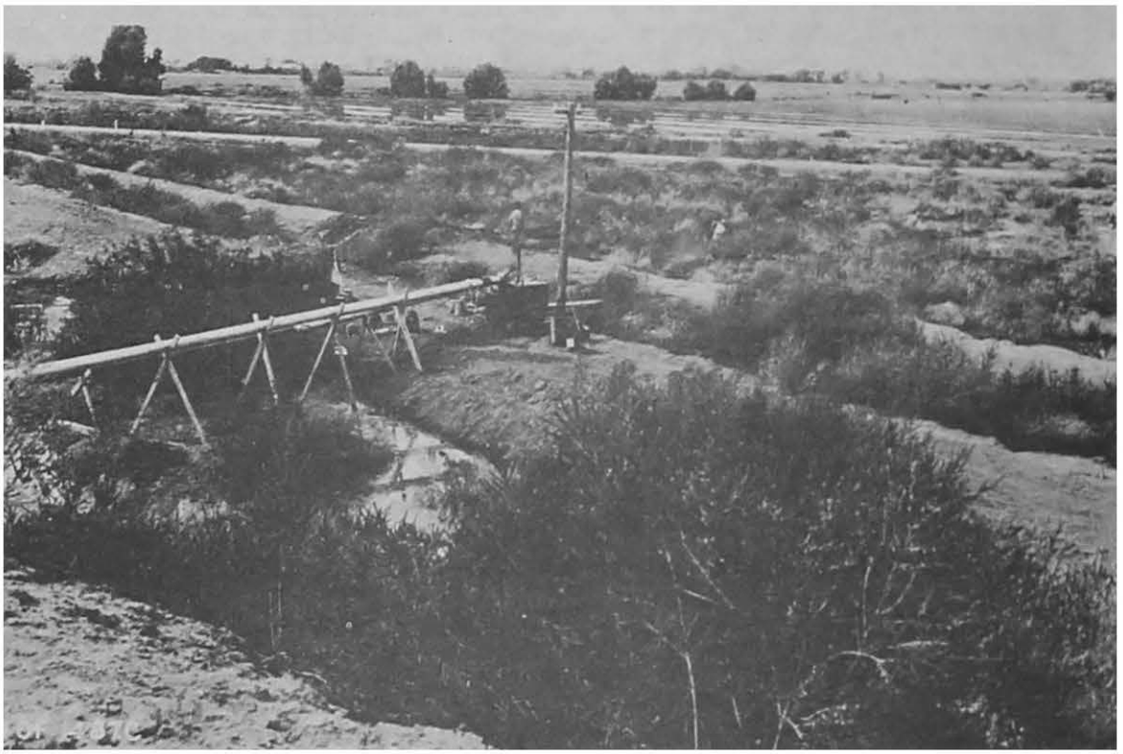
*Numerals in parentheses thus, (51), refer to the bibliography.

A considerable amount of individual, uncoordinated effort to reduce the cost of lining canals has been expended for short periods in previous years, as indicated in the literature and in the correspondence and informal reports on file. However, such effort in the past was usually short-lived and resulted in very limited progress. In recognition of the urgent need for an organized and continuing effort, the Bureau of Reclamation officially inaugurated a Lower-Cost Canal Lining Program in June 1946. Outlined in Circular Letter No. 3398, the Program included functions by office and field forces. It called for laboratory and office research, surveys of existing installations, seepage determinations, field experiments, and equipment development. Progress to date has been retarded by the Bureau's work load and the limited availability of funds and competent personnel. However, the program has produced some worthwhile results and plans for its continuation are unchanged.

It is the purpose of this report to summarize and bring up to date all available data and information on canal lining; to present the results of laboratory research, field installations, economic studies, investigations of seepage losses and seepage measuring devices; and to describe new developments in equipment and methods of constructing lined canals. So far as possible at this time, conclusions and recommendations as to the value of various types of linings and their suitability for use under various field conditions are stated. However, the answer to many of these problems can be determined only after years of continued research and observation of canal lining installations in the field. Therefore, these conclusions and recommendations are not presented as the final results of the Program, and are subject to revision as more complete data are obtained.



Photograph No. 1

Groundwater over cultivated field due to seepage
from canal 700 feet north of field.

SEEPAGE DETERMINATIONS

SEEPAGE LOSS MEASUREMENTS

Many factors enter into the need for and justification of lining. Damage to land by seepage or the cost of a drainage system to prevent the damage may, and often has, amounted to more than the cost of a dependable lining. Where lands are thus damaged by excessive seepage, it is evident that remedial measures are required, although knowledge of the exact amount of water seeping from the canal may be unimportant. However, the relation and slope of the various soil horizons may be such that the seepage water is not visible but flows underground. In many instances the cost of the lining must bear a relation, or be equal, to the value of the water lost. It is often desirable also to determine accurately the effectiveness of a lining installation. The only means of determining the actual loss is by measurement, and since the quantity is generally small in comparison with the total discharge in the canal, extreme care must be exercised to obtain accurate results. For this reason, an important phase of the Lower-Cost Canal Lining Program is to devise the best method of measuring the quantity lost by seepage. The principal methods for determining canal seepage losses are:

1. Inflow and outflow method by use of either current meters, salt velocity, weirs, valves, gates, venturi meters, or Parshall flumes
2. Tappoon method
3. Constant and variable head permeameters
4. Laboratory seepage meter

The inflow and outflow method involves the measurement of the quantity of water flowing into a particular section and the corresponding outflow from the same section, the difference representing the total loss. Current meters are often used for this procedure because no head loss occurs and the instruments are low in cost. However, the accuracy is normally insufficient for the purpose of evaluating seepage losses, although the average of a large number of measurements will increase the accuracy of the results.

Application of the salt-velocity method involves a determination of the time required for a salt cloud to travel from one pair of electrodes to a second pair at a known distance downstream. The passing of the electrolyte may be detected with an oscillograph or an ammeter. The elapsed time for the cloud to pass from one pair of electrodes to the other represents the average velocity of the water from which the quantity may be computed. The chief advantage of this system is that no head loss exists. However, the special equipment and technique required render the method unsuitable for ordinary field usage.

Weirs are generally impractical because of head loss, and even in the few cases where their installation is possible, the fluctuations in head, when using large quantities of water, are sufficient to produce inaccurate results. The installation costs are relatively high, which further disqualifies this method of measuring the flow.

Valves, gates, venturi meters, Parshall flumes, and similar devices may be utilized if properly calibrated. In this connection, it should be remembered that only relative values are required when utilizing the inflow-outflow method. For instance, even though the measuring apparatus at each end of the section being studied indicates discharges 10 percent in excess of the correct amount, the differential will be approximately correct. Hence, the use of identical measuring devices will tend to compensate for any errors.

The tappoon method involves segregating a reach of canal with temporary bulkheads, filling with water, and observing any decrease in volume over a given period of time, thus evaluating the quantity of loss. Or a constant depth may be maintained in the test section by providing a small discharge of known quantity into the reach and measuring the outflow over a fixed crest. The difference between the two quantities then represents the total loss. This procedure permits the measurement of small quantities which is readily accomplished quite accurately with weirs.

Based on present information, the tappoon method is the most accurate means of determining seepage loss. However, it does possess certain disadvantages in that the canal being studied must be taken out of operation, or measurements made off-season when freezing temperatures or a dry subsoil may introduce undeterminable errors

sufficiently large to void the results. A further disadvantage is that the effect of velocity on the seepage loss is ignored, whereas experiments made by running water in short sections of test canals supported on screens show that the loss is greater with still water than with flowing water. (180) A very small velocity makes little difference but a bottom velocity of 1.5 feet per second has a marked influence. A higher velocity further decreases the amount of seepage but in a much lesser degree.

Evaporating, which also is usually neglected, is considered to be negligible. On this subject, Samuel Fortier states (179) that the loss of water due to evaporation "is small in comparison to the volume carried and on an average represents less than one-fourth of one percent of the flow."

The constant and variable head permeameter consists merely of a pipe placed in the canal, while flowing, to segregate a small section. The pipe is then filled with water and the drop in water surface noted over a pre-determined time, or a known quantity is added to maintain a constant head. The loss from the pipe, representing the seepage, is subject to errors if any leak occurs around the end forced into the canal or if a change in density is caused by disturbing the soil. The procedure does possess the advantage that the loss is determined in a local area instead of a considerable length permitting the determination of those sections contributing most to the total loss.

The laboratory seepage meter, Figure 1A, is a modified version of the constant head permeameter. This device is equipped with a flexible bag which is submerged in the canal after filling with a known quantity of water and connected to the cylinder previously forced into the canal boundary. A constant head is maintained on the area segregated by the cylinder and the loss from the bag determined over a given period of time by weighing the bag before and after the test. Another type of permeameter apparatus is shown in Figure 1B which operates with a variable head. In each of these modified permeameter devices, the cylinder forced into the earth is provided with a bleeder valve to release any air trapped while placing the apparatus in position.

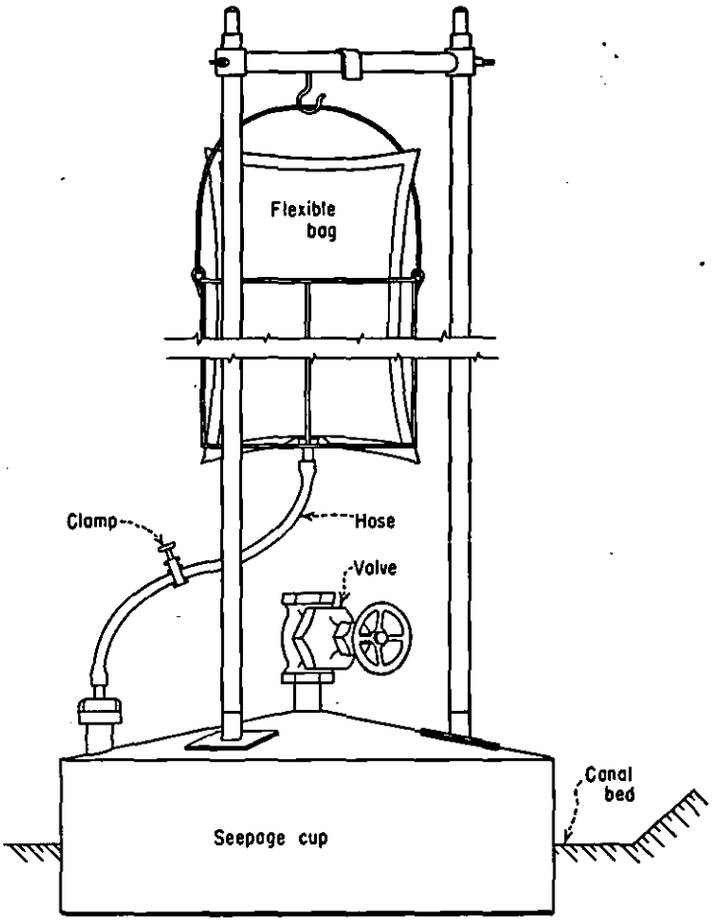
Obviously, none of these permeameters can be used in lined canals, and their application is probably limited to the determination of relative values of loss rather than a method of ascertaining total seepage losses. Additional tests are necessary to establish definitely the relative merits of these several devices.

The preferred procedure of measuring seepage loss from an unlined canal is to apply the tappoon method by dividing the canal into several reaches sufficiently long to obtain a loss great enough for accurate measurement. The application of one of the permeameter devices in the sides and bottom of the canal at predetermined intervals will segregate any particularly porous section. The same procedure is applicable to lined canals except that permeameters cannot be used.

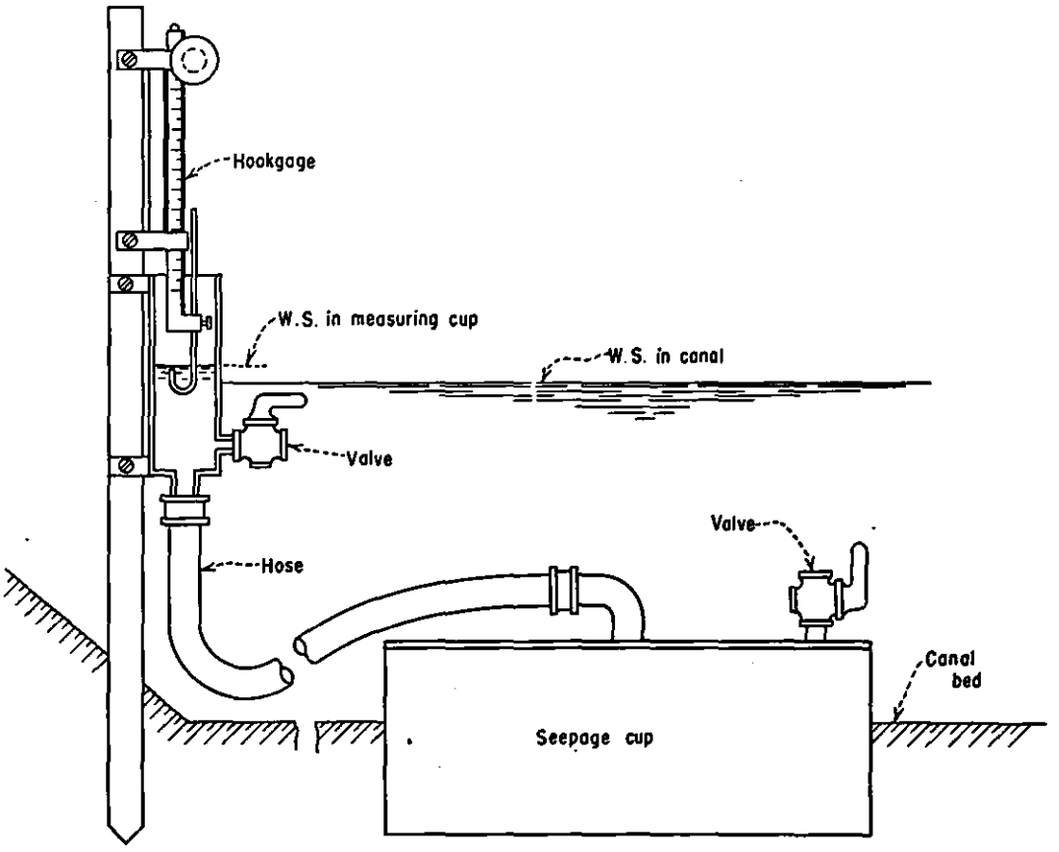
There has been considerable variance relative to the method of expressing the quantity of seepage loss. This has been expressed as: (1) second-feet per acre of wetted area; (2) percent of flow per mile; (3) feet depth over water surface per unit time; (4) percent of total diversion; (5) feet depth over wetted area per day; (6) second-feet per thousands of square feet of wetted area; and (7) cubic feet per square foot of wetted area per 24 hours.

Perhaps the most common expression is percent of total diversion or percent of flow. Although this method may be preferable for operational purposes, it is not suitable when comparing the loss from different canals as this will vary with the extent of the wetted area. Hence, it is proposed to use the expression "cubic feet per square foot of wetted area per 24 hours" as a standard means of expressing the seepage loss in connection with the canal lining program.

In the reports from the regions on existing linings, considerable data were included on seepage losses. Data were given for numerous unlined sections as well as for linings of concrete, shotcrete, asphalt, and bentonite. Many of these data were incomplete as to thickness and condition of the lining at the time of test, but losses were expressed in terms of cubic feet per square foot of wetted area per 24 hours and, therefore, are of value in indicating the probable range. Losses from unlined canals varied from 0.20 to 3.0, while losses in lined sections ranged from 0.0 to 1.2 cubic feet per square foot of wetted area per 24 hours. In the data reported by Rohwer and Stout (3), a maximum



A. LABORATORY TYPE



B. VARIABLE HEAD TYPE

SEEPAGE METERS

FIGURE 1

loss of 8.24 cubic feet per square foot of wetted area was shown for an unlined section through sandy loam. This loss, from a lateral with an average discharge of 13.9 cubic feet per second, was also shown in percent per mile and was equal to 41.3. As a further comparison of the relation of these two units for expressing seepage losses, a main canal and a lateral, both unlined, with average discharges of 111 and 5.75 cubic feet per second, respectively, were both reported to be losing 10.93 percent per mile through seepage. Expressed as cubic feet per square foot of wetted area, however, the loss from the main canal was 2.07 and from the lateral only .595 cubic feet.

When considering the amount of water lost by seepage from a canal, consideration must be given to the fact that the loss may be only temporary with recovery at some lower elevation. For instance, observations on the North Platte River between Whalen and Bridgeport (13) revealed that approximately 65 percent of the water diverted for irrigation returned to the river, and hence was available for irrigation of additional land. In cases where the earth structure is such that the seepage water does not enter a natural stream, drainage ditches or wells may be provided to permit irrigation of additional land or supplement the existing water supply. In fact, it may not be desirable to prevent seepage as in Kern, Tulare, and Fresno Counties of California where seepage during the runoff season is required to build up underground storage for use during the summer and fall for pumping as there are no surface reservoirs for storage. However, in utilizing water from the soil, consideration must be given to the chemical composition due to any salts that may have been dissolved, since the drainage water may be unfit for irrigation purposes.

Location of the groundwater table is necessary to determine its effect on the amount of seepage and assure that the measurements are made under normal conditions. If the groundwater table is sufficiently high, there may be a flow into the canal under study, or conditions may be such that no exchange of water exists between the canal and the adjacent area. During the initial wetting of a canal in the spring season, the loss may be abnormally high until the groundwater is replenished and, unless proper precautions are taken, loss measurements will be in error. Test wells adjacent to the canal at varying distances from the centerline will permit location of the groundwater table and a determination of its stability and effect on the quantity of seepage.

The amount of silt deposited in a canal will certainly affect the seepage rate. If the material through which a new canal passes is such that silt will be carried into the voids reducing the permeability, the loss by seepage will decrease after the first few years operation. Consideration must be given to this fact when measuring the loss from a recently constructed canal.

The effects of precipitation during test periods will introduce errors in the results which may only be avoided by not making observations under adverse conditions. No accurate means exist for ascertaining the quantity of water which will flow into a canal from surface runoff and by percolation. According to Hazen's formula for the flow of groundwater, the velocity is proportional to the temperature Fahrenheit plus 10, hence an increase in temperature from 50 to 70 F would increase the seepage loss 33 percent. (180) Accordingly, the groundwater temperature in the vicinity of the canal is important.

PRELIMINARY INVESTIGATIONS

Preliminary investigations, prior to construction, are of considerable value in determining the probability of excess seepage and the damaging effect of the water so lost to the canal itself and to nearby lands. Perhaps the first and one of the most important requirements is a soil profile. A soil profile is a vertical section of the layers of soil existing along the canal line. The characteristics of the soil profile control all internal water movement except that influenced by nonsoil agencies, such as animal burrows, root holes, and the like. The soil profile should be determined below the canal bottom to a depth at least equal to the expected water depth in the canal, or a minimum of 5 feet. If a complete soil profile is not justifiable, an approximate profile obtained by a few test holes, supplemented by surface observations, may be sufficient for estimating seepage losses. A field man trained in the objectives and procedures of this work should be present at the time of the excavation of the test holes and should log the

materials according to the Casagrande classification. Other descriptive information should be included with the soil classification, such as apparent density of soil; presence of small cracks, holes, or streaks of sand; and apparent permeability of the soil in place. A knowledge of the extent of a particular type of soil found in the test hole or the place of change in material along the canal will assist in determining the soil profile. Another important feature that should be noted on the soil profile is the transverse slope of the ground surface along the canal. Steep slopes encourage seepage because of the possibility that very permeable strata may intersect the canal section or the earth surface near the canal. Sand and gravel formations in the vicinity of old river terraces are often very permeable although they may be overlain with fine-grained soils that may be sufficiently thick to inhibit excessive seepage. The soil structure in such areas, as determined from a few test holes made for reconnaissance purposes, will indicate the seepage possibilities and the number of additional test holes necessary.

Generally, the types of soil that are likely to have heaviest losses are relatively clean sands and gravels. Uniform gravels are highest in permeability followed by well-graded gravel, uniform sand, and well-graded sand in decreasing order. Another type of material that is usually questionable is a very plastic clay (Casagrande CH type) because of its tendency to develop large shrinkage cracks upon drying. This type soil should have small seepage losses when continually wet, but a recent test installation on the Delta-Mendota canal is an exception. Provisions were made for wetting the base (CH type soil) under 195 linear feet of concrete lining to determine the destructive effects on the concrete due to the expansion properties of this particular soil. Water was added to the base for 3 months at about 2,000 gallons per hour, indicating a high percolation rate. The strong affinity for water and possibly minute fissures in the soil account for this high loss. Other soils that will have moderately heavy seepage losses are the very fine sands of the SP or SF-silty type and silts of ML or MH types. These fine-grained soils are also hard to hold in any regular form of ditch due to sloughing and erosion. Following is a list of soil types with their Casagrande classification symbol and a relative rating as to permeability and probable need for lining:

GP Gravel, uniform--extremely permeable--need lining

GW Gravel, well-graded with few fines--extremely permeable--need lining

SP Sand, uniform--moderately to very permeable--usually require lining or silting

SW Sand, well-graded with few fines--moderately permeable--usually require lining or silting

CH Clay, very plastic--very impermeable when wet, or extremely permeable after drying--special considerations

ML Silt--fairly impervious, but bank section is difficult to hold in place--special considerations

MH Silt, very compressible--fairly impervious, but bank sections difficult to hold--special considerations

GC Gravel with clay binder--may range from moderate to very low permeability

GF-clayey Gravel with excess clay--usually impermeable, good stability

SC Sand with clay binder--usually impermeable, good stability

SF-clayey Sand with excess clay--usually impermeable, good stability

GF-silty Gravel with excess silt--usually impermeable, good stability

SF-silty Sand with excess silt--usually fairly impermeable but hard to hold on bank

CL Clay (lean) usually very impermeable

OL Organic silt--permeability fairly low, but stability is questionable

OH Organic clay--permeability very low, if soil is kept wet, but stability is questionable and shrinkage cracks are probable

More factual test data supplementing the above criteria will be of considerable value in predicting seepage losses. The permeability characteristics of the soils along the canal may be determined by cutting undisturbed samples from the different types of soils for laboratory permeability tests, or by making field percolation tests on the predominating soil types in place. Both of these tests have definite limitations. The laboratory test is necessarily made on a small sample, which may contain small holes or permeable streaks which are not continuous in the natural structures, but would extend through the test specimen and thereby give faulty permeability results. The analysis of field percolation tests is questionable because the material types, structural arrangement, and lateral flow at the point of test, are frequently not representative of the area.

The laboratory permeability test follows the standard procedure** except that the materials are placed in plastic, permeability cylinders and tested without any superimposed load other than the water head. Undisturbed cohesive samples are cut to fit the cylinder, whereas noncohesive materials sampled loose are placed in the cylinder at their in-place density. Where permeability tests are required on noncohesive soils, the in-place density must be determined.

The field percolation tests may be made by either measuring the amount of water required to maintain the water level in a test hole of known dimensions at a constant depth, or by measuring the amount of water required to maintain the water level in an open-ended standpipe at a constant depth. For best results, it is advisable to continue either of these tests until the rate of water loss becomes fairly constant--usually 3 days or more. The test-hole method is most adaptable to use in material of low-to-moderate permeability where the quantity of water required is not excessive. The pit or hole may be filled with gravel to prevent caving of the sides or erosion of the soil which may settle in the bottom and tend to form a seal. The standpipe method may be used in any soil, but should have a 6-inch layer of gravel in the bottom to avoid disturbance of the soil when the water is added. Further details on the procedure for field percolation tests are contained in the Bureau of Reclamation Earth Materials Investigation Manual, November 1947.

**Laboratory Procedure in Testing Earth Materials for Foundation and Construction Purposes, Bureau of Reclamation, July 1946.

In his text, Irrigation Practice and Engineering (180), B. A. Etcheverry established the following requirements for a good canal lining: "It should be practically watertight to prevent seepage loss and the resulting waterlogging and rise of alkali; its cost should not be excessive; it should prevent the growth of weeds; resist burrowing animals; be strong and durable, preferably not affected by the tramping of cattle; adapted to the construction of a smooth canal of the proper shape to increase its carrying capacity and permit the use of high velocities." These, together with a reasonable amount of flexibility and extensibility, are the desirable characteristics of an ideal lining.

Few, if any, of the canal lining materials in common use possess all of these characteristics, the relative importance of which depends upon problems encountered on each installation. The several types of linings which have come into general use, and those experimental linings utilizing new materials and construction methods, are discussed in the following pages. Some of the newer-type linings are described in more detail than those which have been in common use for many years, not for the purpose of emphasizing them or attempting to sell the idea, but because of the lack of information available elsewhere. Details commensurate with the importance of cast-in-place concrete, for instance, are not given here because such information is readily available in the Bureau's Concrete Manual. (56) The type of lining to be selected for a particular installation will be the one with the characteristics which most nearly fulfill the requirements of the job.

CONCRETE AND MORTAR LININGS

Linings of portland cement concrete and mortar may be cast-in-place, precast, or applied pneumatically. Mixtures in which portland cement is combined with selected natural soils are termed soil-cement and are discussed further under the general heading of Earth Materials.

Cast-in-place Concrete Mortar

The term "cast-in-place" in this discussion refers to linings constructed of premixed plastic concrete which is either troweled, screeded, or formed into place. Canal side slopes are usually made sufficiently flat, as discussed under design, to avoid costly forms; but forming may be necessary in special cases. Some installations exist which were constructed with forms, but linings of this type are rarely built today. Hand or machine screeding is and has been for years the most common method of construction for cast-in-place linings. The fresh concrete is usually dumped from wheelbarrows or dragline buckets and screeded into place by hand or by machinery built for the purpose. Photograph No. 2 shows men hand-screeding a 2-inch concrete lining in a small lateral. One or two strokes with a long-handled steel trowel finishes the operation. The concrete was supplied by wheelbarrows operated on the 2-inch plank at the top of each slope. Transverse screeding has also been used in larger installations as illustrated in Photograph No. 3 where concrete was dumped in the canal from a concrete bucket operated from a dragline. The lining was placed in alternate panels to reduce final shrinkage and the screed was pulled up the side slope by a winch on a tractor at the top of the bank.

In spite of these improvements in construction operations, progress is relatively slow and considerable hand labor is required with attending high cost. On jobs of considerable magnitude, it is usually found economical to utilize one of several types of slip-form or continuous lining equipment. These may be mounted on railroad rails on the berm or, if the lining is unreinforced, may ride directly on the trimmed subgrade. Such equipment, which is described in more detail under the general heading of equipment, places and screeds the concrete across the entire perimeter of the canal as it moves, or is moved, longitudinally along the section. The simpler slip-form (see Photograph No. 11) which rides directly on the subgrade has recently been used economically in small laterals and farm ditches of only 1- or 2-foot bottoms. It is not believed adaptable to use in large canals and obviously cannot be used for placing reinforced concrete.

From a study of the many miles of concrete lining in existence, it is estimated that the average serviceable life of a properly designed and constructed concrete lining is about 40 years. A few canals lined with thin cement mortar have been in service for longer than 50 years. Many concrete linings have given satisfactory service for

over 30 years and are still in good condition. In 1886, a 3/4-inch cement mortar lining was installed in 11.9 miles of the Gage canal, Riverside, California (see Photograph No. 4). It was reported that the condition of the original lining still in service is very good, although about 25 percent of it has been replaced for various reasons. Three miles of the Fruitvale Mutual Water Company's main canal in southern California lined with 1/2- to 3/4-inch of cement mortar in 1880 is reported to be in good condition. The Hodges Conduit, part of the water supply for the city of San Diego, California, which was lined in 1908 with 3-inch reinforced concrete, is still in service and in a generally good condition although there has been some damage from back-pressures. There are many other concrete linings in service today of ages approaching that of the above linings. However, there have been numerous early failures of concrete linings, due to one or more of the many factors affecting their permanence, such as design, location of the canal, groundwater, subgrade conditions, quality of the concrete, and construction methods.

The measured losses from concrete-lined canals vary through a wide range dependent upon the quality of the concrete and the condition of the lining. Since escape of water through unfilled cracks and joints constitutes the principal source of leakage, the maintenance of concrete linings is essential to their most efficient use. A reasonable loss from a properly constructed and maintained concrete-lined canal should not be in excess of 0.05 cubic feet per square foot of wetted perimeter per 24 hours.

Concrete linings are highly resistant to erosion and will permit the safe use of high velocities especially if the water does not carry an appreciable amount of sand. This may be of considerable importance where advantage can be taken of steeper gradients which, with the resulting higher velocities, will permit the use of a smaller canal cross-section. Weed growth cannot penetrate a concrete lining, and while some weeds may be found growing in the larger cracks, the necessity of routine weed removal is nil. Burrowing animals, which cause numerous canal breaks, cannot penetrate such a lining. Concrete linings distribute concentrated loads to a limited extent, and thus are not readily damaged by livestock or canal cleaning equipment and will successfully bridge across small holes and washouts. They also exert an appreciable stabilizing effect on side slopes that might otherwise have a tendency to slide out or slough.

On the other hand, concrete linings cost more to construct than any of the other commonly-used types which may limit its use. Concrete is subject to temperature cracking because of its high coefficient of expansion and low extensibility. If not sealed, these cracks permit seepage and thus affect the durability and life of the lining. Also, the resistance of concrete linings to external hydrostatic pressures due to groundwater conditions, or from a rapid drawdown in the canal water level, is quite limited. In northern climates where considerable below-freezing weather is encountered, frost heaving (Photograph No. 6) is undoubtedly the greatest factor in the destruction of concrete linings. The provision for adequate drainage and proper preparation of the subgrade, as discussed under the general subject of design, is perhaps the most effective protection against frost heaving. Where such conditions are to be anticipated, it may sometimes be advisable to omit lining. Extensive damage may result from freezing and thawing in a concrete-lined canal in year-around use as illustrated by Photograph No. 7. Laboratory tests and field experience have demonstrated, however, that the addition of an air-entraining agent to the concrete mix will greatly minimize this destructive action. It is therefore particularly important that a properly designed mix, with air entrainment, be utilized for concrete linings in locations where severe conditions of exposure can be anticipated.

In an effort to lower the cost of canal linings, there has been an increasing recognition of the desirability and practicability of relaxing the requirements for alignment, grade, and finish, for standard concrete linings on Bureau projects. This idea has taken positive form in more liberal tolerances which have been adopted recently and now appear in Bureau specifications. These permit a departure of 4 inches from established alignment and a departure of 1 inch from established profile grade. A minus variation up to 10 percent in specified thickness is now permissible, provided that average thickness is maintained, as determined by daily batch volumes. For the past year Bureau specifications have been interpreted to require a minimum of hand trowelling. No hand trowelling is required when reasonably workmanlike results are obtained with a long-handled steel trowel as the lining slab emerges from the slip-form canal lining machine. Further simplification of this finishing procedure is under consideration.

Among the various canal linings having the qualifications of a lower-cost lining--low maintenance costs and long life as well as moderate first cost,--the possibilities of a simplified concrete lining have attracted increasing attention. An approach to such a lining has been made on the Gila project in the pit-run concrete lining placed with a slip-form (see description in section on equipment), although either pit-run or screened fine and coarse aggregate might be used. Such a simplified concrete lining may be placed under specifications requiring a minimum of control refinements. Pit-run aggregate may be satisfactorily utilized if of suitable quality and grading, and if series segregation is prevented by wetting and care in handling and batching. Screened material proportioned to contain 50 to 60 percent sand will expedite placing and finishing. A cement content of about 5-1/2 sacks per cubic yard of concrete is adequate for the average job. Pit-run material containing more than 60 percent sand will require additional cement--one-half sack per cubic yard for each additional 10 percent of sand. Batching may be by any method that will maintain the cement content within a variation of 5 percent plus or minus. Entrainment of 3 to 5 percent of air will be required to facilitate workability and finishing, and to improve durability.

Finishing may be largely eliminated where a fairly smooth surface is left by the slip-form. Piano wire alignment and rail guides are not required for operation of the slip-form which may be guided, as was the slip-form used on the Gila project, by a forward pan sliding on the trimmed subgrade. Tolerances need be no more restrictive, and probably less if necessary for significantly lower costs, than the 4-inch deviation from line and 1-inch deviation from grade, which has been established in new specification tolerances for standard concrete linings. Trimming costs in small canals can be greatly reduced by the use of longitudinally operating blades and graders working to liberal but practical tolerances. Wherever practicable, turnouts and similar structures should be constructed in advance of lining operations and should be designed so that the structure is recessed to permit trimming and lining equipment to pass without interference. Suitable blockouts will, of course, be necessary for subsequent construction of bridge piers, checks, and overchutes.

The serviceability that might be expected from such simplified concrete linings is indicated in the 27- and 34-year-old linings of the Franklin County and Burbank irrigation districts near Pasco, Washington (Photographs Nos. 8 and 9). The Franklin County linings are still in service while the Burbank canals were abandoned because of financial difficulties in 1925 after 12 years of service. Both linings are in good condition and have evidently required very little maintenance. Such performance would make a simplified concrete lining highly competitive with other lower-cost lining materials having less prospect of low maintenance and long life.

Reports from India to the effect that relatively weak concrete linings containing impure slaked lime were giving excellent service, led to a series of tests in the Denver laboratories aimed at developing a mix with greater extensibility and less drying shrinkage. Although results in the main were negative, some detail of the series is given here as a matter of record. Tests were conducted using hydraulic lime alone as the binding agent in concrete and in combination with portland cement. Asphalt emulsions were added to regular concrete mixes and the effect of air entrainment beyond the recommended amounts for durability was investigated. It was found that as the air content was increased above 5 percent, the drying shrinkage increased and the strength and elastic modulus decreased. Restrained specimens containing air up to about 5 percent appeared to crack at about the same age as concrete without entrained air. However, specimens containing 7 percent or more of entrained air cracked earlier with increased air. It may be concluded, therefore, that the reduced cracking which should result from a lower elastic modulus, is more than offset by increased drying shrinkage when high percentages of air are entrained. A similar effect was obtained with additions of asphalt emulsion.

In mixes containing hydraulic lime as the binding agent, strength was so low that tests on restrained shrinkage would have been meaningless. Compressive strength at 28 days for concrete containing from 500 to 600 pounds of hydraulic lime per cubic yard, was about 400 psi. Test specimens of hydraulic lime concrete disintegrated completely after less than 10 cycles of freezing and thawing. The use of hydraulic lime in combination with portland cement resulted in a decrease in compressive strength of the concrete at 28 days, in proportion to the amount of hydraulic lime used. Restrained shrinkage tests of concrete containing 10 and 20 percent hydraulic lime indicated less resistance to cracking than the control specimens of all portland cement.

Understandably, the cost of concrete lining varies widely with a number of factors such as location, accessibility of the region where construction is to be undertaken, competition among bidders, rigidity of specification requirements, and general economic conditions. Because of these variable factors, it is almost impossible to establish the cost of concrete linings with any degree of accuracy.

The cost of concrete lining installed by the Bureau of Reclamation has varied considerably as shown in Table 1. This has been especially true for the last few years, and it is impossible to determine a reliable cost for concrete lining under present conditions. However, during the years 1927 to 1940, construction costs were fairly constant. To secure reliable data on the cost of concrete linings, bid prices and cost reports for the linings installed by the Bureau during this period were secured and analyzed. Since it was known that many bidders purposely unbalance their bids, an average of the three lowest bids was utilized where three or more bids were received. If less than three bids were received, an average of the two bids, or the one bid only, was used.

It was realized that the costs of these items would vary with the section of the country, but no attempt was made to take this into account because the questionable accuracy of other assumptions did not warrant it for this purpose. Nor were all projects used; if some local condition, such as a large amount of rock excavation or inaccessibility of the job resulted in unusually high prices, they were eliminated. In making this study, only the major items of construction were considered; namely excavation, compacted embankment, trimming, and lining for lined canals.

An attempt was made to correlate the varying unit cost of each item with the quantity involved or the size of canal. It was found that the cost of concrete did not exhibit any definite tendency to vary with the size of canal, but it did show a good relationship with the quantity of concrete involved. For small yardages of 250 to 400 cubic yards, the average cost was about $19.50 per cubic yard including cost of materials and labor and decreased with increasing quantities to approximately $14.50 per cubic yard for quantities of 8,000 cubic yards and over. An average weighted cost was determined to be about $16.75 per cubic yard such as would be applicable to a yardage of 2,500 cubic yards. This value was used in future computations in this report.

The unit costs of excavation, compacted embankment, and trimming, evidenced a much more definite relationship to the size of canal than to the individual quantities involved. The average unit cost of excavation varied from a high of approximately $0.19 per cubic yard for small canals and laterals of 10 to 30 cfs capacity to $0.10 per cubic yard for canals of 200 cfs capacity and larger. The high cost of excavation in the smaller canals and laterals is undoubtedly due to the small cross-sections and the relatively small yardage which precluded the use of large earth-moving equipment, such as is normally employed for excavating the larger canals. The unit cost of compacted embankment remained fairly constant for canals of all capacities at about $0.20 per cubic yard. The average unit cost of trimming the canal subgrade increased with increasing canal capacities from $0.19 to $0.36 per square yard. This increasing cost for larger canals may be due to the fact that the trimming of the small canals has usually been accomplished in the past with unskilled hand labor or with simple, relatively inexpensive equipment, whereas the large canals required the use of large, costly, specially-built equipment. Although this equipment speeded up operations considerably, it had to be amortized in construction cost on the one contract and actually resulted in higher unit costs of trimming.

Figure 2 shows the average total construction cost for 3-inch reinforced concrete lined canals of various capacities. The data and graphs presented under the heading of Design of Lined Canals were used to determine the dimensions of the cross-sections for the various canal capacities shown. These cross-sections, which represent average Bureau of Reclamation design, were used to compute the yardage of excavation, compacted embankment, trimming, and lining per station for use with the above unit costs. As shown by these graphs, the cost of excavation is about 8 to 15 percent, compacted embankment 3 to 5 percent, trimming 14 to 16 percent, and the lining 64 to 72 percent of the total cost of a 3-inch reinforced concrete lined section. Thus the cost of the lining and the trimming are major items of expense.

A further breakdown of the cost of a concrete lining into the various items of expense is shown by Figure 3. Considering only the lining, the cost of placing the concrete is the

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

CONCRETE LINED CANALS

COST DATA BASED ON CONTRACT BID PRICES AND CONTRACT COST REPORTS

SPECIFICATION		PROJECT AND LOCATION	CANAL SECTION				DATE OPENED	CANAL LINING						COST OF LINING PER SQUARE YARD						
											REINFORCEMENT		CONTR. JOINT SPACINGS	CONCRETE		REINFORCEMENT		SUB-TOTAL	TRIM. SUB-GRADE	TOTAL
No.	SCHED.		Q c. f. s.	b FEET	d FEET	SIDE SLOPE		TYPE	QUAN. CU. YDS.	THICK-NESS	LONGIT.	TRANS.	LONGITUDINAL	TRANS	CONTRACT	GOV'T. MATERIALS	CONTRACT	GOV'T. MATERIALS			
446		Sta. 31 + 33 to 226 + 00 Kittitas Main Canal Yakima Project	1320	12	9.75	1¼:1	4-29-26	Reinforced Concrete	7,600	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	20'-0"	0.833	0.375	0.102	0.127	1.437	0.18	1.617
453	1	Sta. 226 + 00 to 488 + 35 Kittitas Main Canal Yakima Project	1275 to 1320	11 to 12	8.99 to 9.75	1¼:1	12-28-26	Reinforced Concrete	10,000	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	20'-0"	0.667	0.375	0.102	0.127	1.271	0.18	1.451
453	2	Sta. 488 + 35 to 745 + 00 Kittitas Main Canal Yakima Project	1254 to 1275	12	9.50 to 9.59	1¼:1	12-28-26	Reinforced Concrete	2,800	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	20'-0"	0.667	0.375	0.102	0.127	1.271	0.15	1.421
459	2	Sta. 1069 + 00 to 1383 + 00 Kittitas Main Canal Yakima Project	925 to 1210	10 to 12	7.61 to 8.70	1¼:1	6-3-27	Reinforced Concrete	9,450	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @24" p=0.00152	None	20'-0"	0.960	0.375	0.071	0.127	1.533	0.20	1.733
464	1	Sta. 0+00 to 242 + 60 North Branch Canal Yakima Project	929	6 to 10	7.85 to 7.88	1¼:1	12-6-27	Reinforced Concrete	5,800	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	20'-0"	0.960	0.375	0.076	0.127	1.538	0.25	1.788
546	6	Sta. 513 + 00 to 2051 + 00 North Canal Owyhee Project	451	7	5.76	1¼:1	10-25-33	Reinforced Concrete	670	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	Min. 8'-0"	1.000	0.613	0.152	0.183	1.948	0.10	2.048
546	7	Sta. 0 + 00 to 240 + 00 Mitchell-Butte Lateral Owyhee Project	179	5	3.71	1¼:1	10-25-33	Reinforced Concrete	440	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	Min. 8'-0"	1.000	0.613	0.152	0.183	1.948	0.15	2.098
550	1	Sta. 0+00 to 513 + 00 Mitchell-Butte Canal Owyhee Project	1068	10	10.00	1¼:1	11-7-33	Reinforced Concrete	630	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	Min. 8'-0"	1.220	0.498	0.135	0.225	2.078		2.078
623	1	Sta. 0+00 to 587 + 00 Ogden-Brigham Canal Ogden River Project	110 to 120	4	2.96 to 3.09	1¼:1	7-8-35	Reinforced Concrete	1,100	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	Min. 8'-0"	1.542	0.470	0.152	0.163	2.327	0.25	2.577
623	2	Sta. 0+00 to 587 + 00 Ogden-Brigham Canal Ogden River Project	95 to 110	4	2.63 to 2.96	1¼:1	7-8-35	Reinforced Concrete	5,550	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	Min. 8'-0"	1.125	0.508	0.152	0.168	1.953	0.30	2.253
659	1	Sta. 587 + 00 to 1260 + 00 Ogden-Brigham Canal Ogden River Project	35 to 85	2 to 4	1.94 to 2.48	1¼:1	12-20-35	Reinforced Concrete	6,750	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	Min. 8'-0"	1.167	0.360	0.152	0.152	1.831	0.20	2.031
659	2	Sta. 10+00 to 335 + 00 South-Ogden Highline Ogden River Project	15 to 35	2	1.28 to 1.94	1¼:1	12-20-35	Reinforced Concrete	2,640	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	Min. 8'-0"	1.167	0.363	0.152	0.152	1.834	0.20	2.034
675		Sta. 313 + 00 to 576 + 81.5 Yakima Ridge Canal Yakima Project	2201	14	11.2	1¼:1	4-13-36	Reinforced Concrete	9,770	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	Min. 8'-0"	0.833	0.480	0.135	0.190	1.638	0.25	1.888
711	1	Sta. 102 + 00 to 197 + 77 Yakima Ridge Canal Yakima Project	2200	14	11.2	1¼:1	12-14-36	Reinforced Concrete	2,480	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	Min. 8'-0"	0.889	0.533	0.135	0.185	1.742	0.30	2.042
716		Sta. 27+00 to 340 + 00 Heart Mountain Canal Shoshone Project	914	8	7.51	1¼:1	1-11-37	Reinforced Concrete	1,540	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	1.000	0.580	0.116	0.325	2.021	0.25	2.271
721	4	Sta. 1105 + 50 to 1129 + 00 Black Canyon Canal Boise Project	1001	12	9.13	1¼:1	3-12-37	Reinforced Concrete	725	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.889	0.608	0.180	0.262	1.939	0.25	2.189
721	5	Sta. 1159 + 00 to 1179 + 00 Black Canyon Canal Boise Project	1001	12	9.13	1¼:1	3-12-37	Reinforced Concrete	1,010	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.889	0.684	0.180	0.316	2.069	0.25	2.319
721	6	Sta. 1129 to 1159 & 1179 to 1250 Black Canyon Canal Boise Project	1001	12	9.13	1¼:1	3-12-37	Reinforced Concrete	450	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.889	0.690	0.180	0.334	2.093	0.25	2.343
729		Sta. 721 to 804 & 942 to 1120 Yakima Ridge Canal Yakima Project	1300	12	9.07	1¼:1	5-7-37	Reinforced Concrete	5,150	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.878	0.536	0.139	0.232	1.785	0.35	2.135
743		Sta. 303 to 322 & 340 to 712 Heart Mountain Canal Shoshone Project	775	7	7.51	1¼:1	8-2-37	Reinforced Concrete	900	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	1.570	0.540	0.180	0.301	2.591	0.20	2.791
748	1	Sta. 1120 + 00 to 1349 + 00 Yakima Ridge Canal Yakima Project	1300	12	9.07	1¼:1	9-22-37	Reinforced Concrete	13,270	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.733	0.506	0.126	0.198	1.563	0.25	1.813
748	2	Sta. 1349 + 00 to 1595 + 00 Yakima Ridge Canal Yakima Project	1300	12	9.07	1¼:1	9-22-37	Reinforced Concrete	14,100	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.733	0.490	0.126	0.198	1.547	0.25	1.797
751	1	Sta. 157 + 00 to 477 + 25 Black Canyon Canal Boise Project	1089	12	9.39 to 10.36	1¼:1	9-20-37	Reinforced Concrete	1,220	4"	½"ϕ @ 12" p=0.00408	½"ϕ @24" p=0.00204	None	8'-0" to 20'-0"	1.111	0.719	0.180	0.260	2.271	0.48	2.751
769		Sta. 712 + 00 to 920 + 00 Heart Mountain Canal Shoshone Project	704 to 740	7	7	1¼:1	2-10-38	Reinforced Concrete	1,235	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	1.377	0.534	0.301	0.280	2.492	0.06	2.552
776	1	Sta. 209 + 65 to 385 + 50 Contra Costa Canal Central Valley Project	350	7	7.37	1¼:1	5-20-38	Reinforced Concrete	5,550	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @24" p=0.00152	None	8'-0" to 20'-0"	0.500	0.415	0.102	0.117	1.134	0.48	1.614
776	3	Sta. 403 + 50 to 638 + 00 Contra Costa Canal Central Valley Project	291 to 320	7	6.75 to 7.06	1¼:1	5-20-38	Reinforced Concrete	6,830	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @24" p=0.00152	None	8'-0" to 20'-0"	0.500	0.415	0.102	0.117	1.134	0.16	1.294
783	1	Sta. 804 + 74 to 942 + 00 Yakima Ridge Canal Yakima Project	1300	12	9.07	1¼:1	5-9-38	Reinforced Concrete	1,660	4"	⅜"ϕ @ 12" p=0.00229	⅜"ϕ @ 24" p=0.00114	None	8'-0" to 20'-0"	0.944	0.628	0.180	0.188	1.940	0.35	2.290
790		Sta. 482 + 00 to 782 + 80 Black Canyon Canal Boise Project	1042	12	9.2	1¼:1	6-13-38	Reinforced Concrete	7,300	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8' 0" to 20'-0"	0.667	0.722	0.139	0.260	1.788	0.15	1.938
794		Sta. 920 to 947 & 981 to 1229 Heart Mountain Canal Shoshone Project	554 to 615	5 to 6	6.2 to 6.51	1¼:1	8-31-38	Reinforced Concrete	3,970	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	1.222	0.513	0.185	0.269	2.189	0.70	2.889
801	1	Sta. 638 + 00 to 780 + 00 Contra Costa Canal Central Valley Project	269 to 291	7	6.50 to 6.75	1¼:1	9-30-38	Reinforced Concrete	3,840	3"	⅜"ϕ @ 12" p=0.00302	⅜"ϕ @ 24" p=0.00151	None	8'-0" to 20'-0"	0.625	0.402	0.091	0.113	1.231	0.20	1.431
801	2	Sta. 780 + 00 to 1066 + 00 Contra Costa Canal Central Valley Project	256	7	6.35	1¼:1	9-30-38	Reinforced Concrete	7,710	3"	⅜"ϕ @ 12" p=0.00302	⅜"ϕ @ 24" p=0.00151	None	8'-0" to 20'-0"	0.625	0.409	0.091	0.113	1.238	0.20	1.438
813	1	Sta. 1641 + 15 to 1952 + 00 Yakima Ridge Canal Yakima Project	1100 to 1250	12	8.48 to 8.89	1¼:1	12-8-38	Reinforced Concrete	5,670	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.833	0.533	0.139	0.185	1.690	0.40	2.090
813	2	Sta. 1641 + 15 to 1952 + 00 Yakima Ridge Canal Yakima Project	1100	10	8.82	1¼:1	12-8-38	Reinforced Concrete	643	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.889	0.591	0.139	0.195	1.814	0.40	2.214
815	1	Sta. 1952 + 00 to 2387 + 00 Yakima Ridge Canal Yakima Project	1100	10	8.82	1¼:1	1-9-39	Reinforced Concrete	6,800	4"	½"ϕ @ 12" p=0.00408	½"ϕ @ 24" p=0.00204	None	8'-0" to 20'-0"	0.844	0.551	0.139	0.195	1.730	0.35	2.080
865		Sta. 1066 to 1542 & 2349 to 2359 Contra Costa Canal Central Valley Project	172 to 225	6	5.06 to 6.06	1¼:1	8-8-39	Reinforced Concrete	11,150	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @24" p=0.00152	None	8'-0" to 20'-0"	0.520	0.401	0.086	0.117	1.124	0.18	1.304
884		Sta. 714 + 85 to 724 + 00 Yakima Ridge Canal Yakima Project	1300 to 2100	12 to 14	9.07 to 10.93	1¼:1	1-26-40	Reinforced Concrete	990	4"	⅜"ϕ @ 10" p=0.00275	⅜"ϕ @20" p=0.00137	None	20'-0"	0.889	0.655	0.085	0.093	1.722	0.50	2.222
886		Sta. 6 + 10 to 452 + 00 Madera Canal Central Valley Project	823 to 1000	8 to 10	7.08 to 8.98	1½:1	12-21-39	Reinforced Concrete	17,700	3½"	⅜"ϕ @ 12" p=0.00261	⅜"ϕ @ 24" p=0.00131	None	20'-0"	0.390	0.341	0.091	0.229	1.051	0.27	1.321
890		Sta. 1542 + 72 to 1993 + 11 Contra Costa Canal Central Valley Project	125 to 172	6	4.14 to 5.06	1¼:1	1-15-40	Reinforced Concrete	9,300	3"	⅜"ϕ @ 12" p=0.00304	⅜"ϕ @ 24" p=0.00152	None	20'-0"	0.608	0.390	0.089	0.186	1.373	0.14	1.513
947	1	"A" & "B" Canals & Laterals Yuma Mesa Division Gila Project	620	8	10.1	1½:1	1-28-41	Reinforced Concrete	4,680	3"	9ga 6"x6" mesh p=0.00097	9ga 6"x6" mesh p=0.00097	None	10'-0"	0.483	0.441	0.045	0.124	1.093	0.13	1.223
947	2	"A" & "B" Canals & Laterals Yuma Mesa Division Gila Project	350 to 420	8	7.58 to 7.96	1½:1	1-28-41	Reinforced Concrete	6,315	3"	9ga 6"x6" mesh p=0.00097	9ga 6"x6" mesh p=0.00097	None	10'-0"	0.486	0.485	0.045	0.133	1.149	0.131	1.280
947	3	"A" & "B" Canals & Laterals Yuma Mesa Division Gila Project	280 to 350	5 to 8	6.2 to 7.1	1½:1	1-28-41	Reinforced Concrete	7,375	3"	9ga 6"x6" mesh p=0.00097	9ga 6"x6" mesh p=0.00097	None	10'-0"	0.683	0.437	0.067	0.135	1.322	0.183	1.505
947	4	"A" & "B" Canals & Laterals Yuma Mesa Division Gila Project	170 to 280	8	5.3 to 6.3	1½:1	1-28-41	Reinforced Concrete	7,150	3"	9ga 6"x6" mesh p=0.00097	9ga 6"x6" mesh p=0.00097	None	10'-0"	0.485	0.465	0.045	0.126	1.121	0.131	1.252
947	5	"A" & "B" Canals & Laterals Yuma Mesa Division Yuma Project	60 to 70	5	2.8 to 4.5	1½:1	1-28-41	Reinforced Concrete	5,050	2½"	10ga 6"x6" mesh p=0.000 0	10ga 6"x6" mesh p=0.000 0	None	10'-0	0.585	0.419	0.057	0.113	1.174	0.185	1.359
947	6	"A" & "B" Canals & Laterals Yuma Mesa Division Yuma Project	50 to 130	5	3.1 to 5.0	1½:1	1-28-41	Reinforced Concrete	6,585	2½"	10ga 6"x6" mesh p=0.000 0	10ga 6"x6" mesh p=0.000 0	None	10'-0"	0.585	0.445	0.057	0.108	1.195	0.185	1.380

TABLE 1

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

CONCRETE LINED CANALS

COST DATA BASED ON CONTRACT BID PRICES AND CONTRACT COST REPORTS

SPECIFICATION No.	SPECIFICATION SCHED.	PROJECT AND LOCATION	CANAL SECTION Q c. f. s.	b FEET	d FEET	SIDE SLOPE	DATE OPENED	CANAL LINING TYPE	QUAN. CU. YDS.	THICK-NESS	REINFORCEMENT LONGIT.	REINFORCEMENT TRANS.	CONTR. JOINT SPACINGS LONGITUDINAL	CONTR. JOINT SPACINGS TRANS.	COST OF LINING PER SQUARE YARD CONCRETE CONTRACT	CONCRETE GOV'T. MATERIALS	REINFORCEMENT CONTRACT	REINFORCEMENT GOV'T. MATERIALS	SUB-TOTAL	TRIM. SUB-GRADE	TOTAL
955	1	Sta. 1+73.12 to 135+86 New Briar Canal Boulder Canyon Project	350 to 700	8 to 18	5.07 to 5.92	1½:1	4-2-41	Reinforced Concrete	4,570	3"	9 ga 6"x6" mesh p=0.00097	9ga 6"x6" mesh p=0.00097	None	10'-0"	0.850	0.309	0.056	0.118	1.333	0.16	1.493
956		Sta 2387+40 to 2884+50 Yakima Ridge Canal Yakima Project	1000	10	8.4	1¼:1	4-11-41	Reinforced Concrete	1,860	4"	⅜" ϕ @ 10" p=0.00275	⅜" ϕ @ 21" p=0.00131	None	20'-0"	1.000	0.622	0.124	0.193	1.94	0.40	2.340
958	1 to 6	A & B Canals & Laterals Second Section Gila Project	12	1.5	1.1 to 2.0	1½:1	4-18-41	Unreinforced Concrete	4,010	2"	None	None	None	10'-0"	0.445	0.204			0.649	0.19	0.839
958	1 to 6	A & B Canals & Laterals Second Section Gila Project	12 to 50	1.5	2.0 to 3.6	1½:1	4-18-41	Reinforced Concrete	17,980	2½"	10 ga.6"x6" mesh p=0.0010	10 ga.6"x6" mesh p=0.0010	None	10'-0"	0.555	0.255	0.047	0.104	0.961	0.19	1.151
1058	1	Sta 2884+75 to 3314+40 Yakima Ridge Canal Yakima Project	800 to 1000	10	7.48 to 8.37	1¼:1	9-27-43	Reinforced Concrete	1,500	4"	⅜" ϕ @ 12" p=000229	⅜" ϕ @ 21" p=000131	None	14'-9"	1.778	0.760	0.106	0.143	2.787	0.70	3.487
1060	1	Sta 3354+00 to 4713+00 Yakima Ridge Canal Yakima Project	500 to 600	7	6.83 to 7.46	1¼:1	10-12-43	Reinforced Concrete	5,000	4"	⅜" ϕ @ 10" p=0.00275	⅜" ϕ @ 21" p=0.00131	None	14'-9"	2.168	0.760	0.150	0.161	3.239	0.44	3.679
1065	3	Sta 4562+67 to 5150+00 Coachella Canal Boulder Canyon Project	1200 to 1300	12	10.8	1½:1	1-31-44	Reinforced Concrete	42,240	3½"	6ga. 8"x8" mesh p=0.00099	6 ga. 8"x8" mesh p=0.00099	None	15'-0"	1.167	0.360	0.057	0.135	1.719	0.25	1.969
1099	1	Sta.6+10 to 301+60 Friant-Kern Canal Central Valley Project	4000 to 5000	36	15.2 to 17.2	1¼:1	7-18-45	Reinforced Concrete	28,800	3½"	¼" □ @ 12" p=0.0015	¼" □ @ 12" p=0.0015	One each side slope 12" from base slab	14'-6"	0.777	0.336	0.116	0.135	1.364	0.53	1.894
1113	1	Sta.5150 +00 to 5725+00 Coachella Canal Boulder Canyon Project	1000 1260	12	10.8	1½:1	12-10-45	Reinforced Concrete	35,900	3½"	6ga. 8"x 8" mesh p= 0.001	6 ga.8"x 8" mesh p = 0.001	None	15'-0"	1.286	0.370	0.108	0.135	1.899	0.27	2.169
1148	2	Sta. 311+28 to 710+00 Friant-Kern Canal Central Valley Project	4000 to 5000	36	15.2 to 17.2	1¼:1	2-5-46	Reinforced Concrete	38,400	3½"	⅜" ϕ @ 16" p= 0.002	⅜" ϕ @16" p= 0.002	One each side slope 12" from base slab	Max. 15'-2"	0.801	0.348	0.193	0.178	1.520	0.52	2.040
1148	3	Sta. 710+00 to 1144+00 Friant-Kern Canal Central Valley Project	4000 to 5000	36	15.2 to 17.2	1¼:1	2-5-46	Reinforced Concrete	38,200	3½"	⅜" ϕ @ 16" p= 0.002	⅜" ϕ @ 16" p= 0.002	One each side slope 12" from base slab	Max. 15'-2"	0.923	0.348	0.193	0.178	1.642	0.52	2.162
1171	1	Sta. 1144+00 to 1591+66 Friant-Kern Canal Central Valley Project	4000 to 5000	36	15.2 to 17.2	1¼:1	3-5-46	Reinforced Concrete	22,600	3½"	⅜" ϕ @ 16" p= 0.002	⅜" ϕ @ 16" p= 0.002	One each side slope 12" from base slab	Max. 15'-2"	1.093	0.348	0.178	0.178	1.797	0.40	2.197
1171	1	Sta. 1144+00 to 1591+66 Friant-Kern Canal Central Valley Project	4000 to 5000	36	15.2 to 17.2	1¼:1	3-5-46	Unreinforced Concrete	22,600	3½"	None	None	6' each side ℄ and on side slope @ 2' and 14' from base	12'-0"	1.093	0.348			1.441	0 40	1.841
1172	1	Sta. 2321+25 to 2512+25 Contra Costa Canal Central Valley Project	22 to 67	3 to 5	1.84 to 3.04	1¼:1	3-12-46	Unreinforced Concrete	1,550	3"	None	None	None	10'-0"	1.50	0.259			1.759	0.35	2.109
1181	1	Sta. 1591+66 to 1647+75 Friant-Kern Canal Central Valley Project	4000 to 5000	28 to 36	15.2 to 17.2	1¼:1	4-10-46	Reinforced Concrete	2,070	3½"	⅜" ϕ @ 16" p = 0.002	⅜" ϕ @ 16" p = 0.002	One each side slope 12" from base slab	15'-2"	1.950	0.348	0.147	0.178	2.623	1.12	3.743
1183	1	Sta. 686+00 to 1365+00 Delta-Mendota Canal Central Valley Projects	4399	48	16 to 16.2	1½:1	4-30-46	Unreinforced Concrete	47,000	4"	None	None	On ℄ and @ 11' interval along base and slopes	12'-0"	1.260	0.324			1.584	0345	1.929
1183	1	Sta. 686+00 to 1365+00 Delta-Mendota Canal Central Valley Project	4399	48	16 to 16.2	1½:1	4-30-46	Reinforced Concrete	47,000	4"	⅜" ϕ @ 12" p=0.0023	⅜" ϕ @12" p= 0.0023	One on ℄	Max. 15'-6"	1.222	0.324	0.217	0.237	1.998	0.345	2.343
1186	1	Phantom Lake Canal Balmorhea Project	25	3	0.69 to 2.55	1¼:1	4-16-46	Reinforced Concrete	1,640	2½"	⅜" ϕ @ 12" p =0.0037	⅜" ϕ @24" p= 0.0018	None	10'-0"	2.010	0.250	0.102	0.178	2.540	0.30	2.840
1186	2	Inlet Feeder Canal Balmorhea Project	100	4	1.43 to 1.97	1¼:1	4-16-46	Reinforced Concrete	1,880	3"	⅜" ϕ @ 12" p=0.0031	⅜" ϕ @24" p=0.0015	None	10'-0"	2.125	0.300	0.102	0.178	2.705	0.25	2 955
1195	1	Sta. 5725+00 to 6106+06 Coachella Canal Boulder Canyon Project	675 to 800	12	8.5	1½:1	4-15-46	Unreinforced Concrete	9,800	3½"	None	None	None	12'-0"	1.460	0.351			1.811	0.45	2.261
1195	1	Sta. 5725+00 to 6106+06 Coachella Canal Boulder Canyon Project	675 to 800	12	8.5	1½:1	4-15-46	Reinforced Concrete	9,800	3½"	6ga. 8"x 8" mesh p= 0.001	6 ga. 8"x 8" mesh p=0.001	None	15'-0"	1.460	0.351	0.100	0.129	2.040	0.45	2.490
1204	1	Sta. 1647+75 to 1932+75 Friant-Kern Canal Central Valley Project	4000 to 5000	28 to 36	15.2 to 17.2	1¼:1	4-10-46	Reinforced Concrete	14,150	3½"	⅜" ϕ @ 16" p=0.002	⅜" ϕ @16" p= 0.002	One each side slope 12" from base slab	15'-2"	1.800	0.375	0.127	0.178	2.480	0.35	2.830
1204	1	Sta. 1647+75 to 1932+75 Friant-Kern Canal Central Valley Project	4000 to 5000	28 to 36	15.2 to 17.2	1¼:1	4-10-46	Unreinforced Concrete	14,500	3½"	None	None	6' each side ℄ and on side slopes @ 2' and 14' from base	12'-0"	1.800	0.375			2.175	0.35	2.525
1204	3	Sta. 2892+50 to 3369+00 Friant-Kern Canal Central Valley Project	4000 to 5000	36	16.1 to 18.3	1¼:1	4-10-46	Reinforced Concrete	11,500	3½"	⅜" ϕ @ 16" p= 0.002	⅜" ϕ @ 16" p= 0.002	One each side slope 12" from base slab	15'-2"	1.990	0.375	0.127	0.178	2.480	0.35	2.830
1204	3	Sta. 2892+50 to 3369+00 Friant-Kern Canal Central Valley Project	4000 to 5000	36	16.1 to 18.3	1¼:1	4-10-46	Unreinforced Concrete	11,500	3½"	None	None	6' each side ℄ and on side slopes @ 2' and 14' from base	12'-0"	1.990	0.375			2.365	0.35	2.715
1230		Pasco Laterals & Sublaterals Columbia Basin Project	119	5	5	1½:1	5-31-46	Unreinforced Concrete	2,200	3"	None	None	None	6'-0" 9'-0" 12'-0"	1.903	0.333			2.236	0.45	2.686
1230		Pasco Laterals & Sublaterals Columbia Basin Project	96	5	4.5	1½:1	5-31-46	Unreinforced Concrete	16,000	2"	None	None	None	6'-0" 9'-0" 12'-0"	1.683	0.250			1.933	0.45	2.383
1236	1	Sta. 24+00 to 430+00 Main Canal Columbia Basin Project	13,200	24	23.7	1½:1	4-30-46	Reinforced Concrete	4,100	4½"	⅜" ϕ @ 12" p = 0.002	⅜" ϕ @ 12" p =0.002	24" in from each side slope	14'-6"	2.500	0.754	0.271	0.237	3.762	0.20	3.962
1286	1	Sta 2+50 to 350+00 West Canal Columbia Basin Project	5100	12 to 38	16.4 to 16.9	1½:1	5-15-46	Unreinforced Concrete	16,250	4½"	None	None	24" in from each side slope	14'-6"	1.500	0.662			2.162	0.58	2.742
1286	1	Sta. 2+50 to 350+00 West Canal Columbia Basin Project	5100	12 to 38	16.4 to 16.9	1½:1	5-15-46	Reinforced Concrete	16,250	4½"	⅜" ϕ @ 12" p= 0.002	⅜" ϕ @ 12" p= 0.002	24" in from each side slope	14'-6"	1.500	0.662	0.196	0.237	2.595	0.58	3.175
1402	1	A Canal & A Lateral Yuma Mesa Division Gila Project	60 to 145	5 to 9	3 to 5.13	1½:1	7-23-46	Reinforced Concrete	65	2½"	10 ga 6"x 6" mesh p=0.0010	10 ga 6"x 6" mesh p= 0.0010	As directed	10'-0"	2.220	0.338	0.132	0.086	2.778	1.00	3.778
1422	1	Sta 0+25 to 650+00 East Low Canal Columbia Basin Project	4500	20	18.9	1½:1	8-20-46	Reinforced Concrete	30,800	4½"	None	None	On ℄ and each side slope 24" from base	12'-0"	2.130	0.096			2.226	0.58	2.806
1422	1	Sta 0+25 to 650+00 East Low Canal Columbia Basin Project	4500	20	18.9	1½:1	8-20-46	Reinforced Concrete	30,800	4½"	⅜" ϕ @ 12" p=0.002	⅜" ϕ @ 12" p=0 002	8' each side of ℄	14'-6"	2.130	0.096	0.196	0.236	2.658	0.58	3.238
1435	1	Sta L-185+00 to L-774+00 Delta-Mendota Canal Central Valley Project	4600	48	16.56	1½:1	8-21-46	Reinforced Concrete	73,800	4"	⅜" ϕ @ 12" p= 0.0023	⅜" ϕ @ 12" p= 0.0023	22' each side of ℄	15'-6"	1.718		0.183	0.237	2.138	0.36	2.498
1512	1	Sta 6106+06 to 6517+00 Coachella Canal Boulder Canyon Project	425 to 625	8	7.0 to 8.0	1½:1	12-5-46	Reinforced Concrete	9,170	3½"	6 ga 8"x 8" mesh p= 0.001	6 ga 8"x 8" mesh p= 0.001	None	15'-0"	1.978	0.036	0.109	0.129	2.252	0.18	2.432
1512	1	Sta 6106+06 to 6517+00 Coachella Canal Boulder Canyon Project	425 to 625	8	7.0 to 8.0	1½:1	12-5-46	Unreinforced Concrete	9,170	3½"	None	None	None	12'-0"	1.978	0.036			2.014	0.18	2.194
1666		M Canal Tule Lake Division Klamath Project		3	5	1½:1	3-3-47	Reinforced Concrete	430	4"	⅜" ϕ @ 12" p= 0.0064	⅜" ϕ @ 24" p= 0.0032	None	8'-0"	3.92		0710	0.63	526	0 50	576
1693	1	Sta 1993+11 to 2321+25 Contra Costa Canal Central Valley Project	72 to 125	5 to 6	3.46 to 4.14	1½:1	3-17-47	Reinforced Concrete	2,450	3"	⅜" ϕ @ 18" p=0.00265	⅜" ϕ @ 18" p=0.00265	None	15'-6"	1.670	0.260	0.360	0.200	2.490	0.750	3.240
1693	1	Sta 1993+11 to 2321+25 Contra Costa Canal Central Valley Project	72 to 125	5 to 6	3.46 to 4.14	1½:1	3-17-47	Unreinforced Concrete	2,450	3"	None	None	None	12'-0"	1.670	0.260			1.930	0.750	2.680
1693	2	Sta 14+86.5 to 240+00 Clayton Canal Central Valley Project	11 to 38	3 to 4	1.14 to 2.19	1¼:1	3-17-47	Unreinforced Concrete	1,500	2"	None	None	None	6'-0"	2.000	0.170			2.170	0650	2.820
2074	1	Sta 3764+99 to 3771+85 Friant-Kern Canal Central Valley Project	4000 to 5000	36	15.54 to 17.54	1¼:1	2-26-48	Unreinforced Concrete	225	3½"	None	None	6' each side ℄ and on side slopes @ 4.6 and 26.6 from upper edge	Max. 12'-0"	2.920				2.920	0 450	3 370
2074	3	Sta 3883+5 to 4646+06 Friant-Kern Canal Central Valley Project	4000 to 5000	36	14.66 to 17.54	1¼:1	2-26-48	Unreinforced Concrete	75,500	3½"	None	None	6' each side ℄ and on side slopes @ 4.6 and 26.6 from upper edge	Max. 12'-0"	1.410				1.410	0 40	1.810

TABLE I (cont'd)

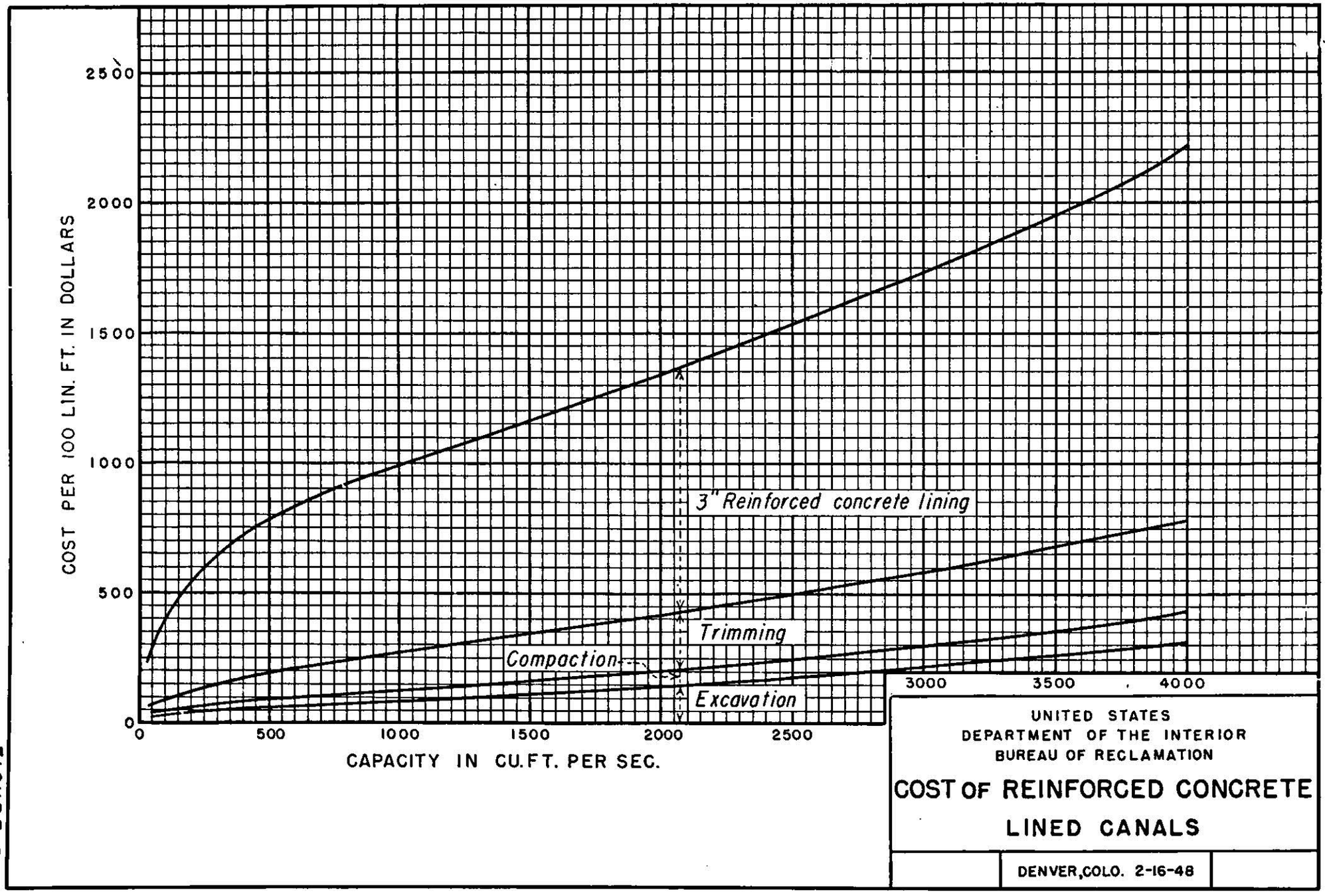

COST PER 100 LIN. FT. IN DOLLARS
2500
2000
1500
1000
500
0
0
500
1000
1500
2000
2500
3000
3500
4000
CAPACITY IN CU. FT. PER SEC.
3" Reinforced concrete lining
Trimming
Compaction
Excavation
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COST OF REINFORCED CONCRETE LINED CANALS
DENVER, COLO. 2-16-48

FIGURE 2

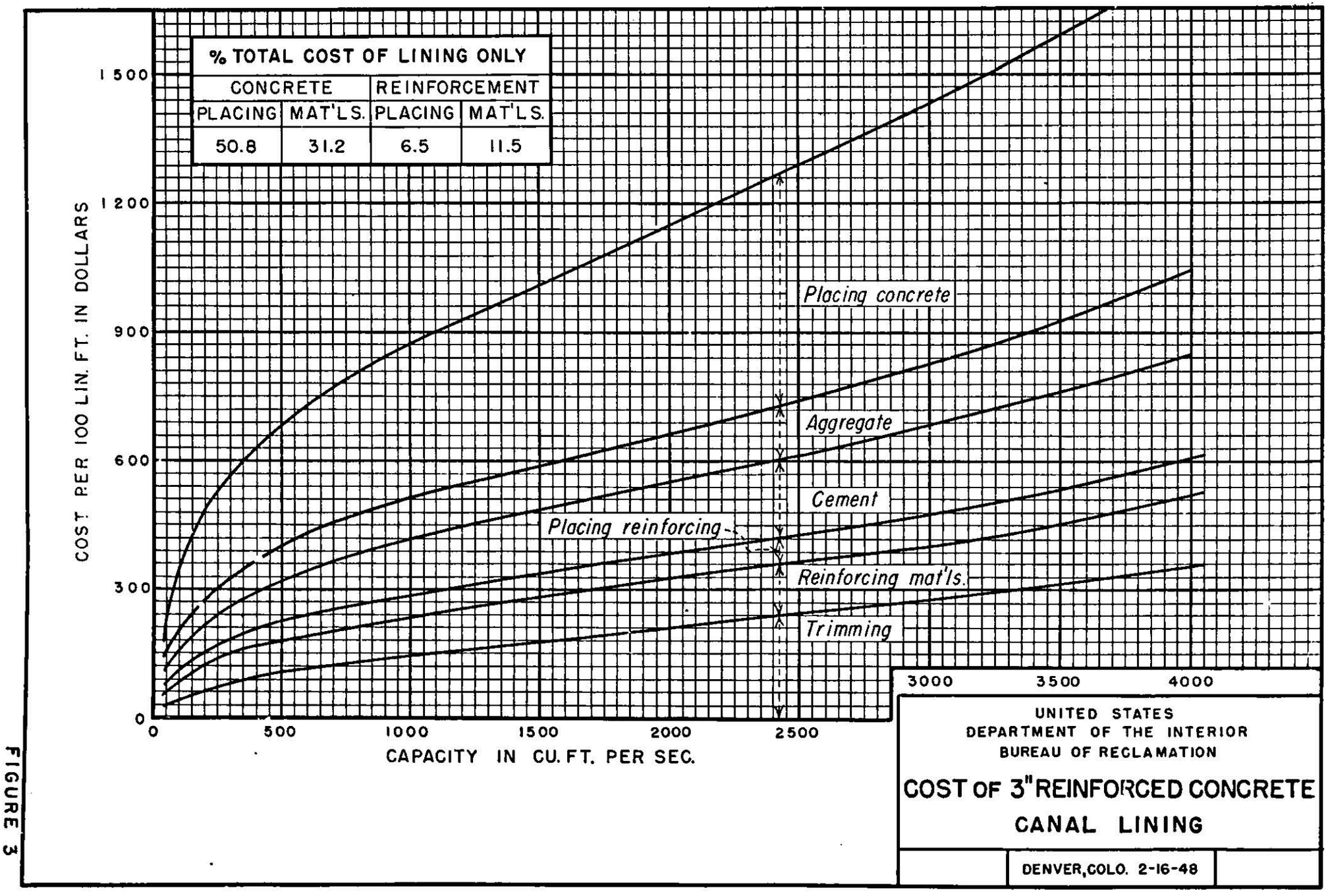

% TOTAL COST OF LINING ONLY
CONCRETE
REINFORCEMENT
PLACING
MAT'L S.
PLACING
MAT'L S.
50.8
31.2
6.5
11.5
COST PER 100 LIN. FT. IN DOLLARS
1500
1200
900
600
300
0
0
500
1000
1500
2000
2500
3000
3500
4000
CAPACITY IN CU. FT. PER SEC.
Placing concrete
Aggregate
Cement
Placing reinforcing
Reinforcing mat'ls.
Trimming
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COST OF 3" REINFORCED CONCRETE
CANAL LINING
DENVER, COLO. 2-16-48

FIGURE 3

largest item of expense and amounts to an average of approximately 50.8 percent of the total, exclusive of trimming. Another important fact shown by the graphs is that the cost of the reinforcing averages about 18 percent of the cost of a 3-inch reinforced concrete lining. Of the remainder, the cost of cement and of aggregates is 18.1 and 13.1 percent, respectively, of the total cost of the lining. Including the cost of trimming, which is usually considered as part of the cost of lining, these data indicate that approximately 63 percent of the total cost of the lining is for construction operations and only 37 percent for materials.

A study of 13 contracts awarded by the Bureau of Reclamation during 1946 for the construction of concrete-lined canals indicates that the average unit cost of a 3-inch reinforced concrete lining during that period was approximately $1.95 per square yard, exclusive of trimming. The average cost of trimming was determined to be about $0.50 per square yard. However, the majority of these contracts was for large canals involving considerable yardages of lining and trimming. Unit costs for smaller canals and laterals will usually be somewhat higher. A study of 1927 to 1940 bid prices and cost reports revealed that the average weighted unit cost of a 3-inch reinforced concrete lining including trimming, was $1.40 per square yard. The Bureau of Reclamation cost index for this type of construction during the first half of 1946 was about 1.7 based on January 1940 costs, and applying this index to 1927 to 1940 prices results in a unit cost of $2.38 per square yard. This figure is probably more nearly representative of the average cost of a 3-inch reinforced concrete lining during early 1946. Applying an index of 2.2 for October 1947 indicates a similar unit cost of $3.08 per square yard for the later period.

The development of a subgrade guided slip-form (Photograph No. 11) for placing concrete lining on the Gila project was a significant step toward economy. This equipment is described further in the section on Canal Construction Equipment. Experience in lining several miles of medium-size laterals has demonstrated the importance of using a traveling-type mixer in conjunction with the slip-form if full advantage of such equipment is to be realized. Stationary-type mixing equipment, even though mounted on a truck, was not capable of supplying concrete at the rate necessary for smooth and economical operation of the slip-form. Although cost records were not kept on the Gila installations, project forces have estimated, from the rate of construction achieved and the necessary manpower required, that a 2-inch unreinforced concrete lining can be placed for about $0.60 per square yard. This figure, however, is not comparable to costs listed in Table 1 since it does not include any charge for overhead or profit and is exclusive of final trimming. The $0.60 per square yard does include all direct labor and materials for mixing and placing the lining and a reasonable rental charge for equipment. This low figure was obtained when using a pugmill-type traveling plant mixer which fed the concrete mix directly into the slip-form hopper. The mix contained pit-run aggregate and about 5.6 sacks of cement per cubic yard.

During the years 1922 to 1945, the Merced irrigation district, California, lined approximately 102 miles of canals, the cost of which varied from year to year. In 1941-42, 28,215 linear feet of existing canal was lined with a 2-inch unreinforced concrete lining. The base width of the lined canals varies from 2 to 5 feet and the water depth from 1.6 to 4.2 feet. The unlined canal section was backfilled with imported soil, reshaped with either team and fresno or dragline to the approximate dimensions required for a lined section, rough graded by hand and then compacted by filling the canal with water impounded to the proper depth by temporary earth dams. After the water had been drained from the canal and the subgrade was sufficiently dry, the section was fine-trimmed and the 2-inch lining installed with screed boards spaced at 6-foot intervals. This lining was placed at a total cost of approximately $0.90 per square yard which included the cost of all labor and material for clearing, backfilling, puddling, trimming, placing lining, installing structures, and engineering. However, this work was accomplished by district forces and the above costs do not include any profit such as would be the case if it were contract work.

Early in 1947, the Turlock irrigation district, California, awarded a contract for the lining of 179,101 square feet of existing canals. A 2-inch unreinforced concrete lining was specified. The method employed in constructing the canal was very similar to that of the Merced irrigation district described above. The canal cross-section varied from a base width of 2 to 3 feet and a depth of 2.25 to 2.5 feet. The total cost, which included labor and materials for removing old structures, regrading existing canal to proper shape, puddling to secure compaction of the fill, trimming of the subgrade and mixing,

placing, and curing, the 2-inch lining was $1.62 per square yard. Of this total, the cost of all concrete materials and their placing was approximately $1.31 per square yard.

Bids were accepted in October, 1947, by the Turlock irrigation district for the placing of 79,100 square feet of 2-inch unreinforced concrete canal lining. The low bids for this work varied from $1.50 to $1.58 per square yard, and it is understood that this cost included regrading of existing canals, puddling, trimming, and placing of the lining. These comparatively low costs for contract work compared to the usual Bureau costs for similar work are attributable to a number of factors. In the first place, a lively competition exists among a number of small local contractors for such work because of a more or less sustained lining program in the local area. But of perhaps greater importance is the fact that the specifications are less rigid and exact than past Bureau specifications have been. These local contractors are thoroughly familiar with the irrigation districts' requirements and know from first-hand experience the degree of workmanship that will be expected and how rigidly the specifications will be enforced. Thus their bid prices do not have to reflect the uncertainties that exist on many construction projects as to the enforcement of the specifications and the degree of inspection.

A considerable amount of concrete lining has been placed in canals in the Lower Rio Grande Valley region in Texas. In 1935, the Hidalgo County Water Control and Improvement District No. 6 installed 4,874 lineal feet of 1-3/4-inch concrete lining in the main canal extension which had a parabolic cross-section with a water depth of 2.58 feet and a top width of 8.34 feet. The lining was reinforced with 4 by 8, No. 12 steel mesh and was installed at a cost of $1.31 per square yard. This work was accomplished by contract and this low price for contract work was due mainly to a plentiful supply of cheap labor.

Concrete Reinforcement Steel

The value of reinforcement steel in concrete canal lining is a point about which a great difference of opinion exists. Many engineers content that reinforcement will be economical over a period of years as a result of a longer serviceable life for the lining and lower maintenance costs. Other engineers feel that under ordinary conditions the benefits of reinforcement are limited and are not equal to the increased difficulties and greater costs of construction resulting from its use.

The main purpose of reinforcement steel is to reduce the size and spacing of contraction cracks that would occur if the slab were not reinforced. This function is commonly referred to as control of the cracking. The formation of cracks due to a volume change resulting from moisture or temperature changes, is dependent upon the degree to which the volume change is resisted by internal and external restraint. The internal restraint is that which develops in a reinforced concrete slab when volume change, resulting from moisture loss, takes place. This volume change is resisted by the reinforcement steel through its bond to the concrete and causes tensile stresses in the concrete and compressive stresses in the steel. With the usual amount of reinforcement, these opposing stresses are sufficient to cause the concrete to fail and to form numerous hair cracks in the slab.

The external restraint of primary importance is that which is developed by frictional resistance between the subgrade and the slab as the latter tends to change its length. Theoretically, the stress resulting from this restraint varies progressively from zero at the free edge to a maximum at the midpoint of the slab. When this tensile stress in an unreinforced concrete slab exceeds the tensile strength of the concrete, a crack results and the stress perpendicular to and at the crack is relieved. The original slab then becomes two separate units and the latter will in turn be cracked in a similar manner if sufficient additional shrinkage occurs.

Had the concrete slab been adequately reinforced, however, the reinforcement would have prevented the formation of an open crack, and as a result, a substantial portion of the stress in the concrete would have been transferred to the steel and across the break. Due to this transfer of stress, the tensile strength of the concrete would be exceeded in a relatively short length and would result in the formation of another crack.

There appear to be two theories as to the ultimate effect of this controlled cracking in concrete slabs of considerable length. One theory, which has not been thoroughly tested in field installations, is based on the idea that the steel does not prevent the shrinkage of the concrete but merely distributes it into numerous hair cracks. When the stress at a crack is transferred to the steel, the bond of the concrete on the steel is destroyed for a short distance on each side of the crack. If the reinforcement is of sufficient cross-sectional area to prevent its failure from this stress, the elongation of the reinforcing over the length on which the bond was destroyed permits the opening of the crack by a small amount. This opening, though, is sufficient to cause the slab near the crack to move slightly on the subgrade so as to reduce the frictional force and also the opposing tensile stress in the steel. This movement at each crack, and the subsequent reduction in steel stress, is sufficient to prevent the accumulation of stress in the steel to a point where the yield-point of the steel will be exceeded. If this analysis is correct, the maximum stress that would be developed in the steel would never exceed that required to produce a tensile failure in the concrete slab. Theoretically, the maximum amount of steel required would be that at which the tensile strength of the steel slightly exceeds the tensile strength of the concrete. This theoretical amount will depend upon the yield-point of the steel and the tensile strength of the concrete. Experience has shown that a considerably smaller amount than that theoretically required will give satisfactory results. This may possibly be due to the reduction in tensile stress resulting from a slight opening of the crack and to the fact, as many engineers contend, that the stress is concentrated in the area of concrete immediately adjacent to the reinforcement instead of being more or less uniformly distributed across the cross-sectional area of the concrete. Most authorities agree, though, that an area of steel equal to about 0.25 to 0.3 percent of the area of the concrete should be adequate for most structures.

According to the other theory, when the stress in a reinforced concrete slab is transferred to the steel, the bond is destroyed as before and a small opening of the crack occurs. This opening, though, is not sufficient to prevent the progressive increase in the tensile stress in the steel at each successive crack. Eventually the yield-point of the steel is exceeded and an open crack occurs. In this case, the minimum amount of steel that would be of value in controlling cracking resulting from external restraint is, theoretically, that amount at which the tensile strength of the steel and the concrete is approximately equal. Actually, this minimum amount could probably be substantially less because of the reduction in stress due to the slight openings of the cracks. An increase in the cross-sectional area of the steel will increase the distance between open cracks resulting from failure of the steel.

In either case, if the steel is to effectively control contraction cracking, it must be used in an amount sufficient to prevent its being stressed beyond the elastic limit when the concrete cracks. While it is known that the actual cross-section area of steel required to accomplish this is less than that required from a theoretical analysis, there is not sufficient evidence to determine the permissible reduction in the theoretical amount. But as stated before, about 0.25 to 0.30 of 1 percent of steel has been found to be satisfactory for control of cracking in other structures and it is felt that this should be fairly applicable to canal linings. The wisdom of using a substantially smaller amount of steel to control cracking appears questionable.

A function of reinforcement steel that is important in concrete highway pavements is that in addition to causing fine hair cracks, it aids in holding the ruptured pieces of slab together and facilitates the transfer of wheel loads across the crack. This last consideration--the transfer of unequal loading across the cracks--is of little importance in canal lining design because it is seldom that the canal lining will be subject to anything but a uniform loading. A comparable situation might occur if there were considerable differential settlement of the canal subgrade which would produce an uneven support for the lining. This settlement, especially in the case of a large canal, would possibly cause serious cracking, and reinforcement would aid in holding the pieces of slab together. For that reason some engineers feel that although reinforcement in sufficient amount to control cracking in concrete linings may not be warranted under normal conditions, a light reinforcement is desirable and justifiable in that although it may not materially effect the size and spacing of the cracks it does aid in preventing the easy displacement of the broken sections of the slab. This should be particularly true for thin concrete linings in which the edges of the cracked slab would have limited interlocking action. As a compromise measure between well-reinforced and unreinforced concrete linings, the linings of the Coachella canal in southern California, and the larger canals and laterals

of the Yuma Mesa division of the Gila project in southwestern Arizona, were constructed with about 0.1 of 1 percent of reinforcement consisting of wire-mesh fabric. This low percentage of steel was deemed sufficient to hold the ruptured pieces of slab in position.

Unbalanced hydraulic pressures under the lining will, if of sufficient magnitude, cause flotation of the lining. Although reinforcement would not aid materially in preventing the cracking of this lining by these forces, it would, in this case also, assist in holding the cracked slab in place. But the benefits of this assistance may be questionable since the reinforcement may increase the extent of the damage by preventing the local failure of a small section that would relieve the pressure in its early stages. Photograph No. 10 is of a section of the Contra Costa canal in California that was badly damaged by flotation. If the lining in this case had not been reinforced, it is reasonable to assume that the damage would have been confined to a smaller area which would have broken through earlier and relieved the pressure, thus reducing the damage considerably.

This is further borne out by the field report on the thin, unreinforced, mortar lining of the Gage canal near Riverside, California, which sustained considerable damage from unbalanced hydrostatic pressure. It was reported that the company's general manager averred that a light reinforcement would have made matters worse since large sections instead of small sections would have been broken out.

The use of reinforcement steel substantially increases the cost of construction of a concrete lining. Recent unreinforced concrete lining installations in small canals and laterals have demonstrated the feasibility and economy of using a simple slip-form canal lining machine which rides upon the trimmed subgrade of the canal prism. See Photograph No. 11. No expensive tracks are required to support the machine. However, they have not been employed for installing reinforced concrete lining because the reinforcement steel which must be laid prior to the placing of the concrete has precluded the use of a machine which rides on the subgrade. Instead, a more complicated machine must be employed, supported on tracks set to correct grade and alinement on the canal banks or supported on some type of tread.

In addition, in order that the reinforcement steel may function effectively and be protected from corrosion, it must be maintained in its proper position in the slab. This requires that the steel be placed accurately. Drill cores from canal lining have shown that displacement and lowering of the steel during placement of the concrete is a common occurrence, and positive means must be taken to insure that no displacement occurs. These drill cores have also shown that unless considerable care is exercised in the control of the concrete mix and in the placement of the concrete with the proper amount of vibratory compaction, honeycombing of the concrete near and particularly under the reinforcement bars may occur. These honeycombed areas, in addition to reducing the strength of the lining, may result in early failure of the steel due to corrosion by water collecting in these pockets.

Reinforcing steel undoubtedly improves the structural quality and strength of concrete canal lining in certain respects. Approximately .25 to .30 percent steel largely controls contraction cracking so as to form practically watertight hair cracks. It aids in holding the separate slabs and broken pieces of slab together, and also increases the factor of safety against canal failures. Under conditions where subgrade conditions are such that considerable settlement or frost heaving can be anticipated or where safety against canal failure is of unusual importance, the use of reinforcement may be desirable and justifiable. However, a decision to use reinforcement should be based upon a consideration of the factors that have been discussed, and the benefits accruing from its use should be carefully balanced against the extra cost.

There are a number of existing unreinforced concrete canal linings which have given satisfactory service for many years. The Riverside Water Company of Riverside, California, has adopted a 2-inch unreinforced concrete-lined section as the standard for all lined canals. Also, the Chief Engineer of the Turlock irrigation district in California was reported to have recommended in 1940 that canals up to a 24-foot perimeter be lined with 2-inch unreinforced concrete, and that a 3-inch unreinforced concrete lining be used for larger canals.

If sufficient reinforcement is used to control cracking, the additional benefits of providing contraction joints in the same lining appear limited. The primary function

of both is to control the contraction cracking, and experience has shown that either one, if properly employed, will satisfactorily accomplish this. However, if both reinforcement steel and contraction joints are used in the same lining, a very important factor, which is often overlooked in the design and construction of the lining, is that the reinforcement should be broken at the joints or grooves if the latter are to be of any benefit. With continuous reinforcement, a hair crack may occur in the joint or groove similar to those that will occur elsewhere in the slab, but the steel will prevent the formation of an open crack that would relieve the stress. Such a crack would then occur only where the stress transferred to the steel by the cracking of the concrete exceeded the yield-point of the steel; this would not necessarily be in joints or grooves. As reported in the Bureau of Reclamation's Concrete Manual, (56) experimental sections of 6-inch reinforced, articulated lining near the New River siphon on the All-American canal have shown that the more or less random transverse cracking associated with continuous longitudinal reinforcement may be largely eliminated by concentrating the shrinkage at transverse joints spaced about 20 feet apart at points where the reinforcement was broken. One method of accomplishing this is to place the reinforcement so that the longitudinal bars are placed with one end of the bars about 2 inches from the groove at one edge of the panel and extending about 6 inches past the groove at the opposite panel edge and into the adjacent panel. These projecting ends are covered either with paper sleeves or bituminous material to prevent bond of the concrete and steel, and serve as dowels between the panels. If contraction grooves and reinforcing steel are used in this manner, the grooves will tend to produce a straight, uniform crack at the break in the reinforcement instead of a jagged, irregular break. This would aid in the filling and maintenance of the cracks.

Contraction Joints

The design of concrete canal linings is similar in many respects to that of a concrete highway pavement. Unlike the analyses of most other engineering structures in which the basic principles of design are commonly accepted and in which the computations can be made with a reasonable degree of accuracy, concrete linings and pavements cannot be subjected to a dependable mathematical analysis. This fact is primarily due to the variable and indeterminate nature of the factors involved in the latter two types of construction. Many of the problems can only be solved by field tests and studies of service records of existing structures. That a number of the problems concerning pavements have not been satisfactorily solved is attested by the wide divergence of opinion and practices among highway engineers as to a properly designed pavement, and by the many different types of slabs that have been adopted by various state and Government agencies engaged in highway construction. During the past 25 years, the United States Public Roads Administration and others have made extensive tests and studies of practically all phases of pavement design in a continuing effort to solve the problems of design and construction. Although this has not resulted in any exact method of analysis, it has given the engineers a better understanding of the problems and has led to a marked improvement in pavements.

A comparable situation exists in regard to canal lining. But due perhaps to the smaller volume of this type of construction, much less consideration and effort have been devoted to the solution of the difficulties and problems encountered. Since there is considerable similarity between the design of pavements and canal linings, many of the results and conclusions reached by highway engineers as a result of these tests and studies should, within limitation, be applicable to canal lining design.

One of the disadvantages of a concrete slab for highway pavements or canal lining is the inevitable cracking to which it is subject. It is impossible to construct an exposed, concrete slab of any great length that will not be cracked by the induced stresses. These stresses result principally from either a temperature or a moisture change in the slab or from a combination of the two. Concrete expands as the temperature increases, and contracts as the temperature decreases. In much the same manner, concrete expands and contracts with changes in moisture content. The amount of contraction or expansion, and the amount of induced stress, will depend upon the water-cement ratio, cement content, kind and size of aggregate, age of the concrete, etc., and these factors affect the modulus of elasticity of the concrete.

The stresses induced in a concrete slab by a temperature variation are of two kinds; namely, stresses caused by the temperature being uniform throughout the slab but varying with time, and stresses caused by a temperature differential between the upper and lower faces of the slab. If the temperature is uniform throughout the slab but varying with time, simple tensile or compressive stresses will be produced. As the concrete cools, the contraction tends to be resisted by the friction of the slab on the subgrade. The magnitude of the resulting tensile stress will depend upon the amount of frictional resistance between the slab and the subgrade and upon the vertical pressure on the subgrade. This stress is normally quite low when the slab is unloaded, but the maximum may occur when a warm slab is suddenly subjected to a full load by filling the canal. Thus, if the temperature of a 3-1/2-inch slab 20 feet long decreases when the canal is suddenly filled to a depth of 10 feet, the resulting tensile stress, assuming a coefficient of friction, "f" of 1.8, would be:

$$S_t = \frac{W\,L\,f}{24\,t} = \frac{(44+624) \times 20 \times 1.8}{24 \times 3.5} = 286 \text{ psi}$$

where:

W = Vertical load in pounds per square foot

L = Length of slab in feet

This stress, while it is dependent on the coefficient of friction which may vary from 0.5 to 3.0 dependent upon the type of subgrade, may be sufficient to cause cracking.

The compressive stress resulting from either a temperature or moisture increase is of little concern for two reasons. In the first place, a slab which is fully restrained at both ends and subjected to a 100° F increase in temperature will develop only about 1,500 psi of compressive stress. This is considerably below the average compressive strength of good concrete. Secondly, the expansion of concrete due even to complete saturation is never as great as the contraction that results from the drying-out of the concrete shortly after the placing. Unless the contraction cracks resulting from the setting shrinkage have become filled with incompressible material, considerable expansion due to an increase in temperature can occur before the cracks will be closed. However, the lining may act somewhat as a thin column, and even this low compressive stress may cause buckling, particularly of thin linings placed on uneven subgrade. But if the contraction cracks or grooves are filled with an elastic material to prevent the entrance of sand or silt, and if the subgrade has been finished to a reasonably uniform surface, such failures are not likely to occur.

When one face of a slab is at a higher temperature than the other, the edges of the slab tend to curl in the direction of the colder face. Assuming a uniformly varying temperature differential between the two faces of the slab, if there were no restraint to this curling action, no warping stress would be induced in the slab. But the restraint varies from a minimum, due to the weight of the slab itself, to complete restraint, as in a slab having no free edges. The amount of the warping stress resulting from this curling action will depend upon the amount of restraint, the temperature differential between the two faces, and the thickness of the slab. An increase in any of these factors will increase the warping stress. As a result of a large number of tests conducted by the Bureau of Public Roads at Arlington, Virginia, it was found that the maximum temperature differential between the two faces of a concrete pavement slab was closely approximated by the rule that the temperature differential is equal to 3h, where "h" is the slab thickness in inches. Using this value for the temperature differential and the Bureau of Public Roads' formula for warping stresses, the stress in a 3-1/2-inch concrete slab under full restraint to warping is only approximately 93 psi, which is considerably below the average 28-day flexural strength of approximately 600 psi for pavement concrete. However, a uniformly varying temperature differential seldom exists throughout the slab thickness, and the temperature stresses may be considerably greater than those indicated for this reason.

Variations in the moisture content of the slab produce stresses in the same manner as temperature variations; an increase in moisture content causes expansion of the slab and vice versa. Tests have indicated that the maximum expansion due to complete saturation is equal to about two-thirds of the contraction which occurs from the initial

drying shrinkage and is approximately equal to the expansion that would be produced by a 100° F increase in slab temperature. The tensile stress produced by a uniform decrease in moisture content is about equal to that produced by a decrease in temperature since the amount of contraction has little effect on the value of the tensile stress. The compressive stress resulting from an increase in moisture content may be larger than will ever occur from a temperature increase but should never exceed the average compressive strength of concrete.

The stress that may possibly cause a large part of the cracking in a concrete canal lining is the warping stress resulting from a differential in moisture content between the upper and lower faces of the slab such as would occur when water is turned out of a canal and the upper face looses its moisture. Little data are available on the variation of the moisture content of a slab or the warping stress that may result. However, such stress may be induced in the freeboard of a concrete canal lining, due to the portion of the lining below the water surface being completely saturated, whereas the freeboard is comparatively dry. This may account for the larger number of cracks in the side-slope slabs than in the base slabs.

There is at present no generally accepted method of computing the value of these stresses because of the many variable and unknown factors. The canal lining cannot economically be designed to resist them and to eliminate cracking, but the latter can be controlled to a limited extent in two ways.

Reinforcing steel can be used in the concrete to cause the formation of hair cracks at relatively close spacing instead of larger cracks at greater spacings as in an unreinforced slab. The effectiveness and value of reinforcing in concrete canal lining has already been discussed.

Another method of controlling cracking in concrete slabs is by the use of contraction joints or weakened planes spaced at the proper intervals so that the cracks will occur in these joints instead of at variable intervals. When a concrete canal lining is placed by hand, it is a common practice to construct the lining in alternate panels, and the bond between the panels, being weaker than the slab itself, concentrates the cracking in the construction joint. In addition, a large number of different types of formed contraction joints have been used in the past for this purpose. The most common type of contraction joint at present is the weakened-plane, sidewalk dummy joint such as the Bureau of Reclamation employs almost exclusively. This joint is merely a narrow groove formed in the concrete to a depth of about one-third of the thickness of the lining. This forms a weakened section where cracking will usually occur.

Due to the undeterminate nature of the stresses which may occur in a lining slab, the spacing of contraction joints is not subject to satisfactory theoretical analysis. It will vary with size of canal, thickness of lining, climatic conditions, type of subgrade soil, and properties of the concrete. The choice of spacing must be based on past experiences and on the performance of existing linings. Almost all reinforced concrete linings designed and constructed by the Bureau of Reclamation during 1935 and 1936 were provided with a transverse contraction joint spacing of 8 feet. This was changed to a spacing of not less than 8 feet nor more than 20 feet for canals designed during 1937 to 1939. During 1940 to 1941 this joint spacing was established at 10 feet for reinforced concrete canal lining, but since 1941 the joint spacing has varied to a certain extent with the size of the canal. For canals of greater than about 500 cfs capacity, the spacing has been about 15 feet, and for canals of less than 500 cfs capacity, the spacing has been about 10 feet. Until recent years, very little unreinforced concrete canal lining was installed by the Bureau of Reclamation. However, a considerable amount of it has now been placed and the joint spacing for this type of lining has been about 3 feet less than for a reinforced concrete canal lining in the larger canals and the same as for a reinforced lining in smaller canals.

A study of all the data and information on this subject, that are available in reports and records of existing linings, indicates that transverse contraction joint spacing of 8 to 10 feet for unreinforced concrete linings in the smaller canals is usually adequate. Only a limited amount of data is available on the performance of the larger canals with regard to contraction cracking, but it appears that a spacing of joints at about 12 feet is satisfactory for unreinforced linings in these larger canals. If contraction joints are to be used in a discontinuously reinforced canal lining, the above spacings can probably be increased by about 3 feet.

Longitudinal joints or grooves are not required in the smaller canals to control the cracking. However, in the large canals of 3,000 to 5,000 cfs capacity, it is the practice of the Bureau of Reclamation to specify one or more grooves in the base slab and each side-slope slab depending on its size.

If a concrete lining is to effectively fulfill its intended purpose, it is important that the open crack be filled with an impermeable, elastic material to prohibit the loss of water through the cracks. In addition, the elastic filler prevents the entrance of incompressible material, such as sand and silt, into the crack, and thus permits considerable expansion of the concrete before compressive stresses are developed. If dummy grooves are used to control cracking, it is advisable to fill the grooves at the time of construction, since filling at a later date would entail prior cleaning of sand and silt from the grooves.

The Bureau of Reclamation follows this practice of requiring the filling of the grooves at the time of construction and provides that the filler may be applied prior to the application of the curing compound and as soon as the concrete has become sufficiently hard to prevent appreciable damage to the groove or the concrete. Further provision is made that the grooves may be filled subsequent to application of the curing compound, providing positive means are employed to insure that the compound does not enter the groove since it prevents a good bond between the filler and the concrete. The material used by the Bureau for filling these grooves is a special cold-applied, internal set-up mastic filler. This filler is made by mixing dry ingredients of powdered asphalt, short-fibred asbestos, diatomaceous earth and powdered limestone with liquid ingredients of an asphaltic fluxoil and a plasticizer to form a workable semiliquid compound.

Thickness of Concrete Linings

The thickness of a concrete canal lining is usually determined from empirical knowledge of similar existing installations. The factors of location, canal size, subgrade conditions, exposure, method of construction, procedures of operation and maintenance, and canal hydraulics must be reviewed and considered in establishing the lining thickness.

In general, the thin unreinforced mortar linings have been most successfully used in the temperate climates such as southern California and the Lower Rio Grande Valley of Texas. In the rigorous climate of Montana, Wyoming, and Washington, heavier reinforced concrete linings have been usually constructed. To provide a measure of resistance to surface disintegration of the concrete and also to frost heaving in the subgrade, a concrete lining which will be subjected to severe climatic conditions is usually made 1/2 inch greater in thickness than a similar lining in a mild climate.

In all locations the size and importance of the canal have been important factors in selecting the thickness of lining. The additional resistance to rupture provided in a thicker lining has resulted in the major canals being lined with a heavier section than considered necessary or justified for lateral distribution systems.

Under present practice, concrete linings designed by the Bureau of Reclamation will vary from a 2-inch minimum to the 4-1/2-inch reinforced concrete lining on large canals of the Columbia Basin project. The 2-inch thickness is generally confined to smaller canals and laterals. The average thickness of linings constructed by the Bureau on the larger canals is about 3-1/2 inches.

An important consideration in establishing minimum thickness for linings of concrete is the construction procedures involved. Any attempt to further reduce the thickness below the minimum previously discussed would involve painstaking construction and inspection to insure uniform work, which would no doubt result in greater cost than a somewhat heavier lining.

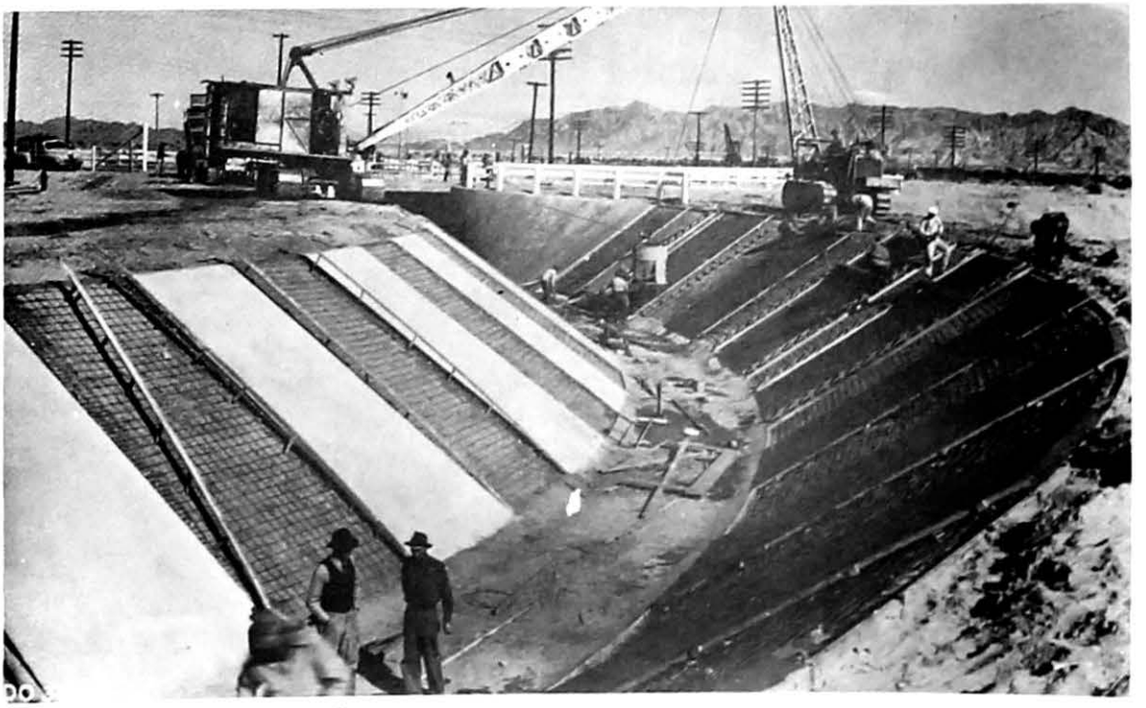
In an operating canal which is properly maintained, the rate of flow is primarily dependent upon the roughness of the surface over which the water flows. The coefficient of roughness used in the hydraulic design of canals represents an evaluation of the degree of roughness of the canal surface and its retarding effect on the flow of water. An important point sometimes overlooked by design engineers is that this coefficient of roughness should not be based on the degree of original surface finish applied to the lining at the time of construction but rather upon the surface finish that will pertain after a few years operation.

If the water conveyed in the canal is relatively clear and if experience in the locality indicates that little moss or algae growth on the lining can be anticipated, the original surface finish will probably be effective throughout most of the life of the lining. In this case, a smooth surface which would increase the carrying capacity of the canal may be warranted. But if the water conveys considerable sand or silt which may be deposited in the canal or if the surface of the lining may become covered with moss or algae growth, these two conditions may have a greater effect on the efficiency of the canal than the degree of original surface finish. Under these conditions a very smooth, hand-troweled surface would be of little value and the cost of securing it would be unjustifiable. Since a majority of the irrigation canals carry water which contains a certain amount of sand or silt and many are subject to the growth of moss or algae, it appears that a reasonably smooth, well-filled slab should normally be adequate.

The coefficient of roughness, "n," of a concrete-lined canal has for many years been assumed by the Bureau of Reclamation and most irrigation engineers to be 0.014 for design purposes. To secure the corresponding degree of surface finish, the Bureau specifications for standard canal lining construction require that the surface be finished so as to produce a finish equivalent in evenness, smoothness, and freedom from rock pockets and surface voids to that obtainable by use of a long-handled steel trowel. In 1939, Fred C. Scobey (176) presented the results of numerous tests to determine the coefficient of roughness for irrigation canals, and in regard to the use of $n = 0.014$ for concrete-lined canals stated, "It is conservative for ordinary conditions, in that modern methods yield original surfaces at least one or two points lower and hence some acquired roughness is discounted."

Photograph No. 2

Hand screeding and finishing--2-inch concrete--
Turlock I. D., California.



Photograph No. 3

Placing reinforced concrete lining--Gila Project,
Arizona, 1941.

Photograph No. 4

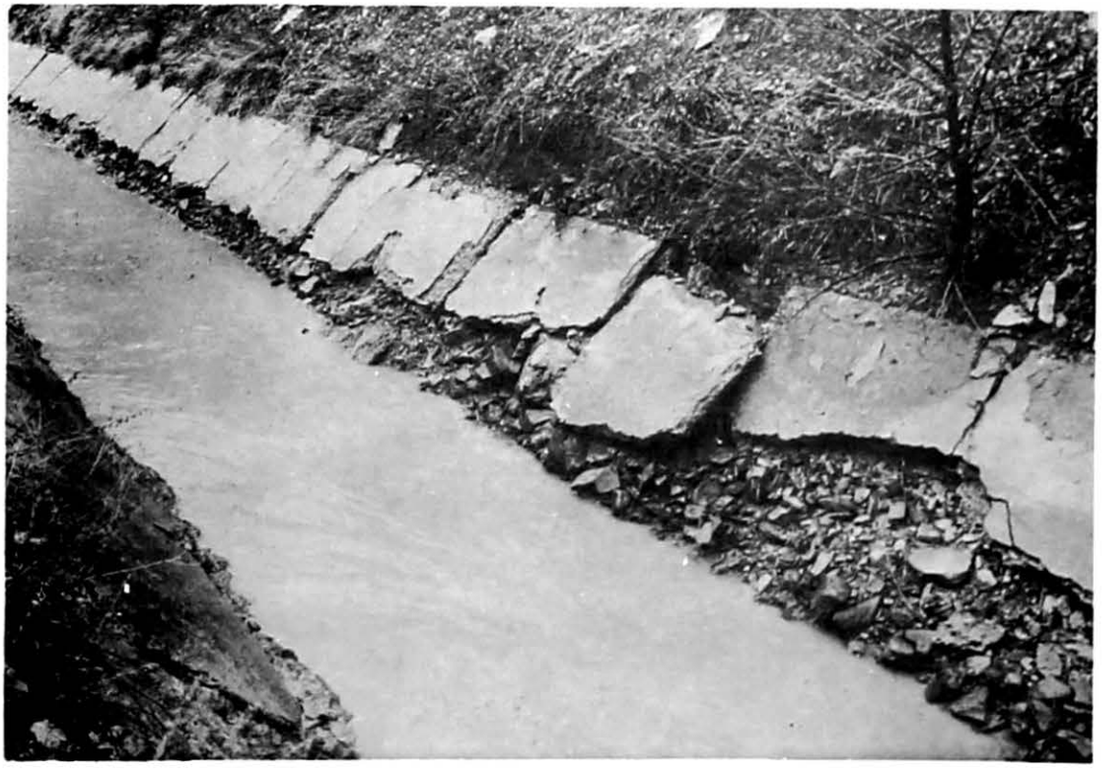
A 3/4-inch mortar lining about 52 years old--Gage Canal--Riverside, California.

Photograph No. 5

Two-inch unreinforced concrete lining about 1 year old at time picture was taken in 1946--Turlock Irrigation District, California.

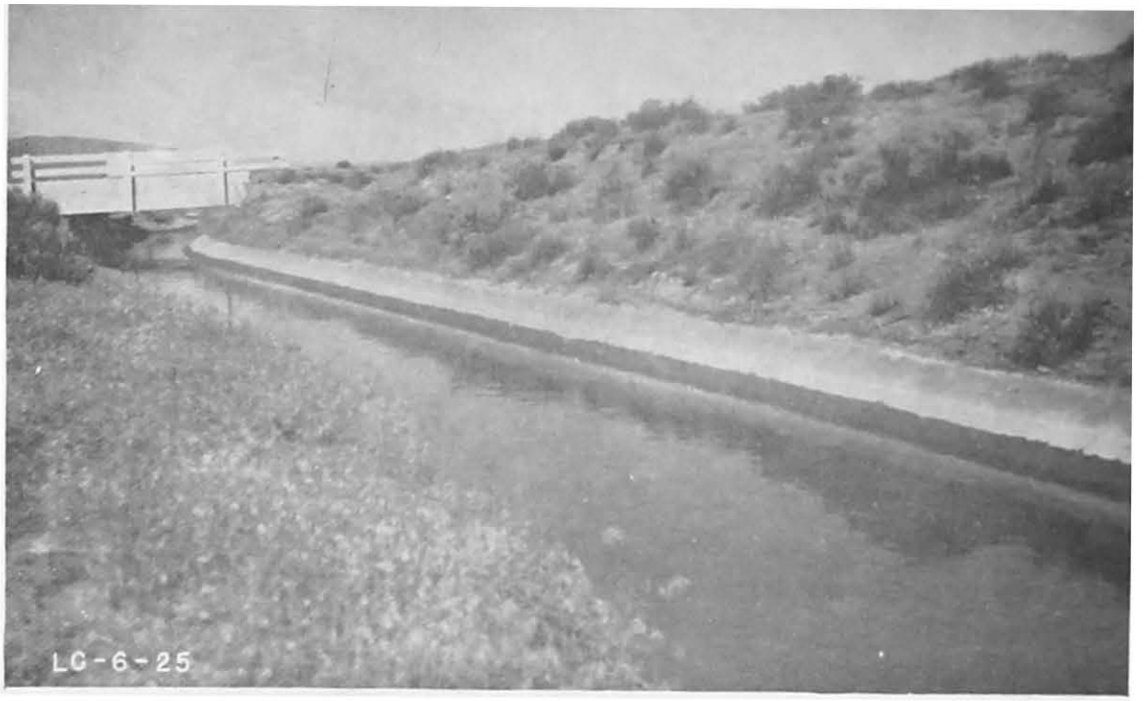
Photograph No. 6

Damage to concrete by frost heaving--Terrace Heights--
Yakima Project, Washington.



Photograph No. 7

Disintegration and spalling of a concrete lining at
and below winter water surface due to freezing
action--Strawberry Valley Power Canal--Utah.



Photograph No. 8

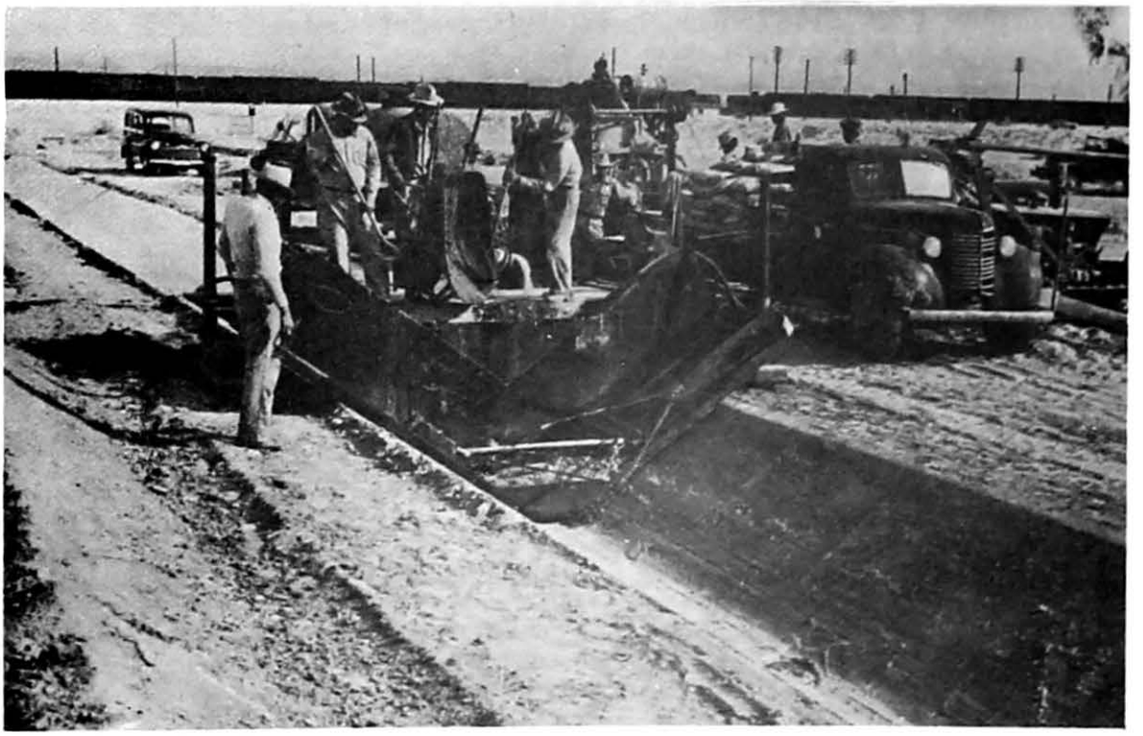
Two-inch unreinforced concrete lining--Franklin County, I. D., Pasco, Washington--Constructed 1921--Picture 1947.

Photograph No. 9

Two-inch unreinforced concrete lining more than 30 years old--Burbank Project--Pasco, Washington.

Photograph No. 10

Failure of a concrete lining due to floatation--
Contra Costa Canal, California.



Photograph No. 11

Simple slip-form lining machine placing unreinforced
concrete--Tracks on canal banks are not required--
Yuma Project, Arizona.

Precast Concrete Linings

Linings built of precast concrete slabs have some advantages over cast-in-place concrete under certain conditions and may be relatively economical. If the joints are sealed with a flexible material and the slabs placed so that the joints are continuous rather than staggered, such a lining will have the ability to conform to slight movements of the subgrade. It is recognized that the hand labor in placing the slabs and sealing the joints cannot readily be avoided, but it has been determined that slabs of satisfactory dimensions can be fabricated at reasonable cost in adaptations of available building block machinery. Slabs could thus be manufactured at low cost under ideal conditions in centrally located plants and distributed to isolated sections to be placed by unskilled labor with a minimum of equipment. This type of lining is particularly adaptable for placement during winter months in localities where cast-in-place concrete would be in danger of freezing or in any climate where the demand for water is such that only short intervals of time are available for lining operations. If the joints in the precast and precured slabs are sealed with an asphalt mastic, such a lining could withstand either freezing temperatures or flowing water as soon as completed without appreciable damage. These precast slabs also appear promising for use by small maintenance crews in lining or repairing short sections or by individual farmers for lining their own small ditches since no particular skill and practically no equipment is required for placing. Perhaps the chief disadvantage of this type of lining is that the usual flat slabs cannot be used in curved sections and is, therefore, limited to tangents.

A 400-foot length of lateral on the Roza division of the Yakima project was experimentally lined with precast concrete slabs in the late fall of 1946. Two types of slabs were used; one 2 feet square and the other 8 inches wide by 2 feet long. The square slabs used in the bottom of the canal were 2-1/2 inches thick with three plain butt edges and one shoulder edge. These were laid two wide, butt to butt, so that a continuous shoulder was provided along the toe of each bank for supporting the slabs on the bank. See Photograph No. 12. The side or bank slabs were only 2 inches thick with three overlapping edges and one bevel edge. These were laid two high so that adjacent slabs lapped, the bevel edge of the lower slab fit into the bottom slab shoulder, and the bevel edge of the upper slab was placed at the top of the lining. All joints were sealed by hand with an asphalt mastic. The 8- by 24-inch slabs were all 2 inches thick with a simple tongue-and-groove on all form edges similar to the common concrete silo stave. These were laid in a round bottom section and sealed with asphalt mastic as shown in Photograph No. 13. Curves were left unlined and later gunited (see Photograph No. 14).

It is too early to predict the probable life or serviceability of this installation. Frost heaving the first winter caused a bulge in the bottom of the square block lining (Photograph No. 15), but project forces anticipated that this would disappear with the return of warm weather and the crack could be easily resealed with asphalt. In addition, storm water entering behind the lining has bulged the 8- by 24-inch slabs on the side slopes at one point as shown in Photograph No. 16. These slabs were all manufactured by hand methods in metal forms and no cost data are available for the precast units. Costs were kept, however, on the placing and sealing operations which were $0.54 and $0.88 per square yard, respectively, for the square slabs and the 8- by 24-inch. A local cement products company which cooperated by prefabricating the test slabs has estimated that it can manufacture the 8- by 24-inch slab in large quantities by machine methods to sell for $0.81 per square yard or about $0.12 each. The silo stave-type slab is better adaptable to machine manufacture but slabs larger than 8 by 24 inches can be made which would be more economical to place. In the field report on the Yakima installation, emphasis was placed on the fact that precast blocks demand a much more exact subgrade than other types of lining.

In special cases where an adequate supply of cheap labor is available or where standby personnel can be utilized to advantage, it may be feasible to manufacture slabs by hand methods. This was done with Civilian Conservation Corps labor in 1940 and 1941 on the Yuma and Carlsbad projects. On the Yuma project about 6 miles of various laterals were lined with a 4-inch cast-in-place concrete base and precast slabs on the side slopes. The precast slabs were 4 feet by 6 feet by 1-1/2 inches thick reinforced with a 3/8-inch rod in each of two ribs on the underside. The slabs were cast at the project yards and hauled to the various laterals as needed. The cast-in-place base was poured first with a 2- by 4-inch shoulder on each edge to support the slabs. The precast slabs were placed on these 2- by 4-inch shoulders and supported at the proper grade and elevation by temporary struts while backfill was tamped behind the slabs. (See Photographs Nos. 18, 19, 20, and 21). Water was then ponded in the lateral and slabs that were displaced by differential settlement of the subgrade were jacked into position

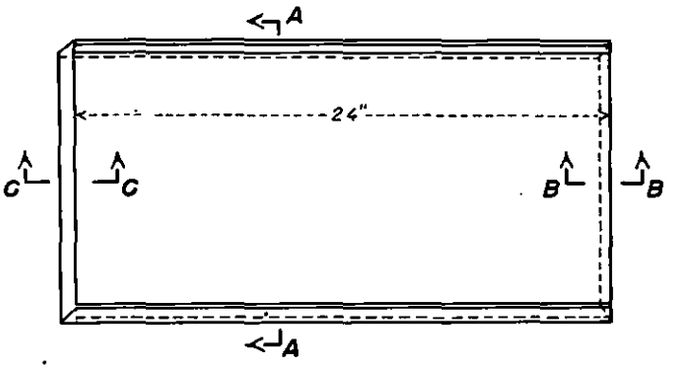
and additional tamped backfill placed under the slab. No cost data are available. The condition of the lining in 1947 was reported as good, but the joints had never been filled and vegetation which has taken root in the joints may cause difficulty. The following comments on this type of lining were stated in a memorandum to the Commissioner from C. B. Elliott, Yuma, February 21, 1942: "In new work where the canal or lateral section could be excavated to neat lines, it is not believed slab lining would be justified. However, on an old project where the existing ditch section averages twice the size required for the replacing concrete section and where numerous interruptions of the work are necessary to supply irrigation water, it is believed there is a place for precast slab lining."

On the Carlsbad project a similar cast-in-place base slab was used, but precast slabs for the side slopes were 3 by 12 by 18 inches and 3 by 12 by 9 inches. These were placed in a staggered pattern, the butt joints sealed with portland cement mortar and backfill tamped behind them as on the Yuma work. Only about 800 linear feet of this type lining was placed by CCC forces on the Carlsbad project but maintenance forces are still using these slabs to good advantage for lining short stretches of lateral and for maintaining check transitions on the main canal. Project forces, however, have expressed the belief that this type of lining is not adapted to contract work. Photograph No. 22 shows a stretch of precast slabs placed by CCC forces in 1941 and Photographs Nos 23 and 24 show maintenance forces placing them in 1947.

A special semicircular precast unit was used to line a 10-mile reach of hillside canal on the Tieton project in 1910. These reinforced units, 4 inches thick and 24 inches long, were cast in metal forms curved to an internal diameter of a little over 8 feet. Joints between the units were sealed with a portland cement mortar. The cost in place was recorded as $5.80 per linear foot or $3.47 per square yard of lining. This was, quite obviously, not a low-cost lining, but is mentioned here by way of general information.

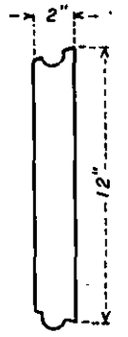
Precast slabs have been used to a large extent by the United States Public Health Service for lining small drainage ditches for malaria control in some of the southern states. They developed a slab with a special tongue-and-groove joint similar to Type B in Figure 4 which weighed about 55 pounds. These were manufactured by hand in a central plant using wooden forms. It was reported that after the forms were constructed, three men could assemble them and with the aid of a concrete mixer could cast 200 slabs per day. It was further stated that after the ditch was in shape to receive the lining, three men could place an average of 300 linear feet of lining, three slabs wide, per day. Joints between the slabs were not sealed in any way. In a recent communication Mr. H. A. Johnson, Senior Sanitary Engineer, Public Health Service, Memphis, Tennessee, wrote, "Vegetation working through the joints of precast linings is very troublesome and is the main factor tending to displace the slabs. After experimenting in the field with cast-in-place and precast types, we are very much of the opinion that cast-in-place linings are much more suitable for our work when they can be installed."

As one phase of the Lower-Cost Canal Lining Program, a series of tests were conducted in the Denver laboratories involving a number of different types of slabs. Full scale models, Photograph No. 17, were constructed using both cement mortar and asphalt mastic for sealing the joints. The models were equipped so that the test lining could be flexed to simulate frost heaving and tests were made with the model full of water to determine when failure by leakage occurred. The type of joint which best met the requirements was a modification of the Public Health Service slab with a beveled edge but a rounded tongue-and-groove (Type A in Figure 4). The two longitudinal edges were formed at an angle equal to one-half that made at the toe of the bank in a trapezoidal section. By reversing one slab, a plane surface or an angle may be obtained without sacrificing the proper fit of the joint. The further use of two different radii in the tongue-and-groove provides additional flexibility and space for the sealing mastic in the cross-sectional crescent as well as the opening on top and at bottom. However, the advantages of this joint over a simple tongue-and-groove, as used in the common silo stave, may be offset by the difficulty of manufacturing in large production machinery and may therefore be higher in cost. The asphalt mastic was far superior to portland cement mortar for flexibility and as a water seal. In fact, it was necessary to coat most of the mortar joints with asphalt to secure a watertight section. Rigidity was obtained by staggering the joints or a very flexible lining resulted if joints were continuous.



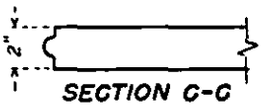
TYPICAL SIDE SLOPE OR BOTTOM SLAB

VOL.= 0.32 CU. FT. WT. 43 LBS.

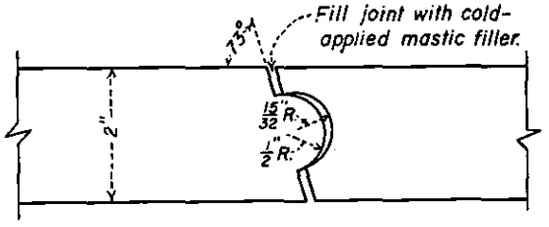


SECTION A-A

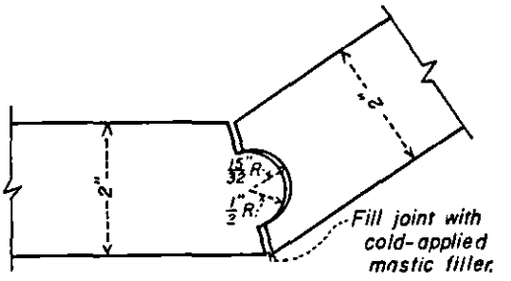
SECTION B-B



SECTION C-C

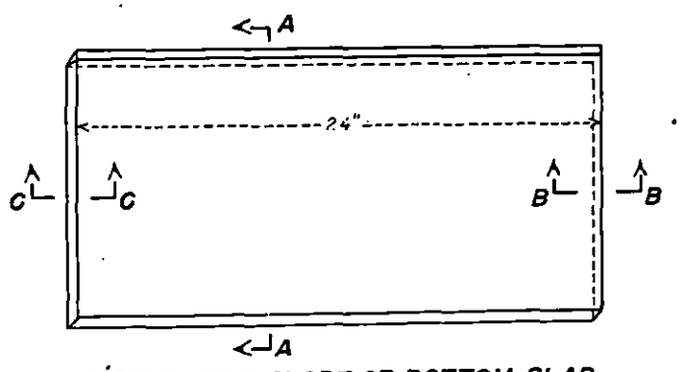


DETAIL OF JOINT ON SLOPE OR BASE



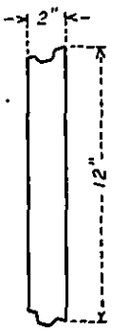
JOINT AT BOTTOM OF SLOPE

TYPE A JOINT



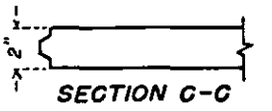
TYPICAL SIDE SLOPE OR BOTTOM SLAB

VOL.= 0.32 CU. FT. WT. 43 LBS.

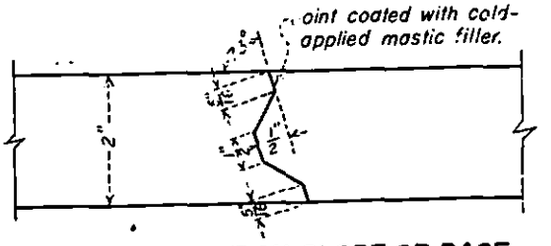


SECTION A-A

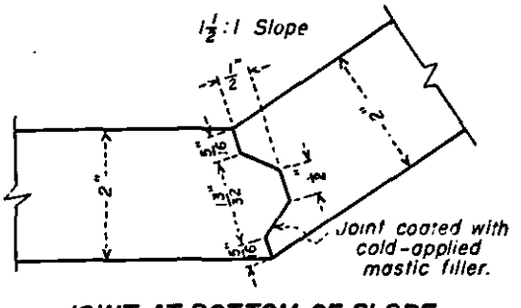
SECTION B-B



SECTION C-C



DETAIL OF JOINT ON SLOPE OR BASE



JOINT AT BOTTOM OF SLOPE

TYPE B JOINT

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
OFFICE OF THE CHIEF ENGINEER

CANAL LINING INVESTIGATION

PRECAST CONCRETE CANAL LINING SLABS

DRAWN	E.W.S.	SUBMITTED	
TRACED	H.E.W.	RECOMMENDED	
CHECKED		APPROVED	

8030-WHP-143

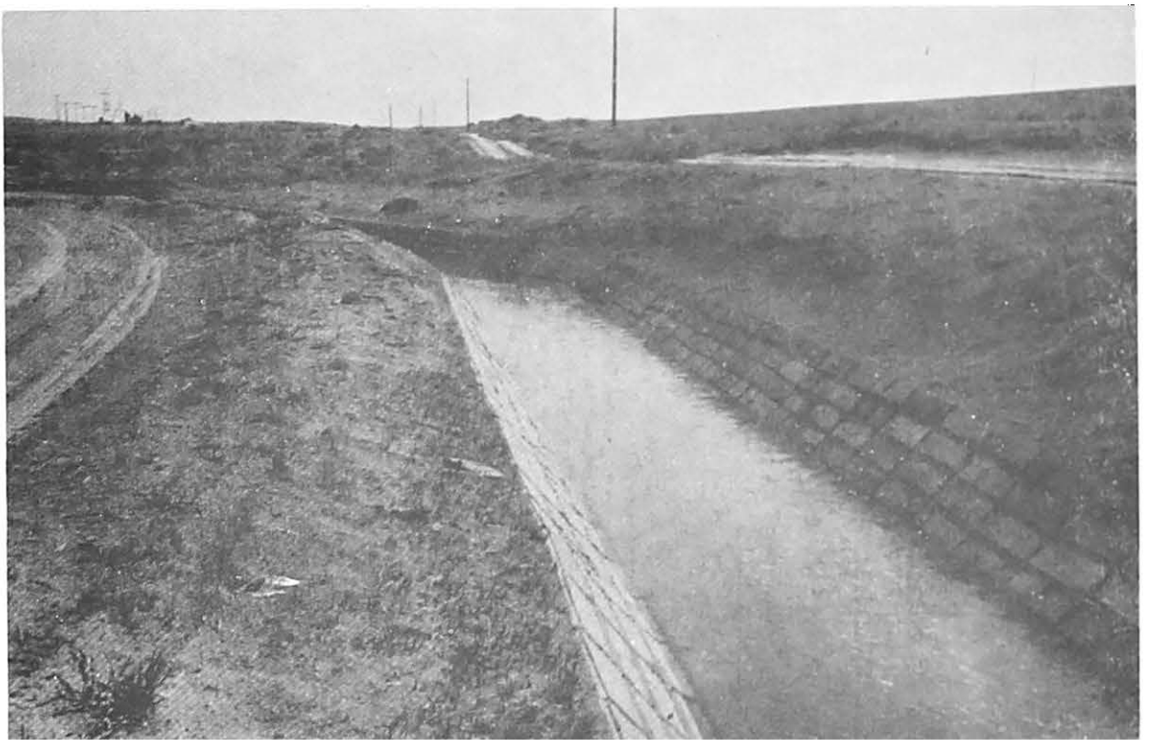
FIGURE 4

Photograph No. 12

Installing 2-foot square slabs of precast concrete lining--Yakima Project, Washington--1947.

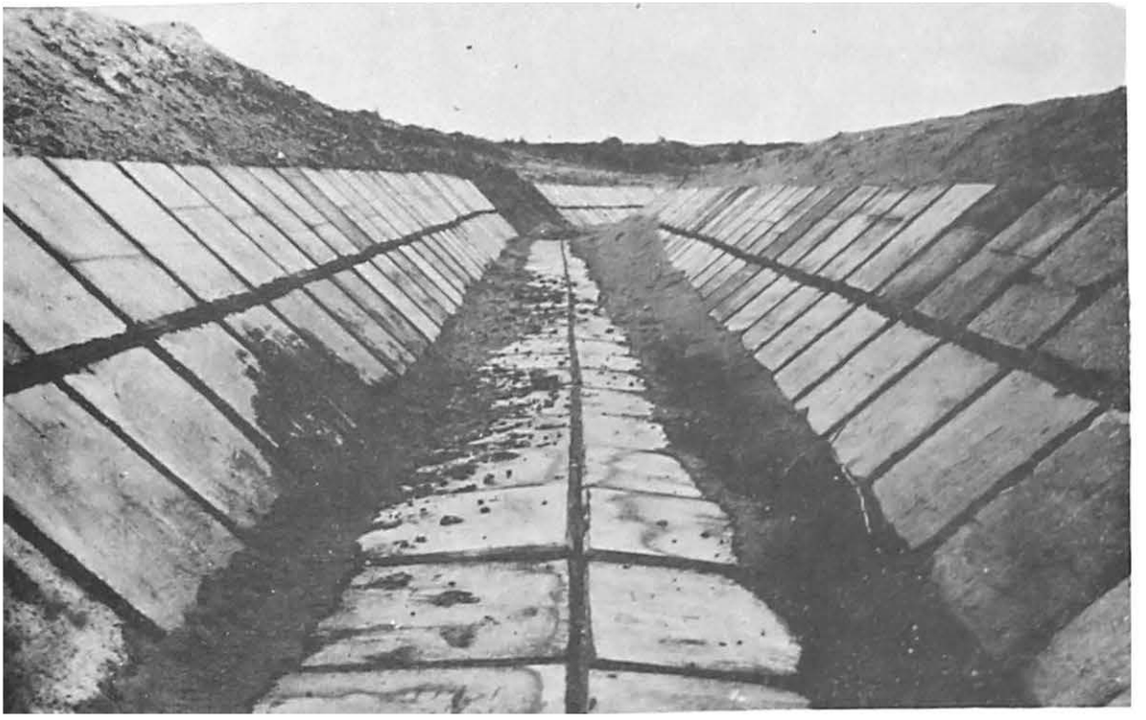
Photograph No. 13

Placing 8-inch by 24-inch precast concrete slabs with tongue and groove joints. Yakima Project, Washington--1947.



Photograph No. 14

Precast concrete lining installed in 1947--Yakima Project, Washington.



Photograph No. 15

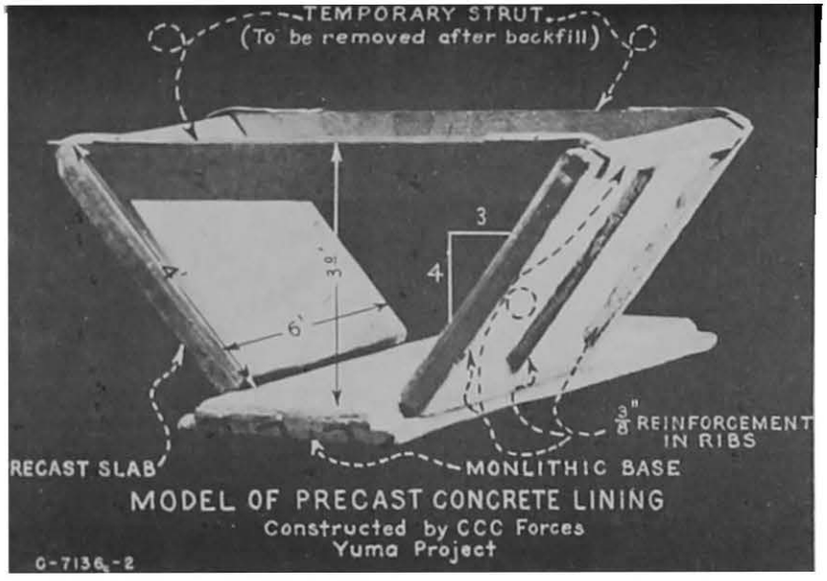
Heaving at the subgrade of precast concrete-lined canal due to frost action--Yakima Project, Washington.

Photograph No. 16

Partial failure of precast concrete lining due to surface water entering behind lining--Yakima Project, Washington.

Photograph No. 17

Full-scale laboratory model for testing flexibility of linings of various types of precast concrete blocks.



Photograph No. 18

Model showing lining details.

Photograph No. 19

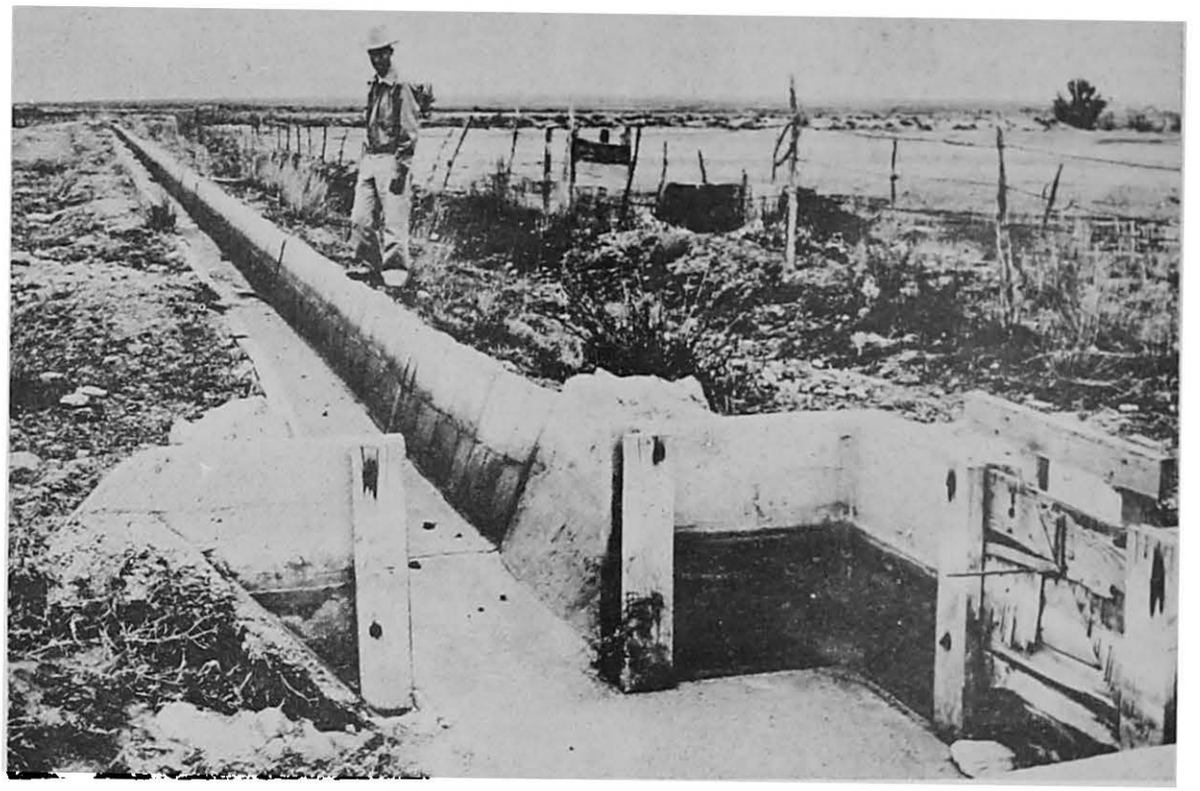
Cast-in-place base slab ready for installation of precast side slopes.

Photograph No. 20

Sliding slabs into place. Note struts for holding slab while backfill is placed and tamped.

Photograph No. 21

Section of lateral showing weed growth through unsealed joints.



Photograph No. 22

Cast-in-place base slab with 3- by 12- by 18-inch and 3- by 9- by 12-inch precast slabs on side slopes--Carlsbad Project, New Mexico--1941.

Photograph No. 23

Bottom course of 3- by 12- by 18-inch precast concrete blocks on a 4-inch cast-in-place base slab--Carlsbad Project, New Mexico--1947.

Photograph No. 24

Placing top course of precast slabs--Carlsbad Project, New Mexico--1947.

Pneumatically-Applied Portland Cement Mortar Linings

Pneumatically-applied portland cement mortar is defined as a surface coating of mortar which consists of an intimate mixture of portland cement, sand, and water shot into place by pneumatic pressure. The resulting coating, if properly proportioned, mixed, placed, and cured is described as being dense, hard, and very strong. This material is often referred to as "Gunite," sometimes incorrectly because "Gunite" is a trade name coined by one manufacturer to describe the sand-cement product of their particular equipment. The term "shotcrete," which has been adopted by both the American Railway Engineering and the Portland Cement Associations to designate pneumatically-applied portland cement mortar, is gaining common acceptance and will be used in this discussion.

Shotcrete has been used for canal linings for over 30 years; one of the early installations being made in Nevada in 1917. During the 1920's this method of construction gained in popularity and was used on a number of canals on both private irrigation and Bureau of Reclamation projects (see Table 2). Commencing in about 1928 extensive work was undertaken by irrigation projects in southern Arizona and the Lower Rio Grande Valley of Texas, using shotcrete linings (Photographs Nos. 25 and 26). In this work both laterals and main canals were lined.

The many installations indicate that a well-constructed shotcrete lining will furnish satisfactory service. In some irrigated areas a large percentage of the lined canals and laterals have been constructed with shotcrete. For instance, it is reported that over 2,200,000 square yards of shotcrete lining have been placed by irrigation districts in the Lower Rio Grande Valley since 1928.

Although the greater percentage of shotcrete linings has been placed on irrigation projects in the temperate climates of southern California, Arizona, and Texas, there are numerous satisfactory installations in locations subject to severe conditions of temperature and exposure. The Southern California Edison Company has recently made extensive use of shotcrete in the lining and repair of their power canals in the Sierra Nevada Mountains where very severe winter weather conditions are encountered.

The shotcrete process is well adapted to canal lining construction of any shape or thickness of lining desired. These features are particularly important in work involving the lining of numerous laterals and farm ditches where a variety of shapes and sizes of cross-section are encountered.

Canals to be lined with shotcrete should have the same quality of subgrade finish that would be provided for a cast-in-place concrete lining. The importance of a reasonably uniform subgrade was emphasized in reports of existing installations on the Roosevelt irrigation district in Arizona, where it was noted that a minimum of buckling and cracking was observed in sections of the canal where the subgrade alinement was reasonably uniform. It is usual practice to require that the subgrade be moist at the time the shotcrete is applied so that water will not be drawn from the freshly placed mortar.

As mentioned previously, the equipment for applying shotcrete will vary in detail with the manufacturer. A system of air locks is usually incorporated into the mechanism for feeding the premixed dry sand and cement into a large flexible hose through which it is then transported to the discharge nozzle by pneumatic pressure. At the discharge nozzle, water introduced through a second hose, is added to the sand-cement mix and the mortar is discharged from the nozzle under pressure. With all equipment, shotcrete is applied to the canal section by holding the nozzle about 3 feet from, and normal to, the surface being covered.

An important requirement for a successful shotcrete application is skilled and well-trained operating personnel. The rate of application and adjustment of the mix, essential to a satisfactory installation, are dependent upon careful operation. In many areas, contractors with shotcrete equipment specialize on this type of work and have developed very economical procedures. The construction procedures and equipment to be used will depend upon the amount of lining to be placed and the size of canal to be lined. On the larger jobs, several shotcrete units are sometimes employed and special mobile equipment is provided. On smaller jobs, the equipment is usually limited to one shotcrete unit such as shown in Photograph No. 27.

In general, the discussions in another portion of the text regarding the design of concrete lining are applicable to shotcrete linings. Questions regarding lining thickness, joint type and spacing, and use of reinforcement with shotcrete linings are best answered by a review of the service records of existing installations. The thickness of shotcrete linings will normally vary from a minimum of 1 inch to a maximum of 2 inches. Most of the Bureau of Reclamation shotcrete linings have been 1-1/2 inches thick, but a large percentage of the installations mentioned in Arizona and Texas are of 1-inch thickness.

Shotcrete linings constructed by the Bureau, as well as those constructed by private irrigation districts, have for the most part contained reinforcement. Usually mesh fabric has been used for this purpose. Typical of Bureau practice are the New Briar canal with 1-1/2-inch thickness of lining reinforced with 9-gage 6- by 6-inch mesh and the "A" canal of the Gila project with a 1-1/2-inch thickness of lining and 12-gage 6- by 6-inch reinforcement. As an average, the area of reinforcement has been about .001 of the concrete area. Experience has indicated this amount of reinforcement to be adequate.

Recently a number of the laterals on the Pasco lateral system of the Columbia Basin project were lined with 1-1/2- and 2-inch unreinforced shotcrete. Late in 1945 two sections of the East Turbine lateral, Yakima project were experimentally lined with shotcrete, part of which was reinforced and part unreinforced for comparison. The installation of this lining is shown in Photograph No. 28. These installations, which are subject to moderately severe weather, will be of interest in future studies to evaluate the benefits of reinforcement in shotcrete linings.

With shotcrete lining construction, as with concrete linings, the increased cost of reinforcement, which often increases the costs 20 percent over the cost of unreinforced lining, must be evaluated against the improved quality of the lining. No doubt many installations with unusual or special conditions justify this expense; however, the more typical installation offers some question as to justification of this additional cost.

The selection of type and spacing of joints for shotcrete linings is controversial. Most of the shotcrete linings installed by private irrigation districts have not been provided with either expansion or contraction joints. The argument favoring this procedure is that the cracks which will occur can be filled as a maintenance operation at less expense than the cost of preformed joints. The Bureau of Reclamation has followed the opposite practice and has consistently provided contraction joints, usually at 6-foot centers, in shotcrete lining. These joints are filled with asphalt mastic at the time of construction. In general, contraction joints at 6-foot centers in shotcrete lining have effectively confined cracking to the joints. This procedure largely eliminates the expense of later cleaning and filling the random cracking. Expansion joints are usually provided only where the lining joins canal structures.

An important consideration in shotcrete construction is the method of handling the rebound which results from a portion of the mortar bouncing away from the surface to which it is applied. Present Bureau of Reclamation practice is to require the rebound to be compacted and smoothed when troweling of the surface is required. In the event the surface is not to be troweled, the rebound is left in place. Specifications reviewed for shotcrete linings by private irrigation districts require that all rebound be removed and the surface of the lining also troweled. The value or importance of troweling the shotcrete linings is influenced by two factors, the method of curing proposed and the hydraulic properties of the canal. Experience has proven that the coverage with a sprayed curing membrane is considerably increased on a troweled surface as compared to the rough natural shotcrete finish. It has been estimated that the more efficient and economical use of curing compound offsets to a considerable extent the cost of troweling. The theoretical hydraulic advantage of a troweled surface is contingent upon the size and location of the canal. A small, lined farm ditch which would probably have a sand or silt deposit on the bottom, which would constitute a major portion of the wetted perimeter, would not justify the expense of a troweled finish for hydraulic benefits. In a large canal, the improvement in hydraulic properties may justify troweling. Insofar as can be determined, the troweling does not improve the quality or strength of the shotcrete lining.

As with all types of canal linings, there are examples of failure in shotcrete linings. In most instances, these can be attributed to the poor quality or workmanship of the original construction. Shotcrete linings due to their thinness are subject to failure or

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

PNEUMATICALLY APPLIED MORTAR LINED CANALS

COST DATA BASED ON CONTRACT BID PRICES AND CONTRACT COST REPORTS

SPECIFICATION		PROJECT AND LOCATION	CANAL SECTION				DATE OPENED	CANAL LINING							COST OF LINING PER SQUARE YARD						
											REINFORCEMENT		CONTR. JOINT SPACINGS		CONCRETE		REINFORCEMENT		SUB-TOTAL	TRIM. SUB-GRADE	TOTAL
No.	SCHED.		Q c. f. s.	b FEET	d FEET	SIDE SLOPE		TYPE	QUAN. SQ. YDS.	THICK-NESS	LONGIT.	TRANS	LONGITUDINAL	TRANS.	CONTRACT	GOV'T. MATERIALS	CONTRACT	GOV'T. MATERIALS			
955	2	Sta. 1+73.12 to 135+86 New Briar Canal Boulder Canyon Project	350 to 700	8 to 18	5.07 to 5.92	1½:1	4-2-41	Reinforced Shotcrete	56,700	1½"	9 ga.6"x6" mesh p=0.00194	9 ga.6"x6" mesh p=0.00194	None	10'-0"	0.670	0.242	0.056	0.118	1.086	0.16	1.246
1104	1	"B" Canal & "A" & "B" Laterals Gila Project	14 to 70	2 to 4	1.1 to 2.8	1½:1	9-20-45	Reinforced Shotcrete	889	1½"	12"ga.6"x6" mesh p=0.001	12"ga.6"x6" mesh p=0.001	None	5'-0" to 6'-0"	1.600	0.172	0.037	0.053	1.862	0.45	2.312
1230		Pasco Laterals & Sublaterals Columbia Basin Project	15	3	3	1½:1	5-31-46	Unreinforced Shotcrete	500	2"	None	None	None	6'-0" 9'-0" & 12'-0"	1.857	0.292	—	—	2.149	0.45	2.599
1230		Pasco Laterals & Sublaterals. Columbia Basin Projects	5 to 15	2 to 3	2 to 3	1½:1	5-31-46	Unreinforced Shotcrete	28,600	1½"	None	None	None	6'-0" 9'-0" & 12'-0"	1.607	0.219	—	—	1.826	0.45	2.276
1402	2	"A" Canal & "A" Lateral Yuma Mesa Division Gila Project	15 to 60	2.5 to 3.2	1.5 to 4.6	1½:1	7-23-46	Reinforced Shotcrete	34,700	1½"	12"ga.8"x8" mesh p=0.00075	12"ga.8"x8" mesh p=0.00075	As directed	6'-0"	1.300	0.203	0.068	0.044	1.615	0.30	1.915
1402	2	"A" Canal & "A" Lateral Yuma Mesa Division Gila Project	15 to 60	2.5 to 3.2	1.5 to 4.6	1½:1	7-23-46	Unreinforced Shotcrete	34,000	1½"	None	None	As directed	6'-0"	1.300	0.203	—	—	1.503	0.30	1.803
1546		Laterals from "A" & "B" Canals Yuma Project	15 to 60	1.5 to 7.0	1.08 to 3.93	1½:1	12-30-46	Reinforced Shotcrete	56,750	1½"	12"ga.6"x6" mesh p=0.001	12"ga.6"x6" mesh p=0.001	None	10'-0"	1.644	0.023	0.082	0.053	1.802	0.43	2.232
1546		Laterals from "A" & "B" Canals Yuma Project	15 to 60	1.5 to 7.0	1.08 to 3.93	1½:1	12-30-46	Unreinforced Shotcrete	56,750	1½"	None	None	None	6'-0"	1.644	0.023	-	—	1.667	0.43	2.097

TABLE 2

damage from frost heaving or hydrostatic pressure behind the lining even more readily than a thicker concrete lining. These problems, however, are common to all rigid types of lining and are not necessarily a weakness of shotcrete construction.

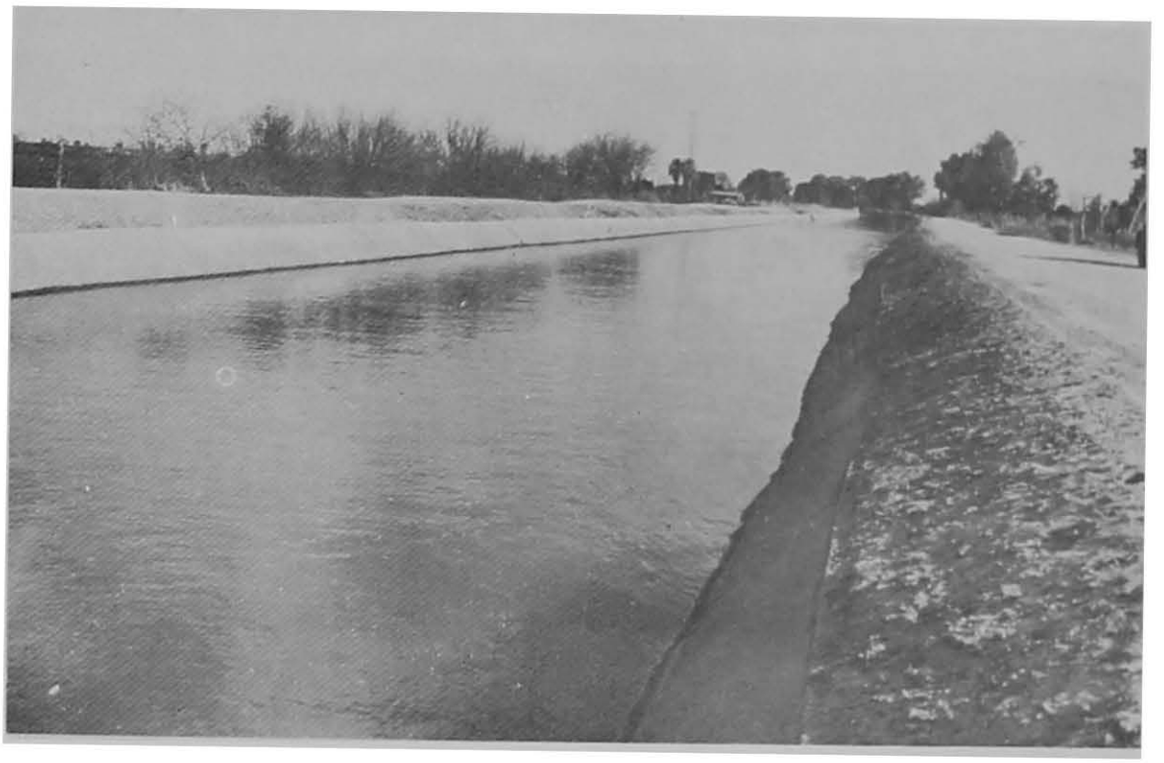
The costs of shotcrete lining construction are dependent principally upon the thickness of lining, use of reinforcement, provision for joints, size of canal, and availability of materials. Recent bid prices for shotcrete lining in new canals on Bureau of Reclamation work, where a 1-1/2-inch thickness of lining with reinforcement and contraction joints was specified, indicate an average price of about $2.30 to $2.50 per square yard including trimming and all materials. Private irrigation districts report a cost of about $1.30 to $2 per square yard for a 1-inch reinforced shotcrete lining without contraction joints. A recent report of a 1-inch reinforced shotcrete lining installed by the Maricopa County municipal water conservancy district near Phoenix, Arizona, indicates a force account cost of less than $1 per square yard. This cost does not reflect the usual items of profit, insurance, or depreciation which are properly included in contract prices. However, if a reasonable allowance is added for these items, the adjusted cost is considerably below present-day averages.

Service records of existing shotcrete lining installations indicate a maintenance and upkeep cost comparable to cast-in-place concrete linings. Many installations with approximately 20 years service are reported to be giving excellent service with a minimum of expense for repair or upkeep.

In addition to being used for the original construction of canal linings, shotcrete has been advantageously adopted for the repair of existing concrete linings. Concrete linings which are badly cracked or have disintegrated and are permitting heavy losses of water from the canal have been effectively re-faced with a shotcrete coating of 1/2 inch to 1-1/2 inches in thickness. This work has been carried out following usual construction methods except that the existing concrete lining has been carefully cleaned to insure adequate bond between the shotcrete and old concrete surface. The Southern California Edison Company has repaired approximately 75,000 square yards of old concrete lining with 1/2- to 1-inch thick reinforced shotcrete lining in the repair of existing concrete canals. Previous similar repair work by this company has proven this procedure to be very satisfactory.

Obviously, the application of a relatively thin shotcrete coating will be of little value in providing permanent repair, if the original lining failure resulted from subgrade heaving or hydrostatic pressure. Unless the cause of these failures can be eliminated, the shotcrete lining will probably fail in a similar manner.

In an attempt to reduce the cost of regular shotcrete linings, approximately 1,300 lineal feet of shotcrete lining were placed on the Gila project late in 1946, utilizing the naturally sandy soils along the canal. This test lining was placed in three test sections using 12, 16 and 20 percent cement content by weight and standard shotcrete equipment. Considerable difficulty was encountered in placing the lining due to the material emerging from the nozzle in slugs or lumps. This necessitated continual adjustment in the water and resulted in a lining of nonuniform mix and of varying thickness. It was also found that proper finishing of the lining was difficult to accomplish as the material was quite sticky and tended to either adhere to the trowel and peel off or sag on the slopes. The installation of these test sections indicated that shotcrete utilizing naturally sandy soils is possible with regular shotcrete equipment, but that production is considerably reduced and resultant savings are questionable. A recent inspection of these test sections revealed that after a little over a year of service these linings, though still serviceable, were excessively cracked.



Photograph No. 25

Reinforced shotcrete lining, 1-1/2 to 2 inches thick, in a generally good condition after about 20 years of service. Consolidated Canal--Eastern Branch, Salt River Valley Water Users' Association, Arizona.

Photograph No. 26

One-inch-thick shotcrete lining constructed in 1947--Lower Rio Grande Valley, Texas.

Photograph No. 28

Placing, finishing, and curing 1-1/2-inch shotcrete lining in 1945--Pasco Pump Laterals--Columbia Basin Project, Washington.

ASPHALTIC CANAL LININGS

Asphalt is a bituminous material distinguishable from tar by some of its physical properties and derived from an entirely different source. Asphalt is obtained primarily from the refining of crude oil while tar is a product usually obtained from the destructive distillation of coal. Before the advent of modern petroleum refining, asphalt for construction purposes in the United States and Europe was secured from natural lake deposits mainly in the British West Indies. Asphalt obtained from the refining of petroleum is more widely used today than from natural deposits largely because of its greater abundance and lower cost. Gilsonite and Grahamite are two natural asphaltic materials occurring in narrow veins in New Mexico and Utah. These are both very hard and brittle but are readily soluble in petroleum naphtha and are used in the manufacture of enamels and similar products. They are comparatively high in cost and therefore not practical for use in paving construction. Rock asphalt which occurs in some of the western states is another form of natural asphalt. This material, formed by the natural seepage of asphalt-bearing crude oil through permeable sandstone and limestone, when crushed has good characteristics for road and street surfacing. Rock asphalt has not been found suitable for use in lining canals due principally to its lack of cohesiveness which renders it very susceptible to erosion.

Asphalt materials are available commercially in many forms ranging from fluid to solid consistencies. Asphalt in the fluid and semifluid states may be either cutbacks or emulsions. Asphalt cutbacks vary in curing characteristics and consistency depending upon the type and amount of fluxing or cutting solvent used. Rapid curing (RC) asphalt is fluxed or cut back with naphtha-type solvents; medium curing (MC) is cut back with kerosene-type solvents; and slow curing (SC) with gas oil types. Six grades of viscosity or fluidity are available in each of the three curing groups. These are designated by numbers from zero to five indicating consistencies from thin to thick. Thus an asphalt cutback (MC-0) would be more fluid than MC-2, but would have a similar curing characteristic. Asphalt emulsions are suspensions of asphalt in water and are designated by Types RS, MS, and SS indicating rapid, medium, and slow set or "break" as it is usually termed. Asphalts of solid and semi-solid consistencies comprise the asphalt cement group which must be mixed and placed hot. Asphalt cements normally vary in consistency from 300 penetration (soft), at testing temperatures, to 30 penetration (hard) and become liquid at temperatures between 250 and 350 F. Desirable combinations of characteristics are developed in cements by special treatment as in the case of air-blown and catalytically blown asphalts. These processes develop unusually high softening points for given penetrations which minimize sagging and running of asphalt at high temperatures although pliable and ductile at low temperatures. Asphalts may also be stiffened and given better weathering properties by adding, or filling with, finely divided materials such as diatomaceous earth, rock dust, portland cement, and silt. The principal advantage of flexible asphaltic materials for use in lining irrigation canals and laterals, is their ability to conform to a reasonable amount of settlement or bulging of the subgrade without rupture of the lining.

The wide range of characteristics and types of asphaltic materials available makes possible a wide variety of types of lining and treatments differing considerably in cost, effectiveness, permanence, and adaptability. Below are listed eight principal types together with the asphaltic materials commonly used in their construction:

1. Hot mix (asphalt cement)
2. Cold mix (cutbacks and emulsions)
3. Prime-membrane (cutbacks and cement)
4. Buried membrane (asphalt cement)
5. Prefabricated surfacing (fabrics and cement)
6. Pneumatically-applied asphalt (emulsions and cutbacks)
7. Embankment injection (oils, emulsions, and cutbacks)
8. Cutoff walls (oils, emulsions, cutbacks, and cement)

The table below lists the location, extent, and date of construction covering the Bureau's experience with asphaltic linings to date. More detailed information on these installations is given along with the discussions of each of the eight types in the following pages.

Project	Feature	Type lining	Length feet	Area sq.yds.	Date Constructed
Central V.	Contra Costa	Hot-mix	528	2,000	April 1939
Yakima	Snipes Mountain	Hot-mix	550	2,200	Nov. 1939
Boise	"D" Line	Hot-mix	270	1,073	April 1943
Boise	"D" Line	Hot-mix	330	920	May 1943
Owhyee	Malheur	Hot-mix	2,600	11,000	Nov. 1944
Col.Basin	Pasco Laterals	Hot-mix	58,000	88,200	May-Sept. 1947
Boise	"D" Line	Cold-mix	20	78	April 1943
Gila	Yuma Mesa	Cold-mix	--	--	April 1946
Boise	"H" Line	Prime-membrane	1,000	1,444	April 1942
Yakima	Roza Laterals	Prime-membrane	4,695	7,575	May-Sept. 1946
Gila	Yuma Mesa	Prime-membrane	2,000	3,500	May-Sept. 1947
All-Amer.	East Mesa	Prime-membrane	--	--	May-Sept. 1947
Col.Basin	Pasco Laterals	Prime-membrane	928	1,850	Sept. 1947
Boise	"D" Line	Prefabricated	30	121	April 1943
Boise	"D" Line	Prefabricated	350	1,800	January 1946
Yakima	Roza Lateral	Prefabricated	30	45	September 1946
Klamath	Coppeck Bay	Buried membrane	408	1,000	September 1947

Hot Mix

Hot-mix asphalt consists of a combination of straight asphalt cement and aggregate which must be mixed, placed, and compacted hot. The gradation and maximum size of the aggregate is a design problem which may be influenced by the characteristics desired and the type of material available. When the mix contains aggregate retained on a standard No. 4 screen, it is termed asphaltic concrete. Mixtures of asphalt and sand are referred to as sheet asphalt. Upon cooling to air temperatures, hot-mix linings develop a high degree of cohesion and stability, resulting in a tough, durable, yet flexible lining. Hot-mix linings are generally considered to be the highest type among the asphaltic group with respect to durability, resistance to external forces, and for hydraulic properties. When placed and compacted by hand labor, this type of lining is relatively high in cost. However, it is especially adaptable to large-scale mechanization, with good indication that low costs can be achieved by the use of properly developed equipment.

Two experimental sections of asphaltic concrete lining were placed in the Contra Costa canal of the Central Valley project in May 1939. The two sections were 220 feet and 308 feet in length, respectively, involving about 2,000 square yards of lining 3 inches thick. The first section was constructed by hand-placing methods near the outlet of the pumping plant at Oakley, California, while the second section was placed with a concrete lining machine near Antioch. The machine-placed section, which was reinforced with 12-1/2-gage woven wire, indicated the feasibility of placing asphaltic concrete by the slip-form method. However, portions of this section developed a number of cracks and bulges during the first summer which appeared to have been caused by expansion of the wire mesh reinforcement. The bulges and cracking were found to be confined to the upper portion of the lining lying above the wire mesh. These were repaired and no further damage, except some minor checking on the berm, has been observed in the machine-placed section. Mud curling at the waterline of the hand-placed section has resulted in some erosion during the 9 years of service. The use of harder asphalts such as are currently recommended for lining construction might have prevented erosion. Creosote, kerosene extract, and sodium chlorate were used as soil sterilants. No appreciable weed growth has occurred.

Asphaltic concrete linings were placed in the Snipes Mountain canal, Yakima project, in November 1939. One area was located within the city limits of Sunnyside, Washington, while the second area was approximately 3 miles south of that city. A total of approximately 2,200 square yards of 2- and 3-inch-thick lining was placed in the two areas.

All material was mixed in a central plant, hauled to the project in flat-bed trucks, and placed by CCC hand labor. (See Photograph No. 29.) Compaction was obtained by a winch-drawn roller attached to a small tractor. All areas of this lining are in generally good condition after seven irrigation seasons with the exceptions of a sodium chlorate treated section of canal lining in the city of Sunnyside which began to show excessive weed growth through the lining at the waterline early in 1946. The use of sodium chlorate in other areas of this installation have proven satisfactory but failed in this area because of the leaching out of the chlorate by irrigation water applied to lawns above the canal.

Two areas in the D-line canal, Payette division of the Boise project were lined with asphalt mixes in April and May 1943. The lining at Milepost 12.5 consisted of approximately 1,070 square yards of lining 1 and 2 inches thick in a 12-foot bottom canal. The hot-mix at this point consisted of a fine sand and an asphalt cement slightly harder than that used in the Contra Costa and Snipes Mountain linings. This amounted to a sheet asphalt mixture and was placed by hand methods. The soil was sterilized with sodium chlorate and no weed growth has occurred. A portion of this lining was subjected to an inflow of storm water over the upper edge which eventually undermined a small portion of the lining and caused a collapse in that area. The failure was repaired by Operations & Maintenance forces early in 1947 and no further damage has occurred. The second section constructed at Milepost 23.8 was lined with 1-, 2-, and 3-inch thicknesses of asphaltic concrete. This material was also placed by hand methods. Sodium chlorate was used as a soil sterilant. The only weed growth observed is a few willow shoots which have appeared at the extreme downstream end of the 1-inch-thick lining. Some bulging of the lining has occurred in the invert in one area due to frost action, but the lining has not ruptured. No difference in appearance is yet noticeable between the sections of different thickness.

Approximately 1,100 square yards of 2-inch-thick asphaltic concrete was placed in the Malheur canal, Owyhee project, in November 1944. (See Photograph No. 30.) An 85-100 penetration asphalt cement was used which was harder than that used in any of the three linings previously described. The aggregate consisted of sand and crushed rock which produced a rough-finish surface requiring a heavy seal of filled asphalt cement. A pipe-enamel seal was used which has since checked badly due to weathering. One area near a turnout gate in a lateral adjacent to the main canal has eroded severely in the bottom due to high velocity and turbulence of the water at this point. Sodium chlorate was used as a soil sterilant and weed growth has been limited to a few willow shoots. This lining was placed to stop severe ground movement in the foundations of the Malheur siphon caused by saturation of the earth by seepage water from the canal. It is reported to have successfully accomplished this purpose.

Almost 90,000 square yards of 2-inch-thick asphaltic concrete lining was placed in approximately 11 miles of the Pasco laterals of the Columbia Basin project, Washington, in the summer of 1947. Only about 25,000 square yards of soil in the lining area was sterilized. Part of this area was treated with sodium chlorate and part with boric acid. A 60-70 penetration asphalt cement and a sand-gravel aggregate was used in the machine-placed hot-mix. This project offered the first opportunity for employing full-scale mechanized equipment for placing and compacting purposes and a practical determination of costs. The contractor on this job used a slip-form type asphaltic lining machine developed for the job. (See Photograph No. 31.) This equipment which is relatively simple in design, and available at a nominal capital investment, satisfactorily placed the hot-mix lining at a rapid rate and at about 90 percent of laboratory compacted density. As disclosed by construction data accumulated on the job, the cost of the 2-inch asphalt hot-mix lining, without soil sterilization, was 79 percent of the cost of the 2-inch portland cement concrete lining and was 87 percent of the cost of 1-1/2-inch shotcrete lining. Percentages apply to the finished lining and include all materials, the final trimming of the earth section, and reasonable charges for equipment rental.

A contract was let late in 1947 and construction is scheduled for the summer of 1948 on 5 miles of the Ygnacio canal to be lined with 28,700 square yards of 2-inch asphaltic concrete. This lining will be constructed with a sand-gravel mix containing 50-60 penetration asphalt cement and compacted to 90 percent of laboratory density. The lining will be placed by machinery developed for, or by, the contractor, and several types of soil sterilants will be used for experimentation. This project is expected to yield further data on the actual cost of full-scale construction on this type of lining and may contribute further towards development of new canal-lining equipment.

Cold Mix

Cold-mix linings are similar to the hot-mix linings in that carefully graded aggregates are used, and are mixed with bituminous material. However, cold mixes utilize one of the liquid asphalts (cutback, emulsion, or powdered asphalt-flux combination). These materials are mixed, laid, and compacted relatively cold, usually at air temperatures between 65 and 100 F. Cold mixes require less complex mixing equipment than the hot mixes and are generally more convenient to use, especially where the quantities of material for a given project or repair job are not large. A considerable disadvantage to this type of lining, with a degree of exception to the emulsified asphaltic materials, is the lack of immediate attainment of high degrees of cohesion in the mixes. Since lining mixes are placed on rather steep side slopes, an appreciable resistance to creep must be developed within a short time after placing, or plastic flow will ensue followed by cracking or slipping of side-slope linings. The lining must be resistant to water erosion, which also requires development of appreciable cohesion. Cold mixes are, therefore, limited in their uses except as they will meet the demands of service. Of the asphaltic materials used for cold mixes, asphalt emulsions develop greatest cohesion in a given time, while MC and RC cutbacks will develop appreciable stability under certain conditions of curing or aerating before placing and compacting. Several reservoir linings have been successfully constructed by emulsion cold-mix methods by agencies other than the Bureau. Two trial canal lining installations have been made by the Bureau; one on the "D" Line canal, Payette division, Boise project, and one on the Gila project near Yuma, Arizona. No cold-mix project has as yet been of a magnitude to permit accurate ascertainment of large-scale construction costs. This cost may be expected to be somewhat lower than hot-mix but, with some possible exception, the lower cost is offset by the disadvantages of cold-mix linings.

An asphalt emulsion, cold-mix lining 2 inches in thickness was constructed at Milepost 12.5 on the "D" Line canal, Payette division, Boise project in April 1943 (concurrently with hot-mix linings at this location which have been previously discussed). The test section was 20 feet in length and 78 square yards in area. This lining employed a local deposit of very fine sand and aggregate. When placed and compacted, this material developed large shrinkage cracks due to the evaporation of the water in the asphalt emulsion. (See Photograph No. 32.) The cracks are still existent after five seasons of irrigation use, and they will probably have an important effect on the ultimate life of the lining. Cracking would have been less severe if a well-graded aggregate had been used. In other cases a slurry of fine sand and emulsion has been broomed into such cracks, filling them and apparently effectively minimizing any effect of such cracks. This emulsion lining was not so treated and has not been considered entirely satisfactory due to the severely cracked condition. The actual mix has retained excellent life however, indicating that this type of lining might be successful if proper design and construction methods were used.

An asphalt emulsion lining utilizing the existing fine sand on the Yuma Mesa of the Gila project was constructed in 1946. It is understood that this construction is performing satisfactorily.

Prime-Membrane

Prime-membrane-type asphaltic lining provides a simple low-cost method of canal lining in which all materials are applied by consecutive spray applications. (See Photograph No. 33.) The construction sequence usually starts with a prepriming operation in which light oil such as diesel fuel is sprayed on the earth or subgrade to act as a soil sterilant and to expedite the absorption of the prime application. The prime application consists of SC, MC, or RC cutback asphalts of low viscosity, applied as a spray and absorbed by the earth. The prime acts as a stabilizing material for the soil forming a relatively waterproof, stable layer to which the membrane, or surfaced course is bonded. The membrane may be either a blown asphalt cement having a softening point above 150 F, or an asphalt cement to which diatomaceous earth has been added to prevent flow of the membrane at temperatures near 140 F. (See Photograph No. 34.) The preprime generally requires about 0.5 gallon per square yard of diesel fuel or similar oil, the prime consists of from 0.5 gallon per square yard to, in some cases, 2.0 gallons per square yard, while the membrane usually requires approximately 0.75 to 1.0 gallon a square yard. The cost of small experimental projects have been in the neighborhood of $0.85 to $1 a square yard, but costs of about $0.50 are believed possible in large-scale installations. In designating canals for prime-membrane lining, a careful study

of soil conditions is necessary. Soils should have sufficient capillarity so that the prime materials will be properly absorbed, and the material should be stable after absorption of the prime. Soils which have a low capillarity usually do not prime well. Such soils may be either very tight clays or soils high in clay, or some types of fine sand which are neither highly permeable nor have high degrees of capillarity. Some desert sands of the latter description have been found to be very difficult to adequately stabilize by priming due to poor absorption of the prime material. It is essential, therefore, that soils proposed for prime-membrane lining be thoroughly tested for their suitability. Prime-membrane linings are readily damaged by canal cleaning operations and, except where unusually stable subgrade conditions exist, by the hoofs of heavy animals.

The first prime-membrane-type lining constructed by the Bureau was placed in the "H" Line canal, Payette division, Boise project, in April 1942. Eleven test sections, totaling 1,444 square yards in a 4-foot bottom canal, were constructed. The test section included numerous variations of soil sterilant, priming material, quantities, and type of membrane used. At this date, after six irrigation seasons, one section is in very good condition (see Photograph No. 35) and four others are either in fairly good condition or are still integral and functioning. The best section is one which was primed with an RC-0 (low viscosity) to a depth of approximately 2 inches and then surfaced with an 85-100 penetration asphalt cement containing 19 percent of diatomaceous earth. The other sections were primed with either RC-1 or RC-2, with membranes of either asphalt cement and diatomaceous earth or RC-2 and diatomaceous earth.

In the early spring of 1946, test areas of prime-membrane lining were constructed in lateral 85.6-6 of the Yakima project north of Prosser, Washington. The purpose of these installations was to determine the optimum means of obtaining deep primes in a silty soil, in preparation for a series of larger test installations. In these experiments, it was found that a light preprime of kerosene or diesel fuel would permit much deeper primes than could be obtained in either dry or water-damp soil. Most of the trial installations in this lateral were destroyed by later construction operations on the lateral, but one section primed with RC-1 and with an RC-2 membrane over which a light covering of stone chips was placed, was still in good condition in July 1947. The data obtained from these tests were used to good advantage in the installations which followed at a later date.

Eleven test sections of about 50 square yards each were constructed in the East Turbine lateral, Yakima project, in May and June 1946 to determine the depths of penetration obtainable with different asphaltic materials and to determine the stabilization effected in a loose blow-sand soil by these methods. By prepriming with diesel fuel, up to a 4-inch depth of prime was obtained with about 3 gallons per square yard of cutback asphalt. A very good degree of stability was also obtained. In August 1946, several different types of membrane, including diatomaceous earth-filled asphalt cement, were placed over the previously primed sections. In November 1947 all of these experimental sections were in good condition, although no water had been in the canals. Final conclusions relative to the effectiveness of the East Turbine lateral linings will await completion of the system and admittance of water to the canal.

Several hundred square yards of lateral 69.1 of the Roza division, Yakima project, were primed with diesel oil and SC-1 in August 1946. A membrane consisting of 85-100 penetration asphalt cement and 6 percent diatomaceous earth was then applied. This lining was in fairly good condition in November 1947, although no water had been admitted to the lateral. As with the East Turbine lateral, final conclusions on the lining must await actual use of the lateral.

A prime-membrane-type lining 928 feet in length was constructed in the main pump lateral, Pasco, Washington, in the summer of 1947. A firm solid subgrade was primed with diesel oil and, in one short section, with a chlorate-boric acid water solution. A prime of 0.5 G.S.Y. of RC-0 was used, and 900 feet of the canal given an asphaltic membrane coating of a 60-70 penetration asphalt cement filled with 20 percent of diatomaceous earth. On the remaining 28 feet of length, several special catalytically-blown asphalts were used. In this prime-membrane lining, a relatively shallow depth of prime was used, as compared to the deeper primes used on previous experimental projects. The reduced prime thickness was used in consideration of the stable subgrade which existed in this area, and to secure a reduction of labor and materials cost for this type

of lining. This lining will be observed for determination of the serviceability of the lining under the existing subgrade and prime conditions, and for behavior of the special asphaltic membranes.

Buried Membrane

Buried asphaltic membrane linings are constructed by overexcavating the canal section and applying an asphaltic membrane by spraying a heated asphaltic cement to a thickness of from 0.1 to 0.2 inch after which an earth, sand, gravel, or rock covering, 1 foot or more in thickness, is placed. (See Photographs Nos. 36 & 37.) The purpose of such a lining is essentially to stop leakage through permeable soils. Since the asphaltic membrane is protected from weathering and mechanical injury, such a lining is expected to have an exceptionally long life. The buried membrane would not be disturbed by normal maintenance operations such as weed removal. An earth cover, however, unless protected by a gravel blanket, would be subject to erosion as is a loose earth lining. No factual cost data is available, but it is estimated that on large scale installations costs may not exceed $0.50 per square yard. No soil sterilants would be required for buried membrane linings. The probable low cost, ease of construction, indicated efficiency, and expected long life of this type of lining give it a high rating among methods for seepage control.

The first trial installation of buried asphalt membrane lining was made in a 408-foot length of the M-2-a-2 lateral of Klamath project, Oregon, in September 1947. This lateral had a 5-foot bottom width, 3.2 feet normal water depth, and a lining perimeter of approximately 22.5 feet. The soil in the canal was largely pumice, light in weight, and subject to severe cracking. It was feared that the loss of water through such cracks might be large and the buried membrane lining was placed for this reason. The trapezoidal canal section was overexcavated by 1 foot on the bottom and sides. The exposed surface was then wetted and lightly rolled to crush large soil lumps. Three types of asphalt were applied, two of which were blown asphalt cements, and one an asphalt emulsion. The asphalt cements produced very satisfactory membranes sufficiently tough to withstand foot traffic during the morning hours, while the emulsion membrane was considered unsatisfactory because of its failure to form a sufficiently thick, impermeable membrane. Portions of the asphalt cement membrane were given a light tack coat of RC cutback before placing the soil cover. The soil cover was expertly placed by a dragline, so that no further manipulation was required. Portions of the cover were wetted down by a hose, some were wetted and rolled, and others were left dry. Tests to determine seepage losses through this lining will be made by project forces.

Prefabricated Surfaces

The plastic and flexible properties of asphalt, both in mixtures with dust or fine mineral aggregate and in membranes of high softening point asphalt cement, has led to a study of prefabricated surfaces reinforced by vegetable or mineral fabrics or felts, for canal lining purposes. Asphalt saturated felts have long been used as roofing materials, and the remarkable serviceability of asphalt-impregnated prefabricated bituminous surfacing ($P^{B}S$), used for advance air fields during the war, furnishes background for use of prefabricated linings for canals and laterals. For practical use, the cost of such material must be low, or it must have other advantages of use which will overshadow a possible high initial cost. The use of very low-cost prefabricated linings is believed feasible for use in small laterals and ditches where rolls of the lining, similar to P.B.S. or prepared roofing, could be laid in place either transversely or longitudinally at a very low cost, or with inexpensive unskilled labor. This material may have a serviceable life of from one or two seasons to perhaps four or five seasons, depending on the quality of the material and the corresponding cost. Another type, suitable for larger canals, would be blocks or large sheets of prefabricated lining which would be higher in cost both in materials and in placing, but which would be long lasting, erosion resistant and sufficiently strong to stand up under livestock passage. Three experimental field installations of the latter type have been made.

McCorkle type prefabricated asphaltic block linings (see Photograph No. 38) were constructed at Milepost 12.5 and Milepost 6.7, "D" Line canal, Boise project, Idaho, in April 1943 and January 1946. The installation at Milepost 12.5 consisted of a 30-foot length of canal containing approximately 121 square yards of lining. The blocks used were 1-1/8 inches in thickness, and sized 34 by 60 inches. An asphalt saturated cotton canvas was used as a reinforcing backing to a 1 inch thickness of a fine sand-asphalt cement hot-mix. The fabric extended out 6 inches on two adjacent sides of the block to permit a degree of bonding between blocks. The soil was treated with sodium chlorate sterilant and the blocks placed by hand. The joints were sealed with a hot-asphaltic compound. The installation at Milepost 6.7 was made with a similar type of block, except that the blocks were reduced in size to 34 inches by 42 inches to permit easier fabrication, handling and placing. In November 1947 both of these linings were still in good condition.

An experimental prefabricated asphaltic lining made by the Shell Development Company, Emeryville, California, was placed in a 30-foot length of 2-foot bottom lateral on the Roza division in September 1946. (See Photograph No. 39.) This prefabricated lining utilized sheet asbestos (similar to furnace paper) as a reinforcing medium, which supported a 1/2-inch thickness of fine sand-asphalt hot-mix material and was made into sheets about 10 feet long and 30 inches wide. The sheets were sufficiently flexible so that they could be wrapped around a barrel at a temperature of 75 F. The sheets were placed transversely in the canal, the edges being bent over to form a berm. The joints were sealed with an asphaltic compound. In November 1947 this lining was in fairly good condition except for some surface cracking that had developed in the bend on the berms. This type of lining has an advantage of fewer joints than the McCorkle type, thus reducing the probability of leakage. The large sheets are somewhat more difficult to handle, however. Practical construction would require fabrication on the project. The high cost of this type of lining as compared to cast-in-place hot-mix linings reduces its practicability except for small installations and special purpose linings.

Pneumatically Applied

Pneumatically applied asphaltic linings may be constructed in the same general manner as portland cement shotcrete linings. (See Photograph No. 40.) In the asphaltic type of lining, sand or fine aggregate is proportioned into the air stream of the aggregate hose utilizing commercial equipment designed for use with shotcrete. At the nozzle, the asphaltic material is sprayed into the stream of aggregate, resulting in a mixing of the asphalt and aggregate as it is ejected from the nozzle. The mixed material is sprayed on the surface to be covered in the same manner as portland cement shotcrete. An asphalt emulsion of medium-setting type has been the most satisfactory material used for this purpose. Rapid curing asphalt cutback has been used but does not furnish as stable a surface as the emulsion. The rebound from the asphalt-type lining is generally not as great as that from the cement type. This type of lining tends to be somewhat more permeable than conventially placed linings. While several reservoir and canal linings of this type have been constructed by other organizations, the Bureau of Reclamation has used it in only one experimental section. This type of lining is indicated to be more expensive to construct than the hot-mix machine-placed linings, but where extremely irregular sections or other factors prevent the use of a hot-mix, the pneumatically applied lining may be economically feasible.

Several sections of old cracked concrete lining were covered with a pneumatically-applied asphaltic mix in March 1947 in the Ridenbaugh canal near Boise, Idaho. This cover lining was placed to a 1/2-inch thickness, using a fine sand and two grades of asphalt emulsion. A medium-setting emulsion, with a small amount of slaked lime added to the sand to facilitate breaking of the emulsion, gave the best results. A quick-setting emulsion tended to break in the mixing nozzle and could not be satisfactorily applied. The lining adhered well to the concrete surface and depressions in the old surface were also well leveled. This lining was in good condition when inspected after one irrigation season in November 1947 but has not been in service long enough to permit complete evaluation.

Injection and Cutoff Walls

Asphalt or oil injection of canal banks has been tried on a limited basis for reducing seepage losses through permeable sands. While this method is not, strictly speaking, a lining, it is nevertheless a component of the Lower-Cost Canal Lining Program. In the oil injection method, pipes are either jetted into the permeable soil or sand or placed in augered holes which penetrate the leaking strata. A low-viscosity oil, either a diesel fuel, fuel oil, or a prepared asphaltic cutback, is then pumped under pressure into the permeable strata. As each elevation becomes saturated, the pipe is slowly withdrawn. In quite permeable strata, plugs up to 10 feet in diameter are created with smaller plugs formed in less permeable materials. A method developed by the Shell Development Company, Emeryville, California, employs an asphalt emulsion especially produced for the type of sand in which it is to be used. It is understood that the Shellperm method, as the process is called, has been successfully used in several installations. The method of injecting oil into water permeable strata holds promise of being an economical method of controlling seepage of water through canal embankments. While oil injection has been used with apparent success by several organizations, only one test installation has been made by the Bureau. Further test installations are planned for construction on the Klamath project in 1948.

A test installation of the oil-injection method of controlling leakage through a sand embankment was made on the Yuma project, Arizona, in September 1946. In this installation, 1,800 linear feet of canal bank were treated by a heavy fuel oil of 24 A.P.I. gravity, injected into the water-permeable embankment sand by jetting pipes to a depth of approximately 9 feet. Fuel oil under high pressure was jetted into the canal bank at points staggered on 16 inch centers in two rows 1 foot apart. An average of 164 linear feet of bank was treated per day with approximately 4.1 gallons of oil per linear foot. Costs were estimated at about $0.40 per linear foot of canal bank. The effectiveness of the installation has not yet been determined.

Asphalt cutoff walls may be constructed by excavating a trench through a leaking embankment and then either mixing the removed material with an asphaltic oil and replacing it in the trench, or by coating the walls of the trench with an asphaltic membrane, followed by backfilling the trench with the original or other suitable materials. Such a method has been successfully used by the Imperial irrigation district of California. No experimental installations of this type have as yet been made by the Bureau, but several installations are planned on the Klamath project for construction in 1948. The method appears to have very good possibilities for use in sandy or gravelly soils where trenches can be excavated by trenching machines, and where the soil is sufficiently stable so that caving of the ditch sides during excavation does not occur. Further data on this method should be available by the end of 1948 upon completion of the Klamath installation.

Photograph No. 29

Hand-placed asphaltic concrete lining November 1939--
Snipes Mountain Canal, Washington.

Photograph No. 30

Asphaltic concrete lining in Malheur Canal--placed
November 1944--Owyhee Project, Oregon.

Photograph No. 31

Asphaltic concrete lining being placed in Pasco Laterals, Washington--June 1947.

Photograph No. 32

Cold-mix lining (asphalt emulsion and fine sand) in "D" Line Canal--Boise Project, Idaho. (Note shrinkage cracks.)

Photograph No. 33

Applying SC priming oil over diesel oil preprimed earth, prime membrane lining--East Turbine Lateral--Yakima Project, Washington.

Photograph No. 34

Applying diatomaceous earth-filled asphalt cement membrane over primed soil, experimental area of Pasco Laterals, Washington.

Photograph No. 35

Section of prime-membrane lining in "H" Line Canal--
Boise Project--after six irrigation seasons.

Photograph No. 36

Applying asphaltic membrane to subgrade for buried-
membrane lining--Klamath Project, Oregon--September
1947.

Photograph No. 37

Placing earth cover over asphalt buried membrane lining--Klamath Project.

Photograph No. 38

McCorkle-type prefabricated asphalt lining "D" Line Canal, Boise Project, Idaho.

Photograph No. 39

Placing Shell Oil Company prefabricated asphalt lining--Yakima Project, Washington.

Photograph No. 40

Applying asphalt cover pneumatically over old p. c. concrete lining--Ridenbaugh Canal--Boise Project--March 1947.

EARTH MATERIAL LININGS

A wide variety of earth materials can be economically utilized by several methods to reduce canal seepage losses to an acceptable minimum. The permeability of some in-place soils can be greatly reduced and their stability improved by compaction or by mixing with other soils. Relatively impervious borrow material is often available near the canal or within a reasonable hauling distance which can be used for lining. Such material may be placed loose, compacted, or, under favorable conditions, may be introduced into the operating canal for deposit along the earth section. Many soils may be stabilized and rendered less permeable by chemical treatment, oil pentration, or by the addition of binders such as portland cement. The uses of oil and asphalt for lining canals are covered elsewhere in this report.

Soil linings not treated with cements, chemicals, etc., are, of course, susceptible to weed growth, particularly the loosely placed linings. Aquatic weeds in the canal reduce the carrying capacity of the canal considerably and yearly removal of weeds on Bureau projects is an expensive operation. Mechanical removal of weeds is often destructive to earth linings and in many cases prohibitive in cost. However, inexpensive chemical weed removal is becoming widespread and effective, reducing the importance of the weed problem. Densification of the soil reduces weed growth considerably; therefore, thin, compacted linings with protective covers are more desirable from this standpoint. Earth linings could be sterilized with the same materials used under asphaltic linings, but the added cost of about $0.10 per square yard is usually not justified. Furthermore, sterilants would be effective for only a few years because of dilution or leaching out by the water in the canal.

Compacted In-Place Soils

In some cases relatively fine-grained soils can be compacted in place to adequately control seepage. By compacting some materials from 90 percent of maximum density to 100 percent, the permeability may be reduced to 1/100 of the original rate. The data pertinent to construction consist of a determination of the in-place density, the maximum density, and optimum moisture by standard laboratory compaction, determining the percentage of compaction (percent of density) required to decrease the permeability from the in-place permeability to an acceptable amount. The construction procedure consists of scarifying the soil, adding moisture, and compacting by sheepsfoot, flat rollers, or other equipment to the required density. A protective cover of loose soil or gravel, 6 to 12 inches thick, should be provided if scouring or surface shrinkage is anticipated.

Mechanical Stabilization

Soil stabilization and decreased permeability may be accomplished by improving the frictional and cohesive properties of the soil. The incorporation of granular soils with clayey soils improves the frictional properties of the clayey soils. The addition of clayey soils to granular soils improves the cohesive properties of the granular material. Thus, by the proper combination of mechanically different soils, better cohesive and frictional properties are obtained.

Loosely-Placed Earth Borrow

Probably the most prevalent type of earth lining now in operation is the loose earth blanket of selected fine-grained soils. Satisfactory results can be obtained with this type of lining in many cases, providing the soils are fine enough to be impervious in the loose state, and are sufficiently stable against erosion. The results obtained from the loose earth linings are not as positive, permanent, or efficient as the results obtained from compacted earth linings of comparable materials. The advantage of the loose earth lining is the ease with which it may be installed, either during the construction period or after the canal is in operation. Very little trimming or reshaping of the canal section is generally required and the installation cost is usually low. A 12-inch layer of soil is recommended for this type of lining. In some cases the primary benefit to be derived

from this type of lining is in the fact that, although most of the soil may be removed by erosion or cleaning operations, the fine materials may penetrate into the canal banks to provide a seal. Laboratory testing for this type of lining includes determination of the permeability of various available materials in the loose condition and whether any movement of the fines would influence the permeability rate. The construction procedure consists of dumping the loose soil on the canal bottom and banks and shaping or spreading to grade and line. In some instances loose linings have been "puddled" in by dragline bucket after the canal has been filled with water, to increase the efficiency of the lining.

Compacted Earth Borrow

Thin compacted linings, 6 to 12 inches in thickness, of selected fine-grained soils which are protected by a blanket of coarse soil or gravel, 6 to 12 inches thick, provide a suitable impervious lining. The maximum density and optimum moisture of materials for this type of lining are determined in the laboratory by the compaction test and permeability is determined by the percolation test. The laboratory tests on compacted materials indicate that severe freezing and thawing would be detrimental to thin linings because the expansion caused by the freezing and subsequent thawing increases the permeability of the material. However, no field reports on existing thin linings have indicated a failure because of freezing and thawing action. Repeated wetting and drying of thin linings should have no effect on the stability or porosity if properly protected against erosion by a sand-gravel blanket. Specifications for placing this lining should include a moisture and density control for obtaining satisfactory compaction. Density requirements for canals are usually established at about 93 percent of maximum laboratory density because of the difficulty in attaining higher density on the canal slopes with present construction equipment. Whether transverse or longitudinal compaction methods are used, auxiliary equipment is needed on the berm to assist the compaction equipment. Longitudinal compaction can be accomplished by anchoring the rolling equipment on the slopes to equipment on the berm. In Photograph No. 41, transverse compaction has been accomplished, in some stabilized test sections by pulling the rolling equipment up the slope by cable through a deadman to a tractor which operated longitudinally along the berm. Another method of transverse compaction utilizes a sheepsfoot roller and dragline as illustrated in Photograph No. 42. The most suitable soils for thin compacted earth linings contain small amounts of clay and considerable sand. Soils arranged in order of suitability for thin compacted linings are:

SC Sand with clay binder

GF - clayey, Gravel with excess clay

SF - clayey, Sand with excess clay

CL Clay (lean)

CH Clay, very plastic (not suitable for canals that will be alternately wet and dry unless protected with a 12-inch gravel-sand cover).

Heavy compacted linings are commonly used for slope linings because conventional rolling equipment can be used longitudinally to compact horizontal layers. (See Photograph No. 43.) The lining thickness depends on the width of available equipment, but 3 feet (normal to the slope) would probably be the minimum thickness possible with a single-drum roller. This type of lining can be constructed of coarser material than a thin lining and no protective cover is required. The soils described above as suitable for thin linings are also suitable for the heavy linings and a few other types can also be used. These are: gravel with clay binder (GC), gravel with excess silt (GF-silty), and sand with excess silt (SF-silty).

In December 1944, about 700 linear feet of compacted earth lining was placed in the Bitterroot main canal, Montana. At this section the canal has a base width of 22.0 feet, side slopes of 1-1/2 to 1, and a bank height of 9 to 10 feet. The normal water depth is 5.25 feet. Before placing the lining, the canal was overexcavated on the bottom

and lower bank and evened up with a Cletrac "60" dozer. The canal having been in operation for 35 years (constructed 1907-1910), the subgrade was in a good state of compaction and no further treatment was considered necessary. The material used for the lining was a sandy-clay material containing sufficient gravel so that it was found unnecessary to place a protective layer of gravel over the lining. The borrowed material was shoved into the canal with the dozer, spread out on the bottom and along the slope of the lower bank to a height of 14 inches above normal water surface. The upper bank was not lined. Compaction of the lining was done by traveling with the Cletrac tractor. The thickness of lining was 12 inches on the bottom of the canal, but on the side slope it varied from 20 inches at the bottom to 14 inches at the top. The lining has performed well under operation and there have been no reports of either sloughing or erosion. Aquatic growth, silt deposits, and the amount of cleaning required does not vary from that required for unlined sections of the canal. No measurements were made to determine the seepage losses before and after placing of the lining. However, before the lining was placed, there were about five acres of land below the canal which had become too wet to farm. Since the lining has been placed, this land has dried up to the extent that farming is again possible.

A compacted clay lining, 5 to 6 inches thick with a 1-inch layer of gravel rolled into the surface was constructed by WPA labor in 1941 on the Melville "C" canal, Delta, Utah. The canal has a base width of 16.0 feet, side slopes 2:1 and height of 3.5 feet, and is 4,200 feet long. Reshaping of the existing canal, placing and spreading of clay and part of the compaction was done by hand labor. A smooth roller was substituted for hand-compaction early on the job. The density of the compacted lining varied from 74.5 to 94.5 pounds as compared with the natural condition of the clay of 97.1 pounds per cubic foot. A clay having a permeability of 0.13 feet per year was used for the lining. A 1-inch layer of gravel was spread over the clay before compaction. The cost was $0.425 per square yard and this lining has served its purpose for 6 years without repairs. However, considerable erosion is evident at the toe of the bank slope. On the outside of curves and under two bridges the lining has been entirely removed by erosion because of the higher velocities caused by restricted sections. Ninety percent of the lining is covered with a small leaf moss which is unaffected by chemicals used so far. Permeability measurements indicate an increase of from 0.12 cubic feet per square foot per year immediately after the lining was placed to 0.42 cubic feet per year in 1944. A high water table, which was lowered after the lining was placed, is again evident.

Earth linings have also been constructed in which the slope lining was placed in a loose condition and the base lining was compacted. This procedure allows the densification of the base material for maximum imperviousness and stability by conventional rolling equipment and provides a loose lining of moderate impermeability on the side slopes where compaction is difficult with present equipment. During 1940, 5 miles of this type of lining was constructed on the All-American canal. The base lining consisted of 4 inches of clay soil thoroughly mixed with 4 inches of base soil and compacted. (See Photograph No. 44.) The slope lining consisted of 6 inches of loose clay soil. The lining involved 153,200 cubic yards of material and was placed under contract at a cost of $0.064 per square yard of base surface and $0.096 per square yard of slope surface. A field report on the condition of this lining late in 1946 stated that wave action had produced some sloughing and probably much of the lining had been removed from the side slopes. However, there was no indication of seepage losses on farm land below the canal, and it may be that an effective seal coating or film still remains which may be covered over by the coarser materials washed down from the banks. Prior to the installation of the earth lining, ponds appeared on the lower side of the canal, and some land was waterlogged. Seepage losses in the unlined canal were never fully determined with certainty, but estimates ranged as high as 1 percent of inflow per mile of canal. Current meter determinations after the lining had been installed indicated that losses had been reduced to 0.1 of 1 percent per mile. Late in 1947, however, ponds of seepage water were again observed in the fields below the canal and it was necessary to install additional drains to remedy the situation. The actual condition of the lining is not known since the canal is in year-round service.

Silting

Seepage losses are frequently reduced by silting operations. Silt may be picked up from the canal section and deposited elsewhere, may come from the water supply, or may be artificially introduced into the canal. The introduction of silt into the canal is probably the least expensive method for reducing seepage losses, if the silting material is available near the canal. The seepage reduction by silting depends upon forming a thin impervious surface membrane of silt and clay on the wetted perimeter of the canal. This membrane is highly susceptible to attrition at the water surface, puncture, deterioration by weathering, and destruction by cleaning operations. Some silting operations in coarse sands and gravels will provide a more-or-less permanent impervious lining if the silt penetrates into the coarse materials to a considerable depth. Artificial silting can be accomplished by dumping select material into the canal by means of a dragline or by sluicing the fine materials into the canal stream by means of a slotted flume which can be placed across the canal. The effectiveness and practicability of silt linings are dependent upon the available silting materials, the velocities of the canal, and upon the structure of the formation through which seepage occurs. This type of lining, although satisfactory in some cases, should not be regarded as permanent or impervious as other types of earth linings.

Both loose earth lining and silting operations were used to reduce the seepage losses in the main canal of the Vale project, Oregon. The operations continued over a period of years and were effective in reducing the seepage losses from 75 percent of total flow in 1930 to 20 percent in 1940. Where heavy visible seepage losses occurred, top loam was placed on the slopes and the bottom of the canal approximately 6 inches thick. From 1930 to 1935, 50,214 cubic yards of earth were placed at a cost of from $0.50 to $0.65 per cubic yard ($0.08-1/3 to $0.10 per square yard, 6-inch thickness). This earth blanket reduced the seepage losses from 75 to 40 percent. The remaining 20 percent reduction was accomplished by silting operations. Selected soils near the canal were washed into a notched trough extending across the canal, which spread the silt over the water surface. The silt placed by this method was carried as far as 50 miles, which adhered to and was drawn into the pores of the natural soil. This silting method was found very effective in reducing the seepage through the minute seams found in the underlying rock formations. The cost of silting amounted to about $0.50 per cubic yard of material handled.

During 1946, 1947, and 1948, Government forces on the Provo reservoir canal, Utah, installed more than 2 miles of loose and compacted linings and silt treatment considerably reducing seepage losses. The loose earth lining was semi-compacted in the bottom by equipment working in the canal. Compaction of the 6-inch slope lining was accomplished by the use of a 3/4-cubic-yard dragline and 7,000-pound single roller with hookup similar to that shown in Photograph No. 42. Silting was accomplished by dumping clay from existing bridges and other structures at points of maximum turbulence. Clay material was also dumped along the canal bank and down the side slopes from the operating road. This spilling operation caused a noticeable cross current which assisted in spreading the material across the bottom section of the canal and on the opposite bank. Clay introduced by these several methods was carried in suspension and deposited along the perimeter of the canal for a considerable distance downstream. Although these several lining operations were confined to less than 2 miles of canal length, current meter measurements indicated an overall reduction in seepage losses over approximately 11 miles from 1.73 percent per mile to 0.5 percent per mile. Cost of the compacted lining was reported as $0.17 per square yard; the silt lining, or the silt treatment, $0.14 per square yard; and the loose lining ranged from $0.16 to $0.29 per square yard depending on the distance the clay had to be hauled. In transmitting a report on these installations in November 1947, the project engineer stated, "In view of the proven results obtained from the type of clay canal lining completed to date on the project, it is now planned to eliminate approximately 2 miles of the original proposed 4.3 miles of concrete lining on the Provo Reservoir Canal."

Bentonite

Bentonite may be defined as earth material which contains 75 percent or more of clay materials of the montmorillonite group. Chemically pure bentonite is a natural hydrous silicate of alumina. However, because of various other chemicals that may be included in the natural soil, all bentonites do not exhibit the bentonitic characteristics in the same degree. The characteristics of bentonite, whether beneficial or detrimental

to the engineering problem in question, include slipperiness, imperviousness, high water absorption, and swelling. Disregarding the bentonitic grouping from a chemical standpoint, and considering them from an engineering construction standpoint, the bentonite can be classed in two groups. The Wyoming-type bentonites show a strong affinity for water and the wetting is accompanied by commensurate swelling. The Metabentonites have mineralogical properties similar to the Wyoming-type bentonites, but do not exhibit pronounced swelling characteristics. To differentiate between these two groups, chemically, the presence of sodium in the first type favors great swelling, whereas the presence of calcium, magnesium, aluminum, and potassium, in the second type tends to inhibit swelling.

There are several petrographic and chemical tests to determine the presence of bentonite. One physical test--the free-swell test, can easily be performed in the field and will indicate the swelling properties of the clay. Briefly, the test consists of adding 10 cubic centimeters of dry material (passing the No. 28 screen and retained on the No. 48) slowly to a 100-cubic centimeter graduate containing 100 cc of water. After standing 24 hours, the apparent volume of the settled clay is determined. The final volume is expressed as a percentage of the original (10 cc) volume. Other more elaborate and definite physical tests are available where the detrimental properties of bentonites are being considered in relation to an engineering structure. These would include the consolidometer type of swelling test, the uplift test, and the triaxial-shear test which may be performed on remolded or undisturbed soil specimens.

Two types of bentonite linings are feasible using the Wyoming-type or high swelling material. One type is the bentonite membrane lining with a protective layer of gravel or earth. The construction procedure consists of spreading a 1- to 2-inch layer of the loose, dry, high swelling bentonite on the canal section and spreading a protective cover over the membrane. The second type of lining using bentonite consists of a mixture of 15 to 30 percent bentonite with local soil. A 2- to 3-inch compacted layer of soil-bentonite protected by 4 to 8 inches of sand and gravel, has provided linings adequately impervious, stable, and resistant to weathering action. The construction procedure consists of mixing the moist earth and bentonite, spreading and compacting the mixture, and spreading the protective blanket over the compacted lining.

A laboratory research investigation in 1941 on the uses of bentonite as a canal lining material resulted in the following conclusions:

(1) The most satisfactory method of using bentonite for limiting canal seepage losses appears to be in the construction of thin compacted linings of mixed soil and bentonite.

(2) Either high swelling or low swelling bentonites may be used, but, in general, more of the low-swelling material would be required for comparable results.

(3) Bentonite used in mixes should be ground at least to 90 percent passing a No. 28 screen and 40 percent passing a No. 100 screen.

(4) The amount of bentonite required for a satisfactory lining depends upon the swelling properties of the bentonite, the fineness of the bentonite, the thoroughness of mixing, and the porosity of the soil used.

(5) Compaction of a soil-bentonite mix is necessary to insure stability on canal banks. For best results, the soil should be moistened to about optimum moisture before mixing.

(6) In general, the compacted layer should be at least 2 inches thick. Heavier layers are usually required when machine methods are used to insure an adequate minimum thickness. The lining should be covered with a minimum of 6 inches sand and gravel or compacted earth, the thickness depending upon the stability of the material used.

(7) Membrane linings of 100 percent bentonite may be feasible in some localities where local bentonites are readily available, although this type of lining appears to be more susceptible to deterioration through weathering than the mixed lining.

Three reaches of the Frenchtown main canal, Montana, were treated with bentonite to reduce seepage losses in April 1940 after 3 years of operation. A total of 1,055 linear feet, (2,579 square yards) of lining was installed in 4- to 6-foot base width sections. These canal sections were excavated through a light, fluffy soil with some rock strata and gravel seams which resulted in heavy seepage losses and unstable lower slopes. The subgrade was in fair condition of density and no additional treatment was made. The canal was first overexcavated to a depth of 4 to 5 inches, the bentonite was spread out over the section to a thickness of 1/4 inch, and the soil from the overexcavation was placed back over the bentonite membrane. This soil cover was 4 to 5 inches thick and was not compacted. The lining has been in service for seven irrigation seasons, and no sloughing or washing has occurred. However, land below the canal is practically as wet now as before the lining was placed. The protective soil layer has apparently remained in place, yet the project manager states that he has excavated several test holes through the lining without finding any noticeable evidence of the bentonite membrane. The bentonite membrane, in this case, is believed to have been too thin (1/4 inch) to satisfactorily serve as a seal. In linings of this type a bentonite membrane thinner than 1 inch is not recommended.

A satisfactory membrane-type bentonite lining was constructed by project forces in the Huntley project main canal, Montana, during 1940 and 1941. This lining consisted of 2 inches of loose bentonite covered with 12 inches of fine material and gravel and was reported to cost less than $0.35 per square yard to install.

A lining of bentonite-earth mix was placed in 3,550 feet of the Heart Mountain canal, Wyoming, in 1942 and 1943 as shown in Photographs Nos. 45 and 46. The lining mixture was compacted to a thickness of 2 inches and covered with 12 inches of fine material and gravel. This lining has been satisfactory in reducing seepage losses, but was excessive in cost, probably because War Relocation Authority labor was employed.

A 2-inch compacted bentonite-earth mix lining was placed in a canal of the North Platte project in Goshen County, Wyoming, by CCC labor in 1941. The canal had a base width of 6.0 feet, slopes 1-1/2:1 and 2.0-foot water depth. The lined section was 450 feet long and a protective cover was placed over the lining. The canal section was overexcavated .75 feet with a dragline. One part bentonite and four parts sand were mixed in a concrete mixer dry and then a small amount of water was added to assist compaction. The mixture was hand-tamped and covered with the original excavated material. The cost of this lining was $2.00 per square yard. At the close of the 1946 irrigation season, although no explanation is advanced, the lining was apparently worthless. Some slippage had occurred on the side slopes and water loss is again quite noticeable on adjacent fields below the canal.

Soil-cement

The extensive use of mixtures of soil and portland cement for paving air strips and secondary roads suggested the possible use of this material for canal linings. Materials for road and airfield surfacing are usually mixed at optimum moisture and are compacted in place to maximum density, although a plastic mix is sometimes used. The required compaction of the relatively dry mixture, termed standard soil-cement, is much more difficult on the side slopes of a canal than on the flat surface of a highway or airfield and is therefore more expensive. For this reason canal-lining experiments have included the use of the plastic soil-cement which can be placed with a slip-form similar to the type used for placing concrete linings. The advantage of soil-cement lining is its possible low cost resulting from the use of soil along the canal and from the economical methods which have been developed for handling and mixing the material. This advantage is presently offset, in the case of the standard mix, by the need for costly compaction. Plastic soil-cement requires no compaction other than that resulting from the weight of the slip-form.

Not all soils are suitable for use in soil-cement. In general, sandy soils are preferable to those containing high percentages (over 35 percent) of silt and clay. The suitability of local soils for use in soil-cement should be determined by laboratory tests. Soil-cement tests have been developed which determine accurately the mixture of soil and cement as well as the suitability of the soil. The test procedures for standard

soil-cement are covered by A.S.T.M. Standards D 558-44, D 559-44, D 560-44, and D 806-44T. From these tests the following information is obtained:

(1) The economical quantity of cement required to harden a particular soil into a durable surface.

(2) The moisture necessary with each soil-cement mixture.

(3) The density to which the soil-cement mixture should be compacted to obtain maximum effectiveness from the cement.

Because the materials are more difficult to mix in the field, the recommended cement percentage for field use is specified as 2 percent higher than the amount found to provide satisfactory soil-cement in the laboratory in the case of material containing less than 35 percent silt and clay. Should it become necessary to use materials having more than 35 percent silt and clay, the recommended cement percentage for field use should be 4 percent higher than the amount found to provide satisfactory soil-cement in the laboratory.

The construction procedure for soil-cement lining consists of preparing the foundation by trimming and densifying the loose or replaced materials and moistening the foundation just prior to placing to aid in curing the soil-cement. The soil for the mixture is pulverized so that 80 percent of the material, exclusive of rock and gravel, passes the No. 4 sieve. The moisture content at this time should be below optimum moisture but sufficiently moist to provide satisfactory pulverization. After the specified quantity of cement is added, the soil and cement are thoroughly mixed and, in the case of standard soil-cement, sufficient water is uniformly added to attain the specified moisture content for compaction plus sufficient water to compensate for evaporation losses during manipulation, transportation, and placing. The soil-cement is conveyed to and dumped into the canal, spread on the bottom and slopes to provide the required compacted thickness, and then compacted by sheepsfoot, pneumatic, or flat rollers, and finished by additional rolling with pneumatic or flat rollers. (See Photograph No. 47.) The pulverizing, moistening, and cement mixing can satisfactorily be accomplished in a traveling pug-mill type mixer equipped to pick up the soil and cement from a windrow along the berm.

As previously stated, the compaction of slopes in canal lining construction is difficult with conventional equipment. It has been found advantageous, therefore, to increase the moisture above the optimum required for standard soil-cement to produce plastic mixtures. Slip-forms or screeds, similar to those used in placing plastic concrete mixes, (see Photograph No. 11) may then be used to advantage. The increase in moisture content of plastic soil-cement decreases the density and resistance to abrasion, and it is necessary to increase the cement content to compensate for the accompanying loss in durability, as determined by A.S.T.M. Standard tests D 559-44 and D 560-44. A finish, comparable to that achieved with one pass of a steel float, is usually specified. All soil-cement linings must be protected from freezing and cured for 7 days by means of 2 inches of soil, straw, or burlap sacking, initially and subsequently moistened as needed. Commercially available bituminous membranes may also be used for curing.

Test panels of standard compacted and plastic soil-cement lining were placed on the main canal of the W. C. Austin project, Oklahoma, during May and June 1945. These panels included linings 4 and 6 inches in thickness, with variable cement contents. The 10 and 12 percent compacted soil-cement as well as the plastic mix appear to be holding up moderately well. Large pattern cracks have developed but no disintegration around the cracks has occurred. These cracks developed within the first 6 to 12 months after construction and have not become more pronounced since that time. Investigations showed no water loss through cracks. Some noticeable pitting had occurred on the surface of this lining, probably due to unmixed clay balls which have eroded. A standard concrete mixer was used in the construction of this lining, but is not recommended because the mixing is not complete. The pug-mill-type mixer is much more efficient. The material used in this installation contained considerable silt and clay (48 percent) and did not provide the most suitable soil-cement mixture. Sandy, friable soils are preferable for use in soil-cement linings due to the greater ease in mixing and placing as well as the lesser tendency to crack. Erosion from wave action or local runoff has not occurred in these test panels and no weed growth has occurred in contrast to the heavy growth of

thistles on the adjacent compacted-earth lined sections. It was also noted that the lining below the water surface was in better condition and contained less cracking than that part above the water surface.

A plastic soil-cement lining 711 feet long was placed in a farm ditch of the Yuma Mesa division, Gila project, Arizona. This ditch has a base width of 4 feet and a lining height of 2.5 feet. The lining was installed by the Branch of Operation and Maintenance and was completed about December 1945. The canal is through sandy soil which is easily eroded but is especially adaptable to cement stabilization. No compaction of the subbase was provided. Screeds were placed in 10-foot sections so that alternate panels could be placed with soil-cement, thus reducing shrinkage and providing contraction joints. The cement-soil proportions were 1:5 by volume and the water cement ratio was about 1:4. Mixing was accomplished in a conventional concrete mixer and damp sand was used for curing the soil-cement after which the ditch was ponded for about 2 weeks. Twenty-eight-day strengths on two 6 by 12 cylinders were 1089 and 1273 psi. Project estimates placed the cost at about $1.25 per square yard. After the lining was under service for about 15 months it appeared to be very satisfactory and its general condition was good. A few small contraction cracks appeared in the middle of many panels and there were a few localized areas where spalling had occurred. There had been no aquatic growth and, although seepage losses have not been measured, they were believed to be insignificant since seepage on adjacent lands was not indicated. However, during a recent inspection of this lining, it was noted that pattern cracking had increased and the general appearance of the lining was not as good as when previously inspected.

Additional plastic soil-cement linings have been placed on the Gila project by means of a slip-form. These linings were 2 inches thick using a 20-percent cement, by volume, mixture and a water-cement ratio of 1:1.2. Initially, this produced a very satisfactory lining at a minimum cost. However, subsequent sections developed cracks which prompted the project to discontinue this type of lining, without determining the cause of cracking. Further experimentation might have shown that this was caused by variable soil conditions, high temperature at time of placing, lag in placing the protective blanket, stresses caused by the slip-forms, or prestiffening in the cement.

On both the All-American and Gila projects, experimental dry mixed-in-place soil-cement linings have been installed. Construction procedure consisted of shaping the canal, spreading cement on the dry base, and hand-mixing the soil and cement to a depth of 3 inches. The canal was then reshaped and watered with a fog spray to set a top crust of soil-cement. Loose soil was then spread over the section and the section was ponded to allow hydration of the cement. This produced an unsatisfactory lining of nonuniform mixture and thickness, low density, and high porosity. Refinements of construction including better mixing and compaction have not produced an acceptable lining with the mixed-in-place procedure. This method has never been recommended. Photograph No. 48 illustrates the poor condition of this lining in its second year of service. Rigid adherence to control instructions is a necessary prerequisite for obtaining the most satisfactory soil-cement linings.

Three 500-foot test panels of soil-cement placed pneumatically in a lateral of the Gila project are described in the discussion on shotcrete. This method of placing a soil-cement lining did not prove satisfactory.

In cooperation with the Portland Cement Association, the Barber-Greene Equipment Company, and the Madsen Iron Works, the Bureau constructed a 4,480-foot plastic soil-cement lining on the West 11.5 lateral of the W. C. Austin project in May 1947. The canal section has a base width of 4.0 feet, side slopes of 1-1/2:1 with a slope length of 5.41 feet. The lining thickness is 3 inches, and about 4 cubic feet of soil-cement per linear foot of lining was required. The natural soils used were poorly graded fine sands of the SP type. The canal was excavated by dragline and finished with a ditcher. Sections of lining, containing 3, 3.9, 4.5, and 6 sacks of cement per cubic yard of plastic soil-cement were placed to determine the variation in lining performance due to cement content.

The two machines which made total mechanization of the process possible were the traveling plant mixer and the canal paver, shown in Photographs Nos. 49 and 50. The mixer is a pug-mill-type, with an overhead storage bin, and a self-propelled pickup

conveyer preceding the mixer. The sandy soil to be used for the lining was windrowed along the side of the lateral and the loose cement was placed in a groove on the top of the windrow. The windrow was picked up by the conveyer and carried to the pug mill where mixing and moistening were accomplished. From the discharge end of the pug mill, the plastic mixture was conveyed to the paver hopper. The mix was distributed to the slip-form which left a smooth, finished lining when properly operated. A maximum rate of operation, 351 linear feet per hour, was reached after several alterations to the slip-form were made and after operating personnel became familiar with the process. The line, grade, and thickness of the lining was controlled by the front unit (sled) of the paver. The paver was moved continuously by means of a winch attached by cable to a deadman ahead. Considerable difficulty was encountered in stopping, starting, and cleaning the slip-form. This necessitated patching after curing of the completed lining. Also, rapid stiffening of the mixture required rapid placement of the curing medium and repeated cleaning of the slip-form.

The finished lining was cured by four methods: spraying with a paraffin wax and diesel fuel mixture, spraying with a commercial white-pigmented sealing compound, spreading RC-2 bituminous compound, and spreading a damp sand cover. Most of the lining was cured with the damp sand cover because this covering could keep up with the slip-form and gave the best immediate results. The wax-diesel fuel mixture melted with rising temperature, flowed to the bottom of the canal, and excessive cracking occurred in the lining so treated. The RC-2 did not prevent cracking but had a tendency to fill them. The commercial white-pigmented compound was generally satisfactory.

Performance records for this lining are not complete at this time; however, several tentative conclusions are indicated. The placement of plastic soil-cement with a slip-form type of paver was found definitely practicable and economical. The pug mill-type mixer was ideally suited to mix plastic soil-cement and the canal paver, after rebuilding, proved effective in obtaining a reasonably smooth lining with the required thickness. The addition of a mechanical feeder and other improvements on the traveling mix-plant should decrease the cost of operation and increase the rate of production. High cement contents (above that necessary for durability) did not improve the plastic soil-cement, but caused the mix to become sticky and difficult to place. Cracking appeared to increase in direct proportion to the amount of excess cement in the mix. Curing by moist soil proved practicable and produced best results during actual construction. However, the use of white-pigmented sealing compound for the covering of patches resulted in even fewer cracks than the moist soil.

It appeared that most of the cracking in the sections containing lower cement contents was caused largely by the surface tension of the slip-form. This supposition is supported by the fact that most of the cracks are nearly normal to the centerline and that they do not appear in such a pattern in the larger patches which were hand placed. It is believed that a paver, pulled at a continuous constant speed, would provide more uniform surface and less cracking would result.

A total ayerage cost, including labor, rental, cement, supplies, and patching, of $0.78 per square yard was obtained. This figure is exclusive of trimming. A minimum cost of $0.61 per square yard was obtained under the most favorable operating conditions after the machines and the operation had been improved. Progress of 300 feet per hour on a 4-foot bottom lateral is entirely possible.

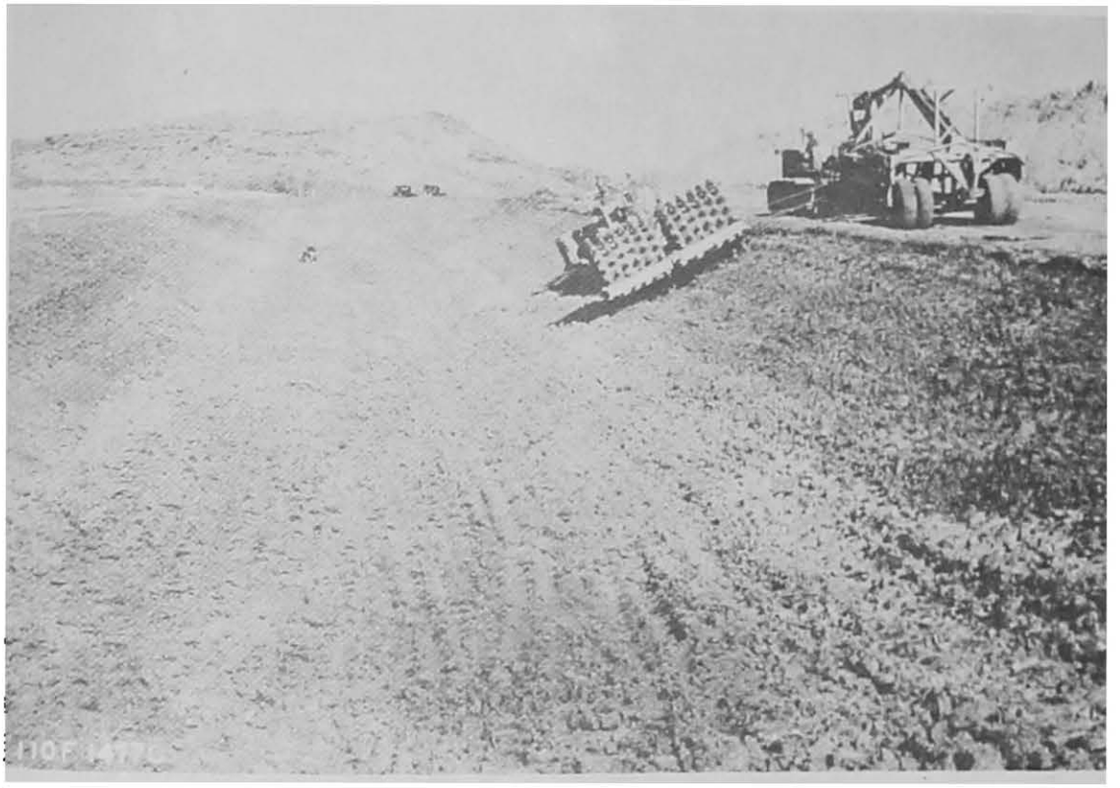
Resin and Chemical Stabilization

Specially treated resins in the form of powder when added to soils containing considerable clay have been used as stabilizing agents for air strips and secondary roads. These resins inhibit the infiltration of water and for this reason have been used experimentally in canal linings. The amount of resin required to stabilize the soil depends on the characteristics of the soil, but from a practical standpoint, it ranges from 1 to 3 percent. Because the resin prevents the entrance of water into the soil, mixing water must be added before the resin is incorporated in the soil. From 12 to 20 percent of moisture is required and maximum compaction is desirable. Linings of this type are mixed and compacted in the same manner as standard soil-cement. After the lining has been compacted, it does not have to be kept wet to cure.

An experimental section of resin-stabilized lining was constructed by the Bureau in the main canal of the W. C. Austin project in 1945. The resin used in this installation was Stabinol, a commercial product which has since been taken off the market because of its generally poor performance. The lining was 6 inches thick and consisted of a sandy loam soil excavated from the canal mixed with 1-1/2 to 2-1/2 percent of resin. The Stabinol consisted of about 70 percent portland cement and 30 percent of a sodium resin compound. The installation on the W. C. Austin project has suffered considerably from erosion, which indicates the necessity for a gravel or other protective blanket over this type of lining.

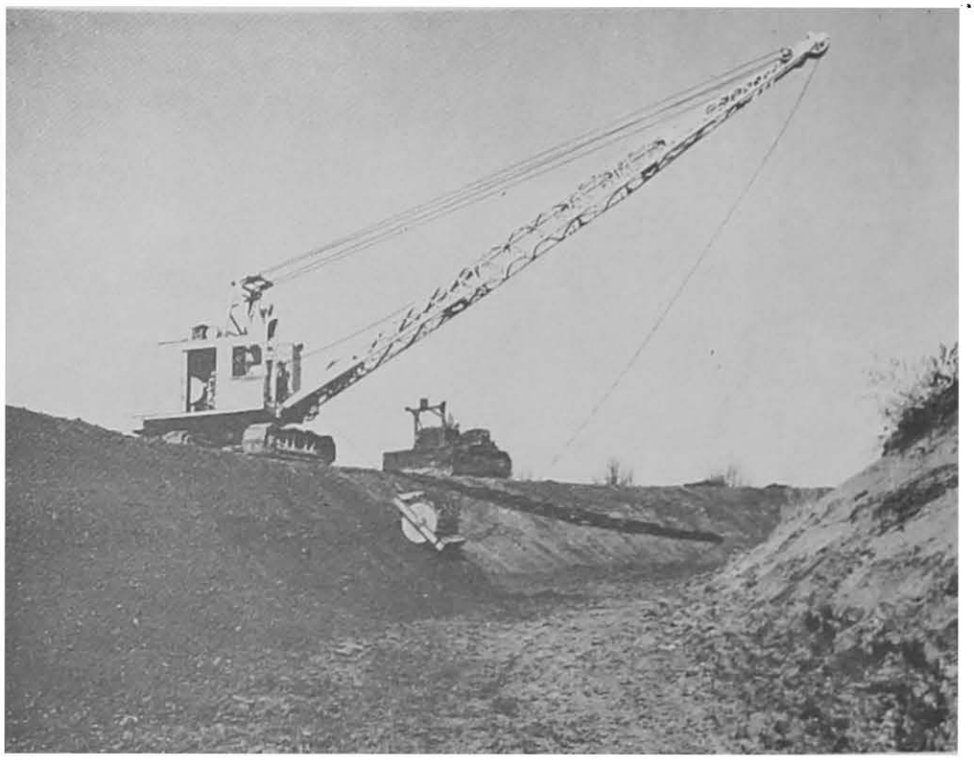
Sodium silicate in combination with sodium and calcium chloride has been used to stabilize sandy soils in deep excavations and to improve the bearing power of soils. Sodium silicate in liquid form produces a gel when added to sandy soils. The addition of sodium and calcium chloride to the sodium silicate sets the gel and forms a solid mass which is hard and impervious. However, the idea is not believed adaptable to canal use because of the exposure and the alternate wetting and drying to which it would be subjected, and no work has been done in this connection in the current Program.

Another possibility exists in the use of sodium chloride with soils containing an excess of calcium. Such soils usually take water readily and are quite permeable. The addition of sufficient sodium in the form of sodium chloride, to replace the calcium and to provide an excess, brings about a change in the soil characteristics rendering it relatively impermeable. Sodium chloride has been so used to seal ponds and reservoirs, but no record of similar treatment of canals is available.



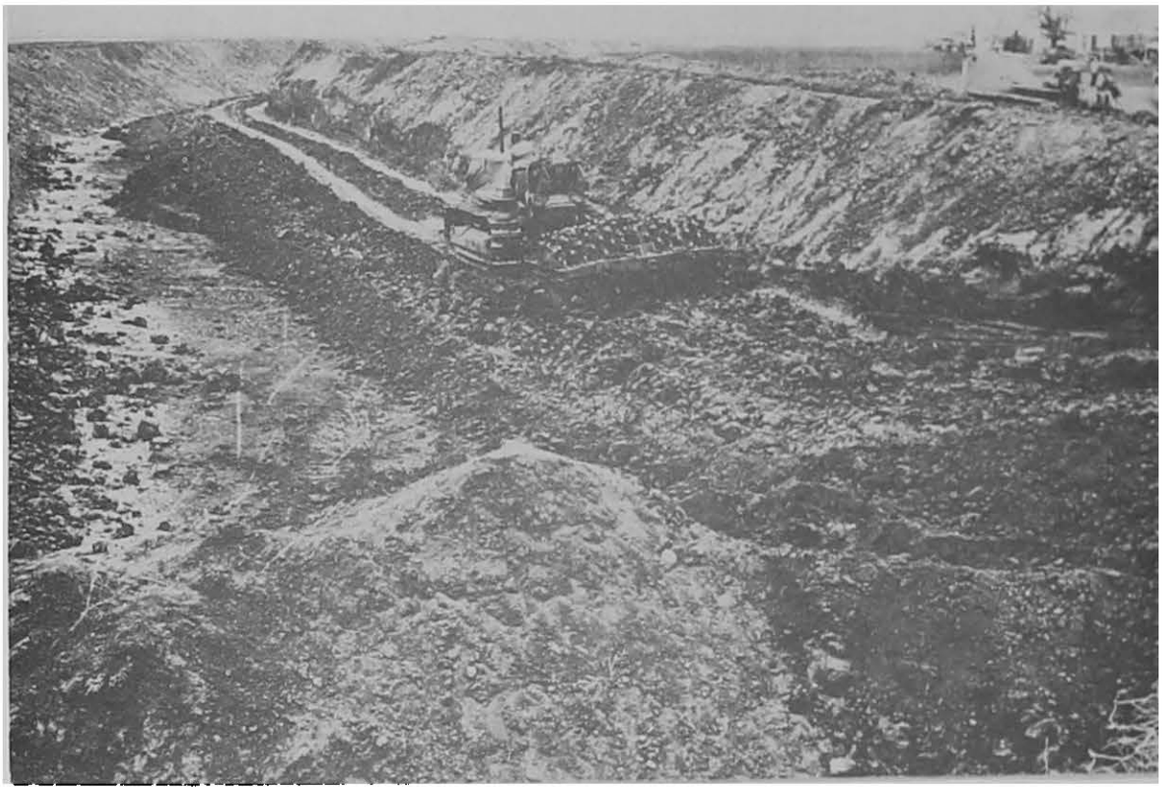
Photograph No. 41

Compacting earth lining on slope. Showing method of holding equipment on side slope--W. C. Austin Project, Oklahoma.



Photograph No. 42

Method of transverse compaction of earth linings--W. C. Austin Project, Oklahoma.



Photograph No. 43

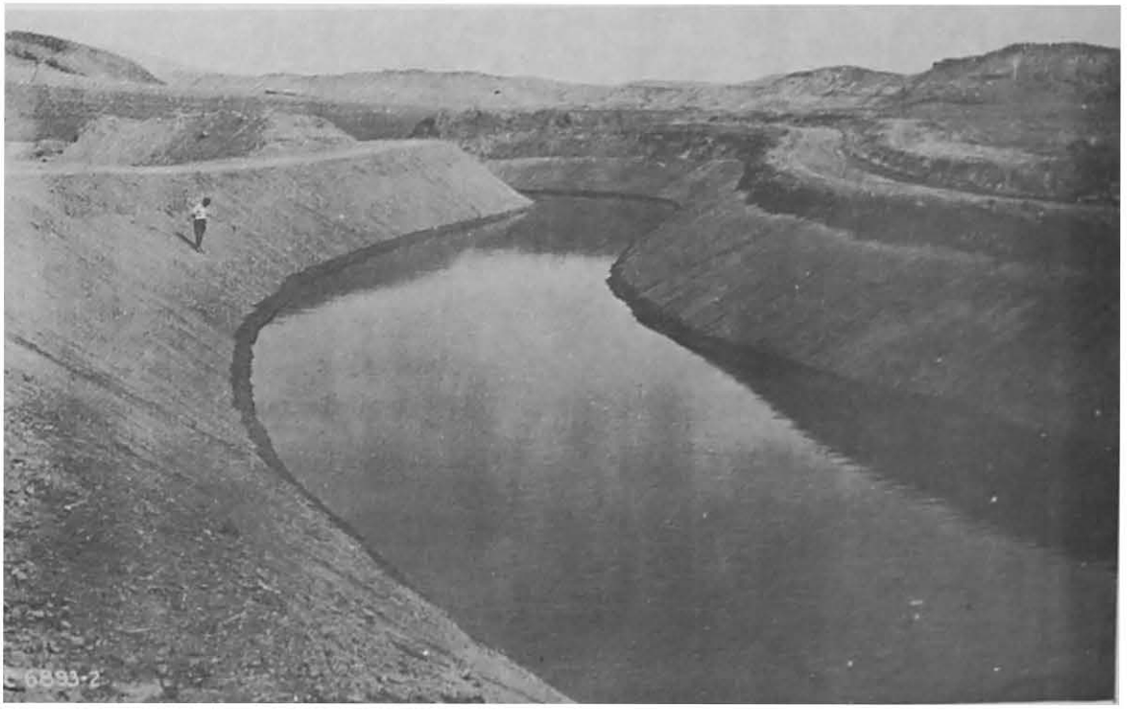
Longitudinal compaction of heavy compacted earth lining--W. C. Austin Project, Oklahoma.

Photograph No. 44

Rolling clay blanket on bottom of canal--All-American Canal.

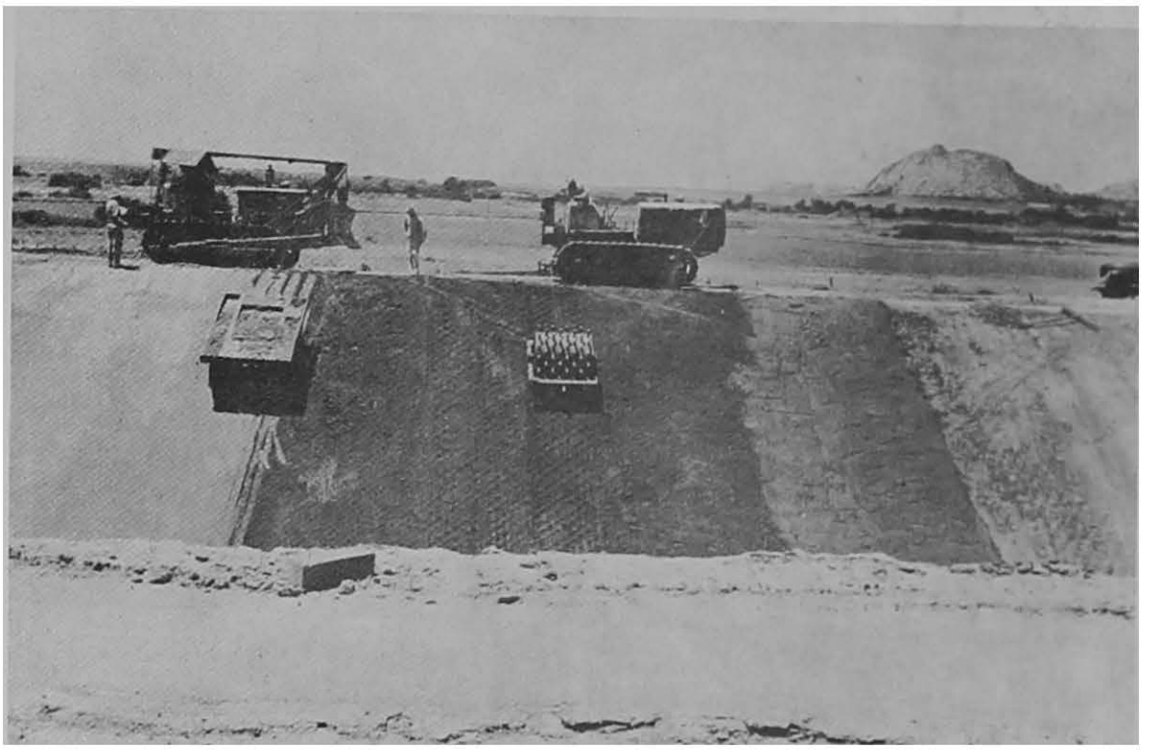
Photograph No. 45

Processing bentonite-earth mix in canal bottom--
Left slope has lining mixed in place--Heart Mountain
Canal--Shoshone Project, Wyoming.



Photograph No. 46

Completed bentonite-earth lining--Heart Mountain
Canal, Shoshone Project, Wyoming.



Photograph No. 47

Compacting soil-cement lining in slope--Main Canal--test section--W. C. Austin Project, Oklahoma.

Photograph No. 48

Erosion and disintegration of dry mixed-in-place soil-cement lining due to low density and nonuniform mixture and thickness of lining--Gila Project, Arizona.

Photograph No. 49

General view of Barber-Greene road mixer and Ekenstam Canal Paver at start of installation of plastic soil-cement lining--W. C. Austin Project. Oklahoma.

Photograph No. 50

Rear view of equipment placing plastic soil-cement lining--W. C. Austin Project, Oklahoma.

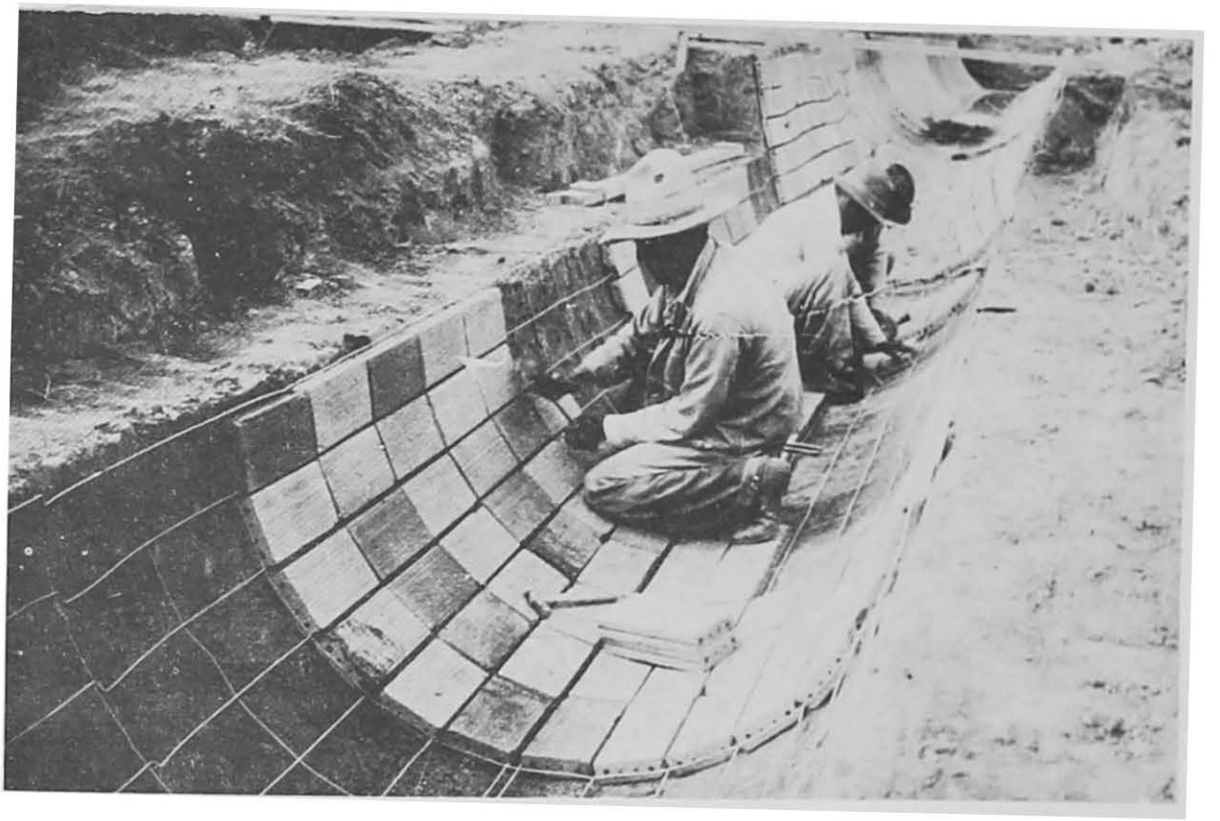
The first extensive use of clay brick for canal lining purposes in the United States of which there is any available information was on a private irrigation district in Texas in 1933. The bricks that were used were ordinary clay bricks salvaged from wrecked buildings. The canal section was a semicircle. The bricks were placed on the subgrade with a sufficient interval between to allow for mortar which was dumped on the bricks and broomed into the openings. The bricks on the side slopes were laid with a trowel. Following the laying of the bricks, mortar was brushed or broomed over the interior surface. No reinforcement was used.

Later, a brick of special design was developed which has been used rather extensively in the Lower Rio Grande Valley (142, 144, 146). Intended primarily for canal lining use, this brick is 1-1/2 by 5-1/2 by 11-1/2 inches in size with longitudinal cylindrical holes which decrease the weight and permit the mortar between the ends of the bricks to enter the holes and serve as dowel pins. Grooves in the longitudinal edges are provided for centering reinforcing mesh which properly spaces the bricks for brooming a thin cement mortar into the joints (see Photograph No. 51). Additional mortar is brushed over the surface to a thickness of approximately 1/4 inch. Although this type of lining has not been in service for a sufficient length of time to conclusively judge its durability, it was reported that an inspection showed no evidence of deterioration, especially where reinforcing had been used, Photographs Nos. 52 and 53. Some small hairline cracks were noted in the reinforced brick linings, but there was no evidence of seepage. In the unreinforced brick linings, there were both longitudinal and transverse cracks which had been repaired with asphalt. It was reported that in the opinion of the managers of the irrigation districts in the Lower Rio Grande Valley who have used brick linings that this type of lining is as satisfactory as gunite for use in small canals and laterals, and in that area is more economical. Total cost of the finished lining, including fine trimming and all materials, was reported as $1.80 per square yard. However, all operations were performed by common labor at $0.30 per hour which is not ordinarily available in other localities.

Brick linings have been used rather extensively in India where an abundance of cheap labor is available and where materials for concrete linings are difficult to obtain. The Haveli canal in India in 1937 was lined with a double layer of 12- by 5-7/8- by 2-1/2-inch tile brick. The bottom layer was bedded in 1/2 inch of 1:6 cement mortar. Both layers were placed with 1/2 inch of 1:3 cement mortar between the bricks and between the layers. The lining was reinforced with 1/4-inch bars at 24-1/2-inch spacings, longitudinally and transversely, on the bottom and 12-1/4-inch spacings, longitudinally and transversely, on the side slopes. A plaster coat was applied to the surface of the last layer of bricks. Except for some damage from settlement of the subgrade, this lining has been satisfactory. The report stated that in future work, brick 10 by 4-7/8 by 2-3/4 inches in size were to be used and that the reinforcing was to be eliminated because experience indicated that any damage from back-pressure or flotation was increased by reinforcing which prevented early failure in small local areas without extensive damage. No cost data are available on this work in India.

Brick linings have advantages which, under certain conditions, recommend their use. The coefficient of expansion of brick is approximately one-half that of concrete or gunite. Therefore, there are fewer temperature cracks and there is no need for expansion joints. The bricks are particularly adapted to use in the more hydraulically efficient, semi-circular section which offers no great difficulty to the placing of a brick lining but which, because of the steep sides, precludes the use of cast-in-place concrete unless forms are employed. The construction of a brick lining requires little equipment and is therefore particularly suited for lining small areas or short, segregated sections of a canal which would require frequent interruptions and movement of equipment. However, the most important advantage of brick linings is that unskilled labor can be used almost entirely in its installation. This factor, plus a readily available supply of local bricks, is the main reason why this type of lining has been found to be more economical than concrete or shotcrete linings in areas where an abundance of cheap labor exists.

However, if it is necessary to employ high-priced labor or skilled brick masons, a brick lining which involves hand labor throughout and which is not susceptible to machine placing would not be as low in cost as a first-class concrete canal lining which, with the machines that have been developed recently, require relatively little expensive hand labor. In addition, unless the brick can be manufactured in sufficient quantity near the construction site, the cost of handling and of transportation would probably prohibit its use for canal lining.



Photograph No. 51

Placing brick in wire mesh reinforcement prior to brooming of thin cement mortar into open joints and over interior surface--Lower Rio Grande Valley, Texas.

Photograph No. 52

Typical brick lining approximately 10 years old--Valley Gravity Project, Texas.

Photograph No. 53

Brick canal lining placed in 1947--Valley Gravity Project, Texas.

STONE LININGS

Probably rubble masonry or stone linings have been used more frequently in the centuries past than any other of our present lining materials. Installations of almost every description and age can be located in various parts of the world, many of them in service at present. A modern example of such construction is the lining installed on the North Fork canal near East Highland, California. It consists of three different types:

1. Stones chinked with broken stones
2. Stones with grouted joints
3. Stones with grouted joints and a plaster finish

The lining is about 10 inches thick and was installed to resist erosion and not to prevent seepage. Reportedly, it has given excellent performance and is in good shape. Such linings are common in that region wherever the grades of the canals were steep and a plentiful supply of natural stones exists.

Rock masonry has been used for the lining of a number of smaller laterals on the Carlsbad project in New Mexico. The typical installation shown in Photograph No. 54 consists of quarried or natural rock 2 to 4 inches in thickness bound together with cement mortar. Most of the work at Carlsbad was accomplished by CCC forces and for this reason the unit costs are of little value for comparative study. The masonry linings on this project were reported to be in very good condition with only isolated damage due to displacement by tree roots.

Rubble masonry provides a strong permanent lining. If given a smooth plaster finish it is an efficient carrier of water. However, its use is rather limited because of the large amount of hand labor required and because a natural supply of stone must be available. In regions of cheap labor the first factor might not prohibit the use of such linings, but in areas of higher priced labor or in areas where natural stones were not readily available, the cost of such linings would exceed that of concrete linings. It was stated in the report on the rock lining on the Carlsbad project that it is believed that rock linings are not adapted to contract work.

Photograph No. 54

Masonry lining installed about 1941 using quarried rock 2 to 4 inches thick with mortar joints--
Carlsbad Project, New Mexico.

SYNTHETIC PLASTICS

A study of materials for lining canals and laterals would not be complete without consideration of the synthetic products popularly termed plastics. Most plastics, manufactured from organic materials, are available in sheets or in various granular forms. Many of the sheet plastics have high mechanical strength and high resistance to rot and weathering. The use of sheets, however, presents a problem in bonding such a lining to the canal section. Sheets of reasonable thickness (0.005 to 0.02 inch) vary in cost from $0.20 to $0.50 per square yard. The granular materials are usually cheaper and may be utilized by dissolving in an appropriate vehicle and spraying or otherwise coating the earth section or old lining. A built-up membrane, using jute fibres or glass fibres as reinforcing, has been suggested for trial.

Investigations to date have been limited to preliminary laboratory tests. At least 16 samples from the leading manufacturers of plastics have been received and are being tested. Rot resistance is determined by tensile tests of samples buried in compost piles. Weathering characteristics are observed after roof exposure. Some experimenting has been done with granular products alone and as admixtures to asphalt, but no significant results have been obtained. Further investigations and possibly one or more experimental field installations of the most promising materials are contemplated.

Weeds do not penetrate portland cement concrete or soil cement, and hot-mix asphaltic concrete made with low-penetration asphalt and compacted to 95 percent of laboratory density, has a high resistance to ordinary weed growth. In contrast, asphaltic prime-membrane, cold asphalt mixes, pneumatically applied asphalt, and most prefabricated asphalt linings are quite vulnerable to weed penetration, which may eventually disrupt the lining. These types of asphaltic linings are not only susceptible to weed growth, but the black heat-absorbing surface acts as a hot bed which actually promotes seed germination and accelerates weed growth. Sterilization of the soil under such linings is usually a prerequisite, therefore, to reasonable serviceability.

Sodium chlorate, borax, and boric acid are effective soil sterilants, and are the materials most commonly used for this purpose. They may be applied as a powder, either scattered over the surface or placed in holes spaced 12 to 18 inches apart, or they may be sprayed on the subgrade in an aqueous solution. Sodium chlorate should be applied at rates from 4 to 8 ounces per square yard of treated surface, while borax and boric acid should be applied at the rate of 16 ounces per square yard. Sodium chlorate is readily soluble in water, and being an excellent oxidizing agent becomes a fire hazard on the clothing of workmen or on dry grass adjacent to the work. Borax and boric acid are both less water soluble than the chlorate, and these boron compounds are fire deterrents. Mixtures of borax or boric acid with sodium chlorate are frequently used because of the benefits of reduced fire hazard and more lasting effects. In such mixtures, borax is usually preferred to boric acid because of its lower cost. Borax may be used alone, but if applied as a powder its slow rate of solubility may result in inadequate early sterilization. Boric acid is more readily soluble than borax and therefore, the two boron compounds are sometimes used in combination. Oxides of arsenic are excellent soil sterilants, but their use is discouraged because of the possibility of contaminating irrigation water.

A number of organic materials, including diesel or fuel oil, have been found to have fairly good soil sterilizing properties. These are rendered more effective by the addition of certain oil soluble chemicals such as pentachlorophenol, sodium dinitro orthocresylate, and two-four dinitro butyl or anyl phenol. While such additives are expensive, they add little to the cost of the sterilant since only small quantities (usually less than 2 percent) are required. Fuel oils fortified with 1.75 percent of two of the above chemicals are specified for alternate use under the asphaltic lining to be placed in the Ygnacio canal, California. Application of the petroleum sterilants will be at the rate of 1 gallon per square yard. Areas will be left untreated for comparison and to determine the need for sterilization.

The cost of sterilization with the inorganic materials should not exceed $0.10 per square yard since the cost of material only, if applied at the recommended rates, will be approximately $0.03 per square yard. By comparison, the cost of fortified fuel oils is about $0.15 per gallon and, at the rate of application proposed on Ygnacio, petroleum sterilization will cost at least $0.20 per square yard. However, if, as anticipated, the latter will provide sterilization for a proportionately longer time, the higher cost may be justified.

CANAL CONSTRUCTION EQUIPMENT

As pointed out in the discussion on costs of concrete linings, the greatest possibilities for reducing the cost of canal linings are in the construction operations. It was stated that 63 percent of the cost of a 3-inch reinforced concrete canal lining was attributable to the cost of trimming, placing, and finishing, whereas, only 37 percent was for materials. In general, this relation exists for other types of lining. Yet a reinforced concrete canal lining is very similar, both in materials and requirements for placing, to a reinforced concrete pavement, and it follows that there should be a fairly close similarity between the two types of construction as to the ratio of the cost of placing and the cost of materials. However, the ratio is practically the reverse for pavement construction as approximately 60 percent of the total cost is for materials and the balance, or 40 percent, is for placing. This great difference is largely attributable to two important factors which are more or less interdependent.

The first factor is the characteristic trapezoidal cross-section of canals as compared to the relatively flat surfaces encountered in pavement constructions. This trapezoidal shape and particularly the steep side slope considerably increases the difficulty of placing the concrete, and, what is more important, it precludes the use of relatively simple, inexpensive equipment that might otherwise be adaptable to other uses.

The other reason for the high cost of placing canal lining is the lack of efficient, mass production, standard equipment, such as has been characteristic of pavement construction. This high degree of efficiency in pavement construction has been the result of a sustained, nation-wide highway construction program which created a highly competitive construction field that, in turn, created a pressing demand for efficient, mass production equipment. Because of the experimental nature of many of the canal lining installations and to the comparatively small volume of such work, the same degree of mechanization has not developed for canal lining construction. It is true that in recent years a few enterprising contractors have designed and built special equipment for trimming the subgrade and placing the linings in large canals which has produced very satisfactory results. However, these machines involved a large expenditure and were built for the particular job at hand with no provision or even little possibility of adapting or modifying them for use on other projects, both because of the lack of such work in the immediate future in most cases and also because of the many inherent difficulties of designing such a machine that would be adaptable to canals of various sizes. As a result, the entire cost of this special equipment was amortized in construction costs on the particular project with no material savings in unit cost.

The construction of a lined canal using any one of the better types of canal lining materials, such as portland cement concrete, asphaltic concrete, shotcrete, etc., has usually been accomplished in three more or less separate and distinct steps: rough excavation of canal section, trimming of canal section to exact line and grade, and placing of the lining itself. A few contractors have devised equipment that successfully accomplished the first two steps, excavation and trimming, in one operation but this does not represent the usual practice.

EXCAVATING

The excavation is usually carried only to approximate line and grade, underexcavating 4 to 12 inches on the bottom and side slopes depending on the depth of cut the trimmer is capable of making. The method and equipment used for the excavation has, of course, varied considerably, depending on the material to be excavated, the yardage involved, and the size of canal. On the larger canals, standard earth-moving equipment, such as draglines, shovels, tractors, and scrapers has been used. Smaller canals have usually been excavated with especially designed machinery or with a standard machine or implement that has been modified and adapted to the particular job with varying degrees of success. Some of these latter machines have produced an excavated surface satisfactory for a lining foundation with little or no trimming.

Photograph No. 55 is a rear view of a remodeled dredge used to excavate a canal with a circular bottom on the Gila project. Because the dredge was not capable of making a cut to the full depth of the canal, it was first necessary to make a "V" cut in the ground with a patrol grader. Very little fine hand grading subsequent to excavation was required.

The ditcher pictured in Photograph No. 56 has been used rather widely in maintenance operations and also for the excavation of new canals. The depth of cut is adjustable by hydraulic lifts. The maximum size of canal that can be excavated with this equipment is one having a depth of cut of 3.5 feet, a bottom width of 5 feet and a top width of 16 feet. An excavator somewhat similar in design and principle of operation has been developed by a contractor operating in the vicinity of Phoenix, Arizona. This machine is shown in Photograph No. 57. In a recent field demonstration, this excavator produced a subgrade surface adequate for lining without further trimming.

Photograph No. 58 is a side view of a specially designed and constructed excavating machine used very successfully on the Gila project, Arizona, in 1946. The machine was self-propelled on regular caterpillar treads. The material was excavated from the canal prism by the large revolving cutting wheel and deposited on a transverse conveyor belt operating through the center of the cutting wheel which, in turn, discharges the material onto the canal bank. Depth of cut was adjusted by raising or lowering the cutting wheel by means of a cable system. The wheel was maintained in a level plane by hydraulic controls, making use of the pendulum indicator seen on the conveyor belt in the front view of the machine (Photograph No. 59). In preparation for excavation with this machine, it was necessary to build the ground surface up to a level plane such that the canal section would be all in cut to a fairly constant depth. A little hand trimming was necessary subsequent to excavation.

TRIMMING

Heretofore, trimming of small canals and laterals has usually been accomplished by hand labor after the excavation was completed to approximate line and grade. On the larger canals specially designed equipment has been devised to do the trimming of the subgrade. Most of these machines were very similar in design and principle of operation and the more recent ones were merely improved models of the earlier machines. Almost universally these machines have consisted of a steel truss frame in the shape of the canal section traveling on tracks laid along the canal bank. The material is trimmed from the subgrade, usually by bucket excavators, and deposited on a system of conveyors which discharge onto the banks. These machines have produced satisfactory surfaces for canal lining but are highly specialized pieces of equipment. They are not adaptable to operations on other types of work nor even on canals of different cross-section without extensive alterations. In addition, the laying of the track along the bank is an added expense which makes the use of such equipment almost prohibitive on small canals and laterals which do not involve sufficient yardage of trimming per lineal foot to warrant such an expense. For this reason, the use of caterpillar tracks on trimming equipment for small canals would be a great improvement.

The machine shown in Photograph No. 60 was used on the Balmorhea project and represents probably the simplest one of the trimming machines that operate on tracks. It was pulled forward by a powered winch and a cable anchored ahead of the machine, and was adjustable as to elevation or depth of trim. The diagonal blades shown in Photograph No. 61 shaved the side slopes and pushed the material down into the center of the canal, from where it was shovelled out of the section by hand.

A rather unusual type of trimmer was used on the Coachella canal, All-American project, in southern California (Photograph No. 62). It consisted of a steel frame supported at the four corners on the usual track laid to grade. The machine was adjustable to grade by means of hydraulic jacks at the supports. Forward movement was obtained by winches and anchored cables. However, instead of the usual bucket excavators, a blade near the front of the machine loosened the dirt on the side slopes and bottom and the flat rectangular plates shown attached to the sprocket chains pushed the material up the slopes and onto the bank. There were two separate systems of these blades and chains, each operating across one-half of the bottom and one side slope.

The macine shown in Photograph No. 63 is another departure from the usual system of bucket excavators. This machine was used for trimming operations on the Yakima project. A long revolving drum fitted with scarifier teeth was set at the proper side slope on each side. This revolving drum cut the sides to the proper grade and slope and moved the material to the bottom of the canal where large screws, one on each side,

trimmed the bottom and moved the material into the path of the elevator buckets to be picked up, deposited on the conveyor belt and discharged onto the canal banks. It also was moved forward by winches and cables anchored ahead of the machine.

The trimmer used on the All-American project in Arizona, is representative of the more common type of trimmer except that it had a number of refinements added (Photograph No. 64). It was of the bucket excavator type. Two systems of buckets, one on each side, traveled across the canal section in guides and trimmed the section to proper line and grade. The excavated material was dumped onto the short conveyor belts and discharged onto the banks. Forward motion was by the system of cables and powered winches shown. Depth of cut was controlled by hydraulic jacks at the four corners. The machine was electrically operated from a motor generator set on the deck.

The trimming machine shown in Photograph No. 65 and also the lining machine in Photograph No. 76 gives an indication of the possibilities of manufacturing equipment whose adaptability and easy adjustment make them readily usable on other projects. These two machines were used in the construction of the Contra Costa canal in California. The canal subgrade was trimmed to line and grade by the usual buckets on a single continuous belt traveling down one slope, across the bottom, and up the opposite slope to dump on the usual conveyor belt system. The machine was electrically operated and driven from a portable 50-kw generator. The most unusual and outstanding feature of this machine is the easy adjustment for changes in depth or width of canal section. Adjustment in bottom width of the canal is secured by insertion of a spacer section in the center of the framework. Approximately 1 foot of variation in depth of canal is possible by means of the hydraulic jacks at each corner support (see Photograph No. 66). An additional 10 inches of adjustment in depth can be obtained by a 180° rotation of the swivel arm connecting the wheel assumbly to the trimmer frame. Exact depth of cut is maintained by operating the machine to a taut wire along the canal bank.

LINING

Previous to the last few years, the placing of concrete canal lining by hand was almost a standard practice except on the larger canals where the volume of material involved and the size of the canal warranted the expense of special equipment. In recent years a few machines have been designed and used for the lining of small canals and laterals. This equipment has been somewhat different in principle of operation from the larger lining machines.

The concrete lining machines used on the larger canals have been, almost without exception, very similar in design and operation. The machines consist essentially of a slip-form, supported by a steel or wood framework in the shape of the canal cross-section, traveling on tracks along the canal bank. The machines are usually adjustable in elevation by means of hydraulic jacks at the four corner supports and are maintained at proper grade by means of a taut wire along the banks.

Photograph No. 67 is a rear view of a canal lining machine used on the Yakima project in Washington, in 1940, showing only the basic framework. The steel plates on the bottom of the slip-form were 3.5 feet wide and formed the slip-form that rode on the freshly placed concrete. These plates were constructed with a slight upward slope toward the front of the machine to aid in compacting the concrete. Just forward of the slip-form is shown the open bottom compartment which held the supply of concrete that fed back under the slip-form as the machine advanced. The supply of concrete was replenished by dumping the concrete through a transverse opening in the top deck just forward of the bulkhead seen through the framework. This bulkhead was sloped forward from the top and guided the concrete into the supply compartment. The rod arrangement extending down along the bottom of the supply compartment was a 3-inch rotating rod equipped with numerous, small, steel fingers which worked the concrete to aid it in flowing back under the slip-form and to secure good compaction.

Photograph No. 68 is a front view of a conventional type of liner used on the Coachella canal, All-American project, in southern California. However, it has several added features which were improvements over the older models. It was electrically operated

and capable of self-propulsion as well as being hydraulically raised or lowered. It also had a receiving hopper on the deck capable of holding about 6 cubic yards of concrete. The concrete mix was delivered from transit mixers to this hopper by the belt conveyor shown and was then distributed the full length of the liner by a distribution car traveling under the hopper.

One of the greatest difficulties encountered in placing a reinforced concrete lining is that of maintaining the reinforcing in the center of the concrete slab. For the machine described above, this problem was solved satisfactorily as shown in Photograph No. 69. To keep the reinforcement from rising above the specified depth, a 4-inch pipe bent to the shape of the canal section was welded to the forward side of the concrete compartment at about the reinforcement grade. This pipe is shown directly under the transverse I-beams. The 2-1/2-inch pipe directly below the 4-inch pipe raised the reinforcement to grade as the machine moved forward and the pipe was pushed along under it. This pipe, also bent to the shape of the canal section, was cut at each intersection of the bottom and side slopes and a cable threaded through the three segments. Since the center segment depended entirely on the cable for support, the cable was fastened at both ends to hoists to make it possible to raise or lower the pipe assembly by changing the tension of the cable. To keep the center segment of this lower spacer from working back into the concrete, it was necessary to install a 3/4-inch pull bar on each slope from a projecting member on the machine down to this center segment. This bar is shown along the bottom of Photograph No. 69 just under the reinforcement.

Recently several machines have been developed that satisfactorily place concrete lining in small canals and laterals. Photograph No. 70 is a front view of one type of machine which was used for placing reinforced concrete lining on the Balmorhea project in Texas. It consisted of a frame traveling on tracks along the canal banks and supporting a steel plate slip-form shaped to the finished section of the lined canal. The machine was self-propelled and traveled forward at a rate of about 165 feet per hour. Photograph No. 71 is a view looking down into the concrete compartment. The slip-form was about 5 feet long at the bottom with the leading edge cut square across the canal section on the bottom and sloping back at a 45° angle on the side slopes. The sloping, inverted pan formed the top of the slip-form, and served as the inclined bulkhead employed in larger lining machines. Located just forward of and connected with the slip-form by four angle iron struts was the shield plate which formed the forward part of the concrete compartment. A large vibrator was attached to this shield plate and vibrations were transmitted to the slip-form through the connecting struts to cause the concrete to flow uniformly under the leading edge of the slip-form. However, the method of vibration using an external vibrator did not prove too successful as the machine did not produce a full and uniform slab unless a wet mix was used, with the result that considerable hand-finishing was required to secure a satisfactory lining. Field experiences have indicated that an internal vibrator, such as a vibrating tube, near the leading edge of the slip-form will produce the best results. The thickness of lining placed by this machine was varied by raising or lowering the slip-form by bolt connections to the frame and the proper thickness was secured by operating the machine with the slip-form in contact with the top edge of the 2-inch by 4-inch ribbons along each bank.

The simple slip-form shown in Photographs No. 72 and No. 73 was developed and used on the Gila project in Arizona for placing unreinforced concrete lining. Essentially, it consists of two metal plate slip-forms separated by a concrete compartment open on the bottom over the canal subgrade. The forward slip-form, shaped to fit the excavated section, rides on the subgrade and is allowed to follow any irregularities in the subgrade surface. The rear slip-form is offset upward a distance equal to the thickness of the lining and rides on the surface of the concrete lining formed as the concrete in the compartment flows under the leading edge of the rear slip-form. The concrete is vibrated by two gasoline-motor powered vibrators. (The vibrator shafts are shown in Photograph No. 73 extending into the compartment from each side). Forward motion is by a caterpillar tractor with a bar extension on one side to make the pull on the machine directly over the canal centerline. Such a relatively simple machine as this is only possible on unreinforced canal linings since reinforcing would prevent using a slip-form riding on the subgrade of the canal. Details of this equipment are shown in Figure 5.

A slip-form very similar to the one described above as used on the Gila project was recently developed and used by a contractor of Phoenix, Arizona. This equipment is smaller than the Gila slip-form and differs in the design of the concrete compartment.

Photographs No. 74 and No. 75 show the form in operation pulled by a bulldozer and a top view of the concrete compartment. The openings in the bottom of the concrete compartment, instead of being continuous across the canal perimeter, are divided so that the concrete is placed on the two side slopes simultaneously and just ahead of that in the bottom. Concrete lining 1-1/2 inches thick is currently being placed with this slip-form in farm ditches with 14-inch bottoms for about $1.30 per square yard. The speed of placement appears to be limited as in the case of the Gila slip-form to the rate at which concrete can be supplied. On the job near Phoenix, as much as 1.75 miles of lining have been placed in an 8-hour day. The lining so placed is of satisfactory finish without additional hand work.

The machine shown in Photograph No. 76 was built to place the lining on the Contra Costa canal and operated in conjunction with the canal trimming machine shown in Photograph No. 65. Concrete was dumped into the hoppers on top of the machine and was fed to the concrete compartment along the leading edge of the slip-form by the concrete chutes indicated. The flow of concrete to the concrete compartments was controlled by manually operated gates. This machine, like the trimming machine, was entirely electrically operated from a 20-kw generator on a trailer. Similarly, easy adjustments in bottom width were obtained by insertion of spacer sections in the frame, and adjustments in depth were made by means of the hydraulic jacks and the reversible swivel arms at each corner support. The manufacturer of these two machines has indicated that it should be possible to mount these machines on treads instead of wheels.

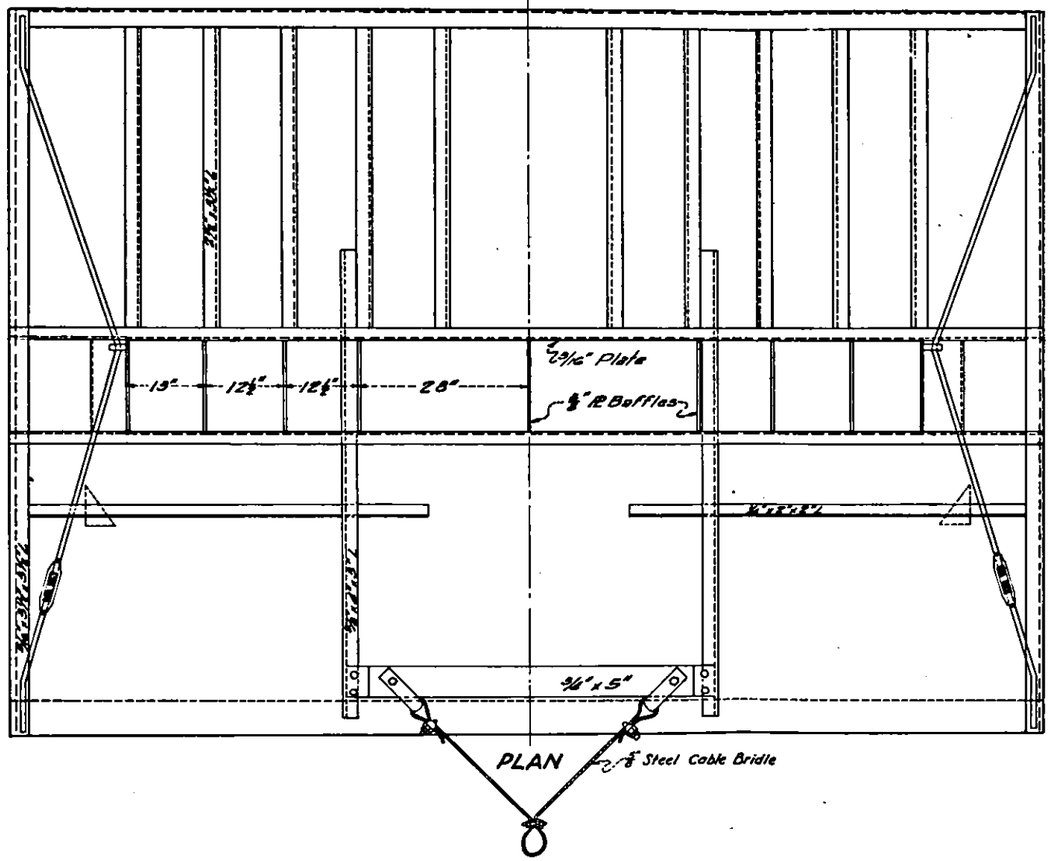
It has been demonstrated in the past that asphaltic concrete can be satisfactorily placed with a machine regularly used for placing portland cement concrete lining. However, a machine manufactured specifically for placing asphaltic concrete was used on the Pasco lateral system in Washington, and after considerable field alteration proved very successful. This machine consisted essentially of a forward sled or guide pan and a strike-off and compaction section separated by an open-bottom, asphaltic concrete distribution area. The guide pan, shaped to fit the canal section, carried a gasoline power unit and cable winch for providing forward motion. The inclined, hinged wings, shown in Photograph No. 77, were intended to trim off any imperfections in the subgrade surface. The asphaltic concrete material was dumped directly on the subgrade in the open center section (Photograph No. 78) and was struck off to the proper uncompacted thickness by the strike-off blades forming the rear bulkhead for this open section. Hot irons were hinged to the rear side of the strike-off blades, one on the bottom and one on each side slope. These hot irons can be seen at the rear edge of the machine in Photograph No. 79. The original machine employed rollers just behind the hot irons to compact the lining. However, because of the scuffing and marking of the lining surface by the rollers, the latter were removed and compaction was obtained by weighting the hot irons. The performance of the machine and the quality of lining produced were very satisfactory. The compaction secured by weighting the hot irons was sufficient to justify elimination of the seal coat except in areas where the lining had been scuffed or damaged.

Both plastic soil-cement and portland cement linings have been placed on the Gila project in Arizona, using the machine shown in Photographs Nos. 72 and 73. Recently, a considerable length of plastic soil-cement was placed on the W. C. Austin project in Oklahoma with a very similar machine. This latter installation was unusual and experimental mainly because it marked the first large-scale installation of plastic soil-cement. A lining machine designed specifically for placing this type of lining was used, and an attempt was made to secure the maximum amount of mechanization and to determine its effect on the cost of the lining. The lining machine, Photographs Nos. 80 and 81, consisted of a forward sled, or guide pan, which carried the power unit and cable winch to provide forward motion by means of cables anchored to a deadman. However, due to a mechanical failure of the cable winch, the machine was pulled forward by a caterpillar for the larger portion of the installation. An open-bottom compartment for receiving and distributing the plastic mix across the canal perimeter separated the forward sled from the slip-form section in the rear. This machine was operated in conjunction with a traveling-plant mixer such as used on highway construction. The traveling-plant mixer picked up the soil material from a windrow which had been formed along the bank with the cement added, mixed the materials and delivered a continuous stream of plastic mix to the mix compartment of the lining machine as the latter moved forward. This operation proved very successful as far as the equipment was concerned except that the lining machine lacked adequate means of distributing the mix across the canal perimeter which

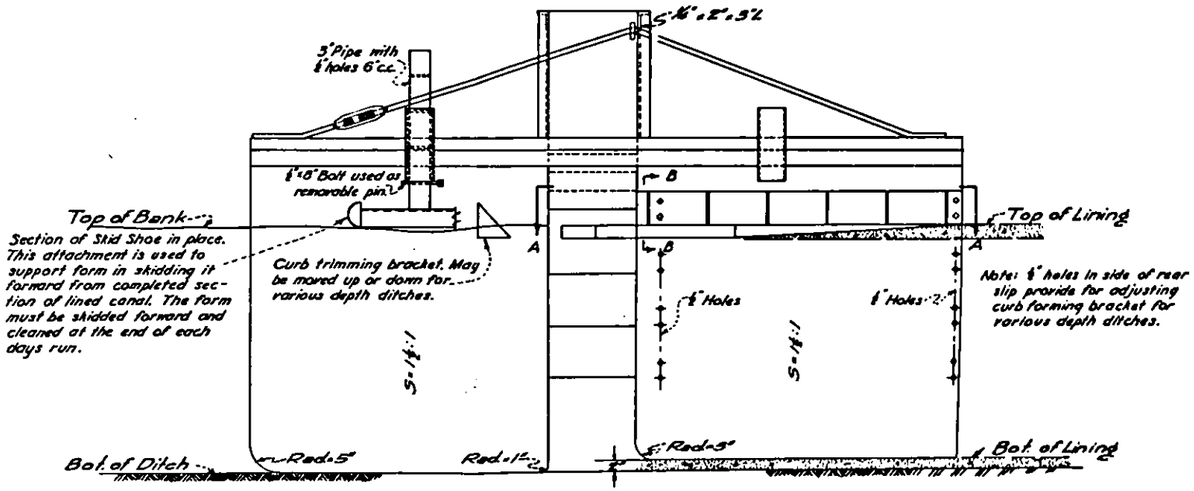
necessitated the use of hand labor, and, after the failure of the winch, it also lacked a means of constant, uniform, forward locomotion. A maximum speed of 351 feet per hour was attained for one an hour run.

Due to the standardization of equipment for shotcrete construction, the variations in construction procedures have been confined to the development of methods for dry-mixing the sand and cement and to the design of hauling equipment for transporting the shotcrete machine and accessory units. On the smaller jobs where only one or two guns have been employed, the usual practice has been to stockpile the aggregate at regular intervals along the canal bank and mix the sand and cement in small concrete mixers. On the larger projects and in particular where several guns have been used simultaneously, it has been found economical to batch and mix the sand and cement at a centrally located plant. This eliminates the necessity of providing separate batching and mixing equipment for each gun.

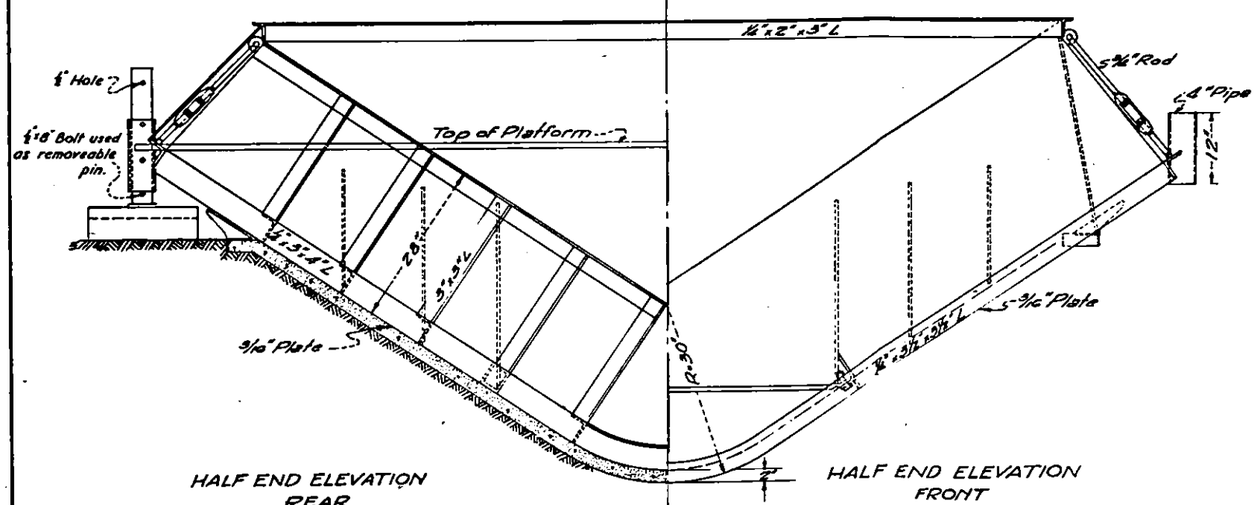
On a number of shotcrete installations the necessary equipment has been mounted on conventional trailers as a single compact portable unit. Photograph No. 82 shows such a unit which was used for lining the canals of the Gila project. The dry ingredients were weight batched in a hopper suspended from a spring scale fastened to a trolley on an overhead beam. The materials were dumped into the mixer skip and raised to the mixer. Subsequent to mixing, the materials were dumped into a rotary drum elevator and lifted onto a 1/2-inch screen from which they fell by gravity into the upper chamber of the gun. (See Photograph No. 83.) The mixer, elevator, gun, and a water supply tank were mounted as a single unit on a trailer. A large truck furnished the motive power for moving the unit and also transported the air compressor.



PLAN

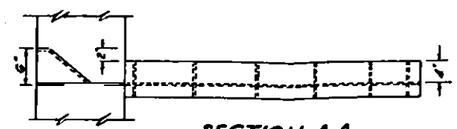


SIDE ELEVATION

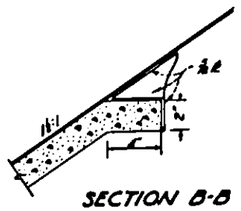


HALF END ELEVATION
REAR

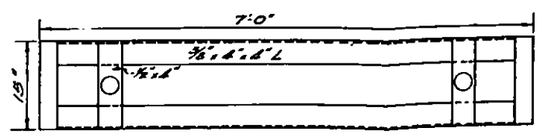
HALF END ELEVATION
FRONT



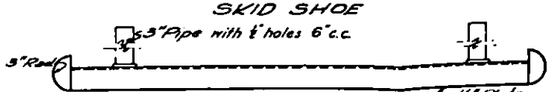
SECTION A-A



SECTION B-B



PLAN
SKID SHOE

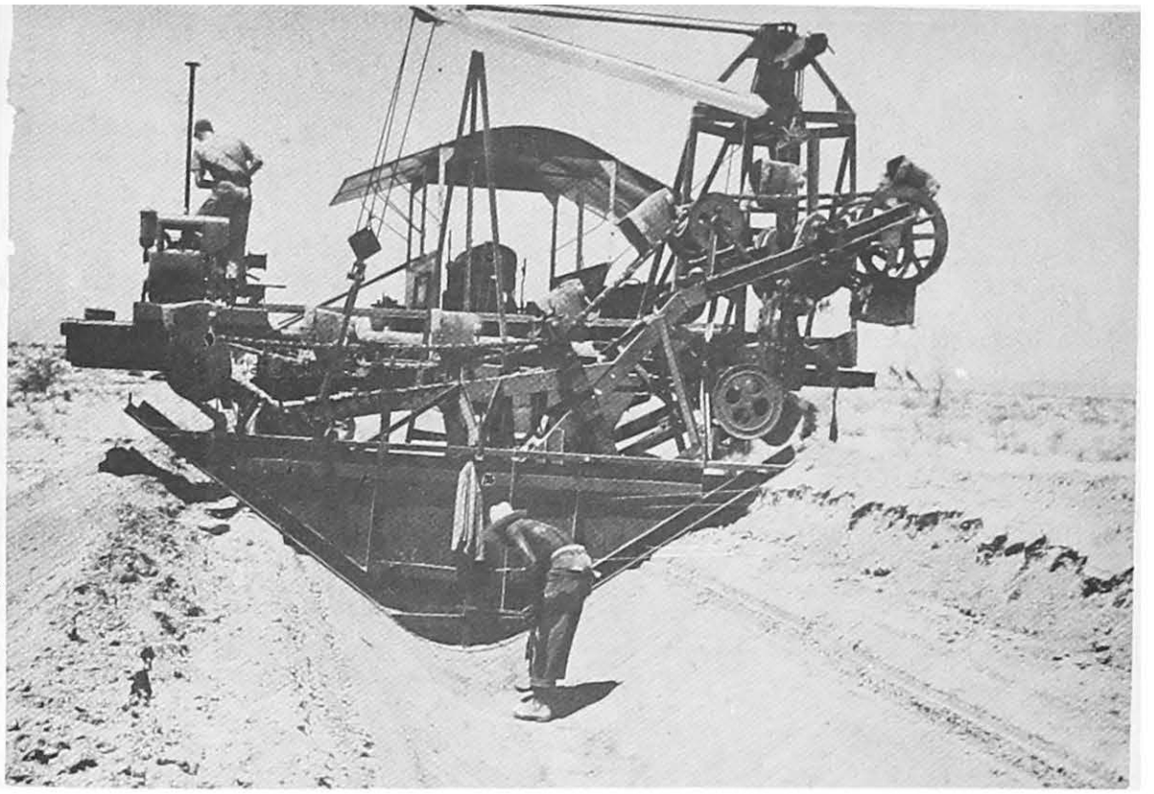


ELEVATION
SKID SHOE

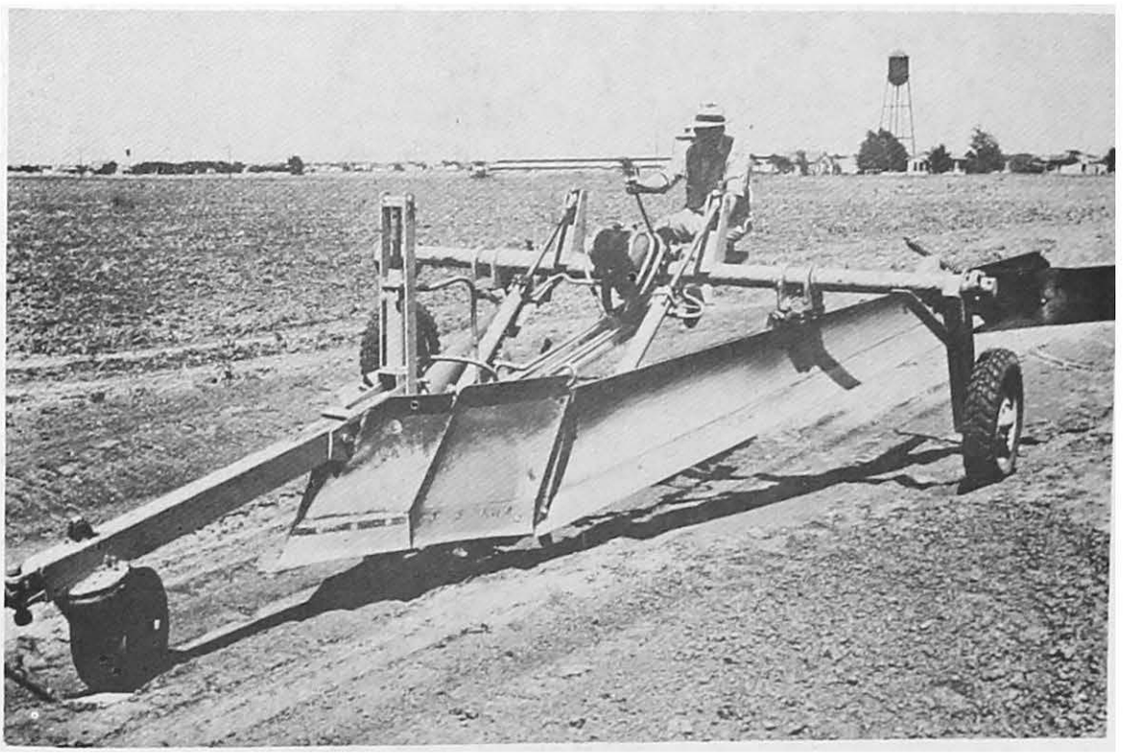
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
GILA PROJECT-ARIZONA
YUMA MESA DIVISION UNIT No. 1

DETAIL OF SLIP FORM

DRAWN ... E.F.B.	SUBMITTED		
TRACED ... A.M.S.	RECOMMENDED		
CHECKED ... R.R.	APPROVED		
G-125-189	YUMA, ARIZONA	JAN. 30, 1948	SHEET 1 OF 1



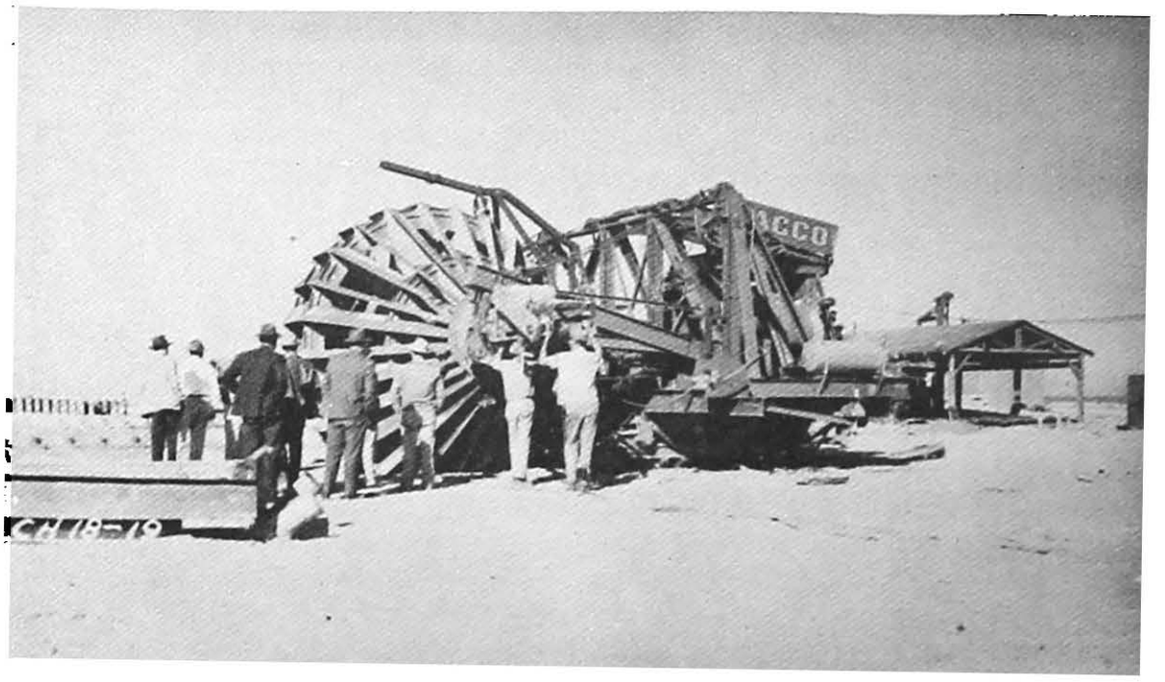
Photograph No. 55
Rear view of converted Ruth Dredger with trimming blade attached. Gila Project, Arizona.



Photograph No. 56

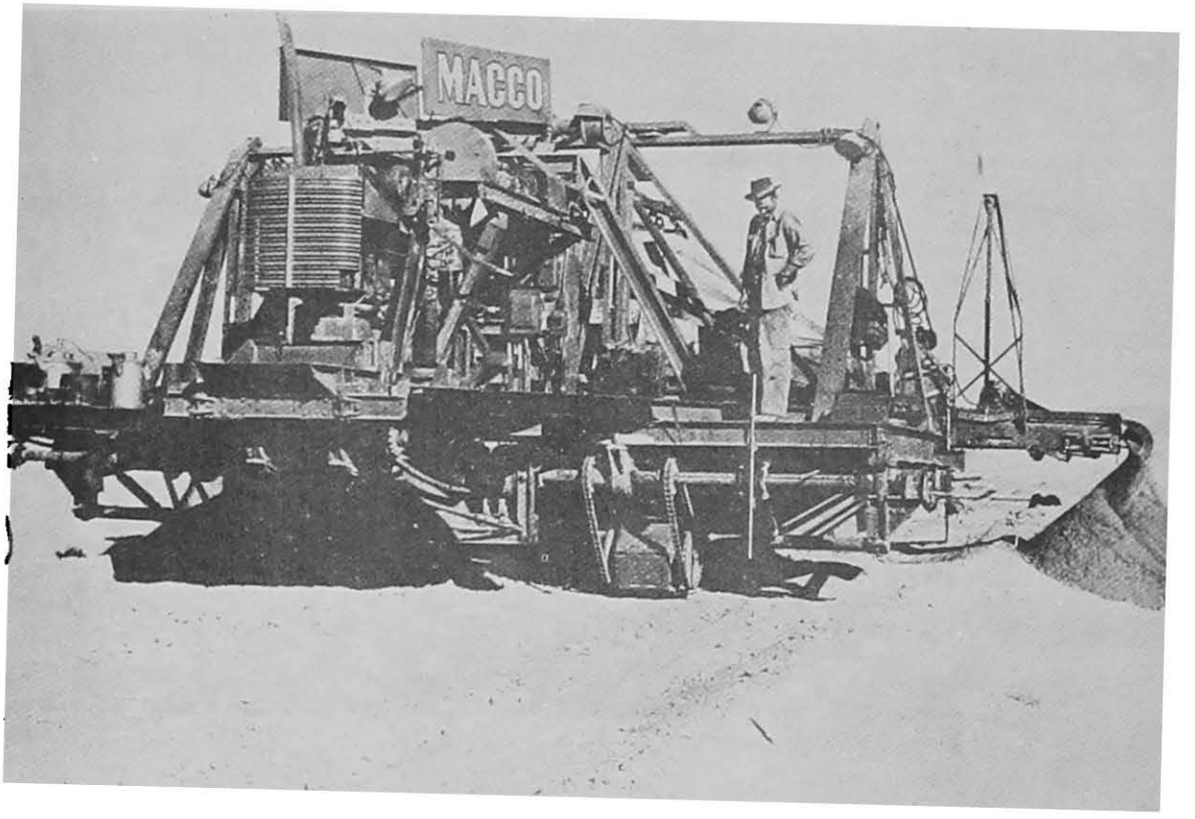
Chattin Ditcher for excavating canals and laterals.

Photograph No. 57
Canal excavator used by a canal lining contractor at Phoenix, Arizona.



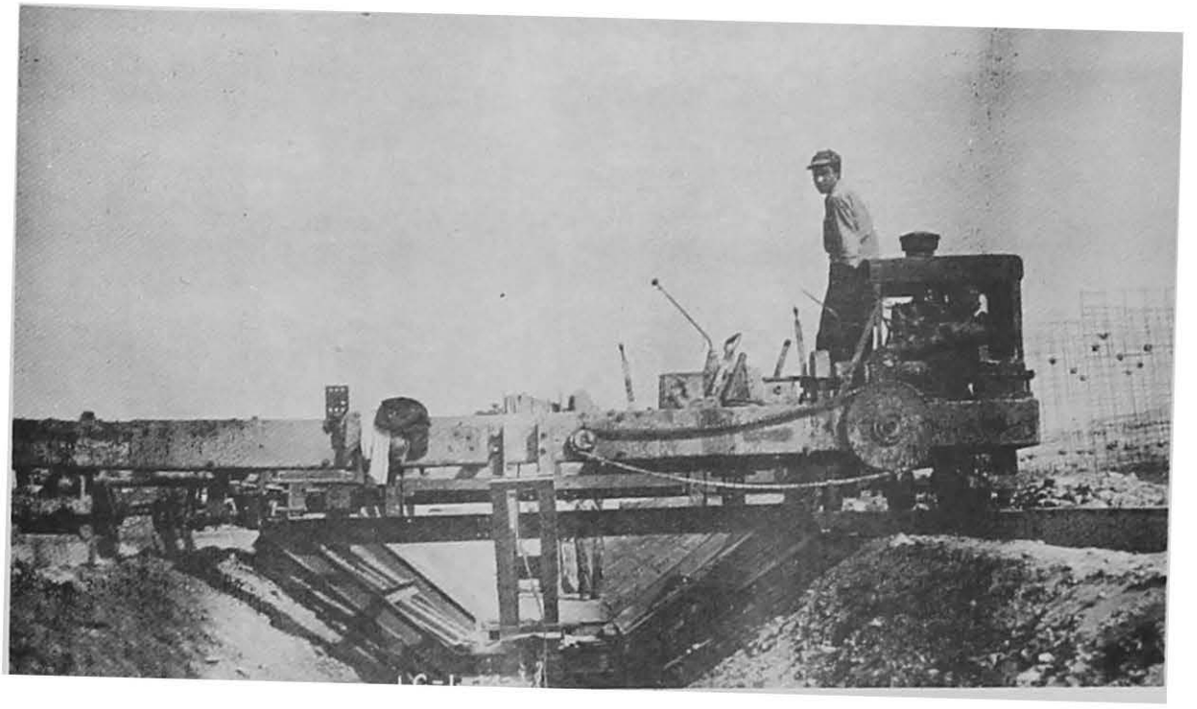
Photograph No. 58

Self-propelled canal excavator--Excavated material was deposited on a transverse belt through center of cutting wheel--Gila Project, Arizona.



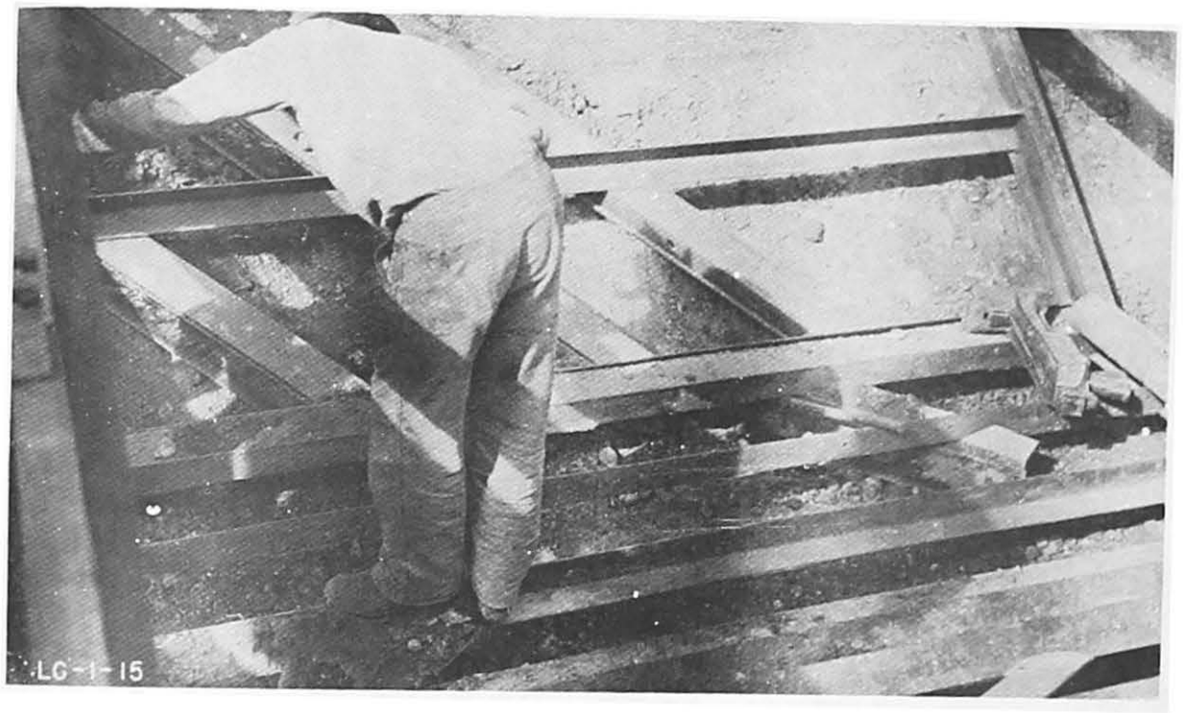
Photograph No. 59

Front view of machine shown in Photograph No. 58--
Note pendulum level indicator on conveyor belt
support at right.



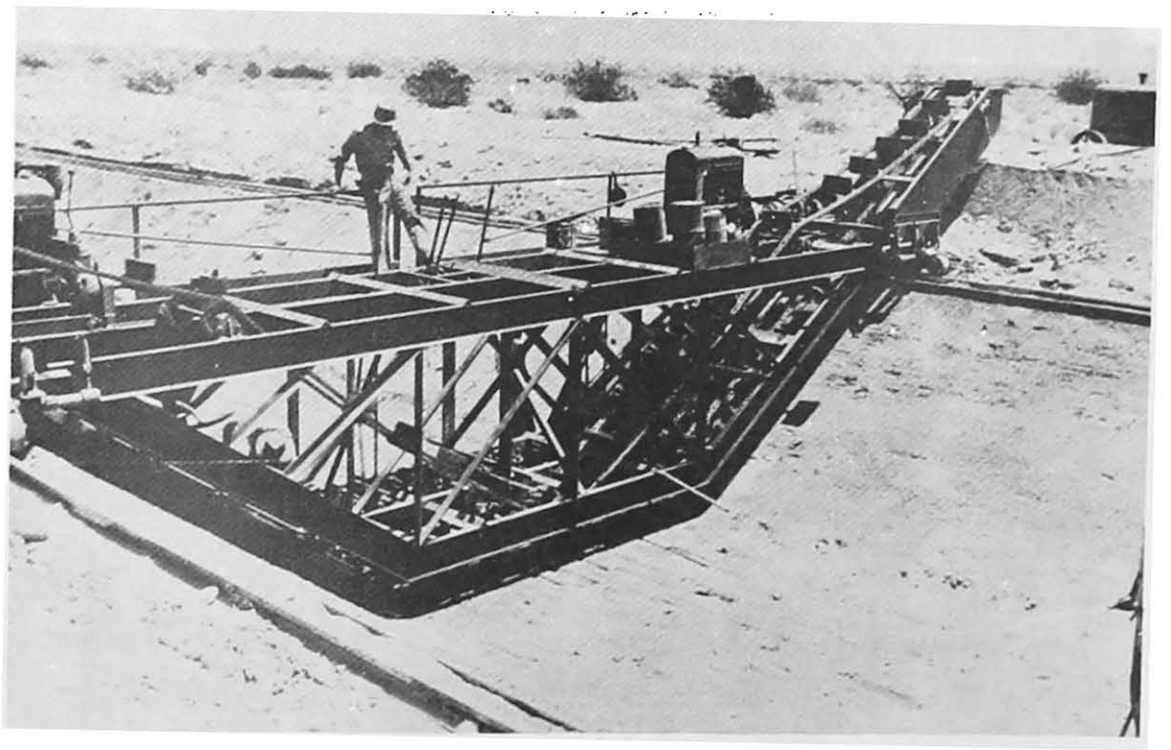
Photograph No. 60

Machine used for trimming subgrade to line and grade
in preparation for placing of lining--Balmorhea
Project, Texas.



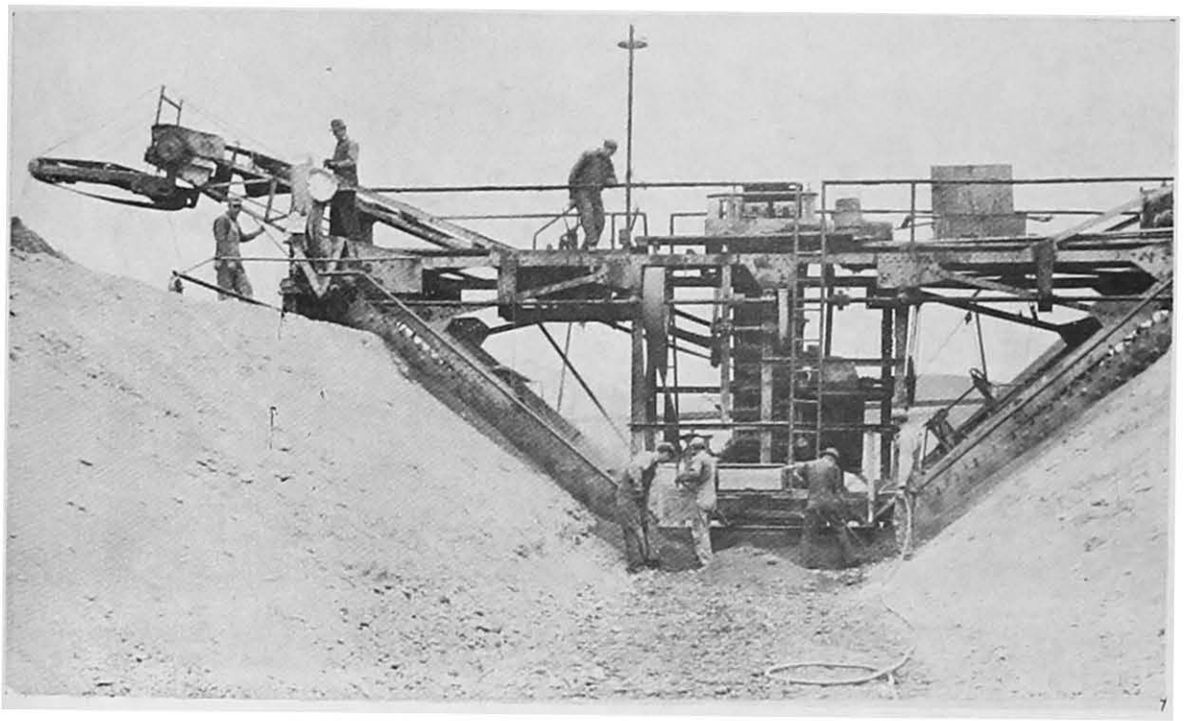
Photograph No. 61

View showing diagonal trimming blades on machine shown in Photograph No. 60



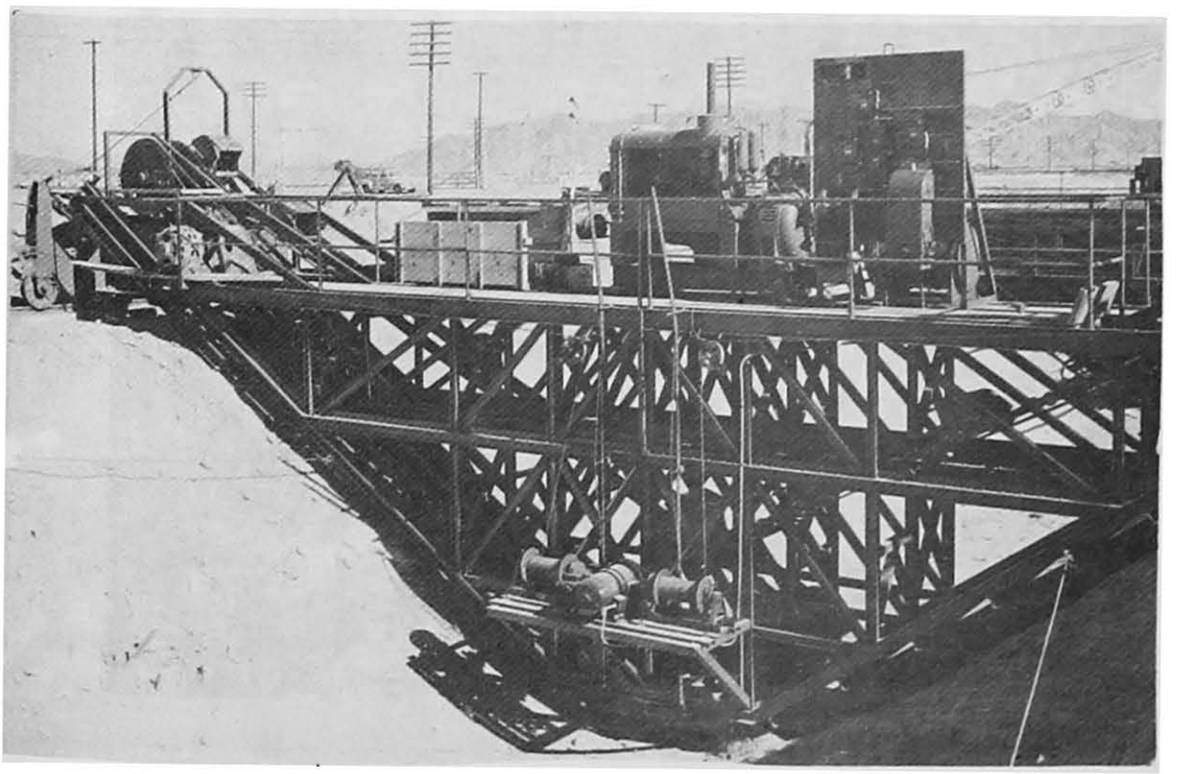
Photograph No. 62

Rear view of trimming machine with trimmed subgrade in foreground--Note rectangular plates on sprocket chain at right which pushed loosened material onto canal banks--Coachella Canal, California.



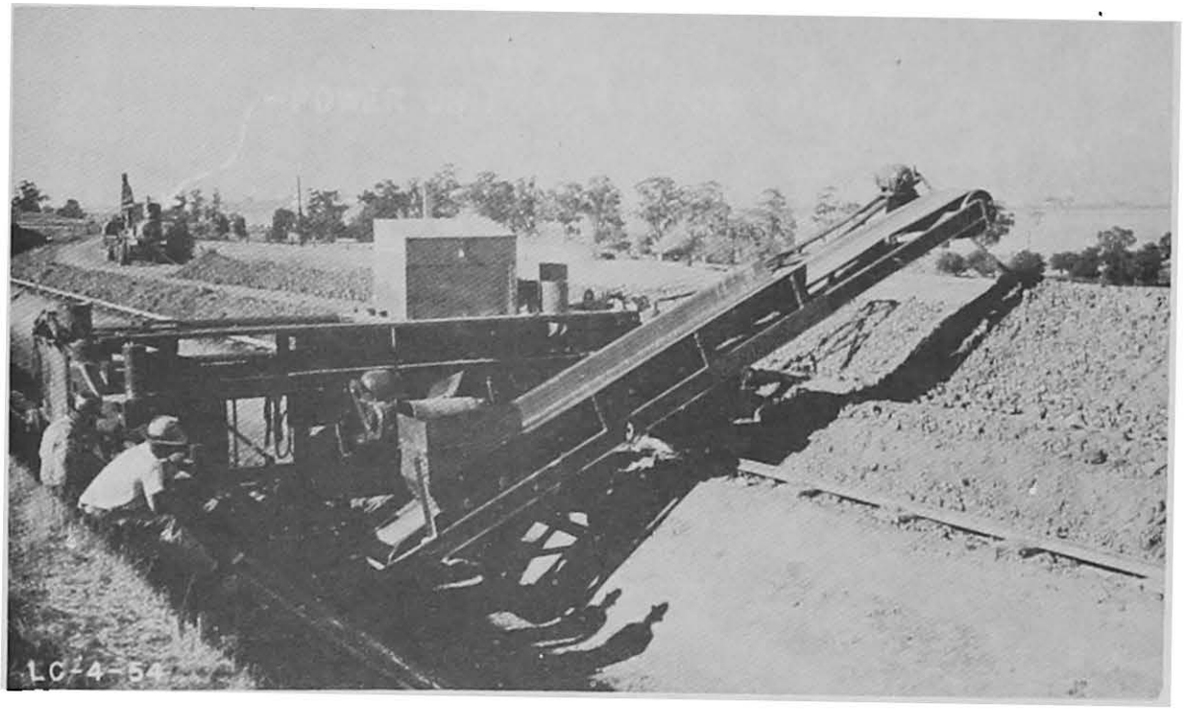
Photograph No. 63

Trimming machine employing revolving drum with scarifier teeth to push material down slope into path of elevator buckets--Yakima Project, Washington.



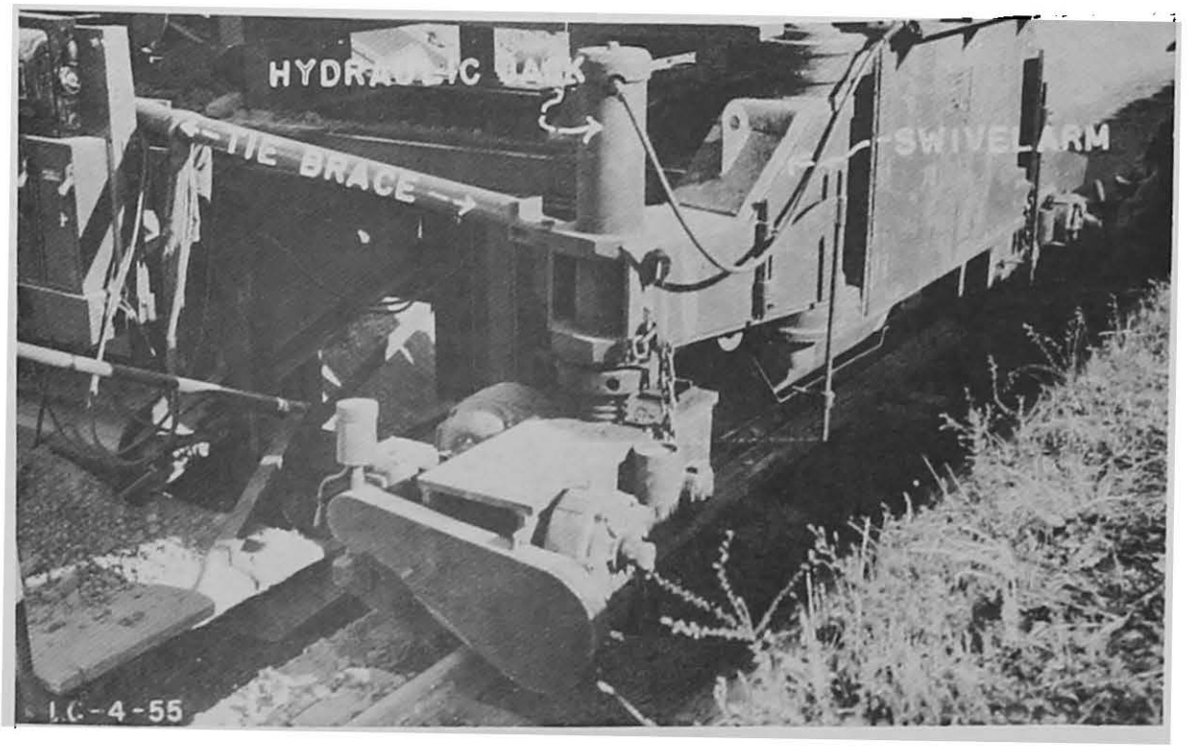
Photograph No. 64

Front view of trimming machine with bucket excavators--All electrically operated with forward motion by means of winches and cables shown--All-American Project, Arizona.



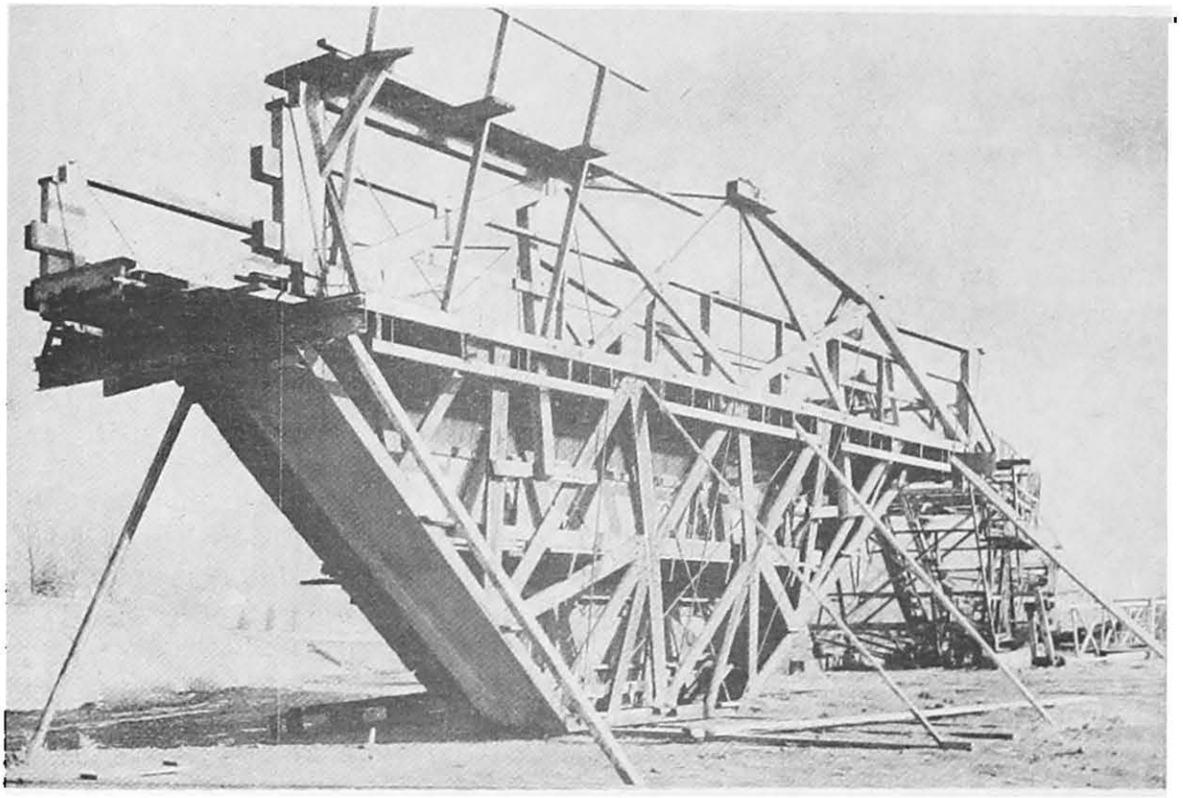
Photograph No. 65

Relatively simple, all electrically operated trimming machine with approximately 22 inches of adjustment in depth and 12 inches adjustment in base width--Contra Costa Canal, California.



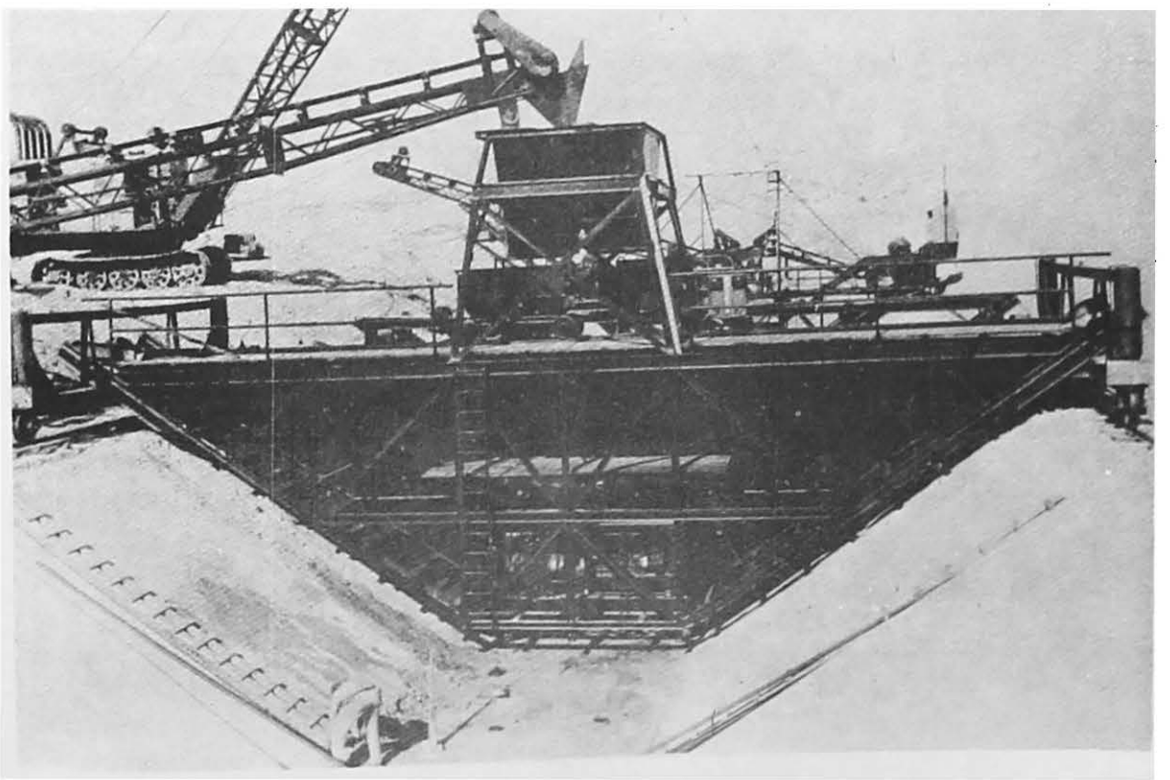
Photograph No. 66

Method of securing adjustment in depth on trimming machine shown in Photograph No. 65. Hydraulic jack permits about 1 foot of adjustment and approximately 10 inches of additional adjustment can be obtained by revolving swivel arm about its horizontal axis--Contra Costa Canal, California.



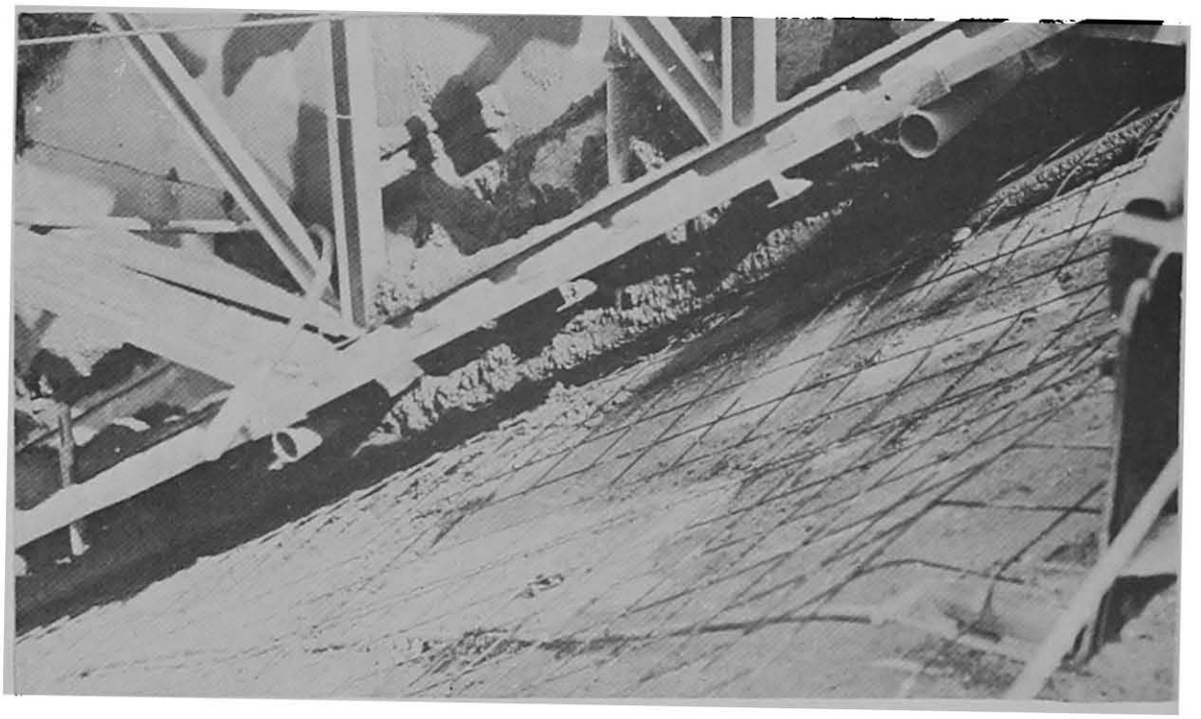
Photograph No. 67

Rear view of basic framework of concrete canal lining machine showing steel plate slip-form and open-bottom concrete compartment at leading edge of slip-form. Note wheel assembly just forward of lining machine--Yakima Project, Washington.



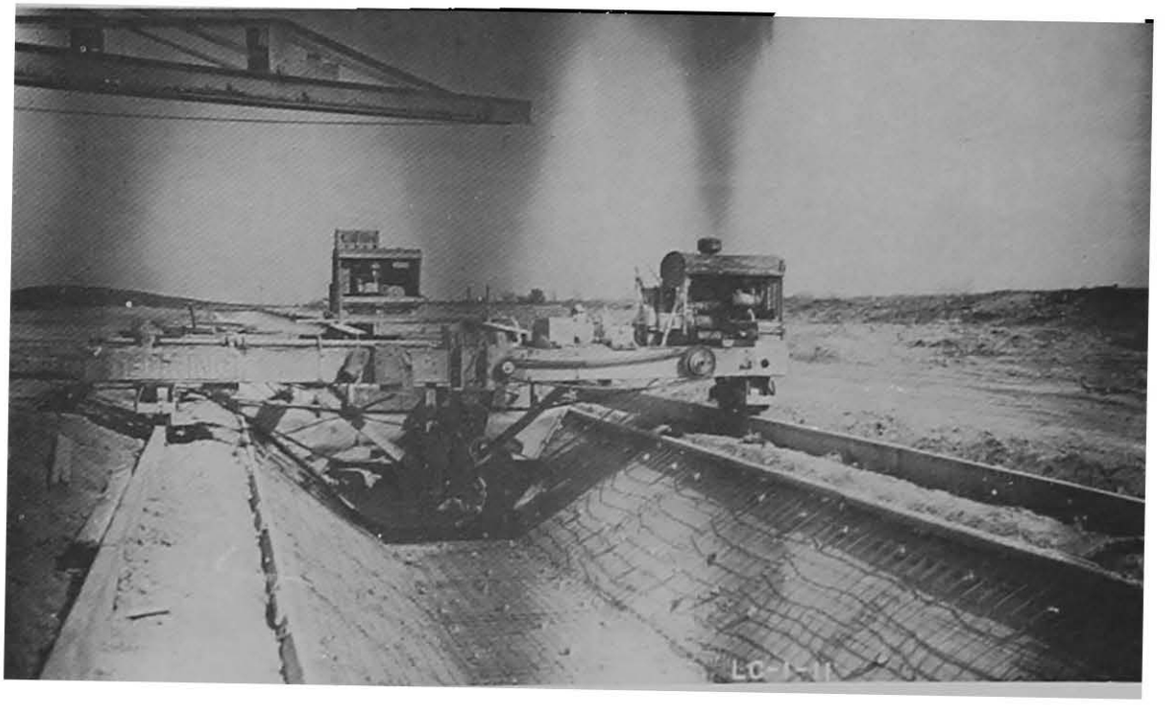
Photograph No. 68

All electrically operated lining machine with conveyor belt for delivering concrete mix to distribution car--Coachella Canal, California.



Photograph No. 69

Method of maintaining reinforcement in center of concrete slab placed by lining machine--Pipe under reinforcement at leading edge of concrete compartment raises reinforcement to proper height--Rod in foreground prevents pipe moving back under machine--Coachella Canal, California.



Photograph No. 70

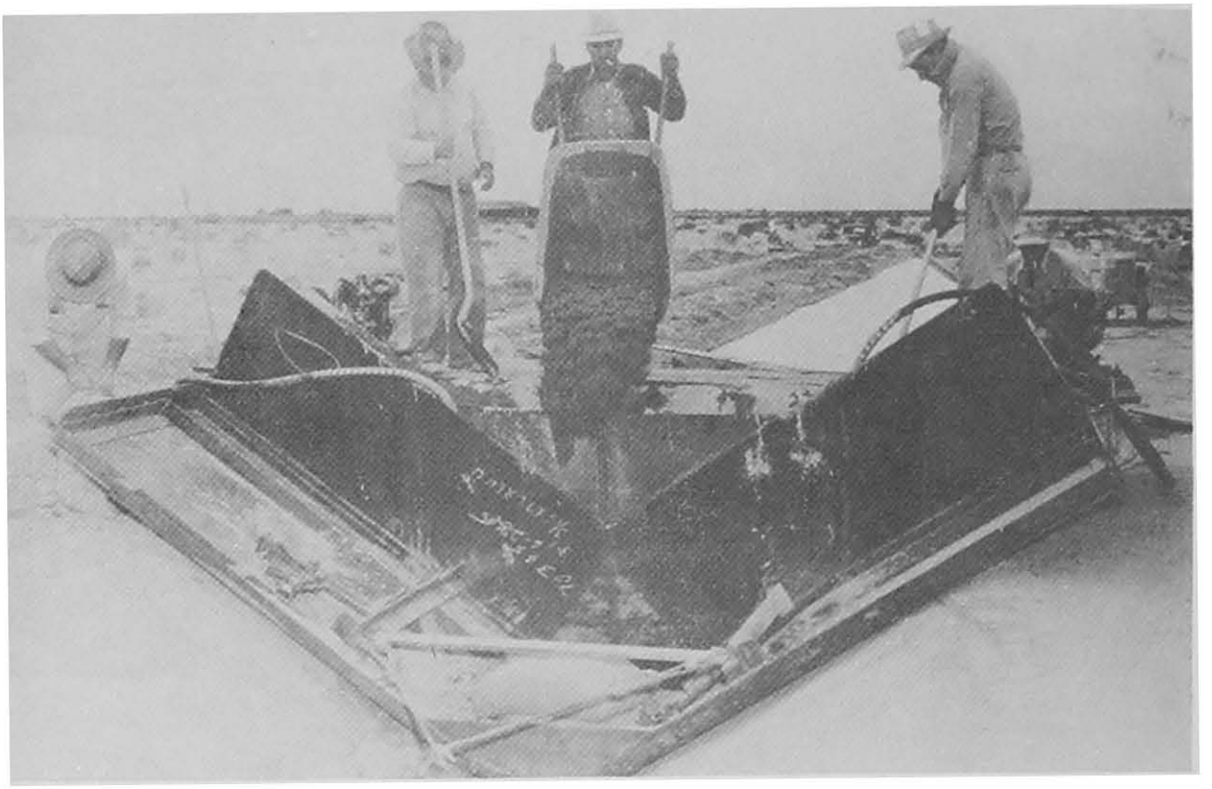
Simple concrete canal lining machine--Balmorhea Project, Texas.

Photograph No. 71

View looking down into concrete compartment of machine shown in Photograph No. 70--Balmorhea Project, Texas.

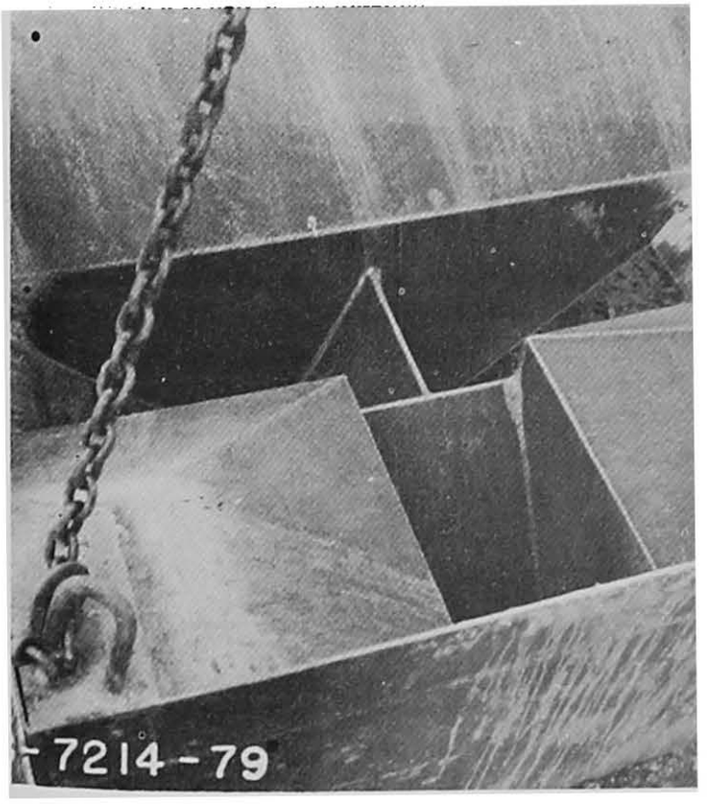
Photograph No. 72

Rear view of slip-form machine for placing unreinforced concrete lining--Forward motion by tractor in background--Gila Project, Arizona.



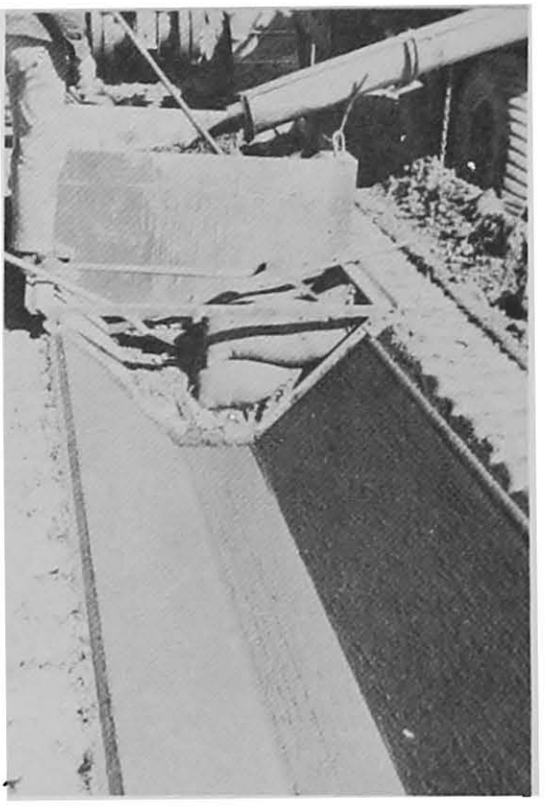
Photograph No. 73

Front view of machine shown in Photograph No. 72 showing forward slip-form which rides on trimmed subgrade- Gila-Project, Arizona.



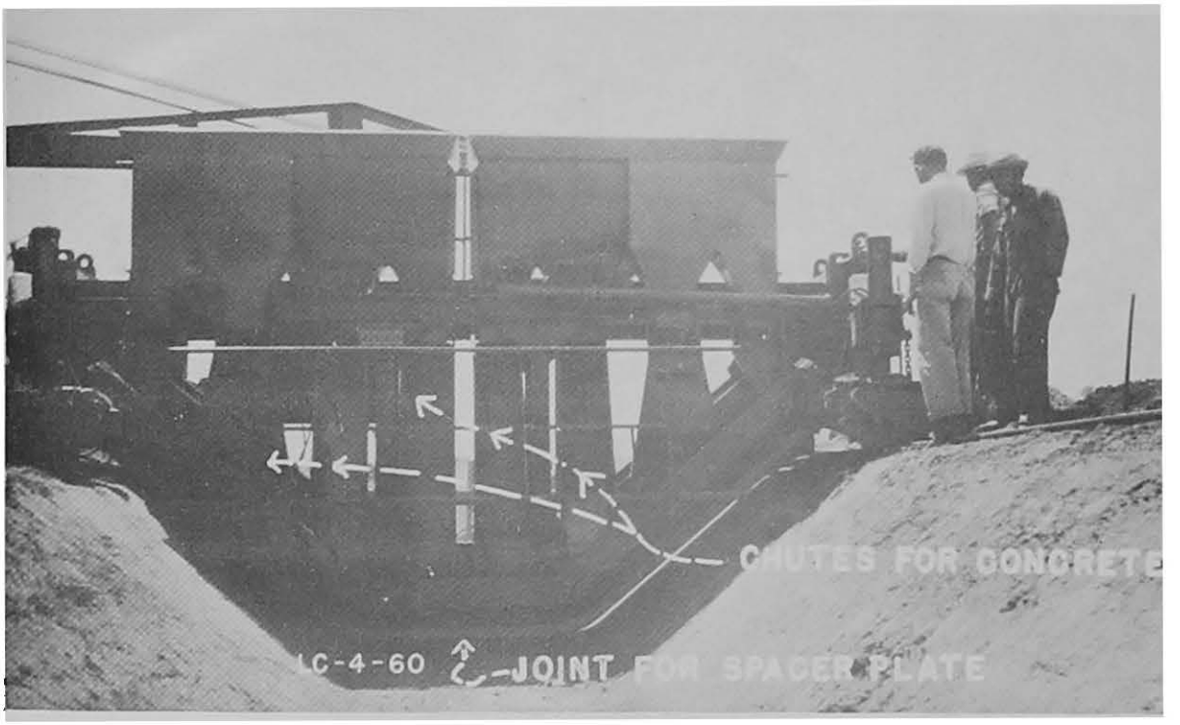
Photograph No. 74

Top view of Fullerform showing concrete compartment.



Photograph No. 75

Rear view of Fullerform in operation.



Photograph No. 76

Concrete lining machine used in conjunction with trimmer shown in Photograph No. 65. Both machines have a similar arrangement for adjustment in canal base width and depth--Contra Costa Canal, California.

Photograph No. 77

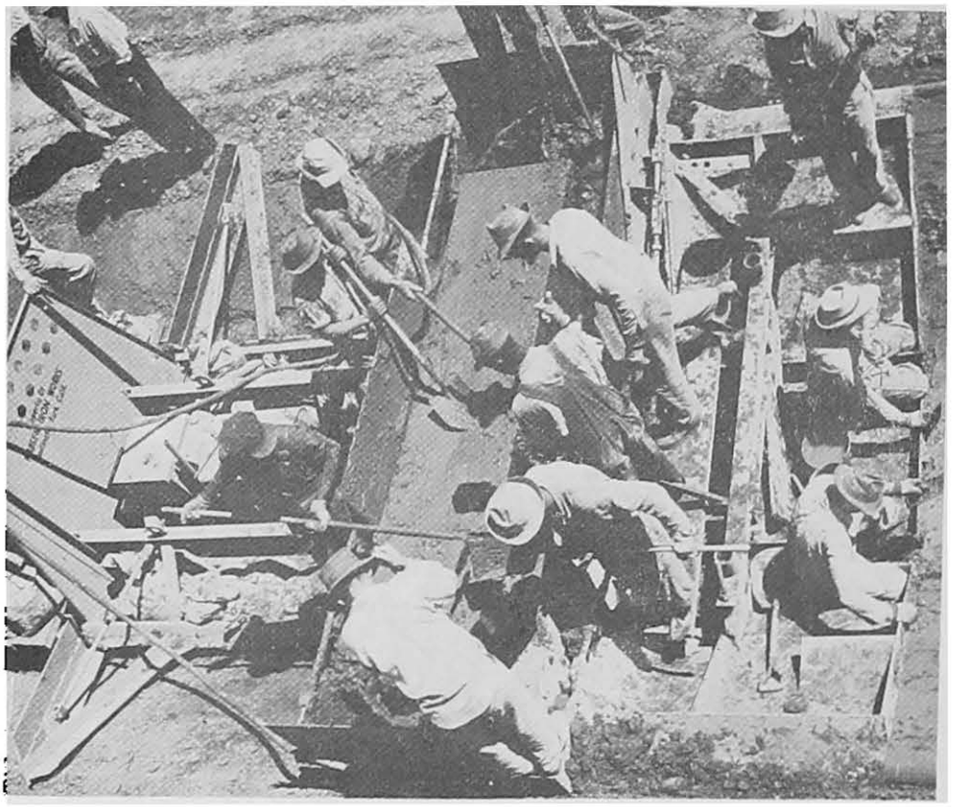
Front view of Ekenstam Canal Paver for placing asphaltic concrete lining--Finished lining in background--Forward pan rides on trimmed subgrade and supports gasoline-motor-driven winch--Pasco Pump Laterals, Washington.

Photograph No. 78

Looking down into asphalt distribution area, of asphaltic concrete lining machine, strike-off blade to rear--Pasco Pump Laterals, Washington.

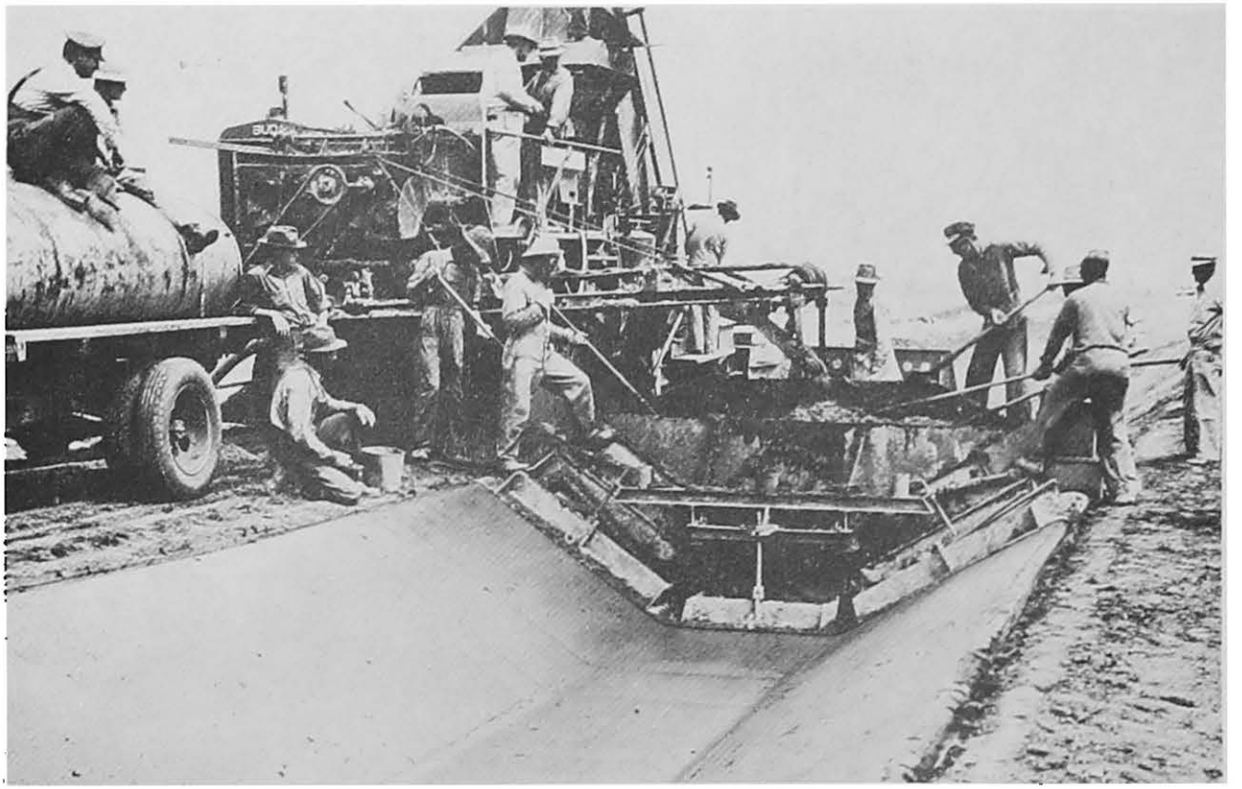
Photograph No. 79

Rear view of asphaltic concrete lining machine showing method of weighting the machine to secure compaction and hot irons at rear edge of machine for compacting and finishing lining--Pasco Pump Laterals, Washington.



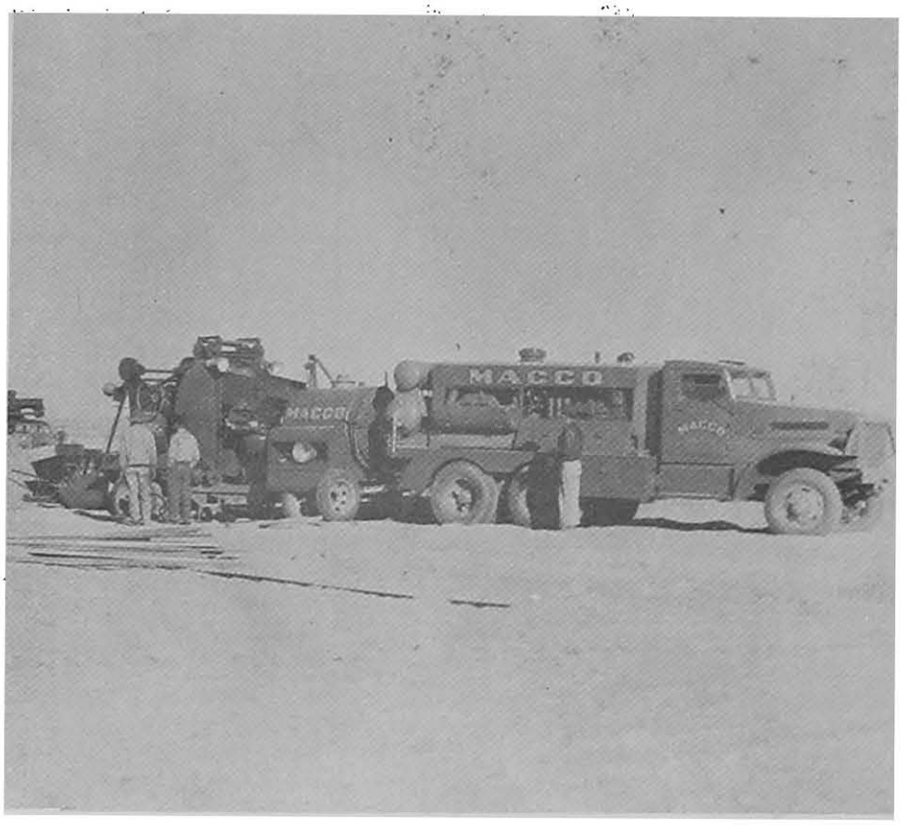
Photograph No. 80

View looking down at lining machine for placing soil cement lining showing forward guide pan with winch and motor to left, mix compartment and rear slip form--W. C. Austin Project, Oklahoma.



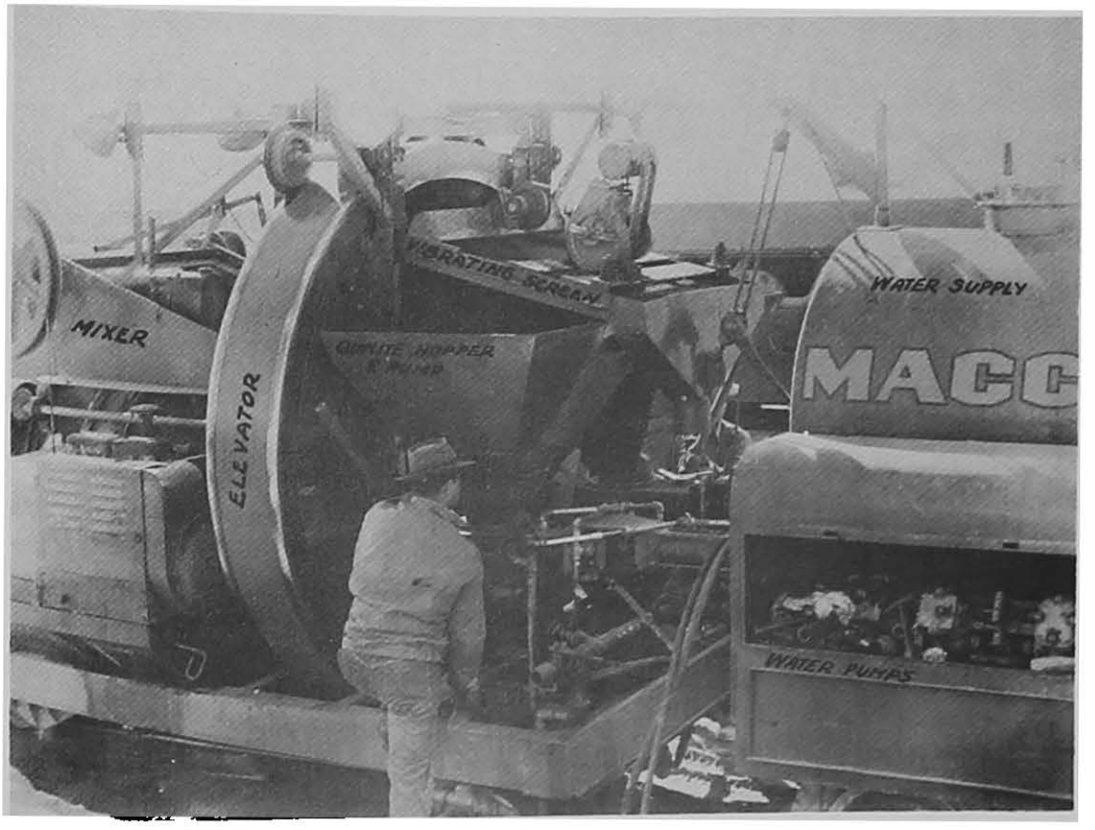
Photograph No. 81

Placing soil cement lining with Barber-Greene traveling-plant mixer and lining machine shown in Photograph No. 80--W. C. Austin Project, Oklahoma.



Photograph No. 82

Mobile equipment for placing shotcrete lining--From left to right: batching equipment on overhead rail, mixer, elevator drum, "gun," water tank, and truck with compressor--Gila Project, Arizona.



Photograph No. 83

Closeup of elevator drum, screens, and "gun" with equipment in Photograph No. 82--Gila Project, Arizona.

DESIGN OF LINED CANAL SECTIONS

The capacity required of a canal is usually determined by the acreage of land to be served, duty of water during periods of maximum demand, and the anticipated transmission losses. Often the canal gradient will be fixed by topographic conditions or by a requirement for maintaining a certain elevation at the lower end so as to irrigate the required acreage. The design of a canal to fulfill these requirements will consist essentially of a selection of the type of cross-section, amount of freeboard, height and top width of the bank, and a determination of the required dimensions of the cross-section. No rigid rule can be established to govern the decision in regard to the above factors, and the experience and judgment of the designer must be relied upon to a large extent for an adequate solution.

TYPES OF CROSS-SECTION

The choice of type of cross-section should be based on a consideration of the comparative economy, practicability, and efficiency of the various possible types of cross-sections and of the purpose and location of the canal. The capacity of a canal of a given cross-sectional area depends upon the velocity of the water. The velocity is in turn dependent upon, and varies with, the square root of the hydraulic radius which is the ratio of the cross-sectional area to the wetted perimeter. Thus the hydraulic radius, and the velocity and capacity vary inversely with the wetted perimeter, and, for a given cross-sectional area, the section having the smallest wetted perimeter will have the greatest capacity. From this analysis, a semicircular section is the most efficient. This type of section has had considerable use in the lower Rio Grande Valley of Texas where the type of soil is favorable to its use and where shotcrete and brick are the most common lining materials. Elsewhere, its use has been rather limited because of several inherent disadvantages. In the first place, the sides of a canal can only be as steep as the natural angle of repose of the earth, and few soils will stand unsupported on the steep side slopes required for a semicircular section. Another disadvantage is that with the exception of brick, pneumatically-applied mortar and masonry, the lining materials cannot be placed without the use of costly forms. In addition, the lining on the almost vertical sides of a semicircular canal offers little resistance to unbalanced, overturning pressures behind the lining.

The next most efficient and yet practical type of cross-section is one with straight sides which are tangent to a circular bottom of a radius equal to the depth of water. The decrease in efficiency of this section as compared to the semicircular section depends upon the slope of the straight sides--the flatter the slope, the lower the efficiency. In any event, the slope should never be greater than the angle of repose of the earth nor such as to necessitate the use of forms to place the lining.

A majority of the lined canals in service today are of a trapezoidal cross-section. The principal reason for this is the relative ease of shaping the subgrade and of constructing the lining with straight surfaces as compared to the curved surfaces in a semicircular or circular-bottom section. The trapezoidal section which has the greatest efficiency for a given cross-sectional area, grade and side slope is one in which the hydraulic radius is equal to one-half the depth or in which $b/d = 2 \tan \theta/2$ where θ, b, and d are the angle of the side slopes with the horizontal, the base width and the depth, respectively. For the maximum efficiency with this ratio, the slope of the sides should be 0.578:1. However, this results in a deep, narrow section with excessively steep slopes and for this reason it is usually modified to result in a shallower section with less steep sides.

As compared to a circular-bottom section, a trapezoidal section has somewhat poorer hydraulic properties but the difference is not of great significance. For instance, a concrete-lined canal with a 6-foot bottom radius, a depth of 6 feet, 1-1/2:1 side slopes and a gradient of 0.0008 would have a capacity of 471 cfs and a wetted perimeter of 25.06 feet. A trapezoidal section of the same water area and gradient but having an average b/d ratio for canals of this capacity, as presently designed by the Bureau, would have a capacity of 466 cfs and a wetted perimeter of 25.53 feet. Thus the use of a trapezoidal section would result in an increase in lining area of only 1.9 percent and a slightly smaller capacity. In view of these slight differences and of the considerable importance of the standardization of canal sections as far as practicable to encourage the development of lining equipment, it is believed to be advantageous to continue designing lined canals with

a trapezoidal cross-section. The widespread adoption of a standard circular-bottom section would be difficult to achieve because of its limited use in the past and the difficulty of adapting such a section to present standard canal structures.

The selection of the ratio of base width to the depth of water in a trapezoidal canal depends to a great extent on the location and purpose of the canal. The latter may be either a diversion canal or a distributary lateral. Diversion canals are usually located on side hills where a relatively deep canal not only permits the use of a smaller section with its better hydraulic properties but also results in considerably less excavation for a given cross-sectional area.

A lateral canal is usually located in comparatively level terrain and, in this case also, a deeper section, because of its greater hydraulic efficiency, would require a smaller cross-section with less wetted perimeter and less lining area. Furthermore, if the water area were all in cut, it would result in less excavation. However, it is usually important to maintain the water level in these laterals at a sufficient height above the ground surface to permit the diversion of water to the land adjacent to the canal and, for this reason, the entire water area cannot be below the ground surface. Therefore, laterals are normally designed for a balanced cut-and-fill wherever practicable. Under these conditions, the volume of excavation required is practically independent of the ratio of the base width to the depth. Similarly, the height of water above the ground surface is nearly constant for all sections of a given cross-sectional area, but the volume of water above the natural ground surface will be considerably increased in the shallower canals. Also, a fluctuation in the volume of water carried by the canal will produce a smaller change in water depth in a shallow section. This latter factor may be important as it facilitates diversion and regulation of flow and also offers more protection against overtopping of the banks due to the entrance of unanticipated storm or drain water. A shallow section, though, has drawbacks in that it requires a greater width of right-of-way and results in an increased area of lining.

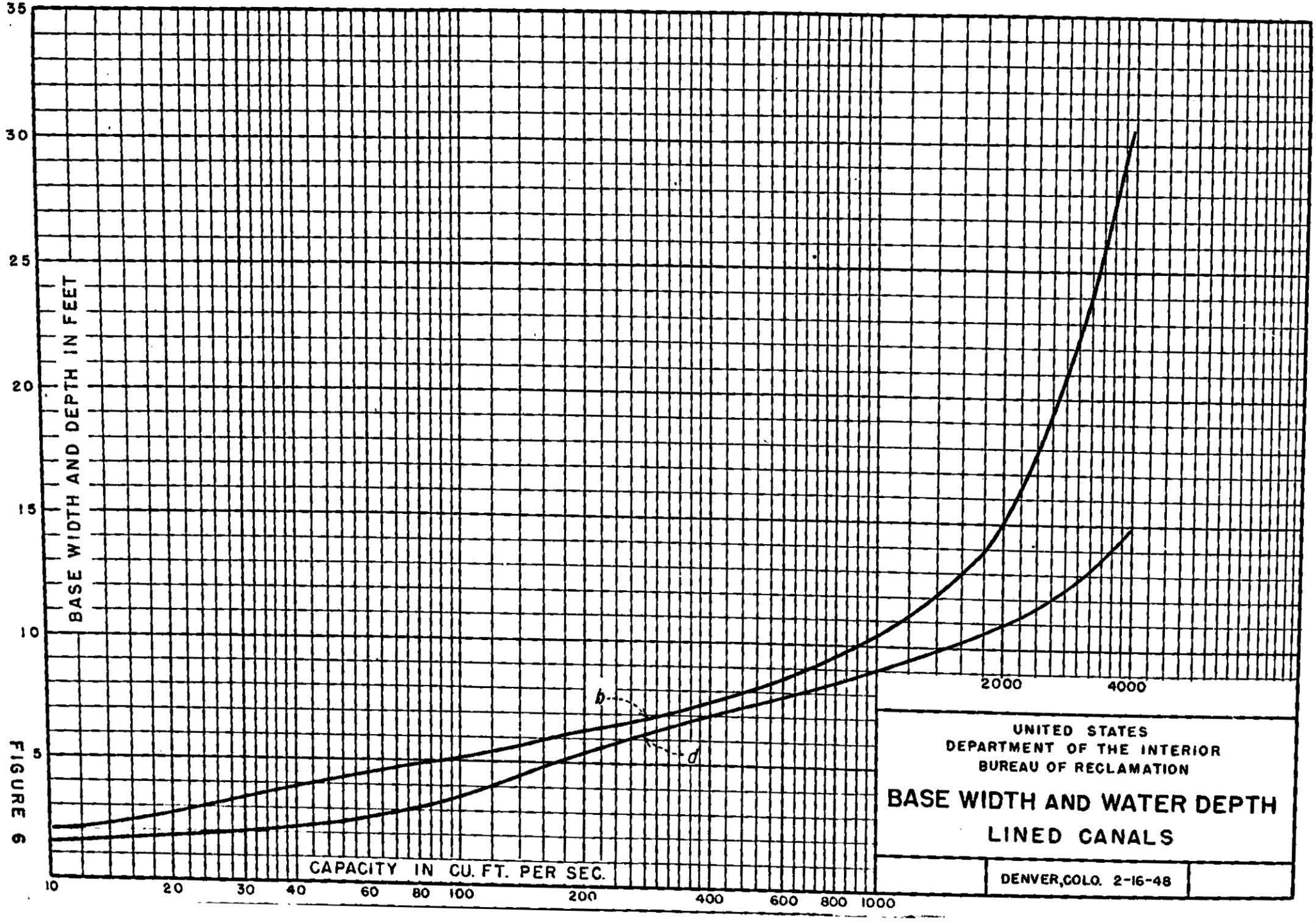
Because of the many variable factors involved in the design of a canal cross-section, the selection of the proper relation of base width to depth will depend largely upon the judgment of the designer. But as a guide in this problem, the base width and depth of a large number of lined canals designed by the Bureau of Reclamation were plotted in relation to the capacity of the canals. Figure 6 was prepared to indicate the average base width and depth for various canal capacities.

PROPOSED STANDARDIZATION

With the increasing mechanization of canal-lining construction, it has become plainly evident that if maximum economy and benefits are to be achieved by the development of mechanized equipment, the canal cross-sections must be standardized so far as practicable. Some equipment manufacturers have evidenced a willingness to specialize in the manufacture of such equipment for rental or sale to contractors, but they have indicated that the development of equipment with sufficient adjustment to accommodate the large variations that have been incorporated in past design is impractical. The large investment required to design and construct such equipment necessitates some assurance that the equipment can be made so as to be usable on future construction. Universal standardization of canal cross-section would provide this assurance and would offer considerable encouragement to equipment development.

The desirability of standardizing lined canal sections has been recognized by the Bureau of Reclamation for a number of years. This problem has been given considerable study in the design and field offices, and as a result standard sections for the smaller canals and laterals have been developed. It is felt that the need and possible benefits of standardization of canal sections is greatest in these smaller canals. The design of the larger canals is usually an individual problem with the size and dimensions of the cross-section being dependent upon local conditions such as topography, subgrade materials, construction problems, etc. For this reason this discussion will be confined to the small canals and laterals not exceeding 6 feet in base width.

It should be recognized in a discussion of the problem that any standards which are developed should have wide application and must, therefore, be adaptable to variations


35
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BASE WIDTH AND DEPTH IN FEET
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CAPACITY IN CU. FT. PER SEC.
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BASE WIDTH AND WATER DEPTH
LINED CANALS
DENVER, COLO. 2-16-48

FIGURE 6

in hydraulic design, types of lining materials, and special field conditions. Since the principal objective is to establish uniform design practices which will result in lower construction costs, the standardization must also offer full opportunity for the most economical and competitive use of available lining materials, methods and equipment. As a consequence of using standard sections, the size of the lined canal will usually be somewhat larger than that actually required. Therefore, if standardization is to be economical the increase in cost due to the use of a larger size of canal must be offset by lower unit costs of construction.

A review of current canal lining design practice shows that the Bureau of Reclamation has, with the exception of a few unusual cases, specified trapezoidal lined canals in the base widths between 2 feet and 6 feet with even increments of change of base width of 1 foot. In the hydraulic design, an "n" factor in Kutter's formula of 0.016 for pneumatically-applied mortar, 0.015 for asphaltic concrete, and 0.014 for concrete has customarily been used. Side slopes have been either 1-1/2:1 or 1-1/4:1. From a standpoint of construction there are objections to the 1-1/4:1 slope in that certain sandy soils are not stable for lining on a slope steeper than 1-1/2:1 and some lining materials cannot be satisfactorily placed on 1-1/4:1 slopes. Due to the improved hydraulic properties, though, there is a saving of about 6 percent in the area of lining required if these sections are designed with 1-1/4:1 side slopes instead of 1-1/2:1. It is questionable, however, if this slight reduction in area of lining would constitute any noticeable cost reduction for a lining project. The theoretical economy of 1-1/4:1 side slopes is probably not sufficient to justify establishing standards for this slope in addition to those required for 1-1/2:1. The Bureau is therefore considering the use of 1-1/2:1 as the standard side slope for the sizes of canals under consideration.

The proposed standard sections for canals of this size are shown in Figure 7. The 2-, 3-, and 4-foot base width canals have been provided with a variation in depth of 1 foot and the 5- and 6-foot base width canals with a variation of 1.20 feet to cover the entire range of flow for canals of these sizes. The average ratio of base width to depth for each base width canal approximates average Bureau practice since the limits were determined from previous canal design. The slopes or canal gradients cover the usual range encountered in canal design.

The possibilities of eliminating one or more of the base widths shown in Figure 7 were carefully investigated. To do so would require a considerably increased variation in depth for each of the remaining sections and would result in canal depths and capacities above the average of present design practices. The elimination of any one of these base widths would probably not result in benefits of any consequence.

Since the depth for each base width of canal as shown in Figure 7 varies through a minimum range of 1 foot, it is not economically practicable to establish one depth for each base width as this would result in an excessive amount of freeboard on the average canal. If equipment can be provided with sufficient adjustment in depth of lining to accommodate the above variations in depth, it would permit the construction of the minimum size of canal cross-section to satisfy the hydraulic requirements. One equipment manufacturer has developed what appears to be a relatively simple method of providing over 1.5 feet of adjustment in depth in a small lining machine, while another manufacturer has stated that equipment with the required adjustment appears to be commercially unsound. Until conclusions on this point can be reached, the standardization, so far as the depths for each base width of canal are concerned, cannot be definitely established.

On the assumption that it may prove economically impractical to manufacture equipment with adjustment to accommodate the entire variation in depth for each base width, it was considered advisable to provide additional standard depth for each base width of canal. In order to minimize the amount of excess freeboard that would result from use of these standard sections for the average canal and yet reduce to a minimum the number of different sections, it was decided that providing three intermediate depths of section for each base width would most satisfactorily meet these requirements. As shown in Figure 7, this results in five different sections for each base width of canal with 0.25-foot increment of change in depth for the 2-, 3-, 4-foot base width and 0.30-foot for the 5- and 6-foot base width.

An economic study was made to determine the increase in the cost of construction of lined canals that would result from the adoption of the standard sections. The standard

sections indicated in Figure 7 were substituted for actual design sections specified for lined canals in several recent Bureau of Reclamation specifications using the same canal gradients. The resultant increase in area of trimming and lining and increased cost of construction using bid prices for the actual work was determined.

The coefficient "n," as used in Kutter's formula for determining flow, will vary from .014 to .016 for materials now commonly employed for lining. It is not planned to alter the range of depths as shown in Figure 7 for this variation; instead, the same ranges will be applied to charts plotted for the different "n" factors. This will not affect the sizes of the canals but will be reflected in a variation in carrying capacity dependent upon the "n" factor used.

The provision of a rounded intersection between the side slopes and the base slab appears to be a desirable feature which could be provided in the standard designs. Since asphaltic concrete linings are usually rounded to an 18-inch radius at this intersection, it would appear reasonable to establish this radius for all standard sections. The effect of this rounded intersection upon the computed hydraulic properties of the trapezoidal section is of negligible importance.

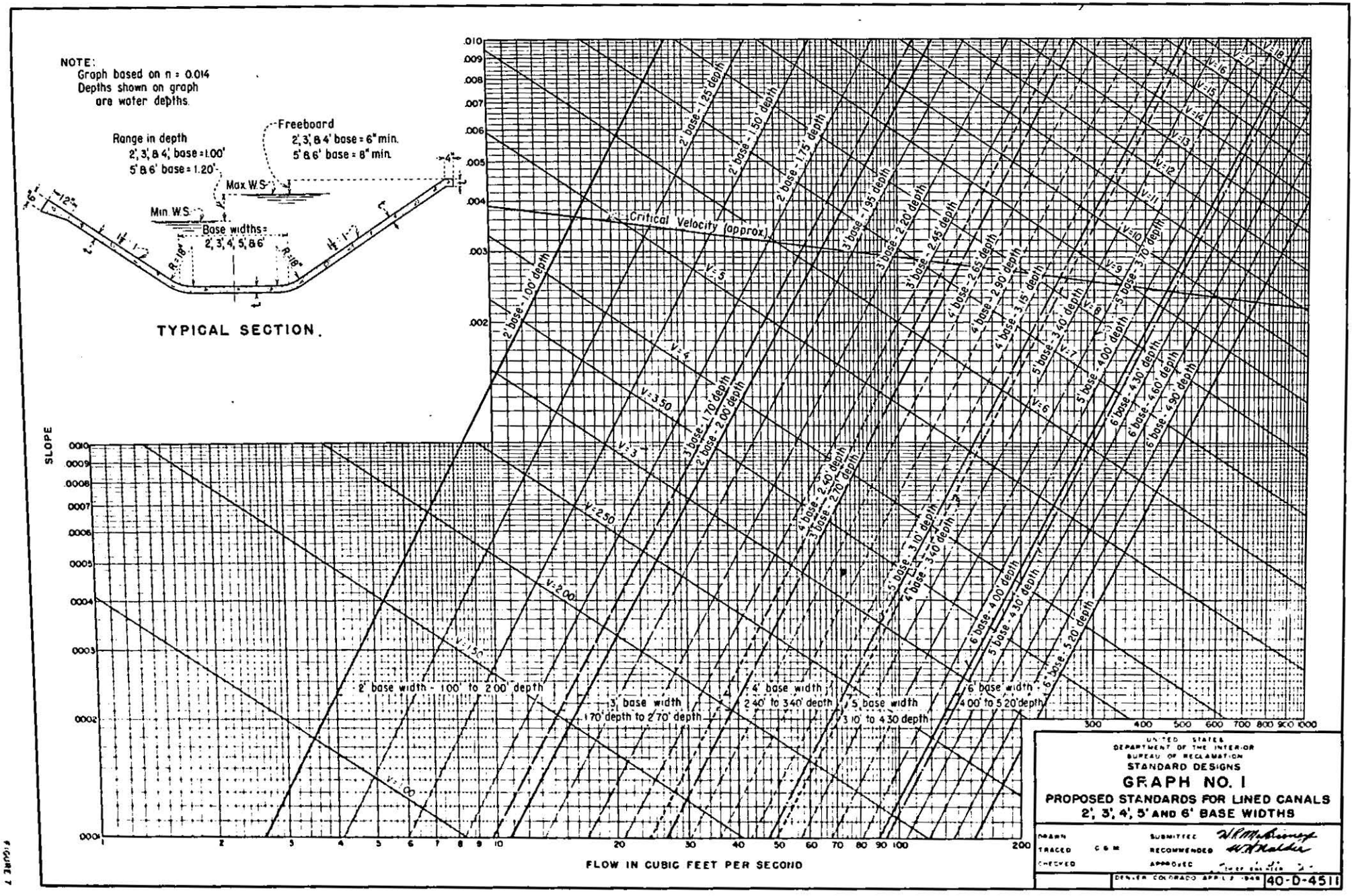
In the design of the upper edge of the lining side slope, it is contemplated that either a 4-inch berm extended horizontally from the top of the lining and of the same thickness as the lining or a thickened lining section in the upper 12 inches of the side slope will be used. Since the 4-inch berm will apply to concrete, pneumatically-applied mortar linings and soil-cement linings, and the thickened edge construction to asphaltic concrete linings, it is not considered that this variation will offer any important problem.

Summarizing the foregoing, it appears that the proposed design standards should provide for 2-, 3-, 4-, 5-, and 6-foot base width canals with 1-1/2:1 side slopes and limiting depths of flow as indicated by Figure 7. The intersection of the side slopes and base will be rounded on an 18-inch radius.

As a result of substituting standard sections as described above for the designed sections of concrete lining provided for the Clayton canal, California, it was found that on the basis of a 2-inch concrete lining for an additional 124 cubic yards of concrete lining and 2,228 square yards of subgrade trimming results from the adoption of standards. This increase amounts to 8.35 percent of the specification quantities. The low bid of $36.00 per cubic yard for concrete and $0.65 per square yard for trimming would have to be reduced to $33.24 and $0.60, respectively, to make the costs of constructing standard sections equal the costs of the sections as provided in the specifications. The lined sections provided in the specification require three different depth sections in the 4-foot base and four different depth sections in the 3-foot base, for a total of seven different sections. The standard sections require three depths in the 3-foot base and one depth section with a 2-foot base. Summarizing, the seven canal sections required by the specifications could be replaced by four standard canal sections with an increase of 8.30 percent in the square yard area of lining.

Following the same procedure for the Contra Costa canal, where a 3-inch lining was specified, the seven canal sections required under the specifications could be replaced with six standard sections. This change increases by 8.19 percent the quantity of concrete and trimming required for the specification canal sections. To accomplish the work at the same cost, using standard sections and the accompanying increase in quantities, the low bid price of $20.00 per cubic yard for concrete lining and $0.75 per square yard for trimming would have to be reduced to $18.48 and $0.69, respectively. It should be noted that in both the Contra Costa and Clayton canals the specifications provide for 1-1/4:1 side slopes, whereas the standard sections which are substituted are designed with 1-1/2:1 side slopes.

To compare the use of standard sections in lieu of designed sections with 1-1/2:1 slopes, a study was made of the specifications for concrete lining on B canal and A and B laterals of the Yuma Mesa division of Gila project, Arizona. The specifications required 28 different depths of canal in a 2-foot base width, 20 different depths in a 3-foot base width, and 3 different depths in a 4-foot base width. In adopting standard sections, three different depths in the 2-foot base, four different depths in the 3-foot base width, and one depth in the 4-foot base width are required. In this manner, 8 standard canal sections replace 51 sections required under the specifications. The area of lining required for the standard sections is increased 3.2 percent over the specification quantities.


NOTE:
Graph based on n = 0.014
Depths shown on graph
are water depths.
Range in depth
2', 3', & 4' base = 1.00'
5' & 6' base = 1.20'
Freeboard
2', 3', & 4' base = 6" min.
5' & 6' base = 8" min.
Max. W.S.
Min. W.S.
Base widths =
2', 3', 4', 5', & 6'
R=18"
TYPICAL SECTION.
Critical Velocity (approx.)
SLOPE
FLOW IN CUBIC FEET PER SECOND
2' base width - 1.00' to 2.00' depth
3' base width
1.70' depth to 2.70' depth
4' base width
2.40' to 3.40' depth
5' base width
3.10' to 4.30' depth
6' base width
4.00' to 5.20' depth
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS
GRAPH NO. 1
PROPOSED STANDARDS FOR LINED CANALS
2', 3', 4', 5' AND 6' BASE WIDTHS
DRAWN
TRACED C & M
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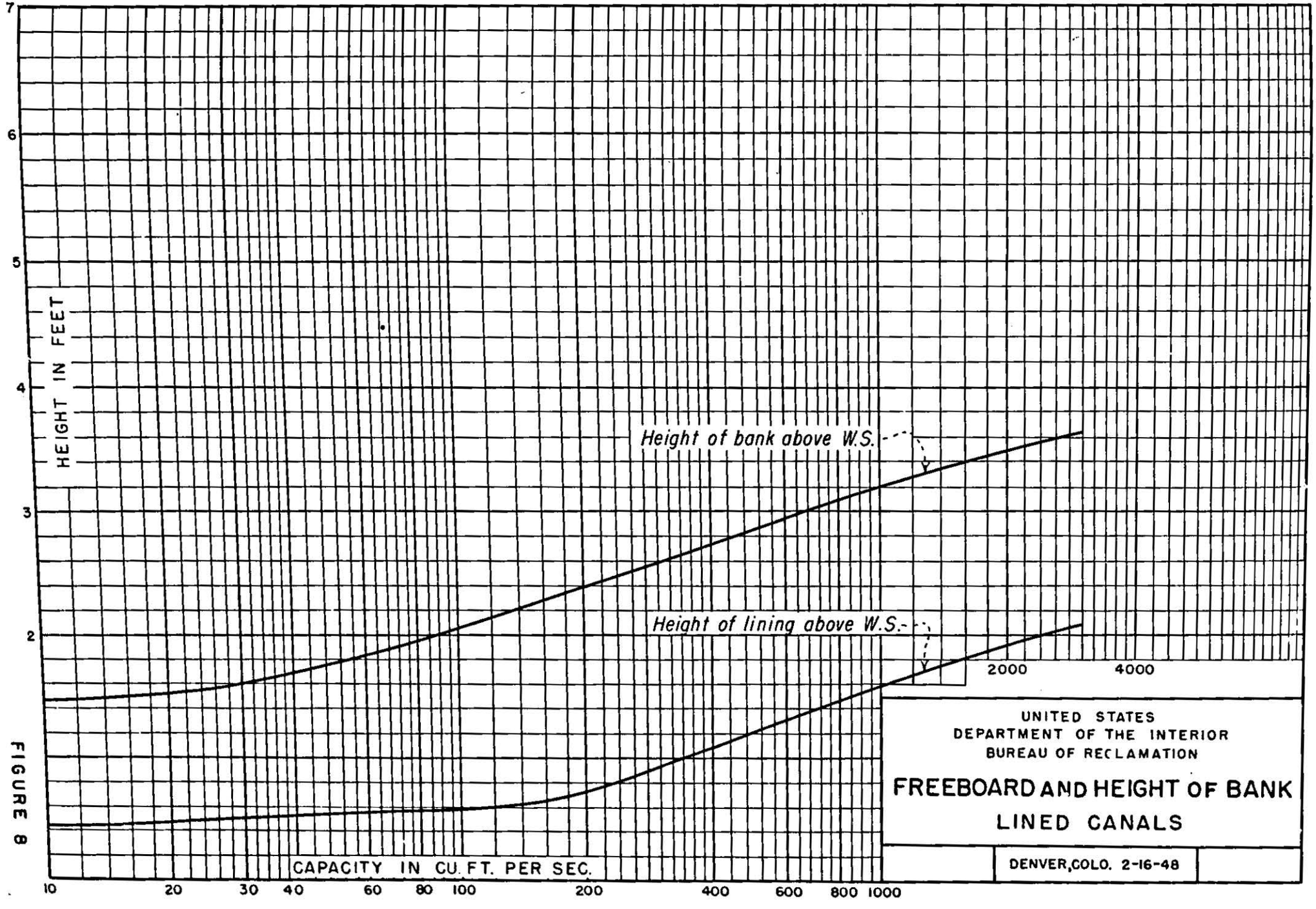
SUBMITTED
RECOMMENDED
APPROVED
DENVER, COLORADO APRIL 2 1948
40-D-4511

FIGURE 7



UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

FREEBOARD AND HEIGHT OF BANK LINED CANALS

DENVER, COLO. 2-16-48

FIGURE 8

A similar study was made of another specification for concrete lining of the A and B canals and laterals on the Yuma Mesa division of the Gila project. The specification requires 17 different depths of 5-foot base width canal on 1-1/2:1 side slopes. The standards require two depths in a 4-foot base width, three depths in the 5-foot base width, and three depths in the 6-foot base. A total of 8 standard sections are therefore required to replace 17 sections required in the specifications. The area of lining required is increased by 2.08 percent through use of the standard sections.

In reviewing the examples which have been discussed, it will be noted that the use of standard sections with 1-1/2:1 side slopes, when used to replace designed sections with 1-1/4:1 slopes, will result in a total increase of 8-1/2 percent in the area of lining required. Since detailed comparisons indicate that about a 6-percent increase in area can be attributed to the conversion from 1-1/4:1 slopes to 1-1/2:1 slopes; the balance or only 2-1/2-percent increase should be attributed to the increased freeboard which results from using standard depths. In those examples where the specifications provided sections with 1-1/2:1 slopes, the total increase in area of lining from standardization averaged about 2-1/2 percent.

In considering the foregoing substitution of standard sections for design sections, it should be noted that the canal gradients provided in the specifications did not permit the most efficient use of the standard sections. If in the original layout of these lined canal systems use could have been made of Figure 7, adjustment in canal gradients could probably have been made to provide more efficient use of the standard sections. With this further factor to be considered, it appears that any increased area of lining as a result of using the proposed standards will be of very small consideration.

FREEBOARD, TOP WIDTH AND BERM

The height of lining above the water surface in lined canals will depend upon a number of factors, such as size of canal, velocity of water, curvature of alinement, probability and amount of storm or drain water entering the canal, fluctuation in water level due to operation of checks and turnouts, and on wind action. In a somewhat similar manner, the height of bank above the water surface will vary with size and location of canal, type of soil, amount of intercepted storm or drain water, etc. Figure 8 indicates the average freeboard and average height of banks in relation to capacities for lined canals designed by the Bureau of Reclamation.

The top width of banks varies from 2 to 16 feet, depending primarily on the size of the canal. If the top of the bank is to serve as a roadway, the width should not be less than 12 feet. Below is shown a table of recommended minimum top widths for small canals where the bank is not to serve as a road:

Capacity (cfs)	Width of bank
1-9	2 feet to 3 feet
10-24	3 feet to 4 feet
25-49	4 feet to 5 feet
50-99	5 feet to 6 feet
100-200	6 feet to 7 feet
200-400	7 feet to 8 feet
400-800	8 feet to 9 feet

The usual practice of the Bureau of Reclamation is to provide a berm along each bank at the top of the lining. The purpose of this berm is to provide a space for the operation of construction equipment such as lining and trimming machines, to receive the material which may wash or slide down from the banks above, and to facilitate maintenance operation. The width will vary from about 2 to 6 feet, depending, of course, on the size of canal, but being determined primarily by the space required for the operation of construction equipment. In some cases, this berm is backfilled to a slope of about 4:1 subsequent to the placing of the lining. This serves to drain intercepted water into the canal and to prevent its entering the subgrade behind the lining and causing serious hydrostatic pressures.

SUBGRADE PREPARATION

A primary prerequisite to the success of any of the rigid types of canal linings is a firm foundation which will eliminate, as far as possible, the danger of cracking or failure due to settlement of the subgrade. The more or less flexible types of lining, such as earth or asphalt, have a tendency to conform to reasonable settlement without failure, and the subgrade need be only firm enough to permit proper compaction of the lining itself. Usually undisturbed soils are satisfactory for a foundation for any type of lining without further treatment, but all fill material which will support the lining should be compacted prior to trimming and placing operations. Natural in-place soils of low density should be thoroughly compacted or removed.

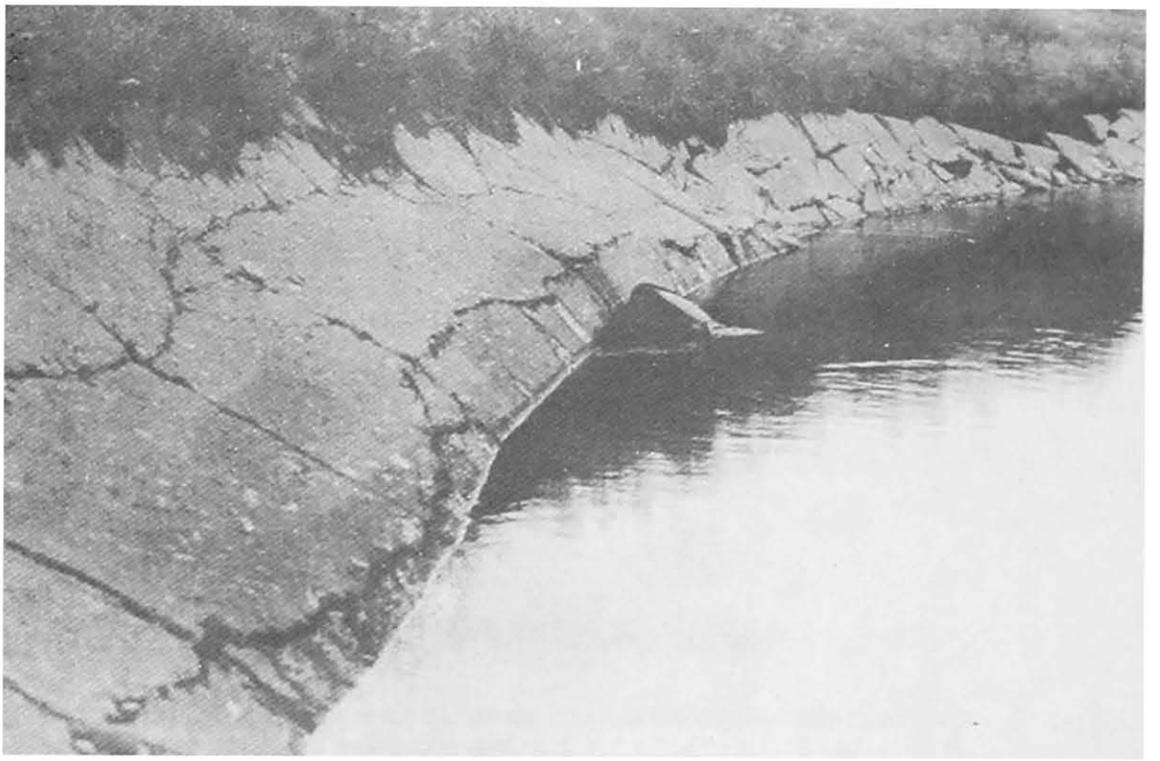
It is the practice of the Bureau of Reclamation to specify that the embankment supporting the lining be constructed of well-compacted materials. These specifications require that after the top soil has been stripped, the entire surface for the embankment shall be plowed thoroughly to a depth of not less than 6 inches, moistened and compacted. The embankment materials shall be placed at a specified moisture content and compacted to a specified density in layers not more than 6 inches thick. The dry density of the soil fraction in the compacted material shall not be less than 95 percent of the laboratory maximum dry-soil density as determined by the Bureau of Reclamation compaction tests, and at a moisture content suitable for achieving this density. The material when distributed and compacted shall be homogeneous and free of lenses and pockets. The top width of the compacted embankment varies with size and location of canal, type of lining, and other pertinent factors, but is usually 2 to 4 feet for canals up to 300 cfs and 6 to 12 feet for larger canals. The outside slope of compacted embankment is normally specified as 1-1/2:1. The compaction of loose soil in cut sections or of soils replacing unsuitable subgrade materials should meet the same requirements for density as those specified for compacted embankment.

When partial backfilling of an existing canal is necessary to reduce the cross-sectional area to that required for a lined canal, the puddling or ponding method has been found to be a satisfactory method of compacting most soils. Subsequent to backfilling, the canal section is rough trimmed to the approximate dimensions required for the lined canal, making due allowances for the settlement of the fill. The canal is slowly filled using temporary earth dams to check the water to the proper height, and allowed to stand for at least 24 hours before being drained. When the subgrade has become sufficiently dry, the section is fine trimmed and lining is installed.

Since most canal linings are installed to prevent seepage, the subgrade is usually relatively free draining. Occasionally, however, it may be necessary to employ lining in areas which are subject to seasonal high ground-water level. During periods of operation when the canal is empty or when the water level in the canal is relatively low, the high ground-water table may result in unbalanced hydrostatic back-pressures on the lining which may be, and often are, sufficient to damage the lining by flotation. A similar situation may occur in areas where the canal is lined for reasons other than to prevent seepage and where the soil is sufficiently water-tight to prevent the free drainage of the leakage from the canal. The accumulation of the water in the soil surrounding the canal may result in a local high ground-water table which, during a period of rapid drawdown of the water level in the canal, may produce damaging hydrostatic back-pressures. In regions subject to freezing temperatures, the canal lining may also be severely damaged by the freezing and resultant heaving of the saturated subgrade. Where these conditions can be expected to pertain, artificial drainage of the subgrade should be provided.

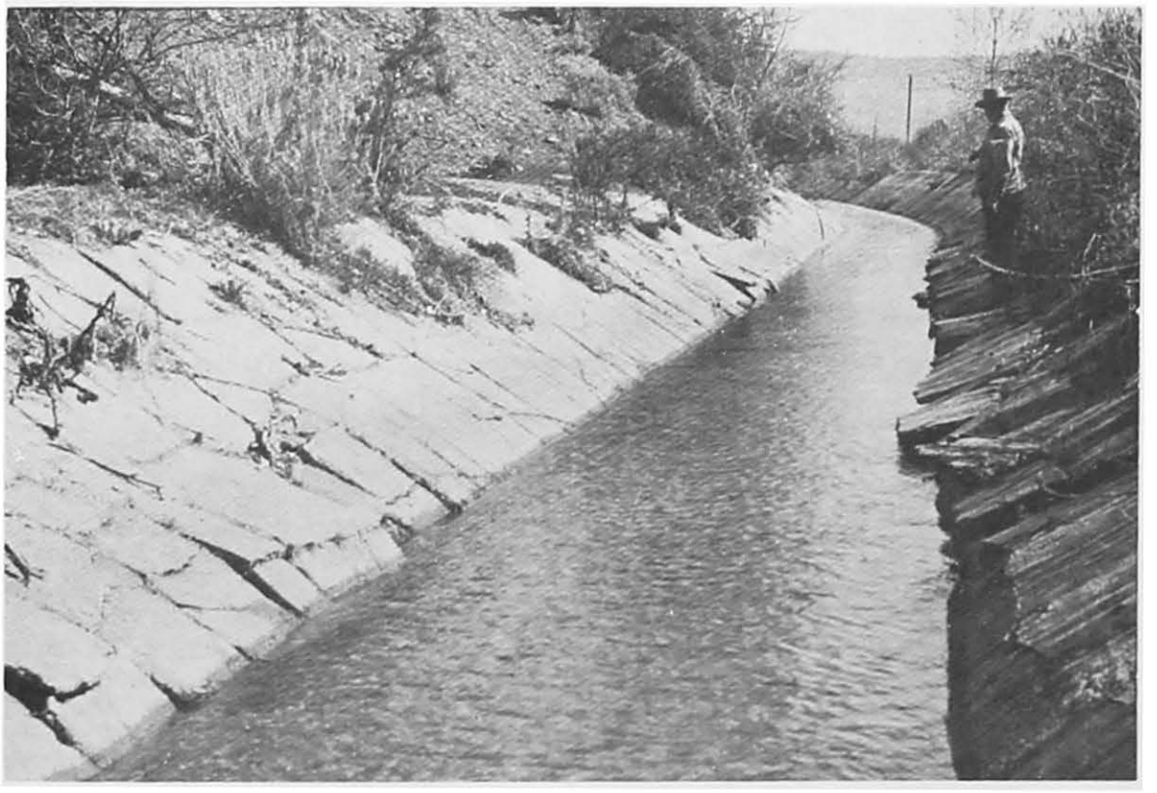
As an illustration of the damaging effect of hydrostatic back-pressures, the Temescal Water Company's canal near Corona, California, was originally lined with a 3/4-inch plaster lining which failed because of flotation by water pressure behind the lining. The thin lining was replaced with a 4-inch unreinforced concrete lining but some damage has been sustained by the heavier lining from the same cause. Similar damage was caused to the Santiago canal of the Irvine Company, Tustin, California, by storm water entering behind the lining. In one instance, a 1,000-foot section of the lining was destroyed.

The field reports from Region 1, in the Pacific Northwest, contained considerable evidence of the damage that may result from a saturated subgrade. It was stated in these reports that the 3-inch unreinforced concrete lining in a section of the Snipes Mountain lateral of the Yakima project was severely damaged by hydrostatic back-pressures resulting from percolating water from an irrigated tract above the canal. (See Photograph No. 84.) Another section of this lining which had been installed in 1914 was completely



Photograph No. 84

Concrete lining severely damaged by hydrostatic back-pressure--Snipes Mountain Lateral--Yakima Project.



Photograph No. 85

Damage to concrete lining by frost heaving of saturated subgrade--Okanogan Project, Washington.

destroyed in the same manner and was replaced in 1940 with a 4-inch reinforced concrete lining which was provided with weepholes and underdrains. It was reported that this appears to be an adequate design. In a section of the Okanogan project main canal at mile 0.2 where the subgrade was saturated by water from a pond above the canal, the lining was badly damaged by heaving of the subgrade from frost action. (See Photograph No. 85.) In these reports it was stated that frost heave is probably the greatest cause of destruction of linings in climates such as are experienced in Region 1.

The artificial drainage provided in areas where these conditions exist usually consists of 4- or 6-inch tile placed in gravel-filled trenches along one or both toes of the inside slopes. These longitudinal drains are either connected to transverse cross-drains which discharge the water below the canal or extend through the lining and connect to outlet boxes on the floor of the canal. The outlet boxes are equipped with one-way flap-valves which automatically relieve pressure greater than the water pressure on the upper surface of the canal base.

CONSTRUCTION TOLERANCES

One of the purposes of the specifications and drawings which become an integral part of a construction contract is to insure that the structure will be completed in the most economical manner consistent with the purpose, safety, serviceability, durability, and appearance of the structure. Yet the total cost is affected to an appreciable extent by the rigidity of the specification requirements just as it is by the dimensions of the component parts of the structure as specified on the drawings. Therefore, the results achieved by each specification requirement should be carefully balanced against the resultant increase in construction costs. All unnecessary requirements and refinements should be eliminated.

The Bureau recognizes the desirability and practicability of relaxing the requirements for alinement, grade and surface finish in the construction of lined canals in the interest of securing lower-cost canal lining. This relaxation can be achieved at very little or no sacrifice in the durability and serviceability of the lining and probably will result in appreciable lowering of construction costs by the elimination of requirements for a comparatively high degree of workmanship where such is not essential. In addition, the relaxation of specification requirements should be an encouragement to the development of relatively simple, inexpensive construction equipment. In the past, the specifications for the trimming of the subgrade and the placing and finishing of the lining have usually been so exacting as to almost necessitate the use of relatively complicated equipment operated on tracts along each canal bank and maintained in correct position by means of a taut guide wire. Larger construction tolerances would probably permit the more economical design and use of mechanized equipment.

The principal elements in lined canal construction for which a modification of specification requirements would achieve the greatest reduction in costs are preparation of the subgrade, placing the lining, and finishing the lining surface. In regard to the preparation of the subgrade, wider tolerances for alinement and grade are probably the most important modifications. In recognition of this fact, the Bureau of Reclamation has recently adopted specification requirements which permit a 4-inch departure from established line and a 1-inch departure from established grade. For the smaller canals and laterals these tolerances could possibly be increased somewhat to provide all the latitude necessary to secure the full benefit of reduction in costs possible by relaxation of requirements for line and grade.

Additional economy should result from lower requirements for compacted embankment. Where compaction is necessary, it is probable that something less than 95 percent of laboratory maximum soil density is adequate to support any lining since unit loads are low, being about 3 pounds per square inch for canals having a water depth of 7 feet. In fact, for the smaller canals and laterals, the compaction resulting from the operation of construction equipment may be sufficient if care is exercised in securing the maximum, uniformly distributed, rolling effect of such equipment and if the soil is damp. Where soil conditions are favorable, the ponding or puddling method of subgrade compaction as described under Subgrade Preparation may be satisfactory and economical.

It is anticipated that the furtherance of equipment development and considerable economy in the placing of the lining can be achieved by permitting a wider tolerance in lining thickness with a specified minimum thickness based on area of lining and yardage of concrete placed. Such tolerances would permit the contractor to weigh the cost of extra concrete for a thicker lining against the additional equipment costs required to provide and operate equipment capable of placing a uniform slab of the minimum thickness.

Finishing requirements also can be relaxed considerably to reduce the amount of expensive hand-labor required for this operation. For concrete linings, when a uniform, well-filled slab emerges from the slip-form, little or no finishing work may be necessary. Bureau of Reclamation requirements are that the finished surface shall be equivalent in evenness, smoothness, and freedom from rock pockets and surface voids to that obtainable by the effective use of a long-handled steel trowel. Light surface pitting and light trowel marks will not be considered objectionable. Where the surface produced by the machine fulfills these requirements no further hand-finishing is necessary.

Where it is necessary to use a float on hand-placed work or elsewhere, the floated surface need receive only one or two passes of a long-handled steel trowel. It should be possible on most hand-placed work to screed the concrete to the proper thickness so as to leave a surface sufficiently smooth that a satisfactory finish can be obtained with about two up-slope passes of the steel trowel. This is the procedure followed by the Turlock and other Central Valley irrigation districts in finishing hand-placed concrete linings and has proven satisfactory. Similar relaxed finishing requirements should be utilized for other types of lining to achieve the maximum economy.

GENERAL ECONOMICS OF CANAL LINING

The economic justification for the construction of canal lining depends upon the estimated value of the benefits which can be expected to accrue as compared to the estimated cost of construction. Since the reduction of seepage from a canal is usually the basic reason for considering the construction of a lining, a most important factor in the economic analysis is a study to determine the quantity and value of the water which can be saved, together with an analysis of the damage or added costs to the project if the water is allowed to escape. Other benefits, such as reduced costs of operation and maintenance, improved canal hydraulic properties, and more efficient operation, must also be considered, when applicable to the study. The relation of the many factors involved and an accurate determination of their importance and value presents an interesting, although complicated, problem.

Normally, the amount of money that can justifiably be spent for canal lining will be determined by the conditions on the project for which the lining is intended. However, in some cases, the water conserved by the lining will be available for use on adjacent projects and the justification for lining may be extended to evaluate the benefits of an increased supply of water for adjacent lands.

VALUE OF WATER LOST

Since the amount of seepage and the unit value of the seeped water are of primary importance, a measurement or estimate of the amount of seepage must be made before the need for lining can be definitely ascertained. The amount of seepage that can be tolerated before it becomes economical to install a lining will vary with individual project considerations. Therefore, it is essential that accurate measurements be made on existing projects and close estimates based on field tests be made on proposed projects for consideration in determining the economic practicability of a lining program.

In considering the amount of seepage, it is necessary to differentiate between seeped water that is irrecoverable and seeped water which is recovered as return flow in a canal or lateral at a lower elevation or in the parent stream for rediversion and use on the same project. The irrecoverable water is a total loss and is a strong recommendation for lining; whereas the recoverable water which is picked up as return flow can be placed to a beneficial use and, unless it causes or aggravates drainage difficulties, it does not offer a substantial justification for lining. If the water, after it has been used a number of times, as on the Shoshone project where it is estimated that some of the water is reused four times, deteriorates in quality by the pickup of salts until it is unfit for agricultural use, it becomes a definite loss just as is the irrecoverable seepage.

In a few irrigated areas, important use is made of the groundwater in providing an additional or supplementary supply of water for irrigation. In these areas, a replenishment of this underground storage is essential. Therefore, the escape from unlined canals of water which contributes to the groundwater supply does not constitute an irrecoverable loss. It may, however, be of concern in project operations in that it may constitute a loss of revenue from canal deliveries.

The determination of the value of water which escapes from a canal by seepage is a most difficult problem. The first consideration is the beneficial use which could have been made of the water had it been retained in the canal system. This analysis must give consideration to the ratio of irrigable lands to available water supply and the acre value of the crops produced. The per acre assessment for the project is not usually a fair determination of the value of the water since this cost reflects the expense of construction, operation, and maintenance which are not necessarily indicative of, or related to, the water supply or income from farming operations. Obviously, these variable factors necessitate an individual determination for each project to estimate the value of the water which can be saved from seepage.

An indirect approach to the determination of the value of water lost in seepage may be undertaken through a study of the repayment ability of the project. This analysis is predicated on the assumption that the maximum value of the water lost in seepage will be equal to the maximum additional charge that can be levied to prevent its loss. As a general rule, the maximum annual charge that a water user can pay for the annual cost of construction, drainage, operation, and maintenance is that which will permit the user to realize a

reasonable return on his investment over a period of years. This maximum annual charge will vary with the projects depending upon climatic conditions, acre value of crops, cost of production, etc. Therefore, the amount which can be justified for expenditure for canal lining divided by the repayment period in years will be the difference between the above maximum annual charge which the water user can pay with the lining installed and the existing annual charges for construction plus anticipated annual charges for operation, maintenance, and drainage. This permissible charge for canal lining can only be determined by a thorough study of a project's repayment ability.

The installation of canal lining need not in all cases necessitate the levying of increased charges against the water users of the project. If there is an area of land which can be brought under cultivation with the water saved by the lining, the cost of the lining could possibly be borne by the newly developed acreage with little or no increase in charge to the original water users. In other cases, the savings in the cost of providing drainage, benefits of reclaiming waterlogged land and savings in operation and maintenance costs may possibly equal the cost of the lining.

DRAINAGE PROBLEMS

Few irrigated farming areas have favorable underground geological conditions which make artificial drainage unnecessary. The extent of the influence of canal seepage on the drainage problem is debatable and difficult to determine in most cases. The effects of canal seepage are most noticeable where land adjacent to the canal has been reduced to a virtual swamp and rendered worthless. The monetary loss from this waterlogged land or the cost of drainage to remedy the situation is directly chargeable to canal seepage and, as such, is justification for canal lining.

It was reported that operating personnel of the Goshen irrigation district of the North Platte project, Wyoming, believed that the lining of 6,310 feet of Lateral 50.9 with a 4-inch reinforced concrete lining would pay for itself in 12 years because, in addition to saving 200 acre-feet of water annually and reducing the cost of operation and maintenance $40 per year for this section of canal, it reclaimed 45 acres of land which each year became too waterlogged to farm. A similar lining in 2,360 feet of Lateral 10.1 on the same district resulted in reclaiming 15 acres of land and partially reclaiming a larger area. A 4-inch reinforced concrete lining in 10,000 feet of Lateral 24 and a short section of another canal on the Pathfinder irrigation district of the North Platte project restored 40 acres of seeped land to agricultural use.

The effects of seepage are not always readily evident. The seeping water from the canals on higher grounds often disappears into a pervious underground stratum to reappear in a low-lying area at some distance from the canal. In this case, waterlogged land will be the result of both canal seepage and deep percolation from irrigation operations on higher terrain. The part that is due to canal seepage can only be estimated. It is doubtful, in such instances, if lining the canal to prevent seepage losses would eliminate the condition but it would certainly reduce the extent and cost of operation and maintenance of the drainage system.

When pumping of the drainage water is necessary, as on the Yuma project in Arizona where the water is pumped back into the river, prevention of the seepage losses would materially reduce the cost of drainage. For example, in 1943 a total of 67,481 acre-feet of drainage water from the Yuma project was pumped a height of 9.92 feet at the Boundary pumping plant at a cost of $0.082 per acre-foot for power only. This was largely the drainage from the Valley and Auxiliary division. A total of 378,828 acre-feet was diverted for use on the Valley division with an estimated canal and lateral loss of 60,622 acre-feet. With a total diversion of 19,985 acre-feet on the Auxiliary division, the estimated canal and lateral loss was 3,963 acre-feet. Assuming that only 40 percent of these losses was picked up by the drainage system, the cost of pumping this seeped water was

$$64{,}585 \times 0.082 \times 0.40 = \$2{,}118 \text{ per year}$$

or $84,720 for 40 years. This cost is for power only so the savings in drainage costs that would result from the installation of lining would be materially greater if reduced operation and maintenance cost were considered.

Many of the existing irrigation projects are favored with a sufficiency of water and unless there is a serious drainage problem or frequent canal failures there is little interest on the part of water users in the conservation of water or prevention of seepage losses by canal lining. One of the most obvious and tangible results of a properly designed and constructed lined canal, even on projects having a sufficiency of water, is the saving in operation and maintenance costs for the irrigation system. This saving alone may in a few instances pay for a canal lining over a 40-year period.

The cost of operation and maintenance per mile of canal will vary with the project and even with the canals on the same project because of various factors, such as climatic conditions, silt load carried by the water, terrain traversed by the canal, and efficiency of operation and maintenance. This is borne out by the wide variations in the costs presented in the field reports on existing canal linings. From the data submitted in these reports, the average cost of maintenance of a concrete lined section is approximately 30 percent of that of an unlined section of equal capacity.

To more accurately ascertain the cost of operation and maintenance of unlined canals, the reports for nine different Bureau of Reclamation projects in various parts of the country were analyzed for the years 1938 and 1939. In this manner, cost data were secured on 3,300 miles of canals including 55.1 miles of lined canals. After making an approximate correction for the lined canals, it was found that the cost of operation and maintenance for unlined canals varied from $78 to $324 per mile with an average of $166 per mile. Cost of maintenance alone varied from $41 to $245 per mile with an average cost per mile of $116. Thus the average maintenance costs were 70 percent of the total cost of operation and maintenance.

Just as there was a wide variation in the amount spent for maintenance on the nine different projects, there was a considerable variation in the amount expended for various items of maintenance. But for the 2 years considered, an average of 40.2 percent of the cost of maintenance was spent on removal of silt and weed control; 13.3 percent for care, repair, and maintenance of banks; 30.7 percent for repair of structures; and 15.8 percent for miscellaneous items.

In an evaluation of the economic benefits of a canal lining with reference to the costs of operation and maintenance, it must be recognized that the application of the factors involved will be dependent upon the type of canal lining being considered. For example, if a lining is being considered for an existing unlined canal, the economic studies for using a concrete lining can properly include benefits anticipated from reduced costs of weed removal, less danger from burrowing rodents, and any other conditions which a rigid, high quality lining will benefit. On the other hand, the economic studies for an earth lining cannot include these factors and must rely almost entirely on the value of seepage prevention for justification. The qualities of a lining, therefore, determine some of the factors which may be considered in the economic analysis.

It is believed reasonable to assume that the cost of these factors of maintenance would be substantially reduced by the installation of a high quality lining, such as portland cement concrete, asphaltic concrete, shotcrete brick or masonry linings. These materials are highly resistant to erosion and, if the lined canal were designed with sufficient velocity to prevent the deposition of silt in the canal, the necessity of routine silt removal would be greatly reduced. For the same reason, the cost of maintaining the canal section against erosion would be similarly affected. In addition, these linings, with the exception of asphaltic concrete which usually requires permanent sterilization of the subgrade to inhibit weed growth, are practically impenetrable by weeds. Canal lining would reduce the cost of weed control and removal.

In areas where rodents are prevalent, many canal failures occur each year in unlined canals as a result of their actions. The cost of repairing the canal and the resultant loss in crops due to a lack of water may be of considerable magnitude. Since any of these high quality linings are practically impenetrable by rodents, their use offers increased safety from breaks resulting from rodent action as well as from erosion. It was stated in the field reports on the canals of the Yuma project that "It has been estimated that for every break and washout in a concrete lined lateral there are about twenty in the unlined laterals." A 3,310-foot section of Lateral No. 1 of the Rock Ranch canal of the Lincoln Land Company, Wyoming, was lined with a 2-inch reinforced concrete lining primarily to prevent washouts which were constantly occurring as a result of rodent action. The manager of the company

stated that the lining had paid for itself in the three years it had been in operation because of the high cost of repairs and the crop damage resulting from a lack of water.

Although, as stated previously, data on the cost of maintenance submitted in the reports on existing lining indicate that the average maintenance cost on a lined canal is only about 30 percent of that on a comparable unlined section, a more conservative figure of 40 percent has been used for purposes of illustration in this report. Assuming a maintenance cost of $116 per mile for an unlined canal, lining would result in a saving of about 116 x 0.60 = $69.60 per mile per year, or $2,784 per mile in 40 years.

It would appear that the cost of operation would be similarly reduced due to better operating efficiency. In addition to the savings in the cost of operation of the irrigation system, additional economics should result from the savings in man-hours required by the farmers in the irrigation of their lands. It has been reported that with lined farm ditches the improvement in farm deliveries may be such as to result in a saving of 50 percent in man-hours required per irrigation. An estimate of the monetary value of the improved operating efficiency, both in the irrigation system and to the farmer, is difficult to determine. In many of these cases, these savings are known to have been appreciable and therefore consideration should be given to an evaluation of these benefits in a study of the feasibility of installing lining.

On a project where it is necessary to pump the irrigation water to gain elevation, the reduction in operating cost that would result from the installation of lining in an existing canal may be substantial. As an example, a pumping plant on a Bureau project raises 525 cfs of irrigation water a total of 29 feet. In 1944, during a 6-month period, 120,111 acre-feet of water was pumped using a total of 6,527,720 kwhr of electrical energy, or 1.87 kwhr per acre-foot per foot of lift. Assuming that 20 percent of the water is lost through seepage in the canals and laterals that could be saved by lining and that power cost $0.0025 per kwhr, the savings in pumping costs alone, if this seepage were prevented, would be 29 x 1.87 x 0.20 x 0.0025 x 120,111 = $3,256.80 per year, or $130,272 for 40 years.

The information and data on the economics of canal lining have been presented merely to indicate the possible savings and benefits that accrue from the use of canal lining. Obviously, all of these benefits and savings cannot be realized on one project nor can they be achieved to the same extent on different projects because of varying conditions. Each installation must be justified by the existing conditions and will involve an economic study of its feasibility. Therefore, it is impossible to present an economic justification of canal lining such that it would be applicable or of great value to any particular project.

However, as an indication of the possible benefits of canal lining, an economic comparison is presented of unlined and concrete lined canals based on the average cost of construction, average cost of operation and maintenance, and average charge for irrigation water. The data previously presented on the cost of construction of concrete lined canals and similar data on the cost of construction of unlined canals were used to determine the yearly construction charge per canal station for a 40-year repayment period. Sufficient information was not available to accurately determine how the cost of operation and maintenance varied with the size or capacity of canal. However, of the 3,300 miles of canals covered by the data previously referred to for this cost, 71 percent of the canals was of 0 to 50 cfs capacity and 19 percent was of 50 to 300 cfs capacity. Therefore, the average cost of $166 per mile for operation and maintenance probably was applicable to a canal of about 50 cfs. For this comparison it was assumed that the cost varied directly with the wetted perimeter from the $166 per mile for a 50 cfs unlined canal to $450 per mile for a 4,000 cfs unlined canal. It was also assumed that the cost of operation would not be affected by lining, and, on this basis the cost of operation and maintenance for a concrete-lined canal was 58 percent of the cost for an unlined canal.

To determine an approximate value of water lost by seepage, it was assumed that the average charge to the water users for irrigation was about $1.50 per acre-foot. For the purposes of analysis, it was assumed that the loss for unlined canals would be 1.0 cubic foot per square foot of wetted perimeter per day and 0.20 cubic foot per square foot per 24 hours for concrete-lined canals. An irrigation season of 130 days was assumed. The comparison of these costs for the two types of construction is shown by Table 3 for illustrative purposes only. The actual water loss in an unlined canal may in some cases be no greater than the loss in some lined sections as shown in the discussion on seepage determinations.

Table 3

COMPARATIVE COST OF CANALS PER 100 FEET

Based on a 40-year repayment period

	Canal capacity in cfs									
	100	200	300	500	750	1000	1500	2000	2500	3000
1. Unlined Canals										
a. Annual construction charge	1.30	1.75	2.20	2.95	4.25	5.90	7.40	9.40	11.50	13.50
b. Annual O&M	3.54	4.08	4.45	5.01	5.72	6.36	7.05	7.50	7.82	8.04
c. Annual loss of water	8.91	12.86	15.56	19.65	24.80	29.50	34.60	37.90	40.70	41.90
Total	13.75	18.69	22.21	27.61	34.77	41.76	49.05	54.80	60.02	63.44
2. 3" Reinforced Concrete Lined Canals										
a. Annual construction charge	10.00	13.60	16.20	19.50	22.50	24.70	29.00	33.50	38.30	43.60
b. Annual O&M	2.05	2.37	2.58	2.91	3.32	3.69	4.09	4.35	4.54	4.66
c. Annual loss of water	1.59	2.30	2.69	3.17	3.56	3.88	4.39	4.95	5.37	6.20
Total	13.64	18.27	21.47	25.58	29.38	32.27	37.48	42.80	48.21	54.46

Based on the above assumptions, the most important factor in the justification of concrete lining is the value of the water saved, although the savings in operation and maintenance are appreciable. As indicated by Table 3, the annual cost of a 3-inch reinforced concrete-lined canal is less than a similar charge for an unlined canal for the range of capacities shown and for the three cost items considered. Similar comparisons could be prepared for other types of linings that may be under consideration.

CONSTRUCTION COSTS

The importance of including canal lining in the original plans and designs of an irrigation project cannot be too strongly emphasized, if studies have proved its economic feasibility. It is only during the planning and designing stages that full advantage can be taken of the many benefits of a permanent canal lining.

On many of the existing projects, it will not be economically feasible to install canal lining even though the need is evident and great. Lining cannot be justified because the benefits and savings will not equal the cost; the benefits and savings will be limited largely to lower seepage losses with a greater supply of usable water, recovery of water-logged lands, lower operation and maintenance costs, improved drainage conditions, and less danger of canal failures. However, had lining been included in the original plans and designs so as to take advantage of additional benefits that accrue from its use, it might have been possible to economically justify the lining. These additional benefits are: reduced storage and diversion requirements, smaller canal sections and structures, less right-of-way, greater range of permissible velocities and canal gradients, and lower lining installation costs as compared to construction undertaken at a later date.

Seepage losses from canals and laterals represent a loss not only of valuable irrigation water that should be conserved for productive agricultural use, but also a considerable loss in additional costs of construction from which no return is received on the investment. Storage reservoirs and dams must be constructed of size to impound not only the useful water but also the water that will be lost in transit to the farms. Canals and laterals must be designed with sufficient capacity not only to transport the useful water but the water that will be lost through seepage as well. Often the canal cross-sectional area required for the latter water is equal to that required for water that will be delivered to the users. Canal structures (bridges, weirs, drops, check, and chutes) must be of increased size. Thus the seepage losses require a considerable increase in construction costs--an increase which could be obviated by the installation of an impervious canal lining at the time of original construction.

In those instances where right-of-way requirements involve the acquisition of expensive agricultural lands, the reduced requirements for a lined section with improved hydraulic properties are of importance. The right-of-way for these canals, in addition to having a high initial cost, imposes a heavy toll on the land it serves. It has been estimated that the area required for canal and lateral right-of-way may often be 1 percent of the total irrigable acreage and, if all corners and restricted areas resulting from the distribution system are included, the area which cannot be cultivated for this reason may approach 3 percent.

A lined canal would reduce the right-of-way requirements by permitting the use of a smaller canal due to elimination of the seepage losses and to improved hydraulic properties. The average unlined canal of 100 cfs capacity, as designed by the Bureau of Reclamation, has a water surface width of 15 feet; whereas a concrete-lined canal of the same capacity has a width of 12.0 feet. And if, for instance, lining of the canal would reduce the required carrying capacity of the canal by 20 percent by eliminating seepage, a lined canal of 80 cfs capacity and a water surface width of 10.8 feet would be satisfactory. This saving in right-of-way requirements would be further increased by the narrower banks that can be used in conjunction with an impervious lining.

In addition, because of the smaller cross-sectional area required for a lined canal, the amount of excavation is materially decreased. This may be of particular importance where the canal location is along a steep slope or requires a deep cut. In such locations an increase in the width of a canal would result in a much greater volume of excavation than would the same increase in fairly level terrain. Such a situation is described in the field report on canal lining in the New York canal of the Boise project. It was reported that the capacity of

this canal was increased in 1909-1912 by enlarging the existing 40-foot earth section to one with a 70-foot base width and also by the lining of selected sections totaling approximately 6-1/2 miles with concrete lining in the existing 40-foot base width section. Most of this lining was placed to reduce seepage and to increase the stability of the canal along the hillside location but some lining was placed through sections of deep cuts where it was found more economical to line the 40-foot bottom canal than to enlarge for an earth section with a 70-foot bottom.

Canal lining, in addition to permitting smaller, less costly structures, may reduce the number required. The maximum permissible velocity in an unlined canal is limited because of the problem of erosion and this, in turn, limits the permissible canal gradient. However, canals often must traverse slopes which for an unlined canal require the use of drops or chutes to avoid destructive erosion. A permanent lining, because of the higher permissible velocities and the steeper gradients, may eliminate the need for many of these structures. A construction contract recently awarded by the Bureau of Reclamation for the construction of approximately 17.4 miles of small, unlined laterals required 94 drops and 2 concrete chutes because of the steep terrain. Although canal lining would not have eliminated all of these structures, it would have certainly reduced the number required.

In certain cases, it may be important to use as flat a canal gradient as possible so as to serve the maximum area, since only the land below the canal can receive water without expensive pumping. A lined canal because of the lower coefficient of friction would permit a much flatter slope than would an unlined canal, thus permitting a larger area to be irrigated.

Another factor that is of importance on a project which requires pumping of the water to gain sufficient elevation to serve the land is the cost of pumping to overcome friction losses in the canal. If lined canals are substituted for unlined canals, the pumping head can be kept the same and the irrigated area increased or the area kept constant and the pumping head reduced. If the latter alternative is used, the savings in cost of pumping may be appreciable. Using the figures from the Bureau pumping plant previously referred to, and assuming a gradient of 0.0001 for a lined canal and 0.00025 for an unlined canal, the latter requires 0.79 foot more head per mile. To pump 120,111 acre-feet of water this additional height, if 1.87 kwhr of power is used per acre-foot per foot of lift at $0.0025 per kwhr, costs 1.87 x 0.79 x 120,111 x 0.0025 = $443.60 per mile per year or $17,744 per mile in 40 years.

Lastly, the cost of installing a permanent lining in a canal that has been in service for some time is more than if it were done at the time of construction of the canal, disregarding all price changes. The cross-sectional area of an unlined canal is greater than for a lined canal of equal capacity and, unless increasing the capacity of the canal is a consideration, it is not economically practical to line the perimeter of the existing canal. Instead, it is the usual practice to backfill part of the cross-section with compacted material, trim the subgrade to exact line and grade and install the lining. This involves placing and compacting of backfill, a costly operation, which would not have been necessary had the lining been installed originally.

ECONOMIC ANALYSIS

Although it may be highly desirable to install lining in a canal for any of the heretofore discussed reasons, such as prevention of seepage loss, reclaiming waterlogged land, lower operation and maintenance costs, mitigation of drainage difficulties, etc., it must be economically feasible to do so. To be economically feasible the capitalized annual value of the benefits resulting from the installation of the lining must be equal to, or greater than, the annual cost of the lining. Some of the benefits are rather intangible and difficult to evaluate. A careful estimate based on wide experience on the part of the estimator may be the only method of securing the required data. Mitigation of drainage difficulties and insurance against canal failures for instance, fall into this category.

Seepage losses can usually be determined or estimated with reasonable accuracy. In existing canals this can be accomplished by direct seepage measurements and in proposed canals it can be estimated from results of soil and permeability tests and borings along the proposed canal location. Operating and maintenance costs on lined and unlined canals

can normally be secured from cost data on existing canals on the same project or on projects operating under somewhat similar climatic, geographical and agricultural conditions. These costs for lined canals will vary from a minimum when the lining is new to a maximum when it approaches the replacement stage and an average cost should be chosen. The total cost of the lining can be estimated with reasonable accuracy and this cost should be amortized over the serviceable life of the lining to determine the annual cost. To be strictly correct, this annual cost should be the annual payment which at interest will accumulate to the replacement cost by the end of the serviceable life of the lining. But such a high degree of accuracy is not consistent with the other assumed and estimated values and assuming that the annual cost equals the total cost divided by the years of serviceable life is usually sufficiently accurate.

The determination of the economic feasibility of a proposed canal lining project, as for any other project, depends upon the yearly cost and benefit considering all pertinent items. Some items may not be available, such as additional storage water or additional land on which to use excess water saved. Water may have a higher value in certain months than earlier or later in the irrigation season. After the various items have been evaluated, the feasibility of the proposal can be reduced to mathematical statements. The formulas presented below are for illustration and are similar to those developed by O. W. Israelsen and D. C. Reeves in Ref. 127.

If C = cost of lining in cents per square foot including all costs incidental to installing the lining, such as trimming, backfill, etc.,

S_1 = seepage loss in lined canal in cubic feet per square foot per 24 hours,

S_e = seepage loss in unlined canal in cubic feet per square foot per 24 hours,

P_1 = wetted perimeter of lined canal in feet,

P_2 = total perimeter of lining in feet,

P_e = wetted perimeter of unlined canal in feet,

d = number of 24-hour days which canal operates annually,

Vw = value of water saved in cents per acre-foot,

L = length of canal in feet,

Y = life of lining in years,

D = annual saving in cents in reduced drainage costs, recovered land, insurance against breaks, improvement in appearance of system, and

M = annual savings in operation and maintenance costs due to lining expressed in cents,

the annual value of water lost by seepage from the unlined canal is

$$\frac{P_e L S_e d Vw}{43560}$$

and that for a lined canal is equal to

$$\frac{P_1 L S_1 d Vw}{43560}$$

therefore, the annual saving in seeped water resulting from the lining is

$$\frac{P_e L S_e d Vw - P_1 L S_1 d Vw}{43560} = \frac{d Vw L}{43560} (P_e S_e - P_1 S_1).$$

and the annual benefits are equal to

$$\frac{d\,Vw\,L}{43560}\,(P_e\,S_e - P_1 - S_1) + M + D$$

The annual cost of the lining is

$$\frac{P_2\,L\,C}{Y}$$

If the lining is to be economically feasible, the annual cost must be equal to, or less than, the annual benefits, or

$$\frac{P_2\,L\,C}{Y} = \frac{d\,Vw\,L}{43560}\,(P_e\,S_e - P_1\,S_1) + M + D$$

$$C = \frac{Y}{P_2}\left[\frac{d\,Vw}{43560}\,(P_e\,S_e - P_1\,S_1) + M + D\right]$$

The above equations are satisfactory for use in connection with existing canals, but they are not correct for making economic studies of the feasibility of employing lining in new constructions. The formulas are based on the cost of the lining only and do not provide for the other savings, such as less right-of-way, fewer structures, less excavation and compaction, etc., which may result from the use of lining. To make them applicable to new construction, the value of "C" should be the difference in total cost of the unlined canal and lined canal, expressed in cents, divided by the total square feet of lining. This would be the cost which would have to be justified. The remainder of the computations would be the same as before.

As an illustration of the method of determining the feasibility of installing lining in an existing unlined canal, it has been assumed that an unlined canal, 5,280 feet in length, is to be lined with 3-inch unreinforced concrete, on the same canal gradient. The unlined canal has a base width of 12 feet, a depth of 4.85 feet, side slopes of 1-1/2:1, and a wetted perimeter of 29.47 feet. The lined section of equal capacity would have a base width of 6 feet, a depth of 4.60 feet, side slopes of 1-1/2:1, a wetted perimeter of 22.58 feet, and a lining perimeter of 25.0 feet. Seepage measurements on the unlined canal indicate a loss of 1.50 cubic feet per square foot per 24 hours and it is anticipated that the loss with concrete lining will be 0.2 cubic feet per square foot per 24 hours. Water has a value of $1.50 per acre-foot and the canal is in operation 130 days per year. Based on previously presented data, the average cost of operation and maintenance for lined and unlined canals of this size would be approximately $125 and $216 per mile, respectively. Other benefits, such as improved drainage conditions, reclamation of waterlogged land, etc., are assumed to be $40 per mile.

Using these data, the annual value of the water saved by lining is:

$$\frac{L\,d\,Vw}{43560}\,(P_e\,S_e - P_1\,S_1) = \frac{(5280)\,(130)\,(150)}{43560}\left[29.47(1.5) - 22.58(0.2)\right]$$

$$= 93{,}856 \text{ cents.}$$

The net benefits are:

$$93{,}856 + M + D = 93{,}856 + 9100 + 4000 = 106{,}956 \text{ cents.}$$

The maximum justifiable unit cost for the lining would then be

$$C = \frac{106{,}956\,Y}{P_2\,L} = \frac{(106{,}956)\,(40)}{(25)\,(5280)} = 32.4 \text{ cents per square foot.}$$

Thus, it is shown that the installation of a concrete lining would be justified on this canal if the lining can be installed at a cost of 32.4 cents per square foot or less based on the comparison of the benefits which can be expected to accrue and the cost of construction. However, further economic studies would have to be made to determine the projects repayment ability to ascertain whether the cost of the lining can be borne and repaid by the water users over the period of serviceable life of the lining.

SUMMARY

The lower-cost canal and lateral lining sought must be reasonable in first cost, and insure satisfactory service without excessive annual costs for maintenance and replacement. Investigations and trial tests in the field and in the laboratory have not resulted in any remarkably new construction material or type of lining. Linings of portland cement concrete remain the most dependable and permanent type in common use and are in general the most expensive as regards first cost. At the other end of the list are silt and loose earth linings which, although low in first cost, are usually only partially effective, are costly and difficult to maintain, or are of questionable value after a few years if not properly maintained. In between these two extremes are several types of asphaltic linings, modifications of portland cement concrete which include soil-cement, and a number of earth material linings.

It has been demonstrated that linings of hot-mix asphalt can be constructed at a slightly lower cost than unreinforced portland cement concrete laid to the same thickness but to closer tolerances. However, experience with asphalt as a lining material is quite limited and its serviceability is not yet well demonstrated. Asphaltic-membrane linings sprayed in place can apparently be placed for considerably lower cost, but are still in the experimental stage.

Since the cost of reinforcement steel in concrete linings currently amounts to more than 20 percent of the cost of such a lining, its elimination will result in considerable saving. In many instances unreinforced concrete is adequate and the possible benefit resulting from the use of reinforcement does not justify the additional cost. The relative long range economy and durability of reinforced and unreinforced concrete lining is now being investigated. Considerably smaller reductions in the cost of concrete may be realized by a slight reduction in cement content or the substitution of pit-run aggregate with a corresponding sacrifice of durability. In localities where concrete aggregates are not economically available a sandy soil may be satisfactorily utilized in a soil-cement mix of plastic consistency which can be placed by the same methods used for concrete but which is of doubtful durability as a canal lining.

Shotcrete, or pneumatically placed mortar, has proved very satisfactory as a lining in many miles of canals and laterals in the southwest, but it is little, if any, lower in cost than machine-placed concrete of almost twice the thickness. Linings of precast concrete blocks are adaptable for use by small maintenance crews or by individual farmers for lining their own small ditches, but the high cost of hand labor in placing prohibits their economical use on a large scale. Limited resistance to external forces renders their general use unattractive.

Thin linings of compacted clay or bentonite are economical if suitable materials are available locally and, when properly maintained, are quite effective. A protective blanket of stable earth or gravel adds to the cost and much to the life of such linings. Heavy-compacted earth linings require more material yardage but permit more economical longitudinal rolling and a wider range of earth types. Other possibilities exist for stabilizing or compacting in-place soils by a number of methods adaptable to limited conditions and soil types.

Regardless of the type of material used, a great percentage of the cost of canal lining in recent years has been attributable to construction operations as compared to materials. It follows, therefore, that one of the greatest possibilities for lowering costs lies in the development and full utilization of mechanized equipment, in the simplification and relaxation of specification requirements consistent with good engineering practices, and the standardization of canal shapes and dimensions. Completely mechanized equipment has been utilized in the construction of large canals for years with a gradual reduction in the basic unit cost. More liberal tolerances in dimension, grade, and concrete finish may reasonably be permitted in smaller canals and laterals and this coupled with standardization of sizes will encourage the development and use of mechanized equipment. Significant progress in this direction has been made since the start of the Bureau's organized effort to reduce the costs of canal lining. New equipment has been developed and proved on several jobs, for placing concrete, soil cement, and asphalt; but the size of the jobs and the development problems encountered did not permit the full reductions in cost that are indicated. Results, however, have been very encouraging and larger scale trials are contemplated.

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