UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

LININGS FOR IRRIGATION CANALS

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INCLUDING A PROGRESS REPORT
ON THE
LOWER COST CANAL LINING PROGRAM

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As the Nation's principal conservation agency, the Department of the Interior has responsibility for most of our nationally owned public lands and natural resources. This includes fostering the wisest use of our land and water resources, protecting our fish and wildlife, preserving the environmental and cultural values of our national parks and historical places, and providing for the enjoyment of life through outdoor recreation. The Department assesses our energy and mineral resources and works to assure that their development is in the best interests of all our people. The Department also has a major responsibility for American Indian reservation communities and for people who live in Island Territories under U.S. administration.

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Frontispiece.—View of Courtland Canal, Bostwick division, Missouri River Basin project, showing compacted-earth lining in the foreground. 271-701-2200.

Preface

THIS PUBLICATION presents instructions, standards, and procedures developed by the Bureau of Reclamation for use in the lining of irrigation canals. It is intended to serve primarily as a guide for engineers, supervisors of irrigation districts, and others concerned with the planning, design, construction, and maintenance of irrigation canals. These instructions, standards, and procedures have been developed over a period of 16 years through the Bureau of Reclamation's lower cost canal lining program.

The publication presents an economic analysis and evaluation of the relative merits of lining or not lining irrigation canals; and provides information on the principal types of canal linings and their design, costs, methods of placement, uses, and serviceability.

The text reveals good service records for several types of low-cost canal linings including unreinforced concrete, shotcrete, asphaltic concrete, and prefabricated concrete blocks; buried membranes of asphalt, bentonite, plastic, or synthetic rubber; and thick compacted-earth linings. Exposed membrane linings of asphalt, plastic, and synthetic rubber, and other types of seepage control such as use of sedimentation and chemical sealants are being investigated and may prove satisfactory for many applications if certain difficulties can be overcome.

Much of the information that is presented on design, costs, methods of placement, uses, and serviceability of the various canal linings is in the nature of tables and discussions of the many lining installations placed under the lower cost canal lining program. These lower cost type linings aggregate more than 50,000,000 square yards installed in over 2,570 miles of canals and laterals, at a savings of many millions of dollars as compared with the more expensive linings formerly considered necessary.

Emphasis is placed in the text on relative costs, as canal linings usually represent a significant percentage of the cost of an irrigation project development. Judicious selection of a serviceable and economical lining will be reflected by a decrease in the overall project cost. In some instances, the selection of economical linings may make possible the construction of certain irrigation projects which could not be economically justified if more expensive linings were required. The many advantages, economic and otherwise, of including plans for canal linings in the initial project development wherever their need can be foreseen, rather than after the canal has been placed in use, are presented. To this end, methods are given for estimating probable seepage losses along the route of a proposed canal, as well as methods for detecting and measuring seepage in existing canals.

Conservation of the Nation's water supplies, particularly in the western States, is becoming increasingly important as the demand for this vital commodity continues to increase and new sources of supply become increasingly scarce. The time is rapidly approaching when the only natural water supplies available will be the salvage of those now being lost through transpiration, evaporation, consumptive waste, and inefficient storage and transportation practices.

The principles of conservation require that full use be made of our natural water supplies, and the greatest results can be accomplished through reduction in the amount of water lost through seepage during transportation to the farmers' fields. It has been estimated[1] that, by lining canals, treating canal subgrades, using closed conduits for the transport of water, and providing other means of seepage control on our irrigation systems, seepage losses can be reduced by an estimated 1.5 million acre-feet by 1980, which is enough water to irrigate several hundred thousand acres of land. These means of seepage control are discussed in this publication.

There are important reasons, in addition to conservation of water supply, for lining canals. Water which seeps from the canals and laterals often collects in lower-lying lands, thereby rendering them unproductive. Dams, reservoirs, and distribution systems must be designed and constructed with greater capacities to provide for the additional water lost in transit, and consequently cost more to construct. The reclaiming of water-logged land, where this is

[1] Select Committee on National Water Resources, "Evaporation Reduction and Seepage Control," Water Resources Activities in the United States, Committee Print No. 23 86th Congress, U. S. Senate, December 1959.

practicable, by construction of drainage systems is costly. Good engineering practice demands that all cost factors—the value of the land, the value of the water, and the cost of engineering features—be properly and carefully evaluated in the economic construction of an irrigation system, and that this evaluation be projected carefully into the future. An important part of this evaluation is the consideration of canal linings to conserve water and reduce seepage and water-logging of valuable land.

In summary, canals may be lined for the purpose of conservation, reducing damage to lowlands from seepage, reducing operation and maintenance costs, or increasing structural safety. Usually more than one benefit accrues. Whatever the purpose or purposes, the guidelines in this publication will be very useful to those persons engaged in this work.

This book was prepared by engineers of the Bureau of Reclamation under the direction of Grant Bloodgood, former Assistant Commissioner and Chief Engineer[2], at Denver, Colo. Special recognition is given P. W. Terrell, Assistant Chief, Canals Branch, Division of Design, and until recently chairman of the Bureau's Lower Cost Canal Lining Committee, for his preparation of design considerations, his helpful comments in the preparation of the manuscript, and his critical review of the publication as a whole. C. W. Jones, representing the Division of Research, as a committee member was responsible for the preparation of some and the coordination of other laboratory studies included in the publication. Preparation of the text was coordinated and the text was edited by R. J. Willson, Maintenance Engineering Branch, Division of Irrigation Operations, formerly a member and presently chairman of the Lower Cost Canal Lining Committee. Final review and preparation of the manuscript for the printer was by E. H. Larson, Head, and W. E. Foote, Assistant Head, Manuals and Technical Records Section.

The Bureau of Reclamation wishes to express its gratitude to those organizations which have permitted the use of material from their cooperative studies and field demonstrations, including the University of Idaho, the University of California, the Arizona State University, the Colorado State University, the Agricultural Research Service of the Department of Agriculture, the Bureau of Mines of the Department of the Interior, the Portland Cement Association, and the Asphalt Institute. Recognition is also given many material and equipment manufacturers who sponsored tests and donated time and material to the program, and contractors who helped in the development of new lining techniques. Lastly, much of the field work would have been impossible but for the able assistance of the many water user organizations on Bureau projects who contributed time, labor, and equipment necessary in the cooperative installation of many experimental linings. To the many who have individually contributed to the preparation of this publication and the accomplishments of the lower cost canal lining program, the Bureau of Reclamation and the members of the Lower Cost Canal Lining Committee wish to express their gratitude.

[2] Mr. Bloodgood retired on February 1, 1963, and was succeeded by Mr. B. P. Bellport who bears the title of Chief Engineer.

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Lower Cost Canal Lining Program

1. ***Purpose of Program.***—Water losses in canals and laterals through seepage have been satisfactorily reduced through the installation of relatively impervious linings, by special treatment of the canal section, or by the use of closed conduits. However, the costs of accepted and dependable methods of seepage control in general use in the past are prohibitive for many irrigation systems. Costs increased considerably along with all construction activities following World War II as the need for water conservation in the irrigation States also increased.

A considerable amount of individual, uncoordinated effort to reduce the cost of lining canals has been expended for short periods in the past. However, such effort was usually shortlived and resulted in very limited progress. In recognition of the urgent need for an organized and continuing effort, the Bureau of Reclamation inaugurated a lower cost canal lining program in June 1946. The program called for laboratory and office research, surveys of existing installations, seepage determinations, field experiments, and equipment developments aimed at reducing the cost of seepage control.

An interim report issued in 1946 outlined the broad scope of the lower cost canal lining program and presented information available at that time on various types of existing linings and their service records. The aims of the program and some of the problems involved were brought to the attention of the engineering profession in February 1947,[1] and an appeal was made to contractors and equipment manufacturers for their assistance in developing lower cost construction methods and greater mechanization of equipment. Progress during the first 2 years was covered in a general information report dated June 1948. In March 1949, a lower cost canal lining committee was formed to direct the program activities. Composed now of one member from each of the Bureau's seven regional offices, and three from the Office of Chief Engineer, the committee meets annually to formulate fiscal year programs and to arrange for funds to finance the work.

The scientific talent of several universities and colleges, the Soil Conservation Service, the Agricultural Research Service, the Geological Survey, and others has been enlisted on various phases of the laboratory and field investigations and studies. New development and condition surveys are reported in an exchange of quarterly reviews and in special reports as required, and an annual report of the program accomplishments and activities is prepared. This publication is intended to summarize all information obtained to date and to supersede all previous Bureau publications on lower cost canal lining, as well as to cover the latest developments on the subject.

2. ***Principal Program Accomplishments.***—After 16 years of organized effort under the lower cost canal lining program, improved and less costly construction procedures have been developed for use with the older types of linings and other means of seepage control; other materials have been adapted or developed for use as lining; and more than 2,570 miles of lower cost type lining has been installed as shown in table 1.[2] This total does not include the many miles of canals and laterals lined by non-Bureau operation and maintenance organizations. Evidence that both lower cost seepage control and significant savings have resulted from these efforts is found in construction records and from a comparison of construction costs based upon bid prices as presented in table 2. Factors contributing to the achievement of the program objectives are discussed below:

[1] Young, W. R., "Low-Cost Linings for Irrigation Canals," Engineering News-Record, vol. 138, February 6, 1947, pp. 192-196.

[2] The total of 2,993.8 miles shown in table 1 includes 419.9 miles of reinforced concrete lining, which is not considered a lower cost type.

TABLE 1.—*Quantities of canal linings placed on Bureau-operated projects*

Type of lining	Square yards	Miles
Exposed Linings		
Asphaltic concrete (hot and cold mixed)	276,000	42.0
Asphalt macadams	11,000	0.8
Asphalt surface membranes (prefabricated and constructed-in-place)	81,000	10.3
Other exposed asphalt linings (mortars, blocks, and slabs)	8,000	0.7
Portland cement concrete (unreinforced)	23,690,000	1,077.5
Portland cement concrete (reinforced)	9,738,000	419.9
Portland cement mortar (pneumatically applied)(shotcrete)	1,915,000	167.6
Soil-cement	37,000	3.7
Other exposed linings (concrete blocks, plastic and rubber surface membranes, etc.)	8,000	6.8
Buried Membrane Linings		
Asphalt (hot-applied)	5,839,000	333.1
Asphalt (prefabricated)	24,000	2.1
Bentonite	300,000	19.6
Plastic	22,000	1.2
Earth Linings		
Thick compacted earth	12,152,000	566.2
Thin compacted earth	1,885,000	74.3
Loosely placed earth blankets	3,207,000	174.9
Bentonite-soil mixtures	108,000	6.1
Soil sealants (chemical, petro-chemical and sediment)	535,000	41.2
Miscellaneous		
Includes resurfacing of existing linings, their undersealing and grouting, and the construction of cast-in-place concrete pipe in lieu of lining	144,000	45.8
Total	59,980,000	2,993.8

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(a) *Liberalization and Simplification of Requirements.*—The liberalization and simplification of lining specifications requirements with respect to line, grade, and finish of hard-surface linings have encouraged greater mechanization of placing equipment and resulted in lower construction costs, and, therefore, lower bid prices. Of course, a general price increase has occurred, in line with other construction costs, but the comparison still holds.

(b) *Elimination of Reinforcement.*—Savings of 10 to 15 percent in total cost have resulted from the elimination of reinforcement steel in concrete linings. Except in specific instances where structural safety is imperative, reinforcement steel is not now used.

(c) *Standardization of Canal Shapes and Sizes.*—Standardizing canal shapes and sizes, within a base width of 2 to 6 feet, has aided in the standardization of slip-form equipment requiring less capital investment, thus contributing to lower construction costs. The standardization of canal design has also resulted in lower design costs and faster completion of engineering details.

(d) *Development of Subgrade-Guided Slip-Forms.*—The development of subgrade-guided slip-forms has made it economically feasible to line small canals and laterals with hard-surface type linings which heretofore have been prohibitive in cost. These slip-forms have now been adopted in the placement of asphaltic and portland cement concrete and mortar linings.

(e) *Use of New Lining Materials and Construction Techniques.*—As shown in table 1, many materials have been adapted for use in the control of seepage from canals and laterals. Asphalt was given early consideration and has been used in many ways, providing an early breakthrough in the reduction of lining costs. It is a comparatively inexpensive material and is manufactured in many types, grades, and compositions. Numerous variations are possible in its application to the lining and waterproofing of canals, some at only a fraction of the cost of linings constructed of other materials. Steam-refined asphalt-cement is used as the binder for asphaltic concrete or is further processed to form cutbacks, emulsions, or airblown asphalts for a variety of uses. The addition of certain catalytic agents during the airblown process imparts desirable characteristics to asphalt for use as a membrane. Buried membranes of asphalt have been developed which may be constructed of prefabricated strips, or of hot, sprayed-in-place asphalt-cement covered with a protective blanket of earth or gravel, at a cost of less than one-half that of most hard-surface linings.

Bentonite is a montmorillonite type clay that has been used for many years as a lining material and under certain conditions has been very satisfactory as a low-cost lining material. Specifications have been written to cover the characteristics it is believed bentonite must possess to be most suitable for the work and for the method of construction, including the necessary earth and gravel cover that must be provided to protect the bentonite when utilized for a membrane type lining. The original cost of linings of this type in areas where deposits of bentonite are nearby has been competitive with buried asphaltic membranes.

Plastic film suitable for canal linings has been and is under test by the Bureau in field installations and in the engineering laboratories; by the Engineering Experiment Station of Utah State University, Logan, Utah;[3] and by others. Until recently, the cost of suitable plastics discouraged their general use as a low-cost canal lining material; but with lowering of production costs through improved manufacturing and fabrication practices, the plastics have become competitive cost-wise with other types of membrane materials. Several recent installations have been made on Bureau projects. While the

[3] Lauritzen, C. W., and Haws, F. W., "1959 Annual Research Report," USDA Agricultural Research Service, SWC, and Utah State University, Logan, Utah, January 1960.

TABLE 2.—*Construction costs of canal linings based on bid prices*

Type of lining	Cost per square yard[1]
Exposed Linings	
Asphaltic concrete (hot, plant mixed):	
2 inches thick	$2.30[2]
4 inches thick	4.90[3]
Asphaltic mortar, pneumatically applied:	
1½ to 2 inches thick	3.70
Asphalt macadam, 2 to 4 inches thick	2.15[2]
Asphalt, prefabricated surface membrane material:	
½ inch thick	1.55[2]
Portland cement concrete:	
2 inches thick, unreinforced	2.11
2½ inches thick, unreinforced	2.41
3 inches thick, unreinforced	2.67
3½ inches thick, unreinforced	3.73
4 inches thick, reinforced	8.11[5]
4½ inches thick, reinforced	9.49[5]
Portland cement mortar, pneumatically applied (shotcrete):	
1½ inches thick, unreinforced	1.97[2]
1½ inches thick, reinforced	2.12[2]
2 inches thick, unreinforced	2.00[2]
2 inches thick, reinforced	2.31[3]
Precast portland cement concrete blocks	3.10[3]
Plastic surface membranes, 17½ mils thick	2.00[4]
Soil-cement:	
Plastic type, 3½ to 4 inches thick	1.10[3]
Standard type, 5 inches thick	1.60
Buried Membrane Linings	
Asphalt, hot-applied, with cover of:	
Earth and/or gravel	1.10
Macadam	2.00[4]
Shotcrete	2.60[4]
Asphaltic materials, prefabricated, with cover of:	
Earth and/or gravel	1.10[4]
Macadam	2.00[4]
Shotcrete	3.00[4]
Bentonite, 1½ to 2 inches thick	1.10
Plastic material with earth and/or gravel cover:	
Polyethelene:	
3 mils thick	1.90[3]
10 mils thick	1.80[3]
17½ mils thick	2.10[3]
Polyvinyl chloride:	
8 mils thick	0.67[3]
10 mils thick	0.75[3]
Nylon fabric, coated with neoprene, with earth and/or gravel cover	3.00[3]
Earth Linings	
Thick compacted earth, 12 inches or more in thickness	0.84
Thin compacted earth, less than 12 inches in thickness	0.57
Loose earth blankets:	
Without gravel cover	0.20
With gravel cover	0.30
Bentonite-soil mixtures:	
Premixed	1.30[2]
Mixed-in-place	0.40[2]
Sealants and Stabilizers	
Waterborne type	0.25[4]

[1] Costs are for contract construction over period 1956-1960, except as noted.
[2] No representative cost data available for period 1956-1960. Cost given is for work done prior to 1956, generally by project forces.
[3] Cost represents construction by project forces and may not include equipment depreciation costs.
[4] *Cost is for experimental installations which reflect field research and study costs.*
[5] *Cost based on short reaches in vicinity of structures.*

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plastics and synthetic rubber films have not been successful when used as exposed linings, they are proving very effective and durable as buried-membrane linings so long as the protective cover remains intact.

Use is being made of relatively thick (2- to 3-foot) linings of compacted, selected earth. Where suitable earth material is available with a minimum of haul and the job is large enough to fully utilize mechanized equipment, this type of lining has proved to be one of the lowest in cost for dependable seepage control.

The use of chemicals and related materials, such as resins, lime, portland cement, asphalt, and petrochemicals, to reduce the permeability and increase the stability of soils traversed by canals and laterals is being studied extensively in cooperation with industry. Some materials have demonstrated the ability to provide impermeability and stability, but many are either expensive or toxic. However, based on laboratory and field tests, some of these chemicals appear promising and a concentrated effort is being made to further their development.

The use of waterborne chemical sealants, especially, appears to offer possibility as an economical means of lining waterways. This method, whereby the sealing material is transported by the flowing water, would be particularly adaptable in the Southwest where canals and laterals are in continuous use. A number of test installations of waterborne chemical sealants have been made, and the immediate reduction in seepage losses averaged 67 percent, the benefits decreasing with time. Full evaluation has not been completed, but it has been demonstrated that water can be saved with such sealants at reasonable cost. Further studies directed toward the development of more effective, durable, and economical sealants are underway.

The use of unreinforced, cast-in-place concrete pipe in lieu of lining has received considerable attention in the last few years. Two types of pipe have been constructed in a continuous placement operation. One type is placed in two parts; the invert is placed first, which is followed by the remainder of the pipe. The other type is constructed monolithically in a single operation by patented machines designed to travel in a previously excavated trench. Using one of these methods, pipe having a diameter of 24 to 48 inches has been constructed for little more than it would have cost to construct concrete lining for a canal of equal capacity.

(f) *Improvement of Methods of Measuring Seepage Losses.*—Development of a lining material that would be sufficiently low in cost as to permit economical lining of all canals and laterals will probably never become possible. Economics normally dictate that lining efforts be concentrated on those waterways or sections of waterways that are the more permeable and hence can benefit more by the effort. Locating the more permeable reaches of a canal or lateral has always been a difficult problem, and in many instances linings have been placed in much longer reaches than necessary because it was not possible to isolate the more permeable sections. Significant savings in lining costs could be realized if it were possible to determine accurately the amount and source of seepage losses from unlined canals.

New devices and test procedures are under development for more economical location and measurement of seepage losses. The reliability and adaptability of these methods are being established by basic investigations and field tests.

3. ***Some Direct Program Benefits.***—It is difficult to evaluate all the benefits derived from canal linings, but experiences with lining installations on the North Platte project in Wyoming and Nebraska, the Huntley project in Montana, the Riverton project in Wyoming, and elsewhere are extremely gratifying. On one project, some 2,100 acres of cultivated land had become so waterlogged by seepage from canals and laterals that it had to be abandoned. Numerous open drains had been constructed, but these were not sufficiently effective since much good farmland continued to be waterlogged and many farmsteads, including the basements in farm homes, gradually became flooded. Linings placed in canals and many principal laterals have now reduced the seepage to the point that cropping is again possible in many of the fields. Some open

drains are now being filled and the area occupied by the drains returned to cultivation. Similar benefits can be claimed on most projects where linings have been placed.

Before linings were placed in canals of the Fort Sumner project in New Mexico, the limited capacity of one canal made it impossible to get sufficient water through unlined sandy channels to serve the project area. On the Milk River project in Montana, the placement of a lining eliminated a serious seepage problem—one that was endangering a rail line by saturation of the subgrade to such a degree that the banks of the nearby river were caving and sloughing badly.

The development of new types of linings has increased the number of competitive materials, thereby stimulating a reduction of contract bid prices.

The above are but a very few of the tangible benefits achieved and the operating problems solved. Other benefits resulting from linings could be enumerated, but it will suffice to state that satisfactory lower cost lining and lining procedures have been developed, and that by the use of the lower cost type linings it has been possible to provide seepage control on waterways that never could have been accomplished with the more expensive linings in general use in the past. The cost would have been prohibitive.

4. *Future Studies.* — The planners, designers, constructors, and operators of canal systems have been provided with durable and lower cost linings developed through studies made over the past 16 years. But full use of even the low-cost types of linings now available will fall far short of meeting the ultimate objective of conserving all possible water and relieving all possible irrigable lands from waterlogging. The Bureau's continuing efforts in its lower cost canal lining program will be directed toward the following:

(1) The devices and test procedures under development for more economical location and measurement of seepage losses will be tested for reliability and adaptability, and other more precise and accurate methods will be sought so that lining requirements can be more precisely determined. Also needed are improved methods for determining probable seepage losses and lining requirements for new canals.

(2) Simplified construction procedures, directed toward lower cost, and specifications covering construction of known types of linings to assure better service will be widely disseminated.

(3) Field evaluation over the years will be continued to determine the service rendered by the various types of linings and other means of seepage control. It is only through service records that the actual cost of a lining can be determined.

(4) New developments in the relatively low-cost asphalts, plastics, and related materials will be closely followed with a view to their possible application as linings. Many such new products are continually being developed.

(5) Search will be continued for improved sealants as the most economical means of providing more universal linings and watertight canals and laterals. The ideal sealant would be one which is low in cost, effective, durable, and easily applied, and of the several types, the chemical sealants appear to be most promising. The assistance of the world's technical and scientific investigators is being enlisted in exploratory investigations to develop improved waterborne chemical sealants. As a part of this effort, all known data on the subject have been assembled into a single reference volume that has been given wide circulation.[4]

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[4] Blackburn, W. C., "A Review of the Use of Chemical Sealants for Reduction of Canal Seepage Losses—Lower Cost Canal Lining Program," Analytical Laboratory Report No. CH-102, Bureau of Reclamation, February 9, 1960.

Planning

5. ***Justification for Lining.***—Even considering the apparent benefits cited in the previous chapter, justification for a particular lining installation may be a complicated procedure. Information on which to base a decision may be insufficient and incomplete, and the benefits that may accrue can seldom be predicted as in the cases cited. It is known, however, what the various linings that have been developed can be depended upon to accomplish; where the different kinds of linings should be and should not be used; how they should be constructed; and how much it will cost to construct them. One presently existing intangible factor is the ultimate cost, which is the sum of the original cost and the cost of maintaining the lining in a serviceable condition. Sufficient information has not yet been accumulated on the costs of maintaining linings nor on the length of time they can be maintained satisfactorily without replacement. Effort is being made to accumulate such data.

6. ***Some Factors Affecting Lining Selection.***—From the advantages and disadvantages of the several types of linings, presented in subsequent discussions, it will be seen that no single type of lining can be recommended for all conditions encountered and that all linings require some maintenance. The planner and designer must take this into consideration, and they, along with the operation and maintenance organization, have a responsibility in the final choice of the type of lining to be used. The planner should include careful determination of land and water values as projected into the future. Many presently constructed unlined conveyance channels would probably have been lined or constructed as closed conduit systems had proper determination been made of future water and land values.

The quantitative determination of seepage in an existing channel and the location of probable areas in a proposed canal or lateral through which seepage will occur, are of major importance to the planner and designer in selecting the reaches of channel to be lined. Chapter III discusses the several accepted methods for such determination, together with their advantages and disadvantages; also some new methods under development which show considerable promise in the location and, in some cases, measurement of seepage in existing canals. The new methods offer advantages of economy over the accepted ones.

Preconstruction investigations should delineate poor subgrade conditions. Permeable soils and soils which may expand or settle upon becoming wet or saturated, or those through which piping may occur, should specifically be indicated during preconstruction and closely observed during construction. Soils not so indicated during preconstruction planning should be given attention by the construction forces to assure the location of all soil areas that may require special treatment.

If linings are constructed at the time of original construction of a project, and loss of water from the canals and laterals is thereby greatly reduced, the sizes of the associated dams, reservoirs, canals, and laterals also can be reduced. The resultant savings would pay, in part at least, for lining of the smaller irrigation system which would deliver the same amount of water, in some cases without extensive drainage construction.

In choosing the type of lining to be included in a particular canal design, the designer must anticipate (1) the service requirements, including the capacity to meet the peak flow demands; (2) provision of facilities which will permit satisfactory delivery of water to adjacent lands; (3) safety to property below the canal, including protection against damage by seepage losses; and (4) most important, conservation of the water supply.

A. TYPES OF LININGS

7. ***General.***—Some of the advantages and disadvantages of the various types of linings available for consideration by the designer are briefly discussed below:

8. ***Hard-Surface and Exposed-Membrane Linings.***—For convenience, these linings are grouped together and discussed in chapter IV under the general heading of "Exposed Linings," the classification being defined in section 19.

Hard-surface linings have been constructed of portland cement concrete and mortar, asphaltic concrete and mortar, prefabricated asphaltic blocks, brick, stone, and soil-cement. They are generally the more costly linings initially, with reinforced portland cement concrete linings being the most costly of those used and usually recommended only where structural safety is a primary consideration.

(a) *Portland Cement Concrete.* — Portland cement concrete is more resistant to erosion than most other lining materials; therefore, it is preferable for higher water velocities. A properly designed and constructed reinforced concrete lining will withstand velocities of any magnitude considered feasible for canals. Linings of concrete, whether reinforced or unreinforced, eliminate weed growth with resulting improvement in flow characteristics and reduction in maintenance costs. Further, burrowing animals, which cause numerous breaks in unlined canals and in canals lined with some types of materials, cannot penetrate concrete.

Portland cement concrete, in general, is susceptible to damage from alkali water and from alternate freezing and thawing action. Concrete linings are susceptible to rupture by outside hydrostatic or other pressures. They will withstand a small amount of cracking to relieve external hydrostatic pressure without significant damage; however, drainage to relieve the outside hydrostatic pressure is generally worth the additional cost.

Unreinforced concrete linings have been constructed at a significant reduction in cost as compared with reinforced concrete linings. Velocities up to 8 feet per second are permissible with adequate water depth. Unreinforced concrete linings are more susceptible to damage by hydrostatic or other pressures under the lining than are reinforced concrete linings, but not to the degree that the difference in cost might indicate. Where unexpected hydrostatic pressures are encountered under the lining, unreinforced concrete will rupture more readily than reinforced concrete, thus relieving the pressure and reducing the area of damage.

A distinct disadvantage of concrete lining is its lack of extensibility, which results in frequent cracks as contraction takes place from drying, shrinkage, and temperature change. Although rather extensive studies are now underway, an entirely satisfactory material for filling and sealing such cracks has not been developed to date.

(b) *Shotcrete.*—Portland cement mortar has many of the characteristics of portland cement concrete. Such mortar is usually pneumatically applied, that is, shot into place by pneumatic pressure,[1] and when so applied is termed shotcrete. Shotcrete linings have the principal advantage of being more easily placed over a rougher subgrade than concrete linings, and therefore, are particularly adapted to use in existing rock cuts where trimming to exact line and grade would be very expensive. The lining may or may not be reinforced with steel, but it has been observed that reinforcement may add many years of satisfactory service to shotcrete linings placed over earth subgrade. Reinforced shotcrete, 1 to 1½ inches thick, can usually be applied for about the same cost as unreinforced concrete twice as thick. Since it is seldom economical to place shotcrete linings thicker than about 2 inches, their use should be limited to small canals or to mild climates where service requirements will not be severe.

Being generally constructed thinner than concrete, shotcrete linings are more readily damaged by hydrostatic pressures and by settlement, expansion or shrinkage of the subgrade. Furthermore, the inherent difficulty in controlling the thickness of the shotcrete application may

[1] Chadwick, W. L. (Chairman), McCrory, J. A., and Young, R. B., "Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure," Committee 805, Proceedings ACI, vol. 47, p. 185, 1951.

result in a lining with areas where the thickness is less than specified, which areas are therefore areas of potential weakness.

(c) *Soil-Cement.*—In mild climates, very good service has been obtained with canal linings made of a mixture of portland cement and a natural sandy soil available at the site, sometimes at a considerable savings as compared with portland cement concrete.

(d) *Asphaltic Concrete.*—Asphaltic concrete may be an economical substitute for unreinforced portland cement concrete in small canals where the cost of asphalt is sufficiently low to offset a possible shorter total life expectancy, and where the aggregate available is suitable for asphaltic concrete but not of sufficiently high quality for portland cement concrete. Further possible advantages of asphaltic concrete are the greater ability of the lining to adjust itself to subgrade changes and the fact that it can be placed in cold weather with a minimum of protection whereas portland cement concrete work must be suspended when freezing conditions exist. This cold weather placement possibility is an advantage in lining canals which must be kept in operation during the irrigation season and, hence, must be lined during weather usually unfavorable for placement of portland cement concrete.

Asphaltic concrete also has been used successfully to repair concrete linings by placing a 1½- to 2-inch-thick resurfacing layer of asphaltic concrete over disintegrated and deteriorated surfaces of portland cement concrete.

Like other relatively new experimental type linings, the life of asphaltic concrete linings is not yet known. Hence, a first cost advantage of this type of lining over portland cement concrete lining might make the use of asphalt appear the more feasible, although ultimate annual costs might actually be higher for the asphaltic lining. Velocities in asphaltic concrete linings must be limited to a maximum of about 5 feet per second, and there is danger of weed growth puncturing or moving the lining unless soil sterilants are used prior to lining placement. This type of lining may also have insufficient resistance to external hydrostatic or soil pressures — a deficiency shared with unreinforced portland cement concrete linings.

(e) *Masonry Type Linings.*—Brick, stone, and rubble masonry linings have never been used widely by the Bureau. Because of the great amount of hand labor involved and the increased cost of such labor in the United States, linings of this type would now be very costly. Some rock masonry linings were placed on Bureau projects during the depression years of the 1930's by Civilian Conservation Corps forces and have given good service with little maintenance expense. Brick linings have been used rather extensively in India where there is an abundance of relatively inexpensive hand labor; they are reported to be very satisfactory.

(f) *Exposed Asphaltic Membranes.* — Thin sprayed-in-place asphalt cements; and prefabricated sheets and rolls of asphaltic materials have been tried experimentally but with limited success, as they are very subject to injury. Tests are continuing with these exposed-membrane linings.

(g) *Exposed Films of Plastic and Synthetic Rubber.*—Films of plastics and synthetic rubber suitable for canal linings have been and are under test. Many of the plastics tested and installed experimentally as exposed-membrane linings have shown low resistance to puncture, and some types disintegrate rapidly upon exposure. Thicker plastics and synthetic rubber with greater resistance to these forces are more expensive. Butyl rubber sheets, 30 and 60 mils thick, have been installed and although costly are proving very serviceable as canal and pond liners. Vandalism has been a problem with these materials in some areas. Sections of the material have even been removed.

9. ***Buried Membrane Linings.***—Hot-applied asphalts, prefabricated asphaltic materials, plastics, and layers of bentonite or other types of clays when placed as buried membrane linings are low in original cost. Another advantage is that in new construction a decision to line a canal or lateral can be deferred until the excavation is in progress and the need for the lining has been definitely established.

Membranes must be protected from damage, which is usually accomplished by covering the membranes with earth, gravel, or both. This has a disadvantage of limiting the permissible water velocity to avoid erosion of the cover.

Another disadvantage is that additional maintenance may be necessary in the control of weeds and willows which can grow in the cover material. Roots of willows have been observed to have punctured asphaltic membranes. For this reason, very little lining of this type has been constructed in hot climates where weed growth is a major problem. Earth covers of gravelly material will reduce the danger from all these factors, but they may increase the cost depending on the availability of gravel or gravelly materials.

A present disadvantage of buried membrane type linings is the uncertainty of the life of the membrane materials as compared with concrete linings. Some bentonite membranes have been in service 22 years, the first linings of this type being installed in 1940. Buried hot-applied and prefabricated asphaltic membranes have only been in service 13 to 15 years and plastics only 9 years.

(a) *Hot-Applied Buried Asphaltic Membranes.*—Hot-applied buried asphaltic membranes should provide one of the tightest linings developed to date, and recent tests of the hot-applied membranes constructed of catalytically blown asphalt cements indicate that little change in the membrane materials has occurred since its application. Positive assertion that these membranes are watertight cannot be made since few seepage tests of representative field installations have been performed. However, improved conditions in the land below such installations indicate a high degree of effectiveness of the linings. Additional advantages of these asphaltic membranes are low first cost, from one-third to one-half that of hard-surface linings; adaptability to placing in cold or wet weather; and adaptability to placing in large quantities with simple and mobile equipment. The latter two features are very advantageous on operating projects where canal work must be done quickly in the period between irrigation seasons, often when weather would not permit placement of other types of lining materials.

(b) *Prefabricated Buried Asphaltic Membranes.*—Prefabricated buried asphaltic membranes, as compared with the hot-applied asphaltic membranes, are more practicable for small jobs for which the shipment of hot asphalt cannot be economically justified. Some of the advantages cited above for the hot-applied asphalt membranes also apply, and the lining can be installed with a minimum of equipment by project maintenance personnel.

It is doubtful whether the prefabricated asphaltic materials can be made as watertight as the hot-applied asphalts because of the many joints that must be made between adjacent strips in the narrow width material (usually 36 inches), although tests made over a number of years indicate that some materials provide adequate impermeability over extended periods. There is a problem of deterioration to consider where materials using organic reinforcement fibers and fillers are used. Fibers and fillers of inorganic materials, on the other hand, have resisted deterioration.

(c) *Plastic and Synthetic Rubber Films.*—The plastic film is an essentially watertight material, even at a thickness of only 1½ mils (0.0015 inch), and it has a high resistance to rupture and rot. Some plastic films with thicknesses of 3 to 20 mils placed as buried membrane linings in 1953 on the Bureau's Huntley project in Montana are performing well. However, to avoid damage during placement, the film should have a thickness of 6 to 8 mils.

With lowered production costs, plastic materials are now competitive cost-wise with other membrane type lining materials and several installations have been made in which 8-mil-thick, black-pigmented, polyvinyl plastic film was used as a buried membrane. The film is light in weight and can be installed by project personnel with ordinary maintenance equipment.

Synthetic rubber also has been used as a buried membrane, and plans by one manufacturer to produce a thinner (15-mil) material could result in a less costly satisfactory lining of this durable material.

(d) *Bentonite Membranes.*—The success of a bentonite membrane appears to be directly related to the quality of the bentonite. There is some indication that the membrane may deteriorate in the presence of hard water. Adequate coverage of the perimeter of the canal is important, and is sometimes difficult to achieve except by careful placement. Placement can be costly if the work must be even partially accomplished by hand. However, many miles of canals and laterals have been lined with bentonite membranes and most are performing very well.

10. *Earth Linings.*—Included under this subject are linings composed of thick compacted earth, thin compacted earth, loosely placed earth blankets, bentonite-soil mixtures, and soil conditioners and admixtures. Sediment sealants have been and are continually used in the silting of leaking channels, particularly during the original puddling and priming of newly constructed waterways. The introduction of silt and other sediments into the flowing water is inexpensive and will reduce seepage to some degree; but generally it has been found that this type of treatment is of only temporary benefit unless the deposited sediment is protected from scour.

(a) *Thick Compacted-Earth Linings.*—Where suitable materials for the construction of a thick compacted-earth lining are available at the jobsite (sec. 45), this is likely the lowest cost permanent type of lining with respect to both first and ultimate costs for use on large canals. A thick compacted-earth lining has an advantage not possessed by any other type of lining in general use. Because of its weight and plastic characteristics, it can withstand considerable hydrostatic pressure without loss of effectiveness, and it can be used in many instances without drains under the lining in areas where the canal prism intersects the ground water table. For similar reasons, a thick compacted-earth lining can be used to advantage over expansive clays which disrupt more rigid type linings. Another distinct advantage of thick compacted-earth linings is the ease of constructing partially lined sections or reaches, as required to cut off permeable strata or areas. The earth lining blends in with the unlined earth sections.

Careful inspection and construction control is required in the construction of thick compacted-earth linings. The soil must be homogeneous when placed, of proper thickness, and compacted at proper moisture content to the prescribed density. Furthermore, if the impermeability which can be attained by available soils does not provide the seepage control desired, the thickness can be increased. If the construction control specified by the Bureau is properly exercised, reasonably low permeability can be normally assured. The thickness of this type of lining is an advantage in that small deficiencies, such as cracks or holes, would not seriously affect the efficacy of the lining as they would were thinner linings employed.

Inspection to determine the location of leaky areas may be more difficult with compacted-earth linings, as compared with hard-surface linings, but repair of such linings is often easily accomplished with normal operation and maintenance equipment.

In severely cold climates, it is possible that frost action may penetrate the entire depth of a compacted-earth lining and destroy the compaction in the same manner that frost restores tilth to clayey farm soil. Although such damage to this type of lining, except for the surface layers, has not actually been experienced, field tests are now being made to determine the effects of frost. Pending results of the tests, it is recommended that frost-susceptible soils be avoided in the colder climates. A similar question may be raised with respect to the loss of compaction in mild climates by the passage of time. However, it appears doubtful that such action will take place except possibly over a very long period of time.

In the design of the canal section for a thick compacted-earth lining, some caution should be used in setting maximum velocities and the maximum degree of curvature to avoid scour, unless satisfactory gravel is available for cover.

(b) *Thin Compacted-Earth Linings.*—Thin compacted-earth linings have some of the characteristics of the thicker linings of this type discussed above. However, they are not well suited for use with certain types of soils which are subject to severe frost action, and extra precaution must be taken to protect the relatively thin linings (usually 6 to 12 inches) from scour and erosion. The Bureau has not used this thinner type of compacted-earth lining to any appreciable extent in recent years, because of the cited problems and the risk of damage that can result from cleaning operations inherent with unlined and earth-lined channels.

(c) *Loosely Placed Earth Blankets.*—Loosely placed earth blankets, unless the soil used is highly impermeable and stable, have limited use. Most frequent use is made of this type of lining on Bureau projects to correct temporary or emergency seepage conditions. The blanket must be protected from scouring and eroding action of the flowing water and the elements by use of stable gravel or gravelly materials.

(d) *Soils With Admixtures.*—Under certain conditions, to provide impermeability and stability to existing or available soils in the area, bentonite or other materials such as portland cement and asphalt emulsoins have been added to soils in the construction of earth linings. Usually this has been done during the construction of compacted type earth linings and may be feasible under some conditions, but the increased cost of the material and its transportation and the cost of thoroughly mixing the additive before placement may be prohibitive. Few linings of this type have been placed, except on an experimental basis.

B. SOIL SEALANTS, STABILIZERS, AND OTHER MEANS OF SEEPAGE CONTROL

11. ***Soil Sealants and Stabilizers.***—Water-borne, mixed-in-place, spray-applied, and subgrade-injected sealants have all been considered as a means of waterproofing or sealing soils to reduce their permeability as well as provide soil stability. Some of those used have provided some benefit and through the efforts of the Bureau [2] attention has been again focused on the possible use of sealants to provide an even less costly method of seepage control. Several manufacturers have been studying such materials as resins, petroleum-based emulsions, plastics, and other related compounds. The future development of a suitable product appears likely, but as yet the designers of our irrigation systems cannot plan on their use; and until field testing and evaluation has proceeded much further, the more conventional linings must be used.

12. ***Other Means of Seepage Control.***—In addition to linings and soil sealants, other means have been used advantageously to control the seepage from channels. Emulsified or hot liquid asphalts and portland cement grouts have been injected under pressure into crevices, joints, and open channels in rock, shattered shale, gravel, sand, and other water permeable materials. The undersealing of hard-surface linings has been accomplished with portland cement grout and asphalt. Cutoffs of portland cement concrete, asphalt, and plastic sheets have been constructed by trenching and installation of the material in the excavated trench to intercept the flow of water from a channel. These and other measures have been taken primarily to correct individual problems and are not widely used because of their high cost, temporary benefit, narrow field of application, or lack of development.

Unreinforced, cast-in-place, concrete pipe having diameters of 24 to 48 inches is now in use where the hydrostatic head does not exceed about 15 feet and where stresses due to fill load, vibration caused by heavy traffic, etc., are not excessive. The pipe costs little more than would concrete lining for a canal of equal capacity. Less right-of-way is required for the concrete conduits and many maintenance problems common to open channels are eliminated. Moss problems are reduced, the drowning hazard in populated areas is eliminated, and lands occupied by open channels can be cultivated and placed in crops.

[2] Blackburn, W. C., "A Review of the Use of Chemical Sealants for Reduction of Canal Seepage Losses—Lower Cost Canal Lining Program," Analytical Laboratory Report No. CH-102, Bureau of Reclamation, February 9, 1960.

Seepage Investigations

13. *General.*—The primary function of most canal linings is to control seepage. Although erosion resistance, safety, or reduced maintenance may be of greater importance in special cases, extensive lining installations are not usually justified if seepage losses are low.

A discussion of the general economics of canal lining will be found in chapter IX. As explained in that chapter, there will be considerable economic advantage to the project if the decision to line the canal sections is made in the preconstruction planning stage rather than at some later date when the need becomes apparent during operation. To evaluate seepage prior to or during construction, then, becomes a very important element of the investigations.

After the project is put into operation, measurements may be required to determine overall seepage losses or losses from certain reaches, or tests may be required to locate the point of origin of ground water causing damage to property. Each of these items presents a separate problem, but methods have been developed to provide usable solutions to each. Admittedly, much remains to be done to improve the accuracy of measurements and to reduce the cost of making them. Investigations are in progress that may give the designer more accurate, versatile, and economical tools to achieve that end.

14. *Preconstruction Permeability Tests.*—The decision to line or not to line a proposed canal often can be reached from visual observations of the soil, provided the soil is of a type that is obviously very pervious or impervious. When the permeability of the canal subgrade materials is in doubt, however, inplace field permeability tests provide a basis for estimating potential seepage losses and deciding the necessity for lining.

(a) *Well-Permeameter.* — The well-permeameter has been used by the Bureau for many years to obtain onsite permeabilities. The test consists of determining the steady-state outflow of water from an uncased well in which the water surface is maintained at constant elevation. Whenever possible, the test is conducted along the canal centerline with the well bottom at canal grade and the water surface at the proposed operating level of the canal.

A photograph and a drawing of the well and equipment used in the well-permeameter test are shown in figures 1 and 2. The well is drilled by an auger, and is backfilled nearly to the operating water surface with a pervious sand which prevents the well from caving. A section of casing with a conical top, providing a chamber in which a float may operate, is lowered into the well and pervious sand is backfilled around it. Through a chain linkage and operating arm, the float actuates a valve on a 50-gallon drum reservoir; thus, a constant water level in the well can be maintained. A length of flexible hose con-

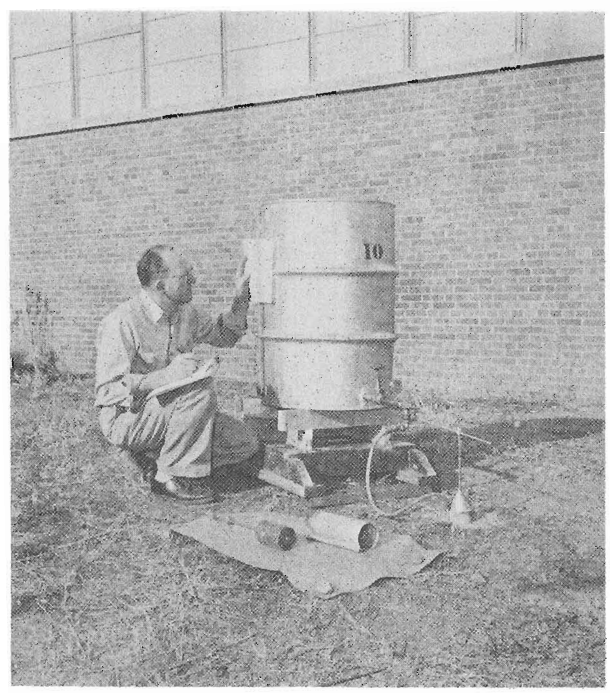

Figure 1.—Well permeameter test apparatus used to measure soil permeability in the bottom of a proposed canal, from which seepage losses may be estimated. PX-D-16608.

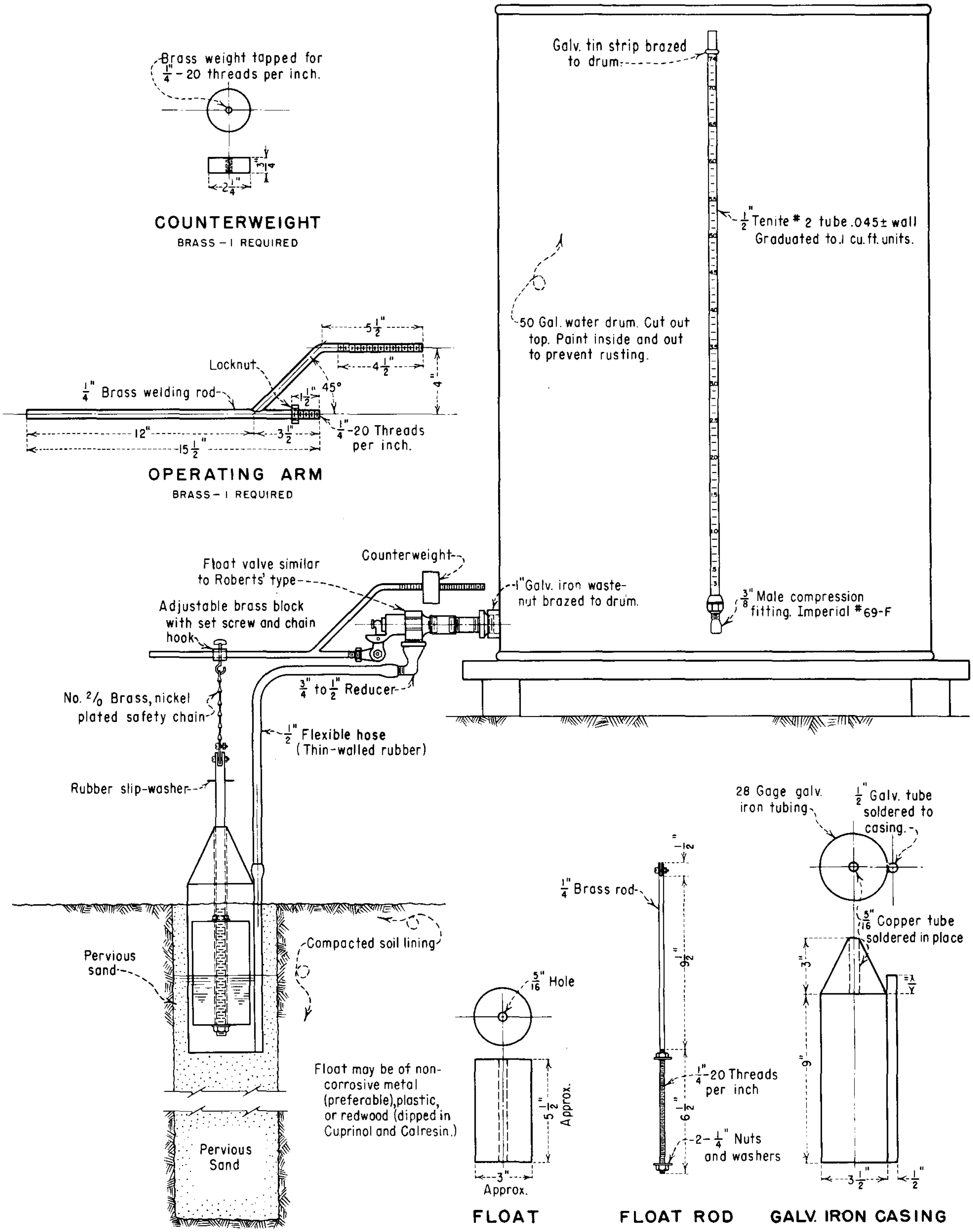

Figure 2.—**Well permeameter equipment for measurement of soil permeability in a canal. 101-D-38.**

ducts the water from the valve to the well. Variations in the desired water level can be accomplished by adjusting the length of chain. The scale indicating level in the water reservoir is graduated to show cubic feet and fractions thereof directly. During a test the volumes of water discharged into the well in measured time intervals are recorded, and a rate of discharge is calculated when the inflow becomes constant. From these data, well dimensions, and prepared nomographs, the permeability coefficient for the soil can be determined. The complete test procedure with required nomographs is contained in the latest edition of the Bureau's Earth Manual.[1]

Some preliminary exploration of the subsurface is required for suitable selection of test sites. The soils where tests are conducted should be representative, and the presence of ground water or impervious soil layers should be known for use in permeability calculations. The test is better used in unsaturated soils, although with some conditions saturated soils under the influence of high ground water can be tested.

After the average coefficient of permeability of the soil and the dimensions of the proposed canal have been determined, the seepage in cubic feet per square foot per day (sec. 17) may be estimated by one of the several formulas available.

15. ***Seepage Measurements After Construction—Accepted Methods.***—Currently accepted methods of measuring the quantity of water lost by seepage from existing canals are limited to ponding, inflow-outflow, and seepage meter determinations. Each method has advantages and limitations. No single method is adaptable to all conditions encountered in the field.

In normal operation of a canal, evaporation losses are generally considered negligible. On this subject, Samuel Fortier states[2] 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." In conducting seepage measurements, however, evaporation may be an important factor, as explained below.

[1] "Earth Manual," first edition, Bureau of Reclamation, 1960.

[2] Fortier, Samuel, "Use of Water in Irrigation," McGraw-Hill, New York, N. Y., 1916, p. 111.

(a) *Ponding Method.*—The ponding method is the most accurate and dependable method of determining seepage now known. Temporary watertight dikes or bulkheads are used to isolate reaches of a canal, water is impounded between the two dikes, and the time rate of drop in the water surface is measured. The rate of drop and the physical dimensions of the ponded reach provide the data necessary to compute the seepage loss in cubic feet per square foot of wetted area per 24 hours. To obtain satisfactory results, the ponded reach must be selected so as to avoid inflow or outflow which cannot be accurately measured.

A modification of the ponding method consists of adding water to the pond to maintain a constant water surface elevation. The accurately measured volume of water added is considered to be equal to the seepage loss, and the elapsed time establishes the rate of loss.

Measurement of evaporation may be necessary when ponding a lined reach in which losses are low and evaporation may be rapid, particularly if a comparison of seepage rates before and after lining is to be made. Ponding tests are normally suspended during periods of precipitation.

(b) *Inflow-Outflow Method.*—The inflow-outflow method utilizes measurements at the upstream and downstream ends of the reach being studied and is no more accurate than these measurements. The quantities of water flowing into and out of the reach of canal are carefully measured, and the difference is attributed to seepage. Existing calibrated weirs or Parshall flumes in the canals can be used for measuring flows. Where permanent installations such as these are not available, or are not located at convenient points, temporary weirs or gaging stations can be installed. Temporary weirs introduce considerable loss in head, which may make their use impracticable. Current meters are used at gaging stations to measure the velocity, from which the rate of flow is derived. When seepage tests are of long duration or when the tests are to be repeated in the future, the gaging stations should be rated. Water stage recorders and the rating curve can then be used to determine flows without frequent recourse to current meter gagings. Flow measurements by the inflow-outflow method are not sufficiently accurate for the close determination of seepage losses in short reaches of canal.

(c) *Seepage Meter Method.*—The seepage meter (fig. 3) is a modified version of the constant-head permeameter developed for use under water. It consists of a watertight seepage cup connected by a plastic tube to a flexible water bag. Water flows from the bag into the cup where it seeps through the 2 square feet of canal subgrade area isolated by the cup. By keeping the water bag submerged, the heads on the areas within and outside of the cup are equal. The seepage rate may be computed from the weight of water lost in a known period of time and the area under the meter.

The seepage meter is not considered an accurate means of measuring seepage loss. Its main value lies in determining approximate locations of relatively high seepage losses. If tests are made at close intervals throughout a reach, a better indication of the average loss rate can be determined.

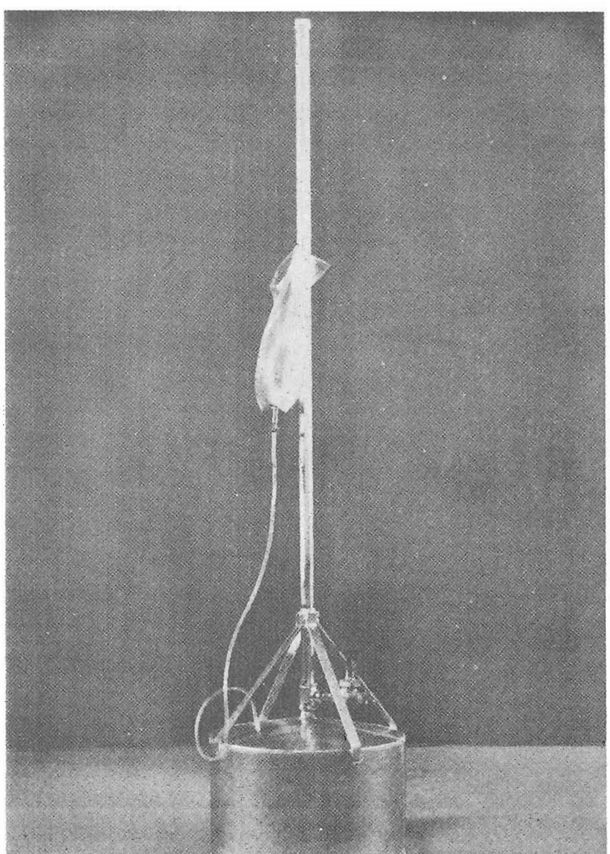

Figure 3.—Seepage meter with plastic bag for use in unlined operating canals. P20-D-21556.

(d) *Limitations of Accepted Methods.*—Since the ponding method of determining seepage requires that the canal be taken out of operation during the tests, this method is best adapted to localities where irrigation is seasonal in nature. Ponding can be conveniently accomplished during the off season, but caution is necessary to avoid making tests at extremely low temperatures or under other conditions which differ appreciably from those prevailing during the operating season. The inflow-outflow method can be employed without interfering with the irrigation schedules. However, when deliveries are made from turnouts in the reach being studied, additional errors in measurement of these flows are likely to be incurred, and they will affect the accuracy of seepage determinations. Seepage meters can be used during normal canal operation; but their use is restricted to unlined or earth-lined canals, whereas ponding and inflow-outflow methods can be used in either lined or unlined canals. Ponding tests can be utilized to investigate short reaches of canal, but the inflow-outflow method requires a reach long enough to obtain measurable losses. The length of pond should be so limited that upstream and downstream depths do not differ appreciably.

16. ***Factors Affecting Seepage.***—Interpretation of the results of tests of seepage from a canal requires a knowledge of the factors affecting it. Principal among these are the following:

(1) Character of material (including permeability) traversed by the canal.
(2) Deposition of silt.
(3) Depth of water in canal.
(4) Relative extent of wetted area.
(5) Inflow of seepage water.
(6) Location of water table relative to the canal.
(7) Percentage of entrained air in the soil.
(8) Velocity of flow.
(9) Action of capillarity and gravity.
(10) Temperature of soil and water.
(11) Ground slope at right angles to direction of canal flow.
(12) Chemistry of the soil and water.

The effects of many of these factors are difficult to determine, and to evaluate their relative importance may require large expenditures of funds and time.[3] Therefore, the factors thought to be least important to seepage loss, such as soil temperatures, velocity of flow, and ground slope at right angles to the flow, generally are not taken into account. Knowledge of fluctuations in ground-water elevations will aid in correlating such fluctuations with the water surface in the canal. Generally, however, even if all the above data could be collected, the effects of these items on the seepage rate are so closely interwoven that it is practically impossible to separate one from another.

17. ***Results of Seepage Tests on Some Bureau Projects.***—Few accurate measurements of seepage losses had been made on Reclamation projects prior to activities in the lower cost canal lining program. Annual project reports contain estimates of seepage losses obtained from measured diversions and deliveries and approximations of wastes. However, these were only estimates and had no accurate basis. The measurement of seepage losses, therefore, was made part of the lower cost canal lining program.

Tables 3, 4, and 5 have been prepared to summarize the results of seepage investigations conducted as part of the program. Table 3 gives comparisons of seepage rates determined from both ponding and seepage meter tests. Table 4 is a compilation of seepage data related to various types of canal lining, and shows before-and-after lining comparisons when these are available. Table 5 contains miscellaneous measurements obtained by several methods that do not fall in the categories of tables 3 and 4. Most of the seepage losses shown in the tables are expressed in cubic feet per square foot of wetted area per 24 hours. The wording of this unit of seepage loss is long and the abbreviation cfd (cubic feet per day) will be used. A few values are expressed in percent per mile and are not directly comparable to the other unit. In these cases insufficient data were available for converting from percent per mile to cubic feet per square foot per 24 hours.

Discussion of all seepage studies in these tables is not warranted in this publication; however, a few typical tests are described below.

(a) *North Platte Project—Wyoming.*—The seepage studies made on the Fort Laramie and Interstate Canals of the North Platte project were initiated because of the visible evidence of seepage in low lying areas near the canal and the need for rehabilitation of the system. Three methods of measuring seepage were utilized: ponding, inflow-outflow, and seepage meter supplemented with ground-water table observations. During the irrigation season of 1949, the inflow-outflow method was used on the Fort Laramie Canal in an effort to determine seepage rates. Gaging stations at miles 35.0 and 39.7, the extreme limits of the reach being tested, were established using current meters for calibration. Fifteen turnouts, located in the reach, were used intermittently or continuously throughout the testing period to make water deliveries to farms. These deliveries were measured by permanent weirs, and the quantities were taken into account in computing the seepage rates. A consistent rate was not found, due in part to inaccuracies of measurements at the 15 turnouts; the weirs used were in disrepair, and each of the 15 measurements probably introduced errors. Also, the accuracy of the gaging station may not have been adequate for the tests.

After the 1949 irrigation season, the ponding technique was applied within approximately the same reach of the Fort Laramie Canal. Seven earth dikes were constructed across the canal to form six ponds. The end dikes were located at miles 36.2 and 38.3 (stations 1911+08 and 2022+84). The rates of drop of the water surfaces in the ponds, together with the physical dimensions of the canal, were used to compute seepage rates for each pond. Evaporation measured during the tests proved to be of negligible importance in computing the rates. The seepage rates for each pond are listed in table 3, in cfd. A number of tests were also performed on various laterals of the Fort Laramie system, as shown in the tabulation.

In conjunction with the ponding tests, seepage meters were used in each pond to advance the Bureau's knowledge of this device and to obtain data for comparison of the two methods of seepage measurement. Several settings were made in each pond to obtain an average rate of loss. The averages only are shown in the table along with losses measured by ponding. In most cases,

[3] Rohwer, Carl and Stout, O. V., "Seepage Losses from Irrigation Channels," Technical Bulletin 38, Colorado Agricultural Experiment Station, Colorado State College, Fort Collins, Colo., March 1948.

TABLE 3.—*Summary of seepage tests—A comparison of results for different methods of measurement*

Location of test reach	Date	Soil type	Method of measurement and seepage rate (cfd)[1]				Length of reach tested, feet	Designed[3]			Remarks
			Ponding		Seepage meter						
			Percent design depth[2]	Rate	Percent design depth	Rate		Capacity, second-feet	Depth, feet	Wetted perimeter, feet	
Central Valley project, California											
Friant-Kern Canal											
Station 2791 + 20 to 2820 + 24	1950	Sandy silt	100	0.067	100	0.064	2,904	4,000 to 5,000	17.2	119	Bottom compacted to 2-foot depth, slope 8 feet horizontally.
North Platte project, Wyoming											
Ft. Laramie Canal											
Station 1911 + 08 to 1938 + 40	1949		63	0.35	35	0.46	2,732	1,200	9	85	Bank grouted with portland cement prior to ponding
Station 1938 + 40 to 1985 + 34	1949		46	0.30	32	0.32	4,694		9	85	
Station 1985 + 34 to 1997 + 90	1949		65	0.40	33	0.25	1,256		9	85	
Station 1997 + 90 to 2001 + 50	1949		82	0.25	41	0.10	360		9	85	
Station 2001 + 50 to 2014 + 61	1949	Silt	72	0.13	40	0.10	1,311		9	85	
Station 2014 + 61 to 2022 + 84	1949	Silt	70	0.18	43	0.20	823		9	85	
Ft. Laramie Lateral 29.4											
Station 6 + 10 to 11 + 98	1949		90	0.38	85	0.25	588		—	—	
Station 11 + 98 to 21 + 82	1949		90	0.29	80	0.31	984		—	—	
Station 21 + 82 to 24 + 44	1949		90	0.23	90	0.28	262		—	—	
Ft. Laramie Lateral 90.4											
Station 130 + 18 to 139 + 97	1949		100	0.63	80	0.69	979		—	—	
Station 145 + 00 to 172 + 74	1949		100	0.45	85	0.15	2,774		—	—	
Station 172 + 74 to 184 + 46	1949		90	0.23	85	0.25	1,172		—	—	
Station 197 + 36 to 201 + 81	1949		90	0.44	80	0.03	445		—	—	
Station 201 + 81 to 210 + 44	1949		95	0.15	90	0.08	863		—	—	
Interstate Lateral 24A											
Station 0 + 89 to 7 + 34	1949		100	0.86	80	1.70	645		—	—	
Station 7 + 34 to 15 + 44	1949		100	0.57	90	1.82	810		—	—	
Station 15 + 44 to 21 + 67	1949	Fine to medium sand	100	0.35	90	0.26	623		—	—	
Station 21 + 67 to 36 + 46	1949		95	0.55	70	1.01	1,479		—	—	
Station 36 + 46 to 44 + 05	1949		95	0.58	80	0.23	759		—	—	
Station 44 + 05 to 51 + 27	1949		100	0.37	85	0.22	722		—	—	
Riverton project, Wyoming											
Wyoming Canal											
Station 1659 + 71 to 1754 + 34	1950	Shattered siltstone and sandstone	94	0.50	94	0.42	9,463	566	—	—	
Station 1754 + 66 to 1801 + 00	1950	Silty sand with outcropping of sandstone	92	0.46	88	0.88	4,634		—	—	
Station 1801 + 34 to 1896 + 59	1950	Medium to fine sand	97	0.72	90	0.28	9,525		—	—	
Station 1896 + 91 to 1964 + 50	1950	A transition zone of medium to fine sand to sandstone and a section of sand with high clay content with strata of gravel	94	0.35	94	0.37	6,759		—	—	

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TABLE 3.—*Summary of seepage tests—A comparison of results for different methods of measurement*—Continued

Location of test reach	Date	Soil type	Method of measurement and seepage rate (cfd)[1]				Length of reach tested, feet	Designed[3]			Remarks
			Ponding		*Seepage meter*						
			Percent design depth[2]	Rate	Percent design depth	Rate		Capacity, second-feet	Depth, feet	Wetted perimeter, feet	
Riverton project, Wyoming (Continued)											
Wyoming Canal (Continued)											
Station 1974+82 to 1985+50	1950	Sand with a moderate amount of fines. Heavy gravel located about 8 inches below canal bottom	95	0.55	90	0.64	1,068		—	—	
Station 1986+43 to 2000+50	1950	Sand with high percentage of fines	84	0.46	84	0.38	1,407		—	—	
Tucumcari project, New Mexico											
Conchas Canal											
Station 2518+40 to 2562+00	1949	Lean clay lining	81	0.40	27	0.10	4,300	700	—	—	
Station 2562+40 to 2587+00	1949	Lean clay lining	81	0.07	46	0.05	2,460		—	—	Bottom compacted to 1-foot depth, slope 3 feet horizonally; clay lined on left side and bottom.
W. C. Austin project, Oklahoma											
Altus 6.8 Lateral											
Station 2+50 to 8+33	1950	Sandy silt	100	0.71	100	0.56	583		—	—	
Station 8+33 to 12+16	1950	Silty sand		1.54		2.82	383		—	—	
Missouri River Basin project, Nebraska											
Courtland Canal											
Station 439+00 to 447+00	1952	Silt	94	1.15	83	0.69	800	685	—	—	
Station 439+00 to 447+00	1953	Silt	96	1.01	96	0.01			—	—	Seepage meter in canal bottom. Bottom compacted to 2-foot depth, no compaction on slope.
Franklin Canal											
Pond 1	1958	Silt	100	0.03	98	0.11	1,400	230	4.80	31	Seepage meter rate is average for reach
Pond 2	1958	Silt	100	0.09	96	0.13	1,400	230	4.80	31	Seepage meter rate is average for reach
Colorado-Big Thompson project, Colorado											
Poudre Supply Canal											
Station 167+50 to 186+00	1951	Silty sand	15	0.24	15	0.05	1,850	1,500	10.76	71.7	"Study of Seepage Losses from Irrigation Channels 1951, 1952" by A.R. Robinson and Carl Rohwer.
Station 167+50 to 186+00	1952	Silty sand		0.15					—	—	

[1] cfd = cubic feet per square foot per 24 hours.
[2] Where design depth is not given, percentage is only an estimate from the best information.
[3] *Values given are approximate and based on best information available at time of test.*

LCCL-T3 (2 OF 2)

TABLE 4.—*Permeability of some typical canal linings*

Feature and project	As designed					As tested				Remarks
	Station		Capacity, second-feet	Water depth, feet	Wetted perimeter, feet	Length of reach, feet	Date	Water depth, feet	Seepage rate, (cfd)[1]	
	From	To								
ASPHALTIC MEMBRANES										
Buried, Hot-Applied										
Shoshone project, Wyoming										
Lateral R-4-S	50+63	60+58	84.7	2.6	21.0	995	1949	2.2	(1.31[2])	Unlined (control reach)
	60+71	71+11				1,041	1949	2.1	0.16[2]	3/16- to 1/4-inch membrane with earth cover
Buried, Prefabricated with Organic Fiber Reinforcement										
Boise project, Idaho										
Lateral CW-9.9	—	—	—	2.0	10.0	300	1950	1.8	(0.54[2])	Before lining (control)
									0.06	1/8-inch membrane with earth cover
Savage Test Lateral	1+49	3+39	—	1.5	9.0	190	1951	0.77[3]	(2.30)	Before lining (control)
	1+12	1+68	—	1.5	9.0	46	1952	1.25[3]	0.09	1/8- to 3/16-inch membrane with earth cover
							1953	0.91[3]	0.24	
							1954	0.97[3]	0.54	
							1955	0.63[3]	0.52	
	2+15	2+67	—	1.5	9.0	52	1952	0.93[3]	0.03	1/8-inch membrane with macadam cover
							1953	0.86[3]	0.14	
							1954	0.83[3]	0.19	
							1955	0.65[3]	0.39	
Buried, Prefabricated with Asbestos Fiber Reinforcement										
Boise project, Idaho										
Lateral 10.2-4.2	0+00	6+45	—	—	—	645	1952	—	(2.73)	Before lining (control)
	0+00	3+15	—	—	—	315	1952	0.85[3]	0.02	3/32-inch membrane with earth cover
							1953	0.89[3]	0.02	
							1954	1.04[3]	0.03	
							1956	0.42[3]	0.05	
Savage Test Lateral	1+49	3+39	—	1.5	9.0	190	1951	0.77[3]	(2.30)	Before lining (control)
	3+52	4+99	—	1.5	9.0	147	1951	0.69[3]	(3.05)	Before lining (control)
	2+68	3+20	—	1.5	9.0	53	1952	0.95[3]	0.05	3/32-inch membrane with earth cover
							1953	0.88[3]	0.04	
							1954	0.95[3]	0.04	
							1955	0.78[3]	0.13	
	3+20	3+74	—	1.5	9.0	54	1951	1.00[3]	0.08	3/32-inch membrane with earth cover
							1952	1.02[3]	0.20	
							1953	0.75[3]	0.05	
							1954	0.89[3]	0.07	
							1955	0.75[3]	0.08	
Buried, Prefabricated with Glass Fiber Reinforcement										
Boise project, Idaho										
Lateral 10.2-4.2	0+00	6+45	—	—	—	645	1952	—	(2.73)	Before lining (control)
	3+15	4+57	—	—	—	142	1954	0.73[3]	0.03	1/16-inch membrane with earth cover
							1956	0.69[3]	0.01	
Savage Test Lateral	1+49	3+39	—	1.5	9.0	190	1951	0.77[3]	(2.30)	Before lining (control)
	1+68	2+15	—	1.5	9.0	47	1952	1.01[3]	0.08	1/16-inch membrane with earth cover
							1953	0.96[3]	0.10	
							1954	0.82[3]	0.29	
							1955	0.71[3]	0.34	
							1958	0.62[3]	1.57	
	3+52	4+99	—	1.5	9.0	147	1951	0.69[3]	(3.05)	Before lining (control)
	3+74	4+99	—	1.5	9.0	130	1951	0.78[3]	0.27	1/16-inch membrane with earth cover
							1952	0.84[3]	0.23	
							1953	0.79[3]	0.07	
							1954	0.74[3]	0.09	
							1955	0.59[3]	0.14	
	4+99	6+51	—	1.5	9.0	152	1951	0.76[3]	0.13	1/16-inch membrane with earth cover
							1952	0.82[3]	0.08	
							1953	0.65[3]	0.07	
							1954	0.75[3]	0.26	
							1955	0.55[3]	0.79	

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TABLE 4.—*Permeability of some typical canal linings*—Continued

Feature and project	As designed					As tested				Remarks
	Station		Capacity, second-feet	Water depth, feet	Wetted perimeter, feet	Length of reach, feet	Date	Water depth, feet	Seepage rate, (cfd)[1]	
	From	To								
ASPHALTIC MEMBRANES (Continued)										
Buried, Prefabricated with Glass Fiber Reinforcement (Continued)										
Boise project, Idaho (continued)										
Willow Creek Pump Canal	0+00	1+38	—	—	10.8	138	1950	—	(0.54[2])	Before lining (control)
							1950	—	0.04[2]	1/16-inch membrane with earth cover
							1951	1.13[3]	0.02	
							1955	0.77[3]	0.33	
W.C. Austin project, Oklahoma										
Altus Lateral 6.8	2+50	8+33	10	1.04	6.0	583	1950	1.04	(0.71[2])	Before lining (control)
	8+33	12+16	10	1.04	10.0	383	1950	1.04	(1.54[2])	Before lining (control)
	5+84	9+08	10	1.04	6.0	324	1950	1.04	0.10[2]	1/16-inch membrane with earth cover
	9+08	12+16	10	1.04	6.0	308	1950	1.04	0.08[2]	
Exposed, Prefabricated										
Boise project, Idaho										
Lateral 10.2-4.2	0+00	6+45	—	—	—	645	1952	—	(2.73)	Before lining (control)
	4+47	5+52	—	—	—	95	6-1953	0.84[3]	0.05	1/2-inch-thick sheets
							9-1953	0.71[3]	0.18	
							1954	0.91[3]	0.01	
							1956	0.87[3]	0.01	
							1958	0.76[3]	0.01	
	5+52	6+45	—	—	—	93	6-1953	0.73[3]	0.48	1/4-inch-thick sheets
							9-1953	0.77[3]	0.05	
							1954	0.90[3]	0.05	
							1956	0.56[3]	0.11	
Savage Test Lateral	1+49	3+39	—	1.5	9.0	190	1951	0.77[3]	(2.30)	Before lining (control)
	0+62	1+12	—	1.5	9.0	50	1953	0.78[3]	0.31	1/4-inch-thick sheets
							1954	0.81[3]	0.25	
							1955	0.67[3]	0.25	
							1958	0.80[3]	0.13	
	3+52	4+99	—	1.5	9.0	147	1951	0.69[3]	(3.05)	Before lining (control)
	6+51	7+00	—	1.5	9.0	49	1953	0.83[3]	0.08	1/2-inch-thick sheets
							1954	0.81[3]	0.18	
							1955	0.58[3]	0.53	
							1958	0.79[3]	0.01	
PORTLAND CEMENT CONCRETE AND MORTAR										
Unreinforced Concrete										
Central Valley project California										
Friant-Kern Canal	3644+07	3673+56	4000 to 5000	17.2	119	2,949	1950	17.2	0.07[2]	3 1/2 inches thick
Rio Grande project, New Mexico-Texas										
West Canal	50+84	273+32	—	—	—	22,248	1949	—	0.83	4 inches thick. Seepage measured by inflow-outflow methods
	273+32	314+76	—	—	—	4,144	1949	—	0.50	
	314+76	342+87	—	—	—	2,811	1949	—	0.26	
Concrete Blocks										
Boise project, Idaho										
"D" Line Canal	Vicinity	mile 13.5	—	—	28.4	400	1951	2.87[3]	(0.43)	Unlined adjacent reach (control)
					25.1	400	1951	2.55[3]	0.20	Lined reach
Shotcrete (Mortar)										
Gila project, Arizona										
Lateral A-8.9-N	—	—	14	—	8.5	1,226	1950	14.0	0.03	1 1/2 inches thick
Lateral B-3.7-1.8	—	—	14	—	8.5	614	1950	14.0	0.03	1 1/2 inches thick

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TABLE 4.—*Permeability of some typical canal linings*—Continued

Feature and project	As designed					As tested				Remarks
	Station		Capacity, second-feet	Water depth, feet	Wetted perimeter, feet	Length of reach, feet	Date	Water depth, feet	Seepage rate, (cfd)[1]	
	From	To								
EARTH LININGS										
Thick, Compacted										
Central Valley project, California										
Friant-Kern Canal	2791+20	2820+24	4500 to 5,000	17.2	119	2,904	1950	17.2	0.07	2 to 3 feet thick
Tucumcari project, New Mexico										
Conchas Canal	2043+60	2072+60	700	8.65	55	2,900	1949	7.0	0.13	Thickness: 1 foot on bottom; 3 feet on side slopes
	2518+40	2562+00	700	8.65	55	4,560	1949	7.0	(0.40)	Unlined adjacent reach (control)
	2562+40	2567+00	700	8.65	55	2,460	1949	7.0	0.07	Thickness: 1 foot on bottom; 3 feet on side slopes
Boise project, Idaho										
"D" Line Canal	Vicinity mile 146		170	—	—	400	1951	2.87[3]	(0.43)	Before lining (control)
							1952	2.79[3]	0.05	18 inches thick, tractor compacted
Missouri River Basin project, Montana										
Helena Valley Canal	1173+97	1213+38	—	4.44	—	3,941	1959	3.76	0.08	Thickness: bottom, 24 inches; slopes, 32 inches
Loose Earth Blankets (Uncompacted)										
Boise project, Idaho										
Savage Test Lateral	11+58	13+98	—	1.5	8.4	240	1951	0.89[3]	(1.96)	Before lining (control)
							1952	0.75[3]	0.54	12 inches thick
							1953	0.72[3]	0.66	
							1954	0.68[3]	0.85	
							1955	0.59[3]	1.27	
Soil-Cement										
Gila project, Arizona										
Yuma Mesa Lateral A-5.0-N	—	—	12	1.35	10	638	1950	1.35	0.03[2]	3 inches thick, mixed in place
W.C. Austin project, Oklahoma										
West Lateral 11.5	197+15	211+00	45	2.13	9	1,385	1948	2.13	0.03[2]	3 inches thick, 15.5 percent cement
							1949	2.13	0.06[2]	
	211+00	223+00	45	2.13	9	1,200	1948	2.13	0.14[2]	3 inches thick, 11.0 percent cement
							1949	2.13	0.20[2]	
	223+00	237+75	45	2.13	8	1,475	1948	2.13	0.07[2]	3 inches thick, 17.5 percent cement
							1949	2.13	0.11[2]	
	237+75	249+75	45	2.13	9	1,200	1948	2.13	(0.95[2])	Unlined adjacent reach (control)
Sediment Sealing (Silting)										
Boise project, Idaho										
Lateral 10.2-4.2	0+00	6+45	—	—	—	645	1952	0.50[3]	0.76	Before removal of silt
							1952	0.58[3]	(2.73)	After removal of silt
Lateral 10.2-3.1	161+90	164+40	—	—	12.3	250	1957	1.42[3]	0.94	Before removal of silt
					16.5	250	1957	1.80[3]	(1.90)	After removal of silt and overexcavation of canal prism
					15.6	250	1958	1.49[3]	0.62	After 1 year of natural silting
Missouri River Basin project, South Dakota										
Angostura unit	Pond No. 1		32	2.6	16.0	2,780	1955	2.5	(1.04[2])	Before bentonite sedimenting
							1956	2.5	1.06[2]	After bentonite sedimenting
	Pond No. 2		22	1.7	11.0	950	1955	1.7	(1.08[2])	Before bentonite sedimenting
							1956	1.7	0.75[2]	After bentonite sedimenting
North Platte project, Wyoming										
Lateral No. 1	Pond No. 1		100	4.0	19.0	3,072	1955	2.9	(0.40[2])	Before bentonite sedimenting
							1956	3.4	0.54[2]	After bentonite sedimenting
	Pond No. 2		100	4.0	21.0	2,161	1955	2.8	(0.40[2])	Before bentonite sedimenting
							1956	2.2	0.63[2]	After bentonite sedimenting

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cfd = cubic feet per square foot per 24 hours

Seepage values enclosed in brackets ( ) indicate condition prior to lining for comparative purposes

[1] Seepage rate determined by ponding and corrected for evaporation unless indicated otherwise

[2] Seepage rate not corrected for evaporation.

[3] Hydraulic radius

seepage meter tests were made at water depths somewhat lower than those at which losses by ponding were established. Obviously, not all seepage meter readings could be made simultaneously, and the depth of the pond was decreasing with time. Therefore, the ponding loss rate must be adjusted for depth if it is to be compared to the computed seepage meter rate. It is worthy of note that all seepage losses measured on the North Platte system were relatively low (maximum by ponding 0.86 cfd), although many areas adjacent to the canal were waterlogged, emphasizing that the rate of loss is not always a reliable indicator of potential damage.

Observations of water table elevations were made on the North Platte project by utilizing ground-water wells located adjacent to the ponded sections. These observations did not yield specific quantitative results, but showed in general that the ground-water table fluctuated rapidly when the canal was filled or emptied. This indicated that the canal was the source of seepage. Wells adjacent to a portland cement-grouted section of the main canal showed the ground water to be as high as before grouting. In contrast, wells located along a ¾-mile section which was lined with buried asphalt membrane in November 1949 became dry or showed a noticeable drop in the water table after installation of the lining.

(b) *Riverton Project—Wyoming.*—A similar series of tests using both ponding and seepage meter measurements was conducted in the extension of the Wyoming Canal, Riverton project. These tests, results of which are shown in table 3, were made to determine the need for lining to control seepage in reaches where high losses were anticipated from visual inspection.

(c) *Central Valley Project—California.*—After the 1950 irrigation season, tests by ponding were conducted in two reaches of the Friant-Kern Canal, one reach having a thick compacted earth lining in sections where soils other than clay were encountered and the other a concrete lining. These linings had been in service less than 3 years and were in excellent condition at the time of testing. The seepage rates were about 0.07 cfd for both earth and concrete. For a given capacity, an earth-lined canal must be much larger in cross section than a concrete-lined one because of the difference in frictional resistance of the two surfaces and the higher velocities permissible with concrete. In the test reaches of the Friant-Kern Canal, the bottom widths of the earth- and concrete-lined sections were 64 and 36 feet, respectively, for almost equal depths. Therefore, the wetted area per foot of canal length was greater for the earth-lined canal than for the concrete, and the rate of water lost was about 30 percent greater for the earth canal.

Seepage tests are seldom made in large canals because of the cost. In the Friant-Kern Canal, the total cost was approximately $30,000 for constructing and removing dikes, pumping water, taking measurements, and all other related operations involving 3 miles of canal. However, the data obtained were considered well worth the expense in establishing percolation rates through concrete and thick compacted-earth lining.

Ponding tests are more commonly made in canals with bottom widths from 6 to 16 feet, for which size range the cost may vary from $1,250 to $2,500 including analysis of data and report. Though these amounts are not prohibitive, it is apparent that efforts should be continued to develop a simpler and less costly method for determining seepage losses.

(d) *Missouri River Basin Project—Nebraska.*—Ponding tests on the Courtland Canal were performed in 1952 to evaluate the seepage loss through loess material before and after compaction of only the canal bottom. The seepage tests were repeated in 1953. Following the initial tests the bottom of the canal, which has a design water depth of 8.5 feet, was scarified and compacted in 6-inch lifts to a 2-foot depth. The seepage rate before compaction as measured with seepage meters was 0.69 cfd and after compaction less than 0.01 cfd. Ponding tests in the same reach in 1953 showed a pronounced decrease in seepage rate with decreasing water depth, as indicated in the following tabulation.

Depth of water, feet	Seepage rate, cfd		Approximate reduction in seepage rate, percent
	Before compaction, fall 1962	After compaction, spring 1963	
8.5	1.21	1.06	12
7.5	1.06	.92	13
6.5	1.90	.76	16
5.5	.73	.57	22
4.5	.55	.37	33

TABLE 5.—*Summary of the results of seepage tests on various projects and on different types of materials*

Canal or lateral reach	Type of lining[1]	Design discharge, second-feet	Design depth,[2] feet	Percent design depth tested	Channel characteristics: Nominal bottom width[3], feet	Channel characteristics: Side slope[3]	Channel characteristics: Wetted area per foot of length[3,4]	Measured seepage rate, (cfd)[5]	Method of measurement	Date	Remarks
Boise project, Idaho											
Black Canyon Canal											
Lateral 0.1-1.0	—	—	2.0	90	2.25	1¾ to 1	10	0.72	Ponding	1950	150 feet tested
Lateral 0.1-1.0	Asphalt	—	2.0	90	2.25	1¾ to 1	10	0.04	Ponding	1950	Prefabricated asphalt membrane, 3-foot base width, 12-inch earth cover
Central Valley project, California											
Contra Costa Canal											
Station 1805+56 to 1857+55	Reinforced concrete	140	4.88	90	6	1¼ to 1	20	0.008	Ponding	1958	
Station 1857+67 to 1873+85	Reinforced concrete	140	4.88	90	6	1¼ to 1	20	0.09	Ponding	1958	
Delta-Mendota Canal											
Station 4535 to 5218	Compacted earth	3,310	15.4	91	62	2½ to 1	137	0.009	Well permeameter	1950-51	
Station 5218 to 5350	Compacted earth	3,310	15.4	91	84	2½ to 1	159	0.009	Well permeameter	1950-51	
Station 5350 to 5485	Compacted earth	3,211	15.4	91	60	2½ to 1	135	0.009	Well permeameter	1950-51	
Madera Canal											
Mile 22.4 to 24.6	—	800	—	50	24	1½ to 1	52	0.21	Stage recorder	1946	In test reach, values represent average conditions.
Mile 24.6 to 35.6	—	625	—	50	20	1½ to 1	48	0.21	Stage recorder	1946	
Madera Lateral 6.2											
Station 62+83 to 104+00	—	340	5.0	100	20	1½ to 1	38	0.006	Ponding	1953	Corrected for evaporation
Colorado-Big Thompson project, Colorado											
South Platte Supply Canal											
Special section	(6)	—	3.6	89	8	2 to 1	24	1.35	Ponding	1954	200 feet tested
Gila project, Arizona											
Yuma Mesa Division											
Lateral A-6.5-W											
Station 0 to 16+21	—	14	—	95	2	1½ to 1	—	3.30	Ponding	1945	Constant head test
Lateral B-3.7-1.8-S											
Station 0 to 19+20	—	14	3.1	100	2	1½ to 1	12	2.18	Parshall flume	1947	
Station 2+30 to 18+60	—	14	3.1	95	2	1½ to 1	12	1.31	Ponding	1945	
Lateral B-3.8-3.3-S	—	14	2.7	90	2	1½ to 1	11.5	3.02	Weir	1945	
Missouri River Basin project, Kans., Nebr.											
Ainsworth Canal											
Special section											
Pond 1	(6)	86	2.5	100	12	4 to 1	27	0.05	Ponding	1952	300 feet tested, dune sand and high ground water
Pond 2	(6)	86	3.3	100	12	4 to 1	34	0.08	Ponding	1952	280 feet tested, dune sand and high ground water
Pond 3	(6)	86	2.0	100	7	3½ to 1	39	0.28	—	1952	100 feet tested, finer sand underlain by hardpan and low ground water
Pond 4	(6)	—	2.5	98	9	2½ to 1	23	2.6	Ponding	1953	200 feet tested, dune sand and low ground water
Courtland Canal											
Station 810+00 to 820+00	Earth	685	8.5	34	28	1½ to 1	54	0.15	Seepage meter	1958	Rate shown is average for reach
Station 832+00 to 845+00	Earth	685	8.5	32	28	1½ to 1	54	0.10	Seepage meter	1958	Rate shown is average for reach

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TABLE 5.—*Summary of the results of seepage tests on various projects and on different types of materials*—Continued

Canal or lateral reach	Type of lining[1]	Design discharge, second-feet	Design depth,[2] feet	Percent design depth tested	Channel characteristics: Nominal bottom width[3], feet	Side slope[3]	Wetted area per foot of length[3,4]	Measured seepage rate, (cfd)[5]	Method of measurement	Date	Remarks
Missouri River Basin project, Kans., Nebr. (continued)											
Hanover-Bluff Canal											
Station 698+42 to 703+45	—	15	2.0	70	5	2 to 1	11	0.30	Ponding	1959	
Helena Valley Canal											
Station 1110+19 to 1120+19	—	225	4.79	100	12	2 to 1	34	1.60	Ponding	1959	
Station 1173+97 to 1213+18	Compacted earth	170	4.44	100	10	2 to 1	27	0.08	Ponding	1959	
Station 1527+74 to 1537+74	—	65	2.75	100	8	1½ to 1	17	2.30	Ponding	1959	
Kirwin Main Canal											
Station 33+00 to 60+00	—	175	5.8	97	14	1½ to 1	34	0.98	Seepage meter	1958	Rate shown is average for reach
Kirwin South Canal											
Station 499+00 to 505+10	—	42	2.55 to 2.70	84	6	2 to 1	16	0.71	Seepage meter	1958	Rate shown is average tor reach
Station 517+00 to 571+00	—	42	2.55 to 2.70	100	6	1½ to 1	18	0.46	Seepage meter	1958	Rate shown is average for reach
Meeker Canal											
Mile 5.8 to 10.5	—	35	2.5	92	—	Approx. elliptical	9	1.7%/mi.	Parshall flumes	1950	Design capacity 35 second-feet
Meeker-Driftwood Canal											
Station 723+50 to 738+58	Earth	250	5.2	100	16	1½ to 1	35	0.05	Seepage meter	1958	Rate shown is average for reach
Middle Loup Canal No. 2											
Mile 0.5 to 12.5	—	67	1.6	90	12	2 to 1	21	0.71	Flume	1949	
Provo River project - Utah											
Provo Canal											
Station 82+00 to 665+48	—	500	5.7	60	16	1½ to 1	36	0.44%/mi.	Current meter	1946	
Station 82+00 to 665+48	—	500	5.7	60	16	1½ to 1	36	0.62%/mi.	Current meter	1947	
Station 665+48 to 752+00	Silt	450	5.3	60	16	1½ to 1	35	2.67%/mi.	Current meter	1946	
Station 665+48 to 752+00	Silt	450	5.3	60	16	1½ to 1	35	2.12%/mi.	Current meter	1947	Canal section varied in test reach,
Station 752+50 to 1040+00	—	400	—	60	16	1½ to 1	35	0.65%/mi.	Current meter	1946	values represent average
Station 752+50 to 1040+00	—	400	—	60	16	1½ to 1	35	0.86%/mi.	Current meter	1947	conditions
Station 1040+00 to 1150+00	—	350	—	60	10	1½ to 1	26	3.54%/mi.	Current meter	1946	
Station 1040+00 to 1150+00	—	350	—	60	10	1½ to 1	26	1.04%/mi.	Current meter	1947	
Rio Grande project, New Mexico, Texas											
West Side Canal											
Station 342+87 to 436+88	—	206	—	—	11.5	1½ to 1	38	0.76	Weir	1949	Sand, sandy loam, and sand fills
Station 436+88 to 696+79	—	206	—	—	11.5	1½ to 1	39	2.10	Weir	1949	
Riverton project, Wyoming											
Wyoming Canal											
Station 2008+00 to 2241+25	—	566	5.7	85	24	2 to 1	52	0.62	Ponding	1950	
Station 2242+18 to 2247+50	—	566	5.7	85	24	2 to 1	52	0.29	Ponding	1950	
Station 2261+83 to 2275+00	—	566	5.7	85	24	2 to 1	52	0.35	Ponding	1950	
Station 2394+81 to 2406+00	—	566	5.7	85	24	2 to 1	52	4.79	Ponding	1950	
Lateral 44.69											
Station 23+00 to 31.50	—	24	1.8	90	4	1½ to 1	10.5	0.96	Ponding	1950	

LCCL-T8(2OF2)

[1] If type of lining is not given, canal is unlined.
[2] Where design depth is not given, percentage is an estimate from best information available.
[3] Values given are approximate and based on best available information at time of test.
[4] When canal cross section was unknown, wetted perimeter per foot of length was obtained from wetted area divided by length of pond for maximum test depth
[5] Unless otherwise indicated, seepage rates are expressed in cubic feet per square foot of wetted area per 24 hours (cfd).
[6] Tests prior to construction in soils reprsentative of proposed canal routes.

The last column in the tabulation on page 23 shows the reduction in seepage rate, for various water depths, which is attributable to bottom compaction.

(e) *Other Studies.*—Seepage measurements on the Angostura unit and the Interstate Canal were performed to establish rates before and after placement of a sediment lining. The initial tests were made at the close of one irrigation season; the lining was placed at the beginning of the following season, and the second series of tests were made after one season of canal operation. Thus, the seepage measurements were conducted at times when seasonal variations of conditions would be at a minimum.

Seepage tests were made on special sections along the alinement of the Ainsworth Canal in soils, and with ground-water conditions, representative of those in which the canal was to be constructed. The sections selected represented a wide range of expected conditions for the purpose of providing design data.

Two ponds were constructed following the 1958 irrigation season in the Franklin Canal, Nebraska. Both were sections in which compacted-earth linings had been installed several years before. Seepage meter measurements were made in these ponds. The purpose of the tests was to learn if the effectiveness of the lining was decreasing with age. The results of both ponding and seepage meter tests are shown in table 3. Ponding rates range from 0.03 to 0.09 cfd. These rates do not indicate that any deterioration of the lining had taken place.

18. ***Experimental Studies.***—It is necessary to estimate the origin, magnitude, and direction of the flow in devising economic remedial measures for reducing seepage from waterways. Suspected areas of high permeability can sometimes be isolated through a knowledge of soil conditions and ground-water levels, and the appearance of water directly associable with nearby hydraulic structures. However, there are many places where the origin and magnitude of seepage losses are not obvious and the requirement exists for more perceptive detection and measurement.

Radioactive and dye tracers for use in the detection and measurement of seepage losses have been under investigation. Also, the use of electrical logging devices has been tested in the field and in the laboratory. Both methods of detection and measurement are being investigated for use with conventional means of measurement such as ponding, inflow-outflow with standard measuring devices, and seepage meters of various types.

(a) *Tracers.*—Four possible methods of using tracers in the detection and measurement of seepage loss have been considered and are briefly described below:

(1) In one method the tracer would be added to the canal water in a form that would remain in suspension or solution and be filtered or attracted by the soil material in the area of maximum leakage. A subsequent survey of the canal sides or bottom possibly would reveal relative surface infiltration rates and the location of the area of greatest seepage loss.

(2) In a second method a soluble tracer would be added which would remain with the water seeping from the canal, to be detected at some distance from the canal. Measurement of the tracer concentration and time of arrival at the observation point would theoretically provide an estimate of the flow velocity through the soil, and also an estimate of the canal seepage loss.

(3) In a third method a tracer would be placed in one or more test wells located near the canal. Measurement of the dilution of the tracer from the test wells or movement to other wells downstream could provide an estimate of the amount of seepage.

(4) In a fourth method tracers would be used to measure the inflow to and outflow from a canal section. With accurate measurements, the difference in flows would show the quantity of water lost by seepage within the reach.

Methods of detecting and tracing the movement of ground water were the object of studies undertaken by the University of California under cooperative agreements with the Bureau of Reclamation and others. The studies were aimed primarily at a better understanding of the velocity variations observed in tracing the flow of liquids through porous media, the developmen' of methods and tracer materials for the direct field determination of the movement of water through the ground, and the application of these procedures to the location and measurement of seepage from water carrier and storage systems. Initial research objectives were the preparation of a comprehensive abstract of the literature pertaining to methods of locating and tracing the movement of ground water, a critical review of the abstracted literature to select those techniques appearing most applicable, and the initiation of a laboratory study to evaluate the most suitable organic, inorganic, and radioactive tracers.

Both geophysical and tracer methods were considered by the University of California, but work was concentrated on tracers when it was concluded that the various geophysical methods for determining the location of ground water would likely not provide quantitative indication of the velocity of the ground-water movement, whereas tracers, when applied to the ground-water stream, would. Among the geophysical methods the gravitational method, the magnetic method, and the seismic refraction method were not believed applicable; and the electrical resistivity method presented problems of interpretation. (Note: One geophysical method, the electrical logging of canals, described in the following subsection, is currently under test by the Bureau of Reclamation (1962) and shows promise of becoming a useful method for indicating leakage along canals and for indicating reaches that are relatively impermeable.)

Based on the existing literature and on the results of preliminary laboratory studies of several of the more promising tracers, no ideal tracer was found to meet a wide range of field conditions. Organic dyes were detectable at low concentrations, but were highly susceptible to absorption; sodium fluorescein appeared to be usable under limited conditions. The chloride ion was found superior to many other tracers in laboratory studies if no density current was induced. Any measurable exchange capacity would disqualify the use of radioactive cations, but not of radioactive anions such as iodine-131. Tritium, the radioisotope of hydrogen, did not appear to constitute a practical ground-water tracer, according to the knowledge at the time, argely because of the high cost of detection procedures.

The report of the first year's activities recommended further studies of the electrical resistivity method, with the objective of developing its applicability to canal seepage location and measurement; and studies of chemical and radioactive ground-water tracers with the objective of delineating the limitations of the tracers currently in use and of finding more satisfactory substances.

During subsequent studies conducted by the University of California, the dispersion phenomena of laminar flow through porous media were investigated theoretically and experimentally for unidirectional flow; theoretical and experimental investigations of dispersion phenomena in laminar flow through porous media [4] were extended to the case of radial dispersion occurring during the movement of water from an injection well penetrating a confined aquifer; and finally, an actual application of the tracer technique was investigated in the field.

A final report of the University of California studies is now being prepared (1962). Further studies of the techniques and equipment involved will be necessary before the use of radioisotopes can be generally accepted for determining accurately the location and amount of seepage occurring; but with refinement of technique and further development of equipment the method offers promise, and further studies are warranted.

(b) *Electrical Logging of Canals to Detect Seepage.*—The technique of electrical logging of drill holes to determine the variations in strata has been used for many years in oil and water wells.[5] The electrical log provides a continuous record of electrical resistivity and self-potential or natural electrical voltage in formations penetrated by the drill. From this record, experienced operators can identify rock formations and secure other information which is valuable in oil field development.

The adaptation of electrical logging of drill holes to the electrical logging of canals to detect

[4] Lau, Leung-Ku, Kaufman, W. J., and Todd, D. K., "Dispersion of a Water Tracer in Radial Laminar Flow through Homogeneous Porous Media," Canal Seepage Research Progress Report No. 5, Hydraulics Laboratory and Sanitary Engineering Research Laboratory, University of California, Berkeley, Calif., July 1, 1959.

[5] Martin, R. I., "Fundamentals of Electric Logging," a manual reprinted from Oil and Gas Journal, Petroleum Publishing Co., Tulsa, Okla., 1955.

seepage has been actively under test by the Bureau of Reclamation since October 1958. The possibility was in fact considered prior to that time. The October 1958 date is mentioned as it marked the first electrical logging of a water-filled canal carried out by the Bureau of Reclamation in an effort to detect seepage. This initial trial was performed by a firm of consulting geologists and geophysicists, of Phoenix, Ariz., and electrical resistivity only was determined. The field tests showed that the electrical resistance of the materials comprising the bottom and the banks of a canal has a relationship to seepage. If these materials are wet gravel and sand or wet silt and clay, they will have a lower electrical resistance than when they are dry. Low-resistance zones may thus indicate seepage.

Under the 1958 tests, logging was completed along some 7 miles of canals in the Central Valley project, and covered selected reaches of the Madera, Contra Costa, Delta Mendota, and Friant-Kern Canals. It was the opinion of the Bureau of Reclamation engineers concerned with the tests that the electrical logging of canals could assist in the location and tracing of seepage, if combined with a knowledge of the soils and geological formations along the canals. In addition, it was felt that further field trials were warranted.

Since the first test in 1958, laboratory studies were initiated and three additional tests were made in canals on the Central Valley project, California, in April 1960; in the Kirwin Canal on the Missouri River Basin project, Kansas, in September 1960; and in canals on Tucumcari project, New Mexico, in May 1961.

A modification of technique was used in the later electrical logging tests in an effort to obtain a more positive indication of the presence of seepage and some general indication as to its amount. The tests involved an electrical property of earth materials known as natural voltage or self-potential. It had been observed that natural voltage may be induced by the slow movement of water through fine-grained materials. The tests determined that natural voltage in the materials surrounding a canal can be measured by use of sufficiently sensitive equipment. The test further indicated that, if the natural voltage in the materials surrounding the canal shows little variation from point to point, the canal is "tight" and there is little or no loss of water. In contrast, if the natural voltage through the surrounding materials changes rapidly at adjacent points along the canal, such reaches may have appreciable seepage.

Like the electrical logging of drill holes, the electrical logging of a canal provides a continuous record on a strip chart of variations in the electrical resistance and in the natural voltage of materials from point to point along the bottom or sides of the canal, or both. As shown in figure 4, these measurements are made by establishing electrical contact with the canal bottom by means of two flat, circular lead disks or electrodes. The electrodes are connected through lead wires to a source of alternating current and the measuring equipment, which consists of a two-pen chart recorder mounted in an instrument truck which travels on the bank and drags the electrodes along the bottom of the canal.

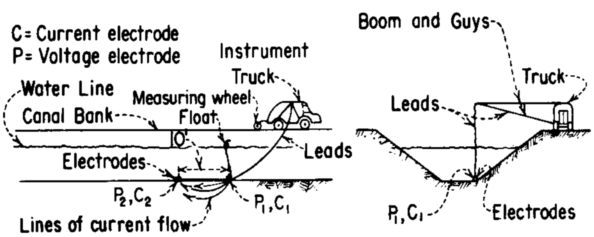


Figure 4.—Setup for electrical logging of a canal to locate seepage. The electrodes are dragged along the bottom of the canal. 288-D-2678.

Some of the Bureau's field tests have included ponding tests and tests made with seepage meters for verification of logging results. There appears to be a considerable degree of verification, although there may be unknown limiting factors to be considered. Electrical logging is believed to have much potential importance in seepage investigations of both lined and unlined canals. When fully developed, it may be able to locate leakage zones as well as tight zones while the canal is in service, at less cost and in much less time than by any other means presently available. (Sixteen miles of canal can be logged in an 8-hour day.) Some of the limitations of the equipment are being studied and additional field trials will be made.

Chapter IV

Exposed Linings

19. ***Types.*** — Exposed linings are considered herein to include all linings except earth linings (defined in chapter VI) that expose the water barrier to the wear, erosion, and deterioration of the flowing water, the elements, operation and maintenance equipment, and other hazards such as stock. This category thus includes all hard-surface (rigid type) linings constructed of asphaltic materials, portland cement concrete and mortars, soil-cement, brick, and stone; and linings consisting of relatively thin membranes of asphaltic materials, plastics, and synthetic rubber placed directly on the canal bed without protective cover. Where such cover is provided, these linings are classified as buried-membrane linings, which are discussed in chapter V.

Portland cement has long been used with much success in the construction of cast-in-place concrete and mortar linings, pneumatically applied mortars (shotcrete), and precast concrete blocks and slabs. More recently, portland cement has been combined with soil to produce soil-cement linings of the plastic and standard types, for use under mild exposures with possible savings in cost. Properly manufactured and applied, plastic soil-cement has been found to approach in serviceability portland cement concrete made with pit-run aggregate, if conditions of exposure are not severe. Brick and stone are other materials that have been used for hard-surfaced linings. (The use of stone in the United States for this purpose dates back to the "early days" in California.)

Several asphaltic materials, including asphaltic concretes and mortars, prefabricated asphaltic blocks, and asphalt macadams, have been used as hard-surface linings. Of these, the asphaltic concretes and asphaltic blocks have generally given good service. Thin, sprayed-in-place asphalt cements, and prefabricated sheets and rolls of asphaltic materials have also been tried, with limited success. Experiments are continuing with these exposed membrane type linings.

Other exposed membrane type linings that have been tried include thin sheets of plastic and synthetic rubber. The plastics and thinner synthetic rubbers have not been successful, as they are too easily damaged, and some of the plastics deteriorate rapidly on exposure. However, the thicker synthetic rubbers offer promise if costs can be reduced. The thinner membrane linings, if protected by a layer of earth, are giving good service.

The following discussion is devoted principally to hard-surface linings, which comprise the great majority of installations and for which design and construction criteria have been generally well established. They may be considered also applicable in general to exposed membrane linings, but these are still in the early experimental stage.

A discussion of the methods of repairing hard-surface linings is included in this chapter.

20. ***General Design Considerations.*** — Since the cost of a hard-surface lining usually amounts to a large percentage of the total cost of constructing a lined canal, the section with the least perimeter for a given area is the most economical. A semicircle has the smallest perimeter for a given area but is not practical because the top portions of the sides are too steep. From experience, the steepest satisfactory side slopes for most large canals, from both construction and maintenance considerations, is 1½ to 1. Steeper slopes may be used on small laterals where the soil materials will remain stable.

Canals provided with a hard-surface lining are usually designed with a base-width to water-depth ratio of from 1 to 2. Small canals normally have a ratio of nearly 1, while the ratio for large canals may exceed 2.

(a) *Subgrade.*—A primary prerequisite to the success of most hard-surface (rigid type) linings is a firm foundation which will reduce, as far as possible, the amount of cracking and the danger of failure due to settlement of the subgrade. Undisturbed soils often are satisfactory for a foundation for lining without further treatment.

Natural inplace soils of low density should be thoroughly compacted or removed and replaced with suitable material. Where this is impracticable, as in reaches through deep loess, concrete or other rigid type linings may not be suitable.

Expansive clays are usually an extreme hazard to rigid type lining because of their tendency to buckle the lining. This unequal movement is in addition to the usual bank instability that is associated with expansive soils. The use of hard-surface linings on expansive soils should be avoided if practicable.

If it becomes necessary to place a concrete or other rigid type lining on expansive clay, there are several ways of reducing or controlling the damage. Clays vary so much in characteristics that the pressure required to prevent expansion may be less than 1 pound per square inch in some types and as much as 150 pounds per square inch in others. If the clay encountered can be controlled by loading the surface with a nonexpansive compacted soil, lining can be placed on this loaded subgrade and satisfactory service obtained. Similarly, if the expansive clay is a thin layer in an otherwise suitable subgrade, it has been found fairly effective to overexcavate the clay and replace it with gravel. Excavation to a depth of at least 24 inches has been the practice to date, but the depth of clay seam and type of clay will influence the amount of excavation required.

Occasionally a hard-surface-lined canal may traverse a reach of expansive clay and no reasonable alternative route or construction type is feasible. The type of construction used for a short reach in the Gateway Canal, Weber Basin project, Utah, has proved effective for 4 years. Here the section was overexcavated and the surface sprayed with asphalt as though for a buried asphalt membrane lining installation. The sprayed surface was then covered with a layer of consolidated free-draining material to form the lining base and then drained to adjacent outlets. Another method successfully used about 10 years ago on the Friant-Kern Canal of the Central Valley project, California, was the installation of a sublining of asphalt on the subgrade immediately before concrete lining was placed. These two treatments have their application, but due to variability of soils and other conditions they may not always provide adequate protection from water movement. Further, both treatments are expensive. The difference in the two situations is that the Gateway location involved both canal water and external water in sand seams in the clay while at Friant-Kern the water was from the canal sides only.

Extensive tests are now being made to determine the possibility of using plastic sheets (polyvinyl chloride) under concrete to obtain extreme watertightness as an alternative to the Friant-Kern type installation.

Rock and boulders are frequently encountered in excavations for canals that will be lined with hard-surface linings. Shotcrete has been used to coat the face of such excavations with some success. The usual subgrade preparation is to overexcavate the rock and refill with compacted material which is trimmed to the required lines to receive the lining. A minimum distance of 3 inches is usually stipulated between bottom of lining and closest rock point, and an average of 5 inches of overbreak is assumed. Care should b exercised in selecting refill material for use over fractured rock or cobbles because of the danger of washing fines into the subgrade voids and thus losing lining support. The selected material must resist such piping and otherwise should be selected for impermeability and ease of placement.

(b) *Embankments.*—Loose embankment is placed over and outside the compacted embankment to provide for operating roads and additional stability. Unsuitable material should be stripped from under uncompacted embankments. Specifications for compacted embankments should require that after the necessary stripping has been done, the entire surface of the subgrade for compacted embankment be plowed thoroughly to a depth of not less than 6 inches, moistened and compacted. The embankment materials should be placed at a specified moisture content and compacted to a specified density in layers not more than 6 inches thick after compaction. The dry density of the soil fraction in the compacted material should not be less than 95 percent of the labora tory maximum density as determined by the Proctor method which is equivalent to ASTM

D-698-58T, method A.[1] The optimum moisture content, suitable for achieving this field placement density, should be specified. The material when distributed and compacted should be homogeneous and free from 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 having a maximum capacity of 100 second-feet and 6 to 8 feet for larger canals. The outside slope of the compacted embankment is normally specified as 1 to 1. Placing operations usually result in a flatter slope, but 1 to 1 is adequate for selected material.

In anticipation that the lining may leak in places or may develop leakage with age, the compacted embankment must be of a selected soil suitable to withstand a waterload without failure and, where sufficiently tight soils are available, without undue leakage. The compaction of loose soil in cut sections, or of soil replacing unsuitable subgrade materials, should meet the same requirements for density as those specified for compacted embankment.

(c) *Backfilling.*—When partial backfilling of an existing canal is necessary to reduce the cross-sectional area to that required for a lined canal, the backfill must be compacted to such a degree that subsequent settlement will not rupture the lining. The required degree of compaction varies with the soil, inplace materials, thickness of the backfill, and type of lining to be used. These related factors must be given special consideration for each such installation. Backfill compaction requirements often are the same as those specified for compacted embankments.

(d) *Underdrains.*—Since most canal linings are installed to prevent seepage, the subgrade is usually relatively free draining and above ground-water level. Occasionally, however, it may be necessary to employ hard-surface or exposed-membrane linings in areas subject to seasonal high ground water. When the canal is empty or when the water level in the canal is relatively low, the high ground water may result in unbalanced hydrostatic back pressures on the lining which are sufficient to damage the lining by flotation, unless it is protected by underdrains.

[1] "Tentative Methods of Test for Moisture-Density Relations Soils, Using 5.5-Pound Rammer and 12-Inch Drop," ASTM D-698-58T, method A, ASTM Standards 1961, part 4, p. 1304.

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 watertight 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.

The location of the canal bottom, with respect to the ground-water table, is especially important. In cold climates, the canal bottom must be at least 3 feet above the water table to prevent heaving from freezing and thawing, except that in free-draining gravels this requirement is not necessary. The greatest danger exists in silts or other highly frost-susceptible soils, especially in areas of frequent cycles of freezing and thawing.

In all instances such as those described above, where hard-surface or exposed-membrane linings are installed in areas subject to high ground water, the probability of damaging the lining can be greatly reduced by providing underdrains. There are two common types of artificial drainage installations. One type 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 to pump pits, 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 relieve any external pressure that is greater than the water pressure on the upper surface of the canal base but prevent backflow. The second type consists of a permeable gravel blanket of selected material or sand and gravel pockets, drained into the canal at frequent intervals (10 to 20 feet) by flap valves in the invert. A drawing of a flap valve for use without tile pipe and in a fine gravel and sand subgrade is shown in figure 5. Both the tile pipe system and the unconnected flap-valve type must be encased in a filter that will prevent piping of subgrade material into the pipe or through the valve. The required grad-

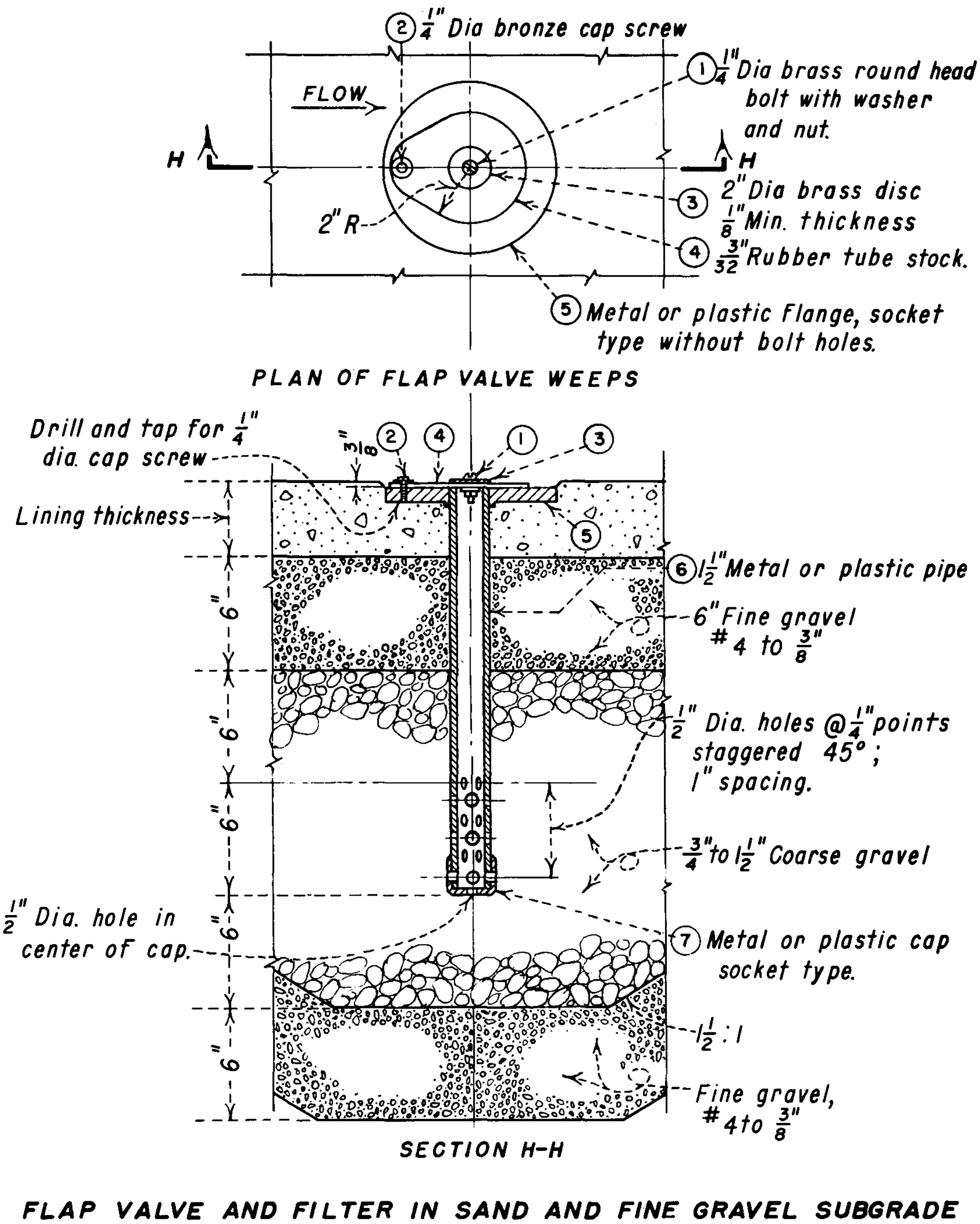


Figure 5.—**Flap valve installation for a canal underdrain. 103-D-636.**

ing of filter materials is given in the Bureau's Earth Manual.[2]

In areas where freezing can occur, the underdrainage system must be designed to be effective in cold weather if the condition of excessive external pressure can develop during the winter season.

(e) *Line and Grade*.—Specifications requirements with respect to line and grade should be as liberal as compatible with good engineerin' for operating conditions of the canal. Curren.

[2] "Earth Manual," first edition, Bureau of Reclamation, 1960.

Bureau specifications for portland cement concrete linings permit departure from established line of 4 inches on curves and 2 inches on tangents, and 1 inch from established grade. These generous tolerances permit the effective use of both rail-guided and subgrade-guided lining machines, which in turn has been instrumental in obtaining lower cost linings. Abrupt departure from and return to alinement and grade should be avoided.

(f) *Water Velocity.*—Hard-surface linings permit higher water velocities than do earth sections. Usually, these velocities in unreinforced lining should be less than 8 feet per second to avoid the possibility of lifting the lining should the velocity head be converted to pressure head through a crack that slopes upstream, or into the current. A mathematical check using a Manning's n of 0.003 less than the design n used for the lining, is also required to make certain the depth of flow does not approach critical depth closely enough to develop standing waves. Reaches most likely to develop these waves are those in which the canal bottom is raised above theoretical grade. (See tolerance in paragraph above.) At the point of maximum upward tolerance the depth should be greater than critical depth when computed with the reduced value of n.

(g) *Coefficient of Roughness.*—In a given canal, the rate of flow is inversely proportional to the roughness of the lining surface. The coefficient of roughness used in the design of canals represents an evaluation of the degree of roughness of the lining surface and its retarding effect on the flow of water. An important point sometimes overlooked is that this coefficient of roughness should not be based on the degree of original surface finish applied to the lining, but rather on the surface that will exist after a few years of operation. The coefficient of roughness, n, recommended for use in Manning's formula for the design of several types of linings is as follows:

Type of lining	n *(Manning's)*
Portland cement concrete lining	[1] 0.014
Shotcrete lining (smoothed with steel-edged screed and rebound removed)	0.016
Shotcrete lining (average)	0.017
Asphaltic concrete lining (machine placed)	0.014
Exposed prefabricated asphalt material	[2] 0.015
Soil-cement	[3] 0.015 or 0.016

[1] Present experience and tests indicate that the n value for linings must be adjusted with the size of the channel if Manning's formula is used. Preliminary results indicate that 0.014 is safe for concrete-lined canals with a hydraulic radius of 10 or less, but that the value should be increased to a possible 0.016 if the hydraulic radius is over 20. The investigation to obtain more reliable data is being continued.

[2] Assumed values based on observation of section only.

[3] Soil-cement may vary in roughness from as good as a well-finished concrete to as poor as a gravel surface. The type of construction to be required must be considered.

(h) *Thickness.*—Figure 6 shows the thickness normally used for several hard-surface linings based on the canal capacity. If surface deterioration in a freezing climate is expected, these thicknesses should be increased and an allowance made for the increased roughness resulting from the deteriorated surface. Increased thickness may also be justified if ice loads are expected.

(i) *Joints and Grooves in Portland Cement Concrete and Mortar.*—A slab of portland cement concrete used as a canal lining is subject to complex stresses resulting from temperatures or moisture change in the slab or from a combination of the two.

The compressive stress resulting from either a temperature or moisture increase is of little concern for two reasons. First, a slab which is fully restrained at both ends and subjected to a 100°F. increase in temperature will develop only about 1,500 pounds per square inch of compressive stress (assuming 0.000005 for the coefficient of expansion and 3,000,000 pounds per square inch for the modulus of elasticity in the relation $E =$ stress $\div$ strain). 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 hardening and drying of the concrete shortly after placing. Unless the contraction cracks resulting from drying shrinkage become filled with incompressible material, considerable expansion due to an increase in temperature can occur before complete closure of the cracks. If the contraction cracks are filled with an elastic material soon

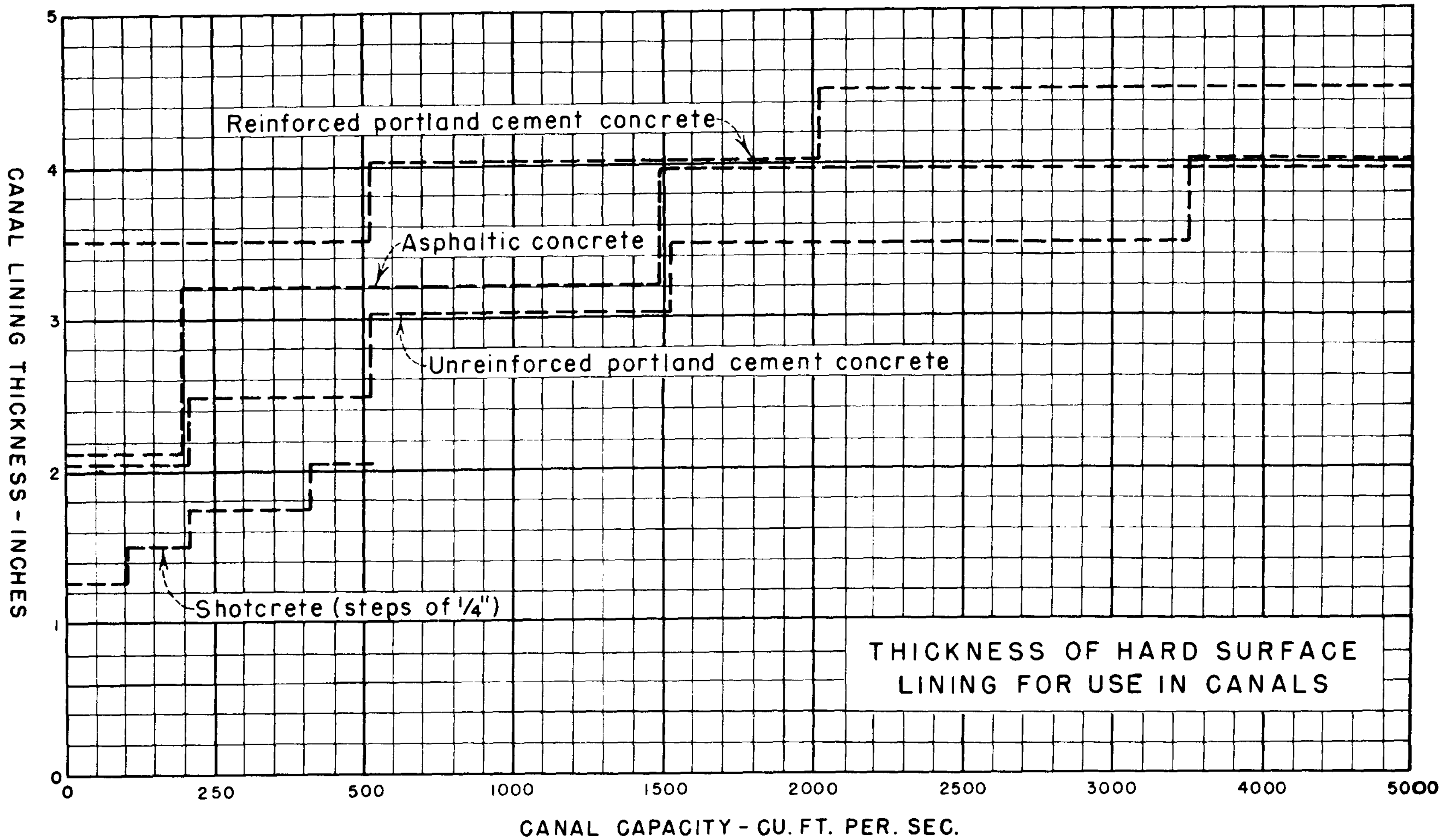


Figure 6.—Determination of thickness of hard-surface lining based on canal capacity. 103-D-706.

after they occur, the entrance of incompressible particles is prevented and expansion joints will ordinarily not be required except where fixed structures intersect the canal.

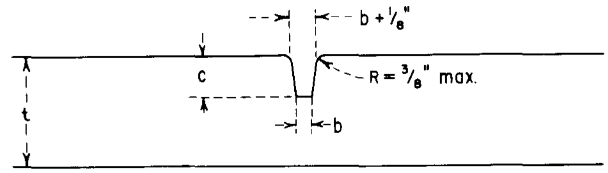
Contraction cracking, which results from tensile stresses produced in hardening or by a moisture or temperature decrease, is of primary concern in concrete lining. Canal lining cannot economically be designed to overcome cracking, but some control of cracking can be accomplished by the use of reinforcement steel or the forming of contraction joints at proper intervals. As discussed previously, the Bureau does not use reinforcement steel for this purpose, except in special cases such as for high-velocity channels. Where lining operations are continuous, Bureau specifications require a weakened-plane type joint or "sidewalk" groove formed in the concrete to a depth of about one-third of the lining thickness. If the grooves are properly formed and spaced, cracking will usually occur at these predetermined planes of weakness.

Both transverse and longitudinal grooves are advisable in concrete canals having lined perimeters of 30 feet or more. The recommended spacing of transverse grooves in unreinforced concrete varies from 10 to 15 feet, depending on the size of canal and the thickness of lining. Table 6 shows the recommended spacing and groove dimensions. A more detailed discussion of the spacing of grooves and the methods of forming them is contained in the Concrete Manuel,[3] under the discussion of contraction joints. Similar grooves are usually provided in shotcrete linings.

Expansion joints also may be required for shotcrete linings if the lining is placed in cold (less than 50° F.) weather. One-inch-wide joints at 100-foot centers were found effective in controlling buckling on one job.

(j) *Freeboard.*—The freeboard for hard-surface-lined canals will depend on a number of factors, such as size of canal, velocity of water, curvature of alinement, stormwater entering the canal, wind and wave action, and anticipated method of operation. As an example, canal reaches between pumping plants must be designed to anticipate unplanned pump outages

TABLE 6.—*Recommended groove dimensions for unreinforced concrete canal linings*



t, inches	b, inches	c, inches	Approximate groove spacing, center to center (feet-inches)
2	1/4 to 3/8	5/8 to 3/4	10-0
2 1/2	1/4 to 3/8	3/4 to 7/8	10-0
3	3/8 to 1/2	1 to 1 1/8	12-0 to 15-0
3 1/2	3/8 to 1/2	1 1/8 to 1 1/4	12-0 to 15-0
4	3/8 to 1/2	1 1/4 to 1 3/8	12-0 to 15-0

Dimension b and c show allowable tolerance

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and provide the required freeboard to accommodate bore waves and level pondage following such outages. The normal freeboard ranges from 6 inches for small canals to over 2 feet for large ones. The height of canal bank above the top of the lining usually ranges from 1 to 2 feet, depending on size of canal and local conditions. For very large canals, an analysis should be made to determine the proper bank height and lining freeboard. Figure 7 may be used as a guide.

A 2- to 6-foot berm is usually provided at the top of the lining for convenience of construction if a rail-guided lining machine is to be used. The tops of hard-surface linings should be turned horizontally into the bank a distance of 6 or 8 inches and backfill placed over this lining berm, except for asphaltic concrete linings which are generally rounded and tapered at the top before backfilling, as shown in figure 8. The backfill should be on about a 4 to 1 slope to the intersection with the earth bank, to prevent drainage water from entering between the lining and subgrade. It has usually been found desirable from a maintenance standpoint to provide a small windrow of earth at the inside top of bank and to slope the top of bank slightly away from the canal.

[3] "Concrete Manual," sixth edition, revised, Bureau of Reclamation, 1956.

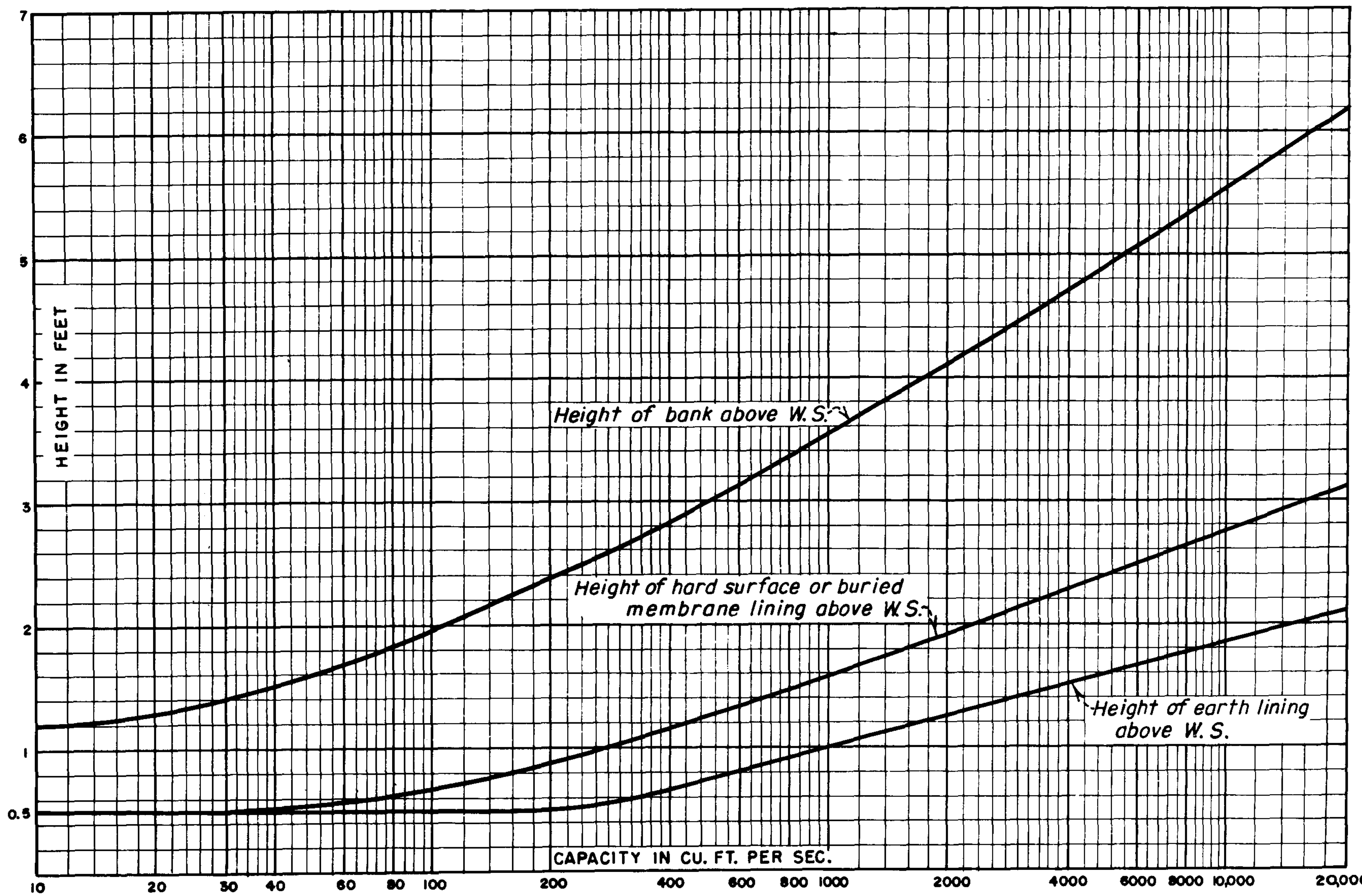

Figure 7.—Bank height for canals and freeboard for hard-surface, buried membrane, and earth linings. 103-D-341.

21. ***Hot-Mixed Asphaltic Concrete Linings.***—Hot-mixed asphaltic concrete is a carefully controlled mixture of asphalt cement and graded aggregate which is mixed and placed under elevated temperature. It is used as a surface lining for canals and as a resurfacing over deteriorated concrete linings (sec. 29). High density is desirable, so care should be taken to obtain the highest practicable degree of compaction. The recommended lining thickness can be determined by use of figure 6. As a resurfacing, the material is usually placed to a minimum thickness of 2 inches.

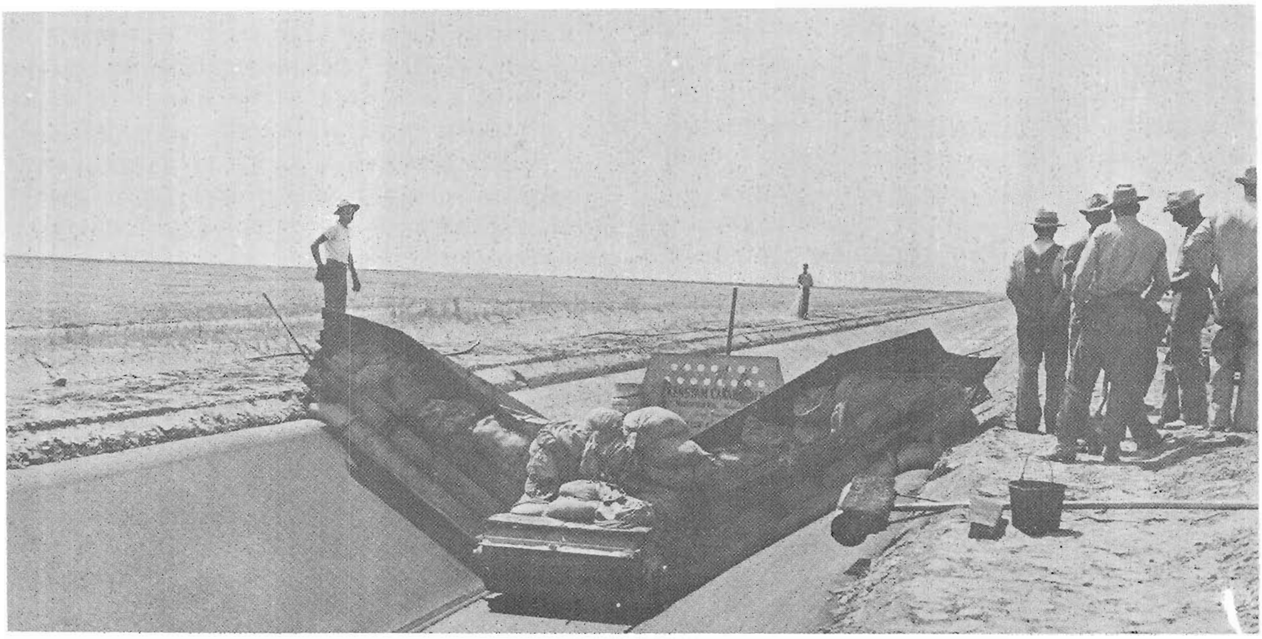

Figure 8.—Slip-form lining machine placing asphaltic concrete in the Pasco Pump Laterals. CH-63-29, 1947.

Asphaltic concrete lining is most satisfactorily placed by using slip-forms of either the subgrade- or rail-guided type because of the resulting economy and uniformly high density. This type of lining appears to be particularly well adapted to smaller canals which permit the use of the less expensive subgrade-guided slip-forms.

More than 330,000 square yards of hot-mixed asphaltic concrete have been placed by the Bureau on various projects. Earliest installations were placed in 1939 in the Contra Costa Canal, Central Valley project, California, and in the Snipes Mountain Canal near Sunnyside, Wash. In 1947, 88,000 square yards of asphaltic canal lining were placed on the Pasco Pump Lateral system near Pasco, Wash. This was the first time that a lining machine especially designed for placing asphaltic concrete (fig. 8) was used in Bureau construction. Several improvements were made in slip-forms used in the construction of subsequent installations.

(a) *Subgrade Problems.* — External hydrostatic pressures are harmful to asphaltic concrete linings, as they are to all hard-surface linings. Also, asphaltic concrete should be used with care when lining canals in heavy clay type soils which may expand upon saturation.

(b) *Weed Problems.*—Weeds are a potential hazard to asphaltic concrete canal lining when certain conditions favorable to weed growth occur. Growths have not presented a serious problem in installations to date, but weed growth is promoted by the heat-absorbing property of the black surface. Conditions favorable for weed growth are: (1) subgrade contamination with weed seeds or rhyzomes of perennial plants at the time the lining is placed; (2) subgrade moisture conditions favorable to seed germination or root growth; and (3) air or lining temperature favoring weed growth maintained for appreciable periods.

When lining is placed in areas previously irrigated or in old canals where such weeds as tules, cattails, or willows are firmly rooted, treatment of the subgrade with a soil sterilant is advisable. Such treatment will increase the cost of the lining installation, but for the average job the additional expense of material and application usually approximates only 10 cents per square yard. Pentachlorophenol in oil distillates, and chlorates and boron compounds in water solutions have been used as soil sterilants. A water solution of polyborchlorate applied by spraying directly to the subgrade prior to placing the lining is recommended for this purpose. Adequate sterilization will ordinarily be accomplished by the use of an equivalent of one-half pound of the powdered polyborchlorate per square yard of subgrade.

(c) *Reinforcement.*—In the 1939 Contra Costa Canal installation, one section of the asphaltic concrete was reinforced with wire mesh. This reinforcement may have contributed to the sloughing of a portion of the side-slope lining during the first summer. However, since this section was repaired, no further trouble has been experienced and this lining is still in fairly good condition. Steel reinforcement is believed to be of no value, or may even be detrimental, so it is not recommended for asphaltic concrete lining.

(d) *Experimental Mixes.*—In 1943, two areas of a canal on the Boise project, Idaho, were lined with hot-applied asphalt mixes. One section was lined with a sand-asphalt mix using a fine pit-run sand found adjacent to the canal. The lining was placed by hand methods to 1- and 2-inch thicknesses. This lining, however, lacked stability due to the poor sand grading, and considerable cracking has resulted from the slow slippage of the lining down the sides. The second section of lining was constructed of asphaltic concrete. This lining, which also was placed by hand, was constructed in 1-, 2-, and 3-inch thicknesses to determine the effect of the thickness factor. After 15 years of service (fig. 9), little difference is discernible in the appearance of the surface of the various thickness sections, except that some willows have grown through the 1-inch lining and external hydrostatic pressures

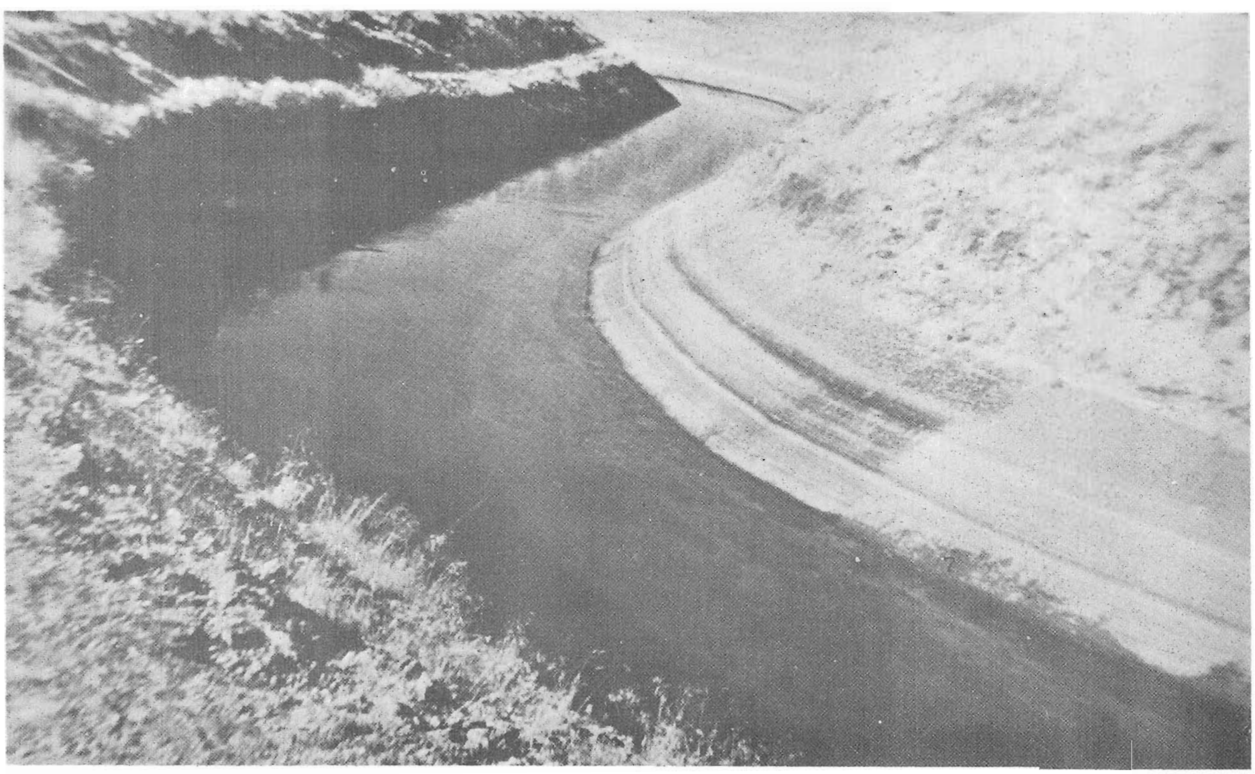

Figure 9.—A hot-mixed, asphaltic concrete canal lining after being in use for 15 years in the northwestern United States. P3-D-15353.

have resulted in some bulging and spalling near the toe of the slopes of the thinner linings.

In 1944, approximately 11,000 square yards of 2-inch-thick asphaltic concrete was placed by hand in a main canal and a small lateral on the Owyhee project, Oregon. Sodium chlorate was used as a soil sterilant because of an abundance of willows in the canal and lateral. The mix contained coarse aggregate and an 85-100 penetration asphalt (harder than any previously used). The coarse aggregate produced a lining surface that was quite rough and somewhat open, and because of this, a heavy seal coat of low-penetration asphalt containing diatomaceous earth and other fillers was applied to the surface of the lining by a squeegee. By 1946, this seal coat had cracked badly above the waterline and minor cracking was evident below the waterline. Since that time, pronounced checking or "alligator cracking" has occurred throughout both sections of the lining. Some of the cracks that originated in the seal coat extended through the asphaltic concrete to the subgrade.

In 1957, it was considered necessary to rehabilitate this lining to prevent complete failure. Accordingly, nine test panels of experimental surface treatment materials were installed to provide data for use in the selection of a low-cost repair method. An evaluation made of these test materials 1 year later indicated that none were entirely satisfactory for repairing the occasional wide cracks. Consideration is being given to other materials and methods for rehabilitating this original 2-inch-thick asphaltic concrete lining.

(e) *Field Performance.*—The first large-scale construction of asphaltic concrete lining was made on the Columbia Basin project, Washington, in 1947. This installation consisted of about 88,000 square yards of 2-inch-thick lining placed by a subgrade-guided slip-form. Although tests indicated a low degree of weed seed infestation in the subgrade soil prior to lining, approximately one-fourth of the area was treated with a sterilant (sodium chlorate and boric acid). A 60-70 penetration asphalt-cement was used in most of this lining, but a small portion contained 50-60 penetration asphalt cement for comparison. Some minor repairs to the lining have been required to date, but it is generally in good condition. Considerable transverse cracking, believed to have been aggravated by extremely low temperatures (—30° F.) during the winter of 1948-49, is shown in figure 10. Wider at the top and diminishing in size toward the toe of the slopes, the cracks are more predominant on the side of the canal or lateral exposed to the sun. They appear to have reached a maximum in size in a few years' time, and those shown in figure 10 are to be cleaned and repaired with pneumatically applied portland cement mortar. At least a partial solution to this problem is believed to be greater lining density, and improvements in mixes, materials used, and placing equipment. Laboratory studies to this end are being continued. Heaving of the bottom and bulging of the side slopes due to hydrostatic head and frost action under or behind the lining is another deficiency, but it appears no more serious than that found in portland cement concrete linings under similar conditions.

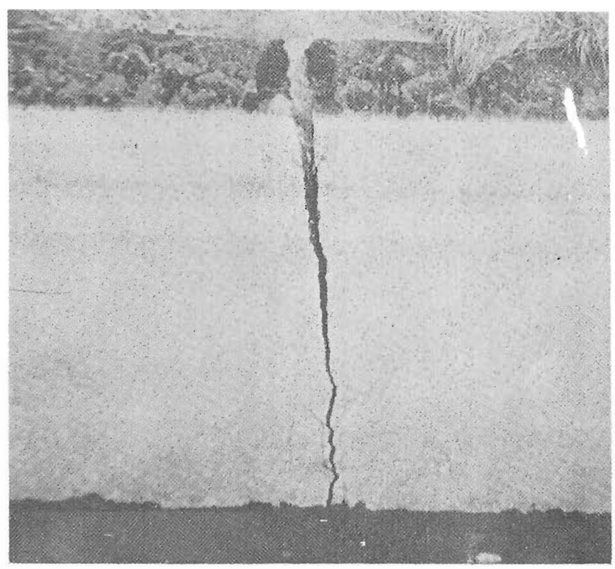

Figure 10.—Typical cracks caused by low temperature (—30° F.) in a 2-inch-thick asphaltic concrete canal lining in the State of Washington. P222-D-15355.

In 1939, asphaltic concrete lining was installed by the Civilian Conservation Corps on the Yakima project, Washington. Since the canal had been in operation for some time prior to lining and was heavily infested with a variety of weeds, various soil sterilants were used before application of the lining in the three sections of the canal. In 1946, some weed eruption through the lining was noted in the section where sodium chlorate had been used as a sterilant; the chlorate sterilant had evidently been leached from the soil during the 7-year period since construction. Consequently, boron compounds (principally borax and boric acid) are favored for use in conjunction with chlorates in sterilant treatments, because the borates are effective sterilants and tend to leach out more slowly. In 1958; with the minor exception of a few distressed areas where weed growth had erupted through the lining, the lining was in very good condition in all sections.

(f) *Mix Design.* — Asphaltic concrete mixes are designed for watertightness. Mixes for canal linings are higher in asphalt content (7 to 10 percent) than those used in highway construction. Certain phases of the mix design are a compromise between high plasticity, to minimize cracking from possible subgrade movements, and hardness to obtain good erosion resistance and slope stability.

Most asphaltic concrete lining placed in Bureau canals and laterals has been placed by subgrade-guided slip-forms developed and built by the contractors. Using local materials where possible, laboratory studies and field experience are used to establish a mix that has a high degree of workability (while hot) and will permit slip-form placement to an adequate density. An average density exceeding 92 percent of the Bureau laboratory standard density has been specified. With the development of new vibratory compactors and pneumatic rollers, it appears reasonable to specify 94-96 percent density without increasing construction costs.

Mix design is generally accomplished by using an immersion-compression test similar to ASTM D-1074-58T.[4] The following tabulation shows gradations used in Bureau specifications for canal linings:

Sieve size	*Percent passing*
3/4 in.	100
1/2 in.	85 to 100
No. 4	55 to 100
No. 10	35 to 60
No. 40	18 to 30
No. 200	5 to 12

[4] "Kennedy's Method of Test for Compression Strength of Bituminous Mixtures," ASTM D-1074-60, ASTM Standards 1961, part 4, pp. 922, 1065-1069.

In the earlier linings, a relatively soft asphalt-cement having a penetration of 100 to 200 was used. Later studies have shown that an asphalt-cement having a penetration of 50-60 is more satisfactory. This somewhat harder asphalt produces a mix that is more resistant to weed growth, is more stable on slopes, and, owing to the thicker films of asphalt on the aggregate, will probably prove to be more durable.

The design of an asphaltic concrete lining involves many factors which can only be adequately evaluated through laboratory testing. Each installation must be given individual consideration relative to selection and usage of materials. Therefore, it is important that prior to construction of major lining installations, the proposed materials are tested in the laboratory to provide data necessary for proper mix selection and control. This information is incorporated into the construction specifications in order to establish the minimum requirements of the asphaltic materials, set the limits for the aggregate grading, and establish minimum acceptable density.

(g) *Construction Methods and Equipment.*— Preparation of the subgrade involves trimming to relatively wide tolerances and may be accomplished with equipment of rather simple design. Satisfactory subgrade preparation has been secured in small canals with plow-type ditchers pulled by one or two tractors.

The carefully proportioned mixtures of sand, gravel, and asphalt-cement, mixed hot in a central plant (usually at about 325° F. in batches of 1,000 to 2,000 pounds), and delivered to the canal in dump trucks, is deposited in the slip-form usually by clamshell bucket as the equipment moves along the canal (fig. 11). In the slip-form, the hot material (which retains heat well) is diverted to the sides and bottom of the canal by wings built in the machine, and the mix is struck off by a template to allow sufficient thickness for later consolidation. This compaction or consolidation operation, which plays a most important part in the formation of the proper surface, is accomplished by a weighted or vibrating ironing plate fastened to the bulkhead behind the strike-off screed. The iron remains hot from contact with the hot mix and is adjustable for varying the pressure on the asphalt mix. No joints are

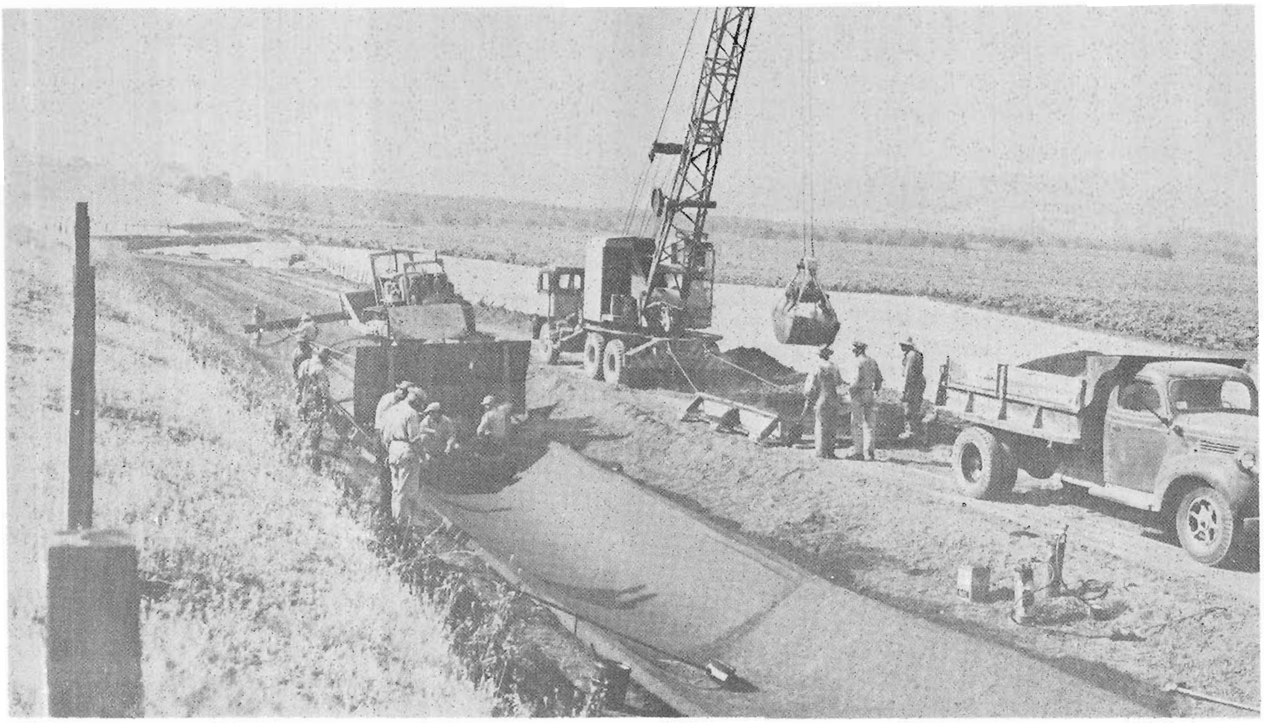

Figure 11.—Hot asphaltic concrete lining being applied to canal prism by tractor-drawn slip-form. CH-102-26.

required in the lining. Since no curing, sealing, or other treatment is required, the lining is ready for use immediately after cooling. In some installations, air temperatures which were below freezing during the placement operation have not caused any harmful effects to the lining.

Where lining protection is required on one side slope of a canal only, it has sometimes been accomplished by placing a 4-inch-thick asphaltic concrete lining on the slope. In some instances the downhill side-slope lining has been joined to a vertical concrete cutoff wall located in the canal bottom near the toe of the slope, as shown in figure 12. In this installation the asphaltic concrete was dumped into a portable hopper from which a short belt conveyor deposited the material onto the side slope. The material was then smoothed and partially compacted by a heavy steel screed pulled up the slope by a dragline. A plate vibrator welded to the steel base plate of the screed aided in compaction of the mix, which was at a temperature of about 300° F. Timbers 4 inches square were set at 11- to 12-foot intervals to support the screed. After the screed supports were removed, the gap was filled with hand-tamped asphaltic concrete. Single-drum smooth steel rollers were used to obtain additional compaction after the asphaltic concrete had cooled somewhat. An efficient crew with adequate equipment can place 1,000 to 1,800 linear feet of lining on 18- to 20-foot-long slopes in 1 day.

(h) *Cost and Maintenance.*—The construction costs of asphaltic concrete linings have varied according to the availability of materials and equipment and the lining thickness. The cost of $2.30 per square yard for 2-inch-thick linings placed by contract during the period 1953-1958 is about equal to the cost of slip-form-placed portland cement concrete linings of the same thickness (see table 1, sec. 2).

Unfortunately, few maintenance and repair costs are available for asphaltic concrete linings. A 2-inch-thick asphaltic concrete lining on the

Figure 12.—Construction of 4-inch-thick asphaltic concrete lining on downhill side slope, New York Canal, Boise, Idaho. The lining is being joined to a vertical concrete cutoff wall near the toe of the slope. CH-538-60, November 1957.

Columbia Basin project over a period of 8 years has averaged about $330 per mile, which includes general repair; removal of sand, silt and debris; and weed control. Of this total, lining maintenance has been estimated at about $31.58. Since the costs are based on limited data, they should not be used for comparison with similar costs for other linings.

(i) *Repair.*—Repairs of asphaltic concrete linings have been accomplished primarily by using portland cement concrete or mortar, because of a lack of suitable available asphaltic materials. Few projects are conveniently located to a source of hot-mixed asphaltic concrete, which would probably be the most satisfactory material for such repairs. The cold-mixed asphaltic emulsions and mortars and ready-mixed asphalt materials presently on the market have not been entirely successful as repair materials. Tests are underway on newer products for the repair of cracks and for the revitalizing and filling of surface crazing.

Although none of the resurfacing materials used to date have been entirely successful, many of them can be considered suitable to a limited degree in the absence of something better, and can be used to delay further deterioration by frequent application.

22. *Cold-Mixed Asphaltic Concrete.*—Cold-mixed asphalt linings are similar to the hot-mixed type previously discussed in that well-graded aggregates and asphalt are mixed and compacted in place. However, it is necessary to cure cold mixes which requires time and favorable weathering conditions. Furthermore, some cold mixes tend to remain soft indefinitely while others (emulsions) contract in curing, thereby creating cracks which must then be filled in some manner. Since cold mixes tend to exhibit low erosion resistance and poor stability for an appreciable period of time after placing, some of the advantages in ease of mixing and placing the materials are nullified.

Cold-mix linings have been constructed in Bureau canals on an experimental basis. These linings have developed numerous shrinkage cracks during curing, making subsequent filling of the cracks with a slurry of the sand and asphalt emulsion necessary. In general, the linings of this type exhibit poor bond between the aggregate and asphalt, they are easily damaged by stock traffic, and there appears to be a general deterioration of the surface with time due to exposure and erosion.

Only limited satisfactory service has been obtained to date from cold-mixed linings. In general, hot mixes are preferred to cold mixes where construction conditions may involve unfavorable weather, where the period available for curing is limited, or where appreciable erosion conditions exist. However, new additives which are now available appear to improve considerably the quality of cold-mix lining, so that satisfactory service may be possible in the future. Testing of these additives is now in progress.

23. *Asphalt Mortars.*—Experimental pneumatically applied asphalt linings have been constructed by spraying a mixture of fine sand and asphalt emulsion on the subgrade. With some modification, the same type of equipment is used as in placing shotcrete. The pneumatic method is particularly advantageous in covering exceptionally rough surfaces where use of a slip-form is impracticable, but the slow rate of application and high cost do not favor general use of the pneumatic method.

One pneumatically applied asphalt lining was placed on a lateral of the Orland project (fig. 13), and another on a larger canal of the Central Valley project, both in 1954. In the Orland tests various proportions of sand, portland cement, asphalt emulsion, and water were used. The material was placed 1½ inches thick over a sterilized earth subgrade. On the Central Valley project the lining was 2 inches thick on the side slopes and 3 inches thick in the bottom. The linings on both projects are being observed regularly for comparison of service with pneumatically applied portland cement mortars. The lining on the Central Valley project is in good condition after 6½ years of service.

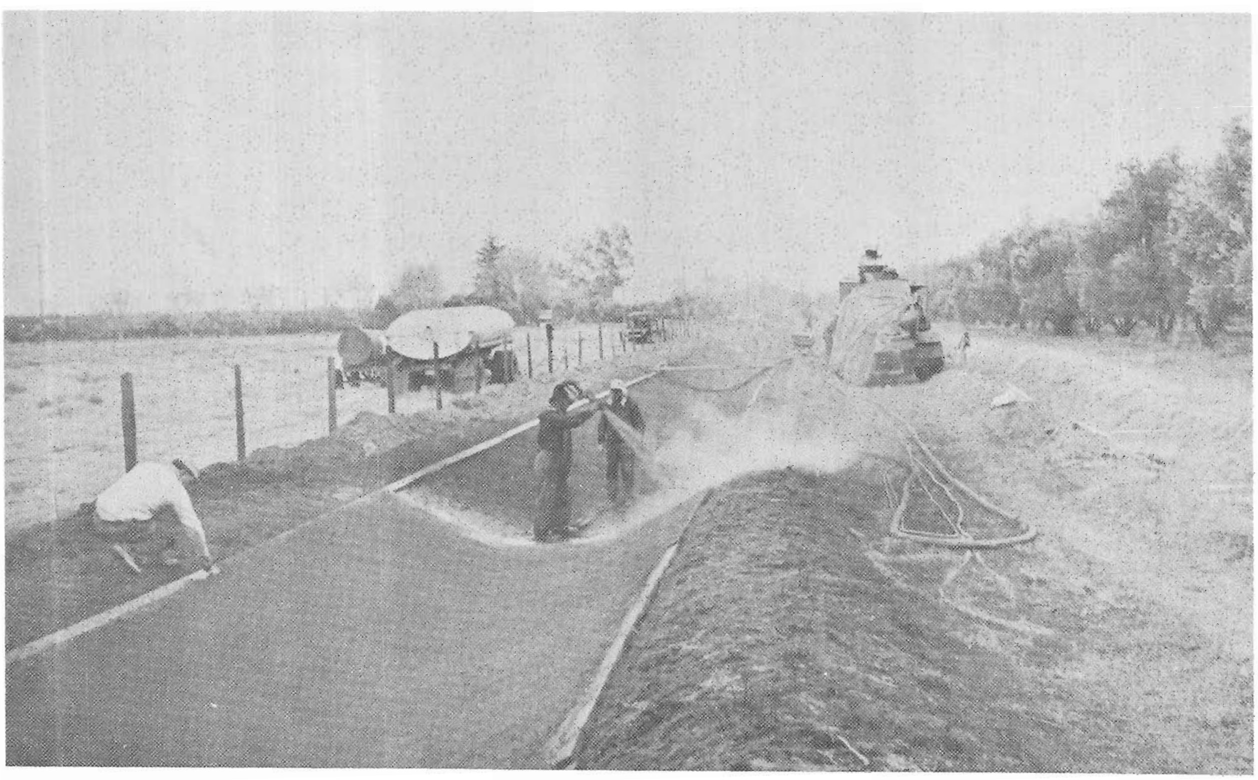

Figure 13.—Installation of pneumatically applied asphalt emulsion mortar canal lining over sterilized earth subgrade on the Orland project, California. P22-D-14665, January 1954.

Two significant changes in the otherwise standard portland cement shotcrete equipment were required in the asphalt applications. A water ring with larger holes and the introduction of the asphalt emulsion into the nozzle through a separate third hoseline provided the thorough mixing required, without binding in the nozzle. Satisfactory mixes were produced using from 8.1 to 11.8 percent of asphalt emulsion (slow setting type), with sand, water, and 0, 3, 5, and 10 percent portland cement based on the dry weight of sand. A total of 775 square yards, 1 inch thick, was completed in 1 day as a resurfacing material for deteriorated portland cement concrete lining.

A pneumatically applied asphalt lining, utilizing a rapid curing cutback and a fairly coarse-graded aggregate, was placed on a farm lateral near Calipatria, Calif., in 1945. This lining was still very soft when examined a year after construction, and had suffered some damage from turbulent water adjacent to check structures.

Pneumatically applied and hand-applied asphalt mortars have also been used in repairing portland cement concrete as discussed in a subsequent section.

24. *Asphalt Macadams.*—An asphalt macadam lining consists of a layer of relatively coarse graded aggregate penetrated with an asphalt to form an erosion-resistant yet flexible surface. This type of lining has been used only on an experimental basis in Bureau work to date. In these installations a number of factors have been investigated which include thickness of the macadam, quantity and type of asphalt, techniques of application, methods of placing the aggregate, type and grading of the aggregate, and side slope steepness. Light sand cover was also used to determine the value of a "choking" course on the surface.

Macadam has been used with and without an underlying membrane of sprayed-in-place asphalt or prefabricated asphaltic materials. The first installations were made in the channels on the Bureau's Experimental Canal Farm at the Denver Federal Center, Denver, Colo., in 1950. These tests were followed by trial installations on the Orland project, California, in 1950; and on the Shoshone and Kendrick projects, Wyoming, in 1951. Later installations were made on the Central Valley project, California, (fig. 14); the Riverton project, Wyoming; and the Huntley project, Montana.

Figure 14.—Construction of penetrated macadam, showing application of catalytically blown asphalt cement at 400° F., on the Madera Canal, Central Valley project, California. PX-D-32054.

Catalytically blown asphalt cements appear to provide a more permanent and superior macadam than other asphalts, and coarser gravels appear to be more easily penetrated by the asphalt. Fewer weeds were observed to be growing through the tougher, more durable catalytically blown asphalt cement macadam. Where asphalt emulsions and cutbacks were used as penetrants, there was a lack of bond between the asphalt and gravel, particularly below the waterline or where the macadam had been broken, and they did not provide the required stability when installed on steep slopes. Sagging near the top of the slopes was also observed. Asphalt emulsion generally was found washed from the surface aggregate, permitting a general and progressive surface erosion. Experiments using pit-run gravels and finely graded materials indicate inadequate penetration of the aggregate by the binder in most instances.

In general, cracking of most macadams with the cracks extending through the lining to the subgrade, has been noted in experimental installations. (However, one reinforced macadam installation on the Kendrick project, Wyoming, remains in good condition after 10 years of service.) It was concluded that it is difficult if not impracticable to construct a macadam lining that is completely watertight without the use of excessive amounts of asphalt and that, in general, macadam should not be considered for use as a lining but can be used for stabilization and as a cover for an underlying seepage control membrane. The use of macadam as a cover over an underlying membrane is more fully discussed in section 40.

25. *Prime-Membrane Linings.*—Prime-membrane linings, the forerunner of buried asphalt membrane linings, were constructed by first priming or penetrating the soil (which must be of a type permitting such penetration) with light fuel oil or distillate, and then placing a surface membrane of asphalt over the soil thus stabilized. After application of the deep penetrating primes, a slow, medium, or rapid curing asphalt cutback was sprayed on at the rate of about one-fourth gallon per square yard per application, until about 2 gallons had been applied per square yard. Under favorable conditions, penetration depths up to 3 or 4 inches were obtained. After a short period of curing, hot asphalt-cement of 85 to 100 penetration filled with 10 to 20 percent diatomaceous earth was sprayed over the surface to form an exposed membrane.

Most of the linings constructed in this manner have suffered failure in a relatively short time. The prime-membrane lining has proved expensive; critical as to weather and soil conditions such as density, capillarity, moisture content and temperature; and subject to excessive injury by livestock. Furthermore, these linings require extended periods of construction and costs usually exceed $1 per square yard. Consequently, the prime-membrane lining is no longer considered a potential low-cost lining type.

The first trial prime-membrane lining was applied on a canal of the Boise project in 1942. Approximately 13,500 square yards of similar lining have been constructed on several projects. These installations are, with only a few exceptions, in poor condition (fig. 15) or have been replaced with other types of linings.

26. *Prefabricated Asphaltic Linings.* — Prefabricated asphaltic materials were developed to permit use of an asphalt membrane but avoid the use of hot materials requiring skilled personnel and special equipment. Thinner (1/8- to 1/4-inch-thick) prefabricated lining materials are designed to be handled and placed in much the same manner as rolled roofing, with lapped and cemented joints. They may be covered with earth or otherwise protected from damage, in which form they are classified herein as buried membranes and discussed in section 39. Thicker materials which may be used without cover or protection have also been developed.

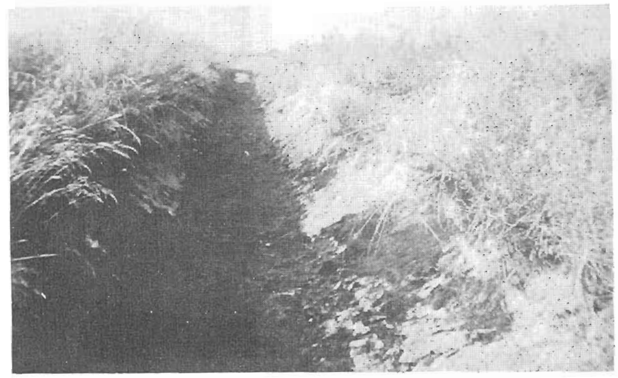

Figure 15.—A prime-membrane canal lining. Note deteriorated condition of lining after only a few years' service. PX-D-32051.

In early experiments prefabricated lining using thick layers of asphaltic mixtures carried on heavy reinforcings was placed without additional protection. Linings of this type were placed on the Boise project, Idaho, in 1944 and 1946 and on the Yakima project, Washington, in 1947. A proprietory block product, which consisted of an asphalt-impregnated canvasback supporting a 1-inch-thick asphaltic concrete, is an example of the original prefabricated, exposed type lining. These and similarly constructed 34- by 60-inch slabs were laid with filled asphalt-cement butt joints. The blocks and slabs (figs. 16 and 17) were in good condition when examined in the fall of 1961; but the asphaltic filler in the joints above waterline had deteriorated, with that remaining in the joints being dry and hard. Joints below the waterline were in better condition and the mastic had remained in place. The lining is still effective in the control of seepage from the canal.

A second early type of prefabricated lining material included a sheet of asbestos which served as a reinforcement backing for a sheet-asphalt mixture one-half inch thick. These

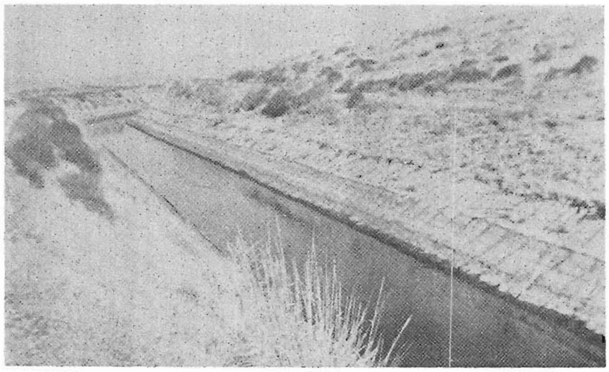

Figure 16.—A lining comprised of prefabricated asphaltic blocks. P-3-D-15392.

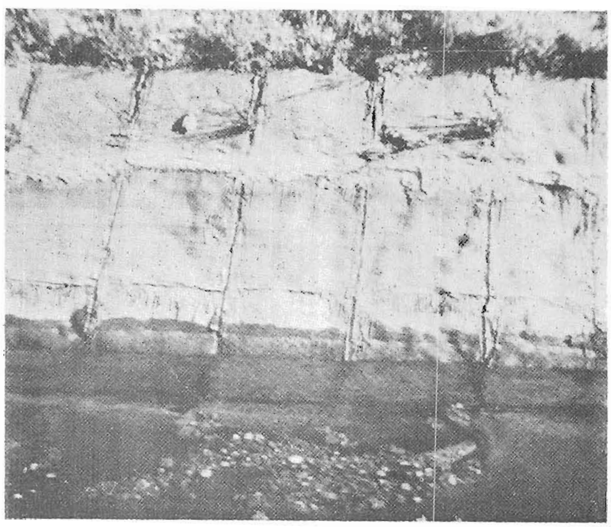

Figure 17.—A lining constructed of hot-paving-mix asphaltic concrete prefabricated slabs. P3-D-15393.

early types of exposed lining materials were heavy and bulky, making shipment for long distances impracticable and material placing costs excessive.

During the past 9 years (present time 1962), the Bureau has installed numerous trial sections of a newer type of exposed prefabricated asphaltic material. This type is essentially a ½-inch-thick sheet manufactured in sections 3 or 4 feet in width and up to about 25 feet in length. The sheet is made using a sandwich type construction consisting of a core of reinforced or filled asphalts with an asphalt-saturated felt on either side and, in addition, a "weathercoat" on the exposed side. The material is installed over a smooth-rolled and sterilized subgrade without special equipment or skilled workmen. Adjacent sheets are joined using a 3-inch lapped joint with a mastic or lap cement (fig. 18). This material also has been used experimentally as a resurfacing over deteriorated concrete linings.

The cost of this newer type of prefabricated asphalt canal lining material is about $0.80 to $0.90 per square yard fob the factory. Depending on the freight to the site of the work and the requirements for subgrade preparation, the installed costs have been from $1.35 to $1.75 per square yard for the completed lining. Observations made regularly of the installations indicate no failures of materials from weathering or maintenance, such as the removal of silt. Since the sheets are light in weight, a few failures have been caused by water getting under them. To protect the lining against this possibility, the joints must be carefully made and properly maintained. Cutoffs at frequent intervals will reduce the extent of damage if lifting should start.

Underwater lining of canals with prefabricated asphaltic materials was tried on the Yuma project in March 1956. A short test section of ½-inch-thick exposed type prefabricated material was installed in a canal while in service (1) to provide information as to the feasibility of this method of lining without removing the water from the canal, and (2) to develop and improve installation techniques. Five sheets of the ½-inch-thick prefabricated lining measuring 4 feet wide by 23 feet long were assembled on a barge into a single panel with 3-inch lapped joints, the barge being anchored near the canal bank. With the panel anchored at the berm, the barge was pulled laterally from beneath the panel while rollers and sand bags were used to submerge the panel. The panel provided lining for one-half the canal's cross section. The same procedure was used from the opposite bank to complete the lining.

The method appeared feasible, but further investigation was considered necessary to provide a watertight seal in the overlapping joints that must be made underwater. This lining method may have value in canals that must remain in constant use and cannot be dewatered for a more conventional lining installation.

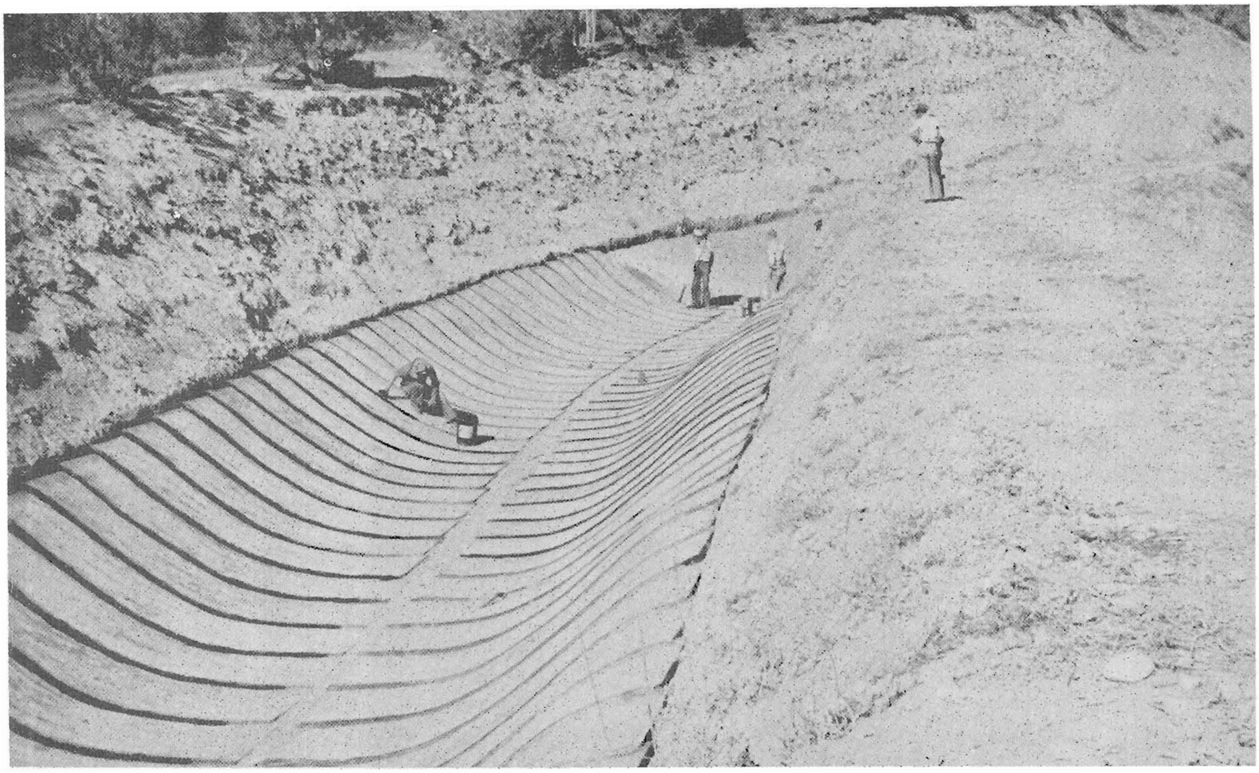

Figure 18.—Installation of exposed prefabricated asphalt canal lining on the Paonia project, Colorado. P551-D-14663.

27. ***Other Exposed Asphalt Linings.***—An experimental jute-reinforced asphalt lining has been under study by the Agricultural Research Service and Utah State University as a part of the cooperative work with the Bureau's lower cost canal lining program.[5] Installations made to date have been of both three-ply and five-ply construction.

Construction of the three-ply lining is begun by spraying catalytically blown asphalt onto the prepared subgrade of a canal at the rate of 1 gallon per square yard. Upon this, a layer of 10-ounce mildewproof burlap is placed while the asphalt is still hot. The burlap is then treated with a second application of the asphalt at the rate of three-fourths gallon per square yard. The five-ply lining includes, in addition, a second layer of burlap and then a third layer of asphalt applied at the rate of one-fourth gallon per square yard.

[5] Lauritzen, C. W. and Haws, F. W., "1959 Annual Research Report," USDA Agricultural Research Service, SWC, and Utah State University, Logan, Utah, January 1960.

After 3 years of service in both hot and cold climates, these catalytically blown asphalt linings appear to be in good condition, and tests show excellent seepage control. Evaluation of the linings will be continued. Field costs of the five-ply lining installed by project forces, including the cost of a soil sterilant applied to the subgrade before placement of the first layer of asphalt, are reported to be about $1 per square yard.

28. ***Portland Cement Concrete Linings.***—(a) *Service History.*—From a study of the many miles of concrete lining now in existence, it is concluded that such a lining, if properly designed, constructed, and maintained, will have an average serviceable life of over 40 years. Long reaches of 3- to 4-inch-thick lining, both with and without steel reinforcement, placed from 1910 to 1916 on the Bureau's Umatilla, Yakima, Boise, and Strawberry Valley projects in Oregon, Washington, Idaho, and Utah, respectively, are in good condition and with a minimum of maintenance should serve for many more years.

Many other linings of comparable age in canals of both Bureau and private irrigation systems attest to their long life.

Portland cement concrete linings placed by hand methods at, or just after, the turn of the century by private irrigation interests in the mild climate of southern California are still in service today. Farther north in the Central Valley of California many miles of lining were placed in canals and laterals after 1920 by a variety of methods, including hand labor in the smaller canals. The many extensive installations of old and new linings in the Central Valley add considerably to the overall service experience. Considering the extent and variety of conditions where portland cement concrete linings have been used, their service history is excellent.

(b) *Causes of Failure.*—There have been failures of portland cement concrete lining due to adverse subgrade conditions, such as loss of support through piping action and bulging of expansive clays. Also, excessive hydrostatic pressures beneath the lining, frost heaving, surface damage from freezing and thawing, poor quality of concrete, faulty design or construction

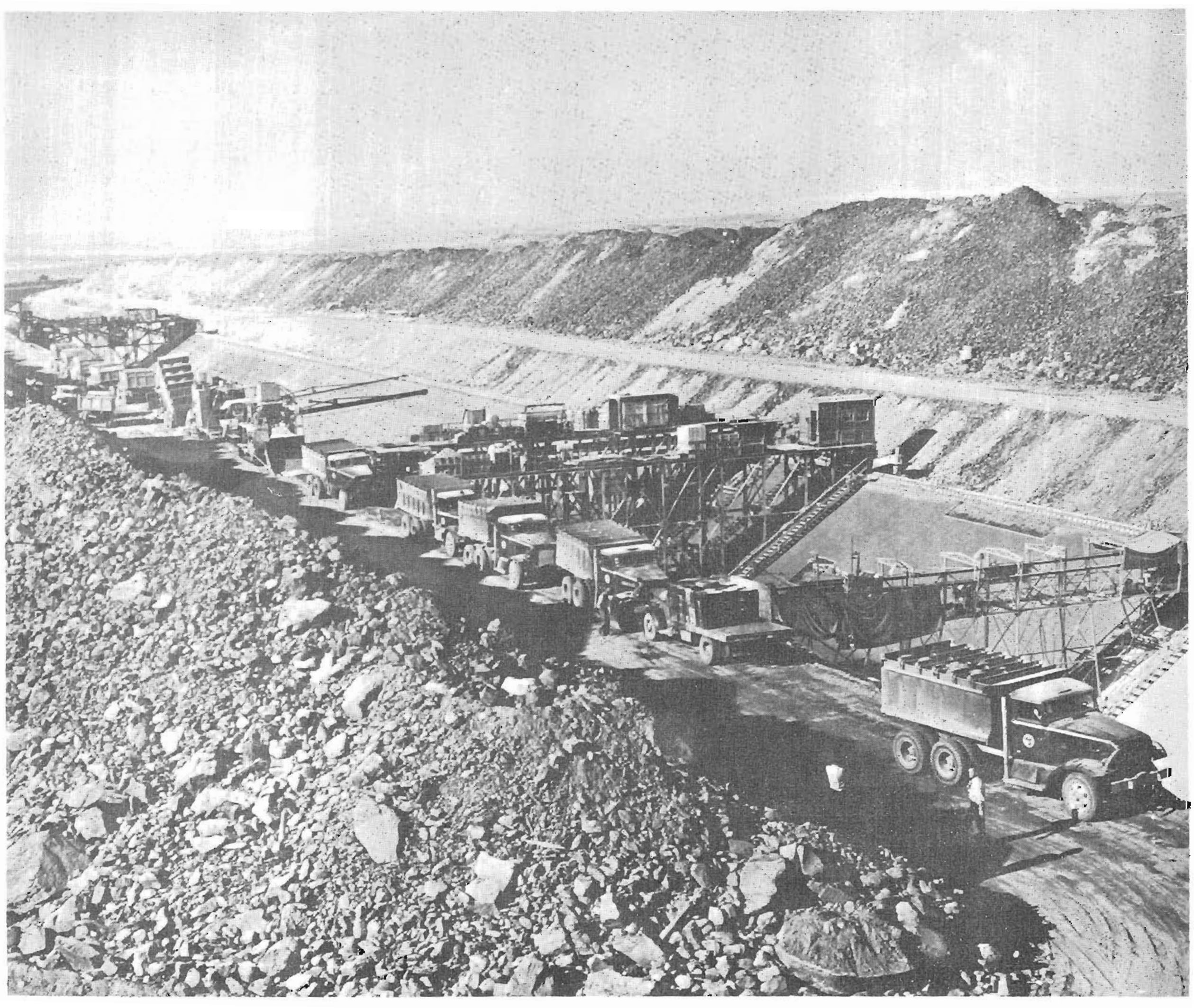

Figure 19.—Concrete canal lining operations on East Low Canal, Columbia Basin project, Washington. Separate jumbos are used for trimming, lining, grooving, and applying sealing compound. Batch trucks and mixers operate on the berm of the canal. PX-D-32055.

methods, or combinations of these and similar factors have caused failures.

In northern climates where considerable subfreezing weather is encountered, frost heaving is undoubtedly the greatest factor in the destruction of concrete linings. In areas where the subgrade is not free draining, provision for adequate drainage is perhaps the most effective protection against frost heave (sec. 20).

(c) *Construction Methods and Equipment—Large Canals.* — Portland cement concrete for canal lining may be economically placed by a variety of methods. Jobs involving considerable lengths of large canals usually utilize longitudinally operated slip-forms supported on rails placed along both berms of the canal (figs. 19, 20, and 21), or track-laying (crawler type) machines such as those used more recently on the Central Valley and Solano projects, California, and the Columbia Basin project, Washington (fig. 22). Short lengths of large canals often do not justify the use of slip-form machines; these may be lined by use of winch-drawn screeds operating transversely up each side slope (fig. 23).

The large slip-form machines supported on railroad rails have been in general use for about 20 years. The equipment is highly mechanized and consists essentially of a framework which travels on rails (one on each bank of the canal) supporting a working platform, either distrib-

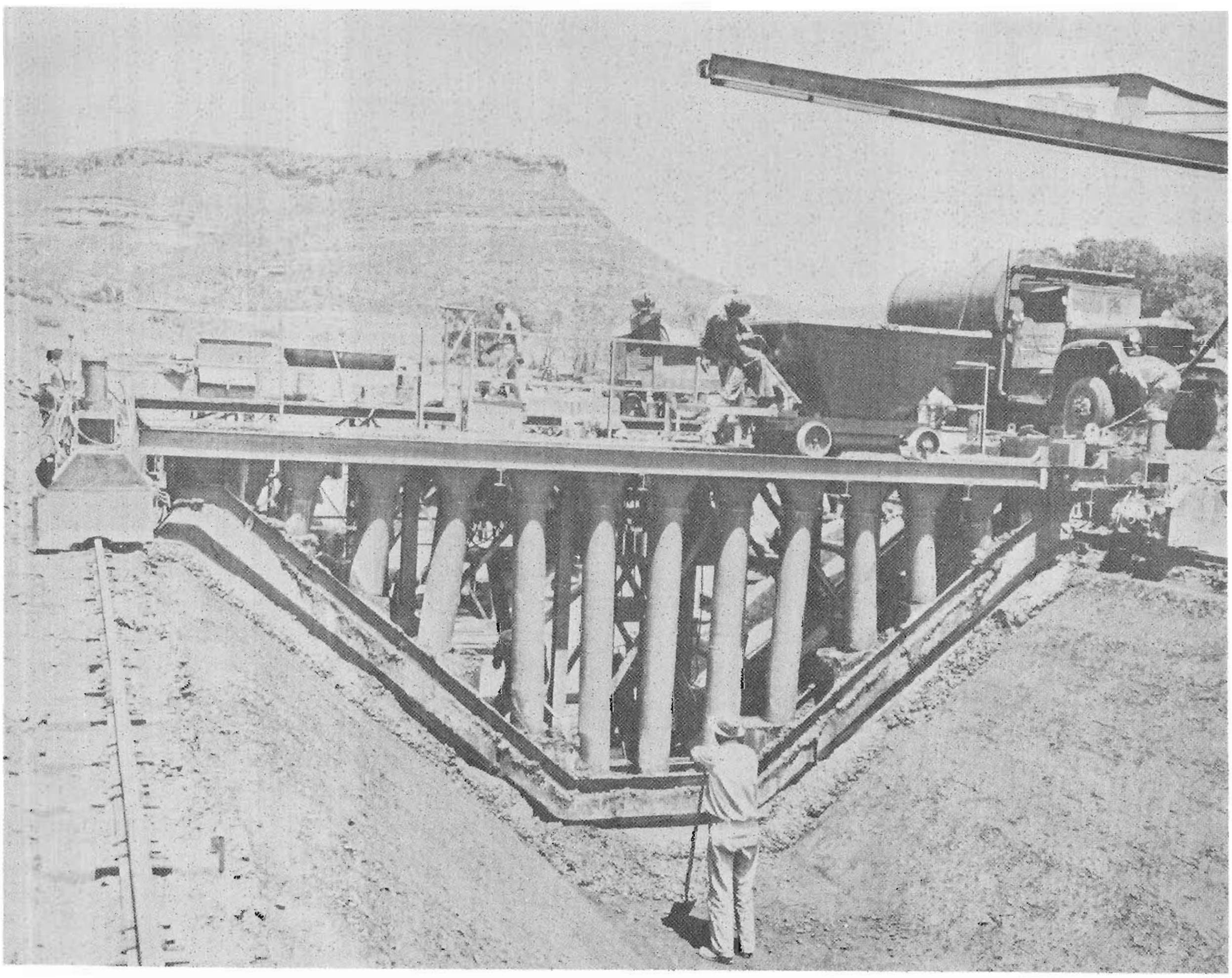

Figure 20.—Rail-supported concrete lining machine with drop chutes progressing along the Charles E. Hansen (Horsetooth) Feeder Canal, Colorado-Big Thompson project, Colorado. 245-704-647.

utor plates or drop chutes, a compartmented supply trough, a vibrator tube in the bottom of the trough, and the slip-form. The latter is a steel plate curved up at the leading edge, extending across the bottom and up the slopes of the canal, which forms and smoothes the finished surface of the lining. If a distributor plate is used, it is fastened to the leading edge of the slip-form and extends upward on a steep incline to the working platform at the top of the machine. The concrete is dumped onto the distributor plate which serves to spread the concrete as it is deposited directly ahead of the slip-form. If drop chutes are used, these supply concrete from a row of hoppers in the working platform to compartments in the trough below. Concrete is usually dumped into the working platform hoppers or onto the distributor plate from a shuttle car loaded by bucket or conveyor belt from mixers on the bank. As the concrete passes out at the bottom of the trough and under the slip-form, it is consolidated by a vibrating

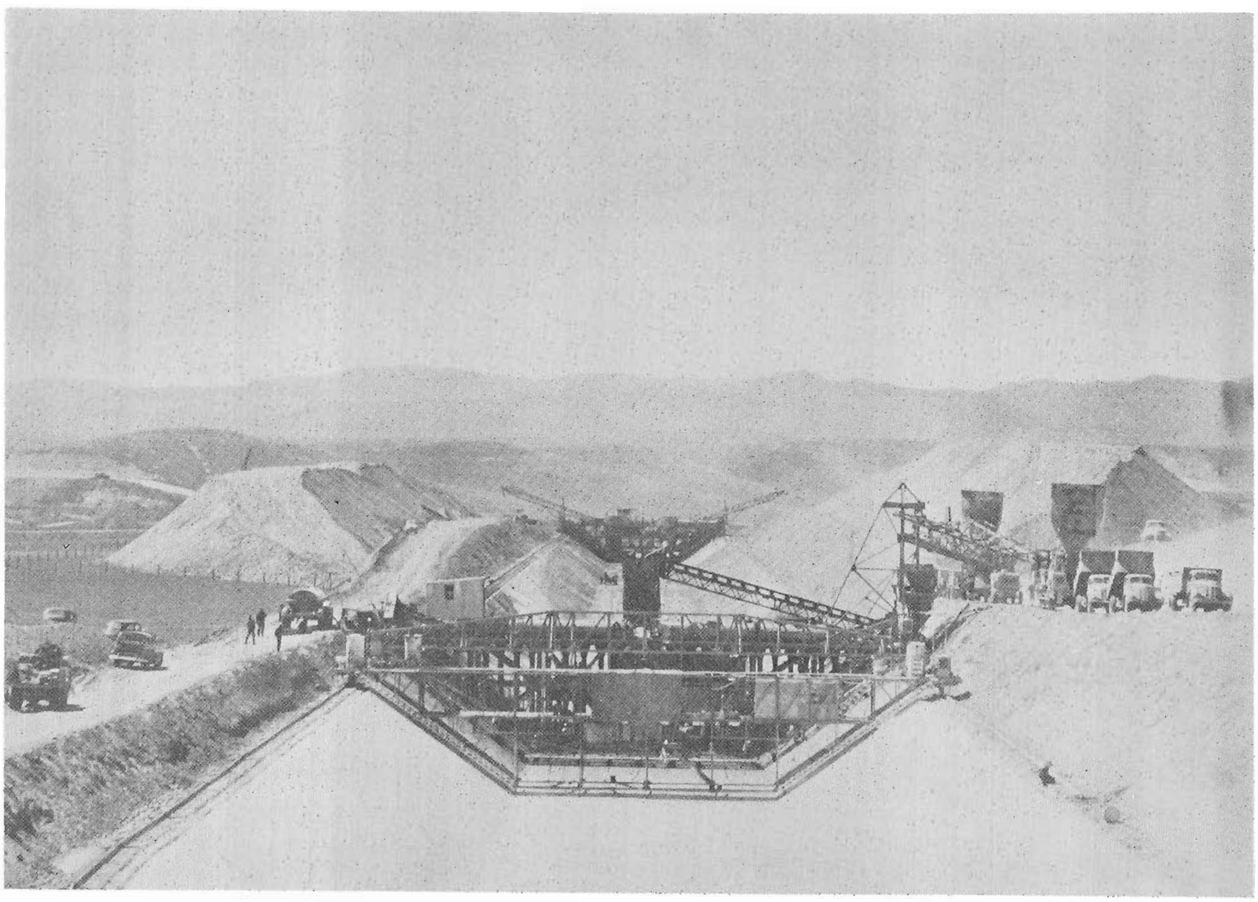

Figure 21.—Fully mechanized concrete lining operations in a large canal. Canal trimmer in background is followed by slip-form lining machine and jumbo for workmen applying sealing compound. Batch trucks and mixer operate on right berm. Delta Mendota Canal, Central Valley project, California. DM-762-CV.

tube, having a minimum frequency of 4,000 cycles per minute, mounted parallel to and a few inches ahead of the leading edge of the slip-form.

Concrete for these large slip-forms is generally mixed in highway pavers which are self-propelled along the bank and are supplied with dry batches by trucks from a central batching plant, although on some jobs the slip-forms are supplied by transit-mix trucks. The rate of lining placement in large canals requiring 10 to 12 square yards of lining per linear foot of canal often exceeds 1,000 linear feet per day.

The track-laying machine (fig. 22), which is supported and driven by crawler-type tracks, is very similar in detail to the rail type previously described except for the method of propulsion. It is hydraulically operated and guided from a control station at the front of the machine. With experienced operators and close inspection of the work, the machine is capable of placing an acceptable lining.

(d) *Construction Methods and Equipment—Small Canals and Farm Ditches.*—Small canals and farm ditches are sometimes lined by hand screeding and finishing. Where relatively inexpensive labor is available or where water users will work cooperatively on lining operations, hand-placing methods may prove fairly economical. Certain private irrigation organizations in California have used this system for years with good success.

If a sufficient length of small canal or farm ditch can be scheduled for continuous lining, much better progress and economies can usually be realized by the use of subgrade-guided slip-forms which operate longitudinally but are supported directly on the subgrade without the use of rails (fig. 24). The subgrade-guided slip-form is believed to have been first used on the Umatilla project, Oregon, in 1915. When the smaller slip-forms are used in ditches, it is often possible for the tractor pulling the slip-form to straddle the ditch. Because this type of equipment rides on the subgrade, it is not adaptable for use with steel-reinforced lining. The weight of the slip-form tends to compact the subgrade ahead of

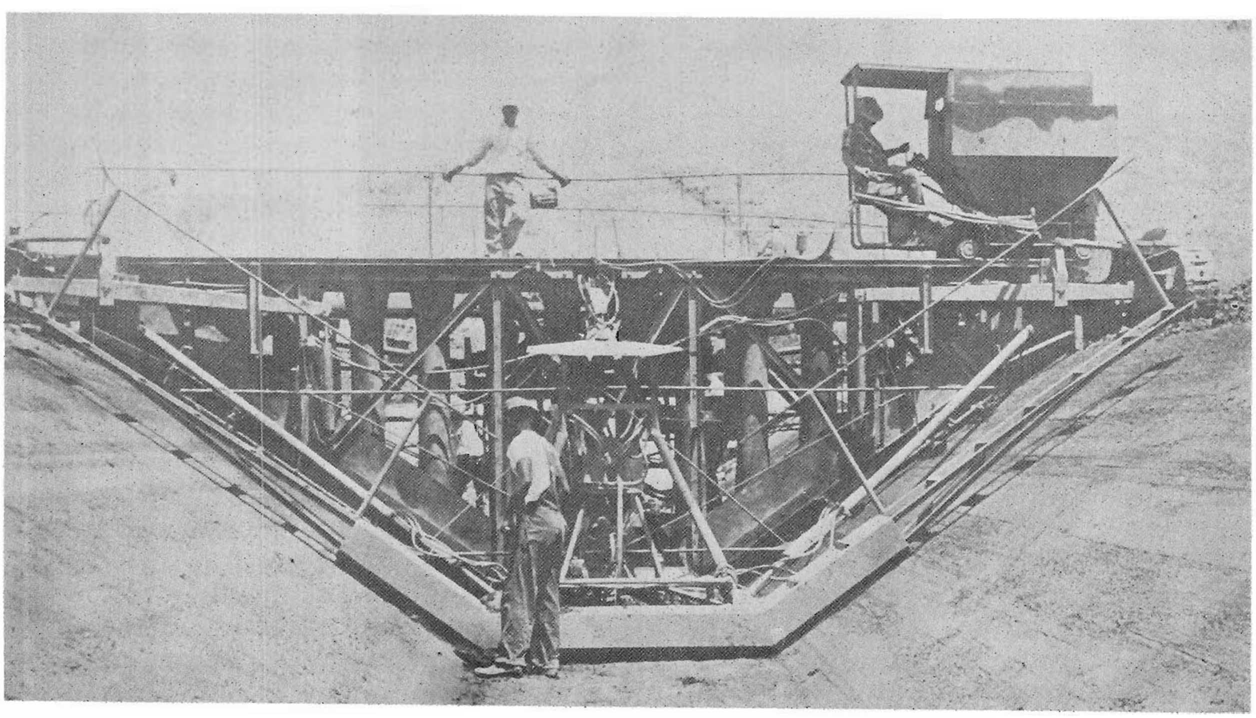

Figure 22.—Closeup view of track-laying (crawler type) slip-form used in concrete lining operations on the Putah South Canal, Solano project, California. The buggy at right of slip-form deposits concrete into the slip-form pockets by means of the metal tubes. Consolidation is by tube type vibrators mounted along the 1½ to 1 slope. The template in the foreground is the device used to control grade and alinement of the slip-form. SO-3517-R2, July 25, 1958.

the lining placement and this is believed to be beneficial. The maximum size of canal in which the subgrade-guided slip-form can be satisfactorily utilized has not been definitely established. It appears that this type of equipment becomes less practical for use in placing lining as the lined perimeter of a canal approaches 40 feet.

Several methods of mixing and supplying the concrete to these smaller slip-forms have been employed with varying degrees of efficiency. Good results have been obtained by utilizing conventional mixers, such as transit truck mixers, highway pavers equipped to dump the batch directly in the slip-form, or trucks delivering from a central mixing plant.

A continuously operated pugmill type traveling plant mixer has also been used for supplying mixed concrete to slip-forms. This type of equipment, originally developed for mixing materials for cold asphalt surfacing and base courses for highways, is equipped to pick up the mix from a windrow on the ground. Aggregate and dry cement are windrowed in proper proportions, and water is added during mixing. The use of a traveling plant mixer increased the speed of placement on the Gila project from an average rate of about 600 square yards of lining per day to better than 4,000 square yards per day, in canals requiring about 1.25 square yards of lining per linear foot. On some of the non-Bureau operations in small farm ditches in the Phoenix, Ariz., area, 1½-inch-thick concrete lining was reportedly placed by slip-forms at the rate of 1 mile per day where a little less than 1 square yard of lining was required per linear foot of ditch.

(e) *Concrete Mixes.*—Details of concrete mix design and recommended practices for both large and small canals are covered in the Bureau's Concrete Manual.[3] Concrete for lining a canal should be plastic enough for thorough consolidation and stiff enough to stay in place on the side slopes. Usually, a mix with an excess of sand is needed for machine-placed lining to give adequate workability. Close control

[3] Op. cit. p. 35.

Figure 23.—Lining a portion of the Courtland Canal, Missouri River Basin project, Kansas, by use of a winch-drawn screed operating transversely up each side slope. P271-701-1641.

of the workability and consistency of the concrete is important, because a variation of as little as 1 inch in slump can seriously interfere with the progress and quality of the work. The use of an air-entraining agent in the mix is recommended and is particulary important where exposure to freezing temperatures is anticipated.

(f) *Finishing.*—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 finished surface which would increase the carrying capacity of the canal may be warranted. However, if the water will carry 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, either of these two conditions may have a greater effect on the efficiency of the canal than the degree of original surface finish; in these circumstances a very smooth, hand-troweled surface would be of little value and the cost of securing it would be unjustifiable. Since a majority of irrigation canals carry water which contains a certain amount of sand or silt and many are subject to the growth of moss or algae, a reasonably smooth surface without voids should be adequate for a concrete lining.

(g) *Curing.*—The proper curing of portland cement concrete in canal lining is equally as

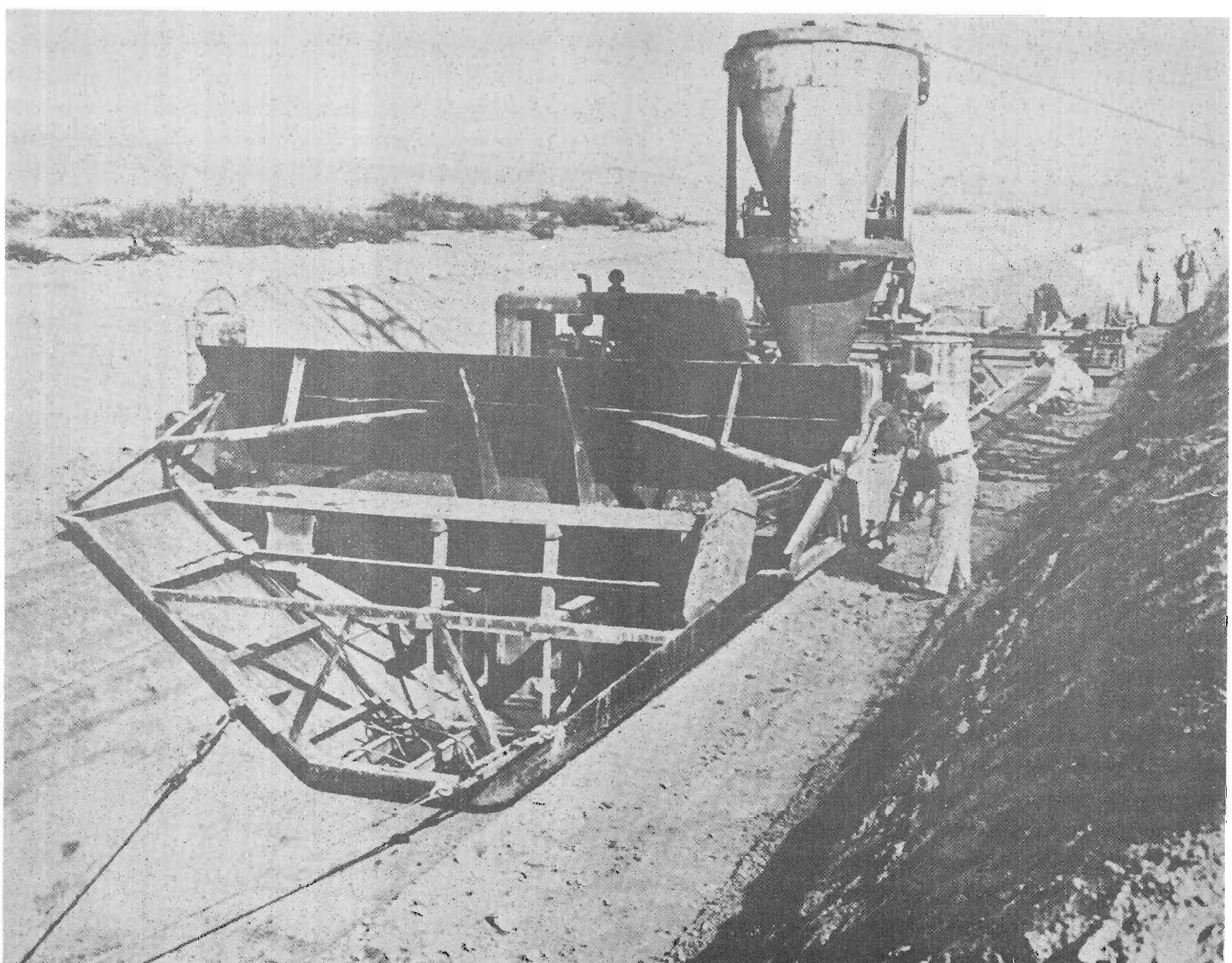

Figure 24.—A subgrade-guided slip-form concrete lining machine following an excavator. PX-D-32056.

important as the curing in any thin structural section. Moist curing is, in general, preferred, but the use of accepted sealing compounds has been found satisfactory. To assure that the lining does not dry out rapidly, the subgrade should be well moistened immediately prior to placing of the concrete.

A more detailed discussion of both finishing and curing of concrete is contained in the Bureau's Concrete Manual.[3]

(h) *Expansion, Construction, and Contraction Joints and Contraction Grooves.*—Expansion joints are not ordinarily required in a concrete lining, except where fixed structures intersect the canal. Fillers for expansion joints are discussed in section 28 (i).

Construction joints are necessary where lining operations are discontinued at the end of the day or for other reasons and are resumed after a considerable time interval. In Bureau work a construction joint[3] is a properly prepared joint where the previously placed concrete and the fresh concrete are bonded. A contraction joint is a butt joint in which the previously placed concrete is painted with sealing compound to assure that no bonding takes place.

As discussed previously, contraction joints should be filled with an elastic material. The recommended spacing for grooves is given in table 6 (sec. 20).

(i) *Joint and Crack Fillers.* — The need for better joint and crack fillers for concrete lining has become apparent from field examinations of existing installations. The Bureau undertook an investigation in 1956 on the New York Canal, Boise project, Idaho. The left side slope and two-thirds of the canal bottom were relined by project forces with portland cement concrete having formed joints. Ample area was thus provided for joint fillers. The right side slope of the canal and the remaining bottom had been relined with portland cement concrete several years previous to 1956. Cracks that had developed in this older lining provided opportunity to evaluate crack filler materials.

Several hundred feet of the random cracks in the older concrete were enlarged to ¾ inch or more in width and 1 inch in depth with a concrete router. Sealants donated by 14 manufacturers were applied to the joints and cracks in November 1956 and March 1957. These sealants were evaluated in March 1957, March 1958, and November 1959. After 3 years of exposure, it was found that four different brands of ready-mixed rubber asphalt base mastic sealers were in good condition. Based partly on the results of the tests, new specifications have been issued for rubberized, cold application, ready-mixed type sealing compounds for use in joints of concrete canal lining.[6] This material will be specified in Bureau canal work as a replacement for the cold-applied, internal-set-up mastic fillers previously used for sealing joints in concrete linings. Laboratory and field tests show that materials conforming to the new specifications are more durable, require no mixing or proportioning, and are easier to apply than the materials previously specified.

(j) *Cost.* — The initial construction cost of portland cement concrete canal lining is influenced by the size and location of the job, specification requirements, competition among bidders, and general economic conditions. Repeated attempts to gather net costs of construction operations on the job have resulted in very little useful information, mainly because of the question of rental rates applied for depreciation of capital investment in equipment. The use of well-established Associated General Contractor rental rates resulted, in some instances, in cost figures greater than the bid prices. Therefore, it appears that bid prices are as good an indication of costs as is available, even though it is realized that bidders frequently increase the price of items of construction scheduled for early completion and proportionally decrease prices for the remaining items in order to increase the early receipts in payment for work completed.

Tables 7 and 8 list representative contract prices for concrete canal linings constructed by the Bureau in recent years. Costs are expressed in dollars per square yard of lining placed, rather than per cubic yard of concrete, for convenience in comparing costs of other types of lining. It should be noted that table 7 contains data on reinforced concrete lining only and table

[3] Op. cit. p. 35.

[6] "Specifications for Sealing Compounds, Rubberized, Cold Application, Ready-Mix, for Joints in Concrete Canal Linings," Bureau of Reclamation, February 25, 1960.

TABLE 7.—*Some representative costs of reinforced concrete canal linings, based on contract prices and specifications quantities*

Specifications No.	Month and year	Feature and project	Canal section			Concrete lining		Cost of lining, dollars per square yard		
			Discharge, second-feet	Base width, feet	Water depth, feet	Quantity, cubic yards	Thickness, inches	Lining	Trimming	Total
4604	February 1956	Upper Meeker Canal, Frenchman-Cambridge division, Missouri River Basin project	284	16	5.03	225	4	11.18	1.00	12.18
4943	August 1957	Wahluke Branch Canal, Columbia Basin project	1,520	12	10.38	80	4.5	8.80	0.69	9.49
4937 sched. II	August 1957	Robles-Casitas Diversion Canal, Ventura River project	500	7	5.56	200	4	4.10	1.00	5.10
4938 sched. I and II	August 1957	Helena Valley Canal, Helena Valley unit, Missouri River Basin project	225	6	4.3	420	4	5.46	0.80	6.26
5085	August 1958	Gateway Canal Revision, Weber Basin project	700	10	7	670	4	7.42	1.38	8.80

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8 contains data on unreinforced concrete lining. Of late years, reinforced concrete linings have only been built where water velocities were high or safety was an important factor. In such cases the linings have usually been of increased thickness, contributing somewhat to the greater costs shown. Furthermore, recent reinforced lining jobs have been small as compared to the larger unreinforced lining jobs, with concomitant greater unit costs. The use of subgrade-guided slip-forms for the construction of thinner linings used in small canals and laterals is reflected in the relatively lower costs evident in recent years.

(k) *Special Laboratory Investigations.*—Reports from India[7] to the effect that relatively weak concrete linings containing impure slaked lime are giving excellent service led to a series of tests in the Bureau's Denver laboratories aimed at developing a lining with greater extensibility and less drying shrinkage; it was thought that an extensible lining might be more resistant to cracking and less affected by temperature changes. Although the results generally were negative, some details of the tests are given here as a matter of record.

[7] Nazir Ahmed, Muzaffar Ahmed, and S. L. Shah, "Problems of Canal Lining in West Pakistan," ICID—Third Congress on Irrigation and Drainage, Transactions, vol. II, R.2, Q7, p. 7.17-7.48, 1957.

Tests were conducted using hydraulic lime alone as the binding agent in concrete and in combination with portland cement. Hydraulic lime was selected as it was believed to have cementing properties similar to the impure lime used in India. Asphalt emulsions were added to regular concrete mixes and the effect of air-entrainment beyond the recommended amounts for durability was investigated.

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. Therefore, the reduced cracking which should result from a lower elastic modulus may be concluded to be 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 only about

TABLE 8.—*Some representative costs of unreinforced concrete canal linings, based on contract prices and specifications quantities*

Specifications No.	Month and year	Feature and project	Canal section			Concrete lining		Cost of lining, dollars per square yard		
			Discharge, second-feet	Base width, feet	Water depth, feet	Quantity, cubic yards	Thickness, inches	Lining	Trimming	Total
4411	June 1955	Texas Hill Canal, Gila project	125	5	5	11,200	2.0	2.03	0.60	2.63
4536	February 1956	Mohawk Laterals, Gila project	60 to 15	2	4.0 to 1.5	6,100	2.0	1.93	0.45	2.38
4555	January 1956	Putah South Canal, Solano project	956	12	10.28	16,500	3.0	1.92	0.40	2.32
4733	September 1956	Putah South Canal, Solano project								
		Section 2	735	10	8.66	27,900 (sections 2–4)	3.0	2.13	0.53	2.66
		Section 3	550	10	7.52		3.0	2.13	0.53	2.66
		Section 4	320	7	6.40		3.0	2.13	0.53	2.66
4881	May 1957	Putah South Canal, Solano project	320	7	6.4	14,600	3.0	2.05	0.48	2.53
4937 sched. II	August 1957	Robles-Casitas Div. Canal, Ventura River project	500	7	5.56	6,400	3.0	2.79	1.00	3.79
4943	August 1957	Wahluke Branch Canal, Columbia Basin project	1,520	12	10.38	9,850	3.5	3.04	0.69	3.73
4986	December 1957	Putah South Canal, Solano project	180	5	5.3	4,960	2.5	2.00	1.34	3.34
5000 sched. I	1957	Esquatzel Div. Canal Columbia Basin project	5,300	8	15.57	55,600	4.5	2.64	0.50	3.14
5084 sched. III	August 1958	Ashland Lateral, Rogue River project	48.0	4	2.5	610	2.0	2.69	1.10	3.78
5086	September 1958	Wahluke Canal, Columbia Basin project	1,190.0	12	9.73	3,610	3.5	3.23	0.50	3.73
5167	April 1959	Wahluke Canal Lateral, Columbia Basin project	42.9	3	2.3	440	2.0	2.08	0.50	2.58
5245	November 1959	H. Lateral, Mercedes division[1]	Avg. 38.0	5	Avg. 3.60	4,550	2.5	2.64	0.25	2.89
			18.5	3	2.28					
5254	December 1959	Block 88 Laterals Columbia Basin project	141.0	5	0.9	530	2.5	2.29	0.50	2.79
			28.36	3	1.584	645	2.0	1.83	0.50	2.33
			36.0	3	2.4	390	2.0	1.83	0.50	2.33
			10.96	2	1.05	835	2.0	1.83	0.50	2.33
5256	December 1959	Wellton-Mohawk Canal, Gila project	276.7	8	4.98	24,000	2.5	1.70	0.60	2.30
5264	February 1960	Wellton-Mohawk Canal, Gila project	240.0	5	5.00	4,220	2.5	1.87	0.40	2.27
			200.0	5	4.70	9,000	2.5	1.87	0.40	2.27
			150.0	5	3.90	4,380	2.5	1.87	0.40	2.27
			110.0	5	3.00	1,400	2.5	1.87	0.40	2.27

TABLE 8.—*Some representative costs of unreinforced concrete canal linings, based on contract prices and specifications quantities*—Continued

Specifications No.	Month and year	Feature and project	Canal section			Concrete lining		Cost of lining, dollars per square yard		
			Discharge, second-feet	Base width, feet	Water depth, feet	Quantity, cubic yards	Thickness, inches	Lining	Trimming	Total
5279	March 1960	West Canal, Columbia Basin project	Normal							
			383.0	10	6.3	6,540	3.0	2.34	0.43	2.77
			251.0	8	6.05	3,630	2.5	1.95	0.43	2.38
			135.0	6	4.1	1,720	2.5	1.95	0.43	2.38
			63.0	4	2.7	4,380	2.0	1.56	0.43	1.99
			35.3	3	2.18	1,640	2.0	1.56	0.43	1.99
			10.0	2	1.0	190	2.0	1.56	0.43	1.99
5284	March 1960	C & C-3 Lateral (schedule I), Mercedes division[1]	31.0	3	2.9	1,275	2.5	2.23	0.32	2.55
		G & G-2 Lateral (schedule II), Mercedes division[1]	59.0	5	3.50	2,280	2.5	2.26	0.32	2.58
			27.2	3	2.60	2,160	2.5	2.26	0.32	2.58
5305	May 1960	Wellton-Mohawk Canal, Snyder Ranch Lateral extension, Gila project	57.5	2	3.20	3,050	2.0	1.33	0.37	1.70
			35.0	2	2.63	3,790	2.0	1.33	0.37	1.70
			17.0	2	1.90	1,120	2.0	1.33	0.37	1.70
			7.7	2	1.23	2,640	2.0	1.33	0.37	1.70
5337	June 1960	K Lateral, Mercedes division[1]	71	5	3.68	600	2.5	2.29	0.41	2.70
			30	3	2.45	2,900	2.5	2.29	0.41	2.70
5387	September 1960	F Lateral, Mercedes division[1]	123	5	4.05	7,850	2.5	2.02	0.33	2.35
			27	3	2.64	4,300	2.5	2.02	0.33	2.35
5426	November 1960	I Lateral, Mercedes division[1]	122	5	4.8	6,100	2.5	2.06	0.32	2.38
			33	3	3.0	2,750	2.5	2.06	0.32	2.38
5548	April 1961	La Feria Lateral, L.Rio Grande Rehab. proj.	57	5	3.30	700	2.5	3.44	0.55	3.99
			28.3	3	3.00	1,190	2.5	3.44	0.55	3.99
5566	May 1961	Main Canal, San Angelo project	131	5	4.64	16,420	2.5	1.65	0.35	2.00
5626	August 1961	Distribution system, San Angelo project	12.7	2	1.76	11,630	2.0	1.49	0.35	1.84
			65.0	4	3.50	2,770	2.0	1.49	0.35	1.84
5602	June 1961	B & D Lateral, Mercedes division[1]	60.0	5	4.45	2,840	2.5	2.19	0.40	2.59
			37.0	3	3.13	1,430	2.5	2.19	0.40	2.59

[1] Mercedes division is in the Lower Rio Grande Rehabilitation project.

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400 pounds per square inch. Test specimens of hydraulic-lime concrete disintegrated completely after fewer 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, which had 100 percent portland cement as the binding agent.

Dry-tamped or zero-slump concrete was also investigated because of the possibility that the lower water content would result in fewer cracks from drying shrinkage and that some economy might be realized from the lower cement content. Very little information was available on zero-slump concrete, and therefore problems of specimen fabrication and mix design were new. Preliminary tests led to the adoption of a 6- by 12-inch cylinder fabricated with an electric tamper. Zero-slump concrete of 0.44 water-cement ratio exhibited considerably less drying shrinkage, greater durability, and about the same strength and permeability as 3-inch-slump concrete of the same water-cement ratio. Furthermore, the dry-tamped concrete contained 1 to 1½ sacks of cement less than the 3-inch-slump concrete with which it was compared. However, no mechanized equipment for placing dry-tamped concrete is available, and until labor saving methods are developed, the high cost of hand placement would far overshadow any benefits from these better characteristics or the saving in cement.

29. *Portland Cement Concrete Linings—Repair.*—In the past, most deteriorated or damaged portland cement concrete linings were repaired or replaced with portland cement concrete or mortar. To develop a less expensive means of repair, the several alternatives described below have been tried. The condition of the existing lining and the condition of the subgrade will influence the kind of repair that should be made. Also, because water delivery is necessary for the growing crops during the summer months, resurfacing and repairs must usually be made during the nonirrigation season accompanied in many instances by unfavorable weather conditions; so the type of repair selected should lend itself to placement in cold weather.

(a) *Asphaltic Concrete.*—One of the early applications of asphaltic concrete for the purpose of rehabilitating portland cement concrete is an installation made on the Boise project, Idaho, in 1944. This installation on the Main Canal, which has a capacity of 3,000 second-feet and a velocity of 6 to 7 feet per second, consisted of the placement of approximately 7,200 square yards of a 1½- to 2-inch-thick asphaltic concrete mat over a badly eroded concrete lining which had been in service since 1909 and 1910. A high sand load carried by the water in the canal contributed to the scour and erosion, particularly of the bottom lining. Inspection in 1958 (fig. 25) showed that the asphalt "half-sole" was still in good condition after having been in service for 14 years.

In this rehabilitation work the asphaltic concrete mat was laid over the old concrete in one layer. Before the mat was placed, the surface of the old lining was cleaned with a rotary brush, primed with kerosene, and given a tack coat of hot 50-60 penetration asphalt. The work was accomplished by contract.

The resurfacing material, consisting of 10 percent asphalt of 50-60 penetration and aggregate, was hot mixed in the contractor's plant at Boise and transported by truck to the site of placement. The temperature of the mix as delivered at the site of the work was about 300° F. Most of the lining was later sealed with an application of hot asphalt of the same grade used in the mix. The lining was rolled with a 12-ton standard roller. No particular difficulty was experienced, other than that it was usually necessary to wait for some cooling to occur before rolling.

In 1956, a rehabilitation program on the above Main Canal, Boise project, included a large quantity of asphaltic concrete resurfacing. The new specifications included some modifications as compared with those used for the 1944 work. A prime coat of SS-1 asphalt emulsion applied at a rate of 0.05 to 0.1 gallon per square yard, followed by a tack coat of hot 50-60 penetration asphalt cement applied at a rate of 0.15 to 0.25 gallon per square yard, was found to be superior to the former surface preparation method. A

Figure 25.—Bottom of the Main Canal, Boise project, Idaho, 14 years after resurfacing with hot-plant-mixed asphaltic concrete. Examination disclosed no cracks or signs of failure in the entire length. The resurfacing was placed in 1944 to repair a deteriorated portland cement concrete lining. P3-D-15373.

minimum compacted thickness of 2 inches of asphaltic concrete was specified for use on both the side slopes and bottoms. A large butane heater was successfully used to dry occasional damp surfaces. This lining is also in good condition (1962) after 6 years of service. More than 100,000 square yards of the hot-mixed asphaltic concrete now have been placed as a repair for portland cement concrete linings. The work was done primarily by project forces, and the resurfacing layer ranged from 1 to 4 inches in thickness.

On another project, some distress and removal of the resurfacing has occurred (fig. 26) due to a poor bond caused by the cold, wet weather during installation or because of hydrostatic uplift pressures. The latter type of failure has been partially overcome by the installation of flap-valve drains similar to the one shown in figure 5 (sec. 20). It was reported that before water was placed in the canal, much of the asphaltic concrete overlay in the bottom had bulged. Flap-valve drains were placed before the canal was put in operation and the lining settled into place after one complete irrigation season.

Figure 26.—Asphaltic concrete resurfacing that has failed at the lower end of the lined reach due to poor bond to underlying concrete and uplift pressure. The condition of the deteriorated portland cement concrete is shown in the immediate foreground. P-33-D-15360.

The aggregate gradation presently specified for asphaltic resurfacing material is as follows:

Sieve size	*Percent passing*
½ in.	100
No. 4	95 to 80
No. 10	80 to 60
No. 40	35 to 14
No. 200	6 to 14

From 8 to 10 percent of a 50-60 penetration asphalt cement, based on the dry weight of the aggregate, is used to obtain a rich workable mix.

(b) *Prefabricated Asphaltic Sheets.*—Damaged concrete linings have been repaired with exposed type prefabricated asphaltic sheets (fig. 27), but the results were not satisfactory with the adhesive method used. Preparation of the sections consisted of removing the moss, sand, gravel, etc., and carefully sweeping the area to remove loose material from the surface of the concrete. A prime coat of SS-1 asphalt emulsion was then applied to the cleaned surface at a rate of approximately 0.15 gallon per square yard. The emulsion was broomed uniformly over the surface.

Prefabricated asphaltic sheets one-half, one-quarter, and one-eighth-inch thick were installed by lapping each joint 3 inches over the previously placed downstream sheet. Lap cement or adhesives were placed at random spots about 3 feet apart to hold the sheets in place. A band of adhesive was also applied to the surface of the concrete lining about 1 foot up from the toe of the side slope to hold the upturned ends of the prefabricated lining in place. Adhesive was applied to both surfaces of the prefabricated material when making a joint. In order to provide a satisfactory bond, the surfaces of the sheets were swept clean of talc, mica, or other material used to prevent the sheets from sticking during shipment. In some reaches, concrete nails were also used to secure the sheets to the side slopes.

Transverse cutoffs were provided at intervals along the lined reach by breaking out the con-

Figure 27.—Installing ½-inch-thick exposed type prefabricated asphaltic material as a surface repair for deteriorated portland cement concrete lining. CH-467-11.

crete lining in a slot 6 inches wide and 6 inches deep extending across the bottom and up each side slope approximately 1 foot. The ends of the asphalt sheets were pressed downward into the cutoff trench after being warmed with a weed burner to prevent cracking, and the trench was then filled with a quick-setting portland cement concrete mixture.

Examination of the above repairs in 1956, 1 year after installation, indicated that the experimental lining had been displaced from two of the three sections, and by the fall of 1958 all the prefabricated lining material had been lost. Lack of adhesion of the sheets to the underlying concrete, inadequate fastening with concrete nails, and hydrostatic uplift pressures all contributed to the displacement and removal of these materials from the surface of the deteriorated concrete.

(c) *Other Asphaltic Materials.*—Catalytically blown asphalt cement has been used to resurface badly cracked concrete linings in the Central Valley project, California. The heated asphalt quid was applied to the damaged lining by spray. This installation, placed in 1954, is being observed.

Several asphaltic materials also were used experimentally in 1947 in an attempt to repair and resurface a 4-inch-thick portland cement concrete lining placed originally in 1910 and 1911 in a small canal of the Boise project, Idaho. Used in the experimental work were pneumatically applied asphalt-sand mixtures; mineral-filled asphalts, applied by hand methods and pneumatically; coal tar; and two types of hot-mixed asphaltic concretes, placed to a thickness of one-half inch. All of the resurfacing materials became badly eroded by 1955, and, except for the two asphaltic concretes, were replaced by hand-applied portland cement mortar in 1956. The asphaltic concretes were found to be in poor condition when examined in 1958, with the one composed of asphaltic cement and aggregate being in somewhat better condition than the one composed of asphalt-emulsion and aggregate.

(d) *Unreinforced Portland Cement Concrete.*—Unreinforced portland cement concrete has heen placed as a surface repair over eroded and teriorated linings of the same type. This method, as well as the asphaltic concrete application described previously, has been used widely on the Boise project in the resurfacing of the older portland cement concrete linings. Since 1956, about 100,000 square yards of the older concrete linings have been rehabilitated in this manner, the resurfacing layer being 2 to 4 inches in thickness. Most of these repairs are in excellent condition.

(e) *Shotcrete.*—On several projects shotcrete has been used to resurface deteriorated concrete linings. The resurfacing generally has been successful under normal operating and exposure conditions. Preparation of the existing concrete surface, care in application of the shotcrete by experienced and conscientious workmen, and proper curing of the shotcrete are controlling factors in the success of the repair. Generally, thin layers of resurfacing have not withstood severe service such as alternate freezing and thawing and high external hydrostatic head where it exists outside the lined canal.

About 500 square yards of the 37-year-old concrete lining on the Boise project, described previously, were repaired with a thin (one-half inch) cover of shotcrete in March 1947. This lining was exposed to extreme conditions of weather, external hydrostatic head, and other hazards. The installation was part of an experimental program to determine which of some 20 variations in materials and methods was most satisfactory for the rehabilitation of old concrete linings. Spalling and cracking were apparent in 1955, after 8 years of service, and when examined again in 1958, the resurfacing had continued to deteriorate. The voids had become more apparent between the old portland cement concrete and the resurfacing material, additional spalling had occurred on the side slopes, and erosion had continued in the bottom.

The condition of the existing lining and the necessity of repair during the off season, during which the weather is sometimes severe in northern climates, has a bearing on the cost of repair work.

30. *Portland Cement Mortar Linings.*—(a) *Service History.*—Some linings of thin portland cement mortar in mild climates are known to have been in service for 60 years or more, and more than 20 miles of ¾-inch-thick unreinforced mortar linings placed from 1880 to 1890 in the Fruitvale and Gage Canals of southern California are still in service. The records indicate that a total of 37 miles of mortar lining was placed in canals of the Okanogan project, Washington, between the years 1912 and 1917. Old reports refer to this as "plaster lining," probably because it is only 1½ inches thick and was applied to the canal perimeter by hand troweling. The lining was unreinforced, and after more than 40 years most of the 37 miles of canal and lateral lining is in good, serviceable condition. Although most of it is cracked and in need of some maintenance, only about one-half mile has failed completely and should be replaced. The cause of such failure has been established as frost heaving, resulting from areas of tight subgrade soil which hold accumulated moisture from irrigation.

(b) *Pneumatically Applied Mortars.*—Most of the portland cement mortar linings placed by the Bureau have been shotcrete linings. Shotcrete is a term adopted to designate pneumatically applied portland cement mortar (mortar which is shot into place by pneumatic pressure).[8] Pneumatic application is accomplished by the use of special equipment available from several manufacturers.[9]

Shotcrete is widely used both for lining and, as discussed previously, for the resurfacing of older linings on irrigation canals and ditches. Because of the small amount of construction equipment required and its mobility, this process is well suited to construction or repair work on small or widely scattered canal lining jobs, and also on laterals and farm ditches with their frequent sharp curves, turnouts and other structures. This type of construction permits placement of lining immediately adjacent to existing structures, in contrast to slip-form-placed concrete which, due to equipment limitations, leaves a space of several feet on the sides of the structure which must be lined later by hand methods. Another advantage of shotcrete construction is that the lining can be placed on the surface of rock cuts.

[8] Chadwick, W. L. (Chairman), McCrory, J. A., and Young, R. B., "Proposed Recommended Practice for the Application of Mortar by Pneumatic Pressure," Committee 805, Proceedings ACI, vol. 47, p. 185, 1951.

[9] The several manufacturers have adopted trade names such as "Gunite," "Bondact," and "Jetcrete" for mortar placed by their equipment.

Most of the advantages of shotcrete lining are limited to its use for some special condition, as mentioned. This type of lining, in general, is not as economical as slip-form-placed concrete for large jobs. Not only is the rate of placement for shotcrete lining very slow in comparison to slip-form operations, but also shotcrete 1½ inches thick usually costs as much as 2 inches of concrete if conditions are favorable to slip-form placement (table 2, sec. 2). Since shotcrete is a mortar containing sand only (no coarse aggregate), considerably more cement is used than in concrete, which makes the shotcrete more susceptible to cracking because of the higher water requirement. The lack of entrained air in shotcrete may further reduce its serviceability in comparison with air-entrained concrete in areas where freezing and thawing occur.

Many miles of shotcrete lining have given satisfactory service for more than 20 years in canals in the relatively mild climates of the Salt River and Middle Gila River Valleys in Arizona, the Lower Rio Grande Valley in Texas, and in southern California. A little over 1 mile of shotcrete lining was placed in canals of the Icicle Irrigation District, Washington, in 1924. This lining was reported to be in good condition in 1945, with years of serviceability remaining. The lining was reinforced with a wire mesh (size unknown) and was 1½ inches thick. For many years extensive use has been made of shotcrete in the lining and repairing of power canals in the Sierra Nevada Mountains, where severe winter weather conditions are encountered.

(c) *Thickness and Reinforcement.*—Shotcrete linings placed by the Bureau are usually 1½ inches or more in thickness. They have sometimes been reinforced with wire mesh, particularly where structural safety is involved. The reinforcement may be 4- by 4-inch or 6- by 6-inch wire mesh of No. 9 or 10 gage. Care should be taken to position the mesh in the center of the lining, otherwise it serves no useful purpose and may even be objectionable. Shotcrete of 1-inch nominal thickness has been used extensively in canals of the Salt River Valley in Arizona, with good success. This thin lining, which is reinforced with a light wire mesh, may crack, heave, and break, but complete failures are few and small in area. Prompt repairs have resulted in a satisfactory low-cost lining, some sections of which have been in use for 30 years and appear good for many more.

Shotcrete lining of similar thickness (1 inch) on the Gila project, Arizona, has not been as successful. There the lining was under the initial handicap of an unstable, sandy soil subgrade as contrasted with a very stable soil in the Salt River Valley. Another factor favorable to the success of a thin lining in the Salt River Valley was the full-time use of the canal in which water was kept most of the time, thereby reducing temperature differentials in the lining. On the Gila project, on the other hand, the lining was subjected to additional stresses from extreme temperature changes and from wetting and drying due to intermittent use of the system.

In 1947, about 3.5 miles of experimental, unreinforced shotcrete lining, 1½ and 2 inches thick, were placed on the Columbia Basin project. In the repair of the lining on the side slopes of one lateral exposed to the sun, which had developed serious cracking and buckling, it was found that some of the original lining was only about one-half inch thick. This illustrates the difficulty of controlling thickness of shotcrete lining during application. Although some of this lining was less than the specified thickness, after 12 years of service the lining appears generally in good condition, and operation and maintenance forces report that little maintenance has been required after the initial deficiencies were corrected.

The experiences cited prove that lining thickness, as well as design details, cannot be arbitrarily set but must be established from engineering considerations on each job.

Shotcrete has been used on several projects as experimental coverings for the protection of asphaltic membrane linings. In these applications the shotcrete is usually one-half or three-fourths inch thick. These are discussed in the section on membrane linings.

(d) *Construction Equipment and Methods.*—The equipment for placing shotcrete varies with different manufacturers. A system of air locks is usually incorporated into the mechanism for feeding the premixed relatively dry sand and cement into a large flexible hose through which it is 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. A minimum air pressure of 45 pounds per square inch is required for hose lengths of less than 100 feet, and it should be increased by 5 pounds per square inch for each additional 50 feet of length exceeding 100 feet. Shotcrete is usually applied to the canal section by holding the nozzle about 3 feet from, and normal to, the surface being covered. See figure 28 which depicts a typical shotcrete installation.

Requisites to the proper application and satisfactory installation of shotcrete are the correct rate of application and proper adjustment of the mix, which may be assured by use of skilled operating personnel. In many areas contractors with shotcrete equipment specializing in this type of work develop very economical procedures. The construction procedures and equipment to be used depend on the amount of lining to be placed and the size of the 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.

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. Latest specifications of the Bureau require that this rebound be removed. The value or importance of troweling the shotcrete linings is influenced by two factors: the method of curing proposed, and the required hydraulic properties of the canal. Experience has proved that the coverage with a sprayed curing membrane is considerably increased on a troweled surface as compared to the rough natural shotcrete finish. The more efficient and economical use of a sealing compound is estimated to offset the cost of troweling to a considerable extent. The theoretical hydraulic advantage of a troweled surface is contingent on the size and location of the canal. For instance, a small lined farm ditch which will probably have a sand or silt deposit over a major portion of the wetted perimeter would not justify the expense of a troweled finish for increased hydraulic proper-

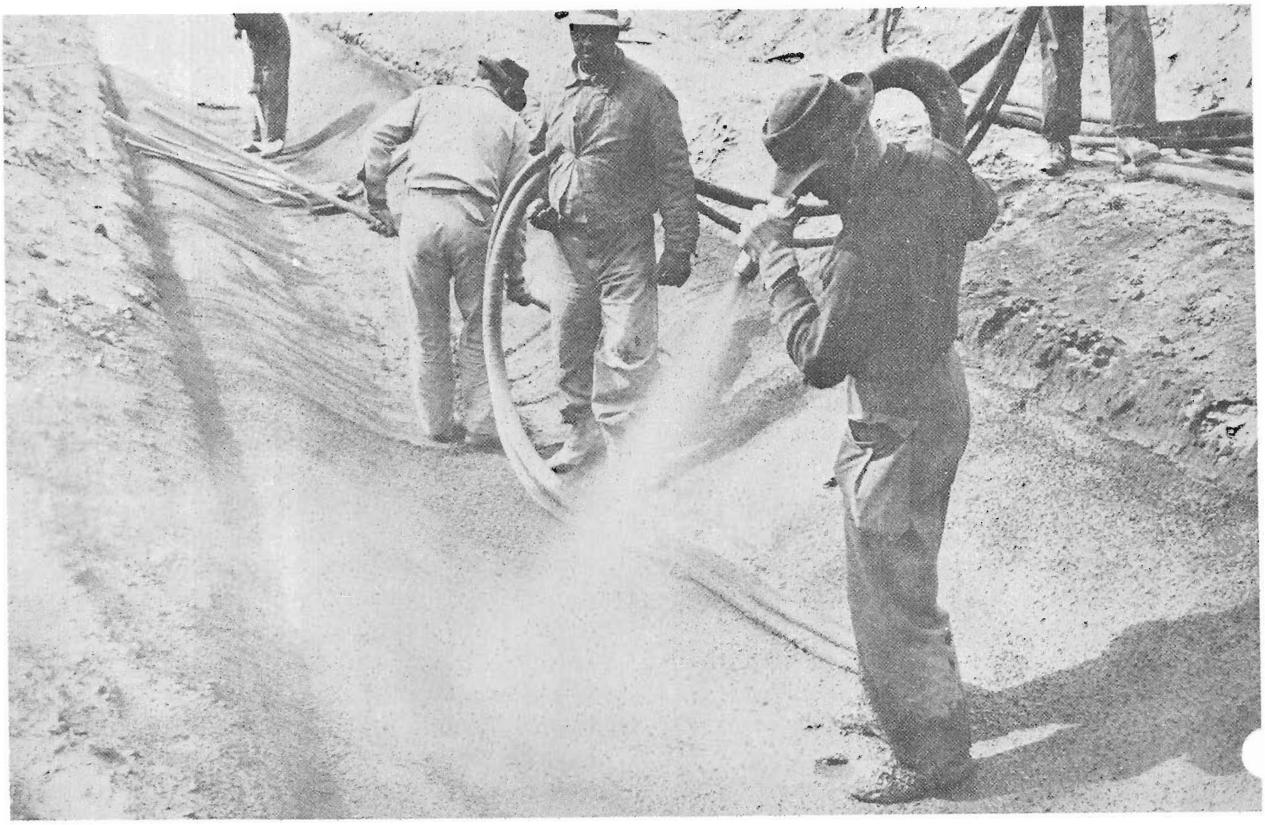

Figure 28.—Placing shotcrete lining in a small canal. C-7330-3.

ties. In a large canal, however, the improvement in hydraulic properties alone might warrant troweling. Insofar as can be determined, troweling does not improve the quality or strength of the shotcrete lining.

(e) *Materials and Mixes.*—Sand for shotcrete should conform to the grading requirement of concrete sand. The term "sand" is used to designate aggregate in which the maximum size of particle is $\frac{3}{16}$ of an inch. Hard particles are desirable because soft grains crumble as they pass through the discharge hose. The resulting increase in fine material requires more water to maintain plasticity and thus results in lower strength and greater shrinkage on drying. Sand should contain 3 to 5 percent moisture for efficient operation of the equipment. Sand which is too dry generates static electrical charges, increases the rebound, and creates difficulty in maintaining uniform movement of the mix through the hose. Conversely, sand which is too wet causes frequent plugging of the equipment. No coarse aggregate is used in shotcrete.

The optimum mix, in place, contains a little less water than that which will cause sloughing and just enough cement for the desired water-cement ratio. Initial proportions of cement to sand usually approximate 1:4.5 by weight. The rebound has a greater percentage of coarse sand particles and a much smaller cement content than the mortar leaving the nozzle. Therefore, the cement content of the materials as mixed should be less than that desired for the mortar in place.

Purposeful air entrainment for increased durability and workability has not been applied to shotcrete. Some limited experimentation in the Bureau's Denver laboratories indicates that air entrainment in shotcrete by the addition of special agents is possible, but it would not be efficient because of lack of mixing at the noz- and it would be very difficult to measure an control. A more detailed discussion of shotcrete, its method of construction, curing, finishing, and the most suitable materials and mixes is contained in the Bureau's Concrete Manual.[3]

(f) *Subgrade Preparation.*—A stable subgrade is important and its preparation for shotcrete lining will vary with subgrade characteristics. Fine trimming of canal sections through stable rock cuts is unnecessary if the hydraulic characteristics of the rough surface lining are satisfactory. With earth subgrades, best results are generally obtained if the subgrade is prepared in the same manner as for cast-in-place concrete lining. If the subgrade is not trimmed to a reasonably smooth alinement, control of the lining thickness is exceedingly difficult and usually results in thin areas forming planes of weakness over high spots. All absorptive surfaces against which shotcrete is to be placed should be thoroughly moistened so that moisture will not be drawn from the freshly placed mortar. At the time of application, however, there should be no free water on the surface of the subgrade.

(g) *Expansion and Contraction Joints.*—Most of the shotcrete linings installed by private irrigation districts have not been provided with expansion joints or contraction grooves. The argument favoring elimination of the joints is that the cracks which do occur can be filled and necessary repairs made as a maintenance operation at less expense than the cost of preformed joints. However, buckling caused by thermal expansion has occurred in some linings placed during cool weather, where expansion joints were not provided. Current Bureau specifications require not only expansion joints adjacent to structures, but also transverse grooves for contraction cracking, as in unreinforced portland cement concrete lining (see table 6, sec. 20). More detailed discussion of contraction joints is contained in the Concrete Manual.[3]

(h) *Cost.*—The cost of shotcrete canal lining is dependent on the thickness, optional use of reinforcement, provision for joints, size of the job, availability of materials, and competitive interest. Table 9 shows some bid prices for shotcrete lining placed by contract on Bureau work. From this list, the average cost, including trimming of the subgrade, for 1½-inch unreinforced shotcrete lining placed since 1945 is just over $2 per square yard, with reinforcement adding about $0.13 per square yard. Very little shotcrete lining has been placed by the Bureau under contract in the last several years; consequently, representative costs are not obtainable. From information on costs supplied by the Salt River Valley Water Users' Association, Phoenix, Ariz., which uses linings of this type in rehabilitation work, the cost of shotcrete lining has increased considerably the last few years, reflecting the general rise in labor and equipment costs.

[3] Op. cit. p. 35.

31. *Precast Portland Cement Concrete Linings.*— The use of precast concrete blocks or slabs for lining small canals and laterals may have some limited application and under certain conditions may be relatively economical. In special cases where an adequate supply of less costly labor is available or where standby personnel can be utilized to advantage, it may be feasible to manufacture slabs by hand methods. Where such is not the case, it will usually be more economical to use smaller blocks manufactured by conventional mass production type building block machinery.

Linings of this type appear most promising for use by small maintenance crews in lining or repairing short sections of canal, or by individual farmers for lining their own ditches. No particular skill and very little equipment are required. The smaller building blocks can be used even on curves but large slabs are limited to use on tangents. Joints in both types should be sealed with either portland cement mortar or asphaltic materials if seepage control is an important consideration. The large amount of hand labor involved in placing the blocks and sealing the joints makes this type of lining slow to install and hence, in most areas of the United States, too high in cost for extensive use.

(a) *Field Experience.* — In 1940 and 1941, Civilian Conservation Corps labor was used to advantage in manufacturing concrete slabs by hand methods for use in lining canals on the Yuma and Carlsbad projects. On the Yuma project, about 6 miles of various laterals were lined on the side slopes with precast slabs and in the bottom with a 4-inch cast-in-place portland cement concrete base. The precast slabs were 4 feet by 6 feet by 1½ inches thick, rein-

TABLE 9.—*Costs of pneumatically applied mortar (shotcrete) canal linings, based on contract prices and specifications quantities*

Specifications No.	Year	Feature and project	Discharge, second-feet	Canal lining		Cost of lining, dollars per square yard			
				Quantity, square yards	Thickness, inches	Mortar	Reinforcement	Trim subgrade	Total
1104	1945	Canal and laterals, Gila project	14 to 70	889	$1\frac{1}{2}$	1.772	0.090	0.45	2.31
1230	1946	Pasco Pump laterals, CBP	15	500	2	2.149	—	0.45	2.60
1230	1946	Pasco Pump laterals, CBP	5 to 15	28,600	$1\frac{1}{2}$	1.826	—	0.45	2.28
1402	1946	Canal and laterals, Yuma Mesa division, Gila project	15 to 60	34,700	$1\frac{1}{2}$	1.503	0.126	0.30	1.93
1402	1946	Canal and laterals, Yuma Mesa division, Gila project	15 to 60	34,000	$1\frac{1}{2}$	1.503	—	0.30	1.80
1546	1946	Laterals from "A" and "B" Canals, Gila project	15 to 60	56,750	$1\frac{1}{2}$	1.667	0.135	0.43	2.23
1546	1946	Laterals from "A" and "B" Canals, Gila project	15 to 60	56,750	$1\frac{1}{2}$	1.667	—	0.43	2.10
3035	1950	Canals, Ft. Sumner project	20	60,200	$1\frac{1}{2}$	1.37	—	0.42	1.79
3174	1950	Canals, Ft. Sumner project	80 and 100	57,400	2	1.66	—	0.33	1.99

CBP = Columbia Basin Project

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forced with a ⅜-inch rod in each of two ribs on the underside. They were cast at the project yards and hauled to the various laterals as needed. The cast-in-place base was placed first, with a shoulder on each edge to support the precast slabs. The slabs were placed on these shoulders and supported at the proper grade and elevation by temporary struts while backfill was tamped behind them. Water was then ponded in the lateral, and any slabs that were displaced by differential settlement of the backfill were jacked into position and additional material tamped behind them.

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 24 inches square and the other 8 inches wide by 24 inches long. The square slabs used in the bottom of the canal were 2½ inches thick, with three plain butt edges and one shoulder edge. These were laid two slabs wide, butt to butt, so that the shoulder edges formed a continuous shoulder along the toe of each bank for supporting the slabs on the bank. The square slabs used in the sides or banks were only 2 inches thick, with three overlapping edges and one bevel edge. These side slabs were laid two high so that adjacent slabs lapped, the bevel edge of the lower slab fitted into the bottom slab shoulder, and the bevel edge of the upper slab formed 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 four edges (type A, fig. 29), simil[illegible] to the common concrete silo stave. These wer[illegible] laid in a round-bottom section and sealed with asphalt mastic. The slabs were used to line tangent sections only. Curves were lined later with shotcrete.

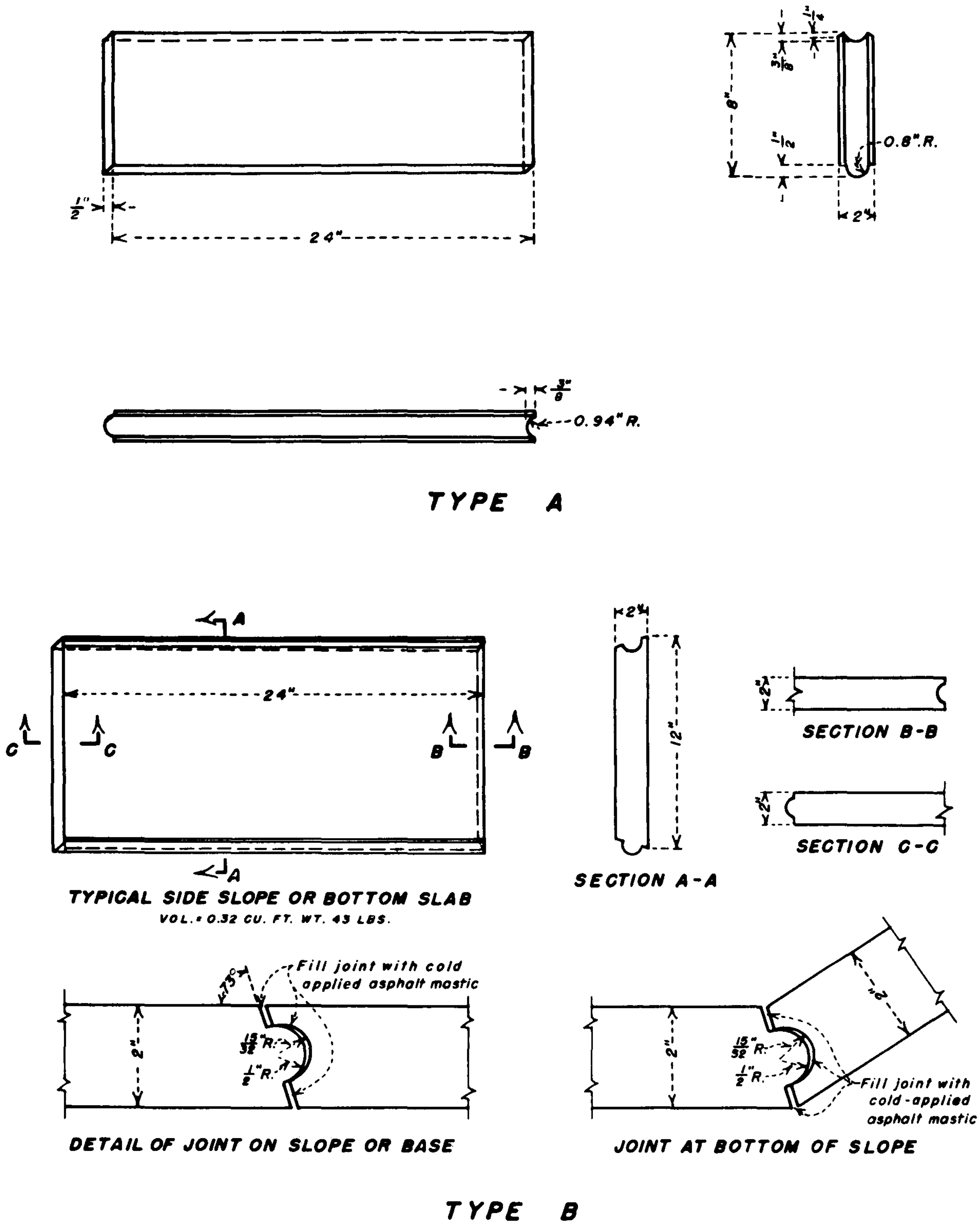


Figure 29.—Suggested designs for precast concrete slabs. 288-D-2638.

Mortar blocks 7¾ by 15¾ inches by 2¾ inches thick, manufactured in conventional building block machinery, were used to line approximately 1,400 square yards of the "D" Line Canal, Boise project, in 1947. The blocks were delivered to the jobsite for 17 cents each, and the total cost of the finished lining including the cost of subgrade preparation was $3.05 per square yard. The bottom of the canal was rounded, with sides sloping at approximately 1½ to 1. The blocks were laid dry with a ¼- to ½-inch space on all sides for receiving a sand-cement grout which was poured into the joints from a bucket, then worked into the joint and smoothed off with a large trowel. This lining was installed to control seepage; but since the blocks were manufactured of a stiff, coarse mortar, they were somewhat porous and did not immediately stop the seepage. As was anticipated, however, the lining was sealed by water-borne silt deposited over the surface during the first irrigation season. Evidence that the lining has stopped the seepage is the drying of the land below the canal which previously had been waterlogged.

32. *Exposed Plastics and Synthetic Rubber.*—Plastics of several types are being used as linings for small reservoirs, and plastics and synthetic rubber have been used experimentally as exposed linings for canals and other waterways. The first linings of this type placed by the Bureau and its cooperators in the lower cost canal lining program were placed in much the same manner as are the prefabricated asphaltic linings; that is, the lining material was furnished in sheets or rolls up to about 5 feet in width, and the adjacent strips were overlapped about 3 inches and cemented together. Some of these materials were placed as exposed linings and others as buried membranes with an earth or gravel cover to hold the material in place and protect it from weathering, erosion, and livestock damage (see sec. 41).

As a canal lining material, good service has been obtained from these plastic materials if placed as a buried membrane, but plastics placed as exposed membranes at a cost of about $2.00 per square yard were in very poor condition after about 2 to 4 years. Figure 30 shows exposed membranes composed of 0.0175- and 0.020-inch-thick polyethelene that failed after 3 years due to exposure to the sun, weather, and erosion. The fringe along the top of the side slopes in the immediate foreground of the photograph is the only remaining evidence of the lining. Accordingly, until a material is developed that will withstand the conditions of exposure, the use of plastics for a canal lining must be confined to those of the buried type unless periodic replacement can be justified; or use might be made of the material by a farmer in the temporary lining of a head ditch.

Butyl-coated fabrics, particularly coated fiberglass, have performed much better than plastics as exposed linings. An installation of the coated fiberglass made on a small lateral in Utah is still in a serviceable condition after 12 years. Other studies are underway utilizing butyl rubber sheeting as an exposed lining.[5] The present high cost of the synthetic rubber and rubber-coated materials is a limiting factor in their more general use.

33. *Brick Linings.*—The first extensive use of clay brick for canal lining purposes in the United States for which information is available in the Bureau was on a private irrigation district in Texas in 1933. The bricks used were ordinary clay bricks salvaged from wrecked buildings. The canal section was semicircular. The bricks were placed on the lower segment of the subgrade with a sufficient interval between to allow for mortar. The mortar was dumped on the bricks and broomed into the openings. On the side slopes the bricks were laid in courses with troweled mortar joints. 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 was used rather extensively for a period in the Lower Rio Grande Valley of Texas. Intended primarily for canal lining use, this brick was 1½ by 5½ by 11½ inches in size with longitudinal cylindrical holes which decreased the weight and permitted the mortar between the ends of the bricks to enter the holes and serve as dowel pins. Grooves in the longitudinal edges were provided for centering reinforcement mesh which properly spaced the bricks for

[5] Op. cit. p. 47.

Figure 30.—A plastic material which has failed in three reaches where placed as an exposed membrane, due to weathering and erosion. The only remnant of the exposed plastic is that along the top of the canal side slopes. Lining in the background is an exposed type prefabricated asphaltic material. P11-D-14991.

brooming a thin cement mortar into the joints. Additional mortar was brushed over the surface to a thickness of approximately one-fourth inch. It has been reported that there is no evidence of deterioration, especially where reinforcement was used. 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. The managers of the irrigation districts in the Lower Rio Grande Valley who have used brick linings believe this type of lining is as satisfactory as shotcrete for use in small canals and laterals. At the time of the above installations (the late 1930's), the use of inexpensive hand labor permitted holding the total cost of the finished lining, including fine trimming and all materials, to $1.80 per square yard. However, it is extremely unlikely that linings of this type could be economically placed in the United States today.

Brick linings have been used rather extensively in India where an abundance of inexpensive hand labor is available and where materials for concrete linings are difficult to obtain.[7 10] The Haveli Canal in India was lined in 1937 with a double layer of 12- by 5⅞- by 2½-inch tile brick. The bottom layer was bedded in ½ inch of 1:6 cement mortar. Both layers were placed with ½ inch of 1:3 cement mortar between the bricks and between the layers. The lining was reinforced with ¼-inch bars at 24½-inch spacings, longitudinally and transversely, on the bottom; and 12¼-inch spacings, longitudinally and transversely on the side slopes. A plaster coat was applied to the surface of the last layer of

[7] Op. cit. p. 55.

[10] Sain, Kanwar, "Canal Lining in India," ICID—Third Congress on Irrigation and Drainage, Transactions, vol. II, R.11, Q7, p. 7.145-7.175, 1957.

bricks. Except for some damage from settlement of the subgrade, this lining has been satisfactory. The report states that in future work, brick 10 by 4⅞ by 2¾ inches in size were to be used, and that the reinforcement would be eliminated because experience indicated that any damage from back pressure or flotation was increased by reinforcement which prevented early failure in small localized areas.

No cost data are available for this work in India. The construction of brick or precast concrete block lining requires little equipment but considerable hand labor. Because of the high cost of both bricks and labor in most localities, brick linings are not economically attractive in this country but may have some limited use.

34. *Stone Linings.* — Stone or rubble masonry linings were more widely used in years past than at present primarily because of the excessive amount of hand labor involved. A number of such linings in southern California have been in service for many years and are still in good condition.

More recently, rock masonry linings have been utilized in small laterals of the Bureau's Carlsbad project in New Mexico. Placed by Civilian Conservation Corps forces, the typical installation consisted of natural rock slabs 2 to 4 inches thick butted together with cement-mortar joints. These linings have given good service with little maintenance expense. The bottoms were wide and flat, and slabs forming the sides sloped away from the canal only a few degrees from the vertical.

Linings of stone are obviously too high in cost to be seriously considered in most localities today. Even where an abundance of suitable stone exists, in most cases the cost of preparing and placing it would make such a lining economically prohibitive.

35. *Soil-Cement Linings.* — Soil-cement offers possibilities for use as a canal lining material in localities where subgrade or adjacent soils are of a sandy nature and other suitable lining materials are not readily available. As the name implies, soil-cement canal linings are made up of a mixture of portland cement and natural soil.

Although other soils can be considered for lining use, laboratory tests indicate that for best results soils for this purpose should be well graded with a maximum size of ¾ inch and contain between 10 and 35 percent fines passing the No. 200 sieve. Soil-cement must be protected from freezing for a reasonable period after placement in cold weather and should be cured for 7 days. Application of a bituminous coating is recommended for curing.

(a) *Standard Soil-Cement.*—Soil-cement linings are divided into two general types, standard and plastic. Standard soil-cement is compacted with the moisture content of the mix at or just above the optimum as determined by laboratory compaction tests. Tests to determine and control the optimum cement content for highway purposes are described in ASTM Standards D-558-57, D-559-57, D-560-57, and D-806-57.[11] These procedures are generally applicable for canal lining work, except that the cement content is arbitrarily increased a minimum of 2 percentage points to insure adequate resistance to the flow of water.

Material mixing for standard soil-cement is best accomplished by traveling mixing machines or stationary plants. Mixing in place in the canal invert and on side slopes of 4 to 1 or flatter has been found satisfactory. Soil-cement for placement on side slopes steeper than 4 to 1 should preferably be deposited directly on the side slopes from traveling mixers of the pugmill type or from trucks hauling from stationary plants. An alternate method is the mixing of the soil-cement on existing soil-cement lining placed in the invert, that has sufficiently cured and hardened to withstand, without damage, the wheel loads that are applied in the mixing and the transfer of mixed material to the side slopes with equipment such as gradals or graders. All lining materials on the bottom and side slopes must be compacted to specified density. To date this type of lining has not been entirely successful.

(b) *Plastic Soil-Cement.*—Plastic soil-cement has higher water and cement contents than standard soil-cement, and a consistency com-

[11] "Standard Methods of Test for Moisture-Density Relations of Soil-Cement Mixtures," ASTM D-558-57; "Standard Method for Wetting and Drying Tests of Compacted Soil-Cement Mixtures," ASTM D-559-57; "Standard Method for Freezing and Thawing Test of Compacted Soil-Mixtures," ASTM D-560-57; and "Standard Methods of Test for Cement Content of Soil-Cement Mixtures," ASTM D-806-57—ASTM Standards 1961, part 4, pp. 1316, 1343, 1349, and 1356, respectively.

parable to that of portland cement concrete. These properties permit placement of plastic soil-cement linings by means of a slip-form similar to that used in the placement of portland cement concrete linings, and therefore frequently at a lower installed cost than for standard soil-cement linings. Plastic soil-cement may be mixed in a pugmill type mixer or in a stationary plant.

(c) *Experimental Installations*—In 1945, several very short sections of the Main Canal on the W. C. Austin project, Oklahoma, were stabilized with standard soil-cement and one section was lined with plastic soil-cement. Plant mixing by means of a 2-cubic-yard concrete mixer, although not entirely satisfactory, was utilized in both instances. The subgrade soils used in the mix had an average gradation of 100 percent passing the No. 4 sieve and 60 percent passing the No. 200. Sixteen percent cement (by volume) was used in the plastic soil-cement and 10 to 12 percent cement (by volume) was used in the standard mix. The lining thickness, or stabilized depth, was between 4 and 6 inches. In 1957, the plastic soil-cement was in fair to good condition. The standard soil-cement was in poor condition on the slopes; the bottom could not be examined.

In 1947, in cooperation with the Portland Cement Association, a 4,480-foot reach of plastic soil-cement lining was constructed on a lateral of the same project to ascertain the cost and feasibility of placing a plastic soil-cement mixture on a production basis. The natural soils were poorly graded silty fine sands of the SP-SM type, based on the Unified Soil Classification System (see table 10, sec. 44). The canal was excavated by dragline and finished with a ditcher to form a base width of 4.0 feet, side slopes of 1½ to 1, and slope length of 5.41 feet. A traveling-plant mixer and a subgrade-guided slip-form (fig. 31) completely mechanized this

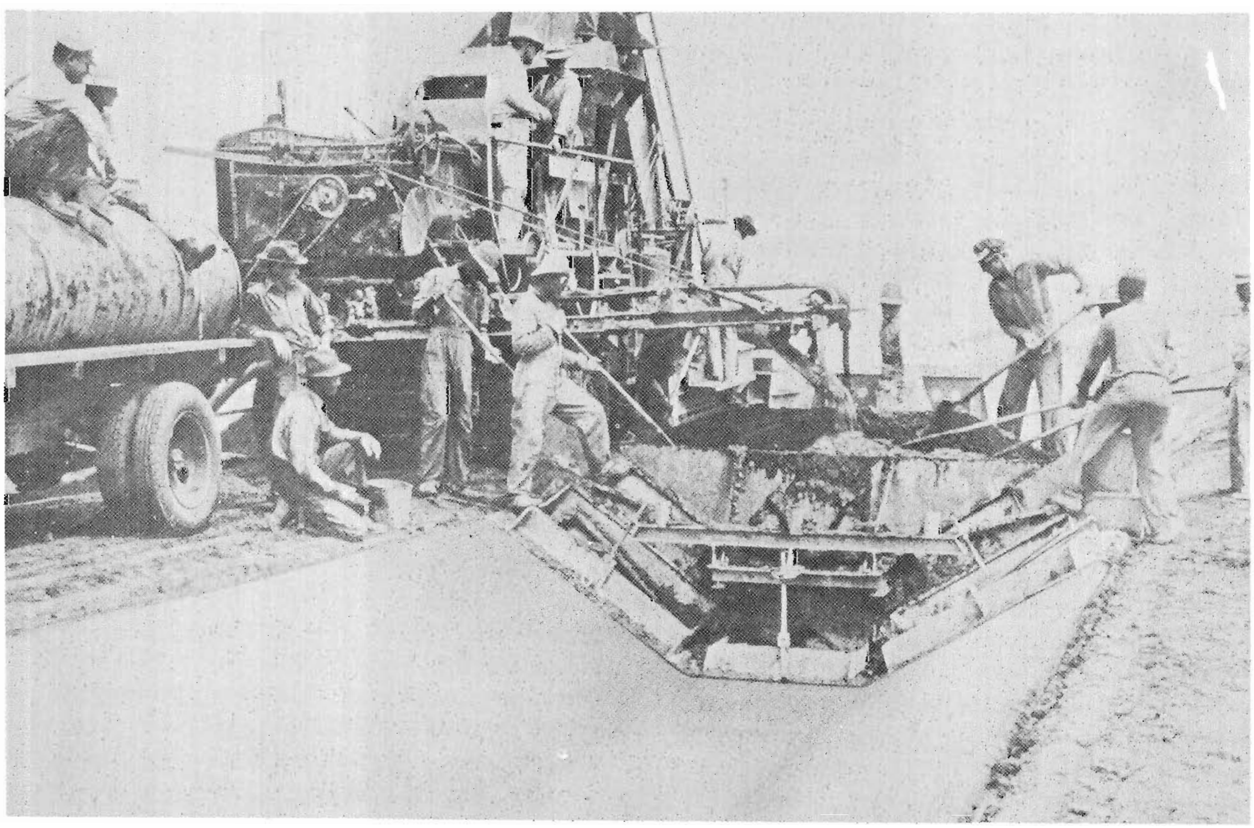


Figure 31.—Traveling plant mixer and subgrade-guided slip-form used in placing plastic soil-cement lining in West 11.5 Lateral of the W. C. Austin project, Oklahoma. PX-D-1026.

lining operation. The mixer was of the pugmill type with an overhead storage bin and a pickup conveyor. The sandy soil to be used in the soil-cement was windrowed along the side of the lateral, and the loose cement was placed on top of the windrow.

The 3-inch-thick lining was placed in different sections containing 3, 3.9, 4.5, and 6 sacks of cement per cubic yard, or 11.1, 14.5, 16.7, and 22.2 percent by volume, respectively, to determine the variations in lining performance due to cement content. With this equipment a lining progress of 300 feet per hour was attained. The average cost of this plastic soil-cement lining was 76 cents per square yard, which included labor, equipment rental, and all materials, but not trimming or contractor profit. It is estimated that with trimming costs, the lining would have cost about $1.10 per square yard. Various curing methods were also tried including application of moist soil, white-pigmented sealing compound, 30 percent paraffin wax with 70 percent diesel fuel, and RC-2 bituminous compound.

When last examined in detail in 1957, it was apparent that the greater the cement content of the plastic mixture, the more durable the lining. The first two methods of curing gave similar results as to quality of the lining, particularly with the leaner mixtures. The RC-2 bituminous compound provided much superior curing and produced a more durable lining so far as spalling is concerned. Surface disintegration or spalling of the lining was more severe below the waterline. All of the lining except the section using the 11.1 percent mixture was still serviceable after six irrigation seasons. The section using the 11.1 percent mixture, which was 3.4 percent below the normally recommended cement content, was badly deteriorated. The remaining sections, having greater cement contents, were in fair to good condition with very little change being noted during the preceding 5 years. The average seepage measured in 1950 was only 0.12 cubic foot per square foot of wetted area per 24 hours, indicating that the lining was still effective at that time in reducing seepage losses. All of the lining continues to be effectively reducing seepage and erosion, and those portions with higher cement contents can be made serviceable for many years with reasonable maintenance.

Mixed-in-place and plastic type soil-cement linings were placed for experimental purposes in 1947 and 1948 on the Yuma Mesa division of the Gila project, Arizona, and on the East Mesa lands in the Imperial Valley of California. Placement of the mixed-in-place lining involved merely spreading dry cement on the shaped section, mixing soil and cement with garden rakes, and wetting by ponding in the ditch section. The plastic soil-cement was placed by subgrade-guided slip-form, by pneumatic methods, and by hand.

After initial cracking and surface deterioration which occurred shortly after placement, deterioration of the mixed-in-place soil-cement lining has not been excessive. With a moderate amount of maintenance the linings should be serviceable for many more years in the mild climate. Plastic soil-cement placed with the subgrade-guided slip-form was found to be in better overall condition than the mixed-in-place lining or the ½-inch- and ¾-inch-thick pneumatically applied plastic type; in fact it compared favorably with experimental portland cement concrete linings placed at the same time in which pit-run and graded aggregates were used. The pneumatically placed plastic soil-cement lining exhibited transverse and axial cracking, and crazing. The hand-placed plastic soil-cement lining had deteriorated very little, and was in very good condition.

An experimental section of plastic soil-cement lining was placed during 1948 on a canal of the Boise project, Idaho. The 300-foot-long reach was constructed using 4.7 sacks of cement per cubic yard, or 14.3 percent cement by volume. Examined in 1958, the lining was in very good condition (fig. 32). There was some evidence of surface scaling, and a few local pockets of erosion and some cracking were observed; however, the lining was still very serviceable and effective in controlling seepage losses. Examination made in 1962 indicated that the lining had deteriorated little, if any, since it was examined in 1958.

Additional standard type soil-cement linings were more recently placed on the Columbia Basin project in 1954 and 1956. Improper mix-

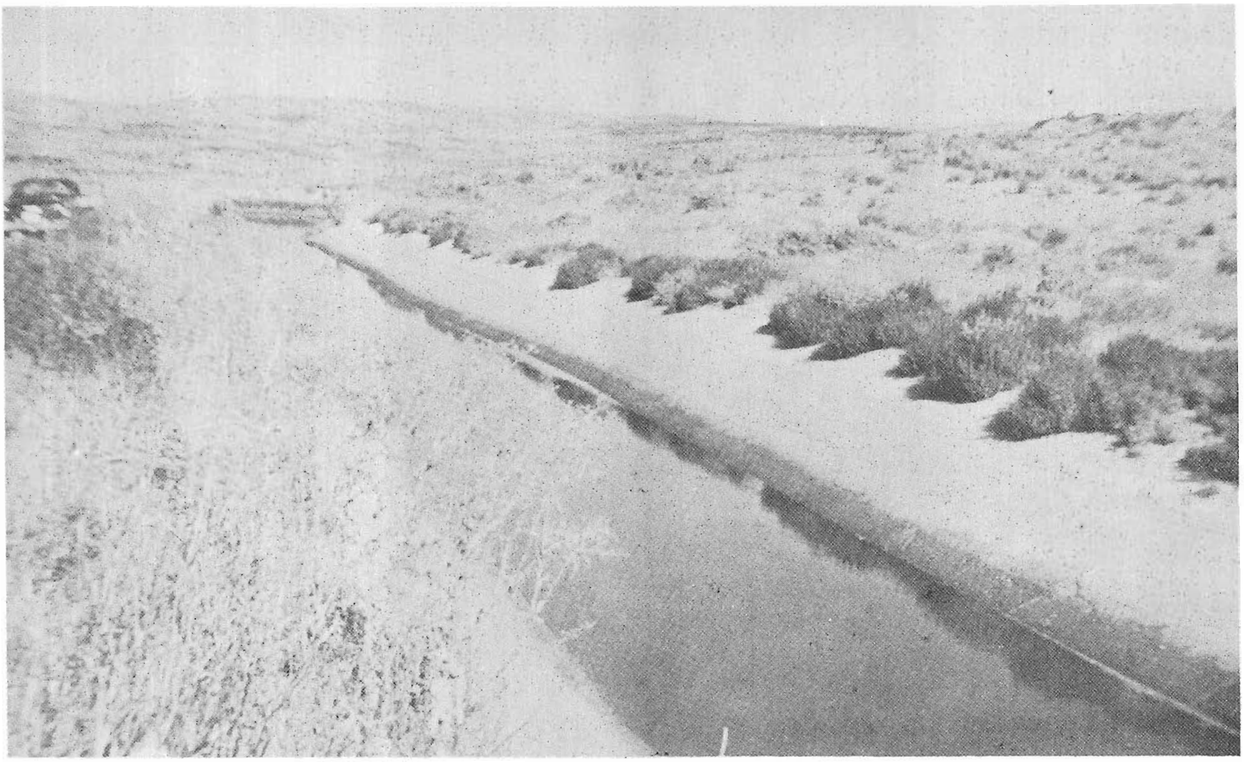


Figure 32.—A plastic type soil-cement lining placed on the Boise project in 1948 which is still in good condition after 14 years of service. Although the lining shows some evidence of scaling, a few local eroded pockets, and some cracking, it is still serviceable and effective in controlling seepage losses. P3-D-15424.

ing procedures and a lack of density were believed to have contributed to the generally poor durability of previous linings of this type, and the Columbia Basin project experiments were initiated to reduce these two deficiencies. However, under the field conditions encountered, the deficiencies were not completely eliminated.

In the 1954 lining of a small lateral, a traveling mixer was used and compaction was accomplished by a plate type vibrator. In the 1956 installation in a larger lateral, a multiple-pass, rotary mixer was used to combine the soil and cement, and the 5-inch-thick lining was placed and rolled in the longitudinal direction on both the bottom and side slopes by a rubber-tired, self-propelled, heavy-duty roller (fig. 33). Both of the above linings contained areas of poor uniformity and compaction. The estimated cost of the later installation, which was performed under contract, was about $1.60 per square yard.

Figure 33.—Compaction of soil-cement on lateral side slopes with a self-propelled pneumatic-tired roller attached by cables to a tractor for support. PX-D-32562.

Chapter V

Buried Membrane Linings

36. *General.*—A buried membrane canal lining consists of a relatively thin and impervious water barrier covered by a protective layer which forms the water-carrying prism. The membranes discussed in this chapter include sprayed-in-place asphalts, prefabricated asphaltic materials, plastic films, and relatively thin layers of bentonite or other types of clay. The membranes are installed to reduce seepage through the banks and bottom of a waterway, and are covered to protect them from exposure to the elements and from injury by turbulent water, stock, plant growth, or maintenance equipment. The need for the protective cover became apparent after early trial installations showed that the membranes had little resistance to field hazards.

Earth and gravel are generally used as the covering material for buried membrane linings; however, protective covers of shotcrete, asphalt macadam, and other materials have been used experimentally in the protection of membranes constructed of sprayed-in-place asphalts and prefabricated asphaltic materials.

37. *Design Considerations.*—(a) *Canal Section.*—The buried membranes that have been used are almost completely watertight if properly placed, and their life expectancy is dependent primarily on the adequacy of the cover material used to protect them from weather, erosion, and mechanical damage. Since earth is usually the least costly cover material, it is the most frequently used. To effectively provide the protection necessary to the safety of the membrane, the channel section must be completely stable so that little or no erosion or sliding will occur.

To arrive at such a section the banks must be made statically stable. This must be accomplished even though the cover is composed of unconsolidated earth which is saturated when the canal is in use, rests on a membrane that introduces a weak shear plane, and is supported by a subgrade that may be either wet or dry. No particular slope will satisfy all subgrade and cover types, but a side slope of 2 to 1 has been found satisfactory in most cases. This is the maximum slope recommended for asphalt or plastic membranes, and a flatter slope probably will be required for membranes of this type if the cover is composed of relatively unstable material such as uniformly graded sands, fine gravels, or silty sands.

A special problem of stability has arisen where the side slopes are 2 to 1 and two layers of cover material have been used. This is discussed in subsection (c) following.

In addition to being statically stable, the channel section must resist the scouring effect of the flowing water. This requirement usually results in a base width to depth ratio of about 4 to 1 in most materials. The design of stable sections has been the subject of many engineering publications over the past century. Each offers some improvement in approach, form, or special applicability. Designs are usually based on assumptions that are not verified at the present time by laboratory or field testing prior to construction in a project area.

In attempting to develop a method of stable channel analysis that uses data obtainable from field or laboratory testing, and is less dependent on experience or judgment, the shear or "tractive force" theory is being studied and is discussed in the following subsection. Obviously, if the transported water will bring a bedload with it, the analysis must include the transport of this material. The question of bedload transport is rarely involved where buried membrane linings have been used, but should the problem be present, a tentative method of analysis is available.[1]

(b) *Tractive Force Theory.*—Permissible velocities in unlined earth canals and in canals lined with erodible materials are usually determined on the basis of the erosion resistance of the earth or lining. It is common knowledge

[1] Terrell, P. W., and Borland, W. M., "Design of Stable Canals and Channels in Erodible Material," Transactions ASCE, 1958, vol. 123, pp. 101-115.

that in larger canals the earth or lining materials will withstand higher mean velocities before scour occurs, than the same materials will withstand in smaller canals. The tractive force theory of design of stable channels is consistent with this and other available data, so it appears the theory can be used to assist in design of earth canals and canals having erodible cover materials used over membrane linings. When developed, the analysis will give the designer a tool for more exact appraisal of the actual safety factor of a section and a better knowledge of potential weak spots.

The theory is based on the assumption that movement of material on the bed and sides of the channel depends on the shear transmitted to the bed and sides by the flowing water.[2 3 4 5] Consider as a hypothetical example (fig. 34) a section of channel infinitely wide, flowing at constant depth in the direction of the arrow. The column of water 1 foot square and with a depth d in feet has a weight $W = wd$, where w is the weight of water in pounds per cubic foot. The shearing force of the column of water on the bed of the channel is $T = W \sin a$ where a is the angle of the slope of the streambed with a horizontal line. For small angles, corresponding to slopes of irrigation canals, the sine of the angle is nearly equal to the tangent of the angle, and the tangent of the angle is equal to the slope, $s = h/L$. Therefore, $T = W \sin a = W \tan a = Ws = wds$. In this equation T is the tractive force in pounds per square foot exerted on a square foot of streambed by the 1-foot-square column of water flowing down the slope. The tractive force is resisted by the streambed, and it is this force, caused by water flowing over the

[2] Glover, R. E., and Florey, Q. L., "Stable Channel Profiles," Hydraulic Laboratory Report No. Hyd-325, Bureau of Reclamation, September 1951.

[3] Lane, E. W., "Progress Report on Results of Studies on Design of Stable Channels," Hydraulic Laboratory Report No. Hyd-352, Bureau of Reclamation, June 1952.

[4] Carter, A. C., "Critical Tractive Forces on Channel Side Slopes," Hydraulic Laboratory Report No. Hyd-366, Bureau of Reclamation, February 1953.

[5] Lane, E. W., "Design of Stable Channels," Transactions ASCE, 1955, vol. 120, pp. 1234-1279.

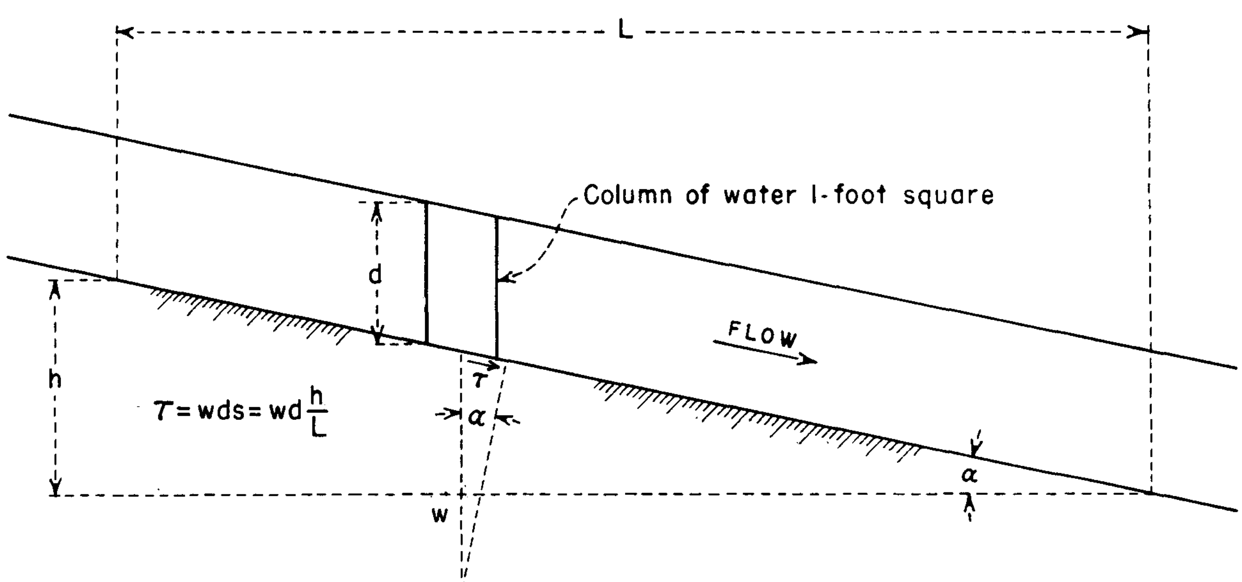


Figure 34.—Tractive force on canal bottom. 288-D-2643.

resistant boundary, that causes the material of the subgrade to move or scour.[6]

Considering a typical canal with a trapezoidal cross section, part of the total tractive force of the flowing water is transmitted to the canal sides, thereby reducing the tractive force transmitted to the bottom to an amount below the value $T = wds$ determined for the bottom of an infinitely wide channel. The amount of tractive force transmitted to the channel sides and the reduction in tractive force wds transmitted to the channel bottom depends on the ratio of bed width to depth and on the slopes of the channel sides. The tractive force on the sloping sides of the channel varies up the slope and for normal sections is less than the tractive force on the bottom because of the lesser depth. However, the slope material tends to roll down, and the combined effect of this action with the tractive force may result in a greater tendency to scour on the side slopes than on the bottom.

Tests have been made and data taken to determine the tractive forces necessary to just begin to move various sizes of noncohesive sand and gravel materials.[6,7] These forces are designated critical tractive forces. Critical tractive forces for uniform granular noncohesive materials are quite well defined, but for cohesive materials and mixtures of both, the critical tractive forces have not been as well established. In the past few years tests to obtain these values have been conducted on sections of canals in the field, supplemented by laboratory materials testing. These data are now being analyzed (1962).

In addition to the above tests, equipment has been developed to evaluate soil properties that influence erosion resistance. Results of this laboratory work and the above field work have been correlated, with encouraging results, and an interim report is being prepared. Additional data on many soil types and further study of the theory are needed to determine the limits of its applicability. At present, the theory appears to be sufficiently accurate to compare canals of different sizes in similar materials, to serve as a guide in the selection of section properties for canals in materials that have been tested, and to evaluate the adequacy of channel banks. The theory and testing equipment may ultimately result in a method whereby obtainable soil and geological data can be used directly in an equation to obtain a theoretically exact section which is at the point of critical tractive force. This theoretical section would probably never be used, but a knowledge of its characteristics would be of considerable value to the designer.

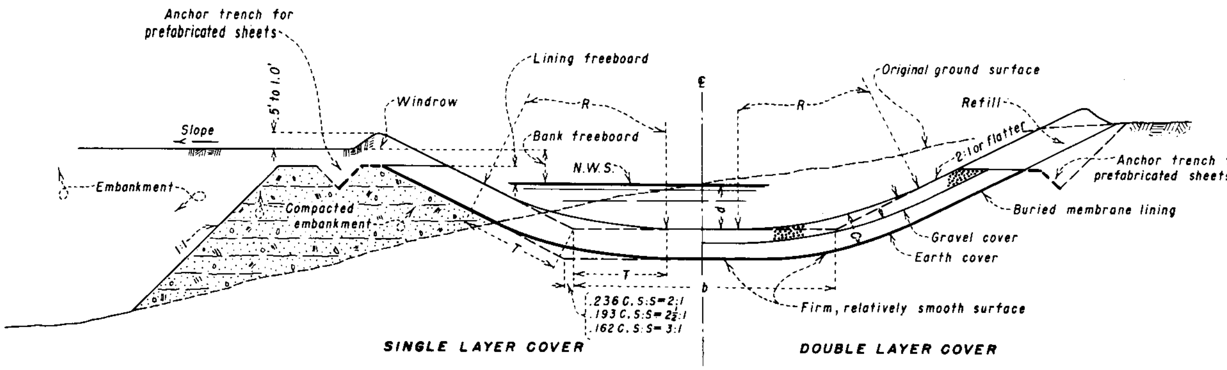
(c) *Cover Material.*—The most expensive part of a buried membrane lining is the excavation for and placement of the cover material. Obviously, for economy, this material should be locally available and placed as thin as possible for adequate protection of the membrane. Figure 35 shows the recommended minimum to be one-twelfth the water depth expressed in inches plus 10 inches. The minimum should be used only when the cover material is a clayey gravel, a gravel-surfaced soil, or some equally erosion-resistant material. Table 10 will be of value in comparing soils for relative erosion resistance and stability. Should local materials be fine grained and noncohesive, a greater total thickness may be required and gravel protection should be provided. An analysis of the relative economy of using a gravel cover with a smaller section, as compared to a larger section and thicker cover of less stable materials, should be made.

Cover thickness discussed above is primarily related to slope stability under operating conditions. Other local conditions which will have a bearing on the side slopes and thickness of cover, are the type of cleaning equipment to be used; the degree of beaching expected; the amount and type of animal traffic on bank slopes and bottom; and localized scour, particularly at curves and structures.

Some difficulty has been experienced with the cover material slipping down the slope. This has occurred where the side slopes are 2 to 1 and when two layers of cover material, a gravel layer over an earth layer, have been used. It has been suggested that possibly a single blended layer of cover material which consists of gravelly material mixed with earth or sand may be

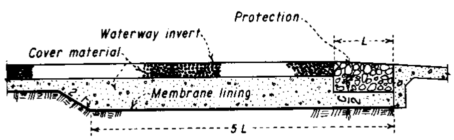
[6] Lane, E. W., and Carlson, E. J., "Some Factors Affecting the Stability of Canals Constructed in Coarse Granular Materials," Proceedings Minnesota International Hydraulics Convention, September 1-4, 1953, pp. 37-48.

[7] Lane, E. W., and Carlson, E. J., "Some Observations on the Effect of Particle Shape on the Movement of Coarse Sediments," Transactions, American Geophysical Union, June 1954, vol. 35, No. 3, pp. 453-462.

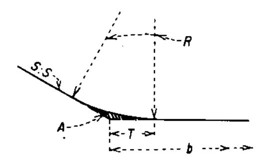


TRANSVERSE SECTION OF WATERWAY

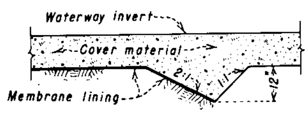
$C = 10'' + \frac{d}{12}$; Where C is in inches and d is water depth in inches. (12" Min.)



LONGITUDINAL SECTION
MEMBRANE AT STRUCTURES



b	T	S:S = 2:1, R=4.23608 T, ΔWP=.00850 R, A=.00425 R²			S:S = 2½:1, R=5.19259 T, ΔWP=.00466 R, A=.00233 R²			S:S = 3:1, R=6.16226 T, ΔWP=.00280 R, A=.00140 R²		
		R	2ΔWP	2A	R	2ΔWP	2A	R	2ΔWP	2A
2	1.0	4.24	.07	.15	5.19	.05	.13	6.16	.03	.11
3	1.0	4.24	.07	.15	5.19	.05	.13	6.16	.03	.11
4	1.0	4.24	.07	.15	5.19	.05	.13	6.16	.03	.11
5	1.5	6.35	.11	.34	7.79	.07	.28	9.24	.05	.24
6	2.0	8.47	.14	.61	10.39	.10	.50	12.32	.07	.42
8	2.8	11.86	.20	1.20	14.54	.14	.99	17.25	.10	.83
10	3.5	14.83	.25	1.87	18.17	.17	1.54	21.57	.12	1.30
12	4.3	18.22	.31	2.82	22.33	.21	2.32	26.50	.15	1.97
14	5.0	21.18	.36	3.81	25.96	.24	3.14	30.81	.17	2.66
16	5.5	23.30	.40	4.61	28.56	.27	3.80	33.89	.19	3.22
18	6.0	25.42	.43	5.49	31.16	.29	4.52	36.97	.21	3.83
20	6.5	27.53	.47	6.44	33.75	.31	5.31	40.05	.22	4.49
22	7.0	29.65	.50	7.47	36.35	.34	6.16	43.14	.24	5.21
24	7.5	31.77	.54	8.58	38.94	.36	7.07	46.22	.26	5.98
26	7.9	33.47	.57	9.52	41.02	.38	7.84	48.68	.27	6.64
28	8.3	35.16	.60	10.51	43.10	.40	8.66	51.15	.29	7.33
30	8.7	36.85	.63	11.54	45.18	.42	9.51	53.61	.30	8.05
35	9.5	40.24	.68	13.76	49.33	.46	11.34	58.54	.33	9.60
40	10.5	44.48	.76	16.82	54.52	.51	13.85	64.70	.36	11.72
45	11.2	47.44	.81	19.13	58.16	.54	15.76	69.02	.39	13.34
50	12.0	50.83	.86	21.96	62.31	.58	18.09	73.95	.41	15.31



LONGITUDINAL SECTION
METHOD OF ENDING MEMBRANE

NOTES

The gradation, type and thickness of the cover material is dependent on tractive forces and velocities in the section, and the type of material available in the area.
ΔWp is the difference between 2T and the arc length. To obtain the wetted perimeter for a section, subtract 2ΔWp from the wetted perimeter for a trapezoidal section of width b.
A is the area of the fillet. To obtain the area for a section, subtract 2A from the area of a trapezoidal section of width b.
For freeboard see 103-D-341

Figure 35.—**Details of buried membrane lining installations. From drawing 103-D-632.**

more stable, particularly during drawdown, than two dissimilar layers. The inability of the finer layer to drain as rapidly as it should may be a contributing factor to the instability of the two layers.

The stability of the cover can sometimes be improved by compaction. If good materials for cover are expensive, selection of less desirable soils and improving their suitability by rolling may be advisable. Compaction equipment should be used with care and only after sufficient cover has been placed to protect the membrane. Also, a long-radius curve connecting the side slope with the base will add materially to the stability. This, of course, will decrease the canal capacity slightly, but a small increase in canal width or depth will compensate for the loss (fig. 35).

Any rapid drawdown of the water surface in a membrane-lined canal will tend to cause the cover to slide down the slope. Rapid fluctuations in the water surface must therefore be avoided, or if this is impracticable, the slope should be flattened and the cover material carefully selected so as to be free draining without loss of fines.

In large canals, wind-generated waves usually cause erosion of most earth materials at the waterline. While this is frequently of little concern in unlined canals, it may cause failure in the cover material of a membrane lining. To provide information on this subject, wind-generated waves have been measured on Bureau canals, and these data, together with the physical conditions at the site, tabulated.[8] [9] Using this information as a basis, a flume and wave-generating machine have been constructed in the laboratory and calibrated to simulate wave heights found in canals. The equipment will be used to evaluate the probable severity of beaching and hence the need for protection of both unlined canals and canals with erodible linings. This information will be of value in project cost estimating and in comparing costs of various lining types. Whether the data can be correlated with soil mechanics procedures to provide an analytical solution will be studied.

In larger canals a beach belt of gravel is sometimes installed to increase the safety at this most critical cover point. Animals will get into the canal, unless adequate fencing is provided, and dislodge the cover. Their hooves will penetrate the saturated bottom cover. Where riprap is used below structures, the cover thickness should be increased to provide additional protection to the membrane. This greater depth of cover should be provided for a distance downstream from a structure at least twice the length of installed riprap to permit the placement of additional riprap should further scour occur. These and other local conditions should be considered when determining cover thickness.

(d) *Miscellaneous.*—Buried membranes will resist some external hydrostatic pressure. However, they can be severely damaged, particularly if side pressures become excessive. Providing for the relief of these pressures is discussed in section 20(d).

[8] Enger, P. F., "Hydraulic Model Studies to Determine the Required Cover Blanket to Prevent Fine-Based Material from Leaking Due to Wave Action—Kennewick Main Canal, Yakima Project, Washington," Hydraulic Laboratory Report No. Hyd-381, Bureau of Reclamation, June 1954.

[9] Carlson, E. J., "Gravel Blanket Required to Prevent Wave Erosion," Hydraulics Division Journal, Proceedings ASCE, vol. 85, No. HY5, March 1959, pp. 109-145.

A windrow along the top of the slope (fig. 35) is advisable to prevent bank drainage from eroding the slopes and to supply some extra material for cover as consolidation takes place during saturation and settlement.

In partial fill sections the degree of compaction for the portion of bank supporting the membrane is not as important as it is for rigid lining, but compaction to 95 percent of laboratory maximum density is believed advisable.

Crushed rock or gravel cover material should never be placed directly on the membrane because of the danger of puncture. Conversely, the membrane should never be placed on a subgrade of gravel or other material containing large voids because of the possibility of rupturing the membrane.

The permissible water velocity in a membrane-lined canal will usually be somewhat less than that in an unlined canal constructed in a soil having the same characteristics as the cover material of the lined canal. The earth covers are generally placed without benefit of maximum consolidation and in this state scour is a hazard to the membrane.

38. *Buried Asphalt-Membrane Linings.* — Essentially, this lining consists of a membrane approximately one-fourth inch in thickness, composed of a special high-softening-point asphalt sprayed in place at a high temperature (400° F.) on a prepared subgrade to form a waterproof barrier that is protected against injury and weathering usually by a layer of earth and gravel (fig. 35). Other protective covers such as shotcrete, asphalt macadam, or in special instances portland cement concrete have been used on an experimental basis.

In addition to providing an effective means of seepage control at low cost, buried asphalt membrane lining can be satisfactorily installed in cold and wet weather such as that frequently encountered in northern latitudes during the nonirrigation season. Late fall and winter, when the canal system is not in use, is frequently the most convenient time for installing canal linings on operating projects. Freezing temperatures and wet subgrade conditions may prohibit the installation of many conventional types of linings. However, asphalt membranes may be placed satisfactorily over frozen subgrades and sometimes, though it is not recommended, the membranes may be installed over a very light covering of snow. Factors contributing to the low construction costs on many membrane lining installations are that they may be constructed with equipment ordinarily in the possession of the average highway contractor and that the lining may be placed during an otherwise slack construction season.

The first trial installation of a buried asphalt-membrane lining was made in a small lateral on the Klamath project, California, in September 1947. Since that date, nearly 6,000,000 square yards have been placed by the Bureau. Nearly 2,000,000 square yards have been placed on the Columbia Basin project, Washington; more than 1,000,000 square yards have been placed on the Riverton project, Wyoming, and a similar amount on the North Platte project, also in Wyoming. Membrane linings of this type are currently being placed on many other projects.

The average contract cost of buried asphalt-membrane linings with an earth or gravel cover has been approximately $1 per square yard, with a maximum of about $1.50 per square yard. The cost depends on the quantity of asphalt required, the location of refineries with respect to the work, working conditions, and the availability of suitable cover materials. Higher costs may result if extremely adverse weather prevails, if the subgrade is such as to require excessive quantities of asphalt, or if gravel or other special types of cover must be used due to high water velocities or undesirable local soil.

(a) *Construction Methods, Materials and Equipment.*—Factors involved in buried asphalt-membrane lining construction include subgrade preparation; cover material thickness and types; and the asphalt quantity, type, and application methods. Ordinarily, subgrades are dragged and rolled to secure a smooth surface which permits obtaining a reasonably uniform membrane of minimum specified thickness without the use of excessive amounts of asphalt. In areas where the subgrade is composed of rough, irregular, inplace rock, angular or fractured rock and gravel, or open gravels and cobbles, a fine sand or soil "padding" is required for satisfactory membrane support. Some water-saturated subgrades that are very unstable in temperatures above freezing become adequately stable for membrane application when frozen.

Properly applied buried asphalt membranes have proved to be efficient and durable, yet provide a low-cost means of seepage control under many difficult conditions of use. Maximum benefit is obtained from this type of construction by the use of an adequate quantity of the proper type of asphalt carefully applied to avoid holidays (pin holes, air bubbles, and voids) and secure uniform thickness, and the use and maintenance of adequate protective cover. Selection of a type and thickness of cover material to withstand the anticipated water velocities constitutes the major problem.

(b) *Subgrade Preparation.*—The canal section must be excavated sufficiently to provide for the required water prism, plus the cover material, before placing of the membrane. In lining existing canals, the flattening and overexcavation of the side slopes can be accomplished by dragline, or by a ditcher or motor patrol if the equipment can be operated directly in the canal section. After excavation, the subgrade surface is prepared for the asphalt application by light dragging and rolling. The object of these preparatory operations is to obtain a relatively smooth surface which will facilitate the construction of an impermeable membrane of uniform thickness.

(c) *Asphalt Membrane.*—Originally, it was thought that a membrane thickness of about $\frac{3}{16}$ inch (0.187 inch), equivalent to about 1 gallon of asphalt per square yard, was adequate. The average minimum thickness now used, however, has been increased to 0.225 inch, or 1.25 gallons per square yard. In many instances, to assure adequate membrane thickness, 1.50 gallons of asphalt per square yard is used. Use of these greater thicknesses has been found to result in a membrane that has greater freedom from holidays, is less easily damaged during covering operations, and will probably have a longer asphalt life. Costs are not greatly increased by use of the additional asphalt.

Asphalt of very good quality has been obtained for Bureau construction. Early research and testing of many special asphalt materials available indicated that the use of certain catalysts in the asphalt refining process would provide a superior asphalt for membrane construction. Continued testing of samples submitted by the cooperative refiners for consideration and acceptance testing during construction have resulted in improvements in specifications and provided better quality membranes.

Specifications now provide that asphalts blown with catalysts of ferric chloride or other salts of iron will not be accepted. For the most part, asphalt used in membranes has been made with catalysts of phosphorus pentoxide or other phosphoric compounds. Laboratory tests to date indicate that such asphalts are generally superior to the asphalts produced with other catalysts. However, continuing research on the part of refiners and the Bureau laboratories indicates that other catalysts, still in the experimental stage, may produce asphalts for membranes equal or superior to those in which the pentoxide catalyst is used.

Normally, the contractor is required to furnish a certified laboratory analysis showing that the materials in each shipment comply with the specifications requirements. It also is Bureau practice to obtain samples of the hot asphalt cement at the distributor spray bar for laboratory testing. These test results not only provide a check on the certified analyses prepared by the refinery, but will indicate the quality of the asphalt at the time of actual placement. Also, samples of the completed membrane, representing at least the start and end of the work, should be tested to obtain a final analysis of the membrane.

Uniformity of application and avoidance of holidays are very important to satisfactory membrane performance. After the asphalt has been heated to approximately 400° F., it is applied to the subgrade at a pressure of 50 pounds per square inch through slot type spray nozzles, using either hand sprays or multiple spray bars mounted on the distributor. While hand sprays are still used under some conditions (fig. 36), most membranes are now applied with long spray bars which extend over an entire side slope or canal bottom (fig. 37). The spray bars are sometimes manually supported at the ends, but frequently are carried by cables or even

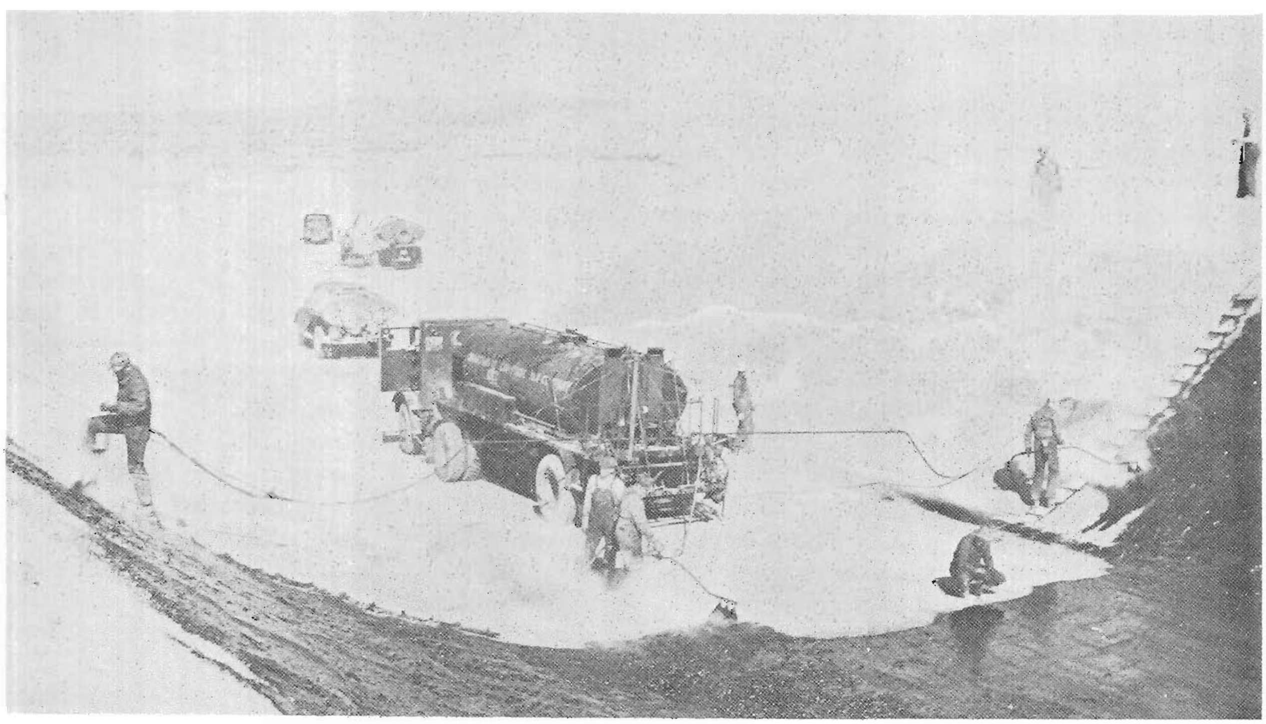


Figure 36.—Constructing a buried asphalt-membrane lining by hand-spray operations on the Wyoming Canal, Riverton project, Wyoming. This method is now seldom used except on small laterals or other areas where space prevents use of large spray bars. CH-179-21.

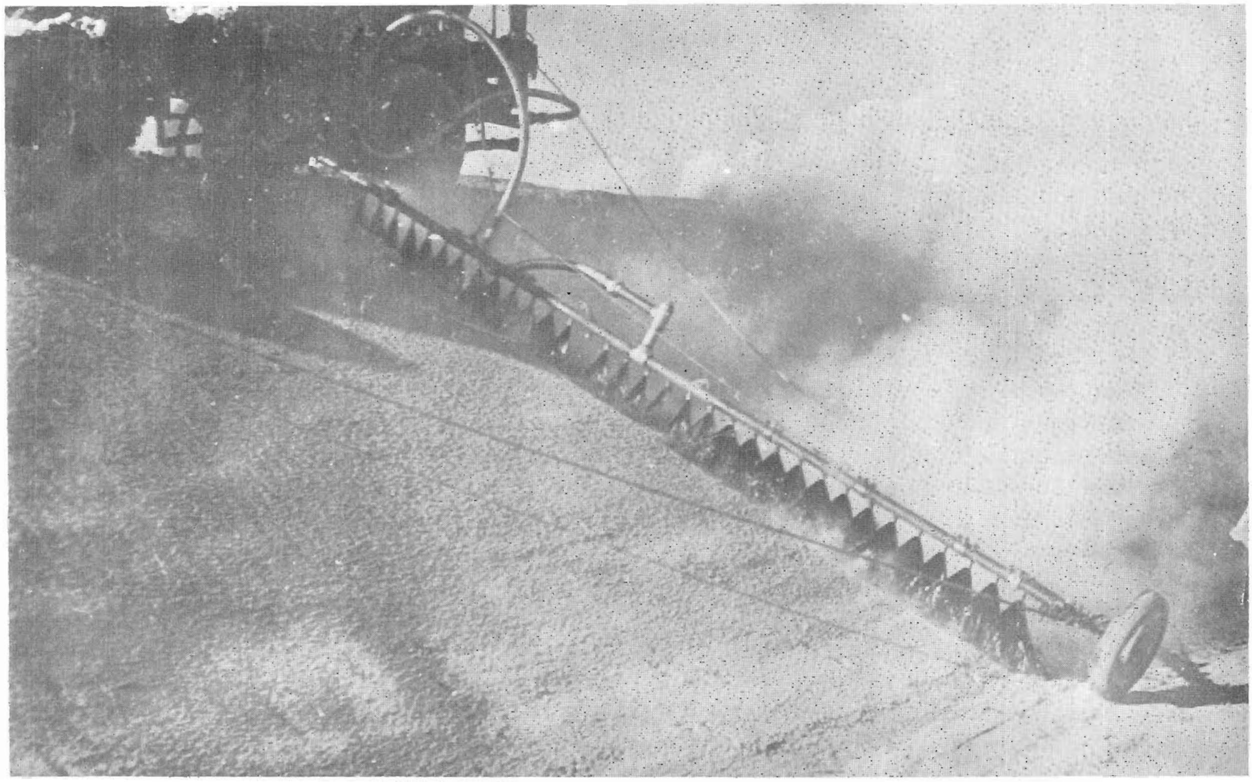


Figure 37.—Efficient application of asphalt-membrane lining with spray bar on the Angostura unit, Missouri River Basin project, South Dakota. PX-D-32264.

directly attached to the distribution equipment to assure their being maintained at a constant distance from the subgrade. The use of distributor spray bars in this way results in a much more uniform membrane thickness and considerable savings in the cost of application. Application of the asphalt is made in two or more passes on side slopes and usually only one in the canal bottom. Hand sprays are advantageous where very rough surfaces or pockets of granular material are being covered, or for retouching holidays in bar-sprayed areas.

Membranes applied over rough surfaces having appreciable quantities of gravel or cobbles may contain holidays. These may result in serious leakage unless careful inspection is made of the membrane and all holidays repaired before placement of the cover. Most instances in which leakage has been noted in completed membrane linings have probably been due to failure to correct holidays in the membrane before placement of the cover. The joining of asphalt membranes at structures should be given special and careful attention, since such areas are particularly subject to settlement and leakage.

(d) *Placement of Protective Cover.*—The hot-applied asphalt cools quickly and is soon ready for application of the cover material. In fact, a few minutes after application the surface may be walked on by construction personnel in covering operations (fig. 38).

Although other materials have been used for the protection of the membrane in experimental installations, earth or a combination of earth and gravel is generally used because of economy (sec. 37(c)). Earth and gravel cover materials should be tested to determine if they meet the design requirements for grading, type, and thickness.

Damaged membrane and serious leakage have resulted from the improper application or choice of protective cover material. Careless placement of earth cover materials containing large rock

Figure 38.—Newly placed asphalt membrane ready for covering. PX-D-19445.

can gouge, tear, crack, or roll the membrane. Blading the cover over the side and down the slopes of a canal frequently results in folding of the top of the membrane. Placement by dragline, whereby the membrane in the bottom is first carefully covered with fine-grained soil and then the slopes are similarly covered from the bottom toward the top, has been the procedure designated in Bureau work. Additional cover of coarser materials can then be applied as needed.

39. ***Prefabricated Asphalt-Membrane Linings.*** — Prefabricated asphalt-membrane linings have been developed to permit the use of an asphalt membrane in smaller canals or in relatively short reaches of large canals where the use of hot asphalt for the sprayed-in-place type lining would require skilled personnel and special equipment. The thinner (⅛- to ¼-inch thick) prefabricated linings are designed to be handled and placed much in the same manner as rolled roofing, with lapped and cemented joints.

Construction procedures similar to those used in the placement of the hot-applied membranes (sec. 38) should be followed in subgrade preparation and placing of protective covers. Earth and gravel covers generally have been provided; however, in some few experimental installations shotcrete and macadam covers have been utilized. Thicker prefabricated asphaltic materials also may be used without cover. These are discussed in section 26.

Buried prefabricated asphalt lining development has been directed towards a relatively thin, lightweight, low-cost material of adequate watertightness and durability, but which may be shipped long distances, stored in hot weather, and placed at low temperatures. One of the original prefabricated linings of this type was developed by the Bureau laboratories and consisted of a layer of catalytically blown asphalt on a heavy sheet of kraft paper. Untreated paper was used so that it would decay rapidly after placing, leaving the asphalt membrane in place. Once installed, this lining performed satisfactorily, but numerous difficulties arose in shipment and handling. In hot weather the asphalt softened and the rolls stuck together; in cold weather the asphalt became so brittle that rapid unrolling cracked the lining.

Other improved types of prefabricated asphalt linings were soon developed. One of these uses a reinforcing of thin glass fiber mat which has been saturated and coated with blown and filled asphalt. This type of lining has proved durable and it ships and handles very satisfactorily; but it is somewhat high in cost ($0.50 to $0.60 per square yard, fob factory). Other materials being tested consist of matted fibers of asbestos, rag, or other organic materials saturated and coated with asphalt.

A large number of trial installations of the covered type of prefabricated lining have been made by the Bureau. Other installations have also been made in cooperation with the project water users and other organizations. The first covered lining was placed in a lateral of the Klamath project in northern California and southern Oregon in December 1949 (fig. 39). This lining was of the paper-backed type and involved some 5,000 square yards. The winter weather required prewarming of the lining to permit placement without cracking. The lining was covered with an average of 1 foot of earth and is giving good service. Similar material was used in an installation on the Yakima project in Washington in 1950 and at the River Laboratory of the Utah State College at Logan, Utah, in cooperation with the Agricultural Research Service.[10] Subgrade roughness and leakage were

[10] Lauritzen, C. W., and Haws, F. W., "1959 Annual Research Report," USDA Agricultural Research Service, SWC, and Utah State University, Logan, Utah, January 1960.

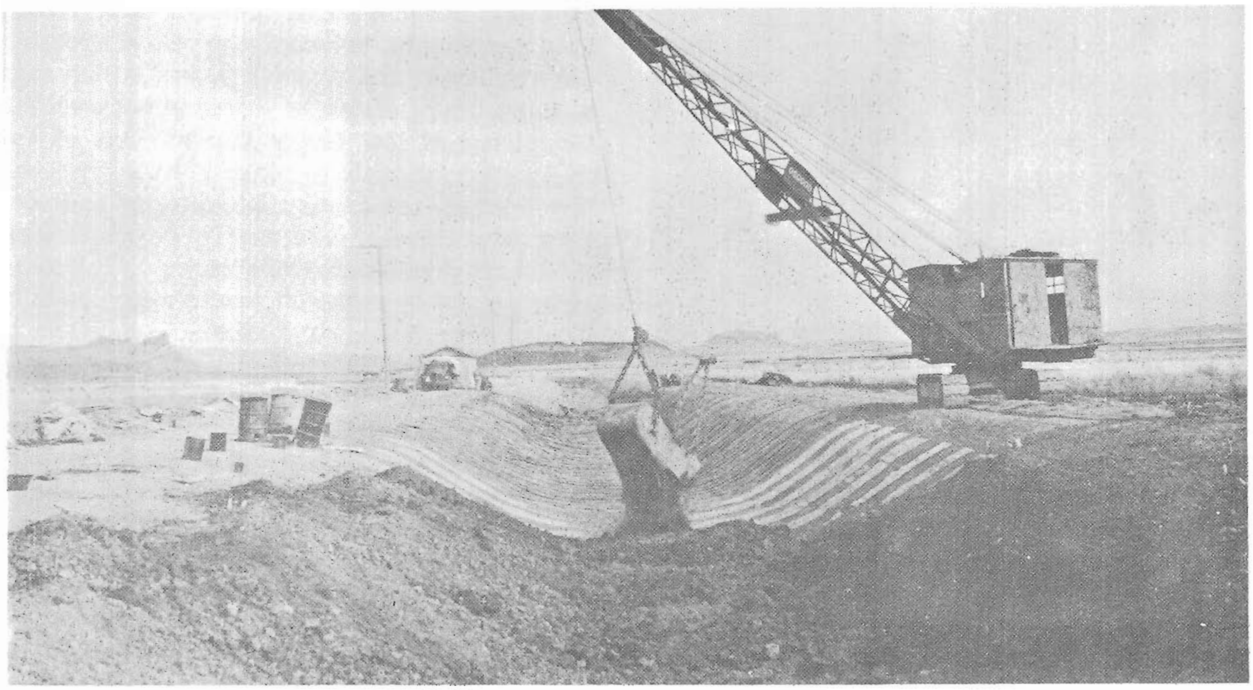


Figure 39.—Placing an earth cover over buried, paper-backed, prefabricated asphalt canal lining on the Klamath Falls project, Oregon. CH-163-25.

determined at the Logan laboratory, and much valuable information was gained by these tests.

In June 1950, after preliminary laboratory tests, glass-fiber-reinforced, prefabricated asphalt linings were installed on the Canal Farm of the Bureau of Reclamation at Denver, Colo. The Canal Farm is an outdoor test area where various materials are installed in test canals and their performance observed. Following this, a field installation was made on the W. C. Austin project near Altus, Okla. (fig. 40). Later installations of similar lining have been made on a number of other projects.

Installations were made on the Boise project, Idaho, in cooperation with engineering personnel of the University of Idaho.[11] The university made studies of construction techniques in connection with the installations, and is continuing observations of the lining durability and effectiveness in controlling seepage. Glass-fiber-reinforced, prefabricated asphalt linings protected with a covering of shotcrete and asphalt macadam have been observed on the Orland project, California (fig. 41), and in a number of private installations during the past few years, and their performance noted.

Asbestos-felt-reinforced, prefabricated asphalt lining was used in an installation on the Boise project in September 1951. This lining, weighing about 50 pounds per roll of 12 square yards, was used in several installations made in 1952.

The results of laboratory and field tests to date indicate that a prefabricated material weighing about 90 pounds per roll and using an inorganic reinforcing is preferred for general use as a canal lining. Future observations can be expected to provide information which may permit the use of other types in special cases.

(a) *Cost.*—The large-scale use of prefabricated lining will depend on its durability and watertightness over long periods, its ease of installation, and its economy. When the cost of the prefabricated material exceeds about $0.35 per square yard, the hot-applied, sprayed-in-place type membrane becomes competitive with the prefabricated material, at least in the larger installations. Isolated areas, remoteness from a

[11] Warnick, C. C., "A Study of the Control of Canal Seepage," Progress Report No. 1, Engineering Experiment Station, University of Idaho, Moscow, Idaho, May 1957.

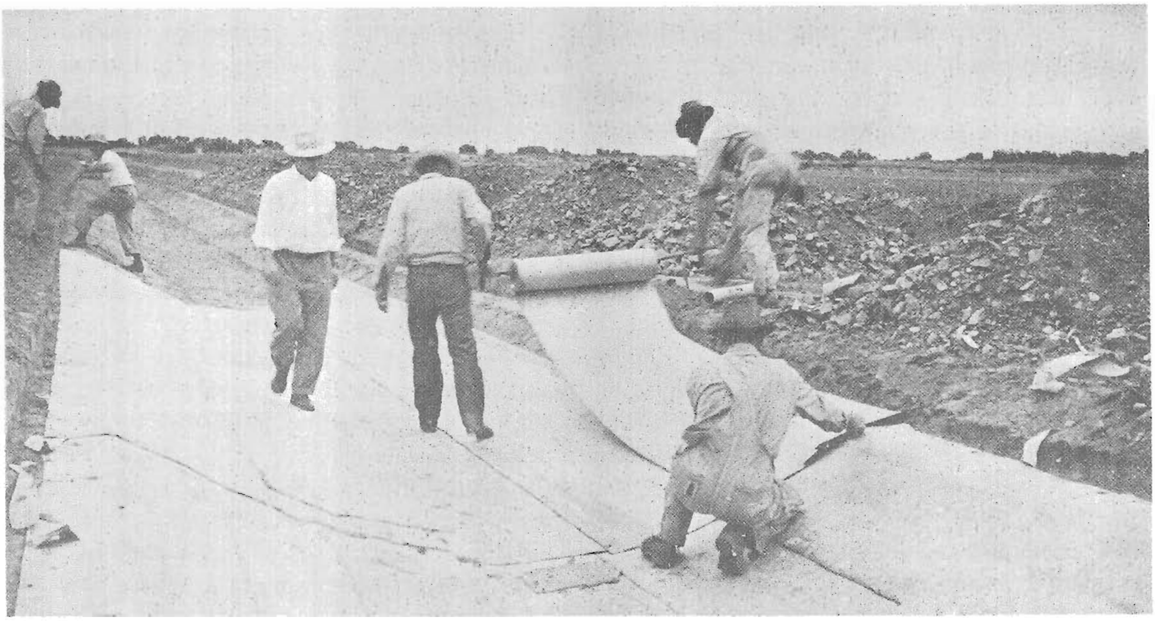


Figure 40.—Placing lightweight, buried, glass-fiber-reinforced, prefabricated asphalt canal lining on the Altus project, Oklahoma. CH-209-21.

Figure 41.—Penetrated macadam cover being placed over lightweight, buried, glass-fiber-reinforced, prefabricated asphalt lining. Two-inch-thick cover consists of pea gravel penetrated with asphalt emulsion. This installation is on the Orland project, California. P868-D-18510, November 1952.

supply of the special asphalts required for sprayed type membrane, lack of special equipment, or piecemeal construction are conditions that favor the use of prefabricated membranes. However, the initial cost of the prefabricated lining material is the most important factor in its large-scale use.

40. *Other Protective Covers for Asphalt Membrane Linings.*—Although more costly than earth and gravel (table 1, sec. 2), several experimental installations have been made using shotcrete and macadam for the protection of asphaltic membranes.

(a) *Shotcrete.*—The rather unusual combination of shotcrete over asphalt was tried first in small outdoor test channels of the Canal Farm at Denver. The lining consists of a thin cover of shotcrete over an asphalt membrane placed directly on the canal subgrade. The idea developed from a need for a stable, erosion-resistant cover for a buried asphalt membrane.

A catalytically blown sprayed-in-place asphalt or a membrane of prefabricated material may be used. From the early laboratory and field tests, it was found advisable to apply a tack coat of RC-O asphalt cutback to the membrane just prior to application of the shotcrete to obtain a better bond between the two materials.

The ¾-inch-thick shotcrete cover for a buried asphalt membrane should be of the same quality as that required for regular shotcrete canal lining discussed in section 30(e). Its application and requirements for curing are also the same, but because the thickness is less than ordinarily employed with regular shotcrete linings, extra care should be exercised in controlling and checking the thickness. Loose shotcrete rebound should be broomed off or troweled into the shotcrete before applying curing compound.

Two field test installations of shotcrete over asphalt membrane were made on the Orland and Riverton projects in California and Wyoming, respectively. About 2,000 square yards were placed on the Orland project in December 1950 and January 1951, using both hot-applied and prefabricated membranes. However, this installation was under the initial handicap of bad weather during construction and subsequent external hydrostatic pressure from unusually high ground water. Several breaks occurred in the bottom of this lining before it was put in service, but the side slopes and much of the bottom were undamaged. The contract costs reported for these installations were $2.17 and $3.04 per square yard for the hot-applied and prefabricated membranes, respectively. These costs are believed to be higher than would result under more favorable construction conditions, because of the rainy weather and saturated subgrade during construction. Furthermore, this job was small and only the lining placement was by contract. All the subgrade preparation was performed by project forces, and because of the unstable condition of the subgrade, mechanized equipment could not be used.

Approximately 4,000 square yards of shotcrete were placed over asphalt membrane in a lateral on the Riverton project in April 1951 (fig. 42). Subgrade conditions were good on this job and all the membrane was hot applied. All the work was done by project forces, including the lining placement.

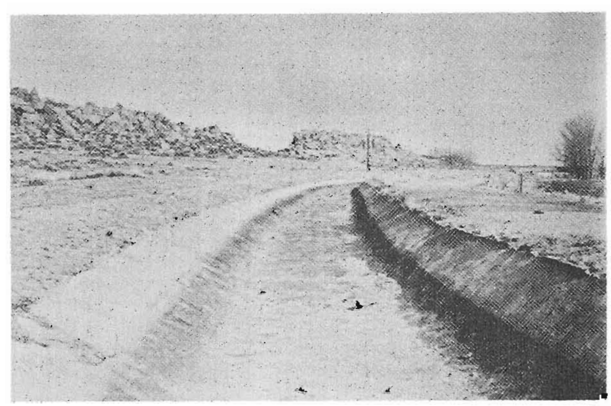


Figure 42.—A general view of a shotcrete-protected hot-asphalt membrane in a serviceable condition, needing only minor repair of a few breaks in the side slopes and bottom, after 5 years of service. IO-1569.

At about the same time that the Riverton lining was under construction, a combination shotcrete and asphalt membrane lining was placed in a reservoir of the Santa Rosa Golf Club, in California. A contractor, who had bid on the Bureau's Orland project installation, proposed a unique type of construction using a prefabricated asphalt membrane. In this construction, instead of fine trimming the subgrade, the surface was filled and smoothed with shotcrete, and shotcrete was placed over the finished membrane. In effect, therefore, the Santa Rosa installation was a "sandwich" lining with shotcrete under and over the asphalt membrane.[12]

(b) *Asphalt Macadam.*—Asphalt macadam, as mentioned in section 24, has also been used experimentally as a protective cover for asphaltic membranes. Like shotcrete, it eliminates the very thick covers of earth and gravel; it also eliminates the need for overexcavation, an important factor where limited right-of-way or rock excavation may be encountered. Tests were made using the macadam as a cover for both the sprayed-in-place type (fig. 43) and prefabricated membranes.

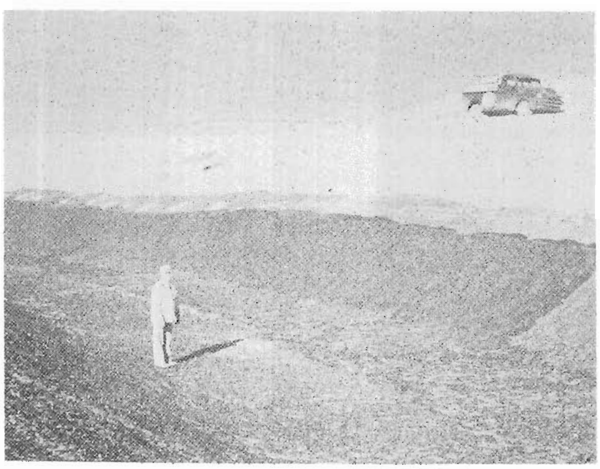


Figure 43.—Completed asphalt macadam cover over an asphaltic membrane lining in the Casper Canal, Kendrick project, Wyoming. CH-293-81, 1951.

The data obtained from the Bureau's test installations indicate that a fairly satisfactory macadam surface can be constructed provided certain procedures are followed. Probably the most important factor is the grading of the aggregate. To secure full penetration and particle bonding, the aggregate must be graded to obtain a relatively open layer. Satisfactory results have been obtained by grading the aggregate from aproximately 95 to 100 percent retained on the ⅜-inch screen (½-inch screen if aggregate is crushed or highly angular) and 100 percent passing the 1- or 1½-inch screen. Aggregate finer than this size range tends to prevent the asphalt from reaching the bottom of a normal 2-inch-thick layer. If thicker layers are used, the minimum-size aggregate must be larger to permit adequate penetration of the asphalt. A grading between coarse and fine within the limits of gradation is advisable to develop stability in the macadam after penetration with asphalt. Aggregate of high angularity will increase stability; but ordinary gravels of relatively low angularity, if well graded between minimum and maximum sizes, will generally develop adequate stability for the purpose. The aggregate should be dry when penetrated, to obtain maximum bond between asphalt and aggregate.

To obtain full asphalt penetration, the asphalt must be applied in a flooding action in one operation. The application bar must be moved slowly, using very hot asphalt, and the bar nozzles must be kept close to the surface. The amount of asphalt required for a 2-inch-thick macadam surface is normally about 2 to 3 gallons per square yard. When used with a membrane, the asphalt requirement including that necessary for the membrane will total about 3½ to 5 gallons per square yard. Contract costs of membrane and macadam surfaces are from $1.75 to $2.50 per square yard. Installations of this type have performed satisfactorily, some since 1952.

41. ***Plastics and Synthetic Rubbers.***—Canal linings of polyvinyl and polyethylene plastics and butyl-coated fabrics have been placed on a limited scale for experimental purposes, as discussed in section 32. Such installations have been made on the Altus project, Oklahoma; the Gila project, Arizona; the Huntley project, Montana; the Boise project, Idaho; the Yakima project, Washington; the Tucumcari project, New Mexico; and the Shoshone project, Wyoming. From reports received and field evaluations made, the plastics installed as buried membranes are performing very satisfactorily. The most recent installations were made on the Altus and Tucumcari projects in the spring of 1961 where both 10-mil-thick polyvinyl and polyethylene plastics were used.

The most promising plastic formulations tested so far appear to be the polyvinyl and poly-

[12] "Gunite-Asphalt-Gunite for Reservoir Lining," Western Construction News, August 1951, vol. 26, No. 8, p. 75.

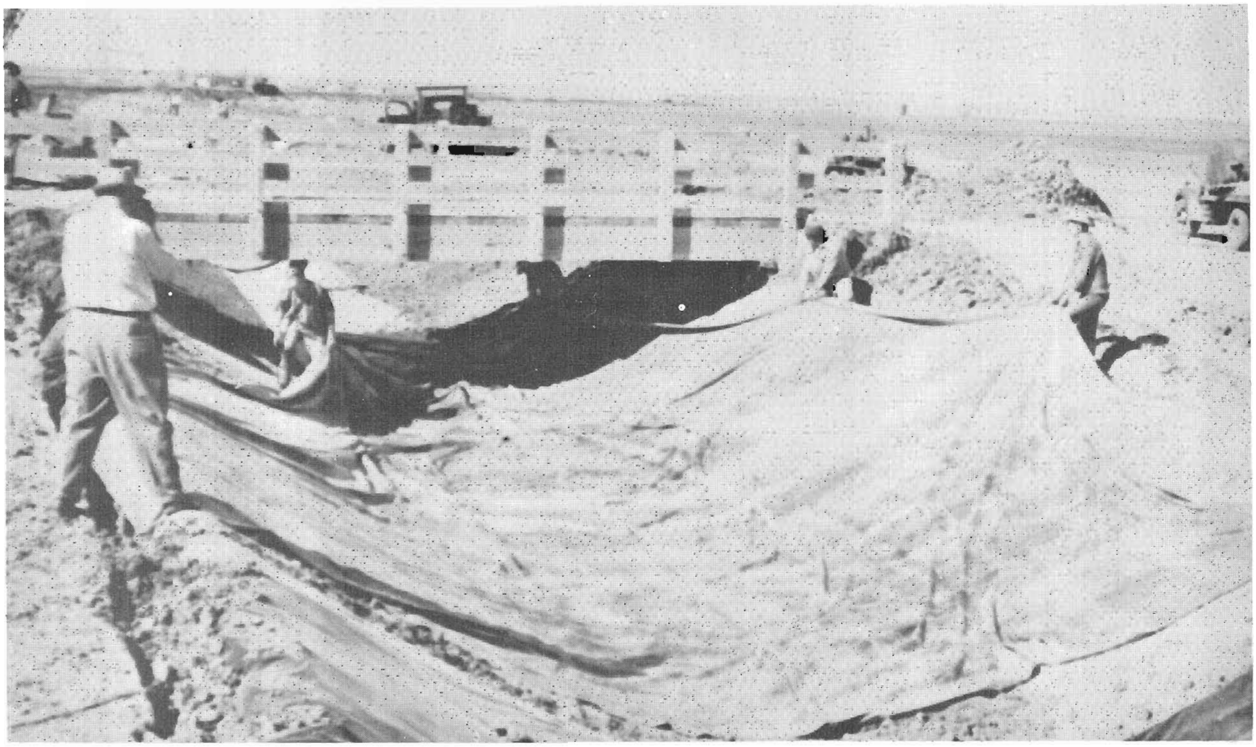


Figure 46.—Unfolding butyl rubber lining material for installation in the Bugg Lateral, Tucumcari project, New Mexico. The membrane was later covered with a protective earth blanket. P257-D-25943.

Figure 47.—Approximately 1,000 feet of 1/32-inch-thick butyl lining in place and ready for cover in the Bugg Lateral, Tucumcari project, New Mexico. P257-D-25961.

laboratories since 1947.[13] The materials are buried in compost in accelerated tests to determine resistance of the plastics to decomposition by bacterial and microorganism attack. The materials are frequently removed from the compost and submitted to various tests, including puncture resistance, tensile strength, and elongation, and their relative durabilities noted.

Some of the materials were pigmented, others were not; some were reinforced during manufacture. The materials varied from 1 mil (0.001 inch) to 25 mils (0.025 inch) in thickness. Plastics and plastic-coated materials tested include those of cellulose acetate butyrate; polyvinyl chloride; and paper, canvas and fiberglass coated with vinyl, polystrene, and polyethylene.

In addition to the tests listed above, samples of the plastics and plastic-coated materials are subjected to natural exposure conditions and simulated field burial conditions by placement in the Bureau's Canal Farm at Denver.

The Agricultural Research Service and the Utah State University, cooperating in the Bureau's lower cost canal lining program, also have done considerable laboratory and field testing of plastic film as a canal and pond lining.[10 14] From these tests and those of the Bureau, and from field observations, it has been found that the films are highly resistant to bacteriological deterioration and that the tensile strength and flexibility of plastics exposed to 10-year composted soil burial were virtually unaffected. Reinforced plastics, where the reinforcement has been of such organic materials as paper, canvas, and burlap, are not desirable because the organic type fabrics are vulnerable to microorganism attack unless completely and carefully saturated with resin.

Plastic film, even of 1½-mil thickness, is essentially a watertight material; however, film less than 6 to 8 mils in thickness has low puncture resistance and would be easily damaged when placed over rough subgrade and covered with angular materials.

[13] Hickey, M. E., "Evaluation of Plastic Films as Canal Lining Materials," Laboratory Report No. B-25, Bureau of Reclamation, July 19, 1957 (Interim Report).

[10] Op. cit. p. 83.

[14] Lauritzen, C. W., "Seepage Control with Plastic Film," Irrigation Engineering and Maintenance, May 1957, pp. 18-19, 32-33.

Ordinary weed growth does not penetrate plastic film of the thickness used in installations made to date, but some of the thinner plastics have been penetrated by the more hardy weeds. Under certain conditions Johnson grass will penetrate 8-mil-thick plastic.

42. ***Bentonite-Membrane Linings.*** — Bentonite, which is an earth material containing a large percentage of sodium type montmorillonite clay, is characterized by high water absorption accompanied by swelling, imperviousness, and slipperiness (low stability). The fact that bentonite does swell and does become impervious on wetting makes it a very useful material in the control of seepage from canals, provided it can be obtained from local deposits at low cost. Bentonite spread as a membrane 1 to 2 inches or more in thickness over the canal subgrade and covered with a 6- to 12-inch protective blanket of stable earth or gravel has been used as a canal lining on some projects for many years.

Bentonite deposits vary greatly in montmorillonite type clay content, being generally accompanied by sand, silt, and clay-sized impurities. Bentonites from various sources, therefore, differ considerably in expansive characteristics, and for this reason some linings of this type have not been entirely successful. For engineering considerations and construction uses, bentonites have been divided into two groups, based primarily on their swelling characteristics: (1) the Wyoming-type bentonite, a high-swelling sodium type montmorillonite clay having a high water absorption capacity, and (2) lower swelling clays, such as calcium montmorillonite, beidellite, and nontronite which although closely related to the Wyoming type have less water absorption capacity because of slight structural and/or compositional differences. Although satisfactory results may be obtained by the use of the low-swelling clays in canal linings, much more material will be required to secure a desired reduction in seepage than would be required with the high-swelling Wyoming type bentonite.

Fine-ground bentonites are the most suitable for membrane lining work. Coarse-ground or pit-run bentonites may be satisfactorily used if a good distribution of particle sizes is obtained, although a greater amount of the material is required for comparable results. Caution should be exercised to insure that the gel-like membrane formed in the presence of moisture will not pipe through very coarse or fissured subgrades.

Specifications for bentonite membrane provide for minimum swelling requirements as determined by laboratory tests. Pit-run bentonite is processed to pass a ¾-inch sieve, and the particles should be reasonably well graded from the finest particles to the maximum size. Bentonite finer than the No. 30 sieve can be used without regard to gradation. Also, a maximum moisture content, such as 20 percent, is specified, with the thickness of the membrane depending on the amount of moisture in the bentonite; the higher the moisture content the thicker the membrane should be to insure adequate solid material. At the present time (1962) an investigation is in progress to improve the requirements for bentonite membrane lining, including possibly establishing requirements for lower-swell types than have been used previously.

In 1940, about 2,600 square yards of bentonite-membrane lining were placed in a canal on the Frenchtown project, Montana. The membrane was only one-half inch thick, with a soil cover 5 inches in thickness. After 7 years of operation, seepage losses from the canal were just as high as they had been before the canal was lined. This apparent failure is believed to have been due to the use of too thin a bentonite membrane, with perhaps an inadequate cover.

Conversely, an example of one of the many satisfactory bentonite-membrane linings is on the Huntley project in Montana. This lining, placed in 1940-41 in a large canal, was comprised of a 2-inch-thick bentonite membrane protected by a 12-inch soil cover. Since the subgrade soil consisted of about 2 feet of sandy loam underlain by relatively clean sand and gravel, excessive seepage losses occured prior to lining. The canal was overexcavated and handleveled. After the pit-run bentonite had been air slaked and dried, it was ground so that all passed a No. 8 sieve, with about 20 percent passing a No. 48 sieve and 5 percent passing a No. 200 sieve. The dried and ground bentonite was placed by hand, as was a 2-inch cover of earth, sand, and gravel. Later, an additional 10 inches of similar cover material was placed over the lined canal section by dragline and bulldozer. This section of lining is entirely satisfactory, and was just as effective in reducing seepage when examined in 1958 as it was when first constructed in 1940 and 1941. In fact, basements of homes in a town ¼ to 1 mile distant which had been flooded during irrigation seasons prior to the installation of lining, subsequently became dry and have remained dry.

Average costs for the 2-inch bentonite membrane placed by project forces on the Huntley project in 1940 and 1941 were slightly over 30 cents per square yard. The cost of 157,628 square yards of pit-run bentonite-membrane lining (1½ to 2 inches thick) placed on the Angostura unit, Missouri River Basin project, South Dakota, by contract in 1954, 1955, and 1956 averaged about $1.24 per square yard.

Earth Linings

43. ***Types.***—Included in the category of earth linings are those composed of compacted earth, loosely placed earth, clay or bentonite-soil mixtures, and soils to which admixtures have been added. The stabilization of soils by physical means or by the use of admixtures, or additives, also is discussed in this section except for soil-cement which is discussed in section 35.

Compacted-earth linings may be the thick type, normally having a 3- to 8-foot thickness measured horizontally on the side slopes and a 12- to 24-inch thickness in the bottom; or the thin type, consisting of a 6- to 12-inch layer of compacted cohesive soil on the slopes and bottom, preferably protected with 6 to 12 inches of coarser soil or gravel.

Loosely placed earth lining generally consists of an earth blanket of selected fine-grained soils placed on the sides and bottom of the canal and spread to a thickness of up to 12 inches.

Clay-soil mixtures refer to gravelly or sandy soil and clay thoroughly mixed and blended. Linings of this type are usually compacted. Bentonite or bentonitic type clays have been used in this way in relatively thin layers varying with local conditions. Expansive clays, otherwise, are not recommended for earth linings.

Resins, chemicals, asphalts, and petrochemicals have been used in the stabilization of soils, as have portland cement and lime. Some soils appear to be well adapted to special methods of chemical stabilization; but the use of chemically stabilized soils for canal linings is now only in the experimental stage. Several experimental reaches, both small and large, of cement- and asphalt-stabilized soil have been constructed and are being observed for serviceability.

Canal linings of natural or processed soils often prove economical for the reduction of seepage and stabilization of sections, if suitable materials are available from the canal excavation or from nearby borrow areas. Cohesive soils of a wide variety may be employed in the construction of either thin or thick compacted-earth linings. Some fine-grained soils placed loosely over the canal subgrade have reduced seepage to satisfactory amounts.

44. ***General Design Considerations.***—The bottom width of an earth-lined canal is usually about three times the depth for small laterals and up to about eight times the depth for large canals. Side slopes in earth canals are 1½ to 1, or flatter, depending on the size of canal and materials available for lining as well as the type of lining to be used. The permissible velocities also vary with the type of lining material and usually range from 1 to 4 feet per second. A Manning's roughness coefficient, n, of 0.025 is used for canals with capacities less than 100 second-feet, and 0.0225 or 0.020 for larger canals.

The minimum freeboards and typical sections for compacted-earth linings are shown on figures 7 and 48, respectively. The bank material on the water face above the water level should be selected to provide adequate impermeability and erosion resistance in case the canal is operated above design capacity. The normal (minimum) bank height is the same as for unlined canals. In canals with a capacity of 1,000 second-feet or larger, the lining freeboard should be selected to meet site conditions and will usually be one-half of the total freeboard.

A thick compacted-earth lining is usually one of the lowest cost permanent type linings where suitable materials are available near the jobsite. It also can withstand considerable external hydrostatic pressure (uplift) without loss of effectiveness, so underdrains are less frequently required. The need for gravel or riprap protection to prevent erosion of the lining should not be overlooked. Linings constructed of silty and sandy materials with little coarse gravel are very susceptible to scour. If these are to be used, the cost of reducing the velocity by use of a larger section, as compared with the cost of maintaining a smaller section with its higher velocity and protecting the lining with a gravel cover, should be included in the evaluation. The criteria set forth in table 10 will assist in selecting materials for linings and gravel protection if needed.

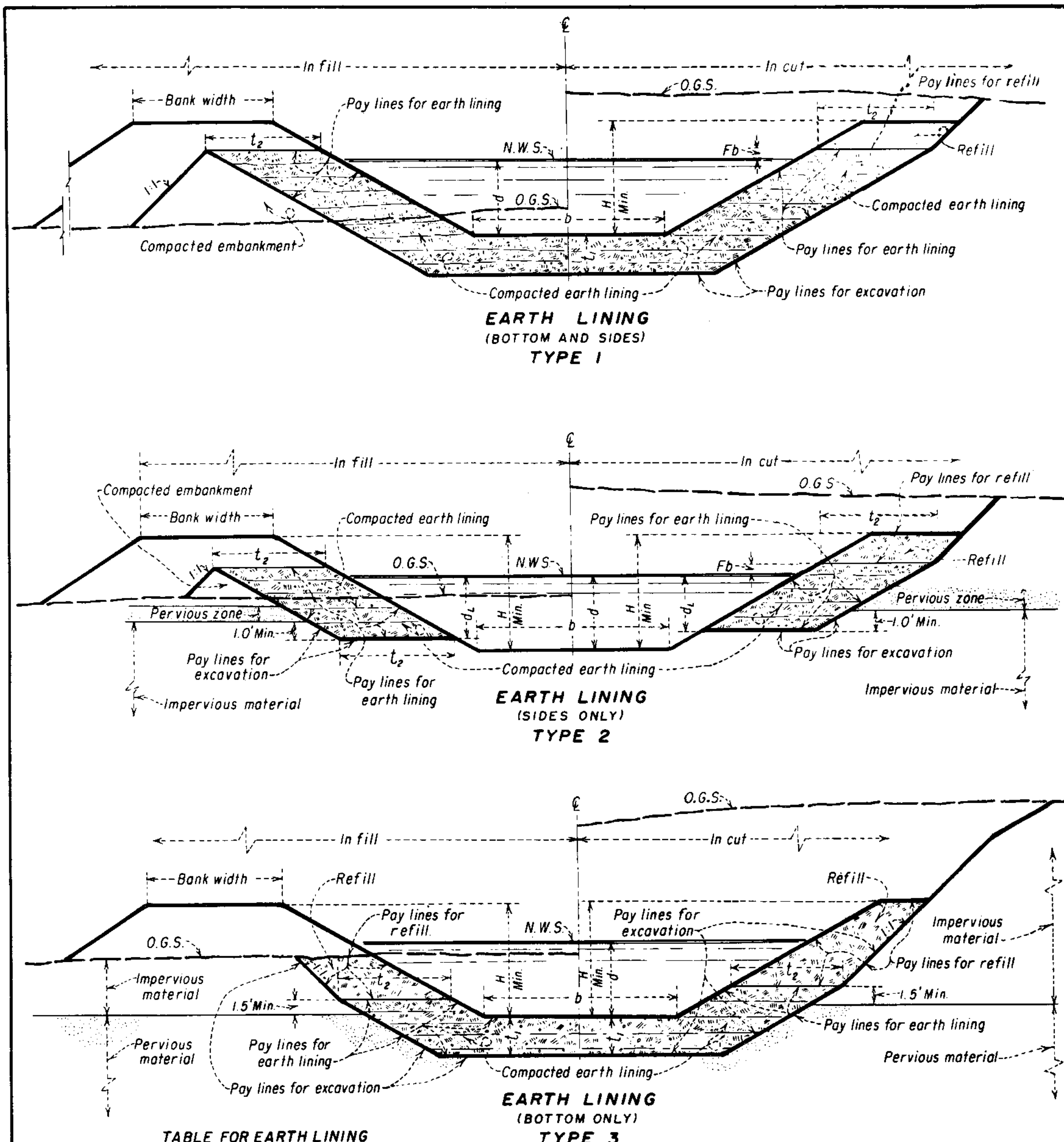


TABLE FOR EARTH LINING

d or d_L	t_1	t_2
2.0' Max.	1.0'	3.0'
4.0' Max.	1.5'	4.0'
6.0' Max.	2.0'	6.0'
Over 6.0'	2.0'	8.0'

NOTES

If lining material requires a protective cover of gravel or riprap to prevent scour or erosion, excavation pay lines shall be extended to provide for the designated thickness of lining plus the gravel or riprap cover.

If a pervious zone containing mostly coarse material is encountered and requires a graded filter to prevent loss of lining material, excavation pay lines shall be lowered on sides or bottom to provide for the designated thickness of lining plus the filter.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

TYPICAL EARTH LINED SECTIONS

DRAWN D.E.C. SUBMITTED
TRACED H.M.O. RECOMMENDED
CHECKED M.K.B. APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO. 101-D-224

Figure 48.—Typical compacted-earth-lined canal sections.

TABLE 10.—*Important physical properties of soils and their uses for canal linings*
(Identifications based on Unified Soil Classification System)

MAJOR DIVISIONS OF SOILS			TYPICAL NAMES OF SOIL GROUPS	GROUP SYMBOLS	SOIL PROPERTIES: PERMEABILITY	SOIL PROPERTIES: SHEARING STRENGTH	SOIL PROPERTIES: COMPACTED DENSITY	SUITABILITY FOR CANALS: EROSION RESISTANCE	SUITABILITY FOR CANALS: COMPACTED EARTH LININGS
COARSE-GRAINED SOILS More than half of material is larger than No. 200 sieve size	GRAVELS More than half of coarse fraction is larger than No. 4 sieve size	CLEAN GRAVELS (Little or no fines)	Well-graded gravels, gravel-sand mixtures, little or no fines	GW	14	16	15	2	—
			Poorly graded gravels, gravel-sand mixtures, little or no fines	GP	16	14	8	3	—
		GRAVELS WITH FINES (Appreciable amount of fines)	Silty gravels, poorly graded gravel-sand-silt mixtures	GM	12	10	12	5	6
			Clayey gravels, poorly graded gravel-sand-clay mixtures	GC	6	8	11	4	2
			Gravel with sand-clay binder	GW-GC	8	13	16	1	1
	SANDS More than half of coarse fraction is smaller than No. 4 sieve size	CLEAN SANDS (Little or no fines)	Well-graded sands, gravelly sands, little or no fines	SW	13	15	13	8	—
			Poorly graded sands, gravelly sands, little or no fines	SP	15	11	7	9 coarse	—
		SANDS WITH FINES (Appreciable amount of fines)	Silty sands, poorly graded sand-silt mixtures	SM	11	9	10	10 coarse	7 Erosion Critical
			Clayey sands, poorly graded sand-clay mixtures	SC	5	7	9	7	4
			Sand with clay binder	SW-SC	7	12	14	6	3
	(For visual classifications, the 1/4" size may be used as equivalent to the No. 4 sieve size)								
(The No. 200 sieve size is about the smallest particle visible to the naked eye)									
FINE-GRAINED SOILS More than half of material is smaller than No. 200 sieve size	SILTS AND CLAYS Liquid limit less than 50		Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	ML	10	5	5	—	8 Erosion Critical
			Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays	CL	3	6	6	11	5
			Organic silts and organic silt-clays of low plasticity	OL	4	2	3	—	9 Erosion Critical
	SILTS AND CLAYS Liquid limit greater than 50		Inorganic silt, micaceous or diatomaceous fine sandy or silty soils, elastic silts	MH	9	3	2	—	—
			Inorganic clays of high plasticity, fat clays	CH	1	4	4	12	10 Volume Change Critical
			Organic clays of medium to high plasticity	OH	2	1	1	—	—
HIGHLY ORGANIC SOILS			Peat and other highly organic soils	Pt	*			**	

* Numbers above indicate the order of increasing values for the physical property named
** Numbers above indicate relative suitability (1 = best)

LCCL-T10

If highly plastic soils are to be used for lining, it is advisable to flatten the slopes to 2 to 1 or flatter because of loss in stability when these soils become saturated. Also, the surface of plastic soils, if of the expansive type, may be easily eroded and the lining become lost unless protected by gravel or by maintaining a low water velocity.

Subgrade treatment for earth linings is quite variable. It may include using subgrade soils as part of the lining; or it may require overexcavation to place a protective layer between the subgrade and the lining. Some fine-grained soils such as loess can frequently be used for linings if they are worked and compacted. In these materials the bottom horizontal layer is not excavated, but is merely plowed and compacted. The sides are roughly trimmed to receive the compacted lining. In sands and sandy gravel no subgrade treatment is required because the lining materials will not pipe into the subgrade. However, if the subgrade contains open voids, as may occur in gravel or fractured rock, it may be necessary to overexcavate and place a sandy gravel filter layer before placing the lining. Particular caution should be exercised to prevent piping if fine-grained cohesionless materials are used for linings. Silty subgrade soils which are dry and of low density are subject to subsidence and the development of cavities when wetted. This may cause piping and settlement of embankment and structures.

If an underdrain system is required because of existing or expected high ground water, the underdrains should empty into the canal through flap valves or into some natural outlet. The underdrain filter material should protect the lining materials as well as the subgrade soils from piping.

The compaction requirement discussed in section 45 applies to all of the lining, but Bureau specifications provide that test samples will not be taken in the outer 2 feet, measured horizontally, of the exposed sloping surfaces of thick compacted-earth linings. This outer layer is difficult to compact to full density requirements without overbuilding and then trimming. Since weathering will probably reduce density in the surface, the cost of initially obtaining this surface compaction is not warranted. The surface of the lining should be trimmed and dragged to provide an even surface in contact with the water.

45. *Thick Compacted-Earth Linings.*—This type of lining has proved to be generally more satisfactory than other types of earth linings and has been used extensively in Bureau work. The thick linings (fig. 49) are constructed of selected impervious soils, both the bottom and side slopes being compacted in successive horizontal layers not more than 6 inches thick after compaction. The usual 3- to 8-foot-wide successive layers on the side slopes can be overbuilt as necessary to accommodate conventional, large earth moving and compaction equipment, then trimmed to required lines. Thus, the actual thickness of the lining is usually about 2 to 3 feet measured normal to the slope. Bottom linings are commonly 1 to 2 feet thick, but vary with the requirements of the job.

Relatively low construction costs are possible with a thick lining if the job is large enough to warrant the use of heavy earth moving equipment and if a suitable soil is available in sufficient quantity without excessive hauling. Properly constructed thick compacted-earth linings have been found by field tests to be highly impermeable (table 4, sec. 17), with losses in the order of 0.07 cfd.

(a) *Soil Suitability.*—Table 10 shows the various soil types, their important physical properties, and their suitability for compacted-earth linings. In general, soils best suited for use in thick compacted-earth linings are gravels and sands with clay binder and poorly graded gravel-sand-clay mixtures; these are preferred because of their low permeability, high stability, and good resistance to erosion. It is often economical to mix coarse subgrade soils with fine soils from borrow to make an impervious, stable, blended lining. The silt or clay that is added to the coarse excavated soils should exceed the percentage determined by laboratory testing to compensate for the less thorough mixing obtainable in the field. The excavated material is blended with the borrow material in layers and compacted as the lining is placed. Thorough mixing is imperative.

Prior to the use of available soils for lining their maximum density, optimum moisture, and permeability should be determined in the labor-

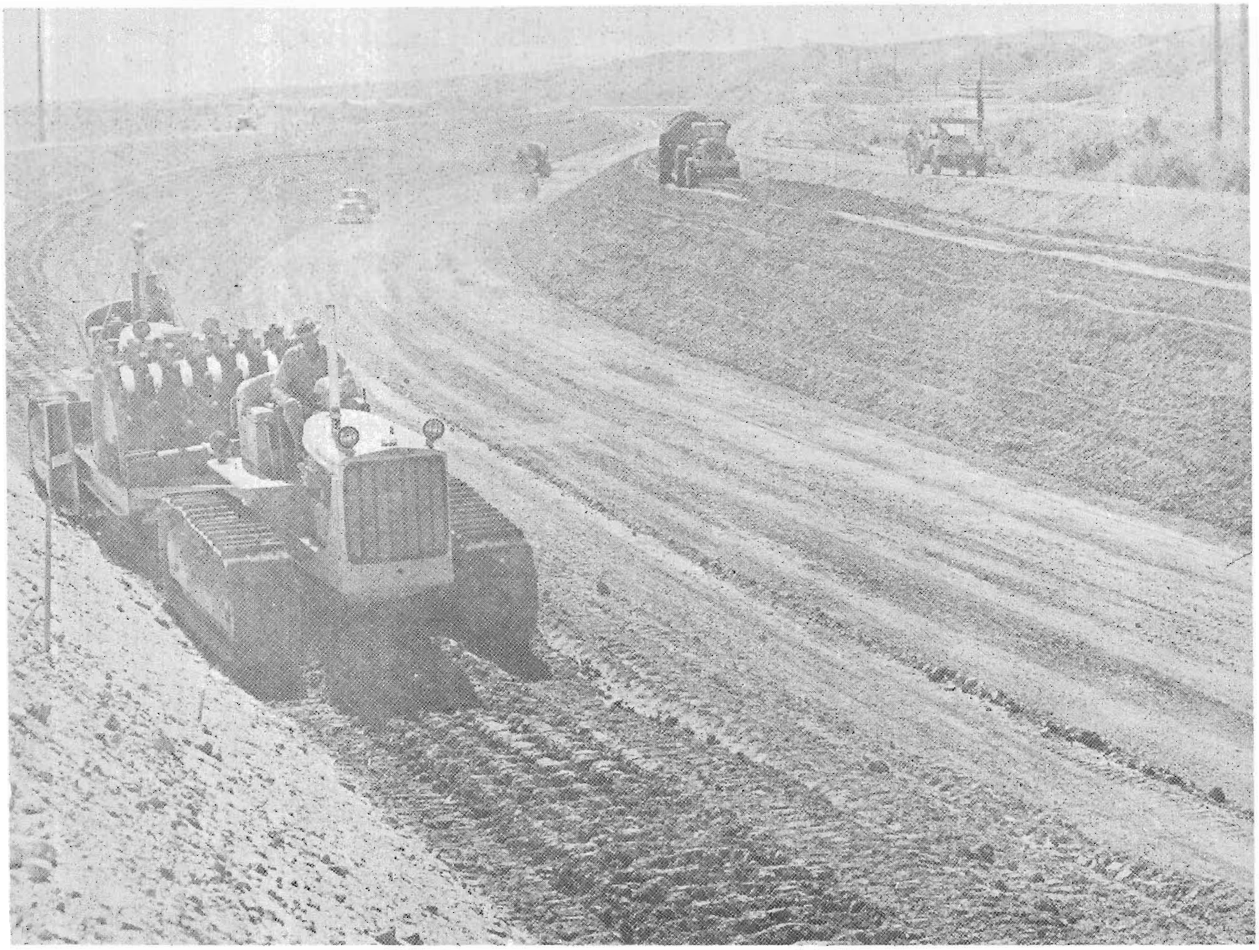


Figure 49.—Installation of thick compacted-earth lining in a canal on the Gila project, Arizona. P50-303-148.

atory. Construction specifications should require a minimum inplace density sufficiently high to provide some excess over apparent permeability requirements. This construction requirement, which is usually 95 percent of Proctor laboratory maximum density, is established from a knowledge of the soil characteristics and the construction practices and equipment. However, the requirement may be based on actual preliminary field tests of the soil as compacted by the equipment that is to be used in the lining construction. Soils that are borderline from a permeability standpoint often may be satisfactory if compacted to higher densities. With the stable side slopes normally used, linings can be placed with moisture content somewhat greater than optimum. Laboratory tests have demonstrated that the permeability of soil generally decreases with increase in placement moisture. Compaction is best accomplished with sheepsfoot rollers.

(b) *Construction Cost.*—The most important factors influencing the unit cost of thick compacted-earth linings are size of the job, source of materials, weather conditions, mixing requirements, subgrade preparation, and cover materials. Of these, only the first three need comment.

A job involving the placement of large quantities of lining in large canals permits the effective use of heavy equipment. Hence, the unit cost of material handling is reduced and the inplace cost per square yard of lining is relatively low.

The source of materials may be a controlling factor influencing unit cost because of the cost

TABLE 11.—*Some representative costs of compacted-earth canal linings based on contract prices and specifications quantities*

Specifications No.	Month and year	Feature and project	Capacity, second-feet	Canal section: Base width, feet	Canal section: Water depth, feet	Canal section: Side slope	Description of lining	Quantity, cubic yards	Cost of lining, dollars per square yard[1]
4604	February 1956	Upper Meeker Canal Frenchman-Cambridge division, MRBP	284	16	5.03	$1\frac{1}{2}$ to 1	Bottom, 24 inches thick Sides, 6 feet thick (horizontal)	5,500	0.84
4934 sched. III	July 1957	Driftwood Canal, Frenchman-Cambridge division, MRBP					Bottom, 24 inches thick Sides, 6 feet thick (horizontal)		
		Section 4	225	16	4.59	2 to 1		37,000	0.62
		Section 5	166	14	3.84	2 to 1		14,000	0.63
		Section 6	125	12	3.46	2 to 1		6,000	0.64
		Section 7	106	12	3.09	2 to 1		14,000	0.65
		Section 8	90	10	3.23	2 to 1		13,000	0.67
4938 sched. I	August 1957	Helena Valley Canal, Helena Valley unit, MRBP					Bottom, 24 inches thick Sides, 6 feet thick (horizontal)		
		Section 2L	350	16	5.64	2 to 1		95,200	1.73
		Section 3L	225	12	4.79	2 to 1		15,900	1.78
		Section 4L	170	10	4.44	2 to 1		18,300	1.82
4999	January 1958	Osborne Canal, Solomon division, MRBP	161	12	4.2	2 to 1	Bottom, 24 inches thick Sides, 6 feet thick (horizontal)	51,000	0.55
*5089	September 1958	Culbertson Canal, Frenchman-Cambridge division, MRBP	400	20	6.18	2 to 1	Bottom, 24 inches thick Sides, 8 feet thick (horizontal)	30,100	0.93
5113	October 1958	White Rock Canal, Bostwick division, MRBP Section 2A	64	8	2.7	2 to 1	Bottom, 24 inches thick Sides, 8 feet thick (horizontal)	500	1.22
5129	January 1959	Osborne Canal, Solomon division, MRBP							
		Section 2	140	12	3.77	$1\frac{1}{2}$ to 1	Bottom, 18 inches thick Sides, 6 feet thick (horizontal)	14,500	0.62
		Section 3	140	12	3.98	$1\frac{1}{2}$ to 1	Bottom, 18 inches thick Sides, 6 feet thick (horizontal)	2,500	0.62
5148	March 1959	Culbertson Canal, Frenchman-Cambridge division, MRBP	380	20	5.95	2 to 1	Bottom, 24 inches thick Sides, 8 feet thick (horizontal)	4,600	0.80
5155	March 1959	Southside Canal, Collbran project							
		Section 1-B	240	14	4.42	2 to 1	Bottom, 24 inches thick Sides, 6 feet thick (horizontal)	37,500	1.64
		Section 2	225	14	4.27	2 to 1	Bottom, 24 inches thick Sides, 6 feet thick (horizontal)	17,400	1.37
		Section 3	130	10	3.40	2 to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	2,100	1.07
5166	April 1959	Helena Valley unit, MRBP							
		North Side Laterals							
		Section L636-1.91	25	6	1.91	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	3,000	1.01
		Section L432-1.63	15	4	1.63	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	1,800	1.08
		East Side Laterals							
		Section L844-2.5	60	8	2.51	$1\frac{1}{2}$ to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	21,200	1.17
		Section L840-2.2	40	8	2.17	$1\frac{1}{2}$ to 1		4,200	1.17
		Section L644-2.8	45	6	2.75	$1\frac{1}{2}$ to 1		8,700	1.21
		Section L642-2.6	40	6	2.59	$1\frac{1}{2}$ to 1		8,000	1.22
		Section L836-2.0	40	8	1.95	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	5,300	0.80
		Section L636-1.9	30	6	1.89	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	4,400	0.84
		Section L530-1.7	20	5	1.73	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	4,200	0.86
5198	June 1959	South Fork Collection Canal, Talent division, Rogue River Basin project							
		Section 2	65	6	3.4	2 to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	6,700	2.57
		Section 4	25	4	2.0	2 to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	1,900	1.71
5218	August 1959	Southside Canal, Collbran project	94	12	2.57	2 to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	600	1.29

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TABLE 11.—*Some representative costs of compacted-earth canal linings based on contract prices and specifications quantities*—Continued

Specifications No.	Month and year	Feature and project	Capacity, second-feet	Canal section: Base width, feet	Canal section: Water depth, feet	Canal section: Side slope	Description of lining	Quantity, cubic yards	Cost of lining, dollars per square yard[1]
5221	September 1959	Culberton Extension Canal, Frenchman-Cambridge division, MRBP					Bottom,24 inches thick		
		Section 1	255	18	4.6	2 to 1	Sides,6 feet thick(horizontal)	500	0.61
		Section 2	248	16	4.8	2 to 1		850	0.62
		Section 3	245	16	4.7	2 to 1		3,000	0.62
		Section 5	195	14	4.25	2 to 1		2,600	0.63
5227	September 1959	White Rock Extension Canal Bostwick division, MRBP					Bottom,24 inches thick		
		Section 3	60	8	2.8	2 to 1	Sides,4 feet thick(horizontal)	2,500	0.60
5242	November 1959	Osborne Canal, Solomon division, MRBP					Bottom,12 inches thick		
		Section 8	60	10	2.6	1½ to 1	Sides,4 feet thick(horiontal)	1,000	0.70
5248	November 1959	Sump 2, contract unit 2, Tule Lake division, Klamath project							
		Q-Canal					Bottom,18 inches thick		
		Station 4+48.9 to 57+60.1	130	10	5.1	2 to 1	Sides,6 feet thick(horizontal)	21,000	0.74
		Station 58+47.1 to 88+63.6	105	9	4.8	2 to 1	Bottom,18 inches thick Sides,6 feet thick(horizontal)	11,300	0.75
		Station 88+63.6 to 133+80	90	8	4.6	2 to 1	Bottom,18 inches thick Sides,6 feet thick(horizontal)	16,200	0.77
		Station 133+80 to 169+04	35	6	3.4	2 to 1	Bottom,6 inches thick Sides,4 feet thick(horizontal)	5,300	0.45
		Station 169+04 to 182+04	25	4	3.0	2 to 1	Bottom,6 inches thick Sides,4 feet thick(horiontal)	1,800	0.47
		Station 182+04 to end	20	4	2.8	2 to 1	Bottom,6 inches thick Sides,4 feet thick(horizontal)	3,200	0.47
		Q-1 Lateral					Bottom,6 inches thick		
		Station 0+00 to 24+87.1	35	4	3.4	2 to 1	Sides,4 feet thick(horizontal)	3,600	0.47
		Station 25+13.6 to 50+34.9	25	4	3.2	2 to 1		3,600	0.47
		Station 50+61.4 to end	20	4	3.0	2 to 1		5,200	0.47
		Q-2 Lateral							
		Station 0+48.84 to 29+90	30	4	3.4	2 to 1		4,500	0.47
		Station 30+16.5 to 56+30	25	4	2.6	2 to 1		3,300	0.47
		Station 56+56.5 to end	20	4	2.8	2 to 1		5,000	0.47
5268	February 1960	Culbertson Extension Canal Frenchman-Cambridge division, MRBP					Bottom,18 inches thick		
		Section 6	140	12	3.8	1½ to 1	Sides,4 feet thick(horizontal)	2,600	0.58
		Section 7	120	12	3.4	1½ to 1	Bottom,18 inches thick Sides,4 feet thick(horizontal)	1,700	0.58
		Section 9	80	9	3.0	1½ to 1	Bottom,18 inches thick Sides,4 feet thick(horizontal)	2,200	0.61
5279	March 1960	West Canal Laterals Columbia Basin project					Bottom,18 inches thick		
		W61 - Section 844-2.7	81	8	2.7	1¾ to 1	Sides,4 feet thick(horizontal)	9,000	0.65
		Section 644-2.8	64	6	2.8	1¾ to 1		4,400	0.67
		Section 644-2.6	64	6	2.6	1¾ to 1		8,400	0.68
		Section 640-2.4	63	6	2.4	1¾ to 1		7,800	0.68
							Bottom,12 inches thick		
		Section 540-1.7	50	5	1.7	1¾ to 1	Sides,3 feet thick(horizontal)	3,600	0.48
		Section 432-1.5	52	4	1.5	1¾ to 1		4,800	0.50
		Section 332-1.1	55	3	1.1	1¾ to 1		800	0.53
		Section 332-1.0	60	3	1.0	1¾ to 1		400	0.53
		W61J-Section 636-2.0	52	6	2.0	1¾ to 1		1,400	0.47
		Section 536-2.0	52	5	2.0	1¾ to 1		2,500	0.48
		Section 536-1.8	51	5	1.8	1¾ to 1		1,900	0.48
							Bottom,18 inches thick		
		W78.8-Section 844-2.7	63	8	2.7	1¾ to 1	Sides,4 feet thick(horizontal)	1,000	0.65
5293	April 1960	Leon-Park Feeder Canal, Collbran project					Bottom,24 inches thick		
		Section 1	350	12	5.45	2 to 1	Sides,6 feet thick(horizontal)	4,100	1.30
		Section 2	150	10	3.65	2 to 1	Bottom,18 inches thick Sides,4 feet thick(horizontal)	400	0.88
5344	June 1960	Main Canal, Hammond project					Bottom,18 inches thick		
		Section 2	90	10	3.2	2 to 1	Sides,4 feet thick(horizontal)	3,800	1.71
5358	August 1960	R-1-a Lateral, sump 2 Tule Lake division, Klamath project	20	4	2.7	2 to 1	Bottom,6 inches thick Sides,4 feet thick(horizontal)	980	1.83

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TABLE 11.—*Some representative costs of compacted-earth canal linings based on contract prices and specifications quantities*—Continued

Specifications No.	Month and year	Feature and project	Capacity, second-feet	Canal section			Description of lining	Quantity, cubic yards	Cost of lining, dollars per square yard[1]
				Base width, feet	Water depth, feet	Side slope			
5359	August 1960	Distribution Canal, Crooked River project, Section D	102	8	3.6	2 to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	6,200	0.86
5368	August 1960	Sherman Feeder Canal, Middle Loup division, MRBP	850	28	8.5	2 to 1	Bottom, 24 inches thick, Sides, 8 feet thick (horizontal)	326,000	0.46
5389	September 1960	Sump 3, contract unit 1 Tule Lake division, Klamath project							
		N-1 Lateral							
		Station 0+00 to 22+92	51	5	3.9	2 to 1	Bottom, 6 inches thick Sides, 4 feet thick (horiontal)	3,800	0.61
		Station 22+92 to 75+74	43	4	3.8	2 to 1		8,500	0.63
		Station 76+00.5 to 102+00	25	4	3.0	2 to 1		3,500	0.63
		Station 102+00 to 109+90	20	4	2.7	2 to 1		1,000	0.63
		N-4 Lateral							
		Station 0+97 to 17+65	30	4	3.2	2 to 1		2,400	0.63
		Station 17+65 to 58+50	25	4	3.0	2 to 1		5,400	0.63
		Station 58+50 to 91+45	20	4	2.7	2 to 1		4,100	0.63
		N-6 Lateral							
		Station 1+50 to 26+75	31	4	2.8	2 to 1		3,300	0.63
		Station 27+01.5 to 66+32	20	4	2.7	2 to 1		4,900	0.63
		N-8 Lateral							
		Station 0+65 to 9+70	25	4	3.0	2 to 1		1,200	0.63
		Station 9+70 to 49+15	20	4	2.7	2 to 1		4,900	0.63
5437	December 1960	Cedar Bluff Canal Smoky Hills division, MRBP Section 1	125	12	4.1	2 to 1	Bottom, 24 inches thick Sides, 6 feet thick (horizontal)	53,600	0.58
5444	January 1961	Madera distribution system extension, part 1, CVP							
		Section 4	15	6	1.79	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	3,900	0.64
		Section 9	15	6	1.48	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	5,400	0.64
		Section 5	10	6	1.43	$1\frac{1}{2}$ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	2,200	0.64
5467	February 1961	Farwell Main Canal Middle Loup division, MRBP							
		Section 2	630	26	7.5	2 to 1	Bottom, 24 inches thick Sides, 8 feet thick (horizontal)	6,100	0.77
		Section 4	430	22	6.32	2 to 1	Bottom, 24 inches thick Sides, 8 feet thick (horizontal)	2,600	0.79
5470	February 1961	Sherman Feeder Canal Middle Loup division, MRBP	850	28	8.5	2 to 1	Bottom, 24 inches thick Sides, 8 feet thick (horizontal)	7,000	1.44
5483	February 1961	Sump 2, contract unit 3, Tule Lake division, Klamath project							
		Q-3 Lateral							
		Station 1+91.3 to 28+24.8	63	8	3.8	2 to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	4,800	0.44
		Station 28+24.8 to 57+30	50	8	3.4	2 to 1		4,900	0.44
		Station 57+30 to 112+17	36	4	3.5	2 to 1		8,500	0.46
		Station 112+17 to 142+07	20	4	2.7	2 to 1		4,000	0.47
		Q-3-a Lateral							
		Station 0+40 to 25+30	30	4	3.2	2 to 1		3,600	0.46
		Station 25+30 to 51+20	25	4	3.0	2 to 1		3,600	0.47
		Station 51+20 to 77+20	20	4	2.7	2 to 1		3,400	0.47
		Q-3-b Lateral							
		Station 0+40 to 23+05	30	4	3.2	2 to 1		3,300	0.46
		Station 23+05 to 47+35	25	4	3.0	2 to 1		3,400	0.47
		Station 47+35 to 85+00	20	4	2.7	2 to 1		5,100	0.47
		Q-3-c Lateral							
		Station 0+00 to 36+42.9	20	4	2.7	2 to 1		4,800	0.47
		R-2 Lateral							
		Station 0+50 to 26+25	25	4	3.0	2 to 1		3,600	0.47
		Station 26+25 to 56+40	20	4	2.7	2 to 1		4,000	0.47
5516	March 1961	Cedar Bluff Canal Smokey Hills division, MRBP Section 1	125	12	4.1	2 to 1	Bottom, 24 inches thick Sides, 6 feet thick (horizontal)	19,400	0.66

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TABLE 11.—*Some representative costs of compacted-earth canal linings based on contract prices and specifications quantities*—Continued

Specifications No.	Month and year	Feature and project	Capacity, second-feet	Canal section: Base width, feet	Canal section: Water depth, feet	Canal section: Side slope	Description of lining	Quantity, cubic yards	Cost of lining, dollars per square yard[1]
5588	June 1961	Main Canal, Hammond project							
		Section 3	55	10	2.47	2 to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	3,300	0.61
		Section 4	55	7	2.83	2 to 1		3,600	0.62
		Section 5	45	7	2.70	1½ to 1		26,700	0.73
		Section 6	35	6	2.41	1½ to 1		8,100	0.75
		Section 7	25	6	1.89	1½ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	10,400	0.52
		Section 8	15	4	1.58	1½ to 1		11,700	0.56
		Section 9	10	4	1.33	1½ to 1		3,400	0.56
		Section 11	10	4	1.30	1½ to 1		400	0.56
		Section 13	10	4	1.36	1½ to 1		2,800	0.56
		Section 14	3	3	0.79	1½ to 1		300	0.61
		Section 15	12	4	1.49	1½ to 1		4,300	0.56
5606	June 1961	Madera distribution system extension, part 2, CVP							
		Section 1	60	8	3.31	1½ to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	27,900	1.42
		Section 2	45	8	2.85	1½ to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	22,800	1.43
		Section 3	30	6	2.58	1½ to 1	Bottom, 18 inches thick Sides, 4 feet thick (horizontal)	7,700	1.50
		Section 4	15	6	1.79	1½ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	2,200	1.03
		Section 5	10	6	1.45	1½ to 1	Bottom, 12 inches thick Sides, 3 feet thick (horizontal)	2,100	1.03

[1] Includes cost of excavation for lining

MRBP = Missouri River Basin project

CVP = Central Valley project

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of excavation or haul. The least expensive linings will be those for which materials removed in the required canal excavation can be used in the lining. Compare the cost of the installations under specifications No. 4934 and 4938, as listed in table 11. Material for the latter installation involved a haul of several miles while material for the former was obtained from canal excavation.

In addition to equipment operation problems, weather conditions influence cost through the necessity of placing lining materials with the proper moisture content. If the soil is too dry, water must be added by some method to obtain fairly uniform distribution for compaction. This is usually done by sprinkling in borrow pits with supplemental sprinkling during placement. If too wet, the soils must be spread to dry or the work delayed. This factor of cost is obviously quite variable, but can frequently be evaluated for a particular project when the contract period is known.

Although thick compacted-earth linings originally appeared more suitable from a cost standpoint for large canals, they have also been found economical for medium and small-sized canals. Narrower linings on the side slopes are built up with the same successive thin layers but only 3 to 4 feet in horizontal width. Compaction in this case is accomplished with a single-drum sheepsfoot roller pulled by a farm type tractor, or by overbuilding inward to permit the use of larger equipment and removing the excess width. Another method that has been used is to build up and compact solid sections of fill, then excavate the center with plow type or other ditchers to form the canal prism.

Thick earth linings that do not meet full compaction or thickness requirements, but which have successfully controlled seepage, have been economically placed by operation and maintenance forces. In these installations, the section was overexcavated from 12 to 24 inches, usually greater at the bottom, and refilled with selected clay type soils. A moderate degree of compaction was obtained by equipment travel in placing the layers. The reported cost of these installations ranged between 20 and 40 cents per square yard. The success and economy of this type of lining depends on a supply of material which is inherently impermeable at low density, and is obtainable in the immediate vicinity.

46. ***Thin Compacted-Earth Linings.***—Thin compacted-earth linings ordinarily consist of a 6- to 12-inch layer of cohesive soil thoroughly compacted and often protected by a 6- to 12-inch cover of coarse soil or gravel. However, cover and lining thickness requirements vary with the type of soil used, the velocity of the water to be conveyed, and other job conditions. For instance, clayey gravels might well be used with no cover if erosive forces are low, but a cover to resist erosion should be considered for silty soils of very low plasticity.

Soil types generally suitable for use in compacted-earth linings are briefly described in table 10. Those suitable for thin compacted-earth linings include: (1) gravel with sand-clay binder (GW-GC), (2) clayey gravels (GC), (3) sand with clay binder (SW-SC), (4) clayey sands (SC), (5) lean clays (CL), and (6) fat clays (CH). Fat clays may not be suitable for canals which are subject to wetting and drying because of swelling and shrinking, unless the lining is protected by a gravel-sand cover. Further description, physical properties, and uses of these soils groups are contained in the Bureau's Earth Manual.[1]

Prior to use of available soil for thin compacted-earth linings, the maximum density, optimum moisture, and permeability should be determined in the laboratory.

Compaction of thin earth lining is best accomplished with sheepsfoot rollers with a final rolling by smooth rollers, but other equipment has also been satisfactorily used. One method is to operate equipment along the berm (fig. 50).

[1] "Earth Manual," first edition, Bureau of Reclamation, 1960.

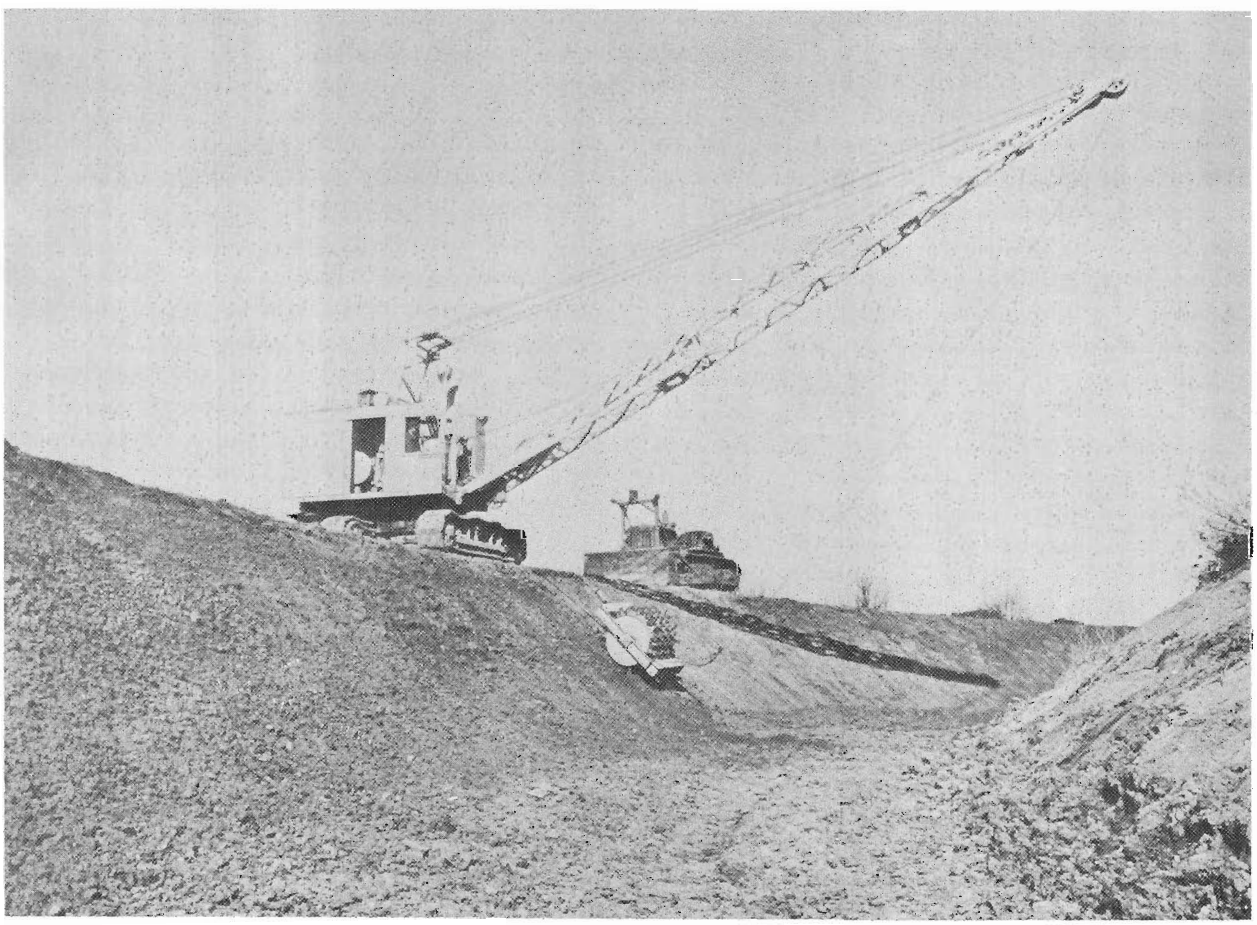


Figure 50.—Transverse compaction of thin earth lining (two 6-inch layers) by use of a single-drum sheepsfoot roller on the W. C. Austin project, Oklahoma. PX-D-32052.

Side slopes have been compacted by the longitudinal operation of compaction equipment with rollers tied to heavy mobile equipment on the berm.

For unlined canals constructed in many fine soils and well-graded coarse soils with fines, seepage losses can be appreciably reduced by compacting the natural canal subgrade to a higher density. This is particularly true when the soils have a fractured or "root hole" structure. The construction procedure usually consists of scarifying, adding moisture, and compacting to the required density by sheepsfoot rollers, flat rollers, or other available equipment. The moisture and density requirements are established by laboratory tests.

A thin compacted-earth lining on the Post Falls unit of the Rathdrum Prairie project, Idaho, is shown in figure 51. This 6-inch lining with a 6-inch gravel cover, constructed in 1945, has performed very satisfactorily under severe freezing and thawing conditions. The average percentage of the laboratory maximum density for the soil of this lining in 1954 was 92 to 94 percent. Corresponding laboratory coefficients of permeability at these densities were about 0.9 foot per year per foot of head.

(a) *Cost.*—The cost of the thin compacted-earth lining described in the preceding paragraph was $0.39 per square yard. A much larger installation of 6-inch compacted-earth lining with a 12-inch gravel cover on the W. C. Austin project, Oklahoma, cost $0.50 per square yard when constructed in 1945. The Bureau has not used thin compacted-earth linings to any large extent because of the risk of relatively severe damage to the lining that can result from erosion or cleaning operations; the excess cost of maintenance of the thin lining may be greater than the difference in initial costs of a thin and a thick compacted-earth lining.

Figure 51.—View of completed canal lining with 6-inch layer of compacted earth and 6-inch protective gravel blanket shortly after construction in 1944. This installation is on the Rathdrum Prairie project, Idaho. 110-F-1484-C.

47. Performance of Compacted-Earth Linings. — Since there is a possibility that changes may occur in a compacted-earth lining due to physical weathering in combination with canal operation, this factor has been given serious consideration. The main question is a possible decrease in density and increase in permeability that could impair the efficiency of the lining in reducing seepage. Although alternate wetting and drying of the soil could conceivably affect the soil properties, freezing and thawing action in the colder areas is thought to be a more important factor in causing changes in the lining.

In 1954, the Bureau began collecting information on compacted-earth linings from selected test areas located in widely scattered projects from New Mexico to Montana, and at present (1962) there are about 17 different test sections. These test sections are predominately in thick compacted-earth linings where there are varying degrees of freezing and thawing action. Density tests have been made at periodic intervals in these linings since they were constructed; in many cases field permeability tests have been conducted in the lining, and in two instances seepage ponding tests have been conducted.

The results of these tests have shown that for some of the linings there has been no significant change in density even under moderately severe freezing and thawing conditions. The greatest decrease in density has been on a canal constructed in 1955, in Nebraska, which had a lining composed of silty loessial soil. The overall average decrease in density of the lining has been about 7 percent, with the greatest decrease being near the lining surface and the least near the bottom. In 1958, two ponding tests in the lining of this same canal showed low seepage losses (0.03 and 0.09 cfd).

Laboratory tests on silty loessial soil[2] have shown that specimens compacted to 95 percent of the laboratory maximum density and subjected to overburden loads of 1 and 2 pounds per square inch to represent the lower portions of a thick compacted-earth lining, retained a dry density of near 92 percent or greater after considerable freezing and thawing action. From this, it was calculated that the resulting seepage loss through a lining would be about 0.1 cfd or less, which is generally considered satisfactory for a canal lining.

[2] Lowitz, C. A., "Evaluation of Earth Lining Materials on Courtland Canal, Bostwick Division, Missouri River Basin Project, Nebraska," Earth Laboratory Report No. EM-563, Bureau of Reclamation, October 16, 1959.

48. Loosely Placed Earth Linings.—This type of lining consists essentially of a loose, uncompacted earth blanket of selected clay soils dumped into the canal and spread over the bottom and banks to approximate line and grade in layers up to about 12 inches in thickness. Seepage can often be reduced to an acceptable amount economically, provided available soils are sufficiently fine to be impervious in a loose state and are adequately stable to resist erosion to a reasonable degree.

Soils considered for loosely placed earth linings should be selected for impermeability in a loose condition and also should be selected to resist piping of soil fines into the subgrade.

Loosely placed earth linings find usage on both large and small jobs and are equally adapted to either contract or force account maintenance work. Very little trimming or reshaping of the canal section is necessary prior to placing a loose-earth lining. Furthermore, equipment requirements for this type of lining are relatively simple (fig. 52).

Unprotected loose-earth linings are subject to excessive erosion and severe damage from maintenance operations. However, some permanent seepage control may result from the unprotected loose-earth linings if the underlying soil contains voids into which the fine-grained lining particles can penetrate and become entrapped. Under less favorable conditions, loose-earth linings, if unprotected, may be effective for only a few seasons. Caution in selecting the lining material should be exercised to insure that the fine soils will not pipe through subgrades having large voids (i.e., very coarse gravels and fissured materials).

Although loose-earth linings do reduce seepage loss, they are not as effective or as permanent as compacted-earth linings. However, loose-earth linings cost much less than compacted linings. Contract costs for 12-inch loose-earth linings in the Coachella and Yakima Ridge Canals were 28 cents per square yard (1944-45). An installation in the Fire Mountain Canal,

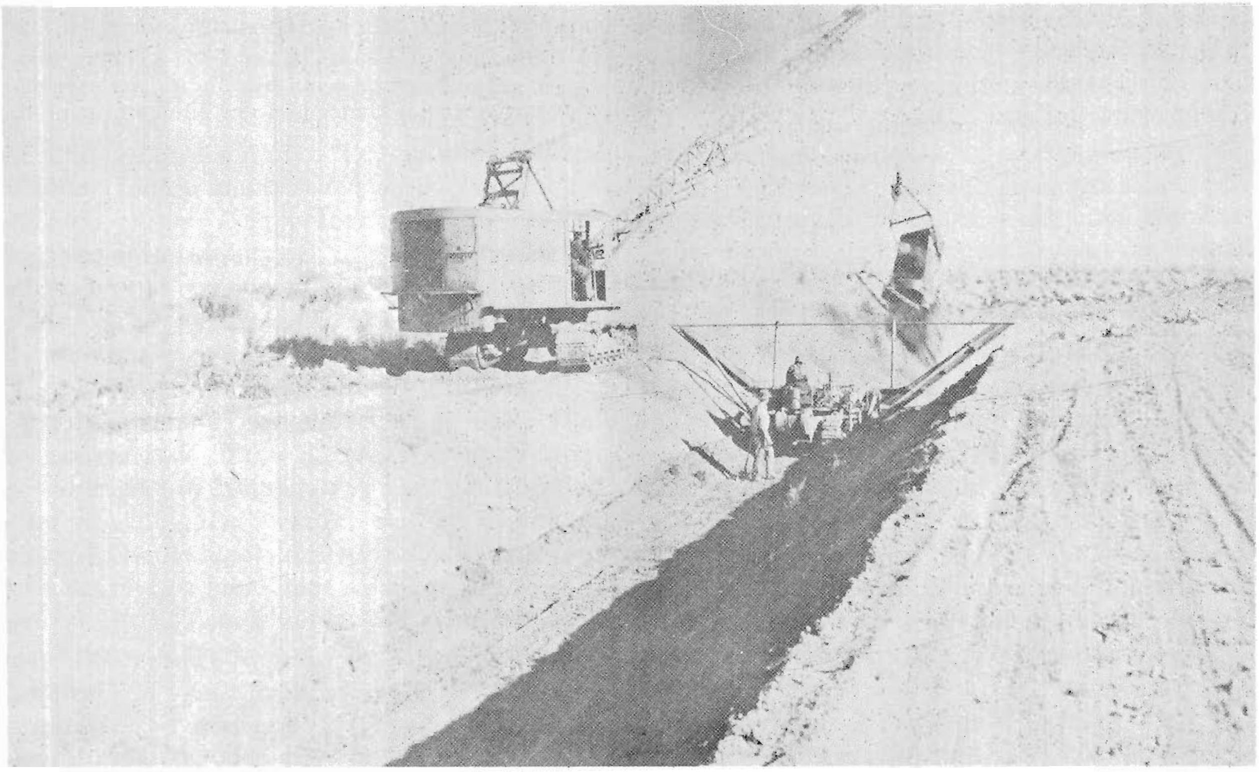


Figure 52.—Shaping loose earth blanket by means of bulldozer blade with extensions to fit the finished trapezoidal section of the Main Canal, Yakima project, Washington. 110-F-1488-C.

Paonia project (1950), cost 24 cents per square yard, based on unit bid prices, for 6 inches of loose earth with a 6-inch gravel cover over the areas subject to high water turbulence. The average life of the full lining for these installations was about 5 years, although some seepage reduction benefits are still apparent (1962).

49. ***Clay or Bentonite-Soil Linings.***—Bentonite, a clay having characteristics discussed under bentonite-membrane linings in section 42, and other clays are premixed with sandy soils and spread over the canal perimeter or mixed in place and compacted to form a 2- to 3-inch and sometimes thicker finished lining. The optimum amount of bentonite for the soil-mix type of lining usually ranges from 5 to 25 percent, but should be predetermined by laboratory test. A protective cover of stable earth or gravel is also recommended over the thin mixed or combination linings.

Bentonite proved effective in the lining of a canal on the Shoshone project, Wyoming, where a 4,000-foot section of this canal was lined with an earth-bentonite mixture in 1943. Although there was considerable seepage through the porous gravel subgrade formation prior to lining, no later seepage has been observed along the lined section.

Selection of a good quality bentonite and adequate control during placement of the bentonite are required. Most linings of this type have been placed by project forces to relieve seepage conditions after a canal or lateral has been placed in operation. Costs for the premixed and placed linings, as shown in table 2 (sec. 2), are considerably greater than for linings in which the bentonite is mixed in place with the subgrade soils.

50. ***Soil Stabilization.***—Many substances have been used to stabilize or seal canal and lateral subgrade materials. These include specially treated resins, chemicals such as sodium silicate in combination with sodium and calcium chloride, a commercial resin-cement, lime, portland

cement, asphalts, petrochemicals, and others including combinations of the above. Under certain circumstances the stabilization may play an important part in the permanence of an earth lining or an unlined canal.

Physical methods of stabilization, including the densification of the natural banks in combination with the special stabilizing materials, have been used experimentally with some success, but high costs have thus far prevented their field application.

(a) *Resins.*—Specially treated resins, in powder form, when added to soils containing considerable clay have been used as stabilizing agents for air strips and secondary roads. These resins tend to waterproof the soil with which they are mixed 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 ordinarily ranges from 1 to 3 percent. Because the resin renders the soil water-repellent, mixing water must be added to the soil before the resin is added. Maximum compaction is desirable. Linings of this type are mixed and compacted in the same manner as standard soil-cement, but no moist curing is required.

Several short test sections in a canal on the W. C. Austin project, Oklahoma, were stabilized in 1945 by mixing 1½ to 2½ percent of a commercial resin-cement product with the subgrade soil. The surfaces of these test sections were badly deteriorated in 1950, after only 5 years of use, indicating unsatisfactory serviceability.

(b) *Chemicals.*—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. When the soil is treated with solutions of sodium silicate and calcium chloride, the chemicals solidify the soil particles to a solid mass which is hard and impervious. However, this method is not believed adaptable to canal use because of the high cost of the chemicals and because such cemented soil has not proved very resistant to wetting and drying or freezing and thawing. Therefore, no work has been done in this connection in the current canal lining investigations.

Another possibility exists in the application of base-exchange principles. A subgrade soil containing clay with an excess of exchangeable calcium ions can be rendered less permeable by the addition of soluble sodium salts such as sodium chloride or sodium carbonate. The sodium ions will replace calcium ions to form sodium soils of much less permeability than the original calcium soils. Sodium chloride has been used to seal ponds and reservoirs, but no record of similar treatment of canals is available. Experiments are being conducted on a limited scale with silicones, lignins, and acrylomides.

The use of chemicals for canal stabilization requires more experimentation and evaluation before it can be recommended for general use. In view of the research now being done by military organizations in the stabilization of roads, it is entirely possible that canal stabilization by these methods can be developed in the future.

(c) *Physical Stabilization.*—The stability of either clayey or granular soils may be improved and the permeability decreased by combining these soils in proper proportions. This operation, termed "mechanical stabilization" in highway construction, may be accomplished at little expense by mixing the soils in place with discs and blades. Compaction of the soil mix will also add to its serviceability and effectiveness.

(d) *Portland Cement, Asphalt and Lime.*—In the stabilization of soil for highway purposes, portland cement, asphalt, and lime have become more or less standard stabilizing admixtures. These materials have been used in the Bureau on an experimental basis in both laboratory and field experiments for canal lining purposes.

Portland cement, when mixed in small quantities with soil, is called "cement-modified" soil in contrast with soil-cement (sec. 35) which contains cement in larger quantities. When 2 to 6 percent, by volume, of cement is used with plastic fine-grained soils, the soil particles become flocculated, perhaps by a combination of base exchange phenomena and cementing action, to form small conglomerate masses of new soil with reduced volume change characteristics. When such treatment is applied to the fine-grained soils which are already impervious, the stability of the soil is improved.

The use of either lime or cement reduces the plasticity, shrinkage, and expansion properties of the soil and increases soil stability, generally in proportion to the amount of admixture used. Although the cement admixture reduces the soil shrinkage under air drying somewhat more than would an equal amount of lime (probably because of superior cementing action), the properties of the lime-treated soil are more favorable in other respects, especially in reduction of plasticity and in increased unconfined strength after wetting and drying action. The recommended amount of lime or cement admixture for this type of field installation is 4 percent.

The Bureau has experimented in the laboratory with the stabilization of expansive clay of the type found in the Central Valley project of California, using both hydrated lime and portland cement.[3] Both of these admixtures, when used in concentrations of 2 to 6 percent by weight, drastically improved the plasticity and stability characteristics of the clay. The laboratory tests were made to devise a possible method of stabilizing sloughing canal banks of a large canal in expansive clay areas. Field tests were not conducted because sloughing in the canal was decreasing each year, and other repair measures were less costly. Stabilization might, however, be economical during original construction in expansive clay areas.

In 1958, after a series of laboratory tests, a field test section of cement-modified soil was installed on a canal in the Frenchman-Cambridge division of the Missouri River Basin project, Nebraska. At the time, thick compacted-earth lining was being constructed. The available materials were silty loessial soils having a low plasticity index, and gravel suitable for cover was costly. The compacted soil was impervious but was subject to erosion by wind, wave action, and flowing water. Compacted linings on the canal slopes were being constructed in layers of 6-foot width and of 6-inch compacted depth. The center 2 feet of the 6-foot width was stabilized with cement; for one test section, the proportion of cement was 2.5 percent by volume, and for another section, 4.5 percent. The required quantities of cement were spread on the top of each loose soil layer by a shop-made, fertilizer type spreader (fig. 53). The cement was mixed at a specified moisture con-

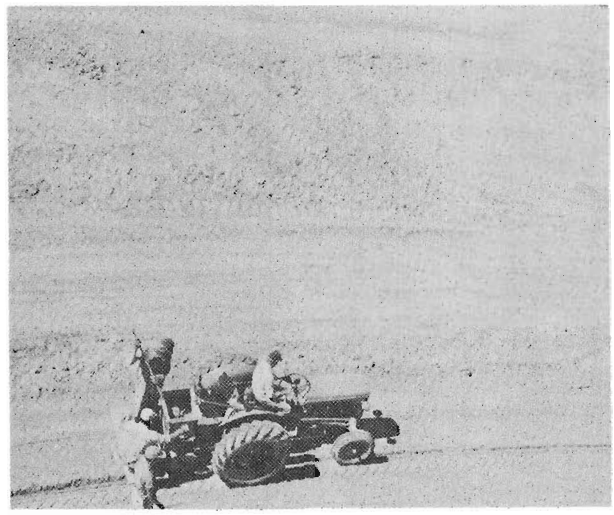


Figure 53.—Cement spreading equipment attached to back of tractor for use in stabilization of test section of thick compacted-earth canal lining. E-1886-13.

tent with the loose soil by a rotary type, traveling mixer (fig. 54). After mixing, the lining was compacted by sheepsfoot rollers in the normal manner for earth lining. After a 7-day curing period for the cement-modified soil, the canal section was trimmed to expose the cement-modified soil in the canal perimeter.

At the same time, and also after laboratory tests, adjacent sections of the same canal were stabilized with small quantities of asphalt emulsion. Each loose layer of earth lining was treated with diluted asphalt emulsion applied with an asphalt distributor in a 5-foot strip (fig. 55). In one test section, sufficient diluted emulsion (Federal Specification SS-A-674b) was applied to provide 1 percent, by weight, of the emulsion in its original strength to the soil; and in another test section, sufficient material was added to provide 2 percent of emulsion. During and following the proper application of moisture to the treated soil, the asphalt-soil mixture was thoroughly intermixed by the same rotary mixer used for the cement-stabilized soil (fig. 54). Afterward, the treated soil layer was compacted

[3] Jones, C. W., "Stabilization of Expansive Clay with Hydrated Lime and with Portland Cement," Bulletin 193, Lime and Lime-Flyash Soil Stabilization, Highway Research Board, Washington, D.C., 1958, pp. 40-47.

with sheepsfoot rollers in the normal manner (fig. 56), and the canal prism trimmed to expose the asphalt-treated soil.

At this time (1962), it is too early to evaluate the results of the cement- and asphalt-modified soils described above, but there are indications that the test sections having the higher concentrations of cement and asphalt are showing less erosion than the other treated sections and the untreated soil lining used as a control.

Figure 54.—Wetting and mixing soil containing cement in preparation for compaction. Only small amounts of cement are used in this soil stabilization method as contrasted with that used for a soil-cement lining. E-1886-19.

Figure 55.—Applying asphalt emulsion to soil with 1,000-gallon-capacity asphalt distributor, for soil stabilization. CH-575-1.

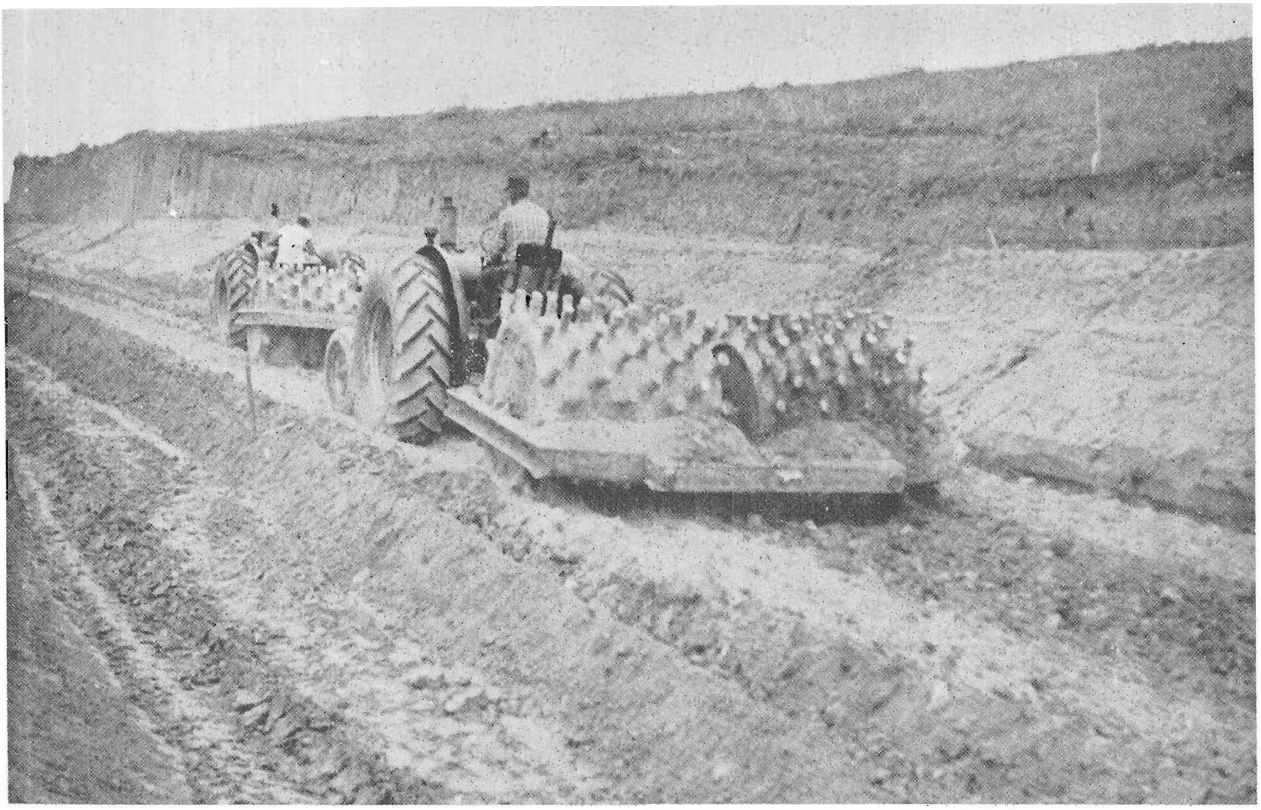


Figure 56.—Sheepsfoot roller compacting asphalt-emulsion-treated soil after mixing was completed with rotary tiller. CH-575-12.

Soil Sealants

51. ***General.***—It is not always practicable to utilize even the lower cost conventional types of linings on some projects, because of economic considerations; and the search for an even less costly method of reducing seepage losses continues. The puddling and priming of new unlined canals with silt, or sediment,[1] is an accepted practice for reducing seepage losses from canal systems. A natural sealing of operating canals occurs if water in the canal carries considerable sediment. In canals where the water is relatively clear, sediments have been added artificially.

One of the more promising fields of investigation at the present time (1962) is that of chemical sealants. Considerable work in this field has been performed by industrial firms as well as the Bureau of Reclamation. Laboratory experiments and field installations have been made, with some of the latter being evaluated over the past several years. Results accumulated to date are encouraging.

52. ***Sediment-Sealing.*** — A waterborne deposition of clay or silt over the wetted area of an unlined canal has often reduced seepage losses significantly. The resulting sediment lining is usually a relatively thin layer. As mentioned above, the sediment may be accumulated naturally as a result of the deposition of waterborne sediment carried into the system from outside sources or from erosion of unlined canals.

Where sediments are added artificially, seepage control is best effected if the silt or clay-sized sediment can penetrate the voids in the subgrade material. Placing of a gravel blanket on the canal subgrade has provided a harbor for the sediment and thus reduced seepage from canals on several projects. The effectiveness of this method of providing seepage control over a period of time is still under study.

Methods used for introducing sediment into the canal water for sediment-sealing have varied with the project.[2] Natural sediments from streamflow are fairly effective as long as the sediment is continuously supplied. When the water source is devoid of sediment, fine-grained soils have been added by various methods, including lowering the ponding reservoir so that sediments in the reservoir will be eroded from its bottom; sluicing hillside material into the canal (fig. 57); building sediment dams in the empty canal and sluicing them downstream with canal water; mixing dry commercial bentonite with and without dispersents in sumps and then dumping it into the flowing canal; dumping clay into the canal and stirring it with an air hose; and many others. All of these trials have saved some water which might, in a dry year, justify their use; but none have provided a permanent lining comparable in effectiveness of seepage control to the usual constructed linings.

The cost per application is quite low, but considering the limited degree of sealing obtained and the cost of periodically repeating the process, it is likely that more water may be saved at less cost over a long period of time by the use of a more conventional lining than by sediment-sealing. However, research work to develop sediment linings is still in progress (1962).

The effectiveness of sediment treatment appears to depend on the suitability of the material used, the velocity of the water in the canal, and the structural formation through which seepage occurs. The relatively thin layer of sediment that is usually deposited is highly susceptible to attrition by erosion, puncture, deterioration by weathering, and destruction by cleaning operations.

53. ***Bentonite Sedimenting.*** — At various times, consideration has been given to the sealing of canals by the introduction of colloidal bentonite in canal water. The theory was that the small bentonite particles would be carried to a con-

[1] Sediment is the more precise and more inclusive term, as sediment contains not only coarse-grained silts but also fine-grained materials such as clay, which are more effective as sealants.

[2] Dirmeyer, R. D., Jr., "Progress Report of Clay Sealing Investigations During 1961," Colorado State University Experiment Station, Civil Engineering Section, Fort Collins, Colo., January 1962, CER62RDD8.

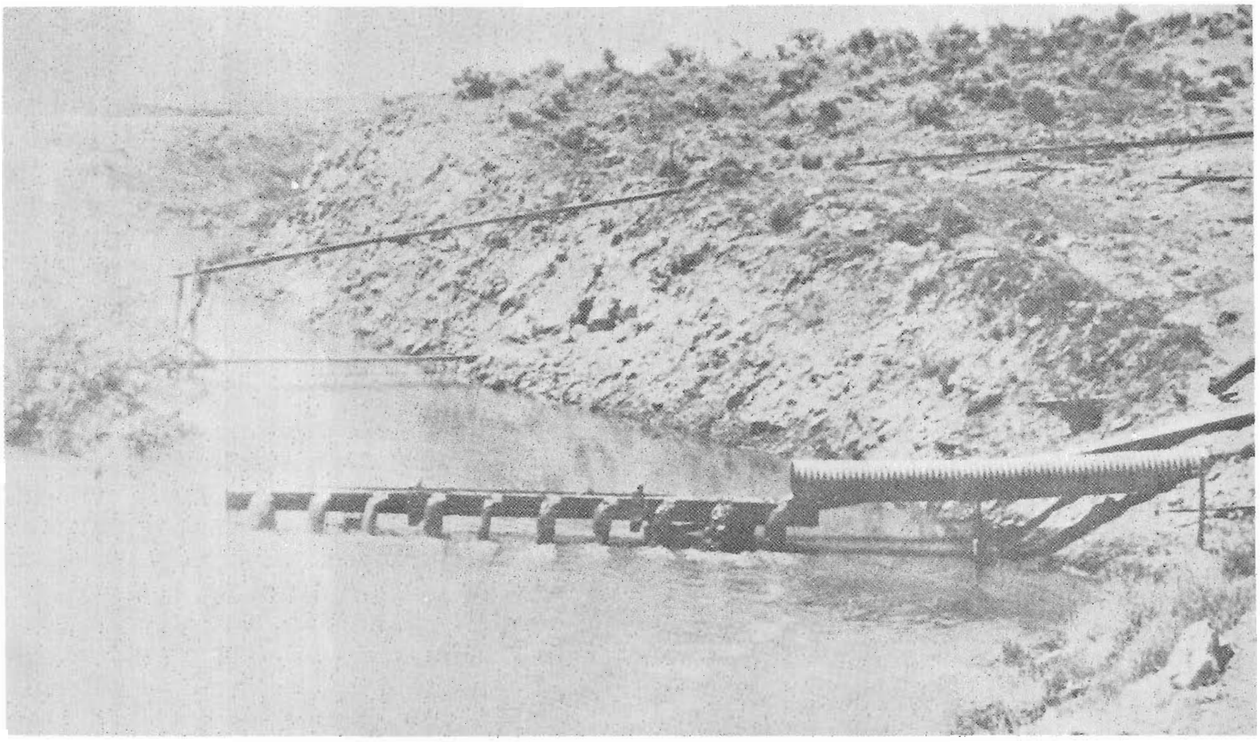


Figure 57.—Silt being washed by pumped water from adjacent deposit and distributed across canal by notched trough on the Vale project, Oregon. PX-D-1018.

siderable depth in the canal bed materials where seepage was occuring, swell upon becoming saturated, and plug the soil voids. The first recorded experiment in the use of bentonite as a sealing agent in canals was conducted in 1940 by the Bureau on the All-American Canal system, Boulder Canyon project.[3] It consisted of sedimenting a large soil sample in a hydraulic recirculating flume. The tests showed that the bentonite formed a surface coating on the soil which contracted upon drying and did not reform to produce an effective seal after once having dried. As a result of the experiment, plans for sedimenting the All-American Canal with bentonite were abandoned.

During the period 1953 through 1958, the lower cost canal lining program assisted in sponsoring a research project on the sealing of canals with bentonite. This project included field and laboratory studies by the Bureau and other cooperating agencies.[4 5] The work of others also has been closely followed.[6] The general bentonite sedimenting procedure in field trials consisted of mixing high-swell, commercial bentonite (containing largely sodium montmorillonite) and a polyphosphate dispersing agent with canal water, and ponding the dispersed bentonite in the canal reaches to be lined for a 24-hour period. About 1 percent bentonite, by weight of water, was used.

Instead of penetrating the soil, the dispersed bentonite generally formed a thin coating on the wetted perimeter of the canal. There was an indication from preliminary seepage tests that this bentonite coating resulted in a seal, as the seepage was reduced immediately after deposition of the coating. However, the thin

[3] Goss, R. E., "The Use of Bentonite in Decreasing the Seepage Loss in a Canal," Laboratory Report No. EM-504, Bureau of Reclamation, July 1957.

[4] Rhone, T. J., "Hydraulic Flume Test Using Bentonite to Reduce Seepage," Hydraulic Laboratory Report No. Hyd-417, Bureau of Reclamation, March 1957.

[5] Shen, R. T., "Sediment-Sealing with Bentonite in a Dune Sand," CER No. 58 RTS 25, Colorado State University Research Foundation, Fort Collins, Colo., August, 1958.

[6] "Minerals and Metals—Canal Lining Materials," Interior Missouri River Basin Field Committee Annual Report, June 1958, p. 91.

seal was subject tc shrinkage upon drying and to erosion from flowing water. There was some indication also that a base exchange of the bentonite in the presence of calcium in the water could have been a factor in early failure of the thin coating. In any event, seepage tests conducted at the end of the first irrigation season after the sedimenting operation definitely showed that the bentonite was not effective in reducing seepage for more than a few months under the particular conditions imposed on it.

Laboratory study[7] has indicated that the soil voids in a dune sand of a lateral on the North Platte project, Nebraska, where an unsuccessful bentonite sedimenting experiment was conducted, were sufficiently large for the bentonite particles to penetrate the soil. Other laboratory tests in the same study showed that bentonite particles are plate shaped, and when sedimented on a glass slide, the long axes of the particles were parallel to the surface of the slide. A logical theory from these studies is that the bentonite particles in a sedimenting operation fall flatwise on the surface of soil and bridge over small soil voids without penetrating into them, even though the voids are much larger than the bentonite particles.

54. *Chemical Sealants.*—(a) *General.*—Chemical sealants, as the term is used here, are chemical products which can be applied to the canal subgrade where they may react chemically to form solid or semisolid gels, may deposit precipitates in the soil voids, or may otherwise render the subgrade impervious to water even by predominantly physical action. Various methods of application may be used such as surface spraying, subsurface injection, or addition of the chemical to the canal water for subsequent deposition on the canal subgrade or penetration into the soils through which the canal has been excavated. The method selected for use will depend upon the type of sealant used, and the environmental conditions existing at the application site.

Bureau engineers believe that an ideal canal sealant should have the following characteristics:

(1) It must be nontoxic to humans, animals, and crops.

(2) It must reduce leakage to 0.1 to 0.3 cubic foot per square foot per day.

(3) It must be capable of nonrestrictive application:
- At most any time of year.
- Under a broad range of water pH and salt content.
- Under a broad range of soil composition.
- In static or dynamic flow conditions.

(4) It must resist damage by animals, equipment, erosion, and hydraulic pressures (20 psi).

(5) It must be durable:
- Not deteriorated by climatic conditions such as freezing and thawing, sunlight, wetting and drying.
- Not deteriorated by soil microorganisms.
- Not deteriorated by reemulsification or chemical change.
- Not deteriorated by reverse hydraulic flow.
- Capable of resealing.

(6) It must be efficient in use of material (low cost).

A few of these requirements of an ideal canal sealant are fulfilled by several products commercially available. However, at this time, the Bureau is unaware of any material that will completely fulfill all of the requirements satisfactorily.

A general summary on the use of chemicals for soil stabilization and sealing has been compiled by the Bureau of Reclamation.[8] This summary includes the experience of the Bureau and related agencies in this field. It also contains a bibliography of all the publications that have come to the attention of the Bureau on the subject which may have a bearing on the use of chemical sealants in canals. This report was sent to about 75 chemical and allied companies in order to stimulate interest in the development of chemical sealants. Responses have been received from about 20 of the companies, asking for more copies of the report, and some expressed considerable interest in possible cooper-

[7] Rubenstein, S., "Physical Characteristics of the Bentonite-Soil Interface—Sediment Sealing Project—Lower-Cost Canal Lining Program," Laboratory Report No. PET-122, Bureau of Reclamation, September 2, 1958.

[8] Blackburn, W. C., "A Review of the Use of Chemical Sealants for Reduction of Canal Seepage Losses—Lower-Cost Canal Lining Program," Analytical Laboratory Report No. CH-102, Bureau of Reclamation, February 9, 1960.

ation in the chemical sealant program. Representatives of several companies have visited the Bureau laboratories in connection with the study in response to the report.

A brief description of the test procedures used for preliminary tests with sealants in permeameters was compiled and distributed to companies requesting the procedure, so that they might do their own preliminary testing. Also, large soil samples of typical canal bed materials were shipped to the interested manufacturers for their laboratory sealant studies. A few companies have developed, or are presently (1962) in the process of developing, chemical soil sealants. The formulations of the sealants are usually not revealed, and sometimes their action on soil to accomplish seepage reduction is not definitely known.

(b) *Waterborne Sealants.*—A low-cost chemical sealant which could simply be added to the water flowing in a canal and which would be effective in reducing the permeability of the canal soils would probably result in a truly low-cost seepage control method. Studies in this direction have been underway for several years, under the Bureau's lower cost canal lining program, with some limited results being achieved. A more concentrated investigation has been included in the program since 1959.

Most of the waterborne sealants tested have been surface sealants or at best have only penetrated the canal subgrade materials to a relatively shallow depth; hence their permanence is questionable, since at only shallow depths the seal is subject to erosion and damage by the traffic of cattle, canal cleaning equipment, etc. Some of the materials tested are relatively low in cost, about 10 to 30 cents per square yard of treated surface; are acceptable within limits; and being low in cost, repeated treatment could provide seepage control. However, the cost of such treatment over a period of years would have to be compared with that of more permanent type linings to determine the relative economy. A material that seals at a greater soil depth would overcome some of these objections, and studies in this direction are continuing. A waterborne type of sealant must necessarily be readily miscible with water, and those tested to date have been generally in the emulsion form.

Two general methods have been used for applying the different waterborne sealants; in both cases, the sealant is introduced into the canal water from drums or tank trucks at a regulated rate to produce a recommended concentration, which is usually in the range of 0.05 to 1.0 percent by volume of canal water. Introduction of the sealant is usually made at a point of turbulence in the canal, as at a drop structure, to facilitate mixing of the sealant with the water. Sometimes supplementary mixing methods are required.

In one general type of application, the sealant is introduced into a flowing canal with the water checked up at canal structures to greatly reduce the velocity of flow through the canal. This allows more time for the sealant to act on the subgrade soil in a given treatment reach as it progresses down the canal, than would be allowed if the velocity of normal water operation were used.

A second general type of application is the ponded method where the treated water is allowed to stand in successive ponds in the canal, which have been formed by temporarily sealing canal structures or placing temporary dams between structures. By this method, the water is allowed to remain in each pond for a sufficient period to give the sealant time to act upon the canal soils. The contact time necessary for proper treatment will vary with the sealant being used and the type of material being treated. The ponding method is more costly, but experience to date has indicated that the resulting sealing effects are somewhat better than for the flowing water method. The seepage reduction obtained by the waterborne sealants has been in the range of 25 to 99 percent, as determined by posttreatment evaluation tests which are generally made within a few days to a few months after application. In only a few cases have posttreatment seepage tests been conducted to evaluate effectiveness after a significant period of time. In such instances, few installations have maintained the same amount of seepage control as that provided immediately after sealing; most have lost some of their effectiveness.

The experimental field treatments of canals and laterals with chemical sealants that have been closely followed by the Bureau, and the results achieved to date, are summarized in table 12. Three proprietary products have been used as follows:

(1) A resinous polymer with heavy atoms (designated RP in table 12), described by the manufacturer as a material mixed in a common diesel fuel. Its function, as also described by the manufacturer, is to increase the ionic attraction of the soil particles for water, thus increasing the thickness of the hygroscopic envelope around each soil particle.

(2) A petroleum-based emulsion (designated PB in table 12), described by the manufacturer as an emulsion of petroleum products and other materials capable of being formulated to penetrate sandy soils to any desired depth, depending upon soil conditions, and deposit a waxy layer at depth.

(3) A cationic asphalt emulsion (designated CA in table 12), which is described by the manufacturer as a petroleum product that possesses unique properties which provide a much tighter bond to some soil particles than is provided by standard commercial (anionic) asphalt emulsion.

The results presented in table 12 should be interpreted with the period of evaluation in mind. It will be noted that some of the sealants are so new that field evaluation of their effectiveness is limited to but a 6- to 12-month period. Plans have been made to check on the effectiveness of chemical sealant installations with a continuing series of seepage tests to determine their performance over a period of years.

(c) *Sprayed-in-Place Sealants.*—A field application of cationic asphalt to seal a sewage lagoon under construction by the City of Woodland Park near Colorado Springs, Colo., was observed. The lagoon, covering a 3½-acre area, had been recently constructed in a riverbed of decomposed granite. Prior to the treatment with the asphalt, water pumped into the lagoon at a rate of about 100,000 gallons per day was soon lost by seepage.

A blend, consisting of cationic asphalt emulsion, white gasoline, and water, was rapidly applied at a rate of 0.3 gallon per square yard to the prepared and prewetted surface by hand spraying and distributor spray-bar methods. The blend was mixed and applied without any premature breaking or excessive rundown on the slopes. Penetration into the surface varied from three-sixteenths to three-eighths of an inch.

The cost for sealing 18,267 square yards was slightly more than 7 cents per square yard, including labor, using the quantities and methods described. The amount of undiluted emulsion applied on this installation was only 0.09 gallon per square yard. Based on laboratory tests, the application rate was far less than that required to provide satisfactory penetration and sealing. However, for this installation, only a temporary reduction of water loss during the initial operation period of the lagoon was required because sewage is expected eventually to seal the lagoon.

Recent information from engineers who were concerned with this installation has indicated that, for the bottom part of the reservoir which has been covered with water, the asphalt emulsion treatment is considered to have performed satisfactorily. Observations of the higher portions of the reservoir which have been exposed above the waterline show that the treatment in these areas has deteriorated.

(d) *Injected Subsurface Sealants.* — Undersealing of unlined canal subgrade materials by injecting asphalt emulsion in the subgrade has been conducted on an experimental basis. This differs from asphalt grouting described in section 56(a) in that placement of a continuous layer of asphalt 6 to 8 inches below the perimeter of the unlined canal is attempted, rather than a cutoff in the banks. This method of seepage control is of interest for use in canals and laterals that are in continuous operation throughout the year and cannot be dewatered for ordinary lining construction or rehabilitation work.

Experiments were conducted with several different machines developed to inject diluted or blended asphalt emulsions into canal subgrade soils. The first such field trial injection of asphalt emulsion was performed at Yuma, Ariz., in 1951. Based on work performed in the Bureau's Denver laboratories, a scarifier type of machine was tried in a project-constructed test canal using various asphalt emulsions and blends of emulsions. This trial was made to determine the possible use of such application for sealing sandy subgrade materials without dewatering the canal. The scarifier equipment and a later slip-form arrangement proved unsuccessful.

TABLE 12.—*Results of the treatment of canals and laterals with waterborne chemical sealants, as of June 1962*

Location	Canal or lateral	Capacity second-feet	Length and area treated	Subgrade material	Method of treatment	Seepage loss data: Method of evaluation	Seepage loss data: Date	Seepage loss data: Loss (cfd)[2]	Seepage loss data: Percent reduction	Time of seepage evaluation	Remarks
						Resinous Polymers					
Boise project, Idaho	Lateral 10.2, bypass A	69									
	Pond A		200 feet	Silty fine to medium sand	Ponding	Ponding	July 1959	0.99	—	Before treating	Gypsum added to ponded water before treatment with sealant.
							July 1959	0.68	31	5 days after treating	
							September 1959	0.56	43	2 months after treating	
							May 1960	1.00	—	10 months after treating	Silt deposited on wetted perimeter removed before evaluation
	Pond B		200 feet	Silty fine to medium sand	Ponding	Ponding	July 1959	1.86	—	Before treating	
							July 1959	0.88	52	5 days after treating	
							September 1959	0.75	60	2 months after treating	
							May 1960	1.40	25	10 months after treating	Silt deposited on wetted perimeter removed before evaluation.
	Lateral 10.2, bypass B	—									
	Pond 3		200 feet	Silty fine to medium sand	Ponding	Ponding	October 1961	0.63	—	Before treating ponds 1, 2, 4 and 5	Untreated control pond for ponds 1 and 2 treated with cationic asphalt (see below) and ponds 4 and 5 treated with resinous polymers.
							October 1961	0.55	13	24 hours after treating ponds 4 and 5	Reduction from natural causes.
	Pond 4		200 feet	Silty fine to medium sand	Ponding	Ponding	October 1961	0.81	—	Before treating	
								0.56	31	24 hours after treating	
	Pond 5		200 feet	Silty fine to medium sand	Ponding	Ponding	October 1961	1.31	—	Before treating	
							October 1961	0.54	59	24 hours after treating	
Boulder Canyon project, California	Coachella Canal	2,500	8 miles	————	Flowing	——	October 1957	—	—		Operational records show saving of about 16,000 acre-feet per year since treatment.
Salt River Valley project, Arizona	South Canal	1,350	4,000 feet	Deteriorated shotcrete lining on sand	Ponding	Ponding	November 1958	0.96	—	Before treating	Seepage losses given are for those at normal operating water depth.
							December 1958	0.25	74	24 hours after treating	
							November 1959	0.16	83	1 year after treating	
							November 1960	0.13	86	2 years after treating	
	Indian Bend Pump Lateral	22	2 miles								
	Pond 1				Flowing	Ponding	May 1960	1.48	—	Before treating	Data on all ponds based on a 2-foot operating depth.
							June 1960	0.59	60	4 days after treating	
							June 1960	0.59	60	1 week after treating	
							December 1960	0.77	48	6 months after treating	
	Pond 2				1st Flowing	Ponding	May 1960	1.03	—	Before treating	
							June 1960	0.82	20	4 days after treating	
							June 1960	0.82	20	1 week after treating	
							December 1960	0.60	42	6 months after treating	
					2nd Ponding		January 1961	0.40	61	Immediately after second treatment	Percentage reduction based on seepage determined prior to first treatment
	Pond 3				1st Flowing	Ponding	May 1960	0.58	—	Before treating	
							June 1960	0.42	28	4 days after treating	
							June 1960	0.49	15	1 week after treating	
							December 1960	0.15	74	6 months after treating	
					2nd Ponding		January 1961	0.10	83	Immediately after second treatment	
Maricopa County Water District, Arizona	Beardsley Canal	15	400 feet	Silty sand	Ponding	Ponding	February 1961	—	—	Before treating	Loss reported is based on data furnished by Agricultural Research Service
							February 1961	—	83	24 hours after treating	
							May 1961	—	15	3 months after treating	Test abandoned
Farmer's Reservoir and Irrigation Co., Colorado	Bowles Seep Canal	30	1,250 feet	Silty fine sand	Ponding	Ponding	June 1960	5.0	—	Before treating	Loss reported is based on data furnished by Soil Conservation Service
							June 1960	2.0	60	24 hours after treating	
							October 1960	3.2	36	4 months after treating	
							July 1961	3.7	26	13 months after treating	

TABLE 12.—*Results of the treatment of canals and laterals with waterborne chemical sealants, as of June 1962*—Continued

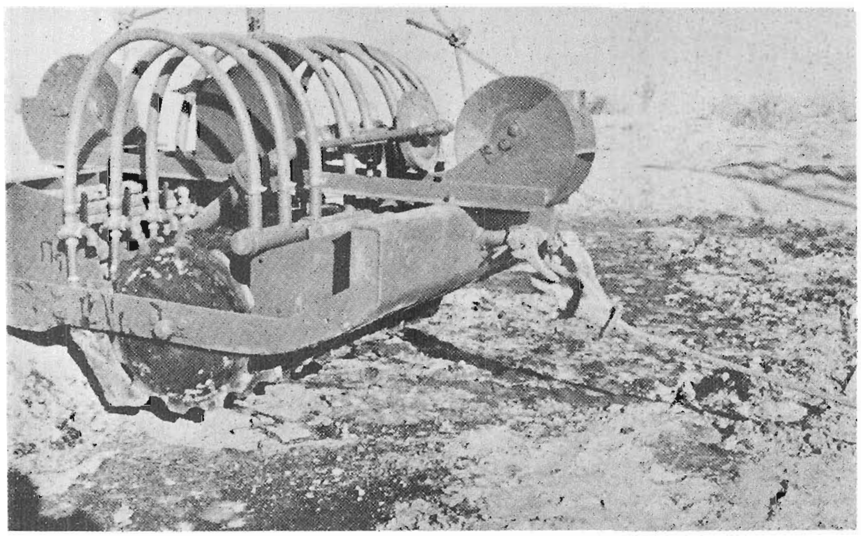
Location	Canal or lateral	Capacity, second-feet[1]	Length and area treated	Subgrade material	Method of treatment	Seepage loss data: Method of evaluation	Seepage loss data: Date	Seepage loss data: Loss (cfd)[2]	Seepage loss data: Percent reduction	Time of seepage evaluation	Remarks
Petroleum - Based Emulsion											
Columbia Basin project, Washington	Lateral WB-5-K	59 to 4	3 miles	Silt with fine sand	Ponding	Ponding	October 1961	1.19	—	Before treating	Loss reported is average of 10 ponds.
							October 1961	0.39	67	Immediately after treating	
							April 1962	1.05	12	6 months after treating	Based on preliminary information.
	Lateral WB-5-G	68 to 4	3 miles	Silty fine sand	Ponding	Ponding	October 1961	0.81	—	Before treating	Loss reported is average of 2 ponds.
							October 1961	0.49	40	Immediately after treating	
							April 1962	0.68	16	6 months after treating	Based on preliminary information.
Central Valley project Madera Irrigation District, California	Allen - Leach Ditch	—	1,000 feet 1,500 sq. yds.	Medium to coarse sand	Ponding	Ponding	May 1961	4.2	—	Before treating	
							May 1961	0.2	95	Near end of treatment period	
							March 1962	1.4	67	10 months after treating	
	Italian - Swiss Ditch	—	1,000 feet 1,000 sq. yds.	Well-graded fine to medium sand	Ponding	Ponding	May 1961	2.5	—	Before treating	
							May 1961	0.1	96	5 days after treating	
							March 1962	1.4	44	10 months after treating	
Klamath project, Oregon Langell Valley Irrigation District	North Canal	—	4,600 feet	Silty fine to medium sand with little gravel over rock	Ponding	Ponding	April 1961	1.9	—	Before treating	
							April 1961	0.5	74	24 hours after treating	
							March 1962	1.4	26	11 months after treating	
Shasta View Irrigation District	"V" Canal	21	3,000 feet	Fine to medium sand, clean to slightly silty	Ponding	Ponding	September 1961	0.6	—	Before treating	
							September 1961	0.3	50	24 hours after treating	
Solano project, California	Putah South Canal Above Rockville Siphon	180	1,400 feet	Concrete lining on sand cushion over rock	Flowing	Visual observation	September 1961				Reported successful from evident reduction in water table adjacent to canal and in nearby town
	Above Mangel Siphon	180	5,500 feet				March 1962				
Eden project, Wyoming	West Side Lateral	120 to 6	6.6 miles 100,000 sq. yds.	Silty fine to medium sand, some shale outcrops	Flowing	Inflow-outflow	June 1961	1.3 to 1.9	—	Before treating	Range given because before and after water conditions were not sufficiently comparable to arrive at single values.
							June 1961	0.9 to 0.7	53 to 46	24 hours to 2 weeks after treating	
							June 1962			1 year after treating	
Cationic Asphalt Emulsion											
Boise project, Idaho	Lateral 10.2, bypass B Pond 1	69	200 feet	Silty fine to medium sand	Ponding	Ponding	October 1961	0.25	—	Before treating	See data under resinous polymers above for untreated control pond 3.
							October 1961	0.04	84	24 hours after treating	Surface seal only, which was badly deteriorated by Spring of 1962.
	Pond 2		200 feet	Silty fine to medium sand	Ponding	Ponding	October 1961	0.66	—	Before treating	See data under resinous polymers above for untreated control pond 3.
							October 1961	0.13	80	24 hours after treating	Surface seal only, which was badly deteriorated by Spring of 1962.
Maricopa County Water District No. 1, (near Pheonix, Arizona)	Pond 1	—	50 feet	Silty sand	Ponding	Ponding	February 1961	2.67	—	Before treating	Treatment and evaluation by Agricultural Research Service.
								2.37	11	24 hours after treating	Ponds 1 and 2 were untreated control ponds.
								1.98	26	96 hours after treating	Ponds 3 and 4 were treated.
	Pond 2		50 feet	Silty sand	Ponding	Ponding	February 1961	2.59	—	Before treating	
								2.24	13	24 hours after treating	
								1.82	30	96 hours after treating	
	Pond 3		50 feet	Silty sand	Ponding	Ponding	February 1961	2.86	—	Before treating	
								1.74	39	24 hours after treating	
								1.68	41	96 hours after treating	
	Pond 4		50 feet	Silty sand	Ponding	Ponding	February 1961	6.29	—	Before treating	
								3.65	42	24 hours after treating	
								2.54	60	96 hours after treating	

[1] Where one value is shown it is capacity at upstream end of reach treated.

[2] cfd = cubic feet per square foot of wetted area per 24 hours.

LCCL-T12 (2 OF 2)

In 1953, results obtained in field tests with a machine using conventional notched farm equipment colters were encouraging. In 1955, consideration was given to experimental work with an improved machine on unlined canals on the Klamath project, Oregon. Although these plans did not materialize, a disk and cutter blade type injection rig was manufactured in 1961 and preliminary tests were made with this machine (fig. 58) in the following spring. A continuous membrane of asphalt emulsion one-half inch in thickness was injected at about a 6-inch depth, with encouraging results. However, mechanical problems have been encountered in the operation of the machine, and further tests to correct these are scheduled for the fall of 1962.



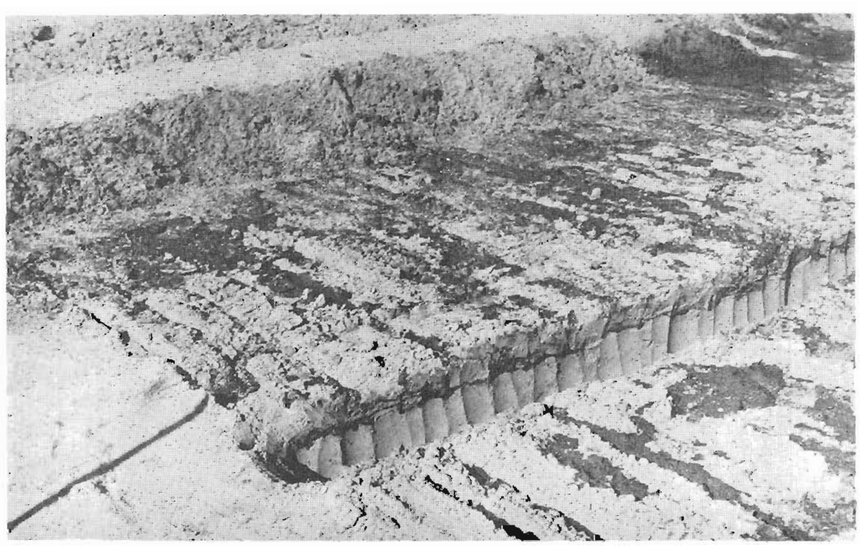


Figure 58.—Undersealing of a canal subgrade by injecting asphalt emulsion beneath the surface. Top photograph shows the machine for injecting asphalt emulsion or other liquid sealants. Excavated trench in the bottom photograph shows continuous membrane of minimum ½-inch thickness formed at about 6 inches depth by use of machine in top photograph. P35-D-30996 and P35-D-31003.

Other Means of Seepage Control

55. ***General***.—In addition to the means, methods, and materials outlined in the previous chapters, the Bureau has used other materials and procedures independently or in combination with linings to reduce seepage in irrigation systems, to stabilize existing canal linings, or as a substitute for linings. These methods and procedures have not become widely used because of some disadvantages such as high cost, short life, narrow field application, or lack of development. Nevertheless, they have been considered and used experimentally, and therefore justify being discussed briefly in the overall treatment of seepage control.

56. ***Grouting***.—(a) *Asphalt Grouting*.—Emulsified or hot liquid asphalt has been injected into crevices, joints, and pore channels existing in rock regions, shattered shale, gravel, sand, or other water-permeable materials. Introduced under pressures as high as 100 pounds per square inch through pipes drilled or driven into the permeable area, the asphalt is frequently forced to travel comparatively long distances.

One trial test using injection means for controlling seepage through a permeable sand embankment was performed in a canal on the Yuma project, Arizona, in 1947. Holes were driven into the embankment on approximately 5-foot centers and an asphaltic type oil was forced into the embankment material. Rather incomplete data obtained from the experiment indicate that some seepage reduction was accomplished but that migration of the asphalt with time has limited the effectiveness of the operation.

A patented method developed by a west coast oil producer, using special asphalt emulsions and breaking agents, is reported to have been used with success in grouting subterranean gravelly streambeds to prevent subsurface flows. The method is reputed to be less expensive and more effective than metal sheet piling commonly used for such purposes. In general, however, asphalt injection in this manner is costly and appears to be impractical for the large scale control of seepage in canals.

(b) *Grouting with Bentonite and Soil Slurries*.—The Bureau and other agencies have injected soil slurries and bentonite slurries into canal and reservoir banks in an effort to fill voids and cracks in loose material and reduce seepage.

In 1953, the Bureau injected a sandy clay slurry into voids and rather large cavities around and above Tunnel No. 5 on the Conchas Canal, Tucumcari project, New Mexico. Apparently the process successfully filled these voids and cavities, because settlement of the surface above the tunnel, which had long been a problem, stopped.

The Bureau's experience indicates that soil slurries are effective in filling larger cavities, with possibly some slight consolidation of loose material. However, the effectiveness of bentonite and soil slurries in plugging finer voids and reducing seepage has been limited. In 1956 and 1957, the Bureau injected bentonite and other clay slurries into a dike near the Madera Canal on the Central Valley project, California, with little apparent success. The thick drilling mud used in drilling the grout holes greatly restricted the lateral movement of the bentonite slurry, and the holes were not spaced closely enough to produce a continuous grout curtain.

In another test, bentonite slurry injected into the banks of the South Branch Canal of the Kittitas division near Yakima, Wash., appeared to have reduced seepage immediately after the injection, but some sections of the canal bank remained wet. It is believed that the grout curtain probably did not extend deep enough in these wet areas.

The Bureau injected bentonite slurry into a reservoir dike and a gravel foundation on the Altus project in Oklahoma in 1942. Immediately after injection and for at least 2 years, a large portion of the seepage appeared to have stopped; later observations have not been made. In these instances the bentonite grout was pumped into large open voids in the dike and into open gravels in the foundation.

In 1952 the Bureau injected bentonite slurry into the banks of the Casper Canal on the Kendrick project in Wyoming. Although seepage was significantly reduced 1 year after the injection, it has largely returned. Much of this increase in seepage after the first year was attributed to the chemical action between the bentonite and the gypsum in the soil.

The experiences discussed above indicate that it is very difficult to grout effectively with bentonite unless void spaces are of sufficient size to permit movement of the bentonite particles. Therefore, bentonite cannot easily be forced into a fine-grained soil mass to develop an effective grout curtain, unless large, connected voids or cracks are present.

57. ***Undersealing Concrete Canal Linings.*** — The undersealing of old concrete canal linings with portland cement grout or asphalt, while not strictly a lining procedure itself, is important as an economical method of lining rehabilitation. In this method, which is similar to conventional highway and airfield pavement undersealing, holes, usually 1½ inches in diameter on 6- or 8-foot centers and adjusted to crack and joint patterns, are drilled through the concrete lining in the area to be undersealed.

(a) *Asphalt.*—Where hot asphalt cement is used as the underseal, it is pumped under moderate pressure from distributors through flexible metallic hose and an asphalt gun into the drill holes (figs. 59 and 60). In this way the asphalt is driven a distance of 5 to sometimes more than 40 feet under the lining from the point of injection. The objectives of this system are to fill all joints and water channels under the lining and to minimize leakage by the injection of asphalt under moderate pressure through most of the drill holes without appreciable lifting of the lining.

Two areas of badly cracked and poorly supported old concrete lining on a canal of the Riv-

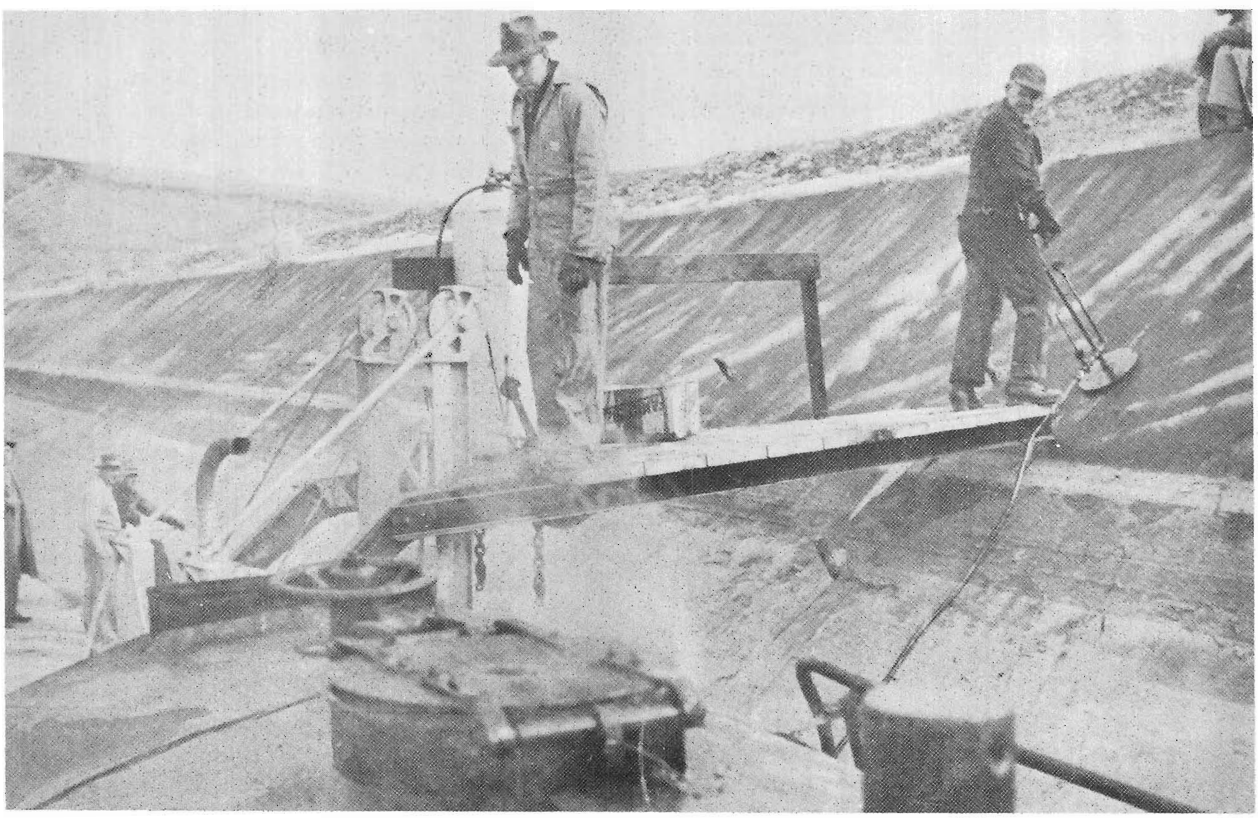


Figure 59.—Asphalt undersealing of the concrete-lined Yakima Ridge Canal, Yakima project, Washington. P33-D-14664, March 1950.

Figure 60.—Closeup view of an asphalt undersealing gun which was fabricated from standard pipe and fittings. PX-D-14667.

erton project, Wyoming, and a 1,000-foot reach of the Yakima Ridge Canal in Washington were undersealed with asphalt in 1949 and 1950. Although definite benefits are apparent, particularly on the Riverton project, some leakage still persists from the Yakima Ridge Canal lining. Possibly the length of section treated at Yakima did not include all the leaking areas, and subsequent leakage may be occurring beyond the end of the undersealed section. This possibility has not been reliably determined, and therefore the effectiveness of the treatment cannot be definitely stated. A somewhat similar experience was encountered in trying to underseal a reach of concrete lining on the Heart Mountain Canal of the Shoshone project, Wyoming.

Asphalt undersealing has been found to be of special advantage in controlling leakage in emergencies, since many linings are close to available sources of asphalt and the repair may frequently be started and completed within a matter of a few hours. This procedure has also been found worthwhile for correcting leakage in canals carrying water for power generation purposes, where long periods of power interruption cannot be tolerated. From 1 to 5 gallons of asphalt are normally required per square yard, and the usual cost of this treatment varies from $1 to $2 per square yard of lining for drilling, asphalt, and application. Bulk asphalt suitable for this use costs about $0.085 per gallon.

(b) *Portland Cement Slurries.*—Portland cement slurries also have been used for undersealing linings. The mixtures generally consist of 1 part portland cement and 5 to 6 parts of fine sand or 3 to 4 parts of silty soil screened over window screen material. Two percent bentonite has been used in some of the slurries. The slurries should not be placed under hydraulic pressure great enough to lift the lining and create additional cracks. In order to assure complete coverage under the lining and afford relief to pumping pressures, holes approximately 1½ inches in diameter should be drilled on about

5- to 10-foot centers through the lining on both the bottom and side slopes before the pumping begins.

58. *Cutoff Walls.* — Asphalt, portland cement concrete, and plastic film have been used successfully in the construction of cutoff walls for the interception and control of seepage through channels of escape, such as those resulting from decayed tree roots in embankments or horizontal pervious strata existing in or below channel excavations. These walls are constructed by trenching through the leakage area.

When asphalt, usually a cutback, is used, it is mixed with the earth removed from the trench or with a selected material. The trench is then backfilled with the mixed material and properly compacted. Although this method has been employed outside the Bureau with quite satisfactory results, the Bureau has not availed itself of the asphalt cutoff method of seepage control because it is more expensive and less adaptable to various conditions than are the other methods. Nevertheless, cutoff walls may fulfill the needs of unusual situations in the control of seepage, and such walls constructed of portland cement concrete placed in the excavated trench or embedment of a plastic film curtain in the trench have been used successfully. Such an installation using 8-mil-thick, black, polyvinyl film, was constructed along the downhill side of a canal on the Boise project, Idaho (figs. 45 and 61). A trench about 400 feet long was excavated by dragline to a depth of 28 feet to intercept an impervious clay stratum. The film, which was fabricated and furnished in one piece 400 feet in length and 30 feet in width, was placed in the trench and anchored at the top and bottom. It was then covered by backfilling the trench. This method proved successful in stopping seepage in this localized area. Comparative costs of linings would be the factor to be considered.

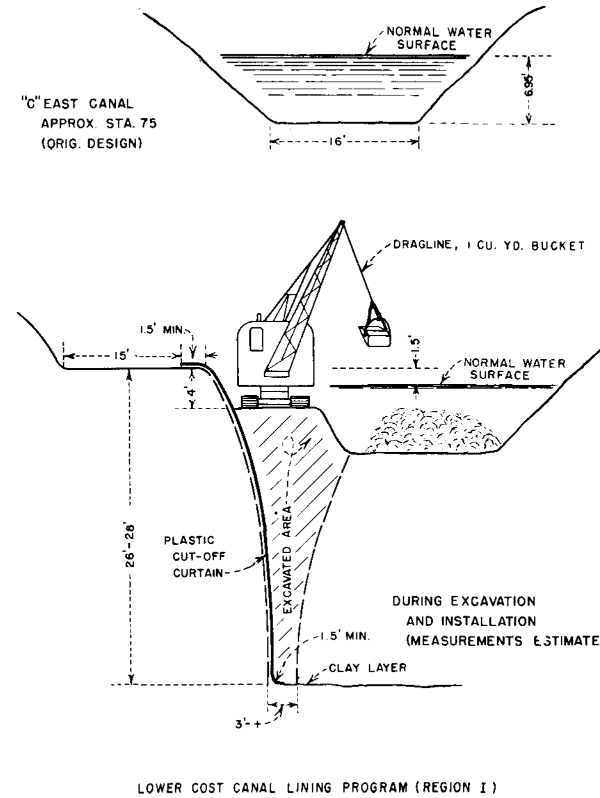


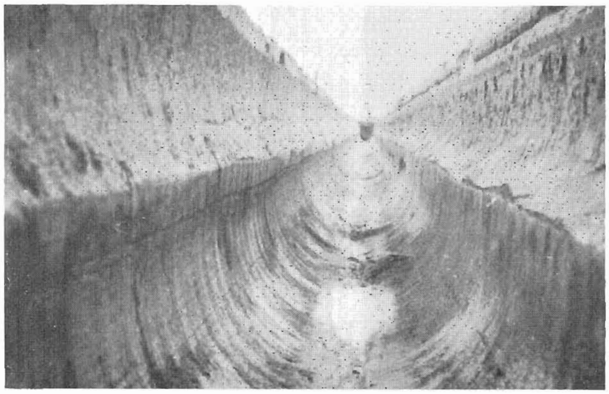
Figure 61.—An experimental installation of a plastic cutoff curtain in an excavated trench along a canal of the Boise project, Idaho. 288-D-2679.

59. *Cast-in-Place Concrete Pipe.*—Various private irrigation districts in the San Joaquin Valley of California have constructed cast-in-place concrete pipe for a number of years. The pipe is generally used in lieu of lined or unlined canals for farm deliveries up to about 15 second-feet, when the cost of constructing the pipe is equal to or less than that of concrete-lined canals. The average pressure heads used in the pipe range from 5 to 8 feet.

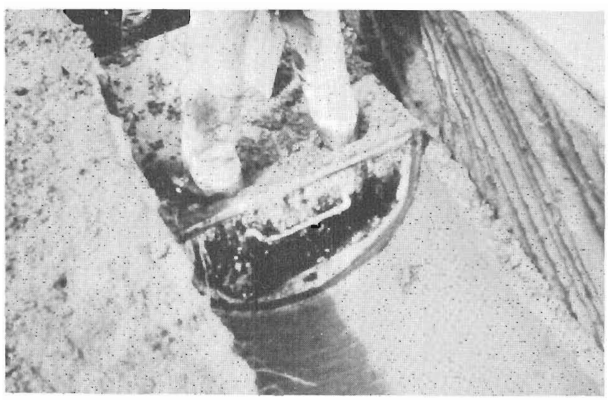
Pipe with inside diameters of 24, 30, 36, 42, and 48 inches has been used, and the Salt River project in Arizona has proposed the construction of 54-inch-diameter pipe. Pipe with diameters of 42 inches and over is more difficult to construct, with consequent higher cost. Pipe with diameters of 24 to 42 inches is most commonly used.

(a) *Two-Part Construction.* — An increasing demand for the cast-in-place method of constructing concrete pipe in the vicinity of Modesto and Turlock, Calif., led to the establishment of numerous contracting firms for constructing these pipelines. There has been considerable competition for the work and reasonable construction costs have resulted.

Most pipe constructed in the past has been fabricated in two parts, with the method of construction varying only slightly among the individual contractors. In general, the preliminary procedure of construction consists of establishment of line and grade, excavation of a trench by power hoe or trenching machine, and shaping of the bottom of the trench to a semicircular shape. Concrete is then placed in the bottom of the trench and the invert formed by a "boat" or semicircular steel form towed by suitable winches or tractors (fig. 62). The invert portion of the pipe is usually given a steel trowel finish, which completes this operation.



A. Excavation for unreinforced cast-in-place, 36-inch-diameter concrete irrigation pipe. This trench was excavated in sandy loam soil in two passes by a rotary-bladed excavator built by a local contractor.



B. Placing and shaping 150° section of pipe invert. One man pulls "tub" or "boat" and second man on form rocks it from side to side with his feet.

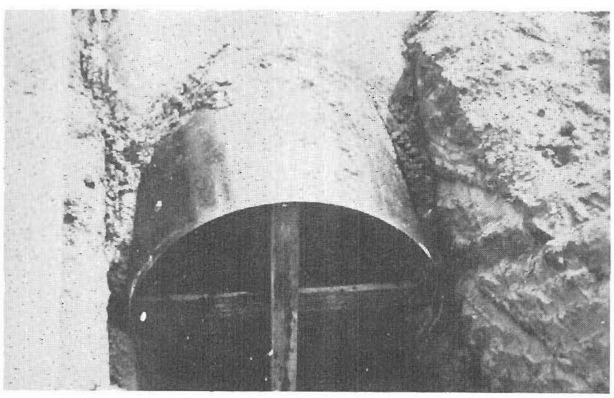
C. Finisher working out imperfections in compaction of invert concrete, and troweling surface.

Figure 62.—Placing lower portion of unreinforced cast-in-place concrete pipe. P214-D-33702, 33705, 33706.

Upon completion of the invert, the top half of the pipe is placed, preferably before the concrete in the invert has set. Narrow pieces of lumber (usually 1- by 4-inch) are placed longitudinally along the invert to serve as a walkway and a bearing plate for timber struts which support the forms for the upper portion of the pipe. These forms, usually 3- or 4-foot lengths of 20-gage or heavier sheet steel or aluminum, circular in section, are then placed and the concrete for the upper part of the pipe placed (fig. 63). The exterior portion of the upper part of the pipe is roughly finished by hand.

(b) *Monolithic Unreinforced Pipe.*—A method of constructing unreinforced cast-in-place concrete pipe monolithically, using a traveling form with which the pipe can be constructed in one operation, has been developed by a California contractor. The Bureau first experimented with this type of pipe on the Orland project, California, in 1954, constructing 36- and 48-inch-diameter concrete conduit. It has also closely followed the installations on the Salt River project, Arizona, where extensive use continues to be made of the monolithic pipe for lateral systems by the Salt River Valley Water Users' Association.[1]

[1] Shipley, H., "Cast-in-Place Pipe for Irrigation," Western Construction News, November 1957, vol. 32, No. 11, pp. 48-55.



A. Form in position for placement of upper 210° section of pipe. About an hour after invert is placed (see fig. 62) 3-foot lengths of curved, 16-gage, sheet steel are set with spreader and support as shown.

B. First batch of concrete being placed on forms. This patch is wet, up to 6- or 8-inch slump, and well spaded into top of invert concrete at both sides of pipe to prevent a leaking cold joint. All other concrete is 2- to 3-inch slump. All concrete contains 60 percent sand and 6 sacks of cement per cubic yard.

Figure 63.—Placing upper portion of unreinforced cast-in-place concrete pipe. P214-D-33704, 33703.

Construction of the pipe is accomplished by a steel sled (figs. 64 and 65), which closely fits the excavated trench. On the sled are mounted an inside form; an outside top form; a hopper; and the necessary operating machinery, including a gasoline engine, a hydraulic pump, spading and vibrating equipment, and a cable drum. The gasoline engine drives the hydraulic pump, which in turn actuates a hydraulic vibrator and a hydraulic-motor-operated winch for providing motion to the sled.

The inside form, outside top form, and hopper are one assembly and are supported by and attached to the sled by hinges to permit the form to negotiate curves in alinement. The hydraulic vibrator is mounted on the inside of the bottom forms; and a spading mechanism, consisting of a spade-shaped tool on each side of the hopper, is moved up and down by a geared connection to the gasoline engine. The device moves the concrete down and around the form while the vibrator consolidates it.

Collapsible forms, which are fed into the front of the hopper assembly, are fastened together by hooks at the top of the form and are held in place by struts resting on the invert. These struts must be placed by hand as the equipment progresses along the trench. The arc of the forms is approximately two-thirds the circumference of the circle, and different size traveling forms are required for each corresponding size of pipe. A completed reach of pipe before backfilling is shown in figure 66.

More recently, a new development in the construction of cast-in-place, unreinforced concrete pipe has occurred. Another manufacturer has introduced a machine for the fabrication of pipe somewhat similar to that described above. Instead of steel forms for the support of the top portion of the pipe, the new development utilizes a tube of balloon cloth inflated pneumatically with about 4 pounds per square inch of pressure. This method of construction has been demonstrated also on the Salt River project (fig. 67).

The monolithic pipe, which is cast in place in one operation, avoids the possible formation of cold joints between the invert and upper sections of the pipe where these are placed in two

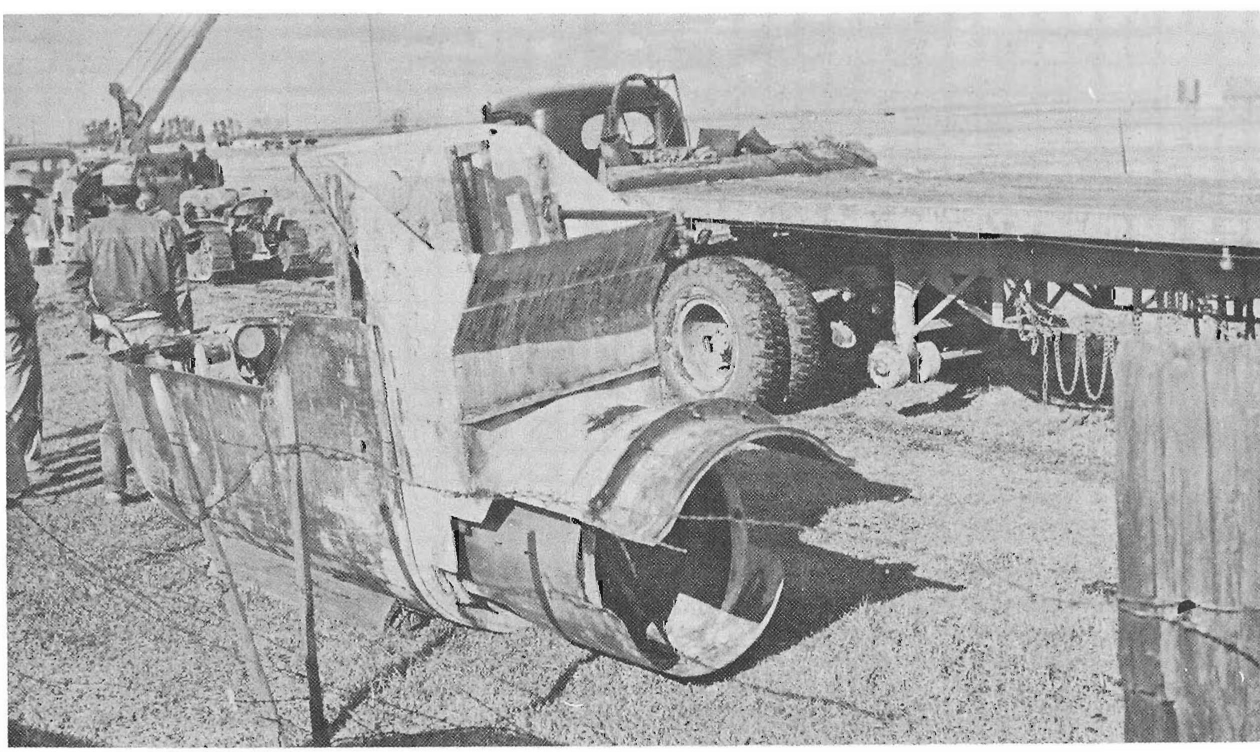


Figure 64.—A monolithic unreinforced concrete pipe machine. CH-414-92.

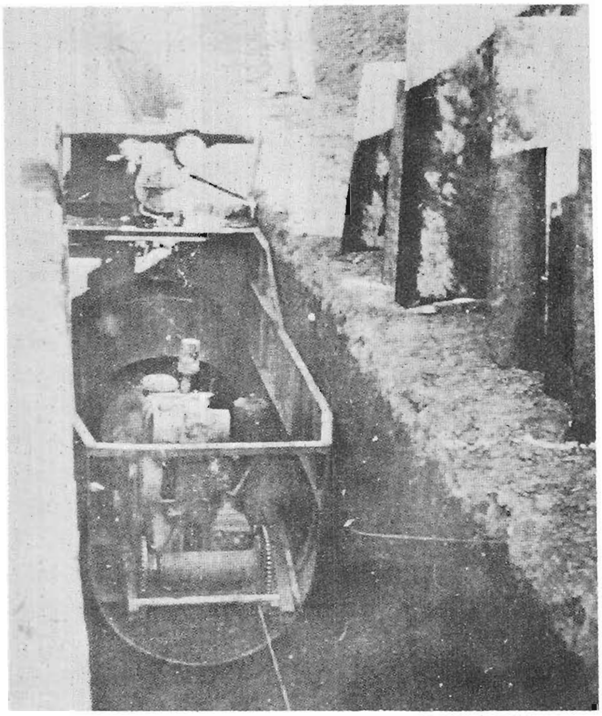


Figure 65.—A monolithic unreinforced concrete pipe machine in operation in an excavated trench. CH-357-62.

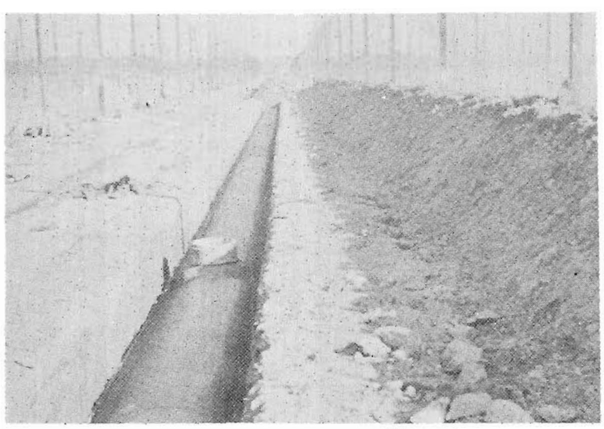


Figure 66.—A completed section of monolithic unreinforced concrete pipe. CH-357-17.

operations, and, accordingly, eliminates one plane of weakness. All unreinforced cast-in-place concrete pipe has its limitations and this applies to the monolithic pipe as well. It is not competitive in many respects with precast pipe. So far, the cast-in-place pipe has been considered primarily as a substitute for canal linings on small laterals where the hydraulic head does not exceed 15 feet. It is not recommended for use under higher hydraulic pressure nor as a replacement for conventional precast reinforced concrete pipe where external loading will be high, and shock, such as that from heavy traffic, is likely to be encountered.

Figure 67.—An inflated tube of balloon cloth serving as an interior form for cast-in-place, reinforced, monolithic, concrete pipe. P25-D-15286 and P25-D-15284.

Cast-in-place pipe is not economical to construct in soils such as sand that are not stable enough to retain a firm vertical-sided trench section. Short reaches of unstable material may be crossed by excavating a wide trench, refilling with stable material, and trenching to fit the machine.

(c) *Limitations.*—The minimum size of cast-in-place pipe that can be constructed with the steel forms is controlled by the necessity for a workman inside the pipe to set and remove the supports for the interior forms. The maximum practical pipe size will undoubtedly depend upon performance as well as competitive cost with other types of pipe or canal linings. One manufacturer of the patented, monolithic pipe machine reports that 60-inch-diameter or larger pipe is practical so far as equipment and procedures are concerned.

(d) *Advantages.*— Any underground water carriage or distribution system has several distinct advantages. Less right-of-way is required; some maintenance problems common to open canals and lateral systems, such as weed and silt removal, are eliminated; the moss problem is reduced; the drowning hazard, especially in heavily populated areas, is eliminated; lands formerly occupied by open canals and laterals can be placed in crops; and the general weed problem in these locations can be better controlled.

(e) *Cost.*—Construction cost data supplied in 1960 by the Salt River Valley Water Users' Association, which has been the largest user of monolithic concrete pipe on a Bureau of Reclamation project, are compared in the following table with the cost of precast pipe and unreinforced portland cement concrete lining for a canal or lateral having an equivalent capacity:

Inside diameter, inches	Wall thickness, inches	Construction cost in dollars per linear foot			
		Monolithic pipe by		Precast pipe by contract	Slip-form portland cement concrete lining by contract
		Project forces	Contract		
30	3	4.03	6.25	8.62	3.35
36	3½	4.84	7.25	10.49	4.16
42	4	5.69	8.25	13.61	5.27
48	4½	6.06	9.75	16.14	5.61

Chapter IX

Economics of Canal Lining

60. *General.*—Canal linings are expensive. In usual terrain a lined canal may cost twice as much as an equivalent unlined earth canal; however, in rough terrain a lined canal may be less expensive in first cost than an unlined canal because of savings in excavation and structures in the smaller sections of a lined canal. In view of the probable increased first cost, a decision to use lining needs justification. Justification may be based on intangible benefits, long-range tangible benefits, or a combination of tangible and intangible benefits. In addition to a possible saving in construction costs, the more common benefits derived from canal lining are (1) a saving in water, (2) reduced damage to lowlands from seepage or reduced drainage cost, (3) greater safety, and (4) reduced operation and maintenance costs.

61. *Economic Analysis.* — Although any one of the above considerations may be the fundamental reason for lining a canal, they all are often involved in the justification. Frequently, it is possible to justify the adoption of a lining program by the consideration of the values assigned to tangible benefits, but when this is not so, it is desirable to determine what monetary value can be assigned to intangible benefits. Such a procedure will bring the intangible benefits into sharper focus for determining the feasibility of lining. To show economic feasibility, the present worth of the annual tangible and intangible benefits resulting from the installation of lining must be equal to, or greater than, the cost of the lining.

Decision to line a canal, based on the expectation of tangible benefits, would be justified principally by savings in water, reduced operation and maintenance costs, and reduced water damage from flooding or seepage or reduced drainage requirements. The reduction of water losses and the prevention of land damage from seepage are gaining in economic importance as complete utilization of natural water resources pproached and as the availability of good agricultural land assumes a more vital role in our national economy.

Seepage losses can be determined or estimated as discussed in chapter III. Operation and maintenance costs for lined and unlined canals can sometimes be secured from cost data on existing canals in the same project or in projects operating under somewhat similar climatic, geographical, and agricultural conditions. These costs for lined canals will vary with the age of the lining, and an average cost should be chosen. The total cost of the lining can be estimated with reasonable accuracy and this total cost compared with the present worth of the annual benefits computed for the expected life of the lining. Although this method is preferred, the opposite approach may also be used by computing a sinking-fund annual cost of the lining and comparing this cost with the annual benefits derived from the lining. However, it is not valid if both methods are used (i.e., present worth plus a sinking fund to replace the lining in some number of years) because this, in effect, doubles the cost of the lining for the first period of useful life.

62. *Importance of Lining During Original Construction.*—The importance of including canal lining (or provision for future lining) in the original construction plans and designs of an irrigation project, provided studies have demonstrated its economic feasibility, cannot be too strongly emphasized and has been stressed in previous discussions. It is only during the planning and designing stages that full advantage can be taken of the many benefits of the installation of a canal lining. When lining is included in the original plans and designs, the cost of the lining might be justified in consideration of reduced storage and diversion requirements, smaller canal sections, smaller and possibly fewer canal structures, reduction of pumping costs where pumping is necessary, and a possible reduction in the right-of-way requirements.

Seepage losses from canals and laterals represent a loss to the intended user not only of valuable irrigation water, but also a considerable loss in the costs of additional construction from which no return is received on the investment. Storage reservoirs and dams must be constructed of sufficient size to impound not only the useful water but also the water that will be diverted in transit by seepage from the canals. The canals and laterals must be constructed with sufficient capacity to transport this excess water that will not be used on the project. A reduction in the canal capacity, made possible by the prevention of seepage, results in smaller structures and a smaller canal cross-sectional area being required, which in turn, represents a material decrease in all quantities.

Canal lining, in addition to permitting smaller, less costly structures, also may reduce the number required. The maximum permissible velocity in an unlined canal is limited to prevent erosion of the section, which in turn, limits the permissible canal gradient. Thus an unlined canal traversing slopes may require the use of drops or chutes to avoid destructive erosion. A more permanent, hard-surface lining, because of its higher permissible velocities and steeper gradients, might eliminate the need for many of these structures.

It is not difficult to compute the savings in operating costs that could be realized through canal lining where appreciable seepage losses are suffered from unlined canals served by pumping plants. Large seepage losses from canals served by high-lift pumping plants will provide strong economic justification for providing an impervious lining to prevent those losses. In projects where lining was not included in the original plan, later lining can save only power costs. Where the lining is included in the original planning, both plant size and standby capacity charges are reduced.

In those instances where right-of-way requirements involve the acquisition of expensive agricultural land, the reduced requirements for a lined section are important. The wider right-of-way for an unlined canal, in addition to having a high initial cost, imposes a heavy toll on the land it serves. It has been estimated that the land 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 hard-surface-lined canal would reduce the right-of-way requirements by permitting the use of a smaller canal through elimination of the seepage losses, but more importantly through use of the smaller water section allowed by improved hydraulic conditions and higher velocities.

63. ***Lining Operating Canals.*** — After the construction of a project has been completed, it may not be economically feasible to install canal lining even though the need is evident and great, simply because the lining cannot be justified without the benefits discussed above. The benefits and savings attributable to canal lining to be installed on an operating project will be limited to lower seepage losses with consequent greater use of available water, recovery of water-logged land, lower operation and maintenance costs, improved drainage conditions, less danger of canal failure, or a combination of these.

64. ***Location of Seepage.*** — Where operating canals are not lined or where old linings have deteriorated badly, excessive seepage may be evident from the water standing in adjacent fields. It is not safe to assume, however, that the water escapes from the reach of canal immediately adjacent to the water-logged land. Seepage water often travels long distances, emerging far from where it escaped. Or, seepage water may sink through underlying strata to escape in natural drainage channels, thus leaving no direct evidence of loss. Seepage measurements are necessary, therefore, to identify the section in which the greatest losses occur as well as to determine the rate of loss, and thereby reduce to a minimum the amount of lining required. It also appears that electrical logging and the use of tracers, discussed in chapter III, may prove of value in locating reaches that are in the greatest need of lining.

65. ***Value of Water Saved.***—Reduction of the loss of water from a canal may be economically important when the water supply available at the head of the canal is limited or when all of the water has to be pumped. Since the amount of leakage and the unit value of the lost water are of primary importance, a measurement estimate of the amount of leakage must be made before the need for lining can be definitely as-

certained. The amount of water loss that can be tolerated before it becomes economical to install a lining will vary with the individual project. Therefore, it is essential that accurate measurements be made on existing projects, and close estimates based on field tests be made for proposed projects, to support the determination of the economic practicability of a lining program.

In considering the quantity of water lost in an unlined canal, it is necessary to differentiate between water that is irrecoverable and water that is recovered as return flow in a canal or lateral at a lower elevation or in a stream for rediversion and reuse. Actually, the only water which is certain not to be recoverable somewhere would be that which evaporates or escapes to the sea. The water that is irrecoverable in a project is a total loss to that project and, if in sufficient quantity, is a strong recommendation for lining; whereas the recoverable water which is picked up as return flow and can be put to a beneficial use on the project does not offer a substantial justification for lining unless it causes or aggravates water-logging of adjacent farmland.

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 crops produced. (The per acre annual assessment for the project is not usually a fair indication of the value of the water, since this reflects the expenses of construction, operation, and maintenance which are not necessarily indicative of, or related to, the water supply or income from farming operations.)

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. Such possibilities would have to be investigated. Obviously, these variable factors affecting the determination of the value of water which might be saved by canal lining necessitate an individual analysis for each project.

With increasing population in a basin, the attendant heavy demands on the natural water supply create a new economic problem in conservation. In this broader plan a project having a water supply adequate for both its use and seepage may be confronted with the necessity of lining its conveyance system to conserve water for beneficial use off the project. A method of equitable payment for the cost of the lining and the right to the salvaged water may require adjudication; but, since the new demand may be domestic supply and hence have a higher priority, the problem must be considered and fairly resolved. In evaluating the economics of lining, this higher priority off-project use will undoubtedly be based on a higher value of water and hence show greater benefits from the lining than were previously possible.

66. ***Drainage Benefits.***—The extent of the influence of canal seepage on the land drainage problem is debatable and difficult to determine in most cases, and the effects of seepage are not always readily evident. The seeping water from canals on higher ground often disappears into a pervious underground stratum and reappears in a low-lying area at some distance from the canal. Moreover, the water-logging of land may be the result of both canal seepage and deep percolation from irrigation operations on higher terrain. It is doubtful, in such instances, if lining the canal to prevent seepage losses would eliminate the water-logging, but it would certainly reduce the extent and cost of operation and maintenance of the drainage system.

The effects of canal seepage are most noticeable where land adjacent to the canal has been reduced to a swampy condition and rendered virtually worthless. The monetary loss from this water-logged land or the cost of drainage to remedy the situation is directly chargeable to canal seepage and, as such, offers convincing justification for canal lining. Similarly, the prevention of damage to railroads, highways, mines, and other improvements from seepage would support a justification for lining the canal. If the overall cost of drainage to alleviate or prevent damages from seepage exceeds the cost of lining the canal, the lining is economically justified.

The cost of constructing, operating, and maintaining a large or lengthy drainage system for the sole purpose of picking up main canal seepage losses may be materially greater than the cost of lining the canal to prevent those water losses, even if the value of the water lost were disregarded. The disposition of the seeped water picked up in a drainage system would influence the economic justification of canal lining in some cases. If the drainage water has to be pumped before disposal, the prevention of the seepage losses by canal lining becomes considerably more important. Thus, the land drainage problem exerts a very marked influence on the justification of the installation of canal lining to prevent seepage.

67. ***Protection of Canal from Failure.***—Canal lining may be justified because it increases the safety of a canal located on fill where burrowing animals are prevalent, or where the canal section is in a combination of rock and earth. In appraising the economic justification for the installation of lining in these cases, lining becomes competitive with other means of taking care of the situation such as constructing wider banks and providing toe drains. An appraisal of the necessity and value of lining to increase safety would consider the possibilities and magnitude of the following items in the event of a complete failure of a portion of the canal embankment: (1) loss of life, (2) damage to improvements, such as farm and town buildings, railroads, highways, utilities, irrigation works, and mines, (3) loss of crops on account of delay in water delivery, (4) loss in power revenues if the canal is operated for power purposes, and (5) damage to the canal itself on account of the high velocity of water rushing through the breach.

68. ***Increased Capacity.***—The necessity of providing increased capacity in a canal that has been in operation a number of years, in order to meet a greater demand for water or to serve a larger area of land, always presents a problem in determining the most economical and practical method of obtaining the increased capacity. If the required additional capacity is relatively large, the only means of attaining the objective would be enlargement of the canal section. However, if the required increase in capacity is relatively small and the operating canal is unlined, the installation of a lining may offer an economical solution to the problem. The prevention of canal seepage losses and the improved hydraulic properties of a hard-surface canal lining, over an unlined section, might provide the increased capacity needed at a lesser cost than enlarging the canal section.

When a canal is located through highly developed property such as an urban area, it may be necessary either to place it in a closed conduit or to line the canal in order to reduce the section to the minimum practicable.

69. ***Reduced Maintenance.***—Many existing irrigation projects have more than adequate water supplies, so, unless there is a serious drainage problem or frequent canal failures, there is usually little interest on the part of the water users in the installation of canal lining. However, one of the beneficial results of a properly designed, constructed, and lined canal, even on projects having an abundant water supply, i the saving in maintenance costs.

In any evaluation of the economic benefits of a canal lining with reference to the costs of maintenance, it must be recognized that the application of the factors involved will be dependent upon the type of lining being considered. For example, if a lining is being considered for a canal, either new construction or as an addition to existing facilities, the economic studies for using a hard-surface type lining can properly include benefits anticipated from reduced costs of weed control, less danger from burrowing rodents, less silt removal, and other conditions which a rigid, high-quality lining will provide. On the other hand, the economic studies for an earth lining or buried membrane cannot include many of these factors and must rely primarily on the value of seepage prevention for justification. The type of lining, therefore, determines some of the factors which may be considered in the economic analysis.

One of the largest items of recurring maintenance costs on many canal systems is weed control and the removal of weeds and water-loving plants from the canal section. Such hi quality, hard-surface linings as concrete, sh crete, and to a less extent, asphaltic concrete, being practically impenetrable by weeds and water-loving plants, would greatly reduce the cost of weed control and removal from the canals.

In areas where rodents and crayfish are prevalent, many canal failures occur each year in unlined canals as a result of holes burrowed in the embankments. The cost of repairing the canals, the resultant possible loss of crops due to lack of water, and, in some cases, the property damage inflicted by water escaping from the canal break may be of considerable magnitude. Inasmuch as any of the hard-surface linings are practically impenetrable to burrowing rodents, and asphalt membrane lining appears to deter such action, their use offers increased safety from canal breaks resulting from such causes.

Any of the hard-surface linings, which will permit high water velocities, could reduce maintenance costs by preventing both deposition of silt in the canal and erosion of the canal during the operating season. These lining materials are highly resistant to erosion, and if the slope of the lined canal is sufficient to use high velocities, the necessity for routine silt removal is greatly reduced or eliminated. For the same reason, the cost of maintaining the canal section against erosion would be similarly affected.

Buried membrane linings or thick compacted-earth linings, utilizing a substantial gravel blanket, will prevent or substantially reduce erosion which might be a problem in an unlined canal or lateral. The gravel blanket will also discourage the burrowing of gophers, muskrats, and crayfish, as well as provide less favorable conditions for weed growth near the water's edge. Thus, it is believed reasonable to assume that the cost of these several factors of maintenance would be substantialy reduced by the installation of one of these canal linings, and such benefits as seem reasonable of attainment should be considered in the economic analysis of the feasibility of installing canal lining either in new project construction or rehabilitation.

70. ***Cost of Maintenance.***—It is possible to obtain rather accurately the construction cost of a canal lining. The annual savings or benefits to be derived from lining, however, are more difficult to evaluate and must include an estimate of the difference in annual maintenance cost between a lined and an unlined canal. Unfortunately, maintenance cost data frequently are inconclusive and incomplete so far as being explicit as to just what the costs include. On water-user-operated projects (including most projects constructed by the Bureau), time and personnel are seldom available for making a careful breakdown of individual maintenance cost items. The costs may be for lining repair only, or they may include the cost of cleaning silt, sand, and other debris from the canal perimeter, etc. Separation of costs for these various maintenance activities is difficult, from most records received.

Good maintenance cost data should include the expense necessary to keep the channels in the condition they were in when transferred from a construction to an operation and maintenance status. Costs incurred for supplemental construction and other items of expense which are properly classed as improvements should be considered as completion of construction rather than maintenance. Weed control expense should preferably include only that expense incurred for control of aquatic and land type weeds within the canal or lateral prism. However, since it is difficult to segregate the cost for control of land type weeds on right-of-way, roads, and outside banks, the total cost for land type weeds is usually included in the maintenance data. This latter cost, although common to all lined or unlined canals, may vary considerably because of seepage through the banks of unlined or ineffectively lined canals, which may stimulate the growth of weeds outside of the canal prism.

Except for repairs that may be necessary to correct faults that develop soon after construction, lining maintenance costs are usually rather nominal. However, ultimately a cleaning job or an extensive repair becomes necessary and the cost of this work may be appreciable, even approaching in some instances the original construction cost. For this reason cost records must cover long periods to be representative. In comparing costs between projects, factors such as climate, period of operation, type of terrain, and service conditions are generally so variable that suitable parallels do not exist. Other variable factors include water velocity, capacity, available construction materials, thickness and types of linings, side slopes, effectiveness of drainage, leakage, rodents, cattle, wind, and stability of soils in adjoining fields. All of these factors have a bearing on maintenance costs, and many are difficult to evaluate. In view of the number and variability of the factors enumerated, it is not surprising that reliable average maintenance cost data that can be used confidently for estimating purposes are difficult to establish.

Though comparison of costs between projects is generally not practicable, operating and maintenance costs for lined and unlined canals can sometimes be secured from cost data on existing canals on the same project or on projects operating under similar climatic, geographical, and agricultural conditions. One project justified lining and the use of buried concrete pipe in lieu of lining on the basis of just such a study, proving that weed control costs, primarily, would justify the use of linings and buried pipe.

The type of lining to be used must also be considered from the standpoint of useful life considering maintenance cost and eventual replacement cost. The lower cost type linings constructed by the Bureau have been in service for only a relatively short period of time, linings of unreinforced portland cement concrete, buried asphalt membranes, and thick compacted earth having been in service only since 1948-50. Replacement of only a very few of the linings of the types mentioned is contemplated in the near future. There is reason to believe that most of the linings will be effective for many more years; but an assumption must be made in computing the maintenance cost or replacement cost, if an annual cost for feasibility estimates is to be prepared.

71. ***Summary of Economic Feasibility.***—Construction costs, the value of water, drainage problems, protection from failure, and increased capacity are factors that can usually be evaluated with reasonable accuracy. Formulas proposed for determining the feasibility and practicability of lining an unlined canal have included these factors, and also factors which consider the life of the lining and its maintenance. As has been pointed out previously, however, these latter type factors must be assumptions based upon limited information. Further, for proposed linings for new canals, the formulas should properly include factors to reflect such items as reduced storage and diversion requirements, smaller and fewer canal structures, and smaller canal sections that would result from lining. As these factors are all difficult if not impossible to evaluate with accuracy, the formulas have generally proved of doubtful value, and proof of feasibility by the use of formulas is not attempted by the Bureau. Rather, consideration is given to the individual and specific factors inherent in a given project or area to be benefited.

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