Design Standards No. 3

CANALS AND RELATED STRUCTURES

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7 CROSS DRAINAGE AND PROTECTIVE STRUCTURES

8 PIPE DISTRIBUTION SYSTEMS

9 BRIDGES

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BUREAU OF RECLAMATION

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Other revisions:

Summary of changes:

This Design Standards has been completely revised and updated to present current Bureau practice in the design of canals and related structures. Among the more significant changes or additions are the following:

1. A statement of the Bureau's new waterway policy, which places emphasis on lining waterways or placing them in pipes.

2. A description of several types of automatic control features, and a discussion of the importance of considering automation of a canal system in the planning and design stage.

3. An increase in the maximum concrete compressive stress from 3,000 to 3,750 pounds per square inch.

NOTE: This is a complete replacement for Design Standards No. 3.

Approved:

Wm H Wolf
Acting Chief Engineer

December 8, 1967
Date

(To be filled in by employee who files this release in appropriate folder.

The above change has been made in the Design Standards.

Signature

Date

UNITED STATES DEPARTMENT OF THE INTERIOR

BUREAU OF RECLAMATION

DESIGN STANDARDS NO. 3

CANALS AND RELATED STRUCTURES

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GENERAL REQUIREMENTS

INTRODUCTION

.1 The following paragraphs deal briefly with total water demand and flow capacity requirements, the two general requirements which must be fulfilled by any type of canal and lateral system. Except where otherwise noted, the material included herein relates to canals and laterals to be used for irrigation. In the structural designs selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated.

WATER DEMAND

.2 The total water demand requirement to fulfill the project purpose must be provided by the irrigation canal system. It is primarily based upon four factors: acreage to be irrigated, estimated duty of water, estimated seepage losses plus an allowance for operational waste, and evaporation loss.

Acreage to be Irrigated

A. The net acreage to be irrigated is based on detailed land classification and preliminary canal location surveys. The exact acreage cannot be computed until final canal locations are made.

Duty of Water

B. The duty of water, or the quantity of water required per acre, to be used for preliminary studies of a canal system, may best be estimated from records of the use of water under similar conditions on similar areas and crops. A detailed study of the soil and subsoil characteristics, irrigable land, drainage conditions, methods of water application, nature of crops, rainfall and evaporation, and other pertinent factors may be required. (See also Part 2 of Volume V, Irrigated Land Use, of the Reclamation Manual, regarding land classification.)

Seepage Losses

C. Seepage loss is usually expressed in cubic feet per square foot of wetted area in 24 hours. It is generally estimated from the loss in water depth in a reach of canal having uniform slope and cross section. For preliminary estimates it may be assumed that, in typical unlined earth canals under usual conditions, about one-third of the total water diverted will be lost by seepage, operational waste, and evaporation. Reported seepage losses frequently include a certain amount of structure leakage, operational waste, and overdelivery to the irrigators. Seepage may at times constitute a gain to the canal rather than a loss, if the ground water is sufficiently high and other natural factors are advantageous. (Seepage of irrigation water from higher lands is sometimes a contributing factor to high ground water along a canal.) Thus, an accurate prediction of seepage loss is extremely difficult to make and the results are at best uncertain. The prediction of seepage must therefore be based on judgment within the limits of existing data and natural factors. The Moritz formula suggests computations of total seepage loss in cubic feet per second (cfs) per mile of canal as follows:

$$S = 0.2C\sqrt{\frac{Q}{V}}$$

where

S = loss in cfs per mile of canal,
Q = discharge of canal in cfs,
V = mean velocity of flow in fps, and
C = cubic feet of water lost in 24 hours through each square foot of wetted area of canal prism.

Observations on eight different projects gave the following average figures for the value of C in earth canals. These factors are suitable

1.2D

GENERAL REQUIREMENTS--Continued

Seepage Losses (Cont'd.)

for rough preliminary estimates, but measurements have shown that actual seepage losses vary widely within each of the general soil types. For design purposes, therefore, it is usually necessary to make estimates of seepage losses in questionable areas on the basis of field tests.

Type of material	Value of C
Cemented gravel and hardpan with sandy loam	0.34
Clay and clayey loam	0.41
Sandy loam	0.66
Volcanic ash	0.68
Volcanic ash with sand	0.98
Sand and volcanic ash or clay	1.20
Sandy soil with rock	1.68
Sandy and gravelly soil	2.20

Seepage losses from properly constructed concrete-lined canals should normally be relatively small. However, subsequent partial failure or poor construction or maintenance of the concrete lining may result in large losses. Other types of lining are susceptible to varying amounts of seepage loss, depending on the type, quality of construction, and related natural factors. The possibility of appreciable losses from lined canals should be kept in mind when preparing initial estimates of water requirements. Technical Bulletin No. 1203 entitled "Measuring Seepage From Irrigation Channels," issued in September 1959 by the Agriculture Research Service, U.S. Department of Agriculture, will prove helpful in estimating seepage losses.

Evaporation

D. Evaporation from canals and laterals is usually such a small quantity compared with seepage that it may be neglected. However, where there are reservoirs along a canal, evaporation should be considered.

FLOW CAPACITY

.3 Flow-capacity requirements of various parts of a system are determined and used as a basis of conveyance design. These flow capacities must satisfy the water demand at the various points in the system. The maximum demand may generally be estimated at 125 to 150 percent of the average demand. Systems operating for a 12-month season may require a capacity large enough to carry, in the maximum month, from 10 to 15 percent of the total annual demand. Those operating for a 7-month season may require a capacity large enough to carry, in the maximum month, from 20 to 25 percent or more of the total annual demand. However, capacities should be made adequate to serve the maximum 10-day demand. The actual maximum demand should be determined by detailed analyses of individual projects.

CANAL LINING POLICY

POLICY

.4 It is the policy of the Bureau of Reclamation, in order to conserve water and to secure other benefits, to consider fully the lining or placing in pipe of all constructed waterways for the conveyance and distribution of project water supplies. In those instances where the recommendations do not call for lining or pipe, full justification for using an unlined waterway will be required. On unauthorized projects for which field studies are not complete, this policy shall be adopted at once. On presently authorized but unconstructed projects, and for those projects on which field investigations are completed, this policy shall be adopted in advance planning studies.

CANAL LINING POLICY--Continued

.5 Justification for using unlined waterways is sometimes very complicated because of the large number of factors to be considered. Consideration must be given to seepage rates with and without lining, the value of water saved, operation and maintenance costs, drainage costs or value of land taken out of cultivation by seepage, canal size, reservoir size, right-of-way, allowable velocities, structure costs, and the various types of lining or pipe correlated with the other conditions. Considerations should also include intangible factors inherent in a given project to be benefited and values assigned these factors whenever possible. The Bureau's publication "Economic Justification for Canal Lining in Irrigation Distribution Systems" presents procedures and guidelines which can be used in making economic studies. JUSTIFICATION

UNLINED CANALS OR LATERALS

.6 An unlined canal or lateral is defined as an open channel excavated and shaped to the required cross section in natural earth or fill without special treatment of the wetted surface. Compaction of bank or fill material for the purpose of stabilization is not considered as a lining operation. See Figure 1. DEFINITION

.7 The cross section selected for a canal or lateral should be such as to carry the maximum capacity discussed in Paragraph 1.3 and should satisfy the proper relationships between bottom width, water depth, side slopes, freeboard, bank dimensions, and future operation and maintenance. The ratio of bottom width to depth usually ranges from 2:1 for small channels to 8:1 for canals with capacities of about 10,000 cfs. The side slopes of a canal depend upon the stability of the material in which it is constructed. Inside slopes of 1.5:1 or 2:1 (horizontal to vertical) are practically standard for earth canals under ordinary conditions; on sidehill locations the inside slope of the uphill bank may be made steeper, if the material will stand, to avoid excessive excavation. CROSS SECTION

Operation and maintenance problems should be considered in the selection of canal cross-section characteristics, such that an overall economy of initial cost and maintenance expense may be obtained. Figure 2 is a curve showing suggested nonsilt, nonscour velocities for clear water running in canals. These velocities ordinarily require modification in Bureau designs due to the variability of soils and sediment in the water. (See Paragraph 1.12.) Figures 3A and 3B include tables of typical earth sections for irrigation canals with 1-1/2:1 and 2:1 inside slopes, respectively.

.8 A canal should divert from a supply source at sufficient elevation (static or pumped) to reach, with proper gradients and by the most economic route, the land to be irrigated. The water section may, at various points along the canal, be partially or entirely in either cut or fill, depending on the location selected to satisfy requirements of safety, structural design, distribution, and least annual cost including maintenance. If the water section is partially or entirely in fill, consideration should be given to the use of compacted embankments or other suitable means of preventing excess seepage and percolation through the fill. At turnouts the canal water surface must be high enough to permit irrigation of the land. LOCATION

.9 The allowable curvature for unlined canals depends on the size or capacity, velocity, soil, and canal section. A small lateral, 20 cfs or less in capacity, flowing at low velocity, 2 feet per second or less, will require only a very small radius of curvature. A large canal, 2,500 cfs or more in capacity, will require a much larger radius regardless of the velocity. CURVATURE AND VELOCITY

Velocities in unlined canals ordinarily vary from 1.0 to 3.5 feet per second. While not an extreme mathematical variable, velocity does have appreciable

UNLINED CANALS OR LATERALS--Continued

CURVATURE AND VELOCITY (Cont'd.)

influence on the radius of curvature required. Water flowing at a velocity of 3.5 feet per second will cause more erosion and develop larger waves than water flowing at 1.0 feet per second around the same radius curves.

The character of the soil has a decided influence on the radius of curvature required. Soil may range from firm to shifting, and its stability may be quite sensitive to the curvilinear flow of the water.

In order to develop a satisfactory rule for determining the radius of curvature required, it is necessary to establish some ratio or ratios of radius of curvature to dimensional elements of a canal section. Since the factors already discussed vary simultaneously with dimensional elements, it is possible to establish such ratios only within general limits. A suggested rule is that the radius to the canal centerline should be from three to seven times the water surface width (the larger ratios for the larger capacities), depending upon the size or capacity of the canal, the soil characteristics, and the velocity. Consideration of all factors is required for an acceptable solution.

FREEBOARD .10

Freeboard in a canal will normally be governed by considerations of the canal size and location, velocity, storm-water inflow, water-surface fluctuations caused by checks, wind action, soil characteristics, percolation gradients, operating-road requirements, and availability of excavated material. The typical earth sections listed in Figures 3A and 3B include the recommended minimum freeboard, and the height of bank above the water surface as shown in Figure 4 may also be used as a guide. These illustrations are based upon average Bureau practice, and it is emphasized that they will not necessarily serve for all conditions. Greater bank heights than those needed for hydraulic reasons may be used where excess excavation exists, provided that undesirable conditions with respect to right-of-way, maintenance, structures, and design elements are not thereby introduced; in the latter event the excess material should be disposed of in some other manner. The use of excessively high banks, particularly on sidehills, increases the hazard of bank sloughing.

BANK TOP WIDTH AND BERM .11

Banks used as operation and maintenance roads may range from 12 feet wide for canals with a capacity of 100 cfs to 20 feet and wider for canals with a capacity of 2,500 cfs or more. Access to waterways should always be provided and is usually accomplished by an operating road on the bank. Where the operating road is not on the bank, the width of bank may be as small as 3 feet for the small laterals. If borrow material is required to build canal or lateral banks, such borrow should be kept to a minimum and the borrow pits should be drained. Operation and maintenance roads should be located at a minimum height above the water surface, to facilitate maintenance of the canal.

Berms reduce bank loads which may cause sloughing of earth into the canal section. Steeper slopes may be used above the berm, provided the material is stable.

Canal and lateral banks should be finished so that, even where there are no regular operating roads, the lines of the bank are regular enough to permit the use of power mowers and other power equipment to control the growth of weeds and maintain the canal sections.

Waste banks and cuts should be made to blend with the surrounding terrain where possible. Every effort should be made to obtain an appearance which does not disrupt the natural terrain and beauty.

FLOW FORMULAS .12

The Manning formula is generally used for open-channel flow. The formula is as follows:

UNLINED CANALS OR LATERALS--Continued

FLOW FORMULAS (Cont'd.)

$$V = \frac{1.486}{n} r^{2/3} s^{1/2}$$

where

V = velocity of water in feet per second,
s = slope of energy gradient in feet per foot,
r = hydraulic radius (water area divided by wetted perimeter), and
n = coefficient of roughness.

A roughness coefficient "n" of 0.025 is generally used for earth canals with capacities less than 100 cfs and 0.020 or 0.0225 for larger canals. Recommended coefficients of roughness are given in the Bureau's Hydraulic and Excavation Tables.

For uniform channel sections covered with sand and gravel, the Manning's "n" may be determined by the Strickler equation,

$$n = 0.0342\, d_{50}^{1/6}$$

where d_{50} equals the size in feet for which 50 percent of bed material by weight is finer.

In unlined canals, the velocity should be such as to prevent cutting of the canal prism or deposition of silt. The maximum velocity allowable to prevent cutting or the minimum allowable to prevent silt deposition will depend upon soil characteristics, sediment in water, and natural factors, but general limits can be set down from experience. The Kennedy formula for sediment-laden water flowing in a bed of similar material is,

$$V_s = CD^{0.64}$$

where

V_s = velocity for nonsilt and nonscour,
D = depth of water in feet, and
C = coefficient for various soil conditions.

Values for the coefficient C are as follows:

For fine, light, sandy soil	0.84
For coarser, light, sandy soil	0.92
For sandy, loamy silt	1.01
For coarse silt or hard soil debris	1.09

A suggested modification of the Kennedy formula for clear water is,

$$V_s = CD^{0.5}$$

Figure 2 shows the relationship of V_s to D for various water depths.

Sand and gravel may be required for protection of the banks against wave action. For clear water flows over these sand and gravel protective layers and other noncohesive granular beds, the nonscour velocity is,

$$V_s = 9\, d_{50}^{1/3}\, R^{1/6}$$

UNLINED CANALS OR LATERALS--Continued

FLOW FORMULAS (Cont'd.)

After a channel is in operation for an extended period, heavy concentrations of fine sediments in the flow may cause cementing (cohesion) of some fine sands in the bed. This often results in an increase in the nonscour velocities of up to 50 percent.

Bureau canal designs are based on the capacity required to supply the maximum 10-day demand period. This results in canals and laterals being operated for most of the year at below design capacity and usually with checks to make deliveries. Because of this, any sediment above colloidal size in the supply water will have to be excluded from the headworks or removed somewhere from the system. Thus canals and laterals are usually designed to avoid scour at maximum discharge, and some provision is made to remove or exclude the sediment at the headworks or remove it from certain reaches of the canal. In transport canals to powerplants or offstream reservoirs, adequate sediment-carrying ability should be provided.

HYDRAULIC BORE .13 Hydraulic bore, which is discussed briefly under Subparagraph 1.23A, may occur in unlined canals. It may be caused by the shutdown of a pumping plant, rapid closure of a check or gate, or as a result of sudden inflow causing a wave in the canal.

LINED CANALS OR LATERALS

DEFINITION .14 Lined canals or laterals may be divided into three groups: hard surface, buried membrane, and earth linings. Hard-surface linings include portland cement concrete linings, shotcrete linings, asphalt concrete linings, exposed prefabricated asphalt linings, brick linings, stone linings, exposed plastic linings, soil-cement linings, and precast concrete linings. Buried-membrane linings include sprayed-in-place asphalt, prefabricated asphalt, plastics, and bentonite. Earth linings include thick compacted earth, thin compacted earth, loosely placed earth, and bentonite soil mixtures. For information on canal linings see current edition of the Bureau publication "Linings for Irrigation Canals."

CROSS SECTION .15 Cross Section.

Hard-surface Linings

A. 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 is the most economical. A semicircle has the smallest perimeter for a given area, but a semicircular section 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 are 1-1/2:1. Steeper slopes may be used on small laterals where the soil materials will remain stable. Hard-surface-lined canals are usually designed with a ratio of base width to water depth of from 1 to 2. Small canals normally have a ratio of nearly 1, while the ratio for large canals may exceed 2. Figures 5 and 6 show standard dimensions and hydraulic properties for small canals with concrete lining. Figure 7 shows normal lining thickness.

The location of the canal bottom with respect to the ground-water table is especially important. If the ground-water table is above the canal bottom, outside hydrostatic pressure may rupture the lining when the canal is emptied or the water surface drawn down. In cold climates the canal bottom must be at least 3 feet above the water table to prevent damage from freezing and thawing. In soils having high capillarity a greater distance above water table is advisable. If hard-surface lining is used

LINED CANALS OR LATERALS--Continued

Hard-surface Linings (Cont'd.)

with high ground water, gravel or tile and gravel underdrains with suitable outlets must be provided to reduce the probability of damaging the lining. Figure 8 shows a flap valve drain outlet for use with a hard-surface lining. The lining must be placed against a stable foundation of existing or compacted material. If expansive clay is present, the treatment consists of overexcavating and replacing with a minimum of 2 feet of nonexpansive material or maintaining a near saturated foundation until the lining is placed. The expansive characteristics of the material will determine the load necessary to confine it. In reaches where expansive clay or high ground water exists, consideration should be given to omitting the lining or relocating the canal. Figures 9 and 10 show typical safety ladders for concrete-lined canals.

Buried-membrane Linings

B. Buried-membrane lining is normally installed only to reduce water loss by seepage. A cover must be provided to protect the membrane from exposure to the elements and from injury by turbulent water, stock, plant growth, and maintenance equipment. The depth of cover depends on cover material, size of canal, water velocity, and canal side slopes. Gravel is generally required at the beach belt in larger canals for protection against wave action. The canal bottom width should be about four times the water depth or greater and the side slopes 2:1 or flatter. Long-radius curves are desirable at the intersection of the side slopes and the bottom to improve stability and to more nearly approach the final shape of the canal section after it has been in operation. When rounded uniformly graded gravels or sands have been used for cover material, 2:1 side slopes have proved to be too steep. There is also danger that the cover material may slough down the bank during placing. Such sloughing may carry the membrane down the slope with it, causing it to crack or tear. Flatter side slopes and placing the cover material on the canal bottom and lower portions of the side slopes first, may prevent damaging the membrane lining. At structures the membrane should be carefully bonded to the cutoff and should be lowered a sufficient distance away from the structure to provide space for extending riprap protection where required. Laboratory tests of the available cover material are desirable in determining the side slopes and mixtures of available materials to be used for best results. Crushed rock or angular cover material should never be placed directly on the membrane because of the danger of puncturing it. Earth, gravelly material, and gravel have been used for cover over membrane linings. A small amount of lean clay will add stability, especially to gravelly materials and gravel. See Figure 11 for details of buried-membrane lining.

Earth Linings

C. Earth linings normally have a 3- to 8-foot thickness on the canal sides, measured horizontally, and a 12- to 24-inch bottom thickness of compacted select material. However, any compacted section 12 inches thick or more is considered to be thick compacted earth lining. Figure 12 shows typical earth-lined sections. Thin compacted earth linings usually have a 6- to 12-inch layer of compacted cohesive soil with a protective cover of 6 to 12 inches of coarse soil or gravel. Loosely placed earth lining generally consists of a loose earth blanket of selected fine-grained soils dumped into the canal and spread over the bottom and sides to a thickness of about 12 inches. Bentonite soil mixtures usually consist of a sandy soil and bentonite mixed together and compacted. The thickness varies with local conditions. The bottom width to depth ratio and side slopes should be about the same as for unlined sections discussed in Paragraph 1.7.

LOCATION

.16 The location requirements for lined canals are about the same as for unlined canals discussed in Paragraph 1.8, but a lined canal may economically follow a more direct route.

LINED CANALS OR LATERALS--Continued

CURVATURE AND VELOCITY

.17 The allowable curvature for lined canals depends on the size and capacity, velocity, material used for lining, and the canal section. Hard-surface linings permit higher velocities than earth sections. Usually these velocities should be less than 8 feet per second to avoid the possibility of converting velocity head through a crack to pressure head under the lining and lifting the lining. A mathematical check using an "n" value 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 at sections where the bottom might be raised above theoretical grade due to construction tolerances.

Buried-membrane linings are usually covered with available material using thickness and canal section changes as required for stability. Velocities which are permissible in ordinary earth canals, where some erosion can be tolerated, may be too high for a buried-membrane lining where shallow scour may entirely remove the protective cover material from buried membrane. It must be realized that for a given velocity clear water may scour, while water carrying considerable sediment may build sandbars, with all other canal conditions being the same. From experience, it appears that the maximum velocity for buried-membrane-lined canals of a given size and shape is about two-thirds of that permissible for unlined earth canals in similar materials (see Paragraph 1.9). The permissible velocities in earth-lined canals vary with the type of lining and material, and usually range from 1 to 4 feet per second.

All influencing factors must be considered in determining the minimum radius of curvature. A suggested guide is that the minimum radius to canal centerline should be from three to seven times the water surface width if erodible linings are used. The smaller ratio is normally used for small canals while the larger ratio is needed for large canals. A concrete-lined canal should have a minimum radius of three times the water surface width.

FREEBOARD

.18 Freeboard for lined canals will depend upon a number of factors, such as the size of canal, velocity of water, curvature of alinement, storm water entering the canal, wind and wave action, and anticipated method of operation. The normal freeboard varies from 6 inches for small laterals to 2 feet or more for large canals. Figure 4 represents average Bureau practice as a guide for determining minimum freeboard and bank height for canals with hard-surface, buried-membrane, and earth linings.

The height of canal bank above the top of the lining usually varies from 6 inches for small laterals to over 2 feet for large canals. (See Figure 4.)

BANK TOP WIDTH AND BERM

.19 A 2- to 6-foot berm is normally provided at the top of hard-surface linings for the construction convenience of trimming and lining machinery. Backfill should be placed on this berm from the top of the lining and sloping upward to the earth bank, to prevent surface drainage from entering the subgrade behind the canal lining. The top width of canal banks and berms for lined canals should be about the same as for unlined canals discussed in Paragraph 1.11.

FLOW FORMULAS

.20 The Manning formula, which is generally used for open-channel flow, is presented and discussed in Paragraph 1.12.

Manning's Roughness Coefficient

A. Values of the Manning roughness coefficient "n" used in design for most lined canals are as follows:

LINED CANALS OR LATERALS--Continued

Manning's Roughness Coefficient (Cont'd.)

Portland cement concrete lining (r less than 4) (For canals with an r greater than 4, see B below.)	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 lining	1/0.015
Soil-cement	2/0.015 or 0.016

For buried membrane and compacted earth linings a roughness coefficient "n" of 0.025 is used for canals and laterals with capacities less than 100 cfs, and 0.020 or 0.0225, depending on the character of the materials, for larger canals. Recommended coefficients of roughness are given in the Bureau's Hydraulic and Excavation Tables.

For channels covered with coarse gravel or cobbles, the roughness coefficient "n" should not be less than that computed by the Strickler equation (see Paragraph 1.12).

Effects of Roughness and Hydraulic Radius

B. A roughness coefficient "n" of 0.014 provides a channel of adequate size for clean, straight concrete-lined canals with a hydraulic radius up to 4. When the hydraulic radius exceeds 4, Figure 13 should be used as a guide in choosing an "n" value. The curve on that figure indicates that a higher "n" value is required for the larger channels when the Colebrook-White equation is used for hydraulic computations and a constant equivalent sand grain surface roughness is assumed. As indicated on the figure, in arriving at points on the curve, the velocity as expressed by the Colebrook-White formula was equated to the velocity as expressed by the Manning formula, and the result solved for the increased coefficient of roughness "n."

The trend of increasing "n" is verified by the data from prototype canal capacity tests plotted on Figure 13. These capacity tests revealed that flow resistance in concrete-lined canals often varies seasonally because of aquatic growths on the lining surface. The most troublesome growth encountered in the western United States is filamentous green algae. Regular treatments with copper sulfate or aromatic solvents are effective in retarding, but not completely eliminating, this algae growth. If it is not feasible to chemically treat a canal to maintain the discharge capability, an increase in "n" should be considered in the original design to accommodate the increased flow resistance which may occur. The capacity tests indicated that "n" values increase seasonally as much as 30 percent in canals heavily infested with filamentous green algae.

Effect of Channel Sinuosity

C. The previously mentioned capacity tests disclosed that flow resistance in concrete-lined canals generally increases with channel sinuosity. They also revealed that canal structure piers located in the flow prism cause significant increases in water depth, especially in canals having very flat invert slopes (in the order of 0.00005). Design methods for accommodating excessive channel sinuosity and for computing pier losses are given in Technical Memorandum No. 661.3/

1/Assumed value based only on observation of section.
2/Soil cement may vary in roughness from as smooth as well-finished concrete to as rough as a gravel surface. The type of construction that is required must be considered.
3/"Analyses and Descriptions of Capacity Tests in Large Concrete-lined Canals," Technical Memorandum No. 661, Bureau of Reclamation, April 1964.

LINED CANALS OR LATERALS--Continued

HYDRAULIC BORE .21 Hydraulic bore, which is discussed briefly under Subparagraph 1.23A, may occur in lined canals. It may be caused by the shutdown of a pumping plant, rapid closure of a check or gate, or as a result of a sudden inflow causing a wave in the canal.

WINTER OPERATION .22 The primary difficulty in winter operation is the accumulation of frazil ice in the canal, especially at the inlets to structures such as siphons or penstocks.

Winter operation can be maintained by designing the system for one of two methods of operation:

A. To be operated at a capacity sufficient to prevent freezing. This presumes that the water temperature at the headworks is sufficiently above freezing to offset the heat loss in the canal.

B. If the first alternative is not feasible, the canal should be designed for operation under an ice cover. A cover forms readily at velocities less than 2.2 feet per second. Further, frazil ice rises to form surface ice at velocities less than 2 feet per second. Once an ice cover is formed, further heat loss is virtually eliminated. The design and operation may be greatly simplified if uniform flow is maintained during the winter months.

In either case, abrupt changes in grade or alinement should be avoided, as turbulence is essential to frazil ice production.

In the design of a power canal for winter operation, consideration should be given to the value of head that can be saved by using a velocity of 2 feet per second on a flat slope. The saving in head may offset the first cost of the larger canal section.

OTHER WATERWAYS

POWER CANALS .23 Power canals convey water from the sources of supply to the penstocks of powerplants. The primary difference between power and irrigation canals is the purpose to be accomplished. The value of power produced should be considered in determining the most economical canal section. Power canals will usually have more sudden changes in flow than irrigation canals. Wasteways are generally required just upstream from the powerplant, and the hydraulic bore must be computed in order to provide adequate freeboard on the canal.

Hydraulic Bore

A. A hydraulic bore is caused by a sudden change of discharge at any point in an open channel. It results in a moving wave going upstream or downstream in the channel. If the wave is caused by suddenly stopping the flow of water in a channel, as is the case in a powerplant shutdown or rapid closure of a check or gate, the hydraulic bore will travel upstream at a high velocity. The momentum, pressure, volume, and gravity effect must all be considered in computing the characteristics of the wave and its effect upon the channel. The downstream leveling of the water surface that occurs after the bore wave has passed must also be evaluated to determine the maximum rise in the water surface.

DRAINAGE SYSTEMS .24 The purpose of a drainage system is to remove excess water from the ground surface or subsoil. A drainage system may be required to prevent waterlogging of the land due to precipitation, irrigation waste, canal seepage, or high ground-water table. The possible need for a drainage system should

OTHER WATERWAYS--Continued

always be considered with the design of an irrigation system. Open or underground drains or a combination thereof may be used to effect an economical system to serve the needs of the area. The general design requirements for open drains are similar to those for irrigation channels. Open-joint tile pipe as well as closed-joint pipe may be used in underground drains. See Figure 14 for discharge curves for concrete or clay drain tile. DRAINAGE SYSTEMS (Cont'd.)

.25 Wasteway channels are sometimes required to dispose of excess water in canals. They are needed to dispose of operational waste or floodwater that has entered the canal, or to empty the canal. The general requirements for wasteway channels are similar to those for irrigation channels, depending on local conditions. Owing to infrequent use of wasteway channels at full capacity, the allowable velocity at full flow is usually greater than for an irrigation canal of similar capacity. WASTEWAY CHANNELS

TYPICAL UNLINED SECTION FOR CANALS AND LATERALS

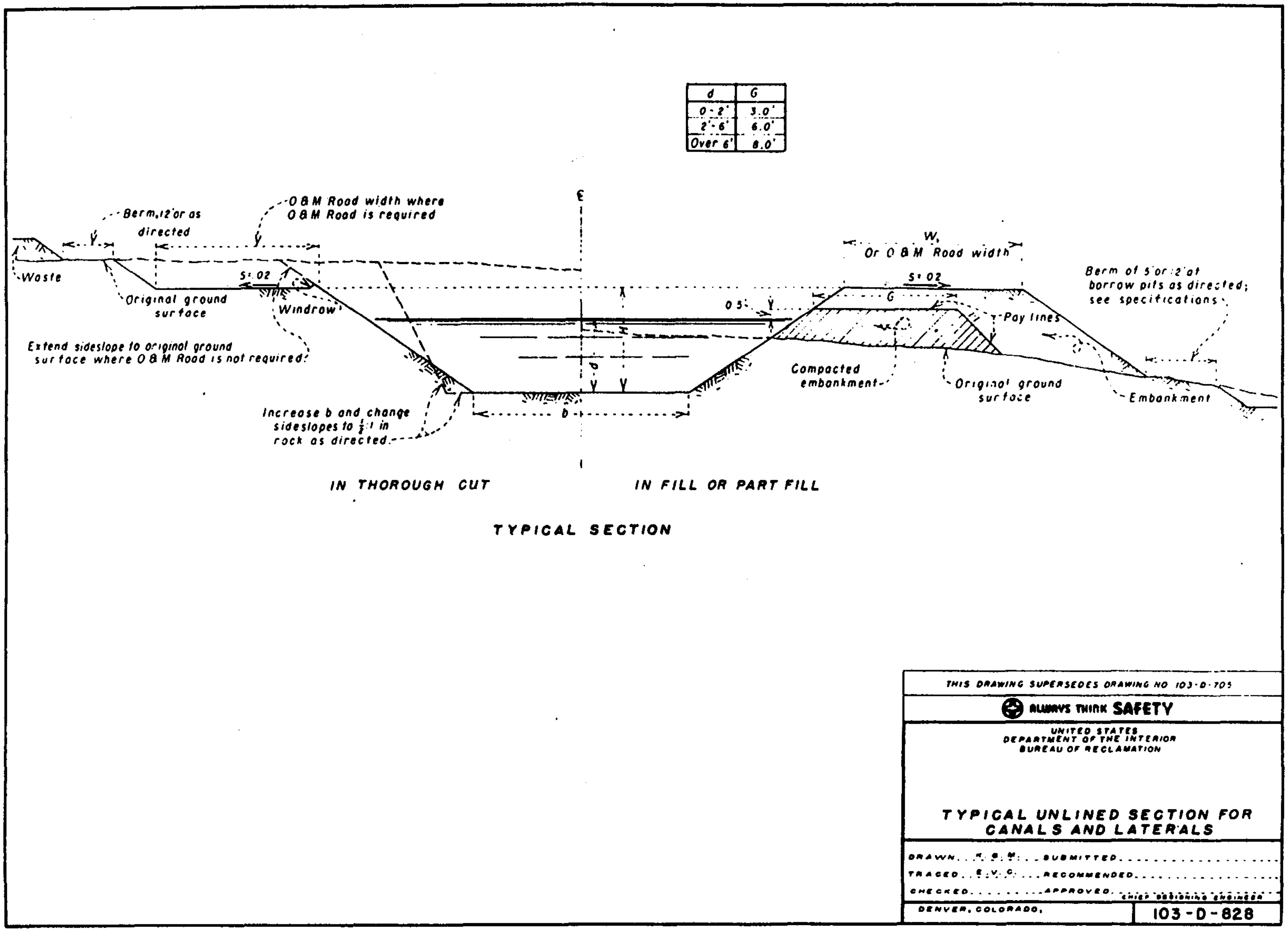
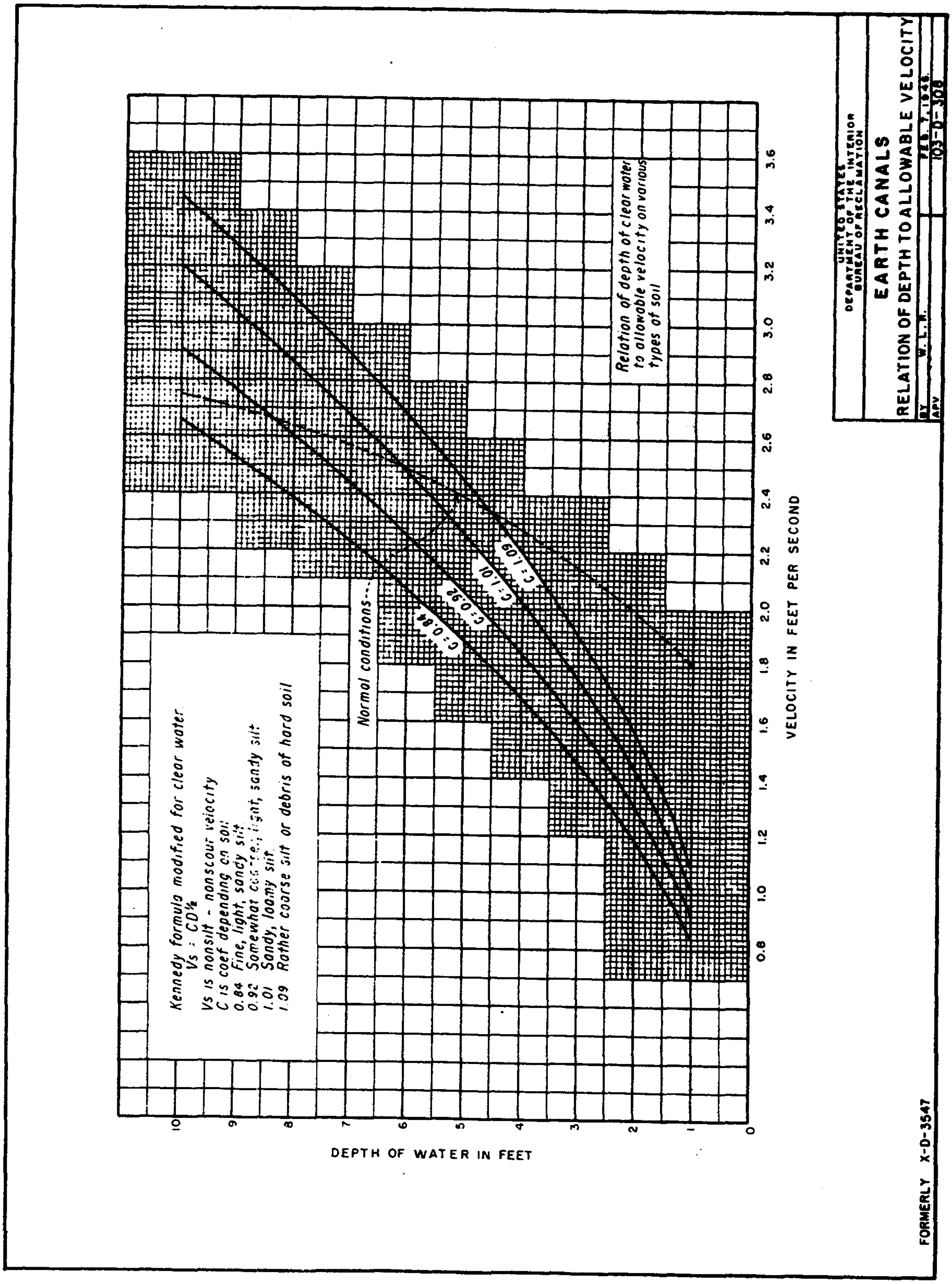


Fig. 2 Par. 1.7

EARTH CANALS--RELATION OF DEPTH TO ALLOWABLE VELOCITY



IRRIGATION CANAL EARTH SECTIONS--INSIDE SLOPES 1-1/2:1 and 2:1.

(An explanation of the tables shown as Figures 3A and 3B)

General

The sections shown in the tables represent canals and laterals constructed or proposed by the Bureau of Reclamation. Some departures may be required in order to meet local conditions. A study of the soils in which the proposed canal is to be excavated will assist in determination of whether, for stability, the wetted slopes of the section should be 1-1/2:1, 1-3/4:1 or flatter. These tables are for 1-1/2:1 and 2:1 inside slopes.

Capacity

The tables should be entered by finding a section giving the required capacity within the preferred lines. Straight-line interpolation of depth and velocity may be used to get capacities other than those appearing in the tables.

Velocity

A careful study should be made of the materials forming the canal section in order to determine the proper velocity for the required capacity. To meet specific conditions the velocities used for the sections shown in the tables may be increased for stable erosion-resistant soils by 10 percent. Velocities given in the table may be reduced by 20 percent if desired.

Freeboard

The freeboards given are for ordinary conditions. Smaller freeboards will generally not be used. For banks subject to wind or water erosion the freeboard may be increased.

Bank Width

Bank widths are based on Bureau practice. This width is given as a minimum and should be taken to the nearest even foot after any necessary interpolation. For small canals these widths will not provide sufficient space for operating roads. Where an operating road on the canal bank is required, the designs should provide for additional width.

Outside Bank Slopes

In this discussion slopes are considered as out and down, horizontal to vertical. In general, the outside bank slope will not be steeper than 1-1/2:1. Flatter slopes are often required due to type of soil and height of banks. On ground subject to sloughing, extra material from cuts should generally be used to provide flatter outside bank slopes rather than to increase the freeboard or bank width.

Fig. 3A Par. 1.7
Sheet 1 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 1-1/2:1)

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 1-1/2:1

(See explanation on preceding page)

d = water depth (ft) Q = discharge (cfs) V = assumed velocity (fps) W = bank width (ft) A = area (sq ft) Fb = freeboard (ft)

d	V	W		Bottom width (ft) 2	3	4	5	6	7	8	9	10	12	14	16	18
1.0	2.00	3.0	A	3.5	4.5	5.5	6.5	7.5	8.5	9.5						
			Q	7.0	9.0	11.0	13.0	15.0	17.0	19.0						
			Fb	1.0	1.0	1.1	1.2	1.2	1.3	1.3						
1.25	2.07	3.3	A	4.8	6.1	7.3	8.6	9.8	11.1	12.3	13.6					
			Q	9.9	12.6	15.1	17.8	20.3	23.0	25.5	28.2					
			Fb	1.1	1.2	1.3	1.3	1.3	1.4	1.4	1.4					
1.50	2.14	3.7	A	6.4	7.9	9.4	10.9	12.4	13.9	15.4	16.9	18.4				
			Q	13.7	16.9	20.1	23.3	26.5	29.7	33.0	36.2	39.4				
			Fb	1.2	1.3	1.3	1.4	1.4	1.4	1.5	1.5	1.5				
1.75	2.18	4.0	A	8.1	9.8	11.6	13.3	15.1	16.8	18.6	20.3	22.1	25.6			
			Q	17.7	21.4	25.3	29.0	32.9	36.6	40.5	44.3	48.2	55.8			
			Fb	1.3	1.3	1.4	1.4	1.5	1.5	1.5	1.6	1.6	1.7			
2.00	2.22	4.3	A		12.0	14.0	16.0	18.0	20.0	22.0	24.0	26.0	30.0	34.0		
			Q		26.6	31.1	35.5	40.0	44.4	48.8	53.3	57.7	66.6	75.5		
			Fb		1.4	1.5	1.5	1.5	1.6	1.6	1.7	1.7	1.7	1.8		
2.50	2.28	5.0	A		16.9	19.4	21.9	24.4	26.9	29.4	31.9	34.4	39.4	44.4	49.4	
			Q		38.5	44.2	49.9	55.6	61.3	67.0	72.7	78.4	89.8	101.2	112.6	
			Fb		1.5	1.6	1.6	1.7	1.7	1.7	1.8	1.8	1.8	1.9	1.9	
3.00	2.34	5.6	A				28.5	31.5	34.5	37.5	40.5	43.5	49.5	55.5	61.5	67.5
			Q				66.7	73.7	80.7	87.8	94.8	101.8	115.8	129.9	143.9	158.0
			Fb				1.7	1.8	1.8	1.8	1.9	1.9	1.9	2.0	2.0	2.0

<-----Preferred lines----->

d	V	W		Bottom width (ft) 7	8	9	10	12	14	16	18	20	22	24	26
3.50	2.38	6.3	A	42.9	46.4	49.9	53.4	60.4	67.4	74.4	81.4	88.4			
			Q	102	110	119	127	144	160	177	194	210			
			Fb	2.0	2.0	2.0	2.1	2.1	2.1	2.2	2.2	2.2			
4.00	2.42	6.9	A	52.0	56.0	60.0	64.0	72.0	80.0	88.0	96.0	104.0	112.0	120.0	128.0
			Q	126	136	145	155	174	194	213	232	252	271	290	310
			Fb	2.1	2.2	2.2	2.2	2.3	2.3	2.3	2.4	2.4	2.4	2.5	2.5
4.50	2.46	7.6	A		66.4	70.9	75.4	84.4	93.4	102.4	111.4	120.4	129.4	138.4	147.4
			Q		163	174	185	208	230	252	274	296	318	340	363
			Fb		2.2	2.3	2.3	2.3	2.4	2.4	2.4	2.5	2.5	2.5	2.6
5.00	2.50	8.2	A				87.5	97.5	107.5	117.5	127.5	137.5	147.5	157.5	167.5
			Q				219	244	269	294	319	344	369	394	419
			Fb				2.3	2.4	2.4	2.5	2.5	2.5	2.6	2.6	2.6
5.50	2.53	8.9	A				100.4	111.4	122.4	133.4	144.4	155.4	166.4	177.4	188.4
			Q				254	282	310	338	365	393	421	449	477
			Fb				2.5	2.6	2.6	2.6	2.7	2.7	2.7	2.8	2.8
6.00	2.56	9.6	A					126.0	138.0	150.0	162.0	174.0	186.0	198.0	210.0
			Q					323	353	384	415	445	476	507	538
			Fb					2.8	2.8	2.8	2.9	2.9	2.9	2.9	3.0
6.50	2.59	10.2	A					141.4	154.4	167.4	180.4	193.4	206.4	219.4	232.4
			Q					366	400	434	467	501	535	568	602
			Fb					2.8	2.9	2.9	3.0	3.0	3.1	3.1	3.1

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 1-1/2:1)

Fig. 3A Par. 1.7
Sheet 2 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 1-1/2:1--Continued

d	V	W		Bottom width (ft) 28	30	32	34	35	40	45	50
			A	177.5	187.5						
5.00	2.50	8.2	Q	444	469						
			Fb	2.7	2.7						
			A	199.4	210.4	221.4	232.4	237.9	265.4		
5.50	2.53	8.9	Q	504	532	560	588	602	671		
			Fb	2.8	2.9	2.9	2.9	2.9	3.0		
			A	222.0	234.0	246.0	258.0	264.0	294.0	324.0	
6.00	2.56	9.6	Q	568	599	630	660	676	753	829	
			Fb	3.0	3.0	3.1	3.1	3.1	3.2	3.2	
			A	245.4	258.4	271.4	284.4	290.9	323.4	355.9	388.4
6.50	2.59	10.2	Q	636	669	703	737	753	838	922	1006
			Fb	3.2	3.2	3.2	3.3	3.3	3.4	3.5	3.5

d	V	W		Bottom width (ft) 16	18	20	22	24	26	28	30	32	34	35
			A	185.5	199.5	213.5	227.5	241.5	255.5	269.5	283.5	297.5	311.5	318.5
7.00	2.62	10.9	Q	486	523	559	596	633	669	706	743	779	816	834
			Fb	3.0	3.1	3.1	3.1	3.2	3.2	3.3	3.3	3.3	3.4	3.4
			A	204.4	219.4	234.4	249.4	264.4	279.4	294.4	309.4	324.4	339.4	346.9
7.50	2.65	11.5	Q	542	581	621	661	701	740	780	820	860	899	919
			Fb	3.1	3.1	3.2	3.2	3.3	3.3	3.3	3.4	3.4	3.5	3.5
			A	224.0	240.0	256.0	272.0	288.0	304.0	320.0	336.0	352.0	368.0	376.0
8.00	2.68	12.2	Q	600	643	686	729	772	815	858	900	943	986	1008
			Fb	3.2	3.2	3.3	3.3	3.3	3.4	3.4	3.5	3.5	3.5	3.6
			A	244.4	261.4	278.4	295.4	312.4	329.4	346.4	363.4	380.4	397.4	405.9
8.50	2.71	12.8	Q	662	708	754	801	847	893	939	985	1031	1077	1100
			Fb	3.2	3.3	3.3	3.4	3.4	3.5	3.5	3.5	3.6	3.6	3.7
			A				319.5	337.5	355.5	373.5	391.5	409.5	427.5	436.5
9.00	2.75	13.5	Q				879	928	978	1027	1077	1126	1176	1200
			Fb				3.4	3.5	3.5	3.6	3.6	3.6	3.7	3.8
			A					363.4	382.4	401.4	420.4	439.4	458.4	467.9
9.50	2.78	14.2	Q					1010	1063	1116	1169	1222	1274	1301
			Fb					3.5	3.6	3.6	3.6	3.6	3.7	3.8
			A						410.0	430.0	450.0	470.0	490.0	500.0
10.00	2.81	14.8	Q						1152	1208	1265	1321	1377	1405
			Fb						3.6	3.6	3.6	3.7	3.7	3.8
			A							459.4	480.4	501.4	522.4	532.9
10.50	2.84	15.5	Q							1305	1364	1424	1484	1513
			Fb							3.7	3.7	3.7	3.8	3.8
			A								511.5	533.5	555.5	566.5
11.00	2.87	16.1	Q								1468	1531	1594	1626
			Fb								3.7	3.8	3.9	3.9

Fig. 3A Par. 1.7
Sheet 3 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 1-1/2:1)

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 1-1/2:1--Continued

				Bottom width (ft)								
d	V	W		40	45	50	55	60	65	70	75	80
			A	353.5	388.5	423.5						
7.00	2.62	10.9	Q	926	1018	1110						
			Fb	3.5	3.6	3.6						
			A	384.4	421.9	459.4	496.9					
7.50	2.65	11.5	Q	1019	1118	1217	1317					
			Fb	3.6	3.7	3.7	3.8					
			A	416.0	456.0	496.0	536.0	576.0				
8.00	2.68	12.2	Q	1115	1222	1329	1436	1544				
			Fb	3.7	3.8	3.9	3.9	4.0				
			A	448.4	490.9	533.4	575.9	618.4	660.9			
8.50	2.71	12.8	Q	1215	1330	1446	1561	1676	1791			
			Fb	3.8	3.9	4.0	4.0	4.1	4.2			
			A	481.5	526.5	571.5	616.5	661.5	706.5			
9.00	2.75	13.5	Q	1324	1448	1572	1695	1819	1943			
			Fb	3.9	4.0	4.1	4.1	4.2	4.3			
			A	515.4	562.9	610.4	657.9	705.4	752.9	800.4		
9.50	2.78	14.2	Q	1433	1565	1697	1829	1961	2093	2225		
			Fb	3.9	4.0	4.1	4.2	4.3	4.4	4.5		
			A	550.0	600.0	650.0	700.0	750.0	800.0	850.0		
10.00	2.81	14.8	Q	1546	1686	1827	1968	2108	2248	2389		
			Fb	3.9	4.0	4.1	4.2	4.3	4.4	4.5		
			A	585.4	637.9	690.4	742.9	795.4	847.9	900.4	952.9	
10.50	2.84	15.5	Q	1663	1812	1961	2110	2259	2408	2557	2706	
			Fb	4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.6	
			A	621.5	676.5	731.5	786.5	841.5	896.5	951.5	1006.5	1061.5
11.00	2.87	16.1	Q	1784	1942	2099	2257	2415	2573	2731	2889	3047
			Fb	4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.6	4.7

				Bottom width (ft)									
d	V	W		30	35	40	45	50	55	60	65	70	75
			A	543.4	600.9	658.4	715.9	773.4	830.9	888.4	945.9	1003.4	1060.9
11.50	2.89	16.8	Q	1570	1737	1903	2069	2235	2401	2567	2734	2900	3066
			Fb	3.8	3.9	4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.6
			A	576.0	636.0	696.0	756.0	816.0	876.0	936.0	996.0	1056.0	1116.0
12.00	2.91	17.4	Q	1676	1851	2025	2200	2375	2549	2724	2898	3073	3248
			Fb	3.8	3.9	4.0	4.1	4.2	4.3	4.4	4.5	4.6	4.7
			A		708.5	773.5	838.5	903.5	968.5	1033.5	1098.5	1163.5	1228.5
13.00	2.95	18.[illegible]	Q		2090	2282	2474	2665	2857	3049	3241	3432	3624
			Fb		4.0	4.1	4.2	4.3	4.4	4.5	4.5	4.6	4.7
			A			854.0	924.0	994.0	1064.0	1134.0	1204.0	1274.0	1344.0
14.00	2.98	19.8	Q			2545	2754	2962	3171	3379	3588	4005	4214
			Fb			4.1	4.2	4.3	4.4	4.5	4.6	4.6	4.7
			A				1012.5	1087.5	1162.5	1237.5	1312.5	1387.5	1462.5
15.00	3.00	21.2	Q				3038	3263	3488	3713	3938	4163	4388
			Fb				4.3	4.4	4.5	4.6	4.7	4.7	4.8
			A				1104.0	1184.0	1264.0	1344.0	1424.0	1504.0	1584.0
16.00	3.00	22.5	Q				3312	3552	3792	4032	4272	4512	4752
			Fb				4.4	4.5	4.6	4.7	4.8	4.9	4.9
			A					1283.5	1368.5	1453.5	1538.5	1623.5	1708.5
17.00	3.00	24.0	Q					3850	4106	4360	4616	4870	5126
			Fb					4.7	4.8	4.9	5.0	5.1	5.2
			A					1386.0	1476.0	1566.0	1656.0	1746.0	1836.0
18.00	3.00	24.0	Q					4158	4428	4698	4968	5238	5508
			Fb					4.9	5.0	5.1	5.2	5.3	5.3
			A						1586.5	1681.5	1776.5	1871.5	1966.5
19.00	3.00	24.0	Q						4760	5044	5330	5614	5900
			Fb						5.2	5.3	5.4	5.5	5.5

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 1-1/2:1)

Fig. 3A Par. 1.7
Sheet 4 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 1-1/2:1--Continued

d	V	W		Bottom width (ft) 80	85	90	95	100	110	120	130	140	150
11.50	2.89	16.8	A	1118.4	1175.9								
			Q	3232	3398								
			Fb	4.7	4.7								
12.00	2.91	17.4	A	1176.0	1236.0	1296.0							
			Q	3422	3597	3771							
			Fb	4.7	4.8	4.8							
13.00	2.95	18.6	A	1293.5	1358.5	1423.5	1488.5						
			Q	3816	4008	4199	4391						
			Fb	4.8	4.8	4.9	4.9						
14.00	2.98	19.8	A	1414.0	1484.0	1554.0	1624.0	1694.0	1834.0				
			Q	4214	4422	4631	4840	5048	5465				
			Fb	4.9	4.9	5.0	5.0	5.1	5.1				
15.00	3.00	21.2	A	1537.5	1612.5	1687.5	1762.5	1837.5	1987.5	2137.5			
			Q	4613	4838	5063	5288	5513	5962	6413			
			Fb	4.9	5.0	5.0	5.1	5.1	5.2	5.2			
16.00	3.00	22.5	A	1664.0	1744.0	1824.0	1904.0	1984.0	2144.0	2304.0			
			Q	4992	5232	5472	5712	5952	6432	6912			
			Fb	5.0	5.1	5.1	5.2	5.2	5.3	5.3			
17.00	3.00	24.0	A	1793.5	1878.5	1963.5	2048.5	2133.5	2303.5	2473.5	2643.5		
			Q	5380	5636	5890	6146	6400	6910	7420	7930		
			Fb	5.2	5.3	5.3	5.4	5.4	5.5	5.6	5.7		
18.00	3.00	24.0	A	1926.0	2016.0	2106.0	2196.0	2286.0	2466.0	2646.0	2826.0	3006.0	
			Q	5778	6048	6318	6588	6858	7398	7938	8478	9018	
			Fb	5.4	5.4	5.5	5.6	5.6	5.7	5.8	5.8	5.9	
19.00	3.00	24.0	A	2061.5	2156.5	2251.5	2346.5	2441.5	2631.5	2821.5	3011.5	3201.5	3391.5
			Q	6184	6470	6755	7040	7324	7894	8464	9034	9604	10174
			Fb	5.6	5.6	5.7	5.8	5.8	5.9	5.9	6.0	6.0	6.0

d	V	W		Bottom width (ft) 70	75	80	85	90	95	100	110	120	130
20.0	3.00	24.0	A	2000.0	2100.0	2200.0	2300.0	2400.0	2500.0	2600.0	2800.0	3000.0	3200.0
			Q	6000	6300	6600	6900	7200	7500	7800	8400	9000	9600
			Fb	5.5	5.6	5.7	5.7	5.8	5.8	5.9	6.0	6.0	6.0
21.0	3.00	24.0	A	2131.5	2236.5	2341.5	2446.5	2551.5	2656.5	2761.5	2971.5	3181.5	3391.5
			Q	6394	6710	7024	7340	7654	7970	8285	8914	9544	10174
			Fb	5.5	5.6	5.7	5.7	5.8	5.8	5.9	6.0	6.0	6.0
22.0	3.00	24.0	A		2376.0	2486.0	2596.0	2706.0	2816.0	2926.0	3146.0	3366.0	3586.0
			Q		7128	7458	7788	8118	8448	8778	9438	10098	10758
			Fb		5.7	5.7	5.8	5.8	5.9	5.9	6.0	6.0	6.0
23.0	3.00	24.0	A			2633.5	2748.5	2863.5	2978.5	3093.5	3323.5	3553.5	3783.5
			Q			7900	8245	8590	8935	9280	9970	10660	11350
			Fb			5.8	5.8	5.9	5.9	5.9	6.0	6.0	6.0
24.0	3.00	24.0	A				2904.0	3024.0	3144.0	3264.0	3504.0	3744.0	3984.0
			Q				8712	9072	9432	9792	10512	11232	11952
			Fb				5.8	5.9	5.9	6.0	6.0	6.0	6.0
25.0	3.00	24.0	A					3187.5	3312.5	3437.5	3687.5	3937.5	4187.5
			Q					9563	9938	10313	11063	11813	12563
			Fb					5.9	6.0	6.0	6.0	6.0	6.0
26.0	3.00	24.0	A						3484.0	3614.0	3874.0	4134.0	4394.0
			Q						10452	10842	11622	12402	13182
			Fb						6.0	6.0	6.0	6.0	6.0
28.0	3.00	24.0	A							3976.0	4256.0	4536.0	4816.0
			Q							11928	12768	13608	14448
			Fb							6.0	6.0	6.0	6.0
30.0	3.00	24.0	A								4650.0	4950.0	5250.0
			Q								13950	14[illegible]50	15750
			Fb								3.0	6.0	6.0

Fig. 3A Par. 1.7
Sheet 5 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 1-1/2:1)

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 1-1/2:1--Continued

				Bottom width (ft)									
d	V	W		140	150	160	170	180	190	200	220	240	260
			A	3400.0	3600.0	3800.0							
20.0	3.00	24.0	Q	10200	10800	11400							
			Fb	6.0	6.0	6.0							
			A	3601.5	3811.5	4021.5	4231.5	4441.5					
21.0	3.00	24.0	Q	10804	11434	12064	12694	13324					
			Fb	6.0	6.0	6.0	6.0	6.0					
			A	3806.0	4026.0	4246.0	4466.0	4686.0	4906.0				
22.0	3.00	24.0	Q	11418	12078	12738	13398	14058	14718				
			Fb	6.0	6.0	6.0	6.0	6.0	6.0				
			A	4013.5	4243.5	4473.5	4703.5	4933.5	5163.5	5393.5			
23.0	3.00	24.0	Q	12040	12730	13420	14110	14800	15490	16180			
			Fb	6.0	6.0	6.0	6.0	6.0	6.0	6.0			
			A	4224.0	4464.0	4704.0	4944.0	5184.0	5424.0	5664.0			
24.0	3.00	24.0	Q	12672	13392	14112	14832	15552	16272	16992			
			Fb	6.0	6.0	6.0	6.0	6.0	6.0	6.0			
			A	4437.5	4687.5	4937.5	5187.5	5437.5	5687.5	5937.5	6437.5	6937.5	
25.0	3.00	24.0	Q	13313	14063	14813	15563	16313	17063	17813	19313	20813	
			Fb	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	
			A	4654.0	4914.0	5174.0	5434.0	5694.0	5954.0	6214.0	6734.0	7254.0	7774.0
26.0	3.00	24.0	Q	13962	14742	15522	16302	17082	17862	18642	20202	21762	23322
			Fb	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
			A	5096.0	5376.0	5656.0	5936.0	6216.0	6496.0	6776.0	7336.0	7896.0	8456.0
28.0	3.00	24.0	Q	15288	16128	16968	17808	18648	19488	20328	22008	23688	25368
			Fb	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0
			A	5550.0	5850.0	6150.0	6450.0	6750.0	7050.0	7350.0	7950.0	8550.0	9150.0
30.0	3.00	24.0	Q	16650	17550	18450	19350	20250	21150	22050	23850	25650	27450
			Fb	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0	6.0

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 2:1)

Fig. 3B Par. 1.7
Sheet 1 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 2:1

(See explanation on page preceding Figure 3A)

d = water depth (ft) V = assumed velocity (fps) W = bank width (ft) A = area (sq ft)
Q = discharge (cfs) Fb = freeboard (ft)

d	V	W		Bottom width (ft)												
				2	3	4	5	6	7	8	9	10	12	14	16	18
1.00	2.00	3.0	A Q Fb	4.0 8.0 1.0	5.0 10.0 1.0	6.0 12.0 1.1	7.0 14.0 1.1	8.0 16.0 1.2	9.0 18.0 1.3							
1.25	2.07	3.3	A Q Fb	5.6 11.6 1.1	6.9 14.3 1.2	8.1 16.8 1.3	9.4 19.4 1.3	10.6 22.0 1.3	11.9 24.6 1.4	13.1 27.2 1.4						
1.50	2.14	3.7	A Q Fb	7.5 16.0 1.2	9.0 19.3 1.3	10.5 22.5 1.3	12.0 25.7 1.4	13.5 28.9 1.4	15.0 32.1 1.4	16.5 35.3 1.5	18.0 38.5 1.5					
1.75	2.18	4.0	A Q Fb	9.6 20.9 1.3	11.4 24.8 1.3	13.1 28.6 1.4	14.9 32.4 1.4	16.6 36.2 1.5	18.4 40.1 1.5	20.1 43.8 1.5	21.9 47.7 1.6	23.6 51.5 1.6				
2.0	2.22	4.3	A Q Fb	12.0 26.6 1.4	14.0 31.1 1.5	16.0 35.5 1.5	18.0 40.0 1.5	20.0 44.4 1.6	22.0 48.8 1.6	24.0 53.3 1.7	26.0 57.7 1.7	28.0 62.2 1.7	32.0 71.0 1.8			
2.5	2.28	5.0	A Q Fb		20.0 45.6 1.6	22.5 51.4 1.7	25.0 57.0 1.7	27.5 62.7 1.8	30.0 68.4 1.8	32.5 74.1 1.8	35.0 79.8 1.9	37.5 85.5 1.9	42.5 96.9 1.9	47.5 108. 2.0		
3.0	2.34	5.6	A Q Fb			30.0 70.2 1.8	33.0 77.2 1.8	36.0 84.2 1.9	39.0 91.3 1.9	42.0 98.3 2.0	45.0 105. 2.0	48.0 112. 2.1	54.0 126. 2.1	60.0 140. 2.2	66.0 154. 2.2	72. 168. 2.2

d	V	W		Bottom width (ft)												
				6	7	8	9	10	12	14	16	18	20	22	24	26
3.50	2.38	6.3	A Q Fb	45.5 108. 2.0	49.0 117. 2.1	52.50 125. 2.1	56.0 133. 2.1	59.5 142. 2.2	66.5 158. 2.2	73.5 175. 2.2	80.5 191. 2.3	87.5 208. 2.3	94.5 225. 2.5			
4.00	2.42	6.9	A Q Fb		60.0 145. 2.1	64.0 155. 2.2	68.0 165. 2.2	72.0 174. 2.2	80.0 194. 2.3	88.0 213. 2.3	96.0 232. 2.4	104.0 252. 2.4	112. 271. 2.4	120. 290. 2.5	128. 310. 2.5	
4.50	2.46	7.6	A Q Fb			76.5 188. 2.3	81.0 199. 2.3	85.5 210. 2.4	94.5 232. 2.4	103.5 255. 2.4	112.5 277. 2.5	121.5 299. 2.5	130.5 321. 2.6	139.5 343. 2.6	148.5 365. 2.6	157.5 387. 2.7
5.00	2.50	8.2	A Q Fb				95.0 237. 2.4	100. 250. 2.5	110. 275. 2.5	120. 300. 2.5	130. 325. 2.6	140. 350. 2.6	150. 375. 2.7	160. 400. 2.7	170. 425. 2.7	180. 450. 2.8
5.50	2.53	8.9	A Q Fb					115.5 292. 2.5	126.5 320. 2.6	137.5 348. 2.6	148.5 376. 2.7	159.5 403. 2.7	170.5 431. 2.8	181.5 459. 2.8	192.5 487. 2.9	203.5 515. 2.9
6.00	2.56	9.6	A Q Fb						144. 368. 2.8	156. 399. 2.8	168.0 430. 2.8	180.0 461. 2.9	192. 491. 2.9	204. 522. 3.0	216. 552. 3.1	228. 584. 3.1
6.50	2.59	10.2	A Q Fb								188.5 488. 2.9	201.5 522. 3.0	214.5 556. 3.0	227.5 589. 3.1	240.5 623. 3.1	253.5 657. 3.2

Fig. 3B Par. 1.7
Sheet 2 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 2:1)

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 2:1--Continued

				Bottom width (ft)									
d	V	W		28	30	35	40	45	50				
5.0	2.50	8.2	A Q Fb	190. 492. 2.8									
5.5	2.53	8.9	A Q Fb	218.0 551. 3.0	229.2 580. 3.0								
6.0	2.56	9.6	A Q Fb	240. 614. 3.1	252. 645. 3.2	282. 722. 3.3	312. 799. 3.4						
6.5	2.59	10.2	A Q Fb	266.5 690. 3.3	279.5 724. 3.3	312.0 808. 3.4	344.5 892. 3.5	377.0 976. 3.6					

				Bottom width (ft)										
d	V	W		16	18	20	22	24	26	28	30	32	34	35
7.0	2.62	10.9	A Q Fb	210 550 3.0	224 587 3.1	238 623 3.1	252 660 3.2	266 697 3.3	280 734 3.3	294 770 3.4	308 807 3.4	322 844 3.4	336 880 3.4	343 899 3.5
7.5	2.65	11.5	A Q Fb	232.5 616 3.1	247.5 656 3.2	262.5 696 3.3	277.5 735 3.3	292.5 775 3.4	307.5 815 3.4	322.5 855 3.5	337.5 894 3.5	352.5 935 3.5	367.5 974 3.5	375 994 3.6
8.0	2.68	12.2	A Q Fb		272.0 729 3.4	288 772 3.4	304 815 3.4	320 858 3.5	336 900 3.5	352 943 3.5	368 986 3.6	384 1029 3.6	400 1072 3.6	408 1093 3.6
8.5	2.71	12.8	A Q Fb			314.5 852 3.4	331.5 898 3.5	348.5 944 3.5	365.5 990 3.6	382.5 1037 3.6	399.5 1083 3.6	416.5 1128 3.6	433.5 1174 3.7	442.0 1198 3.7
9.0	2.75	13.5	A Q Fb					378 1039 3.6	396 1089 3.7	414 1138 3.7	432 1188 3.7	450 1238 3.8	468 1286 3.8	477 1312 3.8
9.5	2.78	14.2	A Q Fb						427.5 1188 3.7	446.5 1241 3.7	465.5 1294 3.7	484.5 1346 3.8	503.5 1400 3.8	513 1426 3.8
10.0	2.81	14.8	A Q Fb							480 1349 3.8	500 1405 3.8	520 1460 3.8	540 1517 3.9	550 1545 3.9
10.5	2.84	15.5	A Q Fb								535.5 1520 3.9	556.5 1580 3.9	577.5 1640 4.0	588 1670 4.0
11.0	2.87	16.1	A Q Fb								572 1642 4.0	594 1705 4.0	616 1768 4.1	627 1800 4.1

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 2:1)

Fig. 3B Par. 1.7
Sheet 3 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 2:1--Continued

Side slopes 2:1				Bottom width (ft)							
d	V	W		40	45	50	55	60	65	70	75
7.0	2.62	10.9	A Q Fb	378 990 3.6	413 1082 3.6						
7.5	2.65	11.5	A Q Fb	412.5 1093 3.6	450 1192 3.7	487.5 1292 3.8					
8.0	2.68	12.2	A Q Fb	448 1200 3.7	488 1308 3.8	528 1415 3.9	568 1522 3.9				
8.5	2.71	12.8	A Q Fb	484.5 1313 3.8	527 1428 3.9	569.5 1543 4.0	612 1658 4.1				
9.0	2.75	13.5	A Q Fb	522 1435 3.9	567 1559 4.0	612 1683 4.1	657 1807 4.1	702 1930 4.2			
9.5	2.78	14.2	A Q Fb	560.5 1558 4.0	608 1690 4.0	655.5 1822 4.1	703 1954 4.2	750.5 2086 4.3	798.0 2218 4.4		
10.0	2.81	14.8	A Q Fb	600 1686 4.0	650 1826 4.1	700 1967 4.1	750 2108 4.2	800 2248 4.3	850 2388 4.4	900 2529 4.5	
10.5	2.84	15.5	A Q Fb	640.5 1819 4.1	693 1968 4.1	745.5 2117 4.2	798 2266 4.3	850.5 2415 4.4	903 2564 4.5	955.5 2714 4.6	1008 2863 4.6
11.0	2.87	16.1	A Q Fb	682 1957 4.1	737 2115 4.2	792 2273 4.2	847 2430 4.3	902 2588 4.4	957 2746 4.5	1012 2904 4.6	1067 3062 4.6

Side slopes 2:1				Bottom width (ft)									
d	V	W		30	35	40	45	50	55	60	65	70	75
11.5	2.89	16.8	A Q Fb	609.5 1761 4.0	667.0 1927 4.1	724.5 2094 4.2	782.0 2260 4.2	839.5 2426 4.3	897.0 2592 4.4	954.5 2758 4.4	1012.0 2925 4.5	1069.5 3090 4.6	1127.0 3257 4.6
12.0	2.91	17.4	A Q Fb		708 2060 4.1	768 2234 4.2	828 2410 4.3	888 2584 4.3	948 2758 4.4	1008 2933 4.5	1068 3108 4.5	1128 3282 4.6	1188 3457 4.7
13.0	2.95	18.6	A Q Fb			858 2531 4.3	923 2722 4.4	988 2914 4.5	1053 3106 4.5	1118 3298 4.6	1183 3490 4.6	1248 3682 4.7	1313 3873 4.8
14.0	2.98	19.8	A Q Fb			952 2837 4.4	1022 3046 4.5	1092 3254 4.6	1162 3462 4.6	1232 3671 4.7	1302 3880 4.8	1372 4088 4.8	1442 4297 4.9
15.0	3.0	21.2	A Q Fb				1125 3375 4.6	1200 3600 4.7	1275 3825 4.8	1350 4050 4.8	1425 4275 4.9	1500 4500 5.0	1575 4725 5.0
16.0	3.0	22.5	A Q Fb					1312 3936 4.8	1392 4176 4.9	1472 4416 5.0	1552 4656 5.0	1632 4896 5.1	1712 5136 5.2
17.0	3.0	24.0	A Q Fb						1513 4539 5.0	1598 4794 5.1	1683 5049 5.1	1768 5304 5.2	1853 5559 5.3
18.0	3.0	24.0	A Q Fb							1728 5184 5.2	1818 5454 5.3	1908 5724 5.3	1998 5994 5.4
19.0	3.0	24.0	A Q Fb								1957 5871 5.4	2052 6156 5.5	2147 6441 5.6

Fig. 3B Par. 1.7
Sheet 4 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 2:1)

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 2:1--Continued

Side slopes 2:1				Bottom width (ft)									
d	V	W		80	85	90	95	100	110	120	130	140	150
11.5	2.89	16.8	A Q Fb	1184.5 3423 4.7									
12.0	2.91	17.4	A Q Fb	1248.0 3632 4.7	1308.0 3806 4.8	1368 3981 4.8							
13.0	2.95	18.6	A Q Fb	1378 4065 4.8	1443 4256 4.9	1508 4449 4.9	1573 4640 5.0						
14.0	2.98	19.8	A Q Fb	1512 4506 5.0	1582 4714 5.0	1652 4923 5.1	1722 5131 5.2	1792 5340 5.2	1932 5760 5.3				
15.0	3.0	21.2	A Q Fb	1650 4950 5.1	1725 5175 5.2	1800 5400 5.2	1875 5625 5.3	1950 5850 5.4	2100 6300 5.5				
16.0	3.0	22.5	A Q Fb	1792 5376 5.2	1872 5616 5.3	1952 5856 5.4	2032 6096 5.4	2112 6336 5.5	2272 6816 5.6	2432 7296 5.6			
17.0	3.0	24.0	A Q Fb	1938 5814 5.4	2023 6069 5.4	2108 6324 5.5	2193 6579 5.6	2278 6834 5.7	2448 7344 5.8	2618 7854 5.8	2788 8364 5.9		
18.0	3.0	24.0	A Q Fb	2088 6264 5.5	2178 6534 5.6	2268 6804 5.6	2358 7074 5.7	2448 7344 5.8	2628 7884 5.9	2808 8424 5.9	2988 8964 6.0	3168 9504 6.0	
19.0	3.0	24.0	A Q Fb	2242 6726 5.6	2337 7011 5.7	2432 7296 5.8	2527 7581 5.8	2622 7866 5.9	2812 8436 5.9	3002 9006 6.0	3192 9576 6.0	3382 10146 6.0	3572 10716 6.0

Side slopes 2:1				Bottom width (ft)									
d	V	W		70	75	80	85	90	95	100	110	120	130
20	3.0	24.0	A Q Fb	2200 6600 5.6	2300 6900 5.7	2400 7200 5.7	2500 7500 5.8	2600 7800 5.8	2700 8100 5.9	2800 8400 5.9	3000 9000 6.0	3200 9600 6.0	3400 10200 6.0
21	3.0	24.0	A Q Fb	2352 7056 5.7	2457 7371 5.8	2562 7686 5.8	2667 8001 5.9	2772 8316 5.9	2877 8631 5.9	2982 8946 6.0	3192 9576 6.0	3402 10210 6.0	3612 10840 6.0
22	3.0	24.0	A Q Fb				2838 8514 5.9	2948 8844 5.9	3058 9174 6.0	3168 9504 6.0	3388 10160 6.0	3608 10820 6.0	3828 11480 6.0
23	3.0	24.0	A Q Fb					3128 9384 6.0	3243 9729 6.0	3358 10070 6.0	3588 10760 6.0	3818 11450 6.0	4048 12140 6.0
24	3.0	24.0	A Q Fb						3432 10300 6.0	3552 10660 6.0	3792 11380 6.0	4032 12100 6.0	4272 12820 6.0
25	3.0	24.0	A Q Fb							3750 11250 6.0	4000 12000 6.0	4250 12750 6.0	4500 13500 6.0
26	3.0	24.0	A Q Fb								4212 12640 6.0	4472 13420 6.0	4732 14200 6.0
28	3.0	24.0	A Q Fb								4648 13940 6.0	4928 14780 6.0	5208 15620 6.0
30	3.0	24.0	A Q Fb										5700 17100 6.0

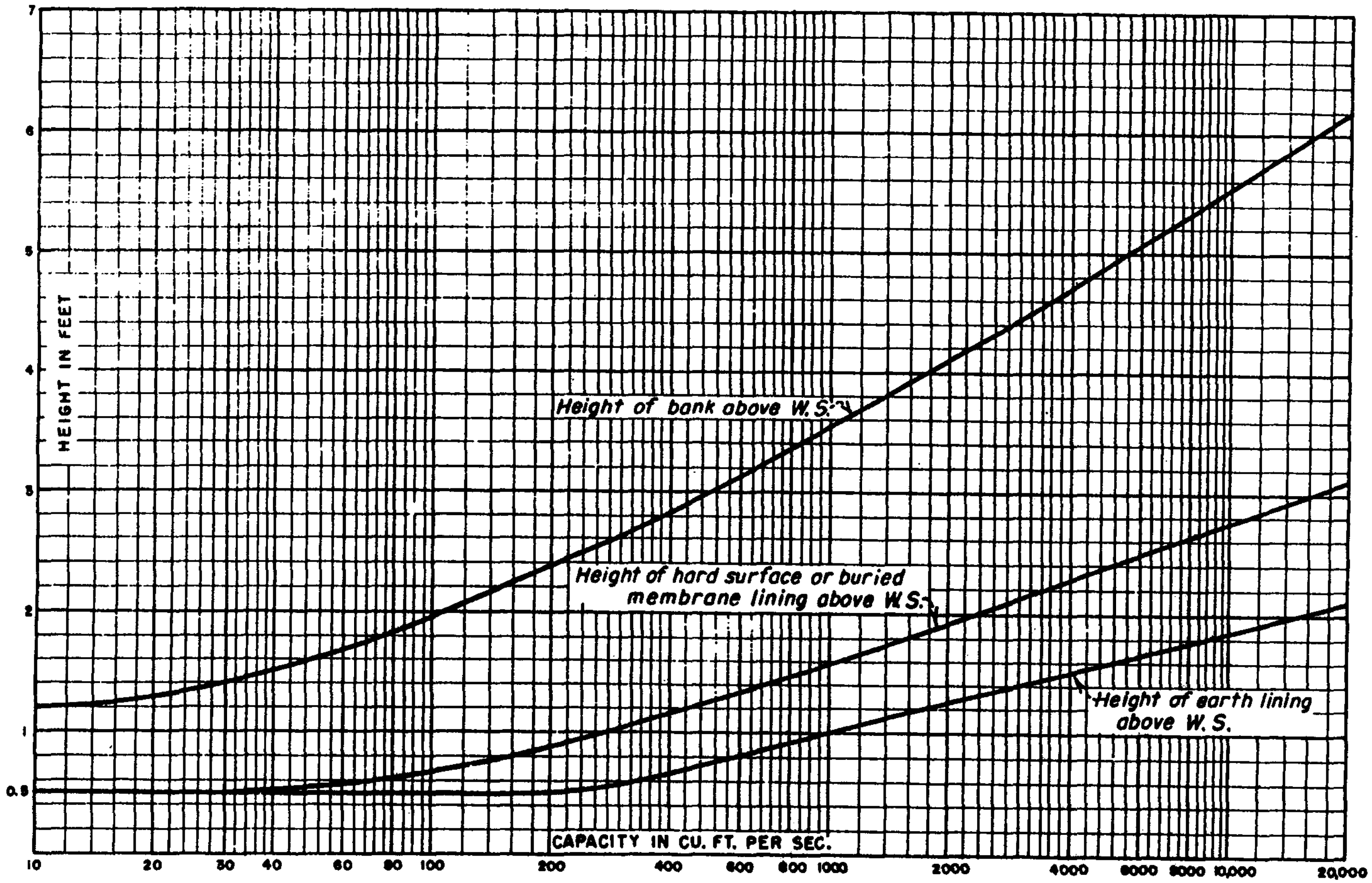
TYPICAL IRRIGATION CANAL EARTH SECTIONS
(Inside slopes of 2:1)

Fig. 3B Par. 1.7
Sheet 5 of 5

TYPICAL IRRIGATION CANAL EARTH SECTIONS SHOWING MINIMUM FREEBOARDS AND BANK WIDTHS--INSIDE SLOPES 2:1--Continued

Side slopes 2:1					Bottom width (ft)								
d	V	W		140	150	160	170	180	190	200	220	240	260
20.0	3.0	24.0	A Q Fb	3600 10800 6.0	3800 11400 6.0	4000 12000 6.0							
21.0	3.0	24.0	A Q Fb	3822 11470 6.0	4032 12100 6.0	4242 12730 6.0	4452 13360 6.0						
22.0	3.0	24.0	A Q Fb	4048 12140 6.0	4268 12800 6.0	4488 13460 6.0	4708 14120 6.0	4928 14780 6.0					
23.0	3.0	24.0	A Q Fb	4278 12830 6.0	4508 13520 6.0	4738 14210 6.0	4968 14900 6.0	5198 15590 6.0	5428 16280 6.0				
24.0	3.0	24.0	A Q Fb	4512 13540 6.0	4752 14260 6.0	4992 14980 6.0	5232 15700 6.0	5472 16420 6.0	5712 17140 6.0	5952 17860 6.0			
25.0	3.0	24.0	A Q Fb	4750 14250 6.0	5000 15000 6.0	5250 15750 6.0	5500 16500 6.0	5750 17250 6.0	6000 18000 6.0	6250 18750 6.0	6750 20250 6.0		
26.0	3.0	24.0	A Q Fb	4992 14980 6.0	5252 15760 6.0	5512 16540 6.0	5772 17320 6.0	6032 18100 6.0	6292 18880 6.0	6552 19660 6.0	7072 21220 6.0	7592 22780 6.0	
28.0	3.0	24.0	A Q Fb	5488 16460 6.0	5768 17300 6.0	6048 18140 6.0	6328 18980 6.0	6608 19820 6.0	6888 20660 6.0	7168 21500 6.0	7728 23184 6.0	8288 24860 6.0	8848 26540 6.0
30.0	3.0	24.0	A Q Fb	6000 18000 6.0	6300 18900 6.0	6600 19800 6.0	6900 20700 6.0	7200 21600 6.0	7500 22500 6.0	7800 23400 6.0	8400 25200 6.0	9000 27000 6.0	9600 28800 6.0

BANK HEIGHT FOR CANALS AND FREEBOARD FOR
HARD SURFACE, BURIED MEMBRANE, AND EARTH LININGS



BANK HEIGHT FOR CANALS
AND
FREEBOARD FOR HARD SURFACE, BURIED MEMBRANE, AND EARTH LININGS

REVISED 3-10-60

103-D-341

PROPERTIES FOR CONCRETE-LINED CANALS--
STANDARD SECTIONS A-1 AND A-2

Fig. 5 Par. 1.15A

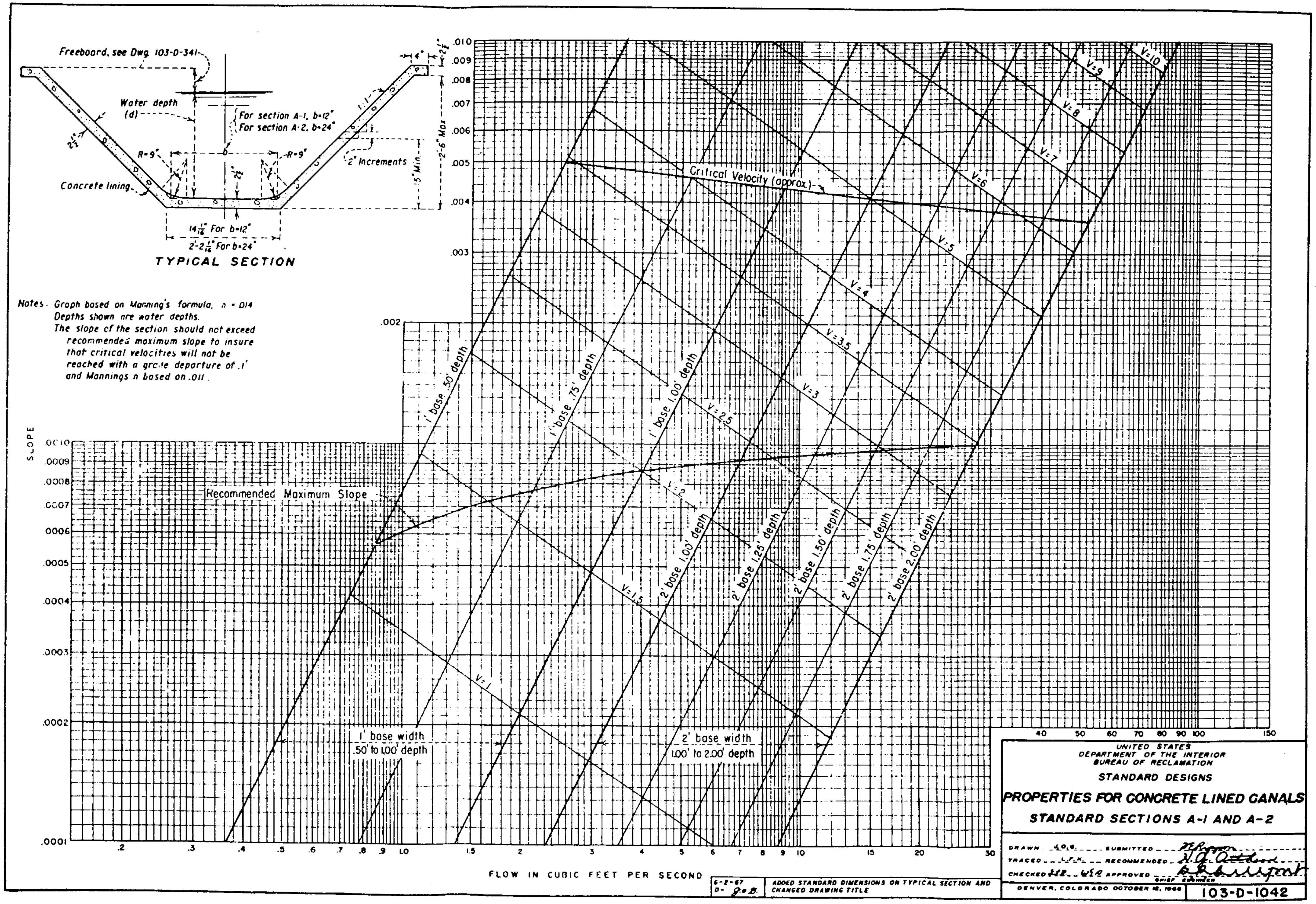
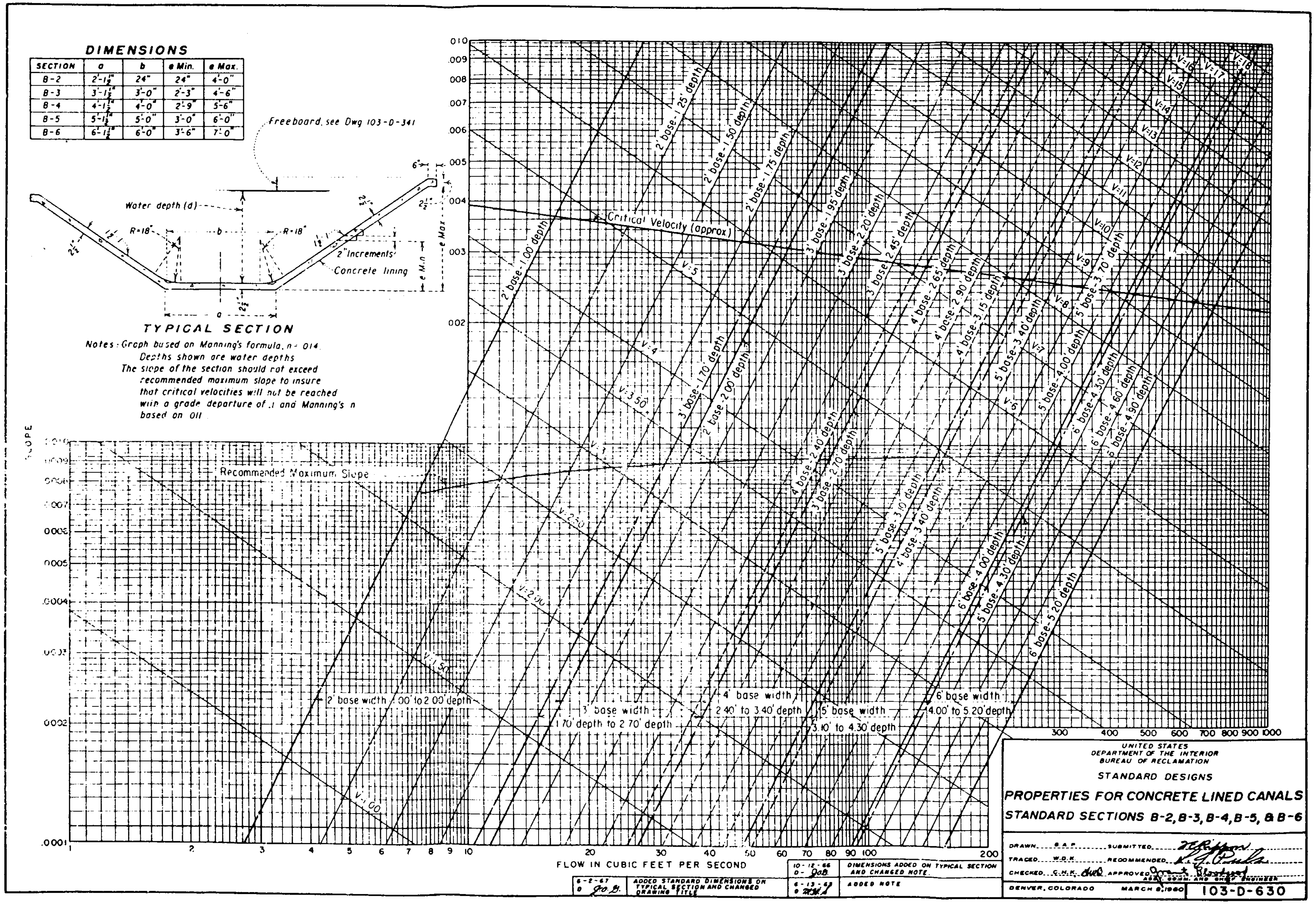


Fig. 6 Par. 1.15A

PROPERTIES FOR CONCRETE-LINED CANALS--
STANDARD SECTIONS B-2, B-3, B-4, B-5, AND B-6



SECTION	a	b	e Min.	e Max.
B-2	2'-1½"	24"	24"	4'-0"
B-3	3'-1½"	3'-0"	2'-3"	4'-6"
B-4	4'-1½"	4'-0"	2'-9"	5'-6"
B-5	5'-1½"	5'-0"	3'-0"	6'-0"
B-6	6'-1½"	6'-0"	3'-6"	7'-0"

THICKNESS OF HARD SURFACE LINING FOR USE IN CANALS

Fig. 7 Par. 1.15A

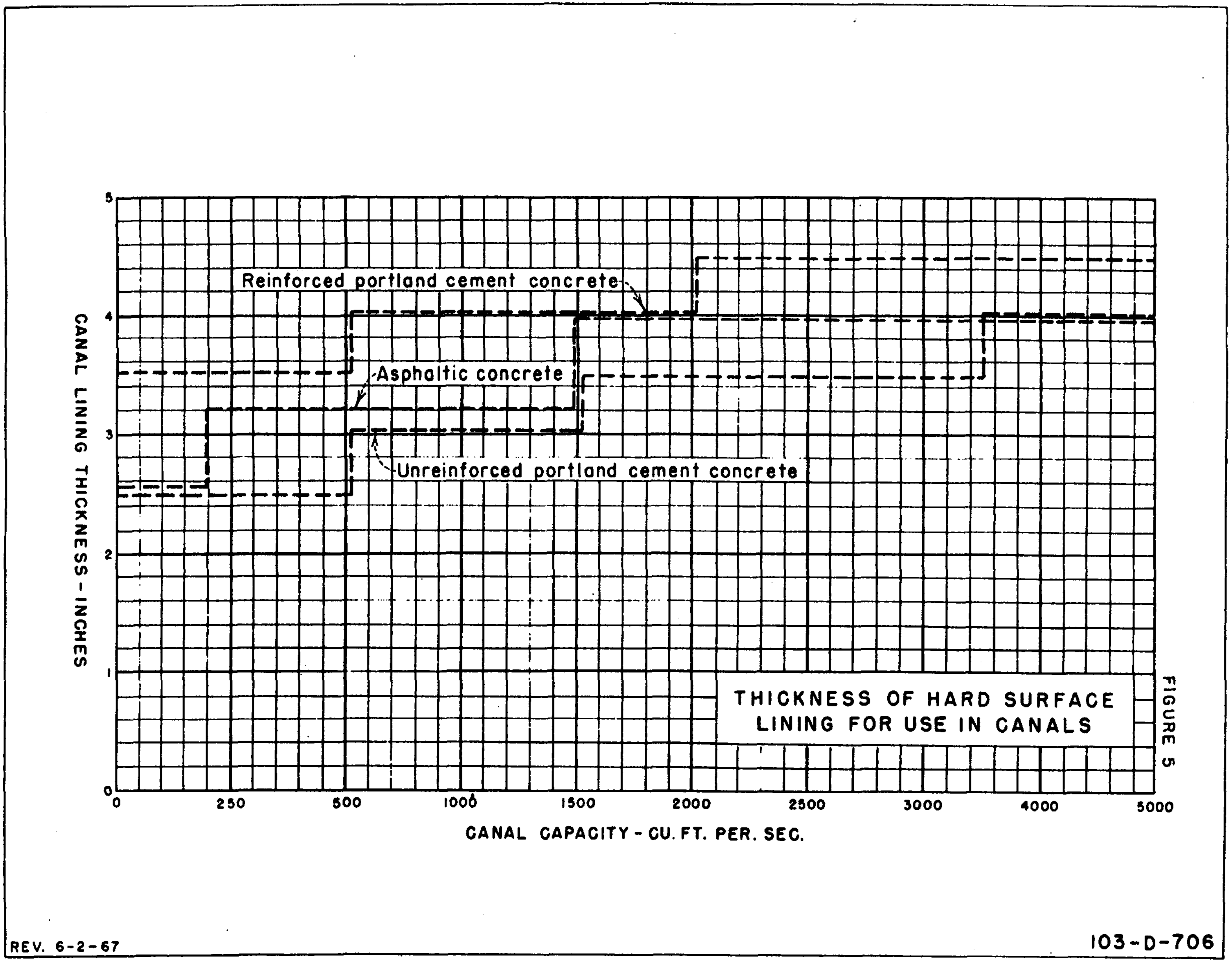
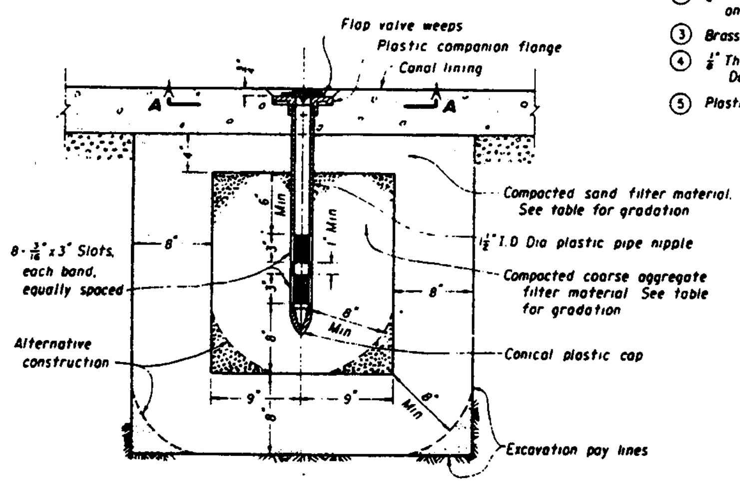


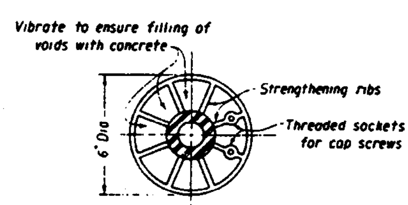
Fig. 8 Par. 1.15A

FLAP VALVE WEEPS

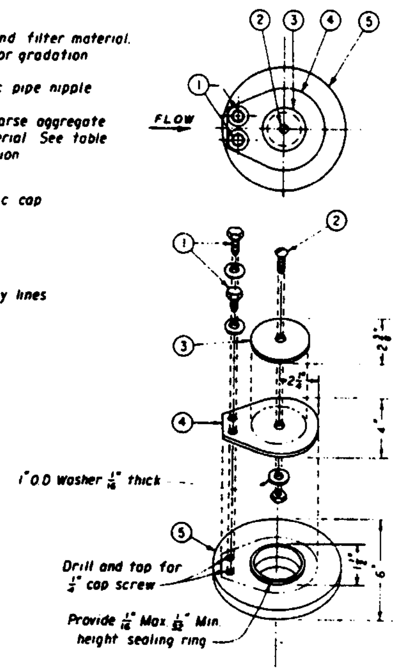
1. $\frac{1}{4}$" Bronze cap screw and washer.
2. $\frac{1}{4}$" N.C Round Head brass machine screw, brass washer and brass nut. Tighten and stake.
3. Brass disk $\frac{1}{8}$" minimum thickness.
4. $\frac{1}{8}$" Thick Neoprene rubber flap with Shore "A" Durometer of 50 to 70.
5. Plastic companion flange.



TYPICAL INSTALLATION



SECTION A-A



FLAP VALVE
DETAIL OF PARTS

GRADATION OF FILTER MATERIALS

FILTER MATERIAL	PERCENT (BY WEIGHT) RETAINED ON STANDARD SIEVE									
	No. 200	No. 100	No. 50	No. 30	No. 16	No. 8	No. 4	3/8"	3/4	1½"
Sand	97-100	93-97	74-85	38-70	15-45	5-25	0-5			
Coarse Aggregate					95-100	75-90	60-80	40-60	20-35	0

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS

FLAP VALVE WEEPS

DRAWN ... SUBMITTED ...
TRACED ... RECOMMENDED ...
CHECKED ... APPROVED ...
DENVER, COLORADO — 103-D-1044

SAFETY LADDER RUNGS FOR
CONCRETE-LINED CANALS

Fig. 9 Par. 1.15A

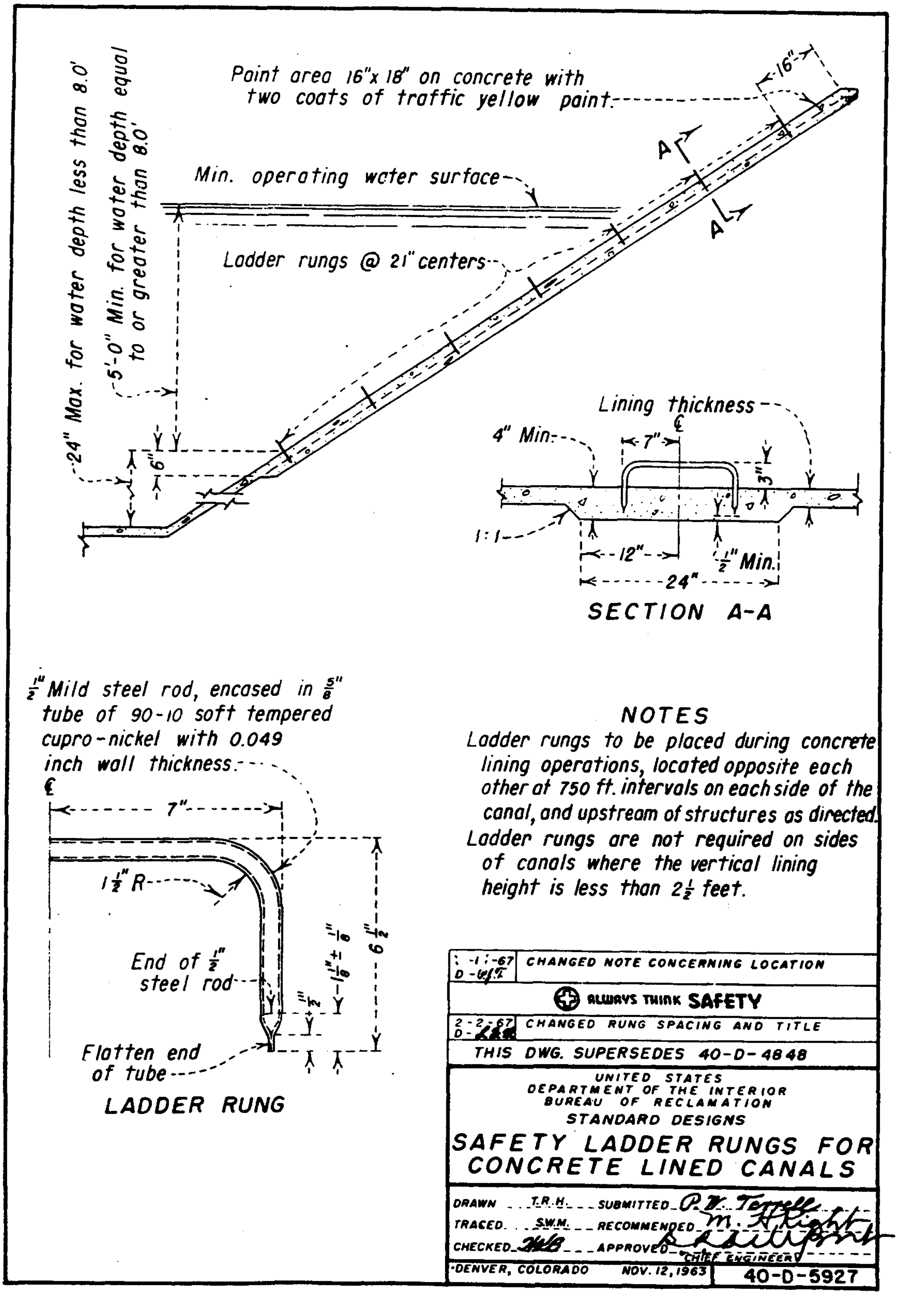
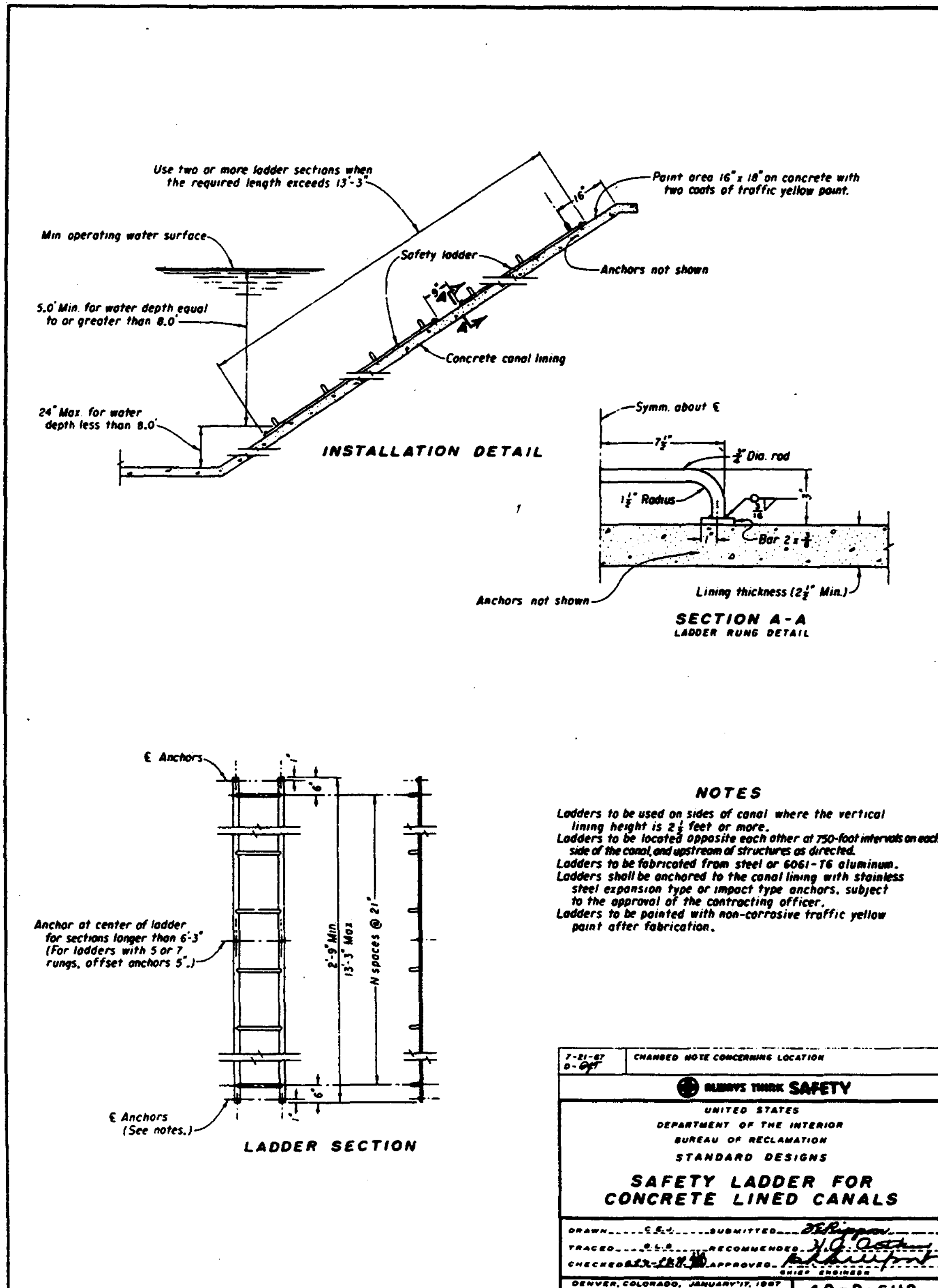
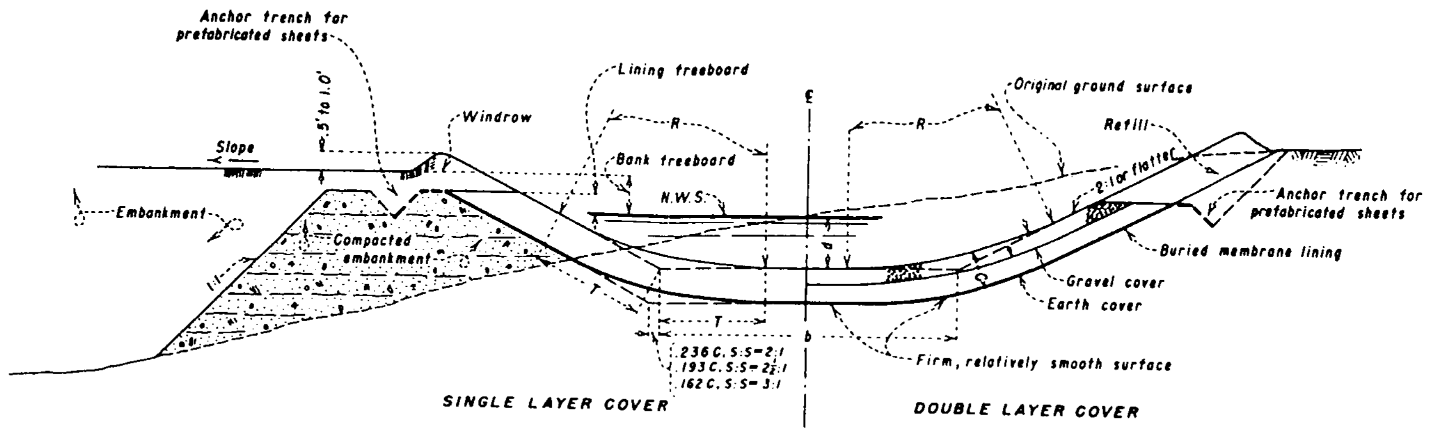


Fig. 10 Par. 1.15A SAFETY LADDER FOR CONCRETE-LINED CANALS

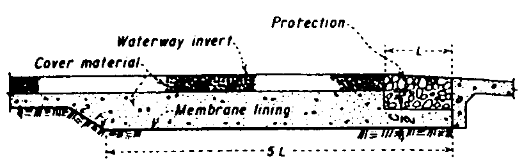


DETAILS OF BURIED MEMBRANE LININGS

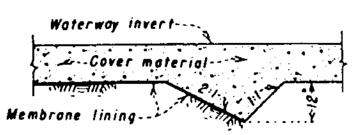


TRANSVERSE SECTION OF WATERWAY

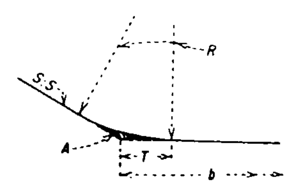
$C = 10" + \frac{d}{12}$; Where C is in inches and d is water depth in inches. (12" Min.)



LONGITUDINAL SECTION
MEMBRANE AT STRUCTURES



LONGITUDINAL SECTION
METHOD OF ENDING MEMBRANE



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	[illegible]3.95	.41	15.31

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.

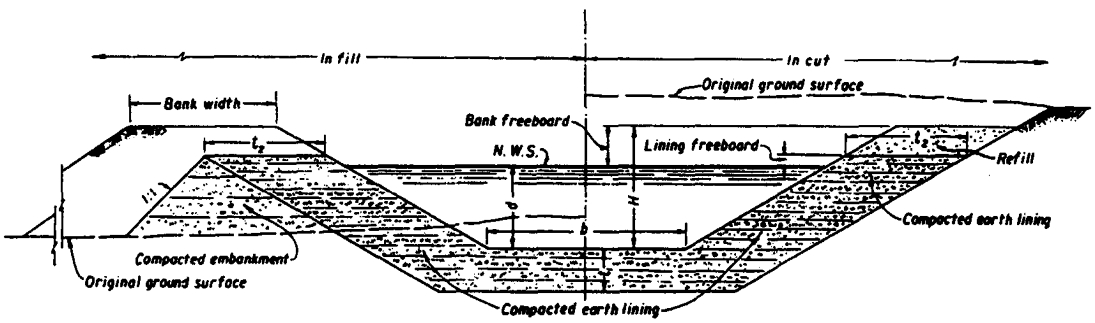
For freeboard see 103-D-341.

DETAILS OF BURIED MEMBRANE LININGS

MAY 26, 1960

103-D-632

TYPICAL EARTH-LINED SECTIONS Fig. 12 Par. 1.15C



TYPICAL EARTH LINED SECTION

TABLE FOR EARTH LINING

d	t_1	t_2
2.0' or less	1.0'	3.0'
>2.0' to 4.0'	1.5'	4.0'
>4.0' to 6.0'	2.0'	6.0'
Over 6.0'	2.0'	8.0'

NOTES

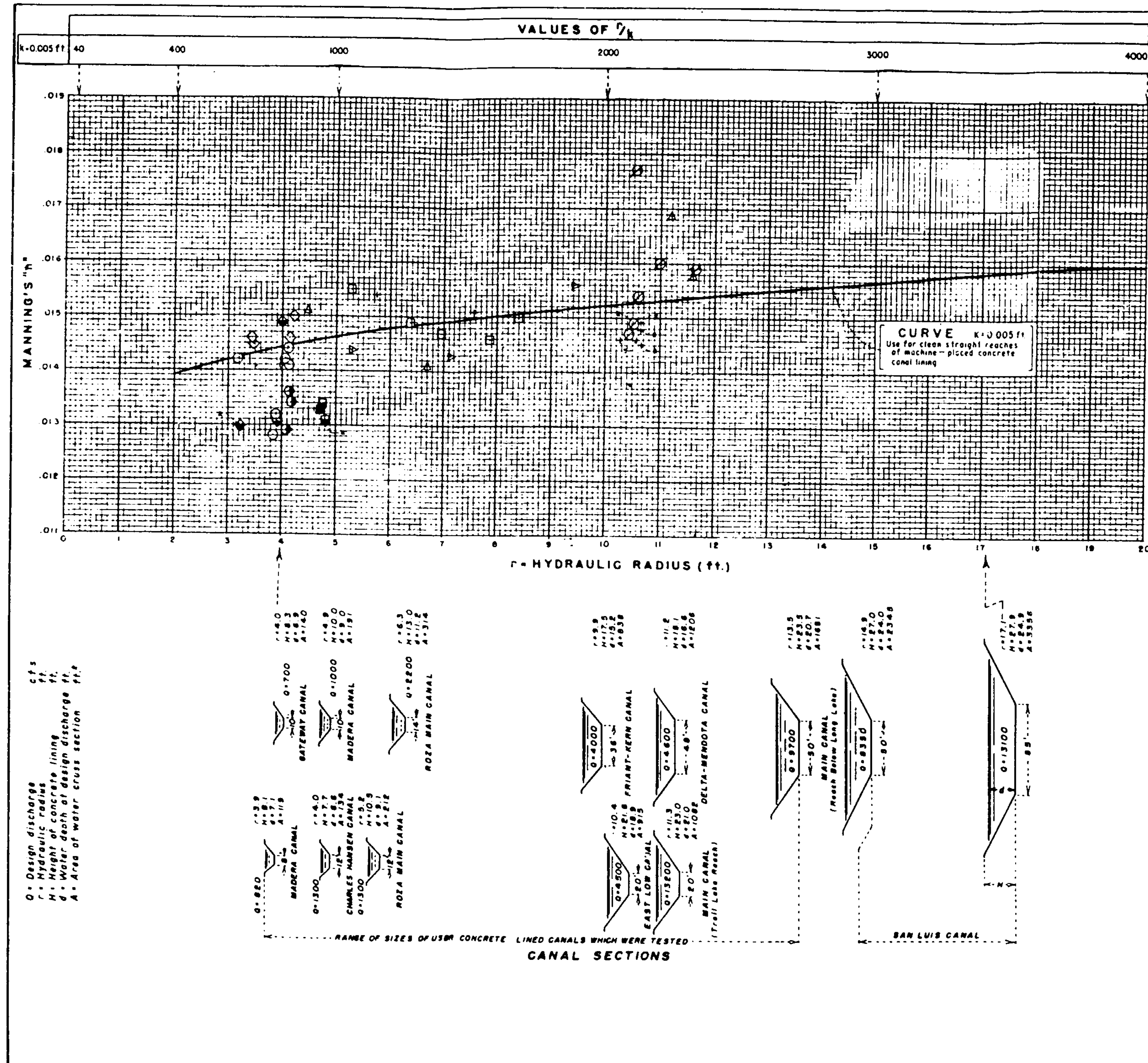
If lining material requires a protective cover of gravel or riprap to prevent scour or erosion, excavation shall be extended to provide for the designated thickness of lining plus the gravel or riprap cover.
For freeboard, see Dwg. 103-D-341 (Rev. 3-10-60)

OCT. 16, 1967

103-D-1043

CONCRETE-LINED CANALS--MANNING'S "n" VALUES FROM PROTOTYPE TESTS

Fig. 13 Par. 1.20B



NOTES

The curve on this chart is based on the following development:

The average velocity of flow (V) in an open channel may be expressed by two formulas.

1. $V = \left[\frac{1.486 r^{\frac{1}{6}}}{n}\right]\left[rS\right]^{\frac{1}{2}}$ (Manning's formula), or by

2. $V = \left[(32g)^{\frac{1}{2}} \log 14.8\left(\frac{r}{k}\right)\right]\left[rS\right]^{\frac{1}{2}}$

Formula 2 is an approximation of the Colebrook-White equation for flow near the turbulent region as developed by Peter Ackers in Hydraulics Research Papers No. 1 and 2, "Resistance of Fluids Flowing in Channels and Pipes," published by the Department of Scientific and Industrial Research of England in 1958.

Equating formula 1 to formula 2 and solving for "n"

$$n = \frac{1.486 r^{\frac{1}{6}}}{(32g)^{\frac{1}{2}} \log 14.8\left(\frac{r}{k}\right)} = \frac{0.0463 r^{\frac{1}{6}}}{\log 14.8\left(\frac{r}{k}\right)}$$

Where n = Manning roughness coefficient
r = Hydraulic radius (ft.)
S = Slope of energy gradient
k = Equivalent sand grain roughness of the rigid boundary surface (ft.) (Typical k values and descriptions of surfaces taken from the research papers mentioned above).
$g = 32.16$ ft./sec.^{2}

Assuming that k is constant for a given type of canal surface, "n" increases with hydraulic radius in the range shown.

The plotted data were taken from straight test reaches or from reaches which contained only 1 or 2 short horizontal curves.

Reaches of Delta-Mendota and Friant-Kern Canals contained structures with piers which extended into the flow prism. Estimated head losses caused by these piers were deducted for computations of "n" values shown.

SOURCE OF PLOTTED TEST DATA

CANAL	SYMBOL	No. OF TESTS	$CuSO_4$ TREATMENT	RANGE OF TEST FLOWS (cfs)
Main (Long Lake)	△	4	No	860 to 8210
Delta-Mendota	⌀	7	No	3370 to 4370
Main (Trail Lake)	⊡	5	No	1090 to 5810
East Low	▷	3	No	875 to 1980
Friant-Kern	+	11	Yes	980 to 4545
Roza Main	⬓	4	No	1065 to 1075
Madera	⊙	7	Yes	485 to 945
Gateway	◑	3	No	724
Charles Hansen (Reach 1)	◇	4	Yes	800 to 1290
Charles Hansen (Reach 2)	◆	4		

Plots of computed "n" values from nine concrete lined canals are shown on diagram at the left and are described below.

This drawing prepared from data given in Bureau of Reclamation Technical Memorandum 661, "Analyses and Description of Capacity Tests in Large Concrete-Lined Canals", April, 1964.

ALWAYS THINK SAFETY

CANAL CAPACITY TEST PROGRAM

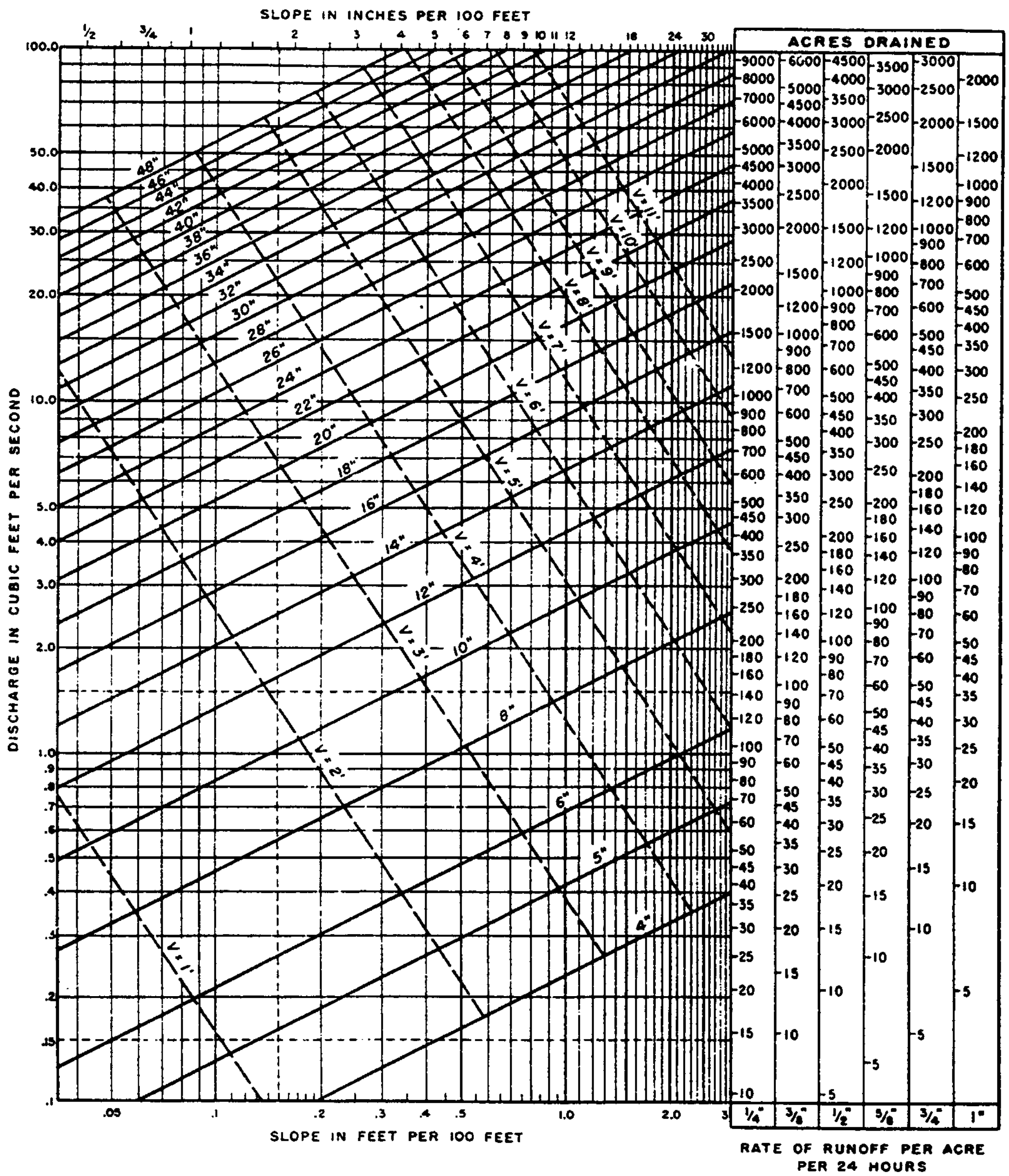
CONCRETE LINED CANALS

MANNING'S "n" VALUES FROM PROTOTYPE TESTS

STRAIGHT REACHES

DECEMBER 6, 1966 103-D-1045

CONCRETE OR CLAY DRAIN TILE--DISCHARGE CURVES Fig. 14 Par. 1.24



DISCHARGE CURVES FOR DRAIN-TILE

BASED ON FORMULA $V = 138 R^{2/3} S^{1/2}$

BUL. 854, U.S. DEPT. OF AGRICULTURE

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GENERAL DESIGN CONSIDERATIONS

.1 This chapter contains design information used for many types of canal structures. The hydraulic and structural designs are generally based on information found in recognized design textbooks. The loadings and standards given in this chapter are adequate for most conditions; however, engineering judgment must be exercised to determine when they do not fit local conditions. In the structural drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated. INTRODUCTION

.2 Design data are prepared or obtained by field forces. It is their responsibility to anticipate the needs of the designers and obtain reliable information on all factors that may influence design. The data should include drawings and other information necessary for the complete design of the structures. The desirable scale for profile drawings is 1 inch equals 10 feet vertical and 1 inch equals 200 feet horizontal, although high density of structures may sometimes warrant a horizontal scale of 1 inch equals 100 feet. Profiles and structures should be oriented on drawings so that the stationing increases from left to right or from bottom to top. For additional information, see Part 133, Design Data Requirements, of the Reclamation Instructions. DATA

REINFORCED CONCRETE DESIGN CRITERIA

.3 The current ACI (American Concrete Institute) Building Code requirements and subsequent ACI standards with exceptions as approved by the Bureau's Division of Design are used as a guide for canal structures. Design criteria for bridges are covered in Chapter 9, Bridges, of this design standards. GENERAL

.4 Most designs for canal structures are based on 3,000-psi strength concrete and reinforcement with a specified minimum yield strength of 40,000-psi or 3,750-psi strength concrete and reinforcement with a specified minimum yield strength of 60,000 psi. Designs may be of the working stress or ultimate strength methods. Figure 1 shows a method for designing a section with combined bending and tension, and Figure 2 shows a method for designing a section with combined bending and compression. ALLOWABLE STRESSES

Reduced steel stresses are used to decrease cracks in the concrete, thereby minimizing leakage. In rectangular boxes the reduced allowable reinforcement stresses for bursting head only (measured from the center of the box) are as follows:

Reduced stress (psi)	Maximum head (feet)
20,000	10
18,000	16
17,000	22
16,000	28
15,000	34

In monolithic pipe the reduced allowable reinforcement stresses for bursting head only are as follows:

Reduced stress (psi)	Maximum head (feet)
16,000	50
14,000	75
12,500	125

2.5

REINFORCED CONCRETE DESIGN CRITERIA--Continued

ALLOWABLE STRESSES (Cont'd.) These reduced stresses are the maximum allowable for bursting regardless of the use of higher strength steels.

BAR SPACING .5 Spacing of parallel bars should be shown on reinforcement drawings as center to center of adjacent bars. ACI 318-63 defines the minimum center to center spacings for parallel bars not in contact. The maximum reinforcement spacing shall be twice the thickness of the member for stress bars and three times the thickness of the member for temperature bars. In either case the maximum spacing is 18 inches with a preferred limit of 12 inches.

SPLICING OF BARS .6 Lengths of splices for reinforcement bars are shown in Figure 3 or Figure 4 as applicable. These lengths are for high-bond bars.

PROTECTIVE COVER .7 Concrete protection for reinforcement is given in Chapter 8 of ACI 318-63, and in Figures 3, 4, and 5 of this chapter. The dimensions given are clear cover, that is the outside of the bar to the surface of the concrete. However, on all design or placing drawings, dimensioning of bar locations should be given to the centerline of the bars unless otherwise noted. If a discrepancy should exist between ACI 318-63 and Figures 3, 4, or 5 as to the amount of protective covering, the greater protective cover indicated should govern. Protective cover over reinforcement in the top face of concrete exposed to scour shall be increased by one-half inch if the water velocity exceeds 10 feet per second and an additional one-half inch for each increment of velocity of 10 feet per second.

BOND AND ANCHORAGE REQUIREMENTS .8 Allowable bond and anchorage stresses shall be governed by ACI 318-63, Chapter 13 for working stress design, and Chapter 18 for ultimate strength design. Tables 14a and 14b of the ACI "Reinforced Concrete Design Handbook" give the maximum allowable bond stresses for various conditions and bar sizes.

SHEAR REQUIREMENTS .9 Allowable shear stresses shall be covered by ACI 318-63, Chapter 12 and Chapter 17, for working stress design and ultimate strength design, respectively.

MINIMUM OR TEMPERATURE REINFORCEMENT .10 Except for very small structures, the following criteria shall be used to determine the cross-sectional area of temperature or minimum reinforcement required in canal structures. The percentages indicated are based on the gross cross-sectional area, not including fillets, of the concrete to be reinforced. Where the thickness of the section exceeds 15 inches, a thickness of 15 inches should be used in determining the temperature or minimum reinforcement.

A. The minimum reinforcement for canal structures shall be No. 4 bars at 12 inches in all exposed faces and where reinforcement is placed in a single layer, and No. 4 bars at 18 inches in unexposed faces with two-layer reinforcement.

B. Single-layer reinforcement:

(1) Reinforced concrete linings 4 inches and less in thickness with discontinuous wire-fabric reinforcement and weakened planes at 12- to 15-foot centers . . . . 0.10 percent

(2) Slabs and linings not exposed to freezing temperatures or direct sun with joints not exceeding 30 feet . . . . 0.25 percent

REINFORCED CONCRETE DESIGN CRITERIA--Continued

MINIMUM OR TEMPERATURE REINFORCEMENT (Cont'd.)

(3) Slabs and linings exposed to freezing temperatures or direct sun with joints not exceeding 30 feet . . . . 0.30 percent

(4) Slabs and linings exceeding 30 feet between joints
Category (2) above . . . . 0.35 percent
Category (3) above . . . . 0.40 percent

(5) Walls and other structural members
Total percentage of horizontal reinforcement to be equal to the sum of those required for both faces as determined below.

C. Double-layer reinforcement:

(1) Face adjacent to earth with joints not exceeding 30 feet . . . . 0.10 percent

(2) Face not adjacent to earth nor exposed to freezing temperatures or direct sun and with joints not exceeding 30 feet . . . . 0.15 percent

(3) Face not adjacent to earth but exposed to freezing temperatures or direct sun and with joints not exceeding 30 feet . . . . 0.20 percent

(4) If member exceeds 30 feet in any direction parallel to reinforcement, add to the reinforcement requirement in that direction because of the increased length . . . . 0.05 percent

(5) If a slab is fixed along any line, double the dimension from line of fixity to free end to determine whether reinforcement is within the less than 30 feet or more than 30 feet percentages shown under (1), (2), (3), and (4) above.

.11 Minimum pin diameters for bent bars shall conform to the tabulation on the applicable Figure 3 or Figure 4. An adequate radius shall be provided to prevent crushing of the concrete, when bends are made at points of high stress (see Figure 6). REINFORCEMENT BAR BENDS

.12 Cantilever walls shall have a minimum thickness at the base equal to 1 inch per foot of height (5 inches minimum) up to 8 feet; above 8 feet the minimum thickness at the base shall be 8 inches plus 3/4 inch for each foot in height above 8 feet. In general, vertical walls over 8 feet high shall have two-layer reinforcement. Transition buttresses normally have the following thicknesses and reinforcement: MINIMUM WALL THICKNESS

Height of buttress (feet)	Thickness of buttress (inches)	No. of layers of bars	Size of bars	Spacing of bars (inches)	Location of bars
0 to 10	8	1	No. 4	12	Center
10 to 15	10	1	No. 5	12	Center
15 to 20	12	1	No. 5	12	Center

2.13

REINFORCED CONCRETE DESIGN CRITERIA--Continued

CUTOFF WALLS .13 Cutoffs are provided to reduce percolation around structures, to prevent movement of structures, and to make transitions more rigid. Cutoffs are required at the ends of structure transitions in concrete-lined canals as well as in other lined or earth canals. Cutoff walls should, in general, be a minimum of 24 inches deep, measured perpendicular to the inside of the structure, for water depths up to 3 feet over the cutoff, 2 feet 6 inches for water depths of 3 to 6 feet, and 3 feet for water depths over 6 feet. For some small structures, 18-inch cutoffs may be satisfactory. The minimum concrete thickness should be 6 inches for 18- and 24-inch cutoffs, 8 inches for 2-foot 6-inch and 3-foot 0-inch cutoffs. In soils that are unusually susceptible to piping, the cutoff should be extended horizontally or vertically, or both, to provide adequate protection against percolation. If minimum cutoffs are specified, a note should be added requiring cutoff extension with unreinforced concrete as directed. This will permit deeper and wider cutoffs to be used where excavation discloses poor soils without complicating reinforcement cutting and bending. The vertical reinforcement in the cutoff is usually the same as the longitudinal reinforcement in the transition floor. If only one layer of reinforcement is used in a cutoff, the vertical reinforcement should be placed in the center of the cutoff wall.

JOINTS IN STRUCTURES .14 Construction and contraction joints are often used in concrete and canal structures.

Construction Joints A. Construction joints are joints which are purposely placed in structures to facilitate construction or which occur in structures as a result of inadvertent delays in concrete placing operations. Construction joints are located to facilitate the contractor's operations, to reduce initial shrinkage stresses and cracks, to allow time for the installation of embedded metalwork, or to allow for the subsequent placing of other concrete, backfill concrete, or second-stage concrete. Bond is required at construction joints regardless of whether or not reinforcement is continuous across the joint.

Contraction Joints B. Contraction joints are joints placed in structures or slabs to provide for volumetric shrinkage of monolithic unit or movement between monolithic units. The joints are so constructed that there will be no bond between the concrete surfaces forming the joint. The joints are made by forming the concrete on one side of the joint and allowing it to set before concrete is placed on the other side of the joint. The surface of the concrete first placed at a contraction joint shall be coated with sealing compound before the concrete on the other side of the joint is placed. If steel bars or dowels extend across the joint, one end of the bar should be coated or wrapped with paper to prevent bonding to the concrete. A sponge-rubber elastic filler should be provided at the joint if expansion is expected. Rubber waterstops with center bulb should be placed across joints where it is necessary to prevent water from passing through the joint. Plastic waterstops may be used in lieu of rubber waterstops for low-head structures.

FILLETS .15 Fillets are often used to provide increased strength or relieve stress concentrations at points of maximum stress, and to facilitate the placement of concrete. The sizes of fillets usually provided at the inside corners of box sections and at the bases of vertical cantilever walls are as follows:

REINFORCED CONCRETE DESIGN CRITERIA--Continued

FILLETS (Cont'd.)

Size of fillet (inches)	Size of box section (feet)	Vertical clear height of cantilever wall (feet)
2 by 2	0 to 4.00	--
3 by 3	4.01 to 6.00	0 to 8.00
4 by 4	6.01 to 8.00	8.01 to 12.00
6 by 6	Over 8.00	Over 12.00

Fillets may be omitted where objectionable in the design of certain structures.

.16 Precast-concrete culvert pipe and irrigation pipe designs shall be in accordance with current ASTM specifications. Precast-concrete pressure pipe and cast-in-place concrete pipe designs shall conform with current Bureau standards. CONCRETE PIPE

LOADINGS

.17 Owing to the nature of some canal structures, unusual loading conditions often exist. The structures are subjected to changing effects such as foundation reactions, temperature stresses, exposure conditions, frost heaving, and varying earth and hydrostatic loadings. GENERAL

.18 The commonly used dead-load weights, in pounds per cubic foot, are as follows: DEAD LOADS

Water	62.5
Dry earth	100
Compacted earth	120
Saturated earth	135
Concrete	150

Some soils may require variations from the above weights. The horizontal fluid pressure of dry earth is usually about 30 pounds per square foot per foot of depth, and that of saturated earth is about 87.5 pounds per square foot per foot of depth. Where the backfill slopes up from the structure, an additional horizontal force is exerted on the wall. Figures 7, 8, and 9 may be used for computing the horizontal earth pressure and moment. Where construction or operating equipment may come close to a structure or where some slope fill may develop, a surcharge equal to 2 feet of dry earth is normally added, resulting in an additional horizontal pressure of 60 pounds per square foot.

.19 Operating platforms where stoplogs are not used should be designed for a live load of 100 pounds per square foot, and where stoplogs are used, 150 pounds per square foot. In addition to the weight of the radial-gate hoists and equipment, the operating platforms for radial gates should be designed for the rated capacity of the hoist acting on either cable. LIVE LOADS

.20 Designs for vertical walls without compacted backfill must include the maximum expected internal water depth on the walls. The internal waterload is reduced by the active dry-earth backfill load. The waterload can usually be neglected when compacted backfill is placed against the opposite side of the VERTICAL WALL LOADS

LOADINGS--Continued

VERTICAL WALL LOADS (Cont'd.)

wall. If the wall is not backfilled, the design should be based on internal water to the top of the wall, plus ice pressure where it may occur, and normal stresses used to resist the resultant moment. The backfill load must be considered regardless of compaction. If the backfill can become saturated, the additional load must be used for design.

SLOPED WALLS

.21 Sloped walls are usually designed to be supported by the foundation upon which it is placed. See Figure 10 for resulting pressures on sloped walls. Buttress walls are often used for large transitions and the maximum loads from both sides of the wall must be considered when determining how the walls will be supported.

HIGHWAY AND RAILROAD LOADS

.22 Figure 11 shows loadings and distribution of loads commonly used for highway and railroad structures. Some State highway commissions and railroad companies have special requirements and standards which must be followed for structures on their right-of-way.

LOADS ON CIRCULAR CONDUITS

.23 Bureau of Reclamation Engineering Monograph No. 6, "Stress Analysis of Concrete Pipe," by H. C. Olander, describes the loadings and a method of analysis for designing buried circular conduits. Also, ultimate load analysis is being used as a method of designing precast and monolithic siphon barrels.

HYDRAULICS

FLOW FORMULAS

.24 The Manning formula (Paragraph 1.12) is generally used for designing canals and related structures except in precast concrete pipe distribution systems. Scobey's formula is generally used for precast concrete pipe distribution system designs with the recommended value of C_s given in the Bureau's Hydraulic and Excavation Tables. These tables also include tabular solutions for Manning's formula and many other tables which are useful in designing canals and structures.

HEAD LOSS

.25 The more common head losses are due to friction, transitions, bends, trashracks, and changes in cross-sectional area. The roughness coefficients for unlined and lined canals are given in Chapter 1.

Friction Loss

A. In open channel systems a roughness coefficient "n" of 0.014 is used for all monolithic-concrete structures except conduits. If constructed with steel forms, monolithic-concrete pipe and tunnels use a roughness coefficient of 0.013. A roughness coefficient of 0.013 is also used for precast concrete pipe. For closed distribution systems carrying clear water, Scobey's formula is used. A coefficient of retardation, C_s, of 0.345 is used for pipe diameters 22 inches and smaller and a C_s of 0.370 for pipe sizes 24 inches and greater.

Transitions

B. Transitions are generally used at the inlet and outlet of structures and where changes occur in the water section. An accelerating water velocity usually occurs in inlet transitions and a decelerating velocity in outlet transitions. The most common types of open transitions to closed conduits are the streamlined warp, straight warp, and broken back. ("Broken back" refers to the intersection of the vertical and sloping plane surfaces on the sides of the transition; the type is also sometimes referred to as "Dog leg".)

Inlet transitions for minimum hydraulic loss and smooth operation should have a submergence or seal of 1.5 (h_{vp} - h_{vc}) or 3 inches minimum measured between the upstream water surface of the inlet transition and the opening in the transition headwall. Outlet transitions should have no

HYDRAULICS--Continued

submergence of the opening in the headwall. If the submergence exceeds one-sixth of the depth of the opening at the outlet, the hydraulic loss should be computed on the basis of a sudden enlargement rather than as an outlet transition. The hydraulic loss in a transition will depend primarily on the difference between the velocity heads at the open end of the transition and at the normal to centerline section of the closed conduit at the headwall, or Δh_v. Coefficients of Δh_v considered adequate for determining hydraulic losses in transitions are tabulated below. Transitions (Cont'd.)

Type of open transition to closed conduit	Inlet	Outlet
Streamlined warp to rectangular opening	0.1	0.2
Straight warp to rectangular opening.	0.2	0.3
Straight warp with bottom corner fillets to pipe opening.	0.3	0.4
Broken back to rectangular opening	0.3	0.5
Broken back to pipe opening	0.4	0.7
Closed transition		
Square or rectangular to round (maximum angle with centerline = 7-1/2°)	0.1	2

Open transitions to multiple closed conduits will involve some additional hydraulic loss. Average friction loss should be added for large transitions, but it may be neglected for small transitions. The slope of the floor on a broken-back outlet transition should be 6:1 or flatter.

The maximum angle between the water surface and the centerline should not exceed 27-1/2° for inlet transitions and 22-1/2° for outlet transitions for the best hydraulic conditions. In some structure designs it may prove economical to use 25° to allow the same structure to be used for both inlets and outlets. A 30° angle is often used on inlet transitions with checks, in which case an additional loss is allowed for the check. Designs should provide for a loss through most check structures of about 0.5 times the difference in velocity head through the check opening and the upstream canal section.

Where an inlet transition connects to a free-flow closed conduit in such a way that the conduit inlet is sealed, the quantity of water that is passed should be determined by the orifice equation. The head should be measured from the center of the opening to the inlet water surface and an orifice coefficient of $c = 0.6$ should be used. A small correction factor is theoretically required when the submergence is less than the height of the opening. When the inlet to a long conduit may operate without sealing, a hydraulic jump may occur that can result in blowback and undesirable operation. Figure 12 can be used to determine the probability of blowbacks existing in any particular structure. Transitions to free-flow conduits can have the control point anywhere between the inlet cutoff and the headwall. If control at any flow is at the inlet cutoff, the upstream channel must be protected from erosion or the design changed to move the point of control to the transition.

C. The head loss in feet for bends in closed conduit may be computed from the formula, Bends

2.25D

HYDRAULICS--Continued

Bends (Cont'd.)

$$h_b = Z\left(\frac{\Delta}{90}\right)^{1/2}\frac{V^2}{2g}$$

$$= \zeta\,\frac{V^2}{2g}$$

where

Z = a coefficient based on radius of pipe bend and diameter of pipe, and

Δ = deflection angle of pipe bend.

ζ may be obtained from Figure 13. Hinds head loss based on Z = 0.25 is also shown and, although conservative, it may be adequate for preliminary estimates. Figure 14 shows methods for computing compound pipe bends.

Trashracks

D. Trashrack losses may be estimated as follows:

Velocity through rack (fps)	Loss in feet
1.0	0.10
1.5	0.30
2.0	0.50

More accurate losses may be obtained from Figure 15.

Piers

E. The cumulative backwater effect of structure piers in the canal prism shall be considered in the design of canals on very flat grades.

Miscellaneous

F. Coefficients for use in the orifice formula for flow through submerged tubes may be obtained from Figure 16. Figure 17 provides a quick method of determining approximate critical depth in trapezoidal sections.

FREEBOARD

.26 The freeboard for the end of transitions adjacent to hard-surface or buried-membrane canal lining is usually the same as that of the lining. In unlined and earth-lined canals, the minimum freeboard at transition cutoffs for siphons, tunnels, and similar structures should be as follows:

Water depth at cutoff (feet)	Minimum freeboard (inches)
0 to 1.25	6
1.26 to 2.00	9
2.01 to 5.00	12
5.01 to 7.00	15
7.01 to 9.00	18
9.01 to 12.00	21

For small structures such as transitions connecting to 24-inch-diameter pipe and smaller, the top of the transition walls may be level. For larger structures, the freeboard at the transition headwall should be greater than at the cutoff; this difference in freeboard will increase as the size of the structure increases.

Chapter 1 gives the recommended freeboard for canals. The earth bank freeboard should be increased 50 percent (1.0 foot maximum) adjacent to siphons, wasteways, and checks without overflows, to prevent washouts at these locations due to more floodwater being taken into the canal than allowed for in

HYDRAULICS--Continued

FREEBOARD (Cont'd.)

designs or by improper operation. The increased freeboard is to be extended away from the structure to a point where the least damage due to overtopping will occur, or a minimum distance of 50 feet.

PERCOLATION

.27 Where water is confined in an area above a point of relief, such as above a check structure, there is a tendency for the water to flow along the structure or through the earth to the lower point of relief. The type of structure and the nature of the soil will govern the amount and rate of the flow. The percolation factor should be at least 2.5:1 as computed by Lane's weighted creep method, and 3.5:1 on a straight path. Straight path factors of 5:1 are common. Larger factors may be required where warranted by the type of soil or importance and type of structure.

The percolation path may be increased by adding length or cutoff walls to most structures. Cutoff walls must be far enough apart to prevent a short circuit between the ends of the cutoffs. The cutoffs must be so spaced that the actual distance between the cutoffs will be at least one-half the weighted creep distance along the structure between the ends of the cutoffs. For computing weighted creep distance, the horizontal distance is considered to be one-third as effective as the vertical distance.

OVERTURNING AND SLIDING

.28 All structures must be checked for stability. Especially, small check structures often require additional length to prevent overturning or sliding for maximum upstream and minimum downstream water surface. The sliding factor, defined as the ratio of the horizontal forces to the total weight reduced by uplift, should not exceed 0.35 for most conditions. Cutoff walls may be added to increase the sliding resistance.

HYDRAULIC JUMP

.29 A treatment of hydraulic jump and critical depth and their application to design is given in Appendix A to this chapter, which is a reprint of an article by Julian Hinds entitled "The Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures." See also Figures 17, 18, and 19.

RIPRAP

GENERAL

.30 Riprap protection is often used adjacent to structures and at other locations in earth-surfaced canals where erosion may occur. The local conditions must be considered in determining the type and amount of protection to be provided. These conditions include the cost of riprap, cost of gravel, danger to structures and crops or to human life should scour occur, rodent protection, type of soil, and velocity of water. In areas where riprap and gravel are scarce, consideration should also be given to stockpiling riprap under the construction contract for later use by operation and maintenance forces. The following protection requirements are to be used as a guide only. Types of protection are identified herein for convenience in discussing the protection requirements. The types shown represent minimum sizes and amounts of material to be used, and adjustments should be made to meet the local conditions mentioned above.

Type 1--6-inch coarse gravel
Type 2--12-inch coarse gravel
Type 3--12-inch riprap and 6-inch sand and gravel bedding
Type 4--18-inch riprap and 6-inch sand and gravel bedding

Except for cross-drainage structures, Type 3 minimum protection should be used where velocities exceed 5 feet per second, regardless of depth.

2.31

RIPRAP--Continued

SIPHONS AND TUNNELS

.31 The following protection is considered minimum for siphons and tunnels in earth-surfaced canals:

d = water depth adjacent to structure (feet)	Inlet	Outlet
0 to 2.00	None	None
2.01 to 3.50	None	Type 1
3.51 to 7.00	Type 1	Type 2
7.01 to 10.00	Type 2	Type 3

Water depths over 10 feet require special consideration.

Protection called for on inlets may be omitted if the velocity is less than 2.5 feet per second.

Where protection is required on inlets, length = d (3.0 feet minimum).

Where protection is required on outlets, length = 2.5d (5.0 feet minimum).

PARSHALL FLUMES, CHECKS, CHECK DROPS, INCLINED DROPS, CHUTES, AND CLOSED-CONDUIT DROPS

.32 The following protection is considered minimum for Parshall flumes, checks, check drops, inclined drops, chutes, and closed-conduit drops with control section on concrete, that is, where critical depth does not occur off the concrete. Where critical depth may occur off the concrete, the next higher type of protection should be used at the inlet.

d = water depth adjacent to structure (feet)	Inlet	Outlet
0 to 2.00	None	Type 2
2.01 to 3.50	None	Type 2
3.51 to 7.00	Type 1	Type 3
7.01 to 10.00	Type 2	Type 4

Water depths over 10 feet require special consideration.

Protection called for on inlets may be omitted if velocity is less than 2.5 feet per second.

Where protection is required on inlets, length = d (3.0 feet minimum).

Length of protection on outlets = 2.5d (5.0 feet minimum).

Where turbulent water may occur at the outlet, the length of protection should be increased to 4d.

Gates or stoplogs near the outlet increase turbulence.

TURNOUTS

.33 Protection is not required on the inlets to most small turnouts. If the turnout capacity is 50 percent or more of the capacity of the canal, the protection recommended for siphon inlets should be used. Protection at the outlets of turnouts should be the same as for siphons, based on the water depth in the lateral adjacent to the outlet transition.

RIPRAP--Continued

CROSS-DRAINAGE STRUCTURES

.34 The following protection is considered minimum for cross-drainage structures with concrete transitions.

Q (cfs)	Inlet	Outlet	Outlet length (feet)
0 to 30	None	Type 2	8
31 to 90	None	Type 2	12
91 to 240	Type 1	Type 3	16
241 to 600	Type 2	Type 4	22

Capacities over 600 cubic feet per second require special consideration.

Where the conduit slope is steep enough to produce a velocity over 15 feet per second at the end of the conduit, use the protection type for the next higher discharge (Type 3 minimum). Where special energy dissipators are provided at the outlet, such as baffled outlets, the protection may be reduced or eliminated as local conditions warrant.

PIPE EARTHWORK

GENERAL

.35 Details of earthwork for pipe used in connection with open irrigation systems are shown in Figures 20 through 28. Details of earthwork for pipe used in closed distribution systems are shown in Figure 29.

AUTOMATIC AND REMOTE OPERATION OF WATER CONVEYANCE SYSTEMS

GENERAL

.36 In the design of individual structures as well as complete systems consideration should be given to automatic and/or remote controls. A dependable source of power is required to operate the conveyance facilities and the communication system. Auxiliary emergency power units are often placed at key points.

MEASURING STATIONS

.37 The need for obtaining frequent information about water stage and rates of flow in distribution systems and for obtaining information from inaccessible stations has made remotely monitored stations necessary on some projects. The capability to be built into a particular water measuring station will depend upon the known and anticipated requirements of the project. In some cases, a local record of the water stage on a strip recorder will be sufficient, while in other cases complete information of water stage, rates of flow, and other parameters must be transmitted to a control center.

GATE CONTROLS

.38 Gates are automated to control and maintain a water stage either upstream or downstream of the gate. The very basic components of an automatic gate are: a water stage sensor, a means of converting this information to commands, and the necessary mechanization to move the gate as commanded. In open flow waterways, the location or elevation of the water surface can be determined mechanically by a float or electrically with a probe. Then by having the float or probes activate electric circuits, the gate can be raised or lowered as required to retain the desired water surface elevation. Automatic gate controls should be considered for use with gates on structures such as diversion dams, canal headworks, and canal checks.

2.39

AUTOMATIC AND REMOTE OPERATION OF WATER CONVEYANCE SYSTEMS--Continued

SUPERVISORY CIRCUITS AND REMOTE CONTROLS

.39 Data such as reservoir elevation, pipeline pressures, canal flow depths, and gate and valve positions may be transmitted to a control center by means of supervisory circuits. Gates and valves may be opened, closed, or adjusted, and pumps started or stopped by means of remote control. It is possible with adequate supervisory circuits and remote controls to operate a distribution system from a control center. The mode of communication is either radio or wire.

SAFETY FEATURES

.40 Automatic closure of gates or stopping of pumps may be incorporated in a system to provide for protection of the canal against excessive flows. Alarms at a central control are sometimes incorporated with the supervisory circuits to warn of impending dangers to the system.

APPENDIX A--THE HYDRAULIC JUMP

The Hydraulic Jump and Critical Depth in the Design of Hydraulic Structures

How Established Principles May Be Applied to the Design of Canals and Other Works—A Study Based on the Laws of Conservation of Energy and of Linear Momentum

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THE hydraulic jump and the critical depth have recently come to be recognized as factors of considerable importance in the design of open channels and related hydraulic structures. An excellent technical discussion of this subject will be found in the *Transactions* of the American Society of Civil Engineers, Vol. LXXX, p. 338, in a paper on "The Hydraulic Jump in Open Channel Flow," by Karl R. Kennison, with discussions by a number of prominent engineers. Also, Messrs. Ward, Riegel and Beebe in "Technical Reports, Part III," issued by the Miami Conservancy District, in 1917, present an interesting discussion of the problem, and submit valuable experimental data on the action of the jump below reservoir outlet works. E. W. Lane, in *Proceedings* of the American Society of Civil Engineers, December, 1919, discusses the occurrence of the hydraulic jump in connection with experimental work on flow through contractions.

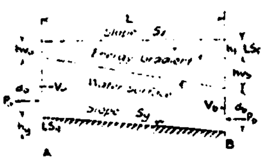


FIG. 1. STEADY FLOW WITH CHANGING VELOCITY. UNIFORM CHANNEL

It is not the intention of this paper to add to the fundamental theories already advanced, but an attempt will be made to show how the established principles may be applied to the design of canals and canal structures.

It is assumed throughout this discussion that the kinetic head is truly represented by the velocity head as computed from the mean velocity.

Practically all formulas previously proposed for the solution of hydraulic jump problems are limited to rectangular sections. While such a limitation simplifies the computations it in no way simplifies the fundamental conceptions, and an attempt will be made to keep the discussion general.

The notation used herein is as follows:

A = Area of water prism = $f(d)$.
b = Width of rectangular channel.
d = Depth of water in any channel.
d_1, d_2, d_a, etc. = Depth of water at Stations 1, 2, A, etc.
d_c = Critical depth.
F = Force producing change in momentum.
$f(d)$ = A = Area of water prism.
$f'(d)$ = T = First derivative of A with respect to d.
g = Acceleration of gravity.
H = Energy of flow = $d + hv$.
h_f = Fall in energy gradient = friction (or other) loss.
h_g = Fall in bottom of channel.
hv = Velocity head.
hv_1, hv_a, hv_b, etc. = Velocity heads at Stations 1, A, B, etc.
hv_c = Velocity head for critical velocity.
L = Distance between consecutive stations.
M = Mass.
P = Hydrostatic pressure on vertical plane across the water prism, the unit force being the weight of a cubic unit of water.
P_1, P_2, P_a, etc. = Values of P at Stations 1, 2, A, etc.
Q = Discharge in cubic units per second.
S_f = Slope of energy gradient.
S_g = Slope of canal bottom.
T = Width of channel at water surface.
t = Time interval.
V = Velocity.
V_1, V_2, V_a, V_b, etc. = Velocities at Stations 1, 2, A, B, etc.
V_c = Critical velocity.
W = Weight of a cubic unit of water.
X = Ratio of T to d for a triangular channel.

As a fundamental basis for discussion it will be necessary to accept the law of the conservation of energy and the law of the conservation of linear momentum. The former law will appear as Bernoulli's theorem, i.e., the elevation of the water surface at any point in a channel plus the velocity head at that point is equal to the same functions at any other point, plus (or minus) intervening losses. Using the notation already given and as shown in Fig. 1 and taking the bottom of the canal at B as datum, this relation may be expressed thus:

$$h_g + d_a + hv_a = d_b + hv_b + h_f \qquad (1)$$

There is no exception to this rule and it is independent of the form or slope of channel, or of channel changes occurring between the two points.

The second law requires that the momentum of a system of particles, considered collectively, cannot be altered by the particles impinging upon each other, but can only be changed by the influence of an external force. The change produced in momentum depends upon the magnitude of the external force and its duration. Stated simply, the law requires that force must equal rate of change of momentum or that force is equal to mass times acceleration. If the acceleration between two stations as A and B, Fig. 1, be uniform, the relation may be written

$$F = \frac{MV_b - MV_a}{t} \qquad (2)$$

where F equals force, MV_a and MV_b are the momentum at A and B respectively and t is the time through which F is applied. If F' is the force acting on a unit volume of water, of weight W, then from equation (2)

$$F' = \frac{W}{gt}(V_b - V_a) \qquad (3)$$

The total force acting on Q units per second for time t will be

$$F = F'Qt = \frac{QW}{g}(V_b - V_a) \qquad (4)$$

Fig. 2 shows graphically the relation existing between the energy of flow (depth plus velocity head), the momentum and the depth for a given discharge in a given channel. This diagram is not affected by the slope or roughness of the channel, and is independent of the method of producing or maintaining flow. The diagram is constructed to scale for 150 sec.-ft. flowing in the irregular channel shown in Fig. 3. The lower of the two curves is obtained by platting values of $d + hv$ over corresponding values of d, and may be called the energy curve, since it represents the energy of flow corresponding to various depths, the bottom of the canal being taken as datum. Values of V and hv may be found for any depth by computing or planimeter-

APPENDIX A--THE HYDRAULIC JUMP--Continued

ing the area. It will be observed that there is one point, C, on the curve for which the value of $d + hv$ is a minimum. The depth, d_c, corresponding to this point is called the critical depth. If flow is taking place at any depth, d_o, other than d_c, there will be a corresponding depth, d_g, having the same value of $d + hv$. The depths d_o and d_g will be called alternative energy depths or alternative energy stages.

Let Fig. 4 represent the profile of a portion of a channel of uniform cross-section having functions as represented in Fig. 2. Then from the energy curve, Fig. 2, it appears that the depth at S may be either d_o or d_g and that it may be made to change from one of these depths to the other at will, provided some means for making the change without depressing the energy line through the point C be supplied.

Such a change in depth with no loss of energy, however, involves a change in momentum and can only be

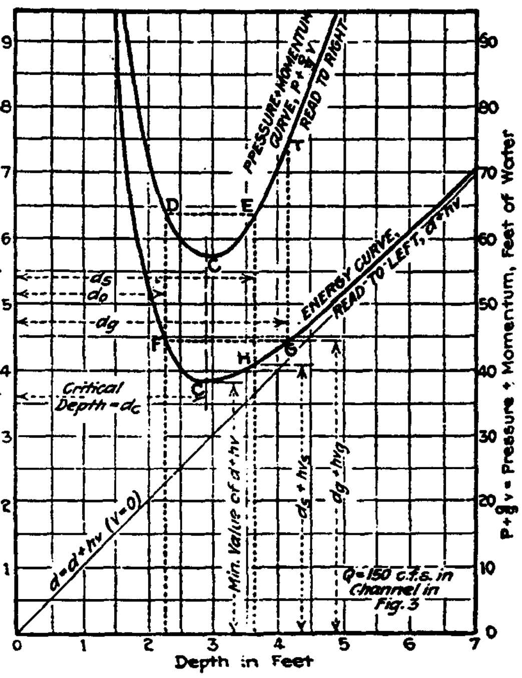


FIG. 2. RELATION BETWEEN ENERGY OF FLOW, MOMENTUM AND DISCHARGE

effected by the intervention of some external force. The required force may be supplied by gravity, friction, unbalanced pressures or by a combination of these factors.

In Fig. 1, let P_a be the hydrostatic pressure on the plane of the cross-section at A, P_b being the corresponding pressure at B, and let F'' equal friction, or any other external force applied between A and B. From (4), taking $W = 1$

$$\frac{Q}{g}(V_b - V_a) = P_a - P_b + F'' \qquad (5)$$

If the change in depth occur without the intervention of the force F'', then equation (5) placing $F'' = o$ and transposing gives the relation

$$\frac{Q}{g}V_a + P_a = \frac{Q}{g}V_b + P_b \qquad (6)$$

which must hold if a change in depth occurs under the influence of the external forces P_a and P_b only.

The momentum curve in Fig. 2 gives values $\frac{Q}{g}V + P$ for various values of d. The abscissæ are the same as used for the energy curve and values $\frac{Q}{g}V + P$ are shown on the right. For a given depth there is always one other depth having an equal value of $\frac{Q}{g}V + P$, this point falling in all cases beyond the critical depth. Therefore, for any depth of flow there is always another depth which we will call the alternative momentum stage, to which the flow may change without the intervention of an external force. Such a change, however, requires a change in the energy of flow.

Since for a given change in depth the change in $d + hv$ is not proportional to the change in $\frac{Q}{g}V + P$ it follows that a change in depth cannot occur without the introduction of some factor to preserve a balance. A change between alternative energy stages without loss may be effected by the application of an external force only, and a change between alternative momentum stages may be accomplished by a change in energy only. All other changes in depth, involving a change in velocity, require both an external force and a change in energy.

There are numerous causes which may produce a change in stage in a canal, but if the channel is straight, of uniform cross-section and roughness, and free from obstructions, changes in stage are generally caused by changes in grade. An analysis of a simple case will be made to show where changes may be expected. Let Fig. 5 be the profile of a canal of uniform cross-section. Let the slopes to the left of K and to the right of N be sufficient to maintain flow at normal depths, d_i and d_n, respectively, both greater than the critical depth, the slope between K and N being sufficient to maintain a normal depth, d_m, less than the critical. Let I be a sufficient distance upstream from K not to be affected by the "drawdown," and let M be sufficiently far below K for uniform flow to be established. Flow at I will be at high stage while at M it will be at low stage. Somewhere between it must pass through the critical depth.

Before proceeding to locate the point of passage it will be well to investigate the properties of the "energy gradient" shown on the profile. This gradient is determined by plotting the velocity head above the water surface at all points. At a given point the total energy, i.e., the sum of the static and kinetic energies, is represented by the elevation of this line. It follows from Bernoulli's theorem that the fall in this gradient between any two points represents the sum of all losses occurring between the two points. The slope of this line at any point represents the slope required to overcome friction and other losses at that point. The energy gradient can never rise in absolute elevation in the direction of flow, since there can be no increase in energy. The distance from any point on the energy gradient vertically downward to the bottom of the canal is $d + hv$. It will be clear from Fig. 2 that there will in all cases be two water depths corresponding to

APPENDIX A--THE HYDRAULIC JUMP--Continued

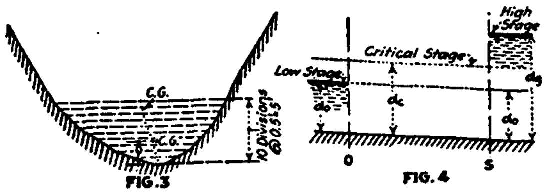


FIG. 3. SHOWING SUBDIVISIONS FOR COMPUTING AREAS AND PRESSURES. FIG. 4. ALTERNATE STAGES

any possible energy gradient. It is also evident that the energy gradient cannot be brought to within less than a certain minimum distance from the bottom of the canal, the two corresponding depths becoming equal at that point. The gradient will generally be a continuous line and can make an abrupt change in height or slope only where a sudden loss occurs. An example of sudden loss, caused by an abrupt change in stage, is shown at O, Fig. 5. So-called sudden changes in open channels are actually more or less extended, but for simplicity they are assumed in all computations to be instantaneous.

Returning to Fig. 5, the water surface must pass through the critical depth between I and K, at K or between K and M. Assume the passages to occur at some point J above K. The energy gradient must under this assumption drop down at J to a minimum height above the bottom of the channel. Therefore, the friction slope from J to K cannot be steeper than the slope of the base of the canal, and, since the slope of the canal is only sufficient to maintain flow at the normal depth, d_i, which is greater than the critical depth, the energy available is not sufficient to overcome friction losses from J to K and maintain flow at or below the critical depth. Hence the critical depth cannot exist at any point above K. If the passage occurs at some point, L, below K the water surface from K to L must be at or above the critical, the velocity will be less than normal, and therefore the friction slope will be flatter than the canal slope; that is, the value of $d + hv$ at L will be greater than at K, whereas if the critical depth occurred at L it should be less. Therefore, the passage cannot occur below K. If the point of passage is at K, $d + hv$ will increase from K to M and the fall in the gradient will be less than the fall in the canal grade. This is logical since the velocity is less than the normal velocity. The fall in the energy gradient from I to K will be greater than the fall in the canal grade, to balance the increase in friction due to velocities in excess of the normal velocities. The point of critical flow will, therefore, come at K.

No reference has been made to values of the slope from K to N except to state that it is sufficient to maintain flow at a depth less than the critical. As long as this slope is sufficient to support flow at a normal depth equal to or less than the critical depth, it may be varied at will without affecting flow conditions above K. For this reason K is called a control.

Flow to the right of M will be uniform at the depth d_m until the flatter slope at N is encountered. For simplicity the loss through shock due to the vertical angle at N is neglected. In actual construction this angle, if sharp, would be relieved by a vertical curve to reduce the shock. Flow to the right of N cannot continue at the depth d_m, since the slope of the canal is insufficient to overcome friction at that depth. The excess of the frictional resistance over the force due to the slope of the bottom of the channel produces a retarding force tending to reduce the velocity and momentum of flow and causing the depth to gradually increase, the velocity head and the depth plus velocity head being decreased to supply the energy necessary to overcome friction.

If this gradual rise in water surface be assumed to continue, along the dotted line YZ in the figure, until the normal depth is reached at Z, there will be some point, as at S, where the depth is equal to the critical depth. Since $d + hv$ is a minimum at S and is not a minimum at Z it follows that the available friction slope from S to Z must be less than the canal slope. But the velocity from S to Z is greater than the normal velocity, hence the required friction slope is greater

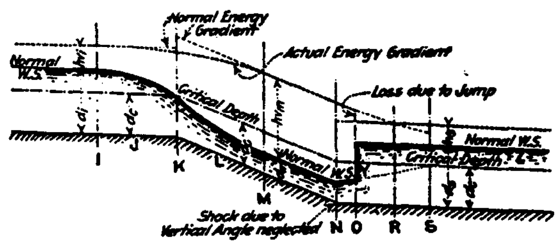


FIG. 5. CHANGES OF STAGE IN UNIFORM CHANNEL

than the canal slope, and the depth cannot change from d_c at S to d_s at Z. It is necessary that the water depth change, between N and Z, from d_m to d_s, but it must not at any point have the intermediate depth d_c nor in fact any depth for which $d + hv$ is less than at Z. The change occurs suddenly through what is known as the hydraulic jump, from some low-stage depth to the depth d_s. The depth d_s and the low-stage depth at R, where the energy gradient for the water surface YZ intersects the normal energy gradient, are alternate energy depths, similar to d_g and d_s in Fig. 2, and if the jump involved no loss of energy it would occur at that point. Referring to Fig. 2 it will be seen that in order for the change to occur at R there must be an increase in $P + \frac{Q}{g}V$, similar to the change from D to T. Such a change in momentum requires the application of an external force. The only external forces available aside from P_e and P_t are the forces of gravity acting through the canal slope and resistance due to friction. These tend to neutralize each other and are negligible in amount. Therefore, F'' in equation (5) may be assumed to be zero, equation (6) must hold, and the jump cannot occur at R, but must take place at some earlier stage where $P + \frac{Q}{g}V$ is equal to the final value of that function. This requirement apparently conflicts with Bernoulli's theorem, but there is automatically introduced a disturbance which produces an internal loss of proper magnitude to preserve the equilibrium of equation (1). The low-stage depth at the jump will be d_v, corresponding to the point D on the momentum curve in Fig. 2. The loss of energy in the jump is equal to $(d_g + hv_g) - (d_s + hv_s)$.

This loss is unavoidable for a change in stage in a channel of constant cross-section and falling grade.

APPENDIX A--THE HYDRAULIC JUMP--Continued

By properly adjusting the shape or vertical alignment of the channel additional external forces may be introduced, and the jump, with the attendant loss of head, may be reduced or eliminated, as will be pointed out later.

The two curves in Fig. 2 approach each other indefinitely to the left of C, while to the right of C they diverge rapidly. A little study of these curves will show that for small heights of jump the loss approaches zero, but that the loss of head increases rapidly as the height of jump increases. For example, in Fig. 2, if the jump occurs from $d = 2.5$ to $d = 3.38$ the loss in depth will be from $d = 3.47$ to $d = 3.38$ or 0.09 ft. The corresponding energy loss is about 0.05 ft. If the jump occurs from $d = 2$ ft. to $d = 4$ ft., the loss in depth will be from 5 ft. to 4 ft. or 1 ft., the energy loss being $5.17 - 4.33 = 0.84$ ft. If the jump occurs from $d = 1.75$ ft. to $d = 4.55$ ft. the loss in depth will be $6.30 - 4.55 = 1.75$ ft. and the energy loss will be $6.40 - 4.80 = 1.60$. Taking the discharge as 150 sec.-ft. this last loss requires the continuous destruction of energy equivalent to about 27 horsepower.

After the depth, d_n, from which the jump will take place has been determined the location of the jump may be obtained by finding the point at which flow will be retarded to that depth. This point may be conveniently found from the following equation, derived from Fig. 1:

$$Ls_o + d_a + hv_a = d_b + hv_b + Ls_f \qquad (7)$$

By assuming values for d_a and d_b all functions at A and B can be computed, including the friction slopes. If the canal slope, s_o, be known and if the friction slope from A to B, be assumed equal to the average of the slopes at these points, all factors in equation (7), except L, become known and L is readily found.

Equation (7) is applicable to any variable flow. The only approximation involved is in the assumption that the average slope is equal to the average of slopes at the computed points. By assuming depths sufficiently close together the error from this source can be reduced as far as desired. However, ordinary friction formulæ are not known to apply accurately to variable flow and extreme refinement in computation is not justified. The friction slope may be determined by Kutter's formula, or by any other friction formula. This equation should not be applied through a control section or a hydraulic jump, but may be used to find the water surface at J, L or N, Fig. 5, or above a check or dam.

The changes in canal slope at points K and N, Fig. 5, are purposely assumed to be great, so that the changes in stage will be marked, but when the normal depth is near the critical, troublesome fluctuations are often produced by very slight unintentional irregularities in the channel. The fluctuations in such cases appear to be out of all proportion to the offending irregularities. This is due to the fact that in the vicinity of C, Fig. 2, the momentum and energy curves are approximately horizontal so that if the amount of energy $(d + hv)$ required at a given point is changed slightly, a comparatively great change in depth must occur to preserve a balance. The possibility of trouble from this source is discussed by J. S. Longwell in an article on "Flow Conditions, Congo Low Line Flume, North Platte Project," published in the *Reclamation Record*, August, 1917, and reprinted in *Engineering News-Record*, Jan. 3, 1918, p. 38. If in the design of a channel it is found that the depth is at or near the critical the shape or slope of the channel should, if practicable, be changed to secure greater stability. Usually the critical velocity can be changed by widening or narrowing the channel, or the normal velocity by altering the slope. If such changes are not practicable, liberal freeboard should be allowed, and extreme care should be used in construction to secure uniformity in grade and cross-section.

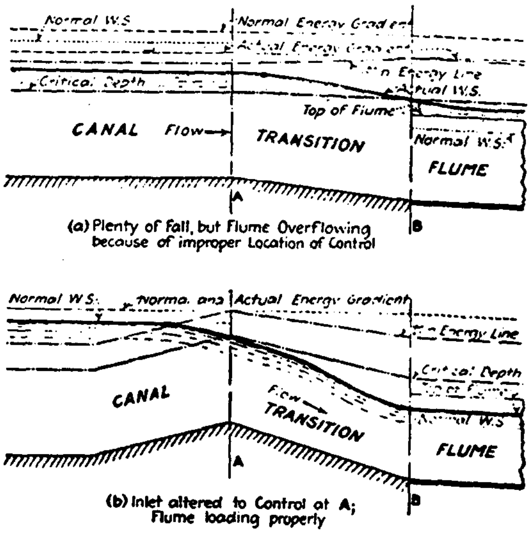


FIG. 6. VARIABLE CROSS-SECTIONS CANAL TO FLUME FOR TRANSITION FROM HIGH TO LOW STAGE

Changes in stage which occur at transitions between canals and flumes, tunnels or other high-velocity conduits, where the cross-section of the channel is variable, involve only the principles already discussed, but additional factors are introduced which affect the mathematical treatment. It will be convenient to consider these transitions under six headings, as determined by the stages between which changes occur, as follows:

(a) Changing from high stage to low stage, increasing velocity.

(b) Changing from one low stage to another, increasing velocity.

(c) Changing from one high stage to another, increasing velocity.

(d) Changing from low stage to high stage, reducing velocity.

(e) Changing from one low stage to another, reducing velocity.

(f) Changing from one high stage to another, reducing velocity.

Case (a) is similar to the example presented in the discussion of Fig. 5, and by arguments already used it can be shown that the control section cannot be above A or below B, in Fig. 6, AB being a variable transition between a canal and a flume. To locate the point of control plot a minimum energy line, as shown in (a) and (b), Fig. 6. This line is obtained by plotting the minimum values of $d + hv$ above the canal bottom, and it represents the minimum possible elevation of the energy gradient at any point. The actual energy gradient cannot fall below this line and if the two gradients intersect it must be at the highest point on

APPENDIX A--THE HYDRAULIC JUMP--Continued

the minimum line. Hence, in Fig. 6 (a), although there is apparently sufficient drop from the normal water surface in the canal to that in the flume, the flume will overflow because of the incorrect location of the control. The drop which should produce velocity head is used up in friction through the lowering of the water surface in the canal. By humping the bottom, as in (b), to bring the minimum energy line at the upper end of the transition into coincidence with the normal energy gradient, the trouble is avoided. Racing in the canal is prevented by the throttling effect of the control at A. The same effect can be secured, if desired, by narrowing the section at A, rather than by raising the grade. The inlet structure may be made to act as an automatic check by shaping it so that the head required to pass any quantity of water at the critical depth is equal to the normal head in the canal above for that quantity. It is theoretically possible to construct such a control check so that it will exactly control all quantities of flow in a given channel, but it is sufficient, for all practical purposes, to design the structure to fit exactly for two discharges, usually full discharge and one-fourth discharge, as in the case of a notched drop.

In changing from one low stage to another, having a greater velocity (case b) it is possible, by contracting the channel or by raising the bottom, or both, to force the water surface up to the critical depth and under extreme conditions a jump may be produced within the transition or in the canal above. Such a contingency is, however, remote and ordinarily this type of transition will not be effected by the critical depth or the hydraulic jump.

The transition from one high stage to another, having a higher velocity (case c) is often accompanied by disturbances attributable to an incorrect control. Fig.

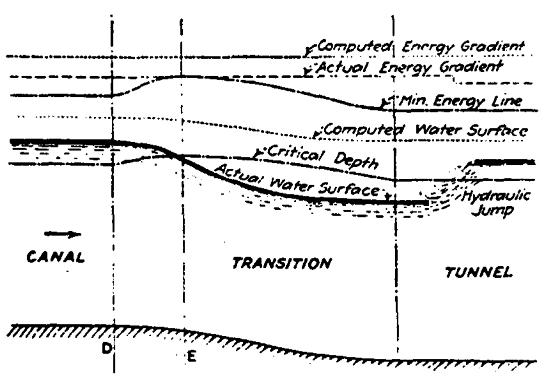


FIG. 7. FAULTY INTAKE: ONE HIGH STAGE TO ANOTHER

7 represents to an exaggerated scale a faulty design recently prepared by the writer for a transition from a segmental open channel to a circular tunnel. The hydraulics for this transition were computed at 2-ft. intervals and no discrepancies were found. However, liberal allowance was made for transitions and friction losses, and a "safe" coefficient of roughness was used to determine the depth in the tunnel. After construction it was found that transition losses were negligible and that the normal depth in the tunnel immediately below the entrance was considerably less than the assumed normal depth. As a result the energy gradient

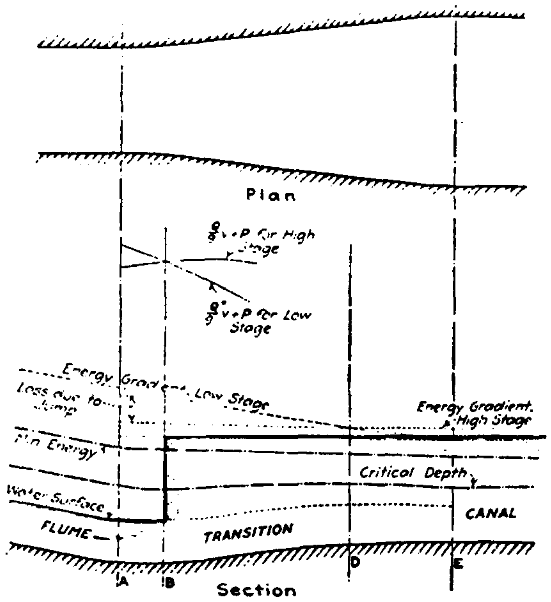


FIG. 8. JUMP WITH VARYING CHANNEL SECTION AND BOTTOM SLOPE

for the tunnel dropped below the summit of the minimum energy line, and the flow passed to low stage at E, causing a jump to occur just below the end of the transition. The transition should have been proportioned to keep the summit of the minimum energy line below the lowest possible position of the energy gradient at F. The jump was particularly objectionable at this location, and was eliminated by bolting cross timbers to the bottom of the channel, thus increasing the friction and bringing the energy gradient up to its computed position.

Transition from low stage to high stage, (case d) may be accomplished either with or without the hydraulic jump. Unless the section of the channel is properly varied or the bottom "humped" the jump is inevitable. Fig. 8 represents a transition in which the variation in channel section is not sufficient to avoid the jump. The energy gradient for low stage is computed from A toward E, using equation (1), the gradient for high stage being computed backwards, from E toward A, in the same way. After these gradients and their corresponding depths are found values of $\frac{Q}{g}V + P$ for the two stages are computed, and plotted to any convenient scale and datum. The jump must occur where this function is equal for the two depths, or at B, the intersection of the plotted lines. By varying the cross-section or the elevations of the flume, transition or canal, the location of the intersection, B, may be varied at will. If this point falls to the left of A the jump will occur in the flume and may cause it to overflow. If it falls to the right of E the jump will occur in the canal section where the resulting disturbances may be objectionable.

If the transition in Fig. 8 be so altered that the minimum energy line at D becomes tangent to the two energy gradients at their point of intersection, the two gradients automatically changing to become tangent to

APPENDIX A--THE HYDRAULIC JUMP--Continued

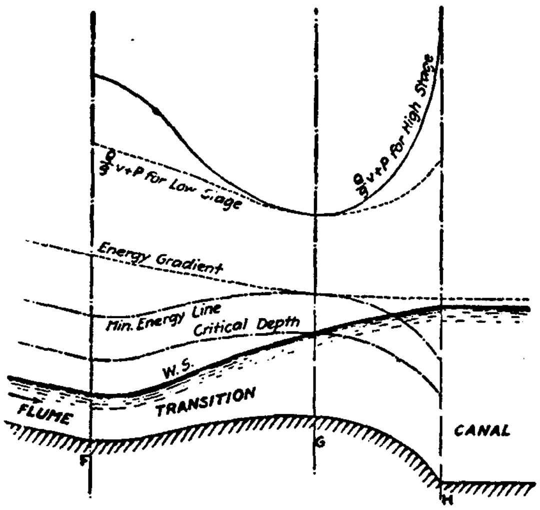


FIG. 9. TRANSITION WITHOUT JUMP

each other at that point, the transition may be accomplished without the jump. Such a transition is illustrated in Fig. 9. The $\frac{Q}{g}V + P$ lines intersect and become tangent at the point G. The excess of the pressure in an upstream direction over that in a downstream direction on the hump in the bottom or on contractions in the sides of the channel supplies the force F'' required in equation (5).

In changing from one low stage to another with a lower velocity (case e) the hydraulic jump and the control section are not often encountered. A contraction in the sides of the channel or a hump or obstruction in the bottom may cause the water surface to rise temporarily above the critical depth, but such contraction or obstruction is not likely to exist in an artificial channel, except by deliberate design. The low secondary dam sometimes placed below an overflow dam to break up the high velocity constitutes such an obstruction, but the normal depth below the secondary dam is usually above the critical so that the conditions of (case d) obtain.

The most usual form of canal transition for reducing velocity is from one high stage to another and such structures are often subject to unexpected irregularities. It is usual in designing transitions of this type to provide for only a partial recovery of head, to allow a factor of safety to take care of imperfections in the structure and of fouling in the canal below. This results in an excess of energy. If the minimum energy line is at all points well below the energy gradient the water surface in the high-velocity channel will be lowered and the excess head will be consumed in increased friction, but if the minimum energy line is high a control is likely to be formed, resulting in low-stage flow for a short distance, followed by a jump back to normal. An actual instance of such an outlet is shown in Fig. 10, where a very gradual change from a 6.1-ft. diameter tunnel to a 8.3-ft. segmental lined section is effected in a length of 100 ft. If the critical depth in the tunnel and in the transition had been lower the water surface would have been further drawn down at J to make the energy gradient from above coincide with that from below, but the high position of the minimum-energy line at J limits the draw down. As a result the flow passes to low stage at J, returning to high stage through the hydraulic jump at K. The location and height of the jump may be determined as in case (d) Fig. 10 is plotted from actual observations. If the required water surface elevation at L were to be increased to bring the energy gradient at that point above the elevation of the minimum-energy line at J, the control at J and the jump at K would be avoided. It will be noticed that the velocities increase from J toward K, reaching a maximum somewhere near K, the effective length of transition being reduced to KL. Under proper conditions the point K may fall to the right of L, and in any event the turbulence below L will be greater than if the transition were effected without the jump. If turbulence is objectionable the outlet should be proportioned to avoid the formation of a control.

The critical depth may be found by constructing either the energy or momentum curve, and finding its lowest point, as in Fig. 2, but it can be more readily determined by means of the equation

$$\frac{A^3}{T} = \frac{Q^2}{g} \qquad (8)$$

where A and T are respectively the area and top width of the water sections for the critical depth. Using

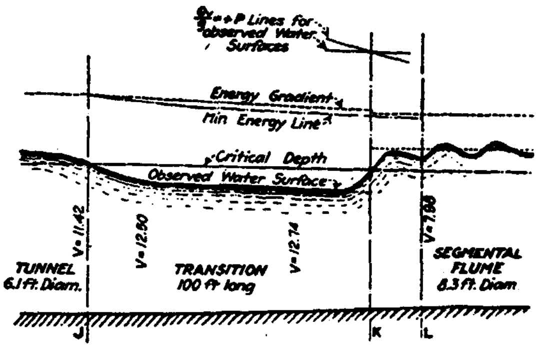


FIG. 10. TRANSITION FROM ONE HIGH STAGE TO ANOTHER: JUMP CAUSED BY EXCESS HEAD

the notation already established, and letting $H = d + hv$, and A = area of section = some function $f(d)$ of the depth, equation (8) is derived as follows:

$$H = d + hv = d + \frac{V^2}{2g}$$

$$= d + \frac{1}{A^2} \times \frac{Q^2}{2g}$$

$$= d + \frac{1}{f^2(d)} \times \frac{Q^2}{2g};$$

differentiating, $\frac{dH}{dd} = 1 - \frac{f'(d)}{f^3(d)} \times \frac{Q^2}{g}$,

where $f^3(d) = A^3$ and $f'(d)$ = the first derivative of A with respect to d, $= T$, the width at the water surface. H is a minimum when

$\frac{dH}{dd}$ is zero, or when $\frac{A^3}{T} = \frac{Q^2}{g}$ as given above.

By substituting AV for Q this equation may be written

$$\frac{A}{2T} = \frac{V^2}{2g} = hv \qquad (9)$$

APPENDIX A--THE HYDRAULIC JUMP--Continued

In a rectangular section of width, b, A is equal to bd, T is equal to b, and (8) may be reduced to

$$\frac{Q^2}{g} = b^2 d_c^3 \text{(for rectangular sections)} \qquad (10)$$

from which d_c is readily determined. Equation (9) reduces to the well-known form

$$hv_c = \frac{1}{2} d_c \text{ (for rectangular sections)} \qquad (11)$$

In a triangular section, where the ratio of T to d is X, equation (8) reduces to

$$\frac{Q^2}{g} = \frac{d^5 X^2}{8} \text{(for triangular section)} \qquad (12)$$

and (11) becomes

$$hv_c = \frac{1}{4} d_c \text{ (for triangular section)} \qquad (13)$$

Similar formulæ may be deduced for any channel having a known mathematical relation betwen A, t and d, but generally the resulting equations are of the $\frac{5}{2}$ deg., and are complicated, and it is preferable to use equations (8) and (9) without further reduction. If d_c is known and Q required these equations may be solved directly, but if d_c is the unknown the solution can best be made by trial.

Valuable assistance in the preparation of this paper has been rendered by D. C. McConaughy and W. H. Nalder and other engineers in the Denver Office of the Reclamation Service.

See Figures 18 and 19 (Paragraph 2.29) for Energy Loss and Relations between Variables in a Hydraulic Jump.

Given:

h	f_s	M_1
b	f_c	T
d_1	n	
d_2		

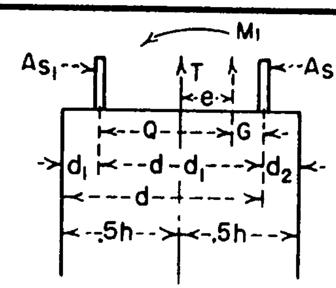
Compute:

$e = \frac{M_1}{T}$

$d = h - d_2$

$k = \frac{1}{1 + f_s \div (n f_c)}$

$j = 1 - \frac{K}{3}$



CASE I $e \leq .5h - d_2$

TENSION OVER ENTIRE SECTION

$Q = .5h - d_1 + e$

$G = .5h - d_2 - e$

$As_1 = \frac{T}{f_s} \times \frac{G}{d - d_1}$

$As_2 = \frac{T}{f_s} \times \frac{Q}{d - d_1}$

CASE II $e > .5h - d_2$

$M_2 = M_1 - T(d - .5h)$; $M_3(\text{allowable}) = .5 f_c\, kjbd^2$; $M_4 = M_2 - M_3$

COMPRESSIVE REINF. NOT REQ'D.

CASE II-A $M_2 \leq M_3$

$As_a = \frac{M_2}{f_s jd}$

$As_b = \frac{T}{f_s}$

$As_1 = 0$

$As_2 = As_a + As_b$

COMPRESSIVE REINF. REQ'D.

CASE II-B $M_2 > M_3$

$As_a = \frac{M_3}{f_s jd}$

$As_b = \frac{M_4}{f_s(d - d_1)}$

$As_c = \frac{T}{f_s}$

$As_1 = \frac{M_4}{f_s} \times \frac{n}{2n - 1} \times \frac{d(1 - k)}{(kd - d_1)(d - d_1)}$

$As_2 = As_a + As_b + As_c$

NOTE: Both M_1 and T are considered positive.
All values to be in inch-lb. units.

REINFORCED CONCRETE DESIGN COMBINED BENDING AND TENSION

DEC. 26, 1946-A.L.A.-E.R.H.-D.K.M. REV. 8-9-67

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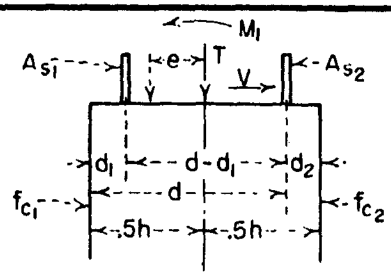
REINFORCED CONCRETE DESIGN--COMBINED BENDING AND COMPRESSION

Given:

h	f_s	M_1
b	f_c	T
d_1	n	V
d_2	v	
	u	

Compute:

$$e = \frac{M_1}{T}$$
$$d = h - d_2$$
$$k = \frac{1}{1 + f_s \div (n f_c)}$$
$$j = 1 - \frac{k}{3}$$



CASE I_a $0 \leq e \leq \frac{h}{6}$

COMPRESSION OVER ENTIRE SECTION AND NO COMPRESSIVE REINF. PRESENT

$$f_{c_1} = \frac{T}{bh}\left(1 + \frac{6e}{h}\right)$$
$$f_{c_2} = \frac{T}{bh}\left(1 - \frac{6e}{h}\right)$$

CASE I_b $\frac{h}{6} \leq e \leq \left(\frac{h}{2} - \frac{kd}{3}\right)$

COMPRESSION OVER PART OF SECTION AND NO TENSILE REINF. REQ'D. NO COMPRESSIVE REINF. PRESENT.

$$f_{c_1} = \frac{T}{bkd}\left(1 + \frac{6e'}{kd}\right)$$
$$e' = \frac{kd}{2} - \frac{h}{2} + e$$

CASE II $e > \left(.5h - \frac{kd}{3}\right)$

$$M_2 = M_1 + T(d - .5h);\quad M_3 = .5 f_c\, k j b d^2;\quad M_4 = M_2 - M_3$$

COMPRESSIVE REINF. NOT REQ'D.

CASE II-A $M_2 \leq M_3$

$$A_{s_a} = \frac{M_2}{f_s j d}$$
$$A_{s_b} = \frac{T}{f_s}$$
$$A_{s_1} = 0$$
$$A_{s_2} = A_{s_a} - A_{s_b}$$
$$f_{c_1} = \frac{M_2}{bd^2} \times \frac{2}{kj}$$
$$v = \frac{V}{bjd}$$
$$u = \frac{V}{\Sigma_o j d}$$

COMPRESSIVE REINF. REQ'D.

CASE II-B $M_2 > M_3$

$$A_{s_a} = \frac{M_3}{f_s j d}$$
$$A_{s_b} = \frac{M_4}{f_s (d - d_1)}$$
$$A_{s_c} = \frac{T}{f_s}$$
$$A_{s_1} = \frac{M_4}{f_s} \times \frac{n}{2n-1} \times \frac{d(1-k)}{(kd - d_1)(d - d_1)}$$
$$A_{s_2} = A_{s_a} + A_{s_b} - A_{s_c}$$

NOTE: Both M_1 and T are considered positive. All values to be in inch-lb. units.

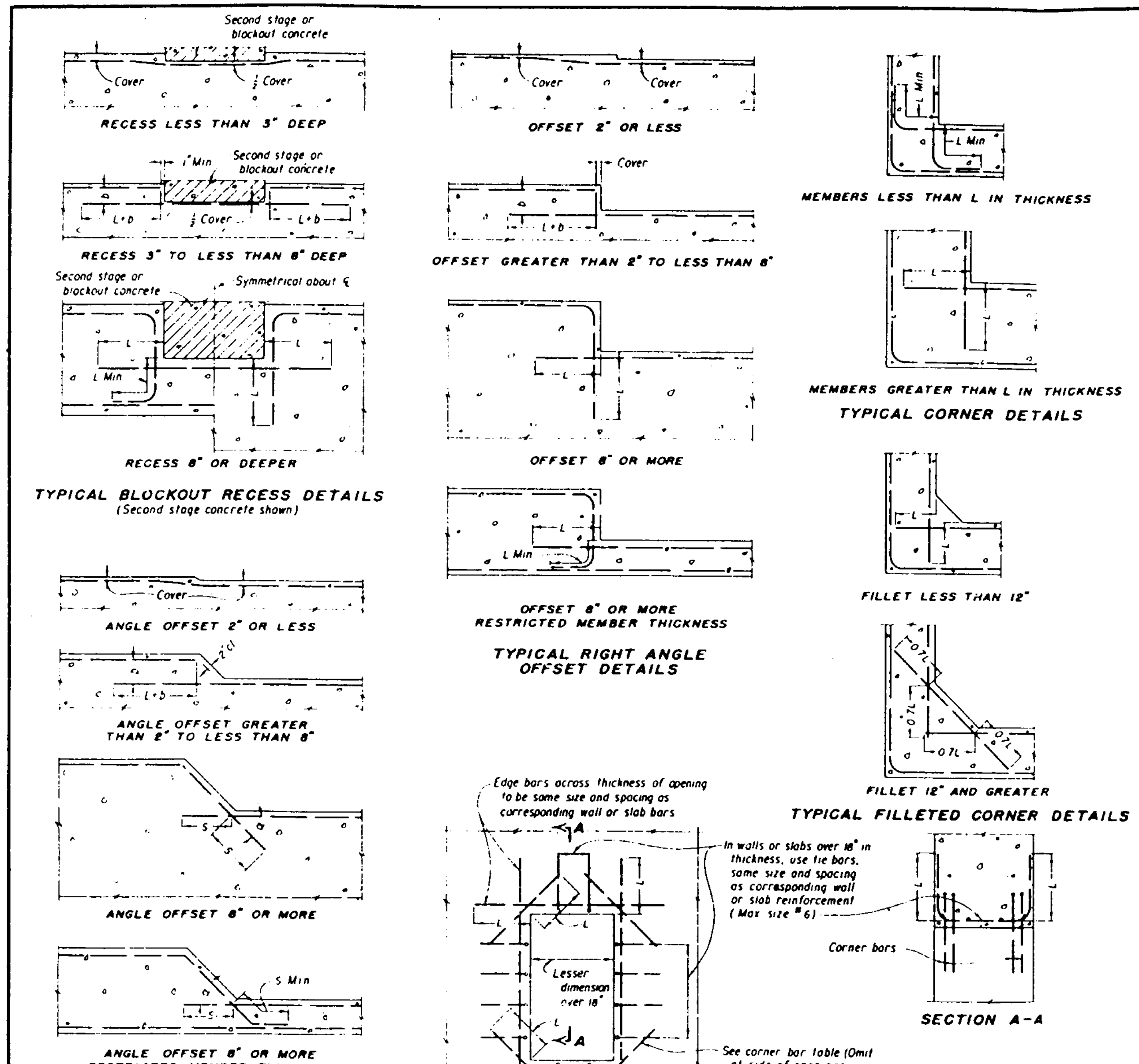
REINFORCED CONCRETE DESIGN COMBINED BENDING AND COMPRESSION

DEC. 26, 1946 - A.L.A.-E.R.H.-D.K.M. REV. 8-9-67

103-D-343

GENERAL NOTES AND MINIMUM REQUIREMENTS
FOR DETAILING REINFORCEMENT--CLASS 40 AND CLASS 50

Fig. 3 Par. 2.6



CORNER BARS	
MEMBER THICKNESS	EACH CORNER (INCLUDES BOTH FACES)
8" or less	1-#5
9" Thru 12"	2-#6
13" Thru 18"	2-#8
19" Thru 24"	4-#8
25" Thru 36"	4-#9
37" Thru 48"	4-#10
Over 48"	4-#11

NOTES
Ties, edge, and corner bars not required for openings 18" or less. For a recess deeper than 4" but not greater than one half wall or slab thickness, use corner bars in face on recess side

MINIMUM ADDITIONAL REINFORCEMENT AROUND OPENINGS IN WALLS AND SLABS

GENERAL NOTES

UNLESS OTHERWISE SHOWN ON THE REINFORCEMENT DESIGN DRAWINGS, THE DETAILS AND NOTES SHOWN ARE TYPICAL FOR ALL REINFORCEMENT DRAWINGS THAT REFER TO THIS DRAWING.

DESIGN:
Structural design is based on 3000 psi specified compressive concrete strength at 28 days with a working stress of 1,350 psi, and working stresses of 20,000 psi tension in reinforcement with a specified minimum yield strength of 40,000 psi

ABBREVIATIONS:

bf = bottom face	br = bottom row	bl = bottom layer
tf = top face	tr = top row	tl = top layer
nf = near face	nr = near row	ns = near side
ff = far face	fr = far row	fs = far side
ef = each face	er = each row	es = each side
if = inside face	ir = inside row	ew = each way
of = outside face	or = outside row	ec = each corner
	mr = middle row	ml = middle layer

spc = space or spaces
eq spc = equally spaced, equal spaces
D = nominal diameter of reinforcing bar
uv = uniformly varying lengths of bars between lengths shown
cl = clear

SYMBOLS:
Bars shown thus #8@12 or #6@7½, indicate a group of identical bars equally spaced
An open circle at the end of a bar indicates a bend with the bar turned away from the observer
A closed circle at the end of a bar indicates a bend with the bar turned towards the observer
Splices shown thus indicate a lapped splice, not a bend in the bar

DIMENSIONS:
Dimensions are to the center lines of the bars unless otherwise shown. Clear cover dimensions are marked "cl"

COVER:
Place the reinforcement so that the clear distance between face of concrete and nearest reinforcement is 1½" for #5 bars and smaller and 2" for #6 bars and larger; except provide a clear distance from face of concrete placed against earth or rock of 2" where member thickness is 9" or less and 3" where member thickness is greater than 9", the clear distance being to the design dimension line
Reinforcement paralleling construction joints shall have a minimum of 2" clear cover

BENT BARS:
Only billet steel shall be used for bars bent to a radius of 50-bar diameters or less
Unless larger radius bends are indicated on the design drawings, all reinforcement requiring bending shall be bent around a pin having the following diameter

FOR CLASS B40

BAR NO.	3	4	5	6	7	8	9	10	11	14S	18S
Standard Bends Diameter of pin (in)	$1\frac{7}{8}$	$2\frac{1}{2}$	$3\frac{1}{8}$	$5\frac{1}{4}$	$6\frac{1}{8}$	7	9	10	11	17*	$22\frac{1}{2}$*
Stirrup and Tie bends Diameter of pin (in)	$1\frac{1}{2}$	2	$2\frac{1}{2}$	$4\frac{1}{2}$	$6\frac{1}{8}$	7	9	10	11		

FOR CLASS B50

BAR NO.	3	4	5	6	7	8	9	10	11	14S	18S
Standard Bends Diameter of pin (in)	$2\frac{1}{4}$	3	$3\frac{3}{4}$	6	7	8	$10\frac{1}{8}$	$11\frac{1}{4}$	$12\frac{3}{8}$	17*	$22\frac{1}{2}$*
Stirrup and Tie bends Diameter of pin (in)	$1\frac{7}{8}$	$2\frac{1}{2}$	$3\frac{1}{8}$	$5\frac{1}{4}$	7	8	$10\frac{1}{8}$	$11\frac{1}{4}$	$12\frac{3}{8}$		

* Special fabrication for bends exceeding 90° and for all grades of steel with yield point of 50,000 psi

REINFORCEMENT DOWELS:
Dowels may be indicated on the drawing thus #8 (d)
Dowels shall have a minimum embedment and projection equal to that required for lap splicing a bar of the same diameter

PLAIN DOWELS:
Plain dowels across contraction joints shall be plain bars painted with one coat of red lead paint. Twist dowels to break bond before final set of first concrete placement. Immediately before the second concrete placement, the projecting half length of dowel shall be uniformly coated with a film of oil. Viscosity of the oil shall have a SAE rating of not less than 250

STANDARD HOOKS:
Hooks shall have 180° bends and extensions of 4-bar diameters but not less than 2½" parallel to the main leg of the bar or 90° bends and extensions of at least 12-bar diameters. Hooks for stirrup and tie anchorage only shall have either a 90° or 135° bend plus an extension of at least 6-bar diameters but not less than 2½" at the free end of the bar. Radius of bend to be as specified in table for bent bars.
A bar ( ) with a standard 180° hook on one end is referred to as an "A" bar
A bar ( ) with a standard 180° hook on both ends is referred to as a "B" bar

PLACING:
Bars at small openings (max. 18") in wall and slabs may be spread apart not more than 1½ times the bar spacing.
Reinforcement may be adjusted in the field to clear waterstops, keys, form ties, anchor bars, seals, recesses, embedded metalwork, and conduits except in heavily reinforced areas. In such cases relocation of the embedded materials must be considered. In no case should bars be bent to greater than 6 to 1 slope.
Where possible, reinforcement shall be placed to maintain a clear distance of at least one inch between the reinforcement and anchor bolts, form ties, or other embedded metalwork
Reinforcement parallel to anchor bolts or other embedded metalwork shall be placed to maintain a clear distance of at least 1⅓ times the maximum size aggregate.
Reinforcement not to be placed between or within 4 inches of main power leads

SPACING:
The first and last bars in slabs and walls, stirrups in beams, and ties in columns are to start and end at a maximum of one half of the adjacent bar spacing.

SPLICES:
Unless otherwise shown the minimum length of lap for splicing parallel bars shall be as given in Table A.
When reinforcing bars of different size are to be spliced, the lap length shall be governed by the smaller diameter bar
Splices are to be made so that the given distances to face of concrete will be maintained
The minimum lap requirement "S" for non-parallel bars shall be as given in Table B.

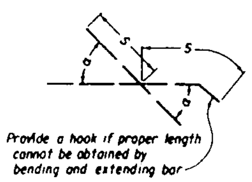
TABLE A
LENGTH OF LAPPED SPLICE "L", IN INCHES*

BAR NO.	L	BAR NO.	L
#2, #3, #4	12	#8	25
#5	15	#9	32
#6	18	#10	41
#7	21	#11	51

* Stagger splices if bars are spaced closer than a clear distance of 8-bar diameters or 8"

TABLE B
LENGTH OF SPLICE "S", FOR NON-PARALLEL BARS

ANGLE α	S
0° To 10°	0.5L
10° To 30°	0.6L
30° To 50°	0.7L
50° To 65°	0.8L
65° To 80°	0.9L
80° To 90°	1.0L



ACCESSORIES:
Bar supports, spacers and other accessories are not shown on the design drawings. The recommendations of ACI 315-65 or other approved supporting systems may be used

REFERENCES:
Numerals in parentheses ( ) following notes and section letters or numbers indicate the number of the drawing upon which the section or detail is shown; for example (524) denotes Drawing No. 557-D-524.
Unless otherwise shown follow the recommendations established by American Concrete Institute's "Manual of Standard Practice for Detailing Reinforced Concrete Structures" (ACI 315-65).

7-3-67 D- C.A.S.	RETRACED - PIN DIAMETERS REVISED
6-5-67 D- C.A.S.	MINOR CHANGES, STEEL CLASS ADDED

ALWAYS THINK SAFETY

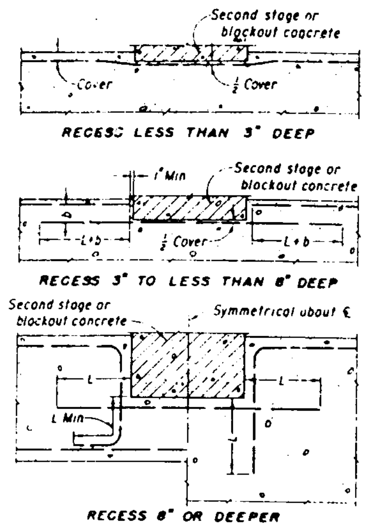
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS

GENERAL NOTES
AND MINIMUM REQUIREMENTS
FOR DETAILING REINFORCEMENT
CLASS 40 AND CLASS 50

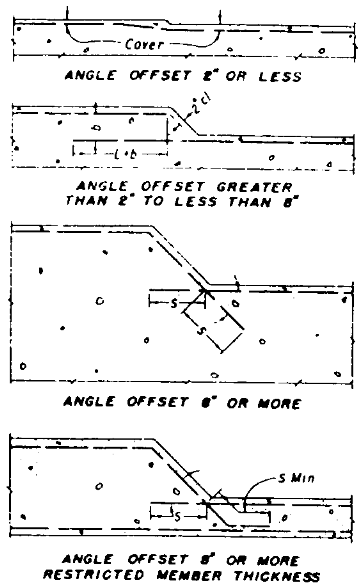
DRAWN C.P.M.-L.M.C. SUBMITTED Wm. H. Wolf
TRACED W.E. RECOMMENDED H. G. Arthur
CHECKED F.B.B.-F.P.M.-S.A.B. APPROVED B. P. Bellport, CHIEF ENGINEER
DENVER, COLORADO, MARCH 26, 1965 — 40-D-7004

GENERAL NOTES AND MINIMUM REQUIREMENTS FOR DETAILING REINFORCEMENT--CLASS 60

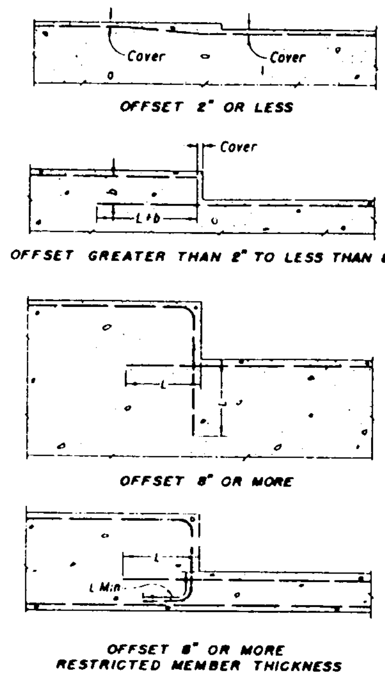
Fig. 4 Par. 2.6



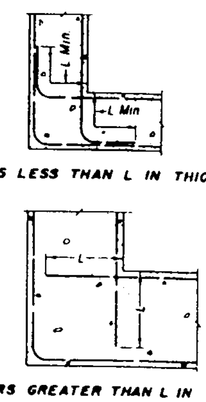
TYPICAL BLOCKOUT RECESS DETAILS
(Second stage concrete shown)



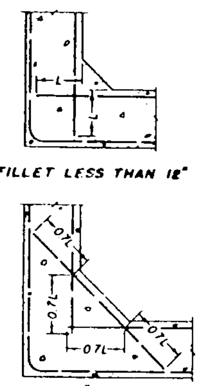
TYPICAL ANGLE OFFSET DETAILS



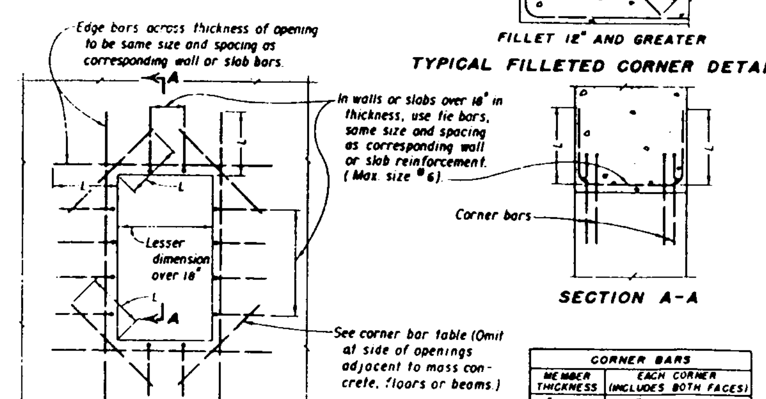
TYPICAL RIGHT ANGLE OFFSET DETAILS



TYPICAL CORNER DETAILS



TYPICAL FILLETED CORNER DETAILS



CORNER BARS	
MEMBER THICKNESS	EACH CORNER (INCLUDES BOTH FACES)
8" or less	1-#5
9" Thru 12"	2-#6
13" Thru 18"	2-#8
19" Thru 24"	4-#8
25" Thru 36"	4-#9
37" Thru 48"	4-#10
Over 48"	4-#11

NOTES

Ties, edge, and corner bars not required for openings 18" or less.
For a recess deeper than 4" but not greater than one half wall or slab thickness, use corner bars in face on recess side

MINIMUM ADDITIONAL REINFORCEMENT AROUND OPENINGS IN WALLS AND SLABS

GENERAL NOTES

UNLESS OTHERWISE SHOWN ON THE REINFORCEMENT DESIGN DRAWINGS, THE DETAILS AND NOTES SHOWN ARE TYPICAL FOR ALL REINFORCEMENT DRAWINGS THAT REFER TO THIS DRAWING.

DESIGN:
Structural design is based on 3750 psi specified compressive concrete strength at 28 days with a working stress of 1,700 psi; and working stresses of 24,000 psi tension in reinforcement with a specified minimum yield strength of 60,000 psi.

ABBREVIATIONS:

bf = bottom face	br = bottom row	bl = bottom layer
tf = top face	tr = top row	tl = top layer
nf = near face	nr = near row	ns = near side
ff = far face	fr = far row	fs = far side
ef = each face	er = each row	es = each side
if = inside face	ir = inside row	ew = each way
of = outside face	or = outside row	ec = each corner
	mr = middle row	ml = middle layer

spc. = space or spaces
eq spc = equally spaced, equal spaces
D = nominal diameter of reinforcing bar
uv = uniformly varying lengths of bars between lengths shown
cl = clear

SYMBOLS:
Bars shown thus #8@12 or #6@7½, indicate a group of identical bars equally spaced.
An open circle at the end of a bar indicates a bend with the bar turned away from the observer.
A closed circle at the end of a bar indicates a bend with the bar turned towards the observer.
Splices shown thus indicate a lapped splice, not a bend in the bar.

DIMENSIONS:
Dimensions are to the center lines of the bars unless otherwise shown. Clear cover dimensions are marked "cl"

COVER:
Place the reinforcement so that the clear distance between face of concrete and nearest reinforcement is 1½" for #5 bars and smaller and 2" for #6 bars and larger; except provide a clear distance from face of concrete placed against earth or rock of 2" where member thickness is 9" or less and 3" where member thickness is greater than 9", the clear distance being to the design dimension line.
Reinforcement paralleling construction joints shall have a minimum of 2" clear cover.

BENT BARS:
Only billet steel shall be used for bars bent to a radius of 50-bar diameters or less.
Unless larger radius bends are indicated on the design drawings, all reinforcement requiring bending shall be bent around a pin having the following diameter.

BAR NO.	3	4	5	6	7	8	9	10	11	14S	18S
Standard Bends Diameter of pin (in.)	2¼	3	3¾	6	7	8	10⅛	11¼	12⅝	17*	22⅝*
Stirrup and Tie bends Diameter of pin (in.)	1⅞	2½	3⅛	5¼	7	8	10⅛	11¼	12⅝		

*Special fabrication for all bends.

REINFORCEMENT DOWELS:
Dowels may be indicated on the drawing thus #8 (d).
Dowels shall have a minimum embedment and projection equal to that required for lap splicing a bar of the same diameter

PLAIN DOWELS:
Plain dowels across contraction joints shall be plain bars painted with one coat of red lead paint. Twist dowels to break bond before final set of first concrete placement. Immediately before the second concrete placement, the projecting half length of dowel shall be uniformly coated with a film of oil. Viscosity of the oil shall have a SAE rating of not less than 250.

STANDARD HOOKS:
Hooks shall have 180° bends and extensions of 4-bar diameters but not less than 2½" parallel to the main leg of the bar or 90° bends and extensions of at least 12-bar diameters. Hooks for stirrup and tie anchorage only shall have either a 90° or 135° bend plus an extension of at least 6-bar diameters but not less than 2½" at the free end of the bar. Radius of bend to be as specified in table for bent bars.
A bar (___⊃) with a standard 180° hook on one end is referred to as an "A" bar.
A bar (⊂__⊃) with a standard 180° hook on both ends is referred to as a "B" bar.

PLACING:
Bars at small openings (max 18") in wall and slabs may be spread apart not more than 1½ times the bar spacing.
Reinforcement may be adjusted in the field to clear waterstops, keys, form ties, anchor bars, seals, recesses, embedded metalwork, and conduits except in heavily reinforced areas. In such cases relocation of the embedded materials must be considered. In no case should bars be bent to greater than 6 to 1 slope.
Where possible, reinforcement shall be placed to maintain a clear distance of at least one inch between the reinforcement and anchor bolts, form ties, or other embedded metalwork.
Reinforcement parallel to anchor bolts or other embedded metalwork shall be placed to maintain a clear distance of at least 1⅓ times the maximum size aggregate.
Reinforcement not to be placed between or within 4 inches of main power leads.

SPACING:
The first and last bars in slabs and walls, stirrups in beams, and ties in columns are to start and end at a maximum of one half of the adjacent bar spacing.

SPLICES:
Unless otherwise shown the minimum length of lap for splicing parallel bars shall be as given in Table A.
When reinforcing bars of different size are to be spliced, the lap length shall be governed by the smaller diameter bar.
Splices are to be made so that the given distances to face of concrete will be maintained.
The minimum lap requirement "S" for non-parallel bars shall be as given in Table B

TABLE A

LENGTH OF LAPPED SPLICE "L", IN INCHES*

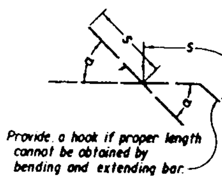
BAR NO.	L	BAR NO.	L
#3	14	#8	36
#4	18	#9	41
#5	23	#10	46
#6	27	#11	54
#7	32		

*Stagger splices if bars are spaced closer than 8 inches or 8 bar diameters, whichever is greater.

TABLE B

LENGTH OF SPLICE "S", FOR NON-PARALLEL BARS

ANGLE α	S
0° To 10°	0.5L
10° To 30°	0.6L
30° To 50°	0.7L
50° To 65°	0.8L
65° To 80°	0.9L
80° To 90°	1.0L



ACCESSORIES:
Bar supports, spacers and other accessories are not shown on the design drawings. The recommendations of ACI 315-65 or other approved supporting systems may be used

REFERENCES:
Numerals in parentheses ( ) following notes and section letters or numbers indicate the number of the drawing upon which the section or detail is shown; for example (524) denotes Drawing No. 557-D-524.
Unless otherwise shown follow the recommendations established by American Concrete Institute's "Manual of Standard Practice for Detailing Reinforced Concrete Structures" (ACI 315-65).

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS

GENERAL NOTES
AND MINIMUM REQUIREMENTS
FOR DETAILING REINFORCEMENT
CLASS 60

DRAWN E.S.T. SUBMITTED Jack W. Hill
TRACED R.L.P. RECOMMENDED H. G. Arthur
CHECKED APPROVED
CHIEF ENGINEER
DENVER, COLORADO, JUNE 20, 1967 — 40-D-6123

PROTECTIVE COVERING FOR REINFORCEMENT

MINIMUM CLEAR COVER OF MAIN REINFORCEMENT

PROTECTIVE COVERING OF REINFORCEMENT FROM OUTSIDE OF BAR TO SURFACE OF MONOLITHIC CONCRETE

	SLABS AND PAVINGS 9" THICK AND LESS	SLABS OVER 9" THICK EXCLUSIVE OF WEARING SURFACE	WALLS	BEAMS (TOP AND BOTTOM)	GIRDERS (TOP AND BOTTOM)	BEAMS AND GIRDERS (SIDES)	COLUMNS	FOOTINGS	SIPHON BARRELS	TUNNELS	GALLERIES IN DAMS
Unexposed to weather, or no fire hazard.(*)	3/4"	1"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"			5"
Fire resistive		1 1/2"	1 1/2"	2"	2 1/2"	1 1/2"	2 1/2"				
Structures exposed to weather, or backfill, or submerged and can be made accessible — Bar Nos. 5 and less	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	1 1/2"	
Structures exposed to weather, or backfill, or submerged and can be made accessible — Bar Nos. 6 and 7		2"	2"	2"	2"	2"	2"	2"	2"	2"	
Structures exposed to weather, or backfill, or submerged and can be made accessible — Bar Nos. 8 and over		2"	2"	2"	2 1/2"	2"	2 1/2"	2"	2 1/2"	2 1/2"	
Structures submerged and can not be made accessible — Bar Nos. 7 and less		2 1/2"	2 1/2"	2 1/2"	2 1/2"	2 1/2"	2 1/2"	2 1/2"	2 1/2"	2 1/2"	
Structures submerged and can not be made accessible — Bar Nos. 8 and over		2 1/2"	2 1/2"	2 1/2"	3"	2 1/2"	3"	2 1/2"	3"	3"	
Structures submerged and can not be made accessible where failure due to rusting would cause loss of life or structure, such as water side of concrete scroll cases, tailrace and backfilled walls of power plants, etc.		3"	3"	3"	3"	3"	3"	3"	3"	3"	
Concrete placed directly against ground, or rock, or surfaces subjected to corrosive liquids.(**)	2"	3"	3"	3"	3"	3"	3"	3"	3"	1" to A line	

* The diameter of the bar shall be used as minimum cover if greater than cover listed.
** Clear cover is to the tie or stirrup where surface is subjected to corrosive liquids.

NOTES

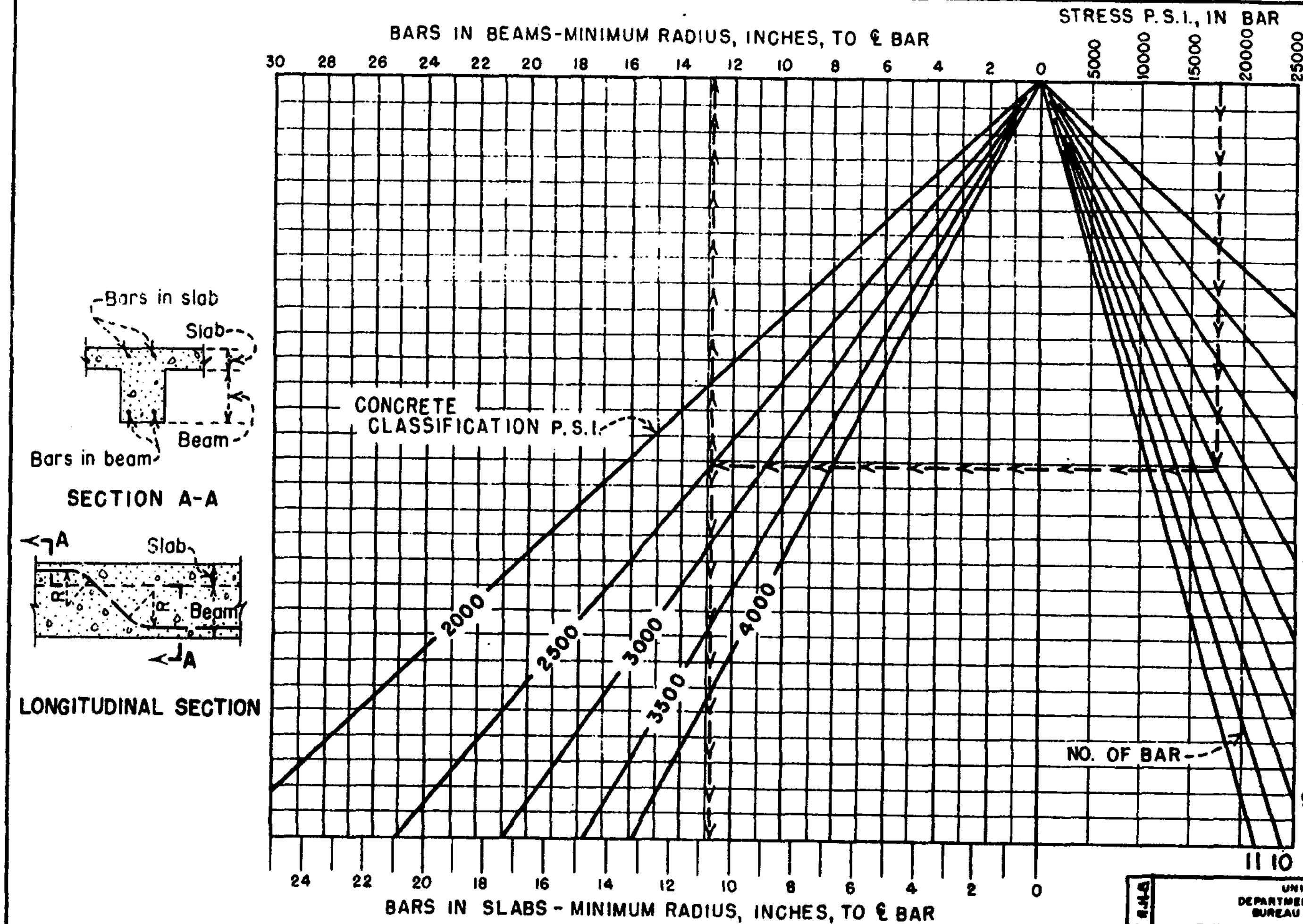
All stair slabs and landings in enclosed wells; use 3/4" clear cover.
At underside of bridge slabs exposed to weather, 1" clear cover shall be provided.
Special consideration shall be given to any design condition not included herein such as scour, other agency requirements, etc.

REV. 7-10-53
REV. 2-6-51 E.L.W.
REV. 6-19-52 E.L.W.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
REINFORCED CONCRETE DESIGN
PROTECTIVE COVERING FOR REINFORCEMENT

DRAWN E.L.W. SUBMITTED [signature]
TRACED D.B.B. RECOMMENDED [signature]
CHECKED H.B. APPROVED [signature]
DENVER, COLORADO. JAN 12, 1951
103-D-143

RADIUS OF BEND FOR BEAM AND SLAB REINFORCEMENT



EXAMPLE:

Given- Calculated stress in #7 bar of 17,500 p.s.i. and allowable ultimate concrete stress of 2500 p.s.i.

Required- Minimum radius of bend for beam or slab bars.

Answer - R = 12$\frac{3}{4}$" in beam. R = 10$\frac{3}{4}$" in slab.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

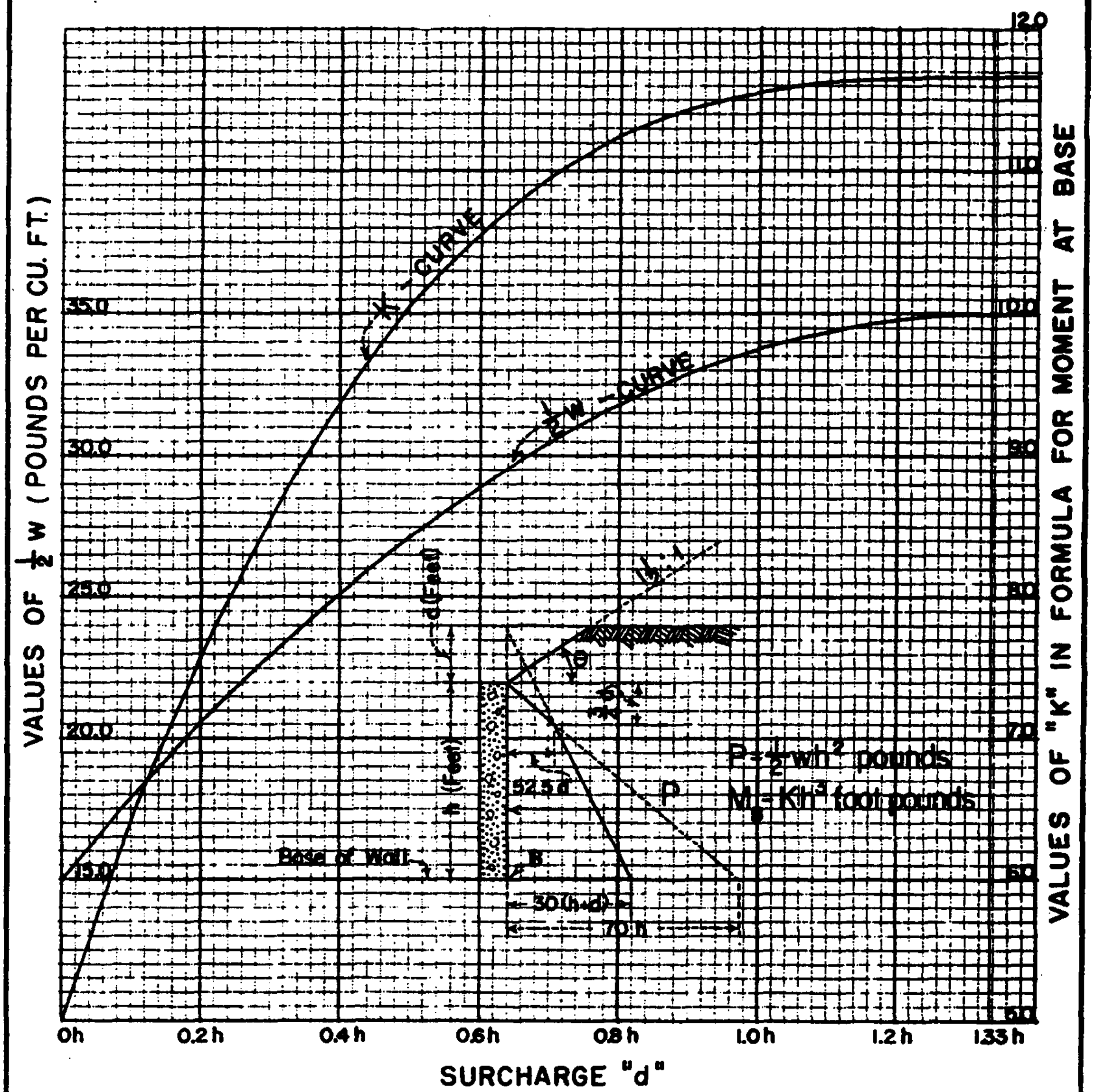
REINFORCED CONCRETE DESIGN

RADIUS OF BEND FOR BEAM AND SLAB REINFORCEMENT

DRAWN A.L.A. SUBMITTED
TRACED C.E.M. RECOMMENDED
CHECKED APPROVED

DENVER, COLORADO, JAN. 12, 1951 | 103-D-142

EARTH PRESSURE AND MOMENT CURVES--ANGLE OF REPOSE = 1-1/2:1 Fig. 7 Par. 2.18



EARTH PRESSURE AND MOMENT CURVES VERTICAL WALL WITH SURCHARGE

ANGLE OF REPOSE = ANGLE OF SURCHARGE = $1\frac{1}{2}$ TO 1

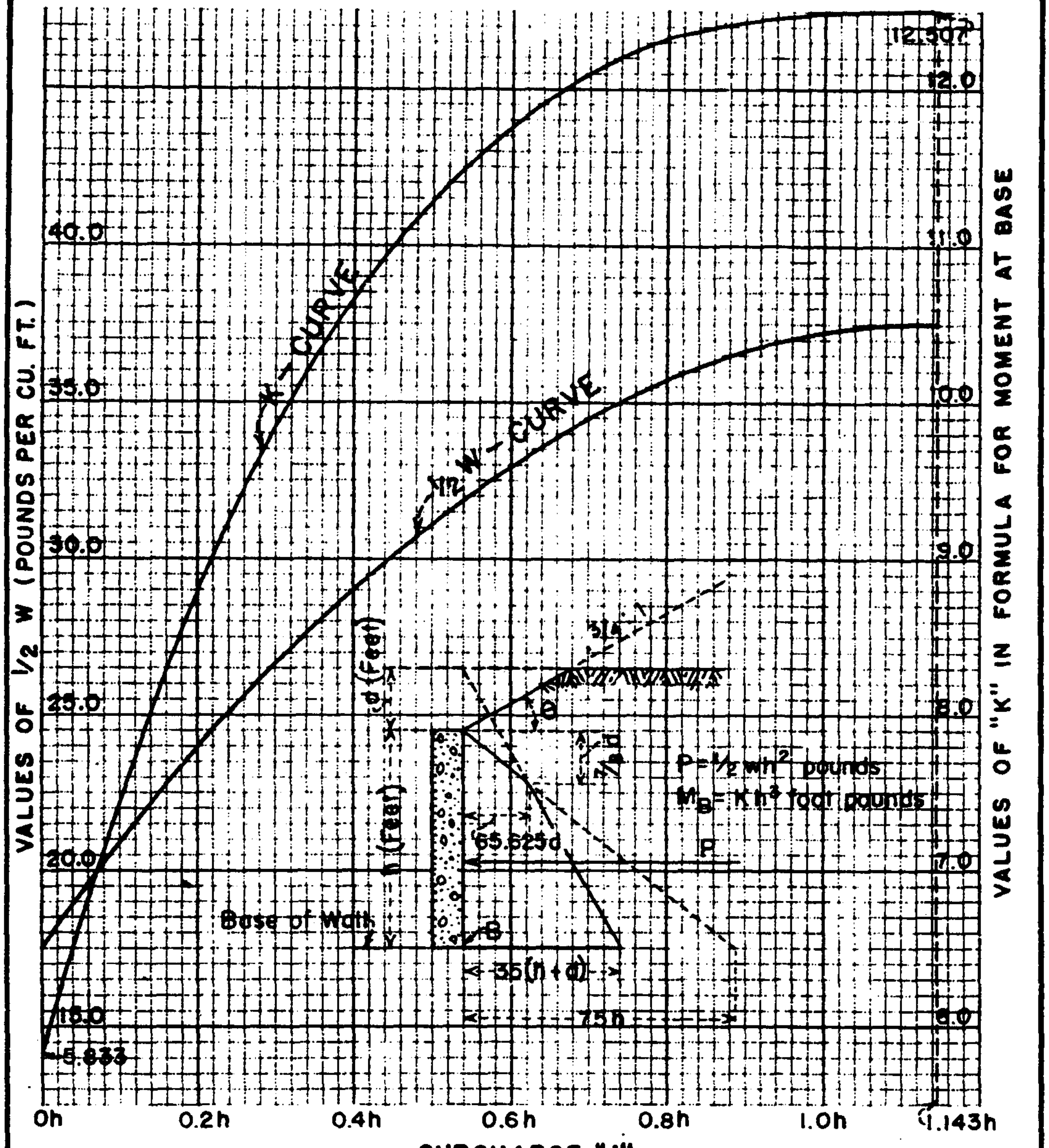
Note: Pressures in pounds per square foot are based on the following equivalent unit weights of fluid:
W = 30# per cu. ft. for no surcharge
W = 70# per cu. ft. for infinite surcharge having slope of $1\frac{1}{2}:1$

DRAWN: W.E.D.
CHECKED: R.S.

REV. 7-12-51-K.H.B. FORMERLY X-D-954 103-D-197

Fig. 8 Par. 2.18

EARTH PRESSURE AND MOMENT CURVES--ANGLE OF REPOSE = 1-3/4:1



EARTH PRESSURE AND MOMENT CURVES VERTICAL WALL WITH SURCHARGE

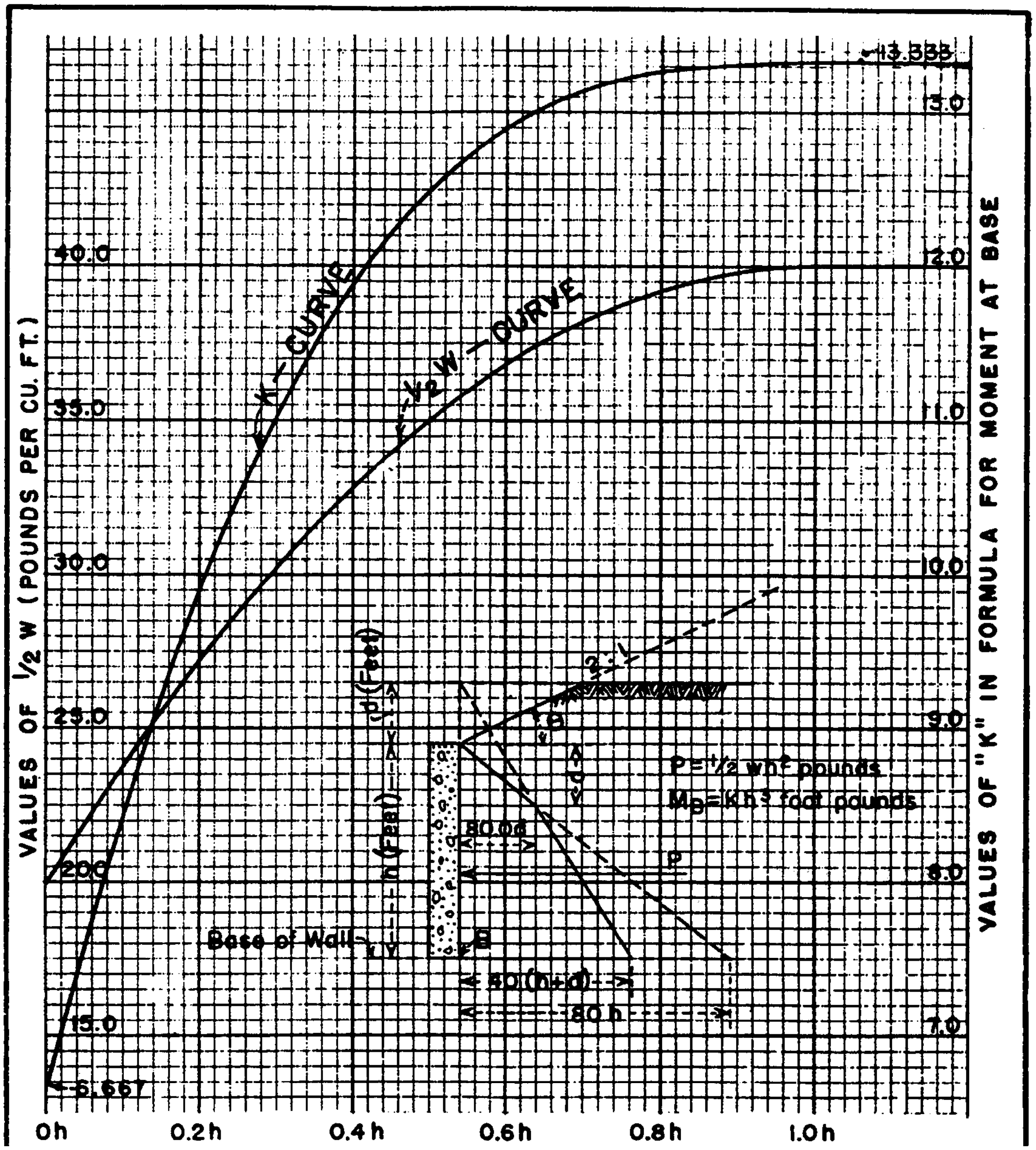
ANGLE OF REPOSE = ANGLE OF SURCHARGE = 1 3/4 TO 1

Note: Pressures in pounds per square foot are based on the following equivalent unit weights of fluid:
W = 35# per cu. ft. for no surcharge.
W = 75# per cu. ft. for infinite surcharge having slope of 1 3/4 : 1

DRAWN: W.Z.P.
CHECKED: C.J.L.
FORMERLY X-D-1790
103-D-198
REV. 7-12-51-H.H.B.

EARTH PRESSURE AND MOMENT CURVES--ANGLE OF REPOSE = 2:1

Fig. 9 Par. 2.18



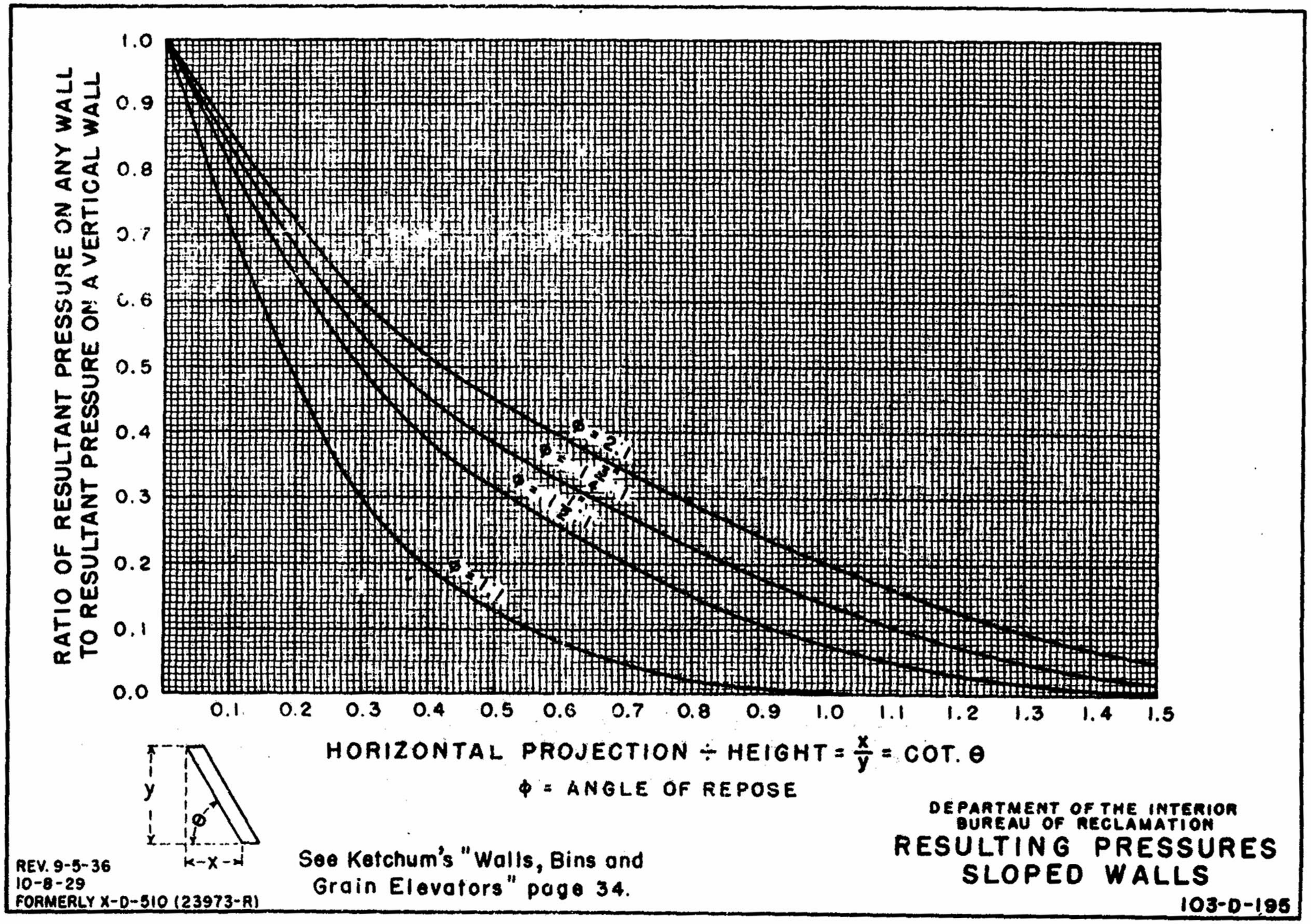
EARTH PRESSURE AND MOMENT CURVES

VERTICAL WALL WITH SURCHARGE

ANGLE OF REPOSE = ANGLE OF SURCHARGE = 2 TO 1

Note: Pressures in pounds per square foot are based on the following equivalent unit weights of fluids
w = 40# per cu. ft. for no surcharge.
w = 80# per cu. ft. for infinite surcharge having slope of 2:1.

DRAWN W.E.O.
CHECKED C.J.L. FORMERLY X-D-1791 103-D-199
REV. 7-12-51 - H.H.B.


RATIO OF RESULTANT PRESSURE ON ANY WALL TO RESULTANT PRESSURE ON A VERTICAL WALL
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
0.1 0.2 0.3 0.4 0.5 0.6 0.7 0.8 0.9 1.0 1.1 1.2 1.3 1.4 1.5
HORIZONTAL PROJECTION ÷ HEIGHT = x/y = COT. θ
φ = ANGLE OF REPOSE
y
θ
x
REV. 9-5-36
10-8-29
FORMERLY X-D-510 (23973-R)
See Ketchum's "Walls, Bins and Grain Elevators" page 34.
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
RESULTING PRESSURES
SLOPED WALLS
103-D-195

9-26-67 REV.
4-3-49 REV.
3-18-46 H.C.D.

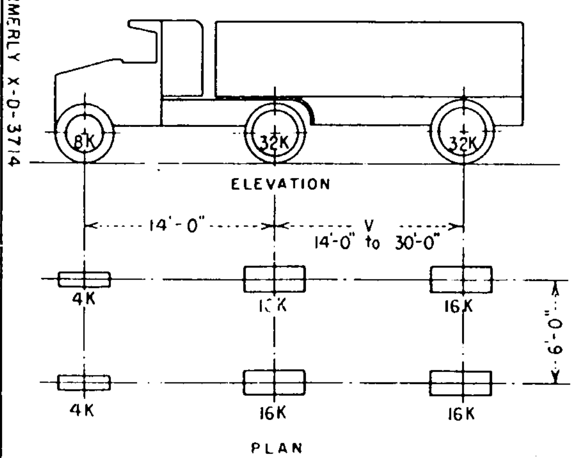
FORMERLY X-D-3714

DISTRIBUTION OF WHEEL LOADS FOR DESIGN OF CULVERTS

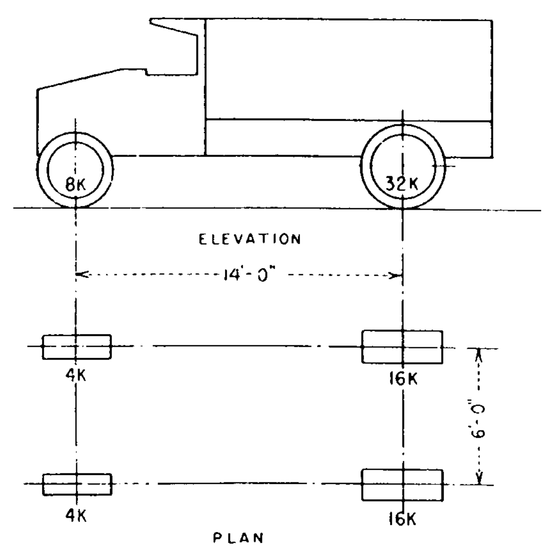
I. HIGHWAY LOADING

A. LOADING: Use truck loadings for all spans.

a. Main highways HS20-44



b. County highways H20-44



B. LOAD DISTRIBUTION

a. Wheels resting directly on slab.

(1.) Spans under 12 feet

$E = .175S + 3.2$, where E = width of slab over which a wheel load is to be distributed and S = span length.

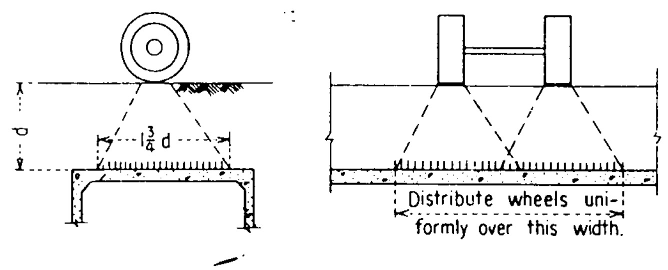
(2) Spans over 12 feet

$E = \frac{10N + W}{4N}$

where N = total number of lanes (@ 10') permissible on the bridge and W = width of roadway.

b. Wheel loads transmitted thru fills.

(1) When the depth of fill is 2 feet or more, concentrated loads shall be considered as uniformly distributed over a square, the sides of which are equal to $1\frac{3}{4}$ times the depth of fill. When such areas from several concentrations overlap, the total load shall be considered as uniformly distributed over the area defined by the outside limits of the individual areas.



(2) For single spans the effect of live load may be neglected when the depth of fill is more than 8 feet and exceeds the span length; for multiple spans it may be neglected when the depth of fill exceeds the distance between end supports or abutments.

(3) In no case shall the stresses for live load as determined by the above be greater than those obtained where the wheel is applied directly on the structure.

c. In no case shall the total width of distribution exceed the total width of supporting slab.

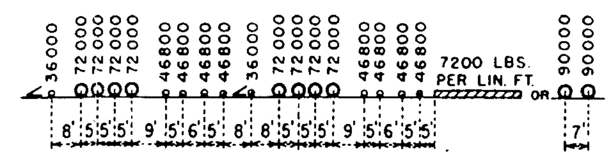
C. IMPACT

For fills	0'	to 1'-0"	Impact	= 30 %
" "	1'-1"	" 2'-0"	"	= 20 %
" "	2'-1"	" 2'-11"	"	= 10 %
" "	3'-0" and over		"	= 0 %

II. RAILROAD LOADING

A. LOADING

a. Main lines use Cooper's E-72 loading



b. Less important lines Cooper's E loadings of smaller capacity may be used such as E-60, E-50 etc. The spacing of the wheels remain the same but the axle loads are reduced in proportion.

B. LOAD DISTRIBUTION

a. Longitudinal distribution

$$l = \frac{d}{2.5}(2.0) + 3.0$$

where l = longitudinal distribution per wheel.
Maximum l = 5.0'
d = depth of ballast and fill below bottom of tie.

b. Transverse distribution

1. For ballasted deck and for cases where the total depth of ballast and fill below the bottom of tie is 6.0' or less, the axle load shall be distributed over a width of 14'.
2. Where "d" is greater than 6.0' the width of distribution W shall be W = 8.0 + d.
3. For multiple tracks where the distribution from several tracks overlap, the width W shall be that determined by the outer limits for the individual tracks.
4. In no case shall the total width of distribution exceed the total width of supporting slab.

C. IMPACT

$I = \%\ \text{impact} = \frac{L}{L+D}$

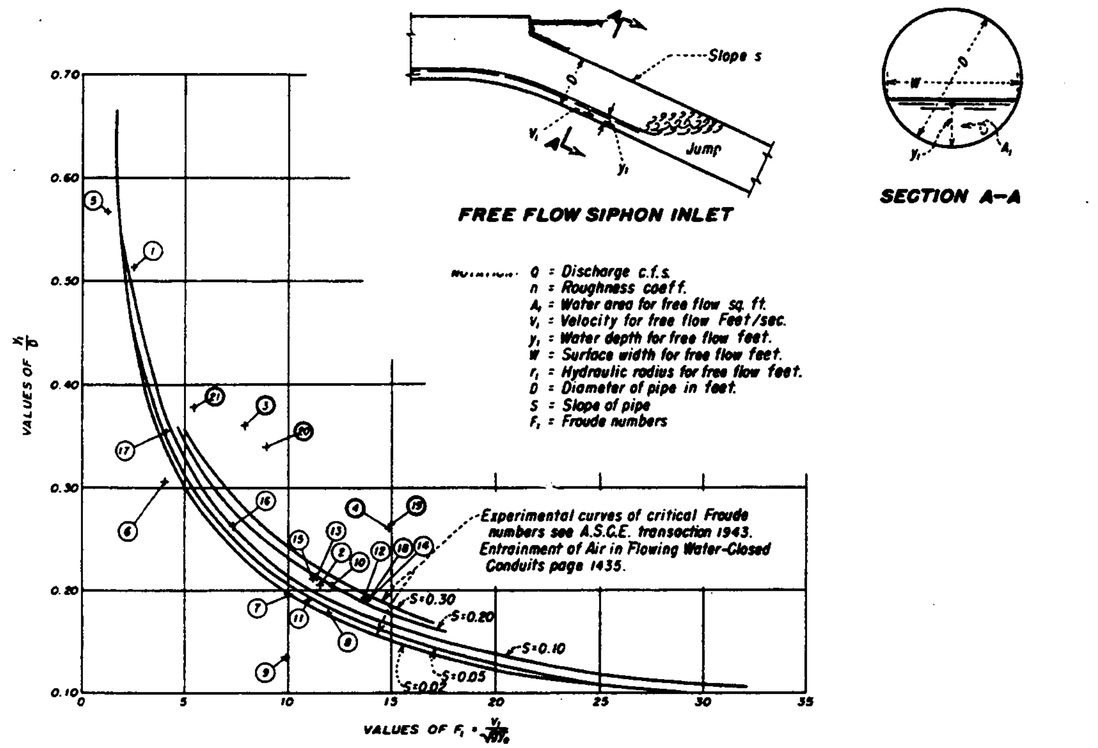
L = Live load

D = Dead load

103-D-229

DESIGN OF FREE-FLOW SIPHON INLETS

Fig. 12 Par. 2.25B



	LOCATION	TYPE OF PIPE	Q	D	n	s	$\frac{y_1}{D}$	F_1
1	Yakima River – Yakima Project	Conc.	925	111"	.010	.82	.511	2.47
2	Malheur River – Owyhee Project	Steel	325	80"	.010	.213	.205	11.60
3	Basin Siphon – King Hill	Wood	250	57"	.012	.17	.360	7.90
4	Basin Siphon – King Hill	Conc.	250	54"	.012	.61	.260	14.90
5	San Diego Sta. 620+00 N	Conc.	95	54"	.010	.0033	.567	1.25
6	San Diego Sta. 961+00 N	Conc.	95	54"	.010	.029	.305	4.10
7	San Diego Sta. 1146+00 N	Conc.	95	54"	.010	.19	.194	10.0
8	San Diego Sta. 1545+15 N	Conc.	95	54"	.010	.2716	.177	11.90
9	San Diego Sta. 1817+50 N	Conc.	95	72"	.010	.1935	.134	9.95
10	San Diego Sta. 1873+50 N	Conc.	95	48"	.010	.3053	202	12.15
11	San Diego Sta. 2200+02 N	Conc.	95	54"	.010	.2198	.185	10.95
12	San Diego Sta. 1303+56 S	Conc.	95	48"	.010	.3643	.192	13.70
13	San Diego Sta. 1217+50 S	Conc.	95	48"	.010	.2563	.212	11.35
14	San Diego Sta. 1073+00 S	Conc.	95	48"	.010	.3714	.19	13.83
15	San Diego Sta. 1073+00 S	Conc.	95	48"	.012	.3714	.211	11.20
16	San Diego Sta. 419+00 S	Conc.	95	48"	.010	.104	.262	7.33
17	San Diego Sta. 171+96 S	Conc.	95	48"	.010	.0336	.355	4.17
18	San Diego Sta. 79+92 S	Conc.	95	48"	.010	.377	.190	13.80
19	High Mesa – Uncompaghre	Steel	42	26"	.010	.535	.264	15.00
20	Lake Valley Crossing P.G.E.	Steel	35	24"	.013	.367	.349	8.95
21	Lake Valley Crossing P.G.E.	Steel	25	24"	.016	.208	.378	5.43

NOTES

Siphon inlets marked thus ◯, have given trouble in operation and air outlets were installed in some cases to relieve the blowing back of air and water.

All other siphons have not given trouble in operation.

Study made indicates that free flow siphon inlets designed so that Froude number will not fall above the critical curves established by experiments will give satisfactory performance.

Proceedure to determine Froude number.

For a given Q, diameter D, slope s and coeff. n, proceed as follows:

a. Calculate V_1 with Manning's Formula.

b. Calculate y_1

c. Calculate $W = 2\sqrt{(D-y_1)y_1}$

d. Calculate $y_e = \frac{A_1}{W}$

e. Calculate Froude number $F_1 = \frac{V_1}{\sqrt{gy_e}}$

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

DESIGN OF FREE FLOW SIPHON INLETS

DRAWN J.W.F. SUBMITTED
TRACED E.P.B. RECOMMENDED
CHECKED E.J. APPROVED
DENVER, COLORADO, FEB. 1, 1949

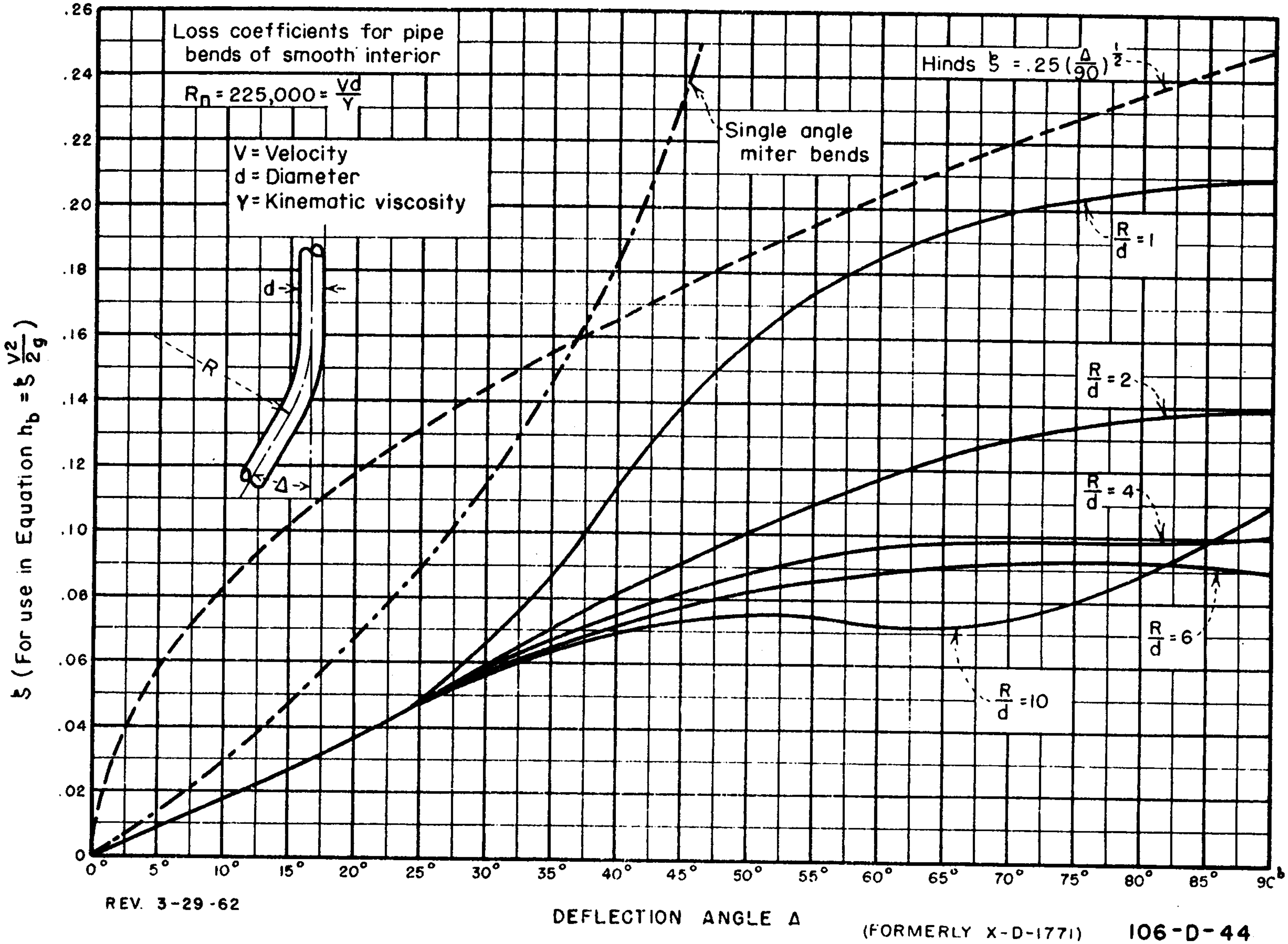
103-D-1

REV. 5-29-62

REV. 8-9-55

Fig. 13 Par. 2.25C

HEAD LOSSES IN PIPE BENDS



COMPOUND PIPE BENDS

In constructing a pipe line to conform to a certain contour, it is sometimes necessary to use both vertical and horizontal angles. The construction is greatly simplified if the pipe is bent to one angle which will accommodate both horizontal and vertical deflections.

The following formula will give the pipe angle which will accommodate two vertical angles and one horizontal angle:

$$\cos X = \cos A \ \cos B \ \cos C \pm \sin A \ \sin B$$

where X = Pipe angle with vertex on YY.
A and B = Projected angles (with the horizontal) in a vertical plane.
C = Projected angle in a horizontal plane.

The minus sign (-) is used when both vertical angles (A and B) are above the horizontal, and the plus sign (+) is used when one vertical angle is above the horizontal and the other vertical angle is below the horizontal.

Two examples will illustrate the use of the formula:

EXAMPLE 1

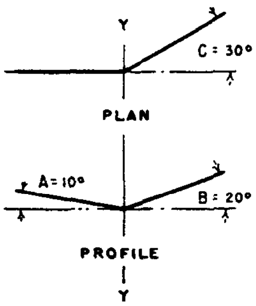


Fig. 1

Both vertical angles above the horizontal.

Find the pipe angles (X) which will accommodate the following angles:

$A = 10°$
$B = 20°$
$C = 30°$

See diagram marked Fig. 1.

The formula is:

$\cos X = \cos A \ \cos B \ \cos C - \sin A \ \sin B$

$\cos X = 0.98481 \times 0.93969 \times 0.86603 - 0.17365 \times 0.34202$

$\cos X = 0.80144 - 0.05939$

$\cos X = 0.74205$

Therefore $X = 42° \ 06'$

EXAMPLE 2

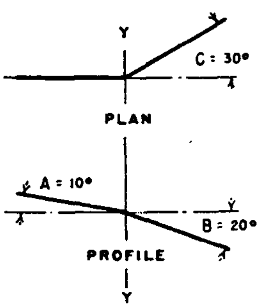


Fig. 2

One vertical angle above the horizontal and the other vertical angle below the horizontal.

Find the pipe angle (X) which will accommodate the following angles:

$A = 10°$
$B = 20°$
$C = 30°$

See diagram marked Fig. 2.

The formula is:

$\cos X = \cos A \ \cos B \ \cos C + \sin A \ \sin B$

$\cos X = 0.98481 \times 0.93969 \times 0.86603 + 0.17365 \times 0.34202$

$\cos X = 0.80144 + 0.05939$

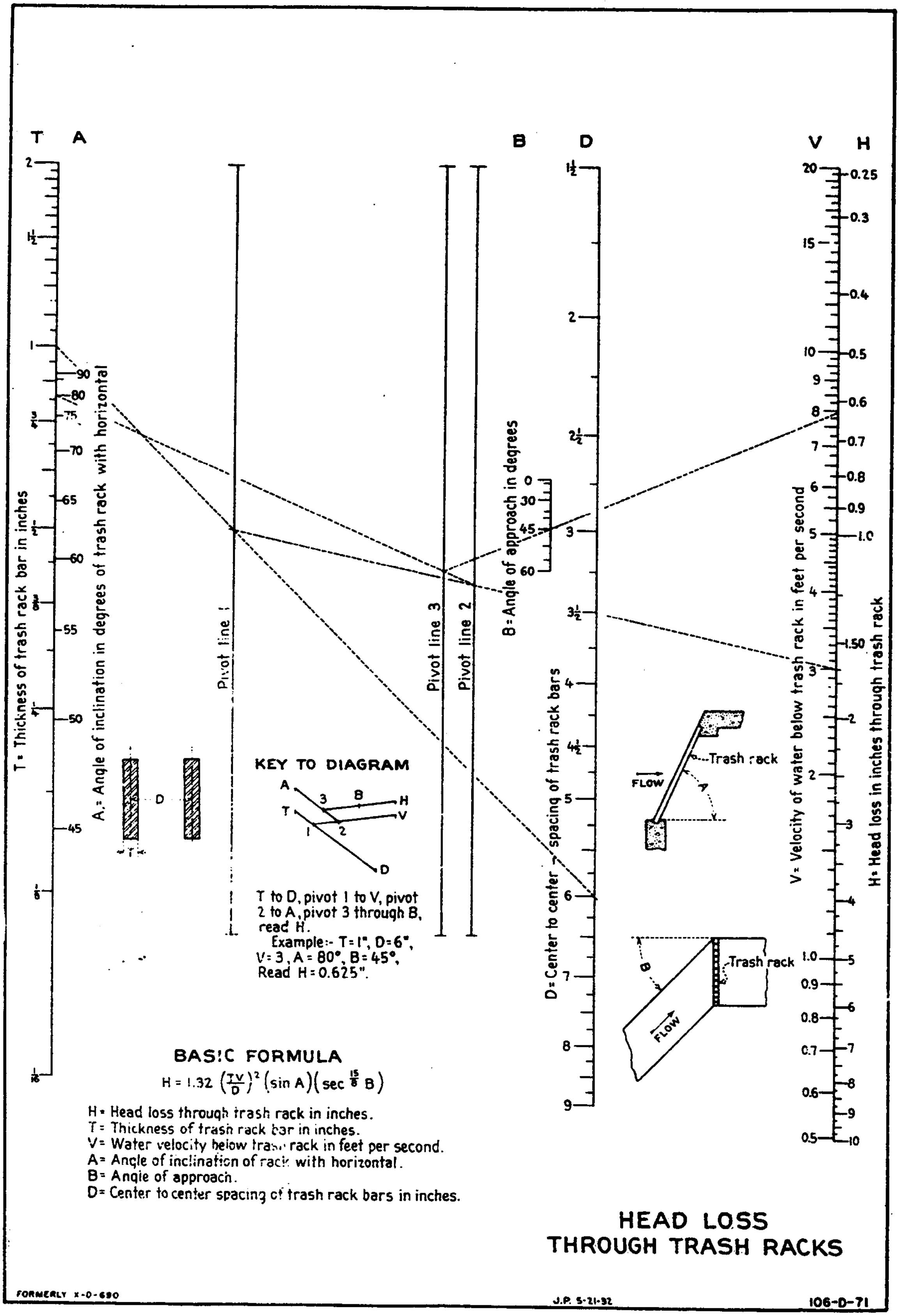
$\cos X = 0.86083$

Therefore $X = 30° \ 35'$

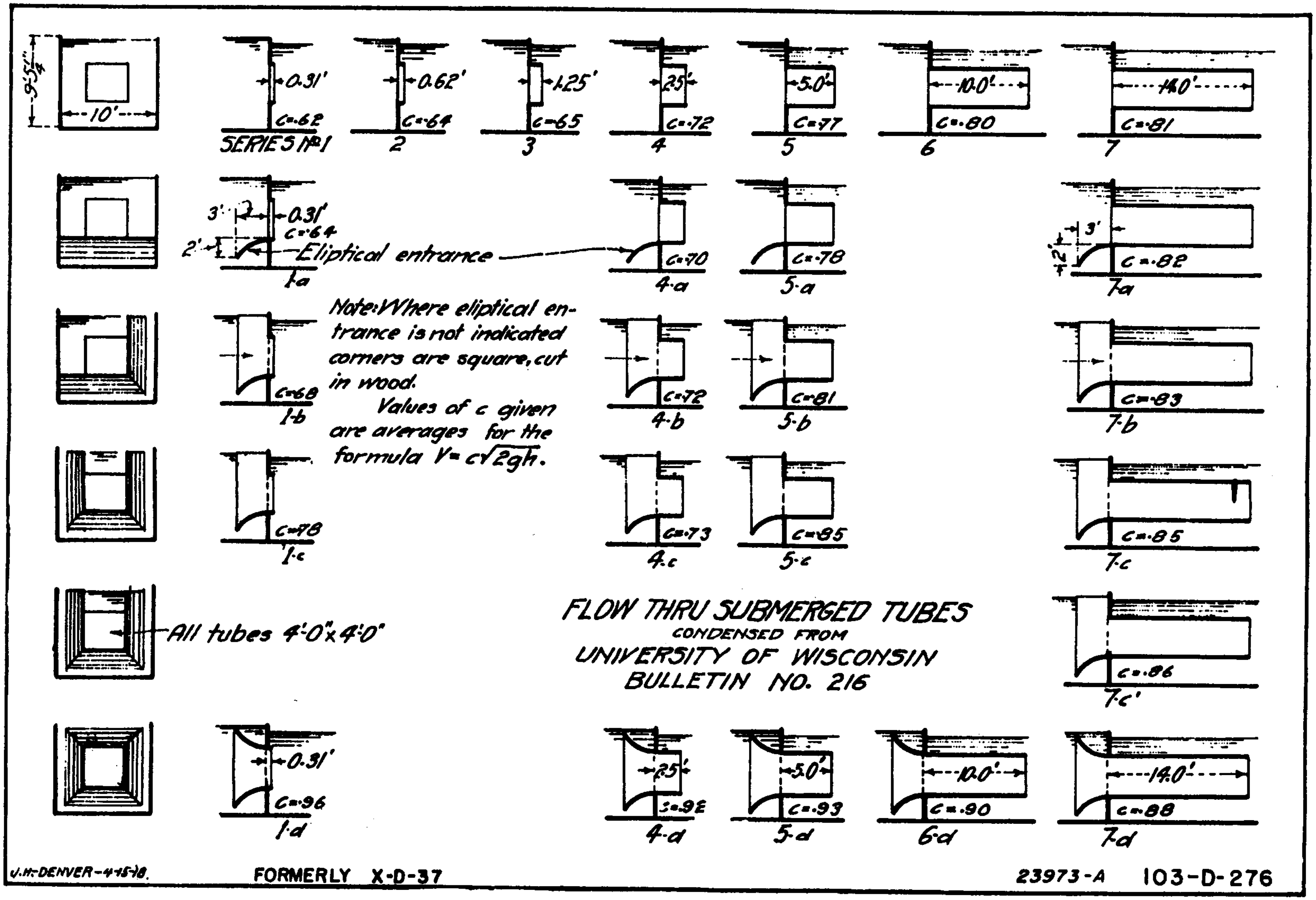
From Page 22 of Bulletin 21 of National Tube Company
P.B.- V.F.S.- 2-20-45 FORMERLY X-D-1101 106-D-32

Fig. 15 Par. 2.25D

HEAD LOSS THROUGH TRASHRACKS



FLOW THROUGH SUBMERGED TUBES

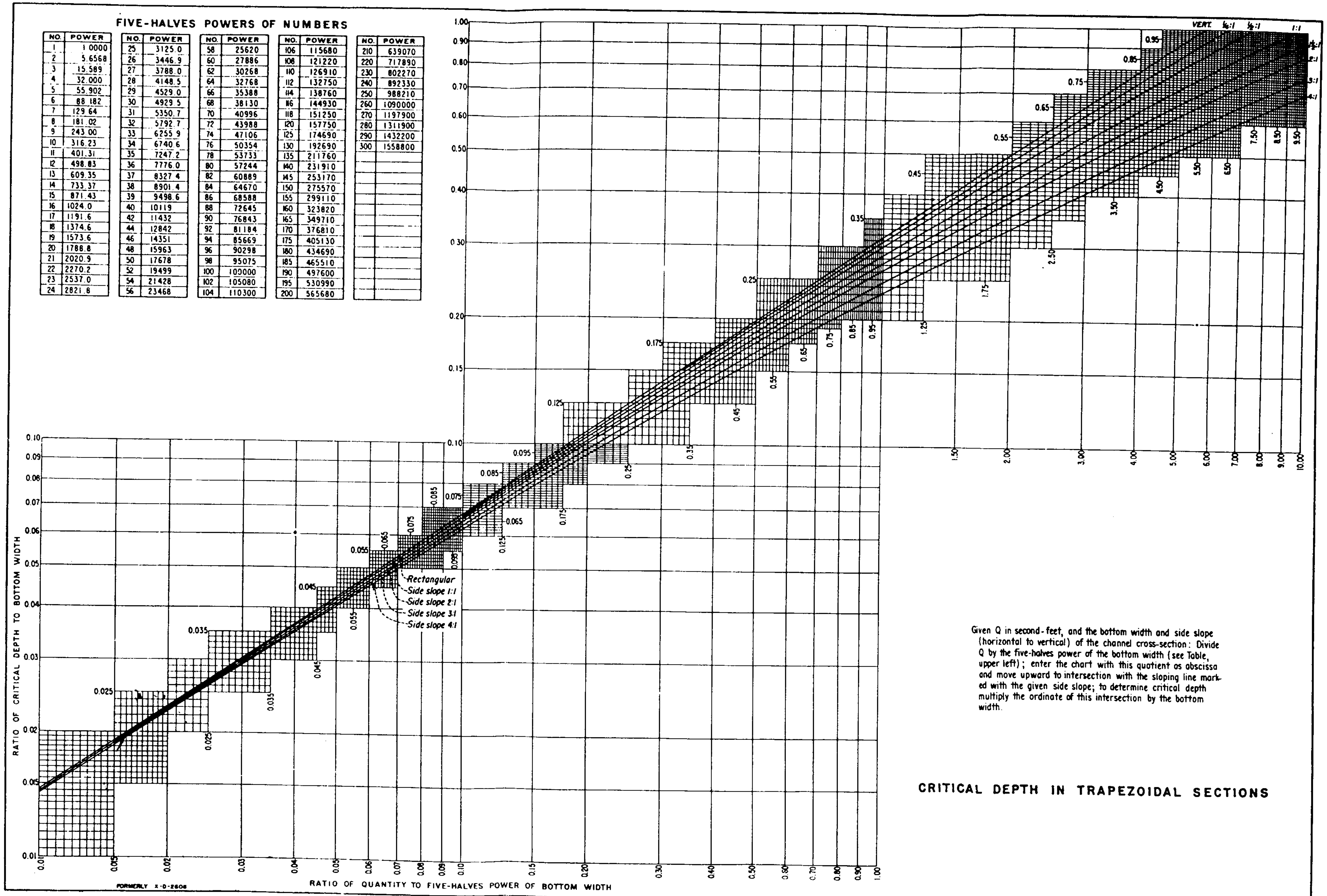


CRITICAL DEPTH IN TRAPEZOIDAL SECTIONS

Fig. 17 Par. 2.25F

FIVE-HALVES POWERS OF NUMBERS

NO.	POWER	NO.	POWER	NO.	POWER	NO.	POWER	NO.	POWER
1	1.0000	25	3125.0	58	25620	106	115680	210	639070
2	5.6568	26	3446.9	60	27886	108	121220	220	717890
3	15.589	27	3788.0	62	30268	110	126910	230	802270
4	32.000	28	4148.5	64	32768	112	132750	240	892330
5	55.902	29	4529.0	66	35388	114	138760	250	988210
6	88.182	30	4929.5	68	38130	116	144930	260	1090000
7	129.64	31	5350.7	70	40996	118	151250	270	1197900
8	181.02	32	5792.7	72	43988	120	157750	280	1311900
9	243.00	33	6255.9	74	47106	125	174690	290	1432200
10	316.23	34	6740.6	76	50354	130	192690	300	1558800
11	401.31	35	7247.2	78	53733	135	211760		
12	498.83	36	7776.0	80	57244	140	231910		
13	609.35	37	8327.4	82	60889	145	253170		
14	733.37	38	8901.4	84	64670	150	275570		
15	871.43	39	9498.6	86	68588	155	299110		
16	1024.0	40	10119	88	72645	160	323820		
17	1191.6	42	11432	90	76843	165	349710		
18	1374.6	44	12842	92	81184	170	376810		
19	1573.6	46	14351	94	85669	175	405130		
20	1788.8	48	15963	96	90298	180	434690		
21	2020.9	50	17678	98	95075	185	465510		
22	2270.2	52	19499	100	100000	190	497600		
23	2537.0	54	21428	102	105080	195	530990		
24	2821.8	56	23468	104	110300	200	565680		

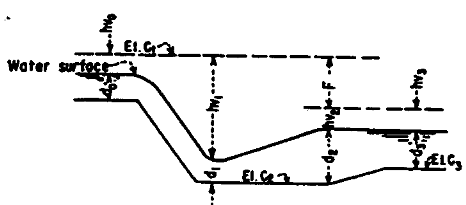


CRITICAL DEPTH IN TRAPEZOIDAL SECTIONS

103-D-278

F/dc	0		.1		.2		.3		.4		.5		.6		.7		.8		.9	
	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c	d_2/d_1	d_1/d_c
0	1.0	1.0	2.07	.680	2.48	.614	2.81	.572	3.09	.541	3.35	.516	3.60	.494	3.82	.477	4.04	.461	4.24	.448
1	4.44	.436	4.64	.425	4.82	.415	5.00	.405	5.18	.397	5.36	.389	5.53	.381	5.69	.375	5.86	.368	6.02	.362
2	6.18	.356	6.33	.351	6.49	.345	6.64	.340	6.79	.336	6.94	.331	7.09	.327	7.23	.323	7.38	.319	7.52	.315
3	7.66	.311	7.80	.308	7.94	.304	8.07	.301	8.21	.298	8.34	.295	8.48	.292	8.61	.289	8.74	.286	8.87	.284
4	9.00	.281	9.13	.278	9.26	.276	9.39	.274	9.51	.271	9.64	.269	9.76	.267	9.89	.265	10.01	.263	10.13	.261
5	10.25	.259	10.38	.257	10.50	.255	10.62	.253	10.73	.251	10.85	.250	10.97	.248	11.09	.246	11.21	.244	11.32	.243
6	11.44	.241	11.55	.240	11.67	.238	11.78	.237	11.90	.235	12.01	.234	12.12	.233	12.24	.231	12.35	.230	12.46	.228
7	12.57	.227	12.68	.226	12.79	.225	12.90	.223	13.01	.222	13.12	.221	13.23	.220	13.34	.219	13.45	.218	13.56	.216
8	13.66	.215	13.77	.214	13.88	.213	13.98	.212	14.09	.211	14.19	.210	14.30	.209	14.41	.208	14.51	.207	14.61	.206
9	14.72	.205	14.82	.204	14.93	.203	15.03	.202	15.13	.202	15.23	.201	15.34	.200	15.44	.199	15.54	.198	15.64	.197
10	15.74	.197	15.84	.196	15.95	.195	16.05	.194	16.15	.193	16.25	.193	16.35	.192	16.45	.191	16.54	.191	16.64	.190
11	16.74	.189	16.84	.188	16.94	.187	17.04	.187	17.13	.186	17.23	.185	17.33	.185	17.43	.184	17.52	.183	17.62	.183
12	17.72	.182	17.81	.181	17.91	.181	18.01	.180	18.10	.180	18.20	.179	18.29	.178	18.39	.178	18.48	.177	18.58	.176
13	18.67	.176	18.77	.175	18.86	.175	18.95	.174	19.05	.174	19.14	.173	19.24	.173	19.33	.172	19.42	.171	19.52	.171
14	19.61	.170	19.70	.170	19.79	.169	19.89	.169	19.98	.168	20.07	.168	20.16	.167	20.25	.167	20.34	.166	20.44	.166
15	20.53	.165	20.62	.165	20.71	.164	20.80	.164	20.89	.164	20.98	.163	21.07	.163	21.16	.162	21.25	.162	21.34	.161
16	21.43	.161	21.52	.160	21.61	.160	21.70	.160	21.79	.159	21.88	.159	21.97	.158	22.05	.158	22.14	.157	22.23	.157
17	22.32	.157	22.41	.156	22.50	.156	22.58	.155	22.67	.155	22.76	.155	22.85	.154	22.93	.154	23.02	.154	23.11	.153
18	23.19	.153	23.28	.152	23.37	.152	23.45	.152	23.54	.151	23.63	.151	23.71	.151	23.80	.150	23.89	.150	23.97	.150
19	24.06	.149	24.14	.149	24.23	.148	24.31	.148	24.40	.148	24.49	.147	24.57	.147	24.66	.147	24.74	.146	24.83	.146
20	24.91	.146	24.99	.145	25.08	.145	25.16	.145	25.25	.145	25.33	.144	25.42	.144	25.50	.144	25.58	.143	25.67	.143
21	25.75	.143	25.83	.142	25.92	.142	26.00	.142	26.08	.141	26.17	.141	26.25	.141	26.33	.141	26.42	.140	26.50	.140
22	26.58	.140	26.66	.139	26.75	.139	26.83	.139	26.91	.139	26.99	.138	27.08	.138	27.16	.138	27.24	.138	27.32	.137
23	27.40	.137	27.48	.137	27.57	.136	27.65	.136	27.73	.136	27.81	.136	27.89	.135	27.97	.135	28.05	.135	28.13	.135
24	28.22	.134	28.30	.134	28.38	.134	28.46	.134	28.54	.133	28.62	.133	28.70	.133	28.78	.133	28.86	.132	28.94	.132
25	29.02	.132	29.10	.132	29.18	.131	29.26	.131	29.34	.131	29.42	.131	29.50	.131	29.58	.130	29.66	.130	29.74	.130
26	29.82	.130	29.89	.129	29.97	.129	30.05	.129	30.13	.129	30.21	.128	30.29	.128	30.37	.128	30.45	.128	30.52	.128
27	30.60	.127	30.68	.127	30.76	.127	30.84	.127	30.92	.127	31.00	.126	31.07	.126	31.15	.126	31.23	.126	31.31	.126
28	31.38	.125	31.46	.125	31.54	.125	31.62	.125	31.69	.125	31.77	.124	31.85	.124	31.93	.124	32.00	.124	32.08	.124
29	32.16	.123	32.23	.123	32.31	.123	32.39	.123	32.46	.123	32.54	.122	32.62	.122	32.69	.122	32.77	.122	32.85	.122
30	32.92	.121	33.00	.121	33.08	.121	33.15	.121	33.23	.121	33.31	.121	33.38	.120	33.46	.120	33.53	.120	33.61	.120
31	33.68	.120	33.76	.119	33.84	.119	33.91	.119	33.99	.119	34.06	.119	34.14	.119	34.21	.118	34.29	.118	34.36	.118
32	34.44	.118	34.51	.118	34.59	.118	34.66	.117	34.74	.117	34.81	.117	34.89	.117	34.96	.117	35.04	.117	35.11	.116
33	35.19	.116	35.26	.116	35.34	.116	35.41	.116	35.49	.116	35.56	.115	35.63	.115	35.71	.115	35.78	.115	35.86	.115
34	35.93	.115	36.00	.115	36.08	.114	36.15	.114	36.23	.114	36.30	.114	36.37	.114	36.45	.114	36.52	.113	36.59	.113
35	36.67	.113	36.74	.113	36.81	.113	36.89	.113	36.96	.112	37.03	.112	37.11	.112	37.18	.112	37.25	.112	37.33	.112
36	37.40	.112	37.47	.112	37.55	.111	37.62	.111	37.69	.111	37.76	.111	37.84	.111	37.91	.111	37.98	.111	38.05	.110
37	38.13	.110	38.20	.110	38.27	.110	38.34	.110	38.42	.110	38.49	.110	38.56	.109	38.63	.109	38.70	.109	38.78	.109
38	38.85	.109	38.92	.109	38.99	.109	39.06	.109	39.14	.108	39.21	.108	39.28	.108	39.35	.108	39.42	.108	39.49	.108
39	39.56	.108	39.64	.107	39.71	.107	39.78	.107	39.85	.107	39.92	.107	39.99	.107	40.06	.107	40.14	.107	40.21	.106
40	40.28	.106	40.35	.106	40.42	.106	40.49	.106	40.56	.106	40.63	.106	[illegible].70	.106	40.77	.105	40.84	.105	40.91	.105

Relation of energy loss, critical depth, and depth before and after jump, for hydraulic jumps in rectangular channels with level floor.



F = Difference in energy levels at upstream and downstream ends of jump.

d_c = Critical depth for flow considered, based on pool width. $d_c = \sqrt[3]{\frac{q^2}{g}}$

d_1 = Depth at upstream end of jump.

d_2 = Depth at downstream end of jump.

$K = d_2 \div d_1$

El. C_2 = El. $C_1 - (d_1 + h_{v_1})$ = El. $C_1 - (d_2 + h_{v_2} + F)$

$$\frac{d_1}{d_c} = \sqrt[3]{\frac{2}{K(K+1)}}$$

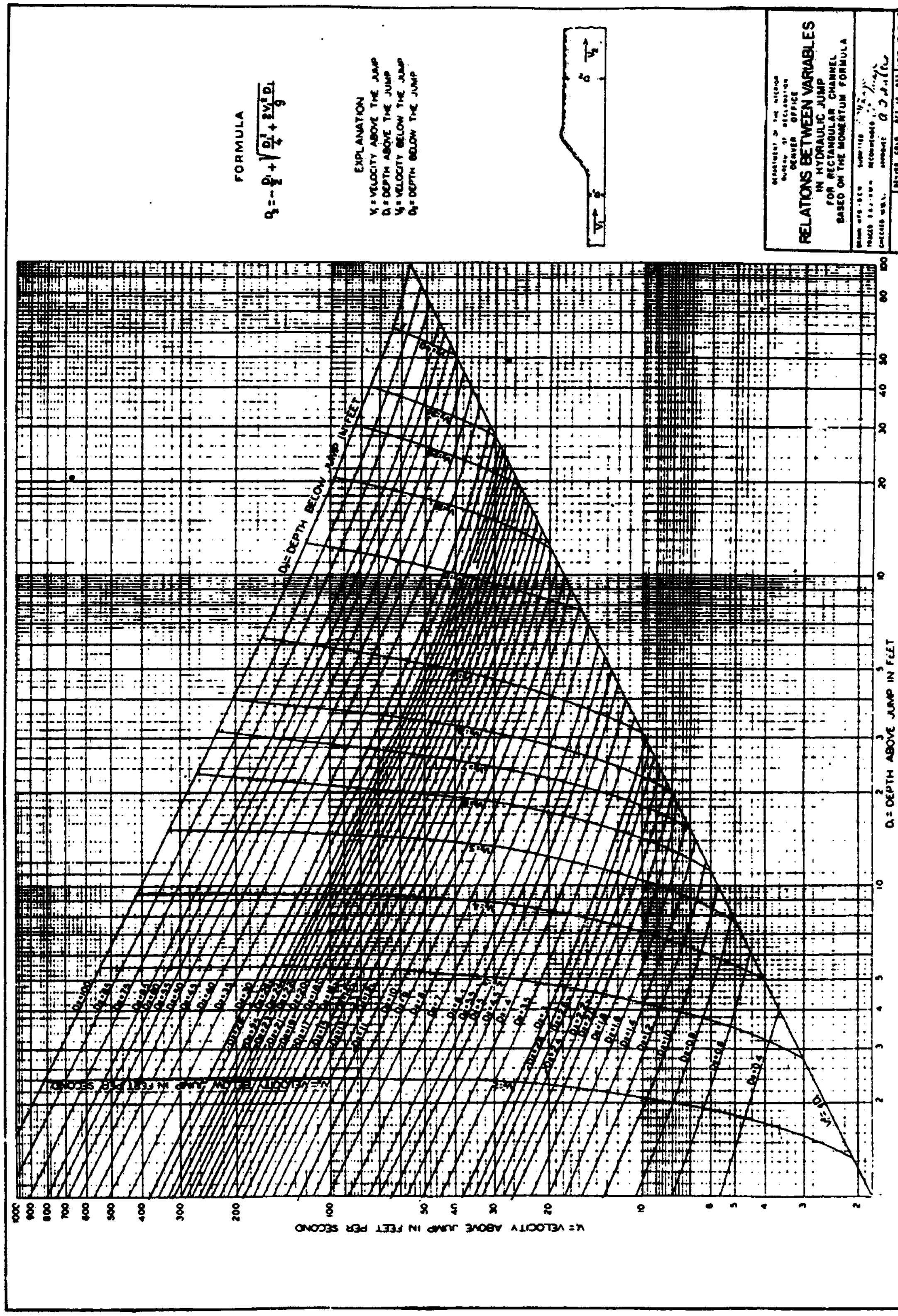
$$\frac{F}{d_c} = \frac{(K-1)^3}{4K}\sqrt[3]{\frac{2}{K(K+1)}}$$

Data from memorandum by C.R.Burky, April 6, 1939

ENERGY LOSS IN HYDRAULIC JUMP

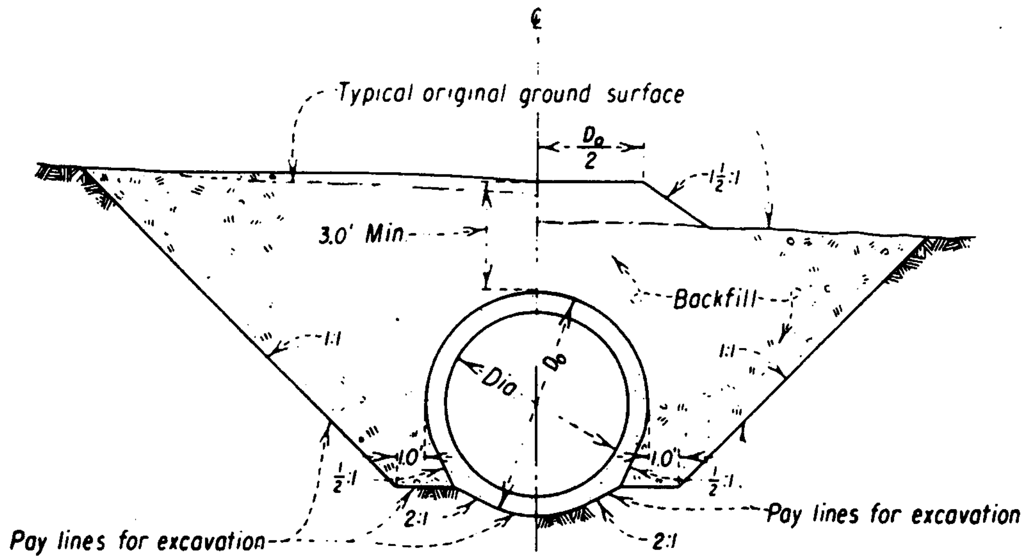
MAY 9, 1947 FORMERLY X-D-3479 103-D-289

Fig. 19 Par. 2.20 RELATIONS BETWEEN VARIABLES IN HYDRAULIC JUMP



EARTHWORK DETAILS, MONOLITHIC CONCRETE PIPE-- EARTH EXCAVATION

Fig. 20 Par. 2.35



EARTH

NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet; unless otherwise indicated on the structure drawings.

Backfill shall be compacted where required by the specifications.

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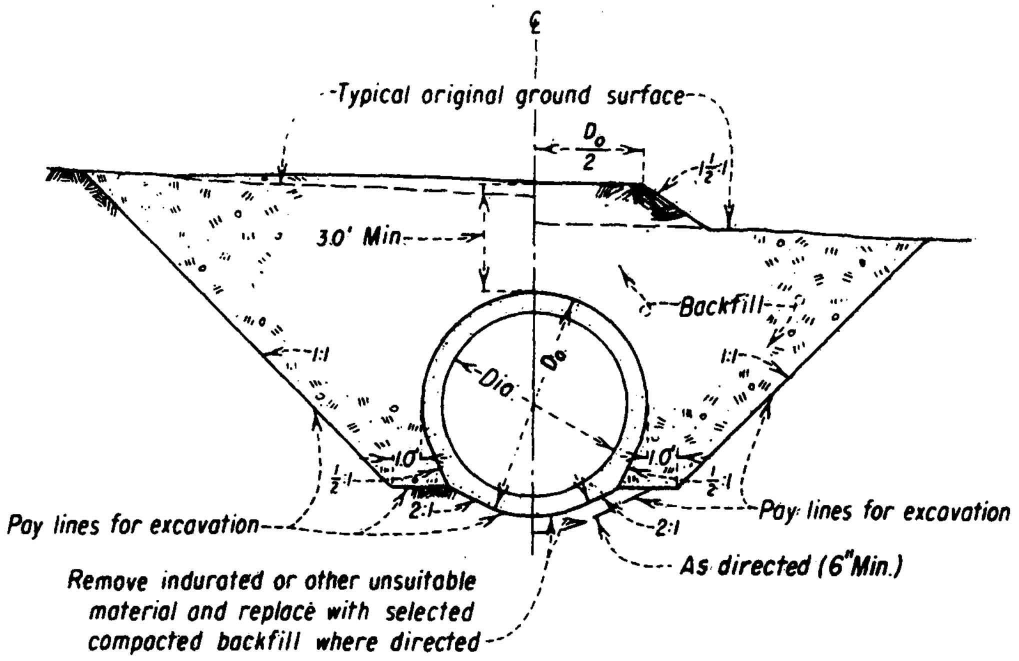
PIPE EARTHWORK DETAILS
MONOLITHIC CONCRETE PIPE
EARTH EXCAVATION

DRAWN A.W.B. SUBMITTED
TRACED T.R.M. RECOMMENDED
CHECKED APPROVED
CHIEF, CANALS BRANCH
DENVER, COLO. AUGUST 9, 1967 | 103-D-1062

Fig. 21 Par. 2.35 EARTHWORK DETAILS, MONOLITHIC CONCRETE PIPE-- UNCLASSIFIED EXCAVATION



NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet; unless otherwise indicated on the structure drawings.

Backfill shall be compacted where required by the specifications.

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PIPE EARTHWORK DETAILS
MONOLITHIC CONCRETE PIPE
UNCLASSIFIED EXCAVATION

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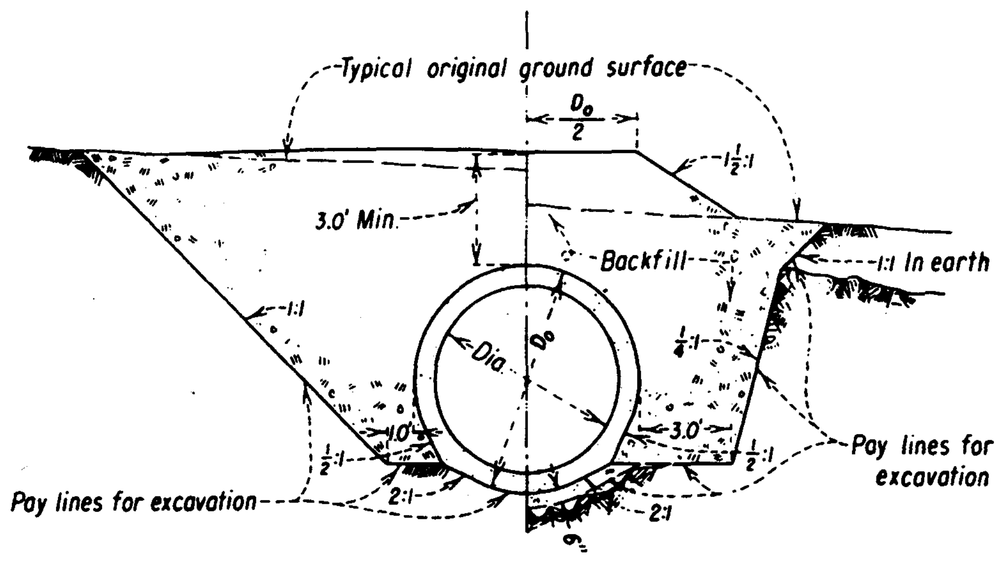
DENVER, COLO. AUGUST 9, 1967	103-D-1063

EARTHWORK DETAILS, MONOLITHIC CONCRETE PIPE--
EARTH OR ROCK EXCAVATION

Fig. 22 Par. 2.35



EARTH **ROCK**

NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet; unless otherwise indicated on the structure drawings.

Backfill shall be compacted where required by the specifications.

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EARTH OR ROCK EXCAVATION

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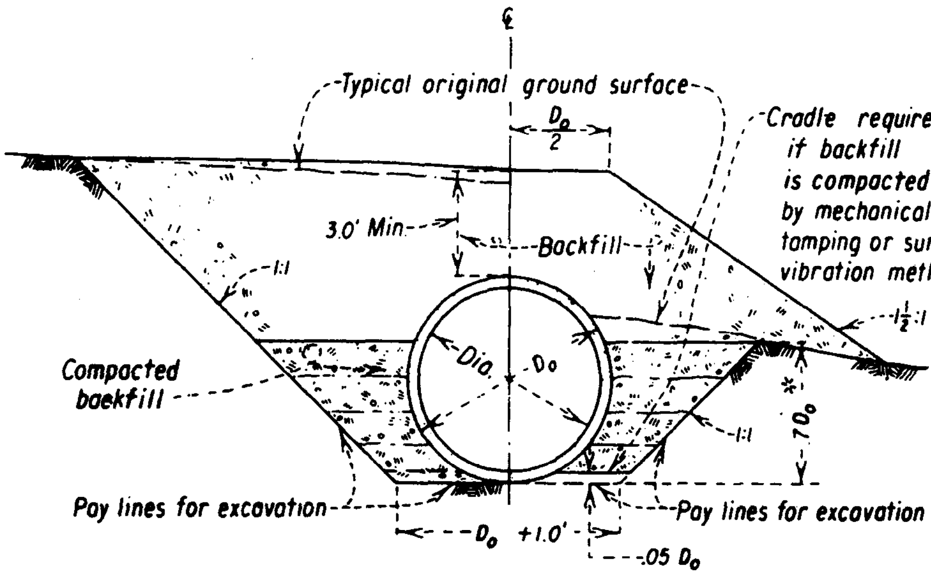
DENVER, COLO. AUGUST 9, 1967	103-D-1064

Fig. 23 Par. 2.35 EARTHWORK DETAILS, PRETENSIONED CONCRETE PIPE-- EARTH EXCAVATION



EARTH

NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet and the maximum shall be 5 ft. for Class A pipe, 10 ft. for Class B, 15 ft. for Class C, and 20 ft. for Class D unless otherwise indicated on the structure drawings.

Backfill shall be compacted as shown unless otherwise specified.

No compaction required for 10 inch dia. pipe.

**Compact backfill to $\frac{3}{8}$ D_o when pipe diameter is 12 inches to 18 inches.*

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PIPE EARTHWORK DETAILS
PRETENSIONED CONCRETE PIPE
EARTH EXCAVATION

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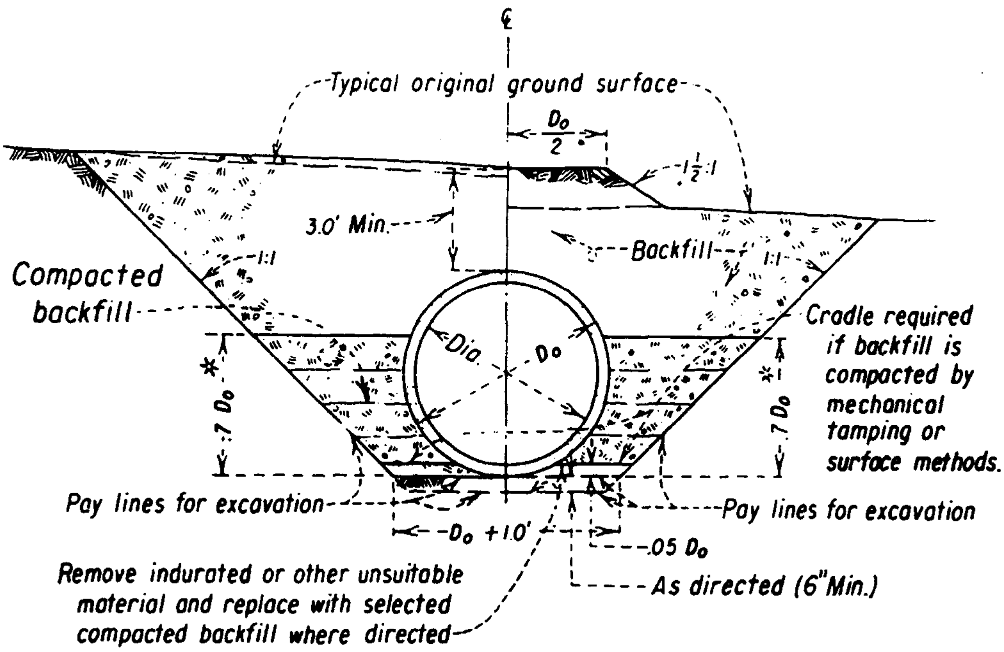
EARTHWORK DETAILS, PRETENSIONED CONCRETE PIPE--UNCLASSIFIED EXCAVATION

Fig. 24 Par. 2.35



NOTES

Backfill over pipe shell be to the original ground surface except the minimum shall be 3 feet and the maximum shall be 5 ft. for Class A pipe, 10 ft. for Class B, 15 ft. for Class C, and 20 ft. for Class D unless otherwise indicated on the structure drawings.

Backfill shall be compacted as shown unless otherwise specified.

No compaction required for 10 inch dia. pipe.

*Compact backfill to $\frac{3}{8} D_o$ when pipe diameter is 12 inches to 18 inches.

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PIPE EARTHWORK DETAILS
PRETENSIONED CONCRETE PIPE
UNCLASSIFIED EXCAVATION

DRAWN A.W.B. SUBMITTED
TRACED T.R.M. RECOMMENDED
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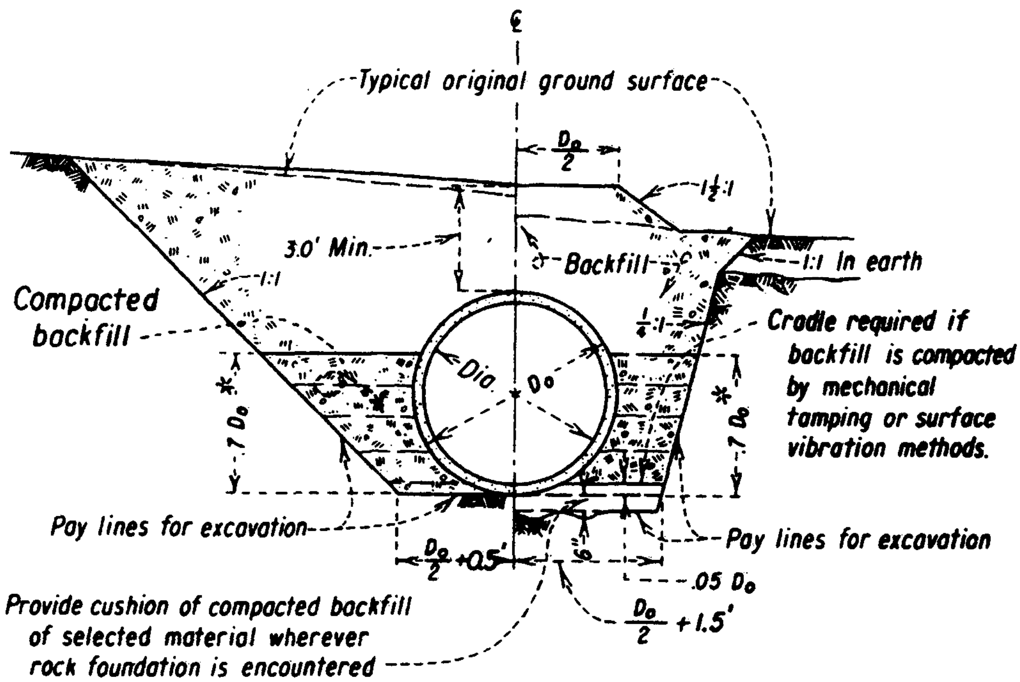
DENVER, COLO. AUGUST 9, 1967 | 103-D-1060

Fig. 25 Par. 2.35 EARTHWORK DETAILS, PRETENSIONED CONCRETE PIPE-- EARTH OR ROCK EXCAVATION



EARTH **ROCK**

NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet and the maximum shall be 5 ft. for Class A pipe, 10 ft. for Class B, 15 ft. for Class C, and 20 ft. for Class D unless otherwise indicated on the structure drawings.

Backfill shall be compacted as shown unless otherwise specified.

*Compact backfill $\frac{3}{8}$ D_o where pipe diameter is 12 inches to 18 inches.

No compaction required for 10" dia. pipe.

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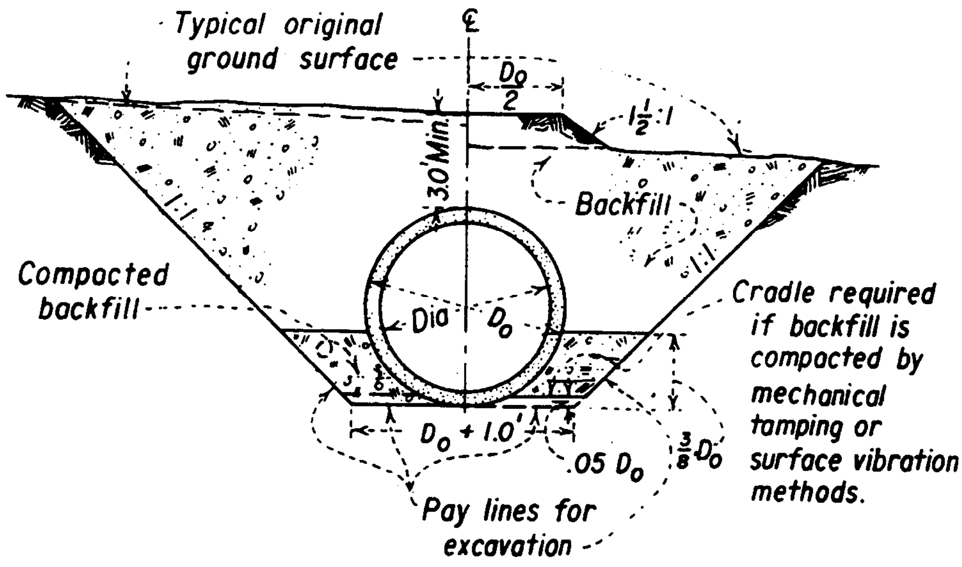
PIPE EARTHWORK DETAILS
PRETENSIONED CONCRETE PIPE
EARTH OR ROCK EXCAVATION

DRAWN A.W.B. SUBMITTED
TRACED T.R.H. RECOMMENDED
CHECKED APPROVED
CHIEF, CANALS BRANCH
DENVER, COLO. AUGUST 9, 1967 | 103-D-1061

EARTHWORK DETAILS, PRECAST CONCRETE OR ASBESTOS-CEMENT PIPE--EARTH EXCAVATION · Fig. 26 Par. 2.35



EARTH

NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet and the maximum shall be 5 ft. for class A pipe, 10 ft. for Class B, 15 ft. for Class C, and 20 ft. for Class D unless otherwise indicated on the structure drawings.

Backfill shall be compacted as shown unless otherwise specified.

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PIPE EARTHWORK DETAILS
PRECAST CONCRETE OR ASBESTOS-CEMENT PIPE
EARTH EXCAVATION

DRAWN A.W.B.	SUBMITTED
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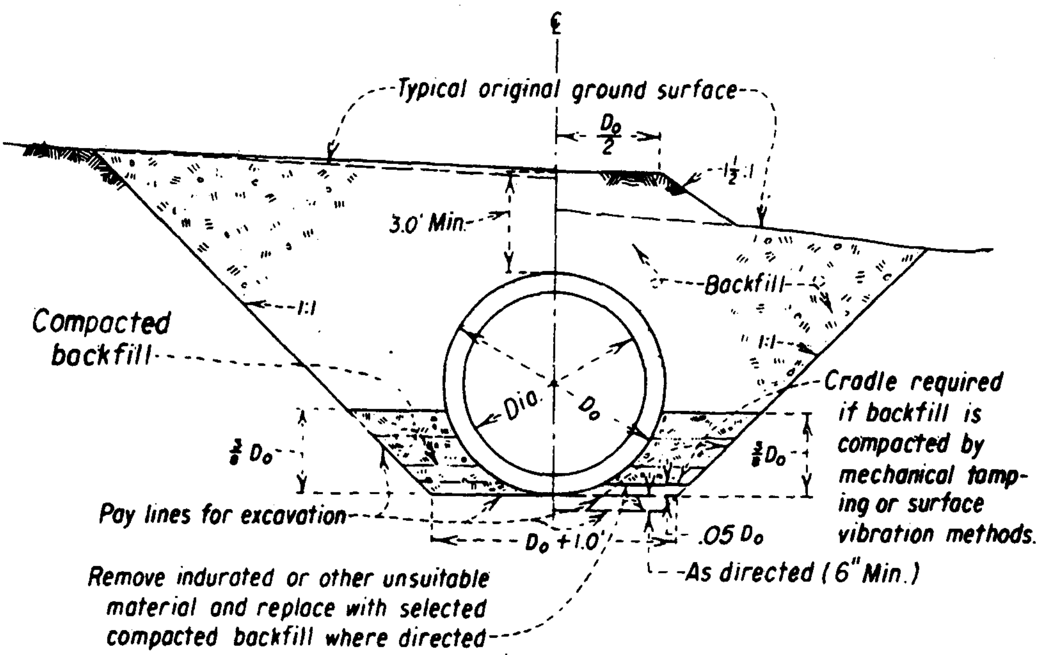
CHECKED	APPROVED CHIEF, CANALS BRANCH
DENVER, COLO. AUGUST 9, 1967	103-D-1056

Fig. 27 Par. 2.35

EARTHWORK DETAILS, PRECAST CONCRETE OR ASBESTOS-CEMENT PIPE--UNCLASSIFIED EXCAVATION



NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet and the maximum shall be 5 ft. for Class A pipe, 10 ft. for Class B, 15 ft. for Class C, and 20 ft. for Class D unless otherwise indicated on the structure drawings.

Backfill shall be compacted as shown unless otherwise specified.

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PIPE EARTHWORK DETAILS
PRECAST CONCRETE OR ASBESTOS-CEMENT PIPE
UNCLASSIFIED EXCAVATION

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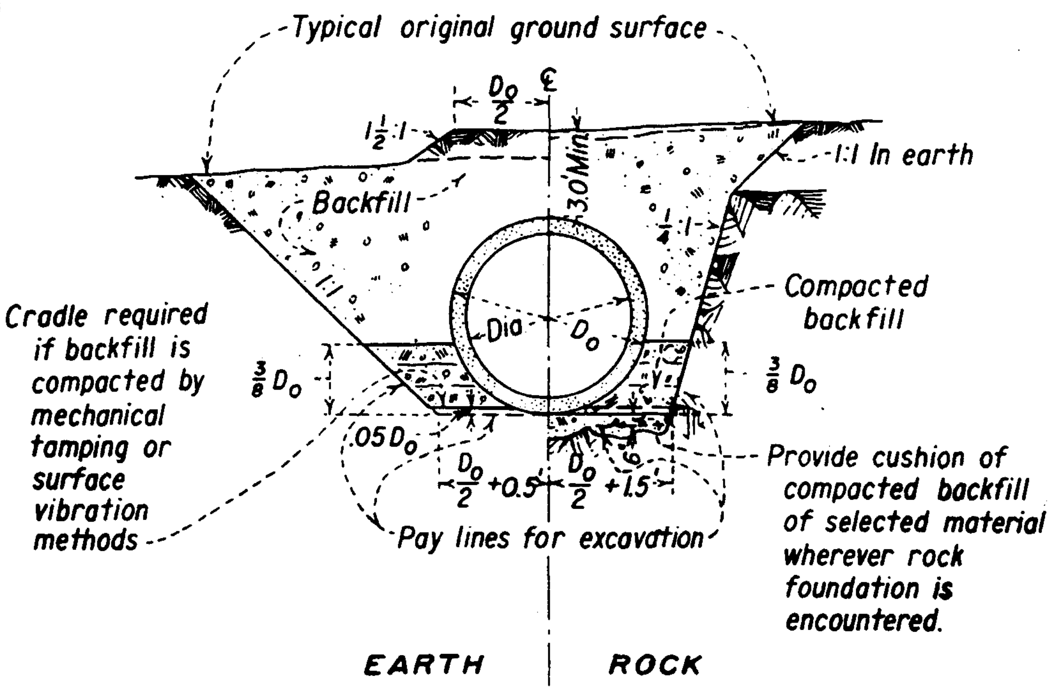
DENVER, COLO. AUGUST 9, 1967 | 103-D-1057

EARTHWORK DETAILS, PRECAST CONCRETE OR ASBESTOS-CEMENT PIPE--EARTH OR ROCK EXCAVATION Fig. 28 Par. 2.35



NOTES

Backfill over pipe shall be to the original ground surface except the minimum shall be 3 feet and the maximum shall be 5 ft. for Class A pipe, 10 ft. for Class B, 15 ft. for Class C, and 20 ft. for Class D unless otherwise indicated on the structure drawings.

Backfill shall be compacted as shown unless otherwise specified.

UNITED STATES
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PIPE EARTHWORK DETAILS
PRECAST CONCRETE OR ASBESTOS-CEMENT PIPE
EARTH OR ROCK EXCAVATION

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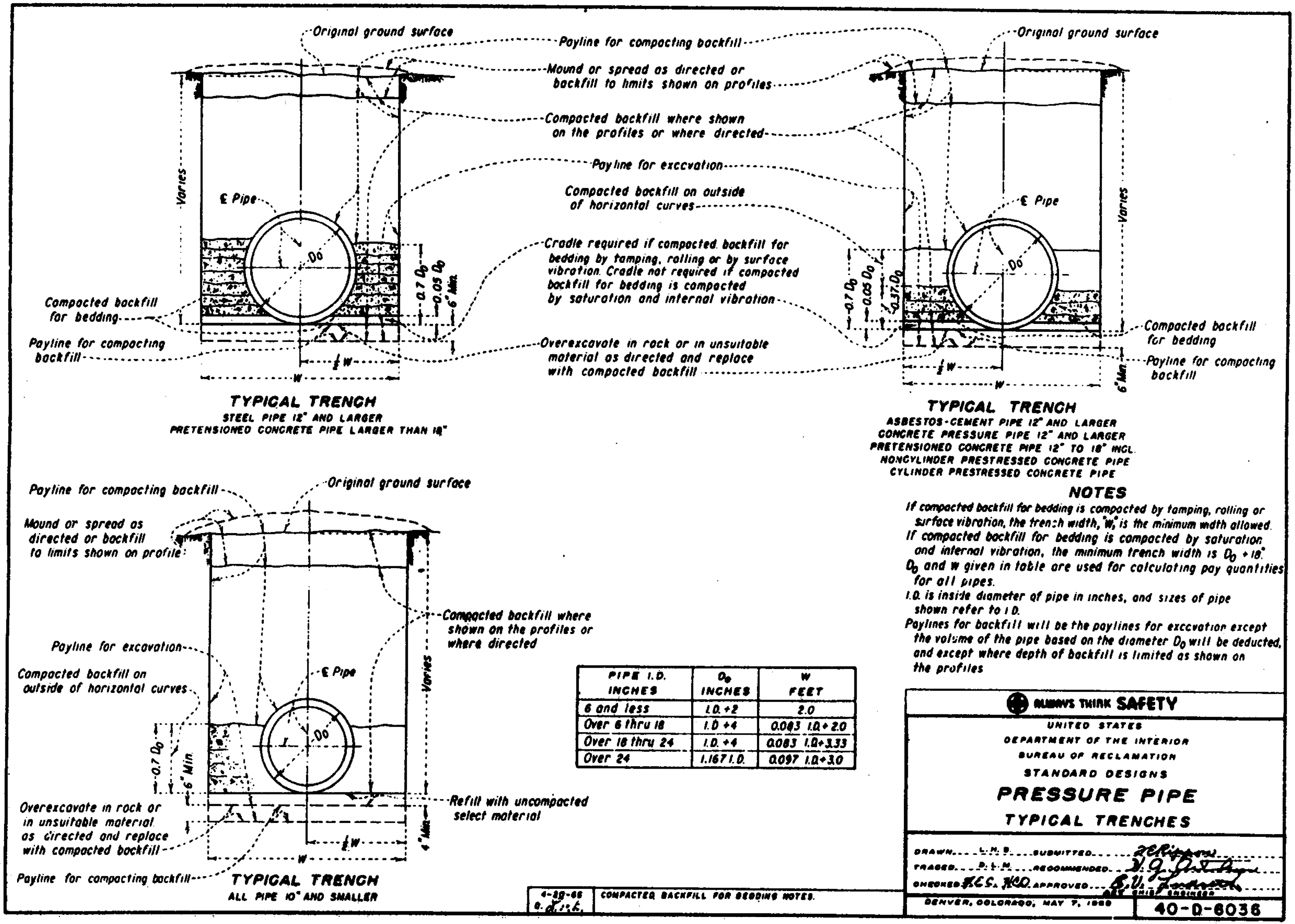
Fig. 29 Par. 2.35

PRESSURE PIPE--TYPICAL TRENCHES



PIPE I.D. INCHES	D_o INCHES	W FEET
6 and less	I.D.+2	2.0
Over 6 thru 18	I.D.+4	0.083 I.D.+2.0
Over 18 thru 24	I.D.+4	0.083 I.D.+3.33
Over 24	1.167 I.D.	0.097 I.D.+3.0

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3.1

GENERAL DESIGN CONSIDERATIONS

.1 To make water flowing in a stream available for irrigation use, it must be diverted by means of a diversion dam and headworks, a diversion headworks alone, or a pumping plant. Elements of a general nature to be considered in the design of diversion dams and headworks are discussed in this chapter. The design of each diversion dam involves the solution of a different set of problems. The objectives of this chapter will therefore be to point out the various features to be considered in the design and to establish broad limits of control. In the structure drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated. INTRODUCTION

.2 The approximate location of a diversion dam is usually established by the requirements of the project plan. These requirements are not considered in this chapter. However, it should be noted that although the general location of a structure is fixed by other than design considerations, the precise location may usually be shifted within certain limits, so that advantage can be taken of natural features of the site to provide for economy of construction. LOCATION

.3 It may be required to place a diversion dam on almost any type of foundation material. Few sites for diversion dams permit construction on a rock foundation, since bedrock is usually at such depth that its use as a foundation is impracticable. Because most diversion dams are founded on pervious or semipervious riverbed materials, only these will be discussed here. Considerations in the design of diversion dams founded on rock are similar to those in the design of gravity dams discussed in Chapter 2, Gravity Dams, of Design Standards No. 2, Concrete Dams. The following items concerning foundations require consideration in the design: FOUNDATION

A. Test borings or test pits are necessary for investigation of the character of the material upon which the dam is to be founded. Test Holes

B. In the design of a dam, the allowable bearing pressure, shearing strength, and sliding factor must be selected either upon the basis of past experience, or from the results of tests of the foundation material in place or tests of samples taken from the test holes or test pits. Design Assumptions

C. Bearing piles are seldom used as supports for diversion dams, although they may be provided where a dam must be built on an unstable foundation material. This type of construction affords very little protection against undermining of the structure and has been a contributing factor in the failure of several dams. Imperial Dam on the Colorado River is an example of a diversion dam built on piles. The design of Imperial Dam is discussed in the Boulder Canyon Project Final Reports, Part IV, Design and Construction, Bulletin 6, Imperial Dam and Desilting Works. Bearing Piles

D. The installation of cutoff walls, aprons, filters, and drains is discussed subsequently in this chapter. The purpose of these features is twofold: to control the amount of seepage under the dam, and to limit the intensity of the uplift so that the stability of the structure will not be threatened. Several factors such as the head on the dam, permeability of the foundation, length of upstream and downstream aprons, depth and tightness of cutoff, and effectiveness of drains enter into consideration of underseepage and uplift. Uplift and Seepage

The magnitude of the uplift forces under the structure and the amount of underseepage for a given coefficient of permeability can be obtained from a flow net. A theoretical discussion of the flow net and tables of representative coefficients of permeability may be found in Engineering Monograph No. 8, entitled "Theory and Problems of Water Percolation," by

3.4

GENERAL DESIGN CONSIDERATIONS--Continued

Uplift and Seepage (Cont'd.)

C. N. Zangar. Most standard texts on hydraulic structures, including "Engineering for Dams, " by Creager, Justin and Hinds, show typical examples of flow nets.

The flow net itself may be constructed either by sketching or by electric tray analogy. For preliminary design it is sufficient to sketch the flow net, provided the foundation material is homogeneous or reasonably close to it. Lack of homogeneity need not rule out sketching the flow net, but the more complex the foundation the less reliable the results since an assumption of homogeneity is basic.

For all final designs and for preliminary designs on complex foundations, a flow net should not be attempted by sketching. Electric analogy is the best method for any foundation, and complex foundations can be simulated by varying the depth of the electrolyte in various regions in accordance with the permeability of the material. The electric analogy is an accurate and inexpensive method of obtaining a flow net which leads to estimating uplift intensities under the dam and quantity of underseepage.

It should be noted that, for all homogeneous foundations (or those assumed homogeneous), the shape of the flow net does not depend on the permeability of the foundation. The permeability coefficient is a factor in the computation for underseepage but not for uplift.

In considering the safety of a concrete dam on a pervious foundation, many engineers have found that Lane's weighted-creep theory is helpful. This theory is fully explained in the ASCE Transactions, Volume 100, 1935, under the title of "Security From Underseepage." Mr. Lane tested his theory by analyzing more than 200 existing dams, both failures and nonfailures. This theory gives another method of computing uplift under the dam, and it enables the designer to judge the safety of the structure by the computation of the weighted-creep-head ratio.

Khosla's method of independent variables, presented in the journal of the Central Board of Irrigation, India, September 1936, is another theory that is useful in estimating the magnitude and distribution of uplift pressures under the dam.

The possibility of piping should always be investigated where a concrete dam is constructed on a pervious foundation. The most critical area is at the downstream toe of the dam or at the bottom of the downstream cutoff. At any depth the submerged weight of the soil above the point in question must exceed the pressure potential in the soil in order to have some margin of safety against piping. Safety against piping may be increased by adding riprap over the questionable area or by providing a filtered weep or drain to relieve the pressure potential, or both.

MAXIMUM FLOOD-FLOW

.4 Diversion dams are designed to safely pass floodflows of 50- or 100-year probable frequency. The selection of the design flood is governed by economic considerations. The responsibilities and procedures for determination of maximum floodflows for diversion dams are given in the Reclamation Instructions, Series 110, Planning, and 130, Design.

FLOOD-FLOW PER FOOT OF CREST

.5 For the design of dams located on a sandy foundation, the flood discharge per linear foot of overflow crest is usually limited because of the cost of protecting against downstream erosion. The discharge per linear foot of dam may also be limited by the allowable upstream water surface elevation.

GENERAL DESIGN CONSIDERATIONS--Continued

.6 The maximum allowable upstream water surface elevation is often dependent on the elevation of railroads, highways, bridges, buildings, and other properties. MAXIMUM UPSTREAM WATER SURFACE ELEVATION

.7 The rating curve of the natural stream below the dam is necessary to permit proper provision in the design for energy dissipation. Information in regard to this downstream water surface elevation may also be necessary to determine the flood discharge in cases of submerged overpour. For a discussion pertaining to effect of submergence on coefficient of discharge, see Hydraulic Laboratory Report No. 182, entitled "Studies of Flow Characteristics, Discharge and Pressures Relative to Submerged Dams," by J. N. Bradley. DOWNSTREAM RATING CURVE

.8 Erosion of the streambed downstream from the dam should be prevented by the proper design for the dissipation of energy of the overpour water and by provision for riprap or rock paving on the banks and bottom of the streambed. A cutoff wall is usually provided at the downstream end of the structure as additional protection against erosion. DOWNSTREAM EROSION

.9 Retrogression of a sandy streambed downstream from a diversion dam should be expected, particularly where an upstream storage reservoir will intercept and retain the natural sand load carried by the stream. In some cases, protection against this lowering of the streambed and the resulting lowering of the tailwater elevation may become very difficult and costly. DOWNSTREAM RETROGRESSION

.10 Where a diversion dam is to be built across a stream meandering down a flat valley floor, there may be danger of the stream shifting to another location during a flood and thus bypassing the dam completely. Considerable riverbank protection may be necessary to guard against this contingency, or a long dike may be required across the valley floor. SHIFTING OF UPSTREAM CHANNEL

.11 In cold climates, ice pressure may become a very important factor in the design of diversion dams. The ice pressure constitutes a relatively larger force on small dams than on large ones. Final selection of the design value should be made after consideration of the importance of the effects of restraint, temperature, and snow conditions. These factors will vary with the individual locality. For a more complete discussion of ice pressures, see "Thrust Exerted by Expanding Ice Sheet," by Edwin Rose, ASCE Transactions, Volume 112, page 871, and "Experimental Investigations by the Bureau of Reclamation," by G. E. Monfore, ASCE Transactions, Volume 119, page 26. ICE PRESSURE

.12 Forces due to earthquakes acting on the structure should be considered in the design of diversion dams. The intensity of the earthquake force to be used in the design will depend on the area where the structure is to be built. Engineering Monograph No. 11, entitled "Hydrodynamic Pressures on Dams Due to Horizontal Earthquake Shock" gives the hydrodynamic pressures on dams due to horizontal earthquake effects. EARTHQUAKE

.13 Contraction joints in diversion dam structures require considerable study. They must be watertight in spite of differential settlement and must provide for contraction and expansion of the concrete to prevent cracking. See Chapter 2, General Design Information for Structures, for further information on contraction joints. CONTRACTION JOINTS

.14 Rubber waterstops have been used successfully for many years to seal contraction joints where movement is probable. Recent specifications permit use of plastic waterstops as an alternative to rubber. WATERSTOPS

GENERAL DESIGN CONSIDERATIONS--Continued

AVAILABILITY OF MATERIALS .15 The economic design of a diversion dam may be considerably influenced by the availability and cost of certain construction materials at the particular site. Such materials are rock for riprap; sand and gravel, or crushed rock for concrete aggregates, roads, blankets, and filters; and earth for compacted fills.

TRANSPORTATION FACILITIES .16 The design of a diversion dam may be influenced by the proximity of railroads and highways which affect the cost of materials to be used.

PROVISION FOR MIGRATORY FISH .17 Some diversion dams are located on streams which support large numbers of migratory and other types of fish, which may be important both commercially and recreationally. Fish screens and fish ladders must be provided in these cases to prevent the entry of fish into the canals and to permit the natural migration of fish to and from their spawning areas.

PROVISION FOR LOGWAY .18 In some cases, it may be necessary to provide logways for the passage of logs being floated to the sawmills.

SPILLWAY SECTION

TYPES OF SPILLWAY SECTIONS .19 In selecting the type of spillway section to be used in the design of a diversion dam for a particular site, the following controlling factors are to be considered: character and strength of foundation; availability of construction material; necessity for a controlled crest and cost. The types of spillway sections most generally used in the design of diversion dams are listed below and discussed in the following paragraphs:

Concrete ogee solid gravity
Concrete dam with control gates on crest
Concrete ogee slab and buttress
Concrete slab on compacted earthfill
Rockfill

Solid Gravity Concrete Overflow Section

A. The usual form of the solid gravity concrete overflow section has a vertical upstream face and a rounded crest with an ogee face downstream. This type is the one most commonly used where no control of the upstream water surface during floodflows is needed; it is most frequently used on a rock foundation. In case the dam is to be built on a sand or gravel foundation, the extent to which this type can be used will depend on the supporting power of the foundation material. Examples of solid gravity diversion dams are shown in Figures 1, 2, 3, and 4.

Concrete Dam with Control Gates on Crest

B. The concrete dam with crest control gates is used where control of the upstream water surface during floodflows is necessary. This control is required where the great value of property and improvements affected by backwater prohibits the use of an uncontrolled crest. The most distinguishing feature of the various gated-crest dams is the type of gates used. The type of gate to be selected depends principally on the conditions under which the gates must operate. Problems involved in selecting the type of gates to be used for a particular dam and a given set of operating conditions require special study. An example of a concrete dam with control gates on the crest is the Putah Diversion Dam shown in Figures 5 and 6.

Slab and Buttress Overflow Section

C. The slab and buttress overflow section is constructed of reinforced concrete and usually consists of a sloping upstream slab and an ogee-shaped downstream face supported on a horizontal foundation slab by equally spaced concrete buttresses. An example of a slab and buttress diversion

SPILLWAY SECTION--Continued

dam is the Imperial Dam on the Colorado River. This type of dam, being much lighter per square foot of area covered than the solid gravity type, is more suitable to foundations with low bearing strength. The difference in first cost between a slab and buttress dam and a solid gravity dam depends on local conditions and the availability of construction material. The slab and buttress type of construction is seldom used for dams of heights less than 20 feet. Slab and Buttress Overflow Section (Cont'd.)

D. The concrete slab on a compacted earthfill type of overflow section may be used economically where the height is 5 feet or less. It consists of reinforced concrete slabs extending upstream and downstream from the crest which rests on a sheet-piling cutoff wall. The downstream slab is constructed on a filter and is weeped to prevent uplift. The Duchesne Diversion Dam constructed across the Duchesne River (Figure 7) is of this type. Concrete Slab on Compacted Earthfill

E. In locations where rock is plentiful and economical to obtain, the overflow section may consist of a rockfill with either a sheet-piling or concrete diaphragm to form the crest and cutoff. The rockfill is placed on top of a filter to prevent the overflowing water from displacing the foundation material. Rock for the rockfill is angular in shape and derrick-placed in such a manner that the rock will resist displacement by a keying or interlocking effect. This interlocking is particularly important in the downstream portion of the section. The minimum size of rock and the slopes are dependent on the amount of overpour per linear foot of crest and the height of dam. Ordinarily, the downstream slope is flatter and the rock is of larger size than in the upstream portion of the dam because of the greater likelihood of displacement. This type of dam is often econômical to construct; but it is limited in height to 5 feet maximum between the upstream and downstream water surfaces and to a capacity of about 30 cfs per foot of width, because of questionable stability against very high velocity flows over the downstream rockfill. Additional percolation path to that furnished by the diaphragm is sometimes provided by an impervious blanket extending upstream under the gravel filter and rockfill. This blanket should be carefully placed and well protected, and should be thick enough to eliminate any danger of a fracture. The rockfill design is not recommended for use on easily erodible foundations. An example of a rockfill design is Robles Diversion Dam shown in Figures 8 and 9. Rockfill Overflow Section

.20 The purpose of a diversion weir is to raise or control the water surface in the river so that the desired flow may be diverted into the canal and yet function properly as a spillway for floodflows. The water surface elevation in the pool formed by the diversion dam is determined by the head required for the canal when flowing at full capacity. When the entire flow of the river at low flows may be diverted, the uncontrolled crest or the top of the gates on a gated crest usually has about 0.5-foot freeboard above the water surface required to supply the canal. This freeboard is necessary to prevent water from being wasted over the dam by wave action. When the minimum flow in the river exceeds diversion requirements, it is necessary only that the minimum low water level of the river upstream of the dam be high enough above the canal water surface to provide the necessary head for the canal. The elevation of a gated crest is usually determined from an economic study of various size gates and by the maximum allowable upstream water surface during floods. ELEVATION OF CREST

.21 There are many factors, depending on the physical features of a given site, to be considered in determining the length of the overflow weir. Because of the many factors involved, no fixed rule can be made for determining LENGTH OF CREST

3.22

SPILLWAY SECTION--Continued

LENGTH OF CREST (Cont'd.)

the crest length. One important factor influencing the crest length of the overflow weir is cost.

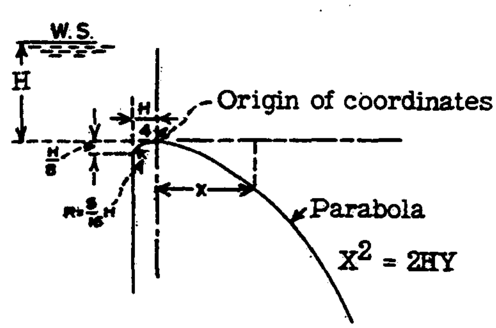
The most economical crest length for a given flood can be determined by comparing the cost of weirs of various lengths with the cost of other features affected by the crest length. Some of the more important features to be considered in making these cost comparisons are the following: cost of relocating public utilities that will be affected by backwater, such as roads, railroads, telegraph, and telephone lines, pipelines, etc.; cost of stilling pool and other protection downstream; cost of nonoverflow section and abutments; cost of headworks and sluiceway; and cost of right-of-way. The economical crest length will be the one giving the minimum combined cost. It may not always be practical to use the economical crest length because of other controlling features, but these cost studies can be used as a guide in selecting the proper crest length. Features other than cost that are to be considered in selecting the crest length are foundation conditions and general shape of the river channel. For example, in a gravel-bedded stream on relatively steep gradient, an examination of the topography at the diversion site and for a distance upstream and downstream will usually give an indication of the stable width of the stream: a length of crest approximating this width should be satisfactory. This criterion will not apply to a meandering low-flow channel in a wide flood plain.

CREST SHAPE

.22 The shape of the crest will depend on the type of spillway section used.

Solid Gravity Concrete Overflow Section

A. The crest of a solid gravity concrete overflow section should approximate the shape of the lower nappe of an overfalling stream over a sharp-crested weir. A simple scheme suitable for most low diversion dams with a vertical upstream face is to shape the upstream to an arc of a circle and the downstream surface to a parabola. The parabola should extend down to its point of tangency with the downstream slope of the dam. The necessary information for defining the shape is shown below.



When a better coefficient of discharge is required or when the head on the crest is high, Engineering Monograph No. 9, entitled "Discharge Coefficients for Irregular Overfall Spillways," should be used to set the shape of the crest.

Concrete Dam with Control Gates on Crest

B. Operational procedure dictates the shape of the crest for a concrete dam with control gates to some extent. If the gates are to be operated at small openings for any extended length of time, then, in order to prevent the jet from springing free or making and breaking, this shape should approximate a parabola which is defined by the equation $X^2 = 4\ HY$. However, if the gates when operated are usually wide open, a better coefficient of discharge for the larger flows can be obtained by use of Engineering Monograph No. 9.

SPILLWAY SECTION--Continued

C. The downstream surface of the crest for a slab and buttress overflow section should be the same as for the solid gravity section if ungated, or the same as for a concrete dam with control gates if gated. The upstream surface usually has a radius at the crest, but the upstream face itself is usually sloping instead of vertical as in the previous cases. Slab and Buttress Overflow Section

D. The crest for a concrete slab on compacted earthfill is a broad-crested weir with flat slopes either way from the crest. Concrete Slab on Compacted Earthfill

E. The crest for a rockfill overflow section is usually a cutoff wall of concrete, wood or steel sheet piling with a flat rock slope each way from the cutoff wall. The upstream slope may be somewhat steeper than the downstream slope because of the water action. Rockfill Overflow Section

.23 Foundation drains for diversion dams generally consist of a main filter located under the downstream toe of the dam. Outlets from the filter drain consist of pipes extending from the filter to the top surface of the apron or to the downstream face of the dam. It is preferable for these outlet pipes to be sloped in a downstream direction so the water flowing over the dam will produce an ejector effect to increase the action of the filter drain. If conditions are such that a deep stilling pool is required where silt-laden water is expected to stand for extended periods of time, the outlet pipes may become clogged with sediment. In this case consideration should be given to placing flap valves on the outlet ends of the pipes, or some other measures should be provided to prevent clogging. FOUNDATION DRAINS

A. The location of the filter is a matter of cost. If the filter is located near the downstream toe of the overflow section, additional upstream apron and cutoff wall will be required to furnish the necessary path of percolation. On the other hand, if the filter is located at the downstream edge of the apron, the apron can be considered as contributing to the percolation path and the length of upstream apron can be reduced. However, with the filter in the latter location, the amount of uplift would be increased, thus requiring more concrete in the dam and apron for stability. Generally, a filter near the downstream toe of the overflow section of the dam will be at the most economical location. Location of Filter

B. The gradation of the foundation material determines the type of filter to be used. The two types of filters commonly used are zoned and graded filters. The criteria for designing protective filters is described in Earth Material Laboratory Report No. EM-425. Design of Filters

.24 Protection of the streambed downstream from the overflow weir is furnished by a downstream apron or an upturned bucket, both supplemented with heavy riprap or articulated concrete blocks. The upturned bucket should be used only in streambeds composed of boulders and coarse gravel, or other material not easily eroded. The riprap or articulated block protection should be placed on a blanket of gravel or rock spalls to prevent the streambed material from being drawn up through the interstices in the protection. In the case of fine streambed material, it may be necessary to construct this blanket as a filter using a layer of sand as the lower course with a layer of gravel on top. STREAMBED PROTECTION

A. One of the most effective means of dissipating the kinetic energy of the water discharging over a weir is by means of a hydraulic jump on a concrete apron. The use of baffle blocks can improve the jump and reduce Hydraulic Jump

3.24B

SPILLWAY SECTION--Continued

Hydraulic Jump (Cont'd.)

the required tailwater depth and length of jump. Engineering Monograph No. 25 gives design criteria for stilling basins and the arrangement of baffle blocks. Unless a model study shows otherwise, the minimum length of apron should be four times the depth of the hydraulic jump, or $4d_2$.

On streams where retrogression can be expected, it is very important to make an estimate of its amount so that the tailwater rating curve can be adjusted to represent future conditions. Otherwise, construction which provides a perfect jump with present tailwater elevations will become worthless because of the future lowering of the tailwater due to retrogression of the river channel.

Upturned Bucket

B. Where high tailwater is available, an upturned bucket may be used for energy dissipation and may be more economical than a concrete apron because of the smaller amount of concrete needed for construction. The upturned bucket should not be used where riverbed material will erode easily and where there is a possibility that degradation of the streambed will lower the tailwater below the design minimum. Engineering Monograph No. 25 gives the design criteria for setting the bucket elevation and dimensions. Woodston Diversion Dam (see Figure 2) is an example of a diversion dam with an upturned bucket to dissipate the energy.

UPSTREAM APRON

.25 An upstream concrete apron has been found very advantageous for diversion dams on pervious sand or gravel foundations. In addition to providing an increased length of path of percolation, the apron provides space for the several rows of sheet piling sometimes required. The upstream apron, or a part thereof, is often tied by steel reinforcement to the dam, thus developing a substantial increase in the sliding resistance of the whole structure. Still another purpose of the upstream apron is to prevent erosion adjacent to the dam during periods of floodflow. A minimum length of approximately three times the height of the overflow section will give good proportions.

Cutoff Walls

A. An upstream cutoff wall or walls under the apron usually furnish a large portion of the required path of percolation under a diversion dam. These cutoff walls may be formed by steel, timber, or concrete sheet piling; by mixed-in-place cement-bound curtains; or by concrete walls. The cutoffs should extend to impervious material wherever practicable. If foundations contain large boulders which would interfere with the driving of sheet piling, trenches for the cutoffs can be excavated. These trenches should be excavated on slopes such that shoring of the excavation will not be required. The cutoff can be constructed in the trench backfilled with impervious material, or if sheet piling is used the trench can be backfilled first and the sheet piling driven into the backfill. Several factors will determine the type of cutoff wall used. Among these factors are the initial cost of construction, cost of maintenance, water tightness desired, type of foundation material, etc. Cutoffs should be spaced no closer than about three times the depth of the deepest cutoff.

Riprap Protection

B. Often it is necessary to furnish some protection against erosion of the streambed and banks upstream from the apron and wingwalls. Riprap is the most commonly used material for this purpose. Since the maximum velocities upstream from the dam are lower than the original river velocities before the dam was built, the riprap usually need not be very heavy. In fact, a coarse gravel blanket will often serve in lieu of riprap.

ABUTMENT SECTION

.26 Where it is not economical or desirable from design considerations to extend the spillway section across the entire river channel, the length which is not closed by a spillway section is closed by a nonoverflow or abutment section. Abutment sections are of concrete tied into the river banks or into earth dikes. GENERAL

A. Concrete abutment sections are designed as low concrete dams. Protection at the banks is provided by cutoff walls and wingwalls. Concrete

B. Earth dikes are designed on the same principles as low earth dams. The top width should be sufficient for the travel of trucks or cars for inspection, maintenance, and access to the spillway section and headworks. The upstream slopes must be protected against erosion, wave action, and the destructive work of rodents. This protection can be provided by riprap, grass, willows, or brush mats. The downstream slope is not usually protected unless the floodwater rises to considerable height on the embankment. Earth Dikes

.27 Freeboard must be provided to prevent overtopping by wave action and for an additional safety factor. The minimum freeboard should be 2 feet. FREEBOARD

.28 Wingwalls are provided to join the spillway section or the headworks to the earth dike or banks. They prevent erosion of the banks and provide resistance to percolation around the end of the spillway section or headworks. They can be vertical, warped, or sloped, depending on the general design and hydraulic requirements. The walls are designed for the earth and waterloads to which they will be subjected. WINGWALLS

.29 Handrailing is provided to prevent injury to the public or operators. The three common types of railing are pipe, concrete, and wood. Additional information concerning safety features of diversion dams, including handrailing, may be found in Chapter 3, Safety Design Standards, of Design Standards No. 1. HANDRAILING

SLUICEWAYS

.30 Sluiceways are required in the majority of diversion dams to reduce the amount of stream bedload entering the headworks and to assist in maintaining a channel to the headworks. For these reasons, it is desirable to place the sluiceway as near the headworks as practicable. The sluiceway should be placed normal to the axis of the spillway so as to discharge its flow in a direction parallel with the flow over the spillway. The angle of the headworks with the sluiceway may vary from almost 0 to 90°. The amount of this angle can best be determined for model studies, unless otherwise limited by local conditions. The angle of the headworks will be discussed further in the following chapter. LOCATION WITH RESPECT TO HEADWORKS

.31 The capacity depends on the amount of stream bedload to be sluiced from in front of the headworks, and should be large enough to maintain a channel to the headworks. Often, an arbitrary capacity of at least twice the capacity of the headworks is chosen provided the amount of water available for sluicing is not limited. The minimum required capacity for sluicing can best be determined by model studies. If trees and logs as floating debris are expected, the minimum width of the sluiceway should be sufficient to accommodate them without danger of clogging. CAPACITY AND SIZE

.32 The sill is usually placed approximately at the level of the bed of the river and should be at least 1 foot and preferably 3 to 4 feet below the sill of the headworks. SILL ELEVATION

SLUICEWAYS--Continued

DOWN-STREAM PROTECTION .33 Because the head on the sluiceway is greater than the head on the overflow spillway, the concentration of flow per foot of width of sluiceway will be greater than that per linear foot of spillway. Also, for the condition where the sluiceway alone is discharging, that is, no water flowing over the spillway crest, the tailwater will be at a comparatively low elevation. For these reasons the sluiceway stilling pool must be placed at a lower elevation than the stilling pool for the spillway, and additional downstream protection should be provided to accommodate the hydraulic jump. The rules for the design of the sluiceway stilling pool are the same as those for the spillway pool.

GATES .34 Either radial gates or slide gates are used to control the sluiceway opening, but in general, open-type radial gates are preferable to pass floating trees and logs. It is not usually necessary to make the height of the open-type gates sufficient to extend from the sluiceway sill to maximum water surface. That is because the most opportune time to sluice is when there is maximum flow in the stream, and thus the sluiceway gate should be in a partially, if not completely, open position. The amount of gate opening is dependent only on the safe concentration of flow per foot of width of the sluiceway. The sluiceway gates can be either hand- or power-operated. Power-operated gates are preferable, particularly if the gate area exceeds 100 square feet. Power-operated gates are also desirable to allow automatic operation on streams where there is appreciable fluctuation in flow. Stoplog-controlled sluiceways are not recommended because of the inflexibility of operation.

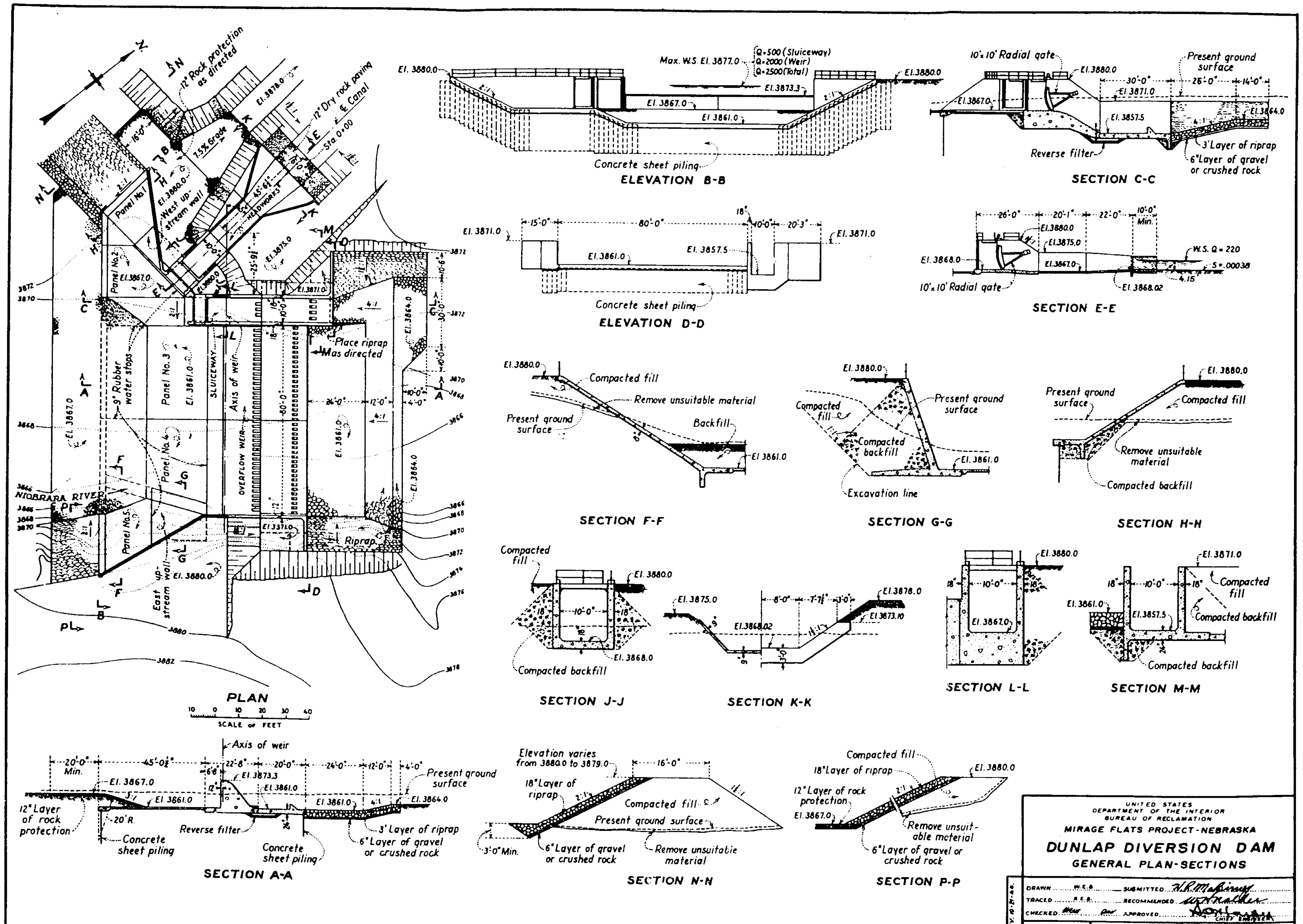
STOPLOG GROOVES .35 Stoplog grooves should be provided for repair and closure in case of damage to the gates.

DESILTING WORKS

GENERAL CONSIDERATIONS .36 As before stated, the sluiceway is primarily used to protect the headworks from an accumulation of stream bedload. The sluiceway is of little or no value in protection against suspended silt load; hence some other means of protection must be employed for suspended silt removal. This protection usually consists of some form of settling basin placed in the canal just downstream from the headworks. The basin should be of such size that the velocity of the water flowing through it into the regular canal section will be low enough to allow the silt to settle in the basin and only the clear water to enter the canal. This velocity will depend on the size of silt particles in suspension, and the length of the basin will depend on the depth and expected rate of settlement. The accumulation of silt in the basin may be removed either by sluicing or by mechanical means.

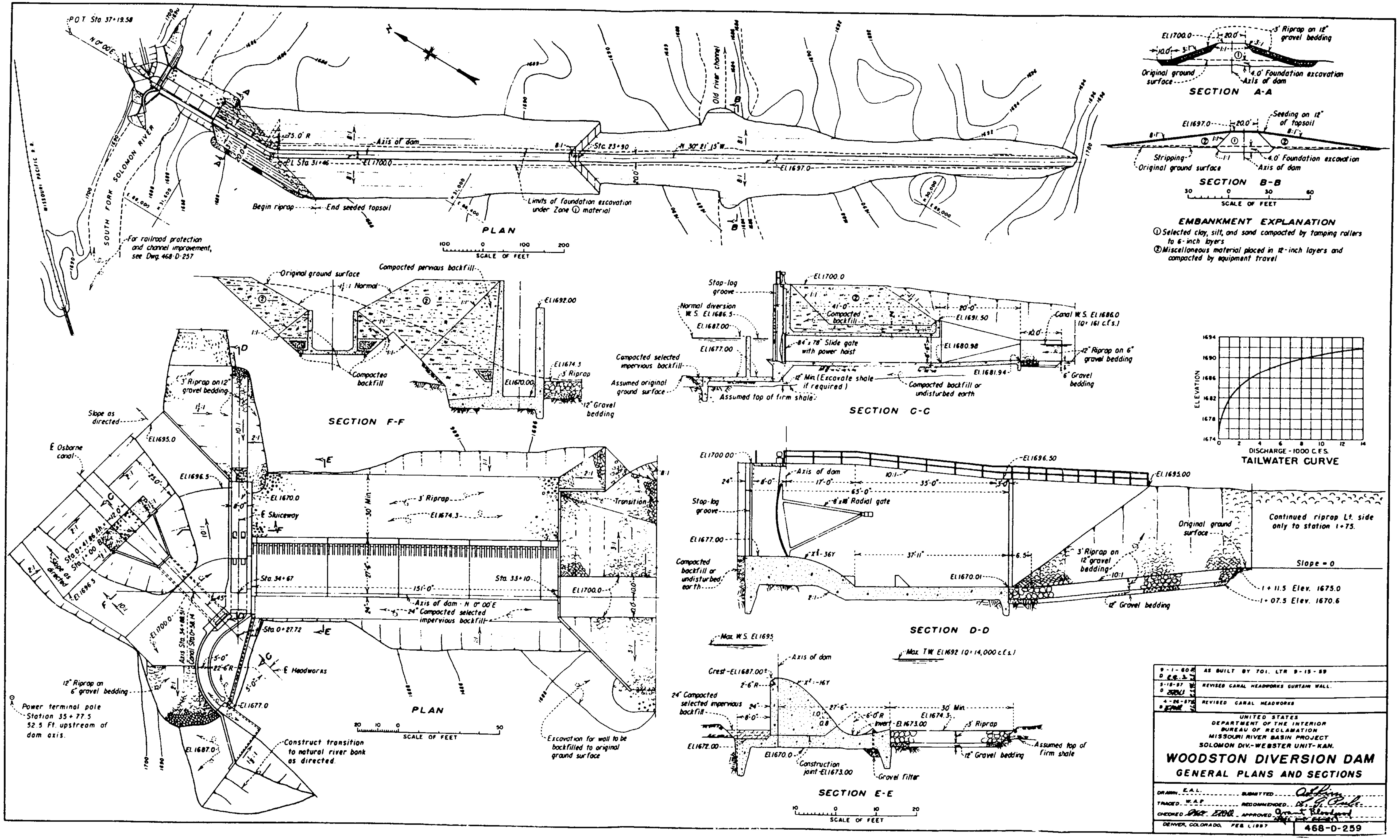
DUNLAP DIVERSION DAM--GENERAL PLAN-SECTIONS

Fig. 1 Par. 3.19A



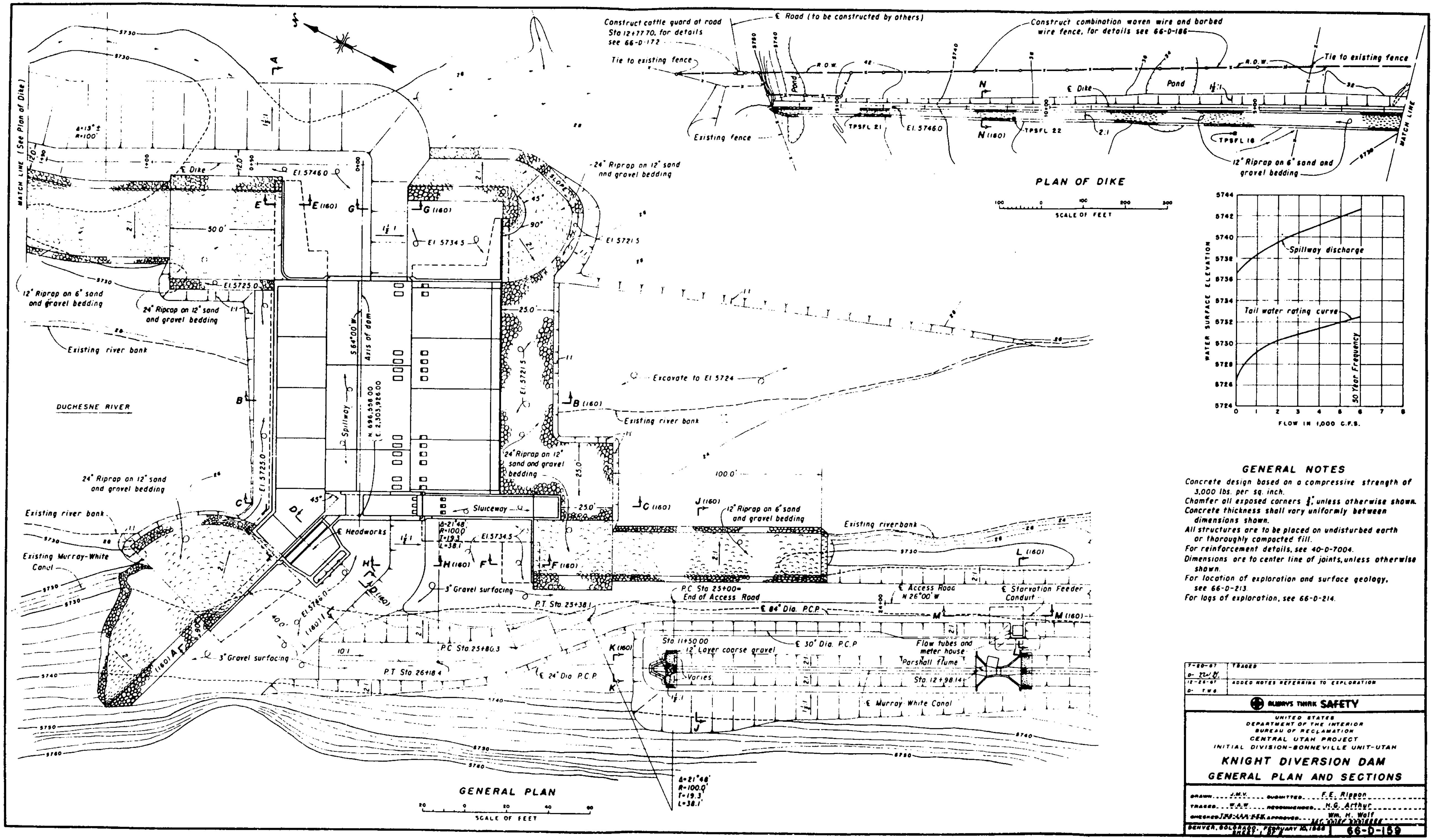
WOODSTON DIVERSION DAM--GENERAL PLAN AND SECTIONS

Fig. 2 Par. 3.19A



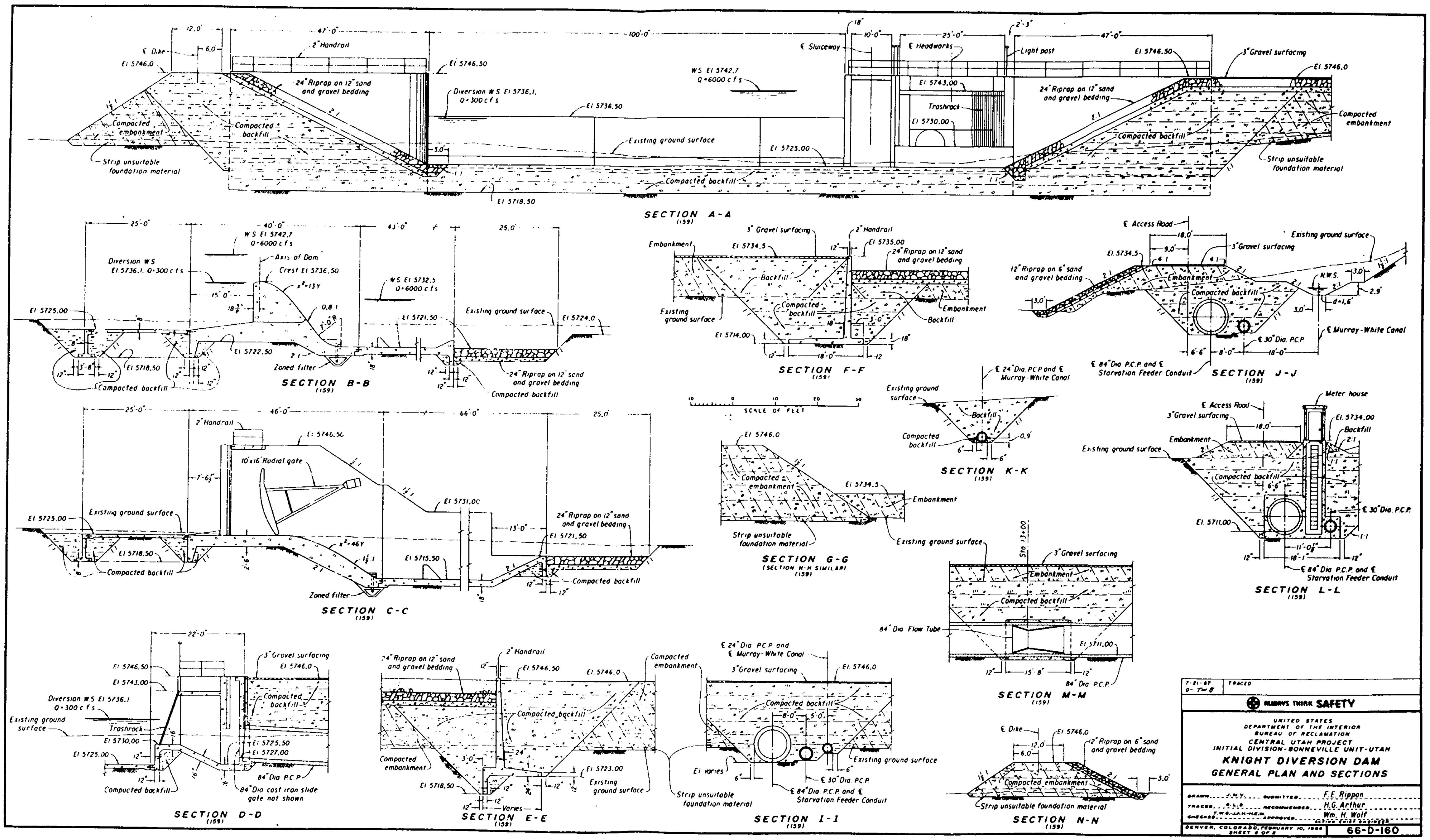
KNIGHT DIVERSION DAM--GENERAL PLAN AND SECTIONS

Fig. 3 Par. 3.19A
Sheet 1 of 2



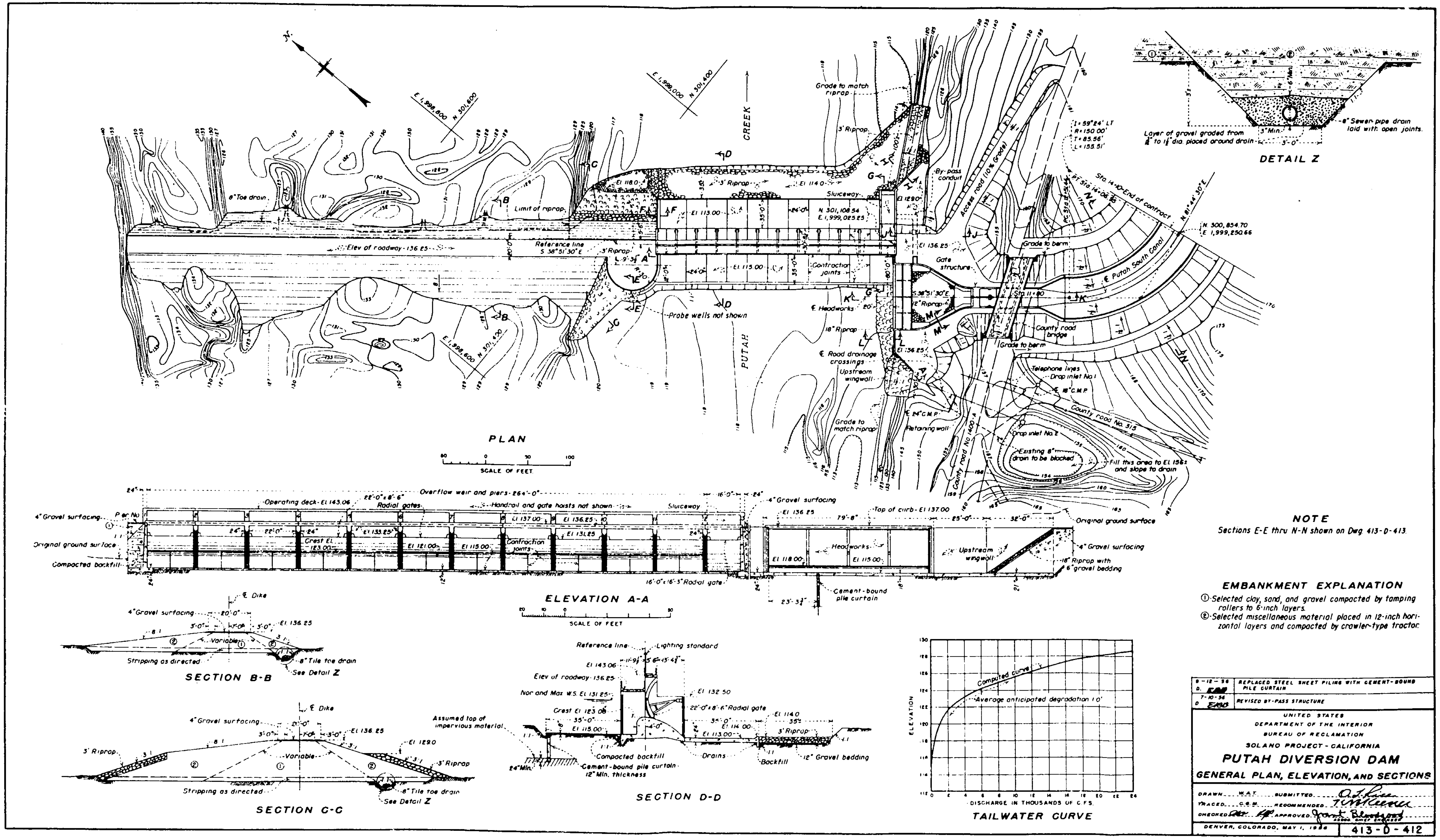
KNIGHT DIVERSION DAM--GENERAL PLAN AND SECTIONS

Fig. 4 Par. 3.19A
Sheet 2 of 2



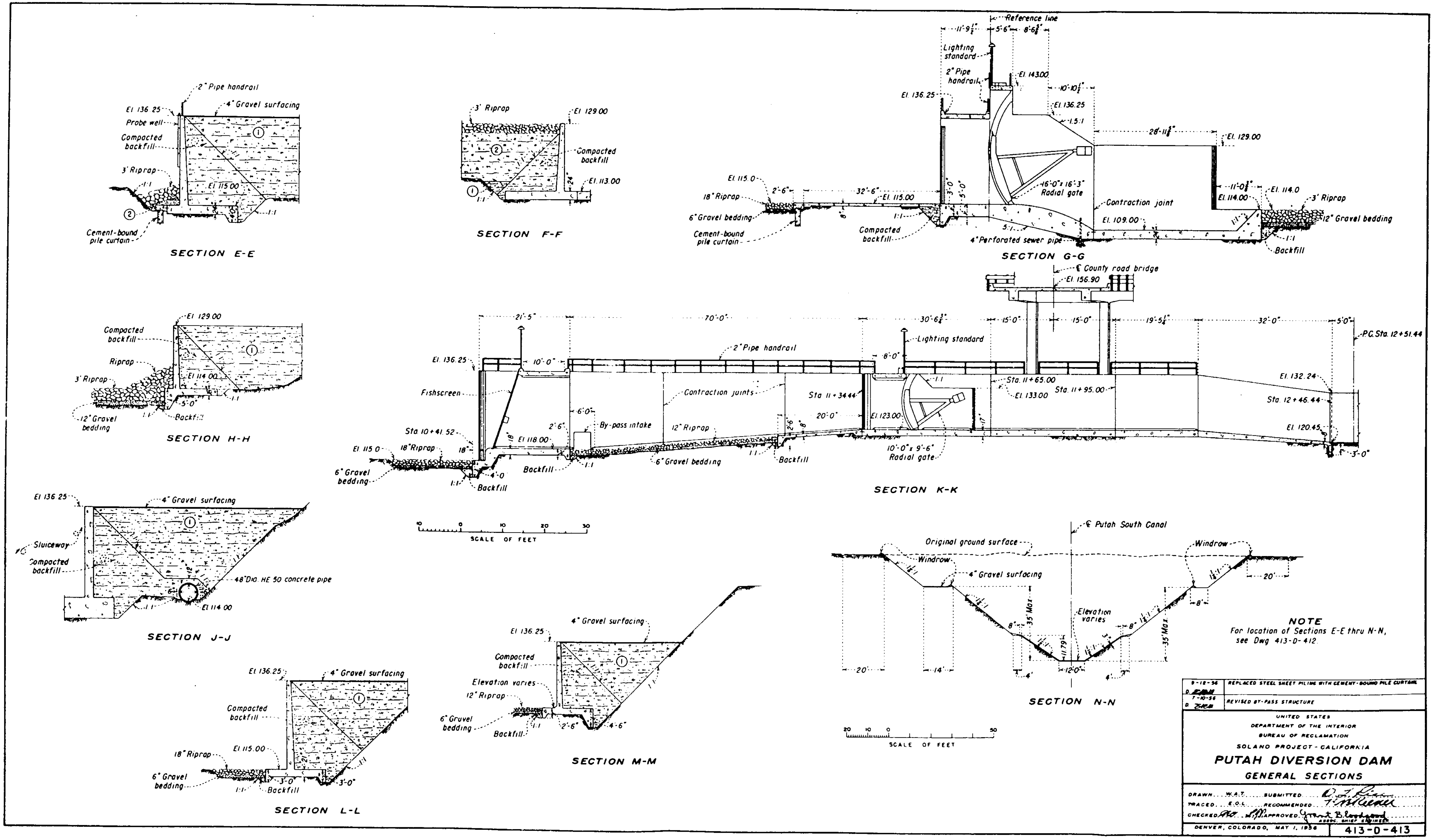
PUTAH DIVERSION DAM--GENERAL PLAN, ELEVATION, AND SECTIONS

Fig. 5 Par. 3.19B



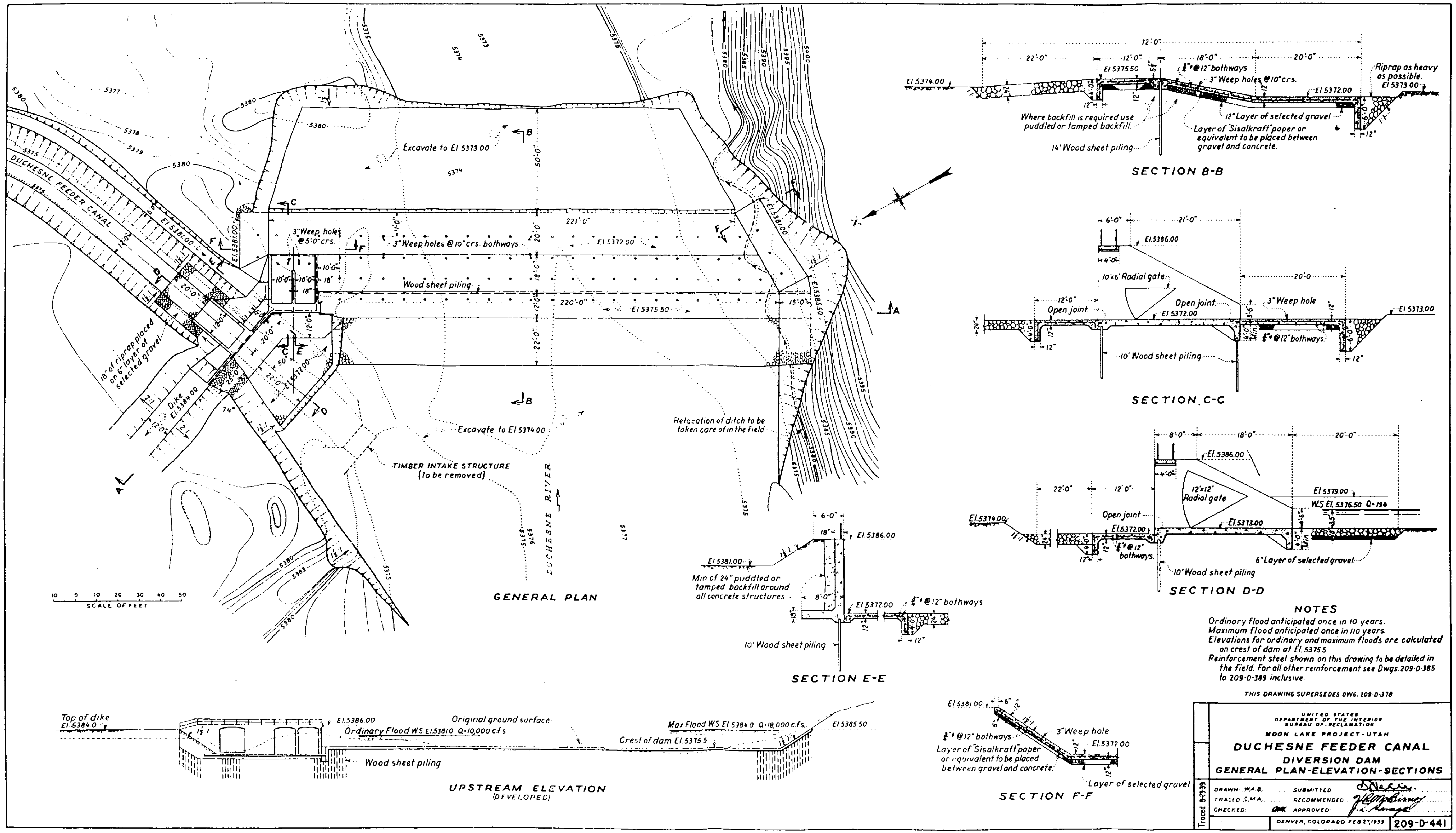
PUTAH DIVERSION DAM--GENERAL SECTIONS

Fig. 6 Par. 3.19B



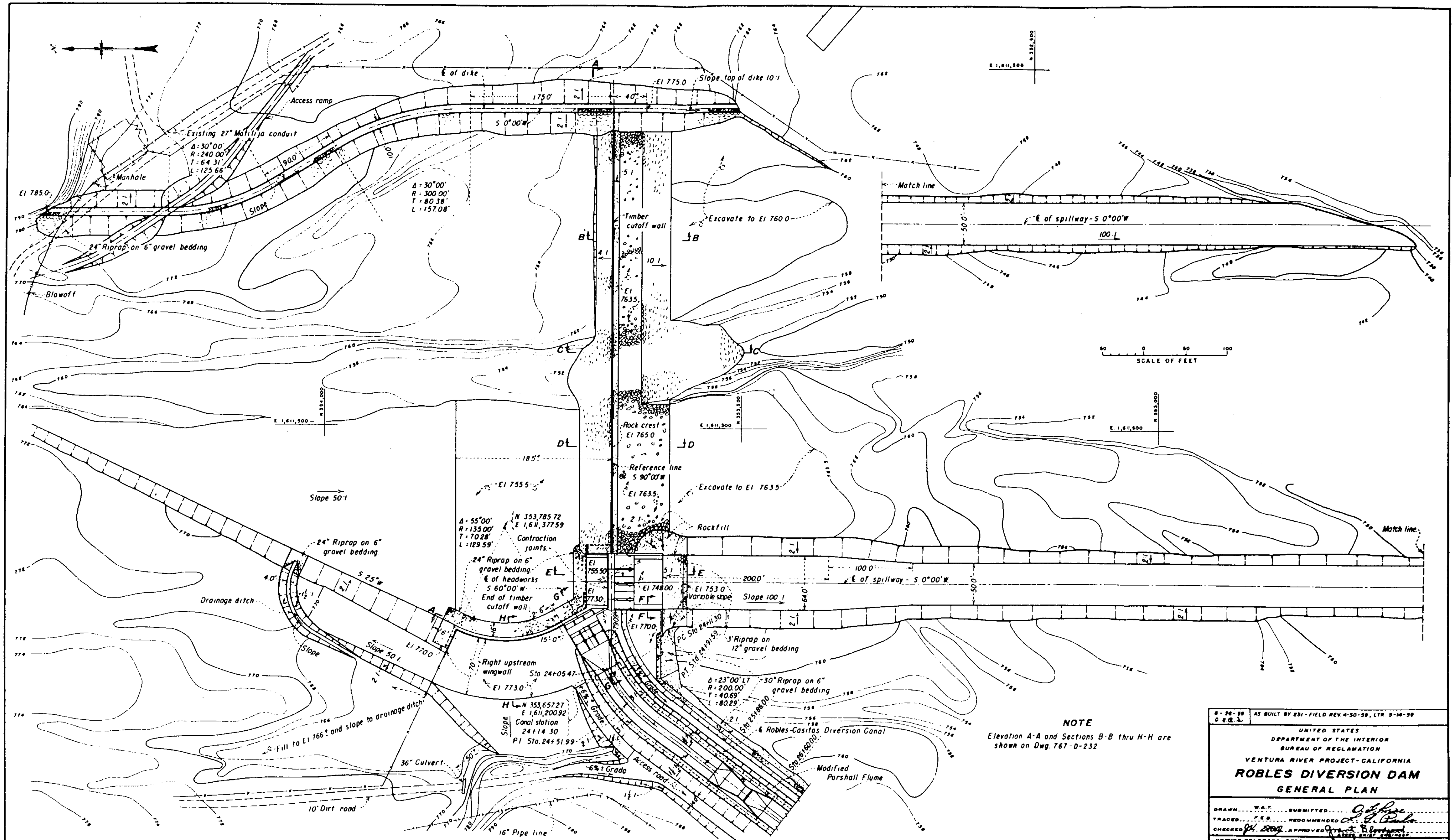
DUCHESNE FEEDER CANAL DIVERSION DAM--GENERAL PLAN, ELEVATION, SECTIONS

Fig. 7 Par. 3.19D



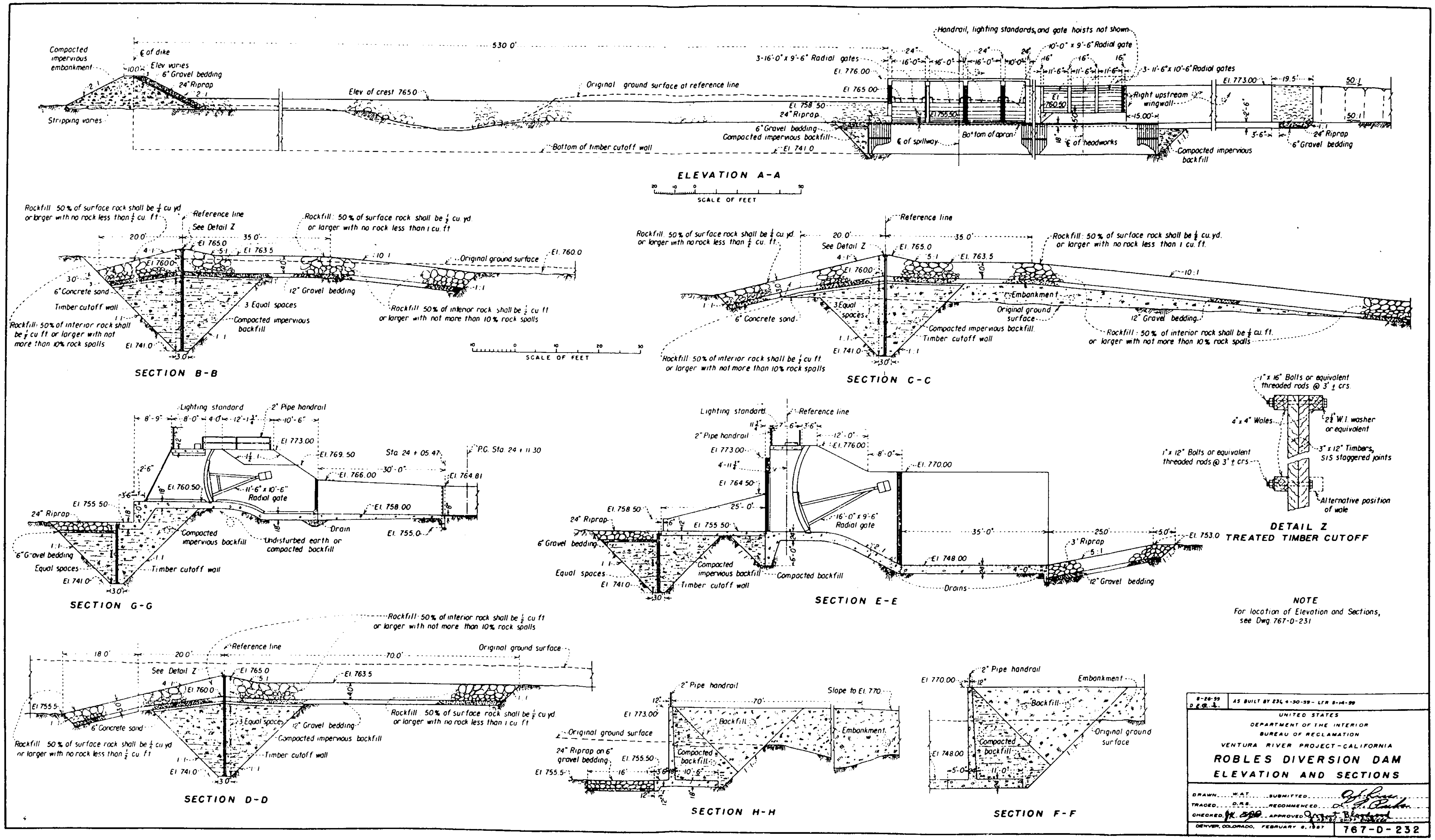
ROBLES DIVERSION DAM--GENERAL PLAN

Fig. 8 Par. 3.19E



ROBLES DIVERSION DAM--ELEVATION AND SECTIONS

Fig. 9 Par. 3.19E



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GENERAL DESIGN CONSIDERATIONS

.1 In this chapter the term "canal headworks" is used to describe those works employed to divert water into a main canal from lakes, equalizing reservoirs, or natural streams. Structures and mechanisms designed to control the diversion of water from large storage dams into main canals are treated as outlet works and described in the Bureau publication "Design of Small Dams." General structural and hydraulic design criteria relating to diversion headworks are given in Chapter 2. In the structural drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated. DEFINITION

.2 The purpose of the headworks structure is to control and regulate the flow into the diversion canal. As a regulating and controlling structure, safety in design and reliability of operation are important. Failure of this structure to operate would mean considerable damage to the canal system and to valuable land below the canal and also possible crop losses. These two factors--safety in design and reliability of operation--should be kept in mind and considerable thought given to them in preparing a design for a headworks structure. FUNCTION

.3 The canal headworks will usually be located along the main bank of a stream where the source of water is to be obtained, and at a point which in general is determined by the position of the diversion canal. It may, however, be located on a branch channel, or slough, or at the edge of a reservoir or lake. LOCATION

.4 When the canal intake or headworks is located on a river, a diversion dam is usually required in order to provide the necessary water surface elevation for diversion at times of low river flow. The location to be chosen should be favorable to the construction of both the diversion dam and diversion canal. The possible sites for a diversion works will be confined to a limited stretch of the stream which will be determined in general by the position of the canal. The exact location within this stretch of the stream is to be determined after considering carefully the geological, the structural, and the economic advantages of each possible site. RELATION TO OTHER STRUCTURES

.5 Where possible, the diversion works should be located on the outside of the bend or the concave bank of the river. In the case where a canal is to be constructed on each side of the river, the diversion dam and headworks structures should be located on a straight portion of the river. It is also preferable to locate the dam and headworks at a point along the river where the position of the canal will be above high tailwater. Such a location will require a higher dam, but will avoid the need for expensive protection work along the canal to protect it against damage from floodflows in the river. EFFECT OF STREAM CURVATURE

.6 In most cases the point of diversion will be located in a valley on the lower reaches of a stream where a pervious foundation of sand, gravel, or silt is usually encountered. In this instance, considerations such as favorable intake location and surface topography rather than geology will determine the site to be adopted. Therefore, the engineer concerned with the design of a diversion dam and headworks should review carefully the possible sites before any extensive foundation exploration work is undertaken. FOUNDATION CONDITIONS

CONTROL STRUCTURE TYPES

.7 The form of the control structure for the headworks depends on the type of gates used, which are the most distinguishing feature of this structure. The type of gates to be selected depends largely on the flow conditions of the river under which they are to operate and on the amount of water to be diverted. FORM OF STRUCTURE

4.8

CONTROL STRUCTURE TYPES--Continued

RADIAL GATE OPEN TYPE .8 Where the river water surface is fairly stable, that is, where there is little fluctuation of the water surface, the radial gate is generally used. For this type of gate installation, the control structure takes the shape of a rectangular flume subdivided into a number of bays by piers which are surmounted by an operating deck. On some of the larger installations where an operating road is involved, a bridge deck is provided in addition to the operating deck. This type of control structure is referred to as the open type and is generally preferred because of its accessibility for repairing the gates and for the removal of debris and drift. The control structure at Roza Diversion Dam (Figure 1) is an example of this type.

RADIAL GATE SUBMERGED TYPE .9 Another form of the open-type control structure is the one used where there is an extreme fluctuation in the upstream water surface. This structure is essentially the same as the one described above except that submerged gates are employed. The vertical-lift gates or the top-sealed gates are the customary submerged types. To close the remaining portion of the structure that is not closed by the gates, curtain or panel walls are used. These walls are designed as reinforced concrete slabs supported against the piers or buttresses. For an example of this type of structure see the Gila Canal headworks, Figure 2.

CLOSED TYPE .10 At reservoirs where the intake structure is constructed through an earth embankment or a small earth dam, the so-called closed type of structure is used. This type is essentially a buried conduit consisting of a single- or multiple-barrel reinforced concrete section which extends through the embankment. The flow through the structure is controlled by top-sealed radial gates or vertical-lift gates which are located near either the center of the embankment or the upstream end of the conduit. To avoid having the conduit under full reservoir pressure, a condition which would increase the danger from leakage if cracks should develop in the walls of the conduit, it is desirable to have the control gates located near the intake to the structure. The disadvantage of this location is that an access bridge would be required from the top of the embankment to the operating deck of the control structure. Trashracks should always be provided at the intake to prevent trees and other drift from getting lodged in the closed conduit.

CATCH BASIN TYPE .11 In mountainous areas where light bedloads may be expected, a special type of submerged intake structure is sometimes used. This type of headworks consists of a catch basin constructed across the riverbed and covered with closely spaced steel trashbars. An essential feature of this structure is an effective sand sluice built at the bottom of the catch basin to remove sand and gravel deposits.

SLUICE AND GATE REQUIREMENTS

STRUCTURAL RELATIONS .12 Where a diversion dam is required the headworks is constructed adjacent to the sluiceway with its axis making an acute angle with the axis of the sluiceway. There are no fixed rules for determining this angle or the elevation of the gate sill. To determine the position of the headworks with respect to the sluiceway, the elevation of the gate sill, and the ~~best~~ arrangement for the upstream wingwalls, model studies are employed. On small structures where the cost of model studies may not be justified, the headworks gate sill should be at least 1 foot and preferably 3 to 4 feet above the elevation of the sluiceway sill which, in turn, should be located at or above the elevation of the riverbed. An angle of from 45° to 60° between the axis of the two structures is usually used when the angle is not fixed by model studies.

SLUICE AND GATE REQUIREMENTS--Continued

.13 Figure 2 in Chapter 3 shows curved guide walls approaching the canal headworks and the sluiceway. This facility creates curvature in normal flows approaching the headworks, so, in effect, diverted water is taken from the outside of the curve where the sediment bedload is least and the heavier silt-laden water is taken through the sluiceway. Model studies must be employed to set the most efficient proportions and curvature of the walls. The top of the inner guide wall is usually set at the overflow weir elevation. SPECIAL SLUICING FACILITIES GUIDE WALLS

A. Figure 3 shows an undersluice arrangement as a special sluicing facility. The top of the undersluice is placed at the same elevation as the headworks gate sill, and the elevation of the sluiceway gate sill is made the same as that of the upstream apron. This arrangement guides the heavier sediment into the sluiceway and prevents it from entering the headworks. A disadvantage of this type is that the undersluice is subject to plugging with waterlogged debris. Success of both of the above facilities depends on continuous sluicing. Undersluice Type

.14 The number and size of gates to be selected depends to a large extent on the judgment of the designer. The total gate opening required for a given discharge is generally based on an allowable velocity of 4 to 6 feet per second. From an operating point of view, a single gate is desirable; but when more than one gate is installed they should be operated in parallel to avoid unsymmetrical flow. On rivers where considerable fluctuation in water surface can be expected during the irrigation season when the gates are in operation, they should be controlled automatically. The controls are operated by means of a floatwell or probewell to control the water surface in the canal downstream from the gate. Requirements for heating of the gates should be considered and the design of operating features should be compatible with remote control requirements. GATE SELECTION

.15 To permit closure of the structure during an emergency and to permit maintenance work on the gates, it is customary to make provision for stoplogs by providing grooves in the piers. The grooves are usually made by forming recesses in the piers, the depth being determined by the allowable bearing stress of the timber. The grooves should not be made too wide because this would permit the stoplogs to turn and bind. Continuous steel angles should be provided at the outer downstream edge of the grooves for protection and to reduce the frictional resistance when removing the stoplogs. STOPLOG PROVISION

SPECIFIC DESIGN CONSIDERATIONS

.16 In the majority of cases the headworks will be constructed on permeable material, and it will be necessary to consider the effect of underflow and the hydrostatic uplift pressure when making stability and design studies. The head causing percolation is to be taken as the difference between the maximum elevation of the water surface in the river and the elevation of the bottom of the canal. Lane's weighted creep theory or electric analogy is used in determining the percolation path required. For the ordinary, open-type structure the required path is furnished by extending the upstream apron and cutoff wall of the dam across and in front of the headworks and providing an additional cutoff wall if necessary. For the closed type of structure, the path is furnished by constructing continuous concrete collars or cutoff walls at intervals along the conduit. Hydrostatic pressure produced by water percolating along the outside of the structure should be considered in designing the walls. To prevent the foundation material from being removed by the percolating water, a filter should be constructed under the structure at its downstream edge. PERCOLATION

4.17

SPECIFIC DESIGN CONSIDERATIONS--Continued

ENERGY DISSIPATION .17 Some means of dissipating the excess energy at times of high water surface in the river should be provided. The hydraulic jump is favored as means of dissipating this excess energy. In order to confine the jump, a stilling pool downstream from the gates is required. In the case of the closed type of headworks, the hydraulic jump may take place in the stilling pool provided in the closed conduit. To avoid undesirable disturbance from the jump and to maintain free-flow conditions, ample freeboard should be provided in the closed conduit from the control gates to the outlet. Where sufficient head is available it would be desirable to have the water flow at supercritical velocity through the conduit and to have the stilling pool located beyond the outlet to avoid objectionable vibration of the structure under the embankment. It is important that consideration be given not only to flow condition at maximum discharge but also at partial discharges. The structure must operate satisfactorily under all possible conditions of flow.

TRASHRACKS .18 Trashracks are desirable at the entrance to some headworks structures and are essential where fish screens and desilting works are involved. Racks usually are constructed of flat steel bars which are set on edge and either joined by through bolts with pipe nipples for spacers or welded to the edges of the crossbars. The welded type is preferred as it provides more space for the rake teeth to pass between the bars. The racks are generally made in panels for ease in handling. To facilitate cleaning, they are usually inclined on a slope of 1 horizontal to 4 vertical and the velocity of flow through the gross area should not exceed 3 feet per second.

TRASHRACK BAR SPACING .19 Rack bars are spaced 1 to 4 inches for fine trashracks and 4 to 12 inches for coarse racks. For headworks where the rotary type of fish screens are installed and heavy bedloads are expected, closely spaced bars should be used at the lower units to prevent the larger sized gravel from becoming lodged against the screens.

TRASHRACK CLEANING .20 On small headworks the racks are usually raked by hand, but on larger installations mechanical rakes are nearly always provided. Some means of disposing of the rakings should be given adequate consideration. A typical mechanical rake, rake gantry, and disposal car that are used at the All-American Canal headworks for cleaning the trashrack are shown in Figures 4 and 5. An example of a bar-type trashrack is illustrated in Figure 6.

PROVISION FOR FISH PROTECTION

NEED FOR PROTECTION .21 The growing importance of sport fishing and the need for protecting the commercial fish industry makes necessary the screening of many irrigation and power canal intakes. To protect these interests, many States have enacted laws requiring the screening of diversions. Since the design requirements for fish screens may vary in the different States, Federal, State, and local agencies concerned with the preservation of fish life must be consulted before any designs of fish screens are made.

CLASSES OF FISH .22 The problem of fish protection involves two classes of fish with quite different habits--the migratory fish, such as the salmon and the steelhead trout, and the sedentary species which confine their movements to limited areas only in search for food. From a commercial point of view, the salmon is the most important. The sportsman is interested in both classes.

SELF-CLEANING MECHANICAL SCREENS .23 To protect the fingerlings from being lost in the canals on their journey enroute from their spawning grounds to the ocean, fine-mesh screens at canal intake are necessary. These screens may have openings as small as four to six meshes per inch which makes them very difficult to keep clean. Keeping

PROVISION FOR FISH PROTECTION--Continued

these fine-mesh screens from being clogged with floating debris and leaves is one of the major problems in designing a fish screen. To overcome this difficulty several types of self-cleaning mechanical screens have been developed, namely the rotary-drum type and the traveling screens which are similar to the industrial water screens. SELF-CLEANING MECHANICAL SCREENS (Cont'd.)

A. The rotary-drum screens are very sensitive to changes in water level and are adaptable only where there will be little fluctuation in water surface. The water depth is limited by the size of the drums. To effectively remove the floating debris without loss of fish over the screens, from 0.1 to 0.3 of the diameter of the drum should extend above the water surface. The required circumferential speed of the drum will depend on the amount of debris being carried by the streams, the maximum speed is about 5 feet per minute. This type of screen consists of structural steel cylinders covered with a fine-mesh screen and supported by a horizontal steel shaft. The drums can either be propelled by means of a waterwheel located downstream or be motor-driven. The fish screens installed at the Florida Farmers Ditch Diversion Dam are of the rotary-drum type. (See Figure 7.) Instead of installing drum screens normal to the flow, recently some fishery authorities prefer placing the screens at an angle of 20° to 30° with the flow. This is to facilitate diverting fish into bypasses. Rotary Type

B. Where there is a wide operating range in the water surface, the traveling or industrial water screens are more adaptable as they can be constructed for almost any depth of water. This type of screen is made up in units. Each unit consists of a continuous belt of overlapping screen trays which are fastened between two parallel link belts operated from two sets of sprockets. The sprockets are mounted on a structural steel frame with the head or driven sprocket located at the top of the frame and the foot or nondriven sprocket located at the bottom. The head sprocket is driven by a gear speed-reducer, roller-chain drive, and motor. To dislodge debris that becomes lodged against the screen trays, a series of water jets are provided on the downstream underside of the trays at a point above the high water level. Traveling Type

.24 At locations where the size of the fish are such that an extremely fine screen is not required, the stationary-bar type of screen is sometimes used. The bar screen can be constructed for almost any fluctuation in water surface but is somewhat difficult to keep clean. An example of this type of screen which is kept clean by means of a motor-driven sweep is the Contra Costa fish screens, Figure 8. BAR SCREENS

.25 The louver system of guiding migratory fish into bypasses is a recent development in fish screens for canal headworks at river diversions. The system consists of a series of vertical bars with 1-inch clear spacings and flow-straightening vanes set in a single row or multiple rows across a channel, each row at an angle of about 15° to the direction of flow. The fish are diverted away from the louver line to a vertical bypass at the downstream end. The bypass covers the full depth of flow and the minimum width is 12 inches. The velocity of flow entering the bypass should be about 1.4 times the channel velocity. The success of this screening system depends on the fish sensing the turbulence around the louver bars. An example of this type screens is the Tehama-Colusa Canal louver fish diverter, Figure 9. LOUVER DIVERTER

.26 Another type of stationary fish screen that appears to have possibilities is the electric fish screen. The principal parts of this screen are the electronic impulse generator which employs the industrial-type vacuum tube, ELECTRIC SCREENS

4.27

PROVISION FOR FISH PROTECTION--Continued

ELECTRIC SCREENS (Cont'd.)

the live electrode, and the ground conductor. The live galvanized pipe electrodes are suspended vertically to swing freely from an overhead beam or catenary cable supported by structural steel supports on each side of the canal. Random lengths of galvanized pipe strung on a cable and joined electrically by a copper wire are placed along the bottom of the channel to form the ground conductor. The electric screen has the advantage of low cost and it allows all types of stream-carried debris to pass through without obstructing the flow of water. This screen is entirely dependent on a reliable supply of power. The efficiency of the electric fish screen has been questioned by Fish and Wildlife authorities and there are some conflicting opinions regarding its effectiveness in stopping the fish. Consequently, a thorough investigation of existing installations should be made when considering the use of this type of screen.

VELOCITY OF APPROACH TO SCREENS

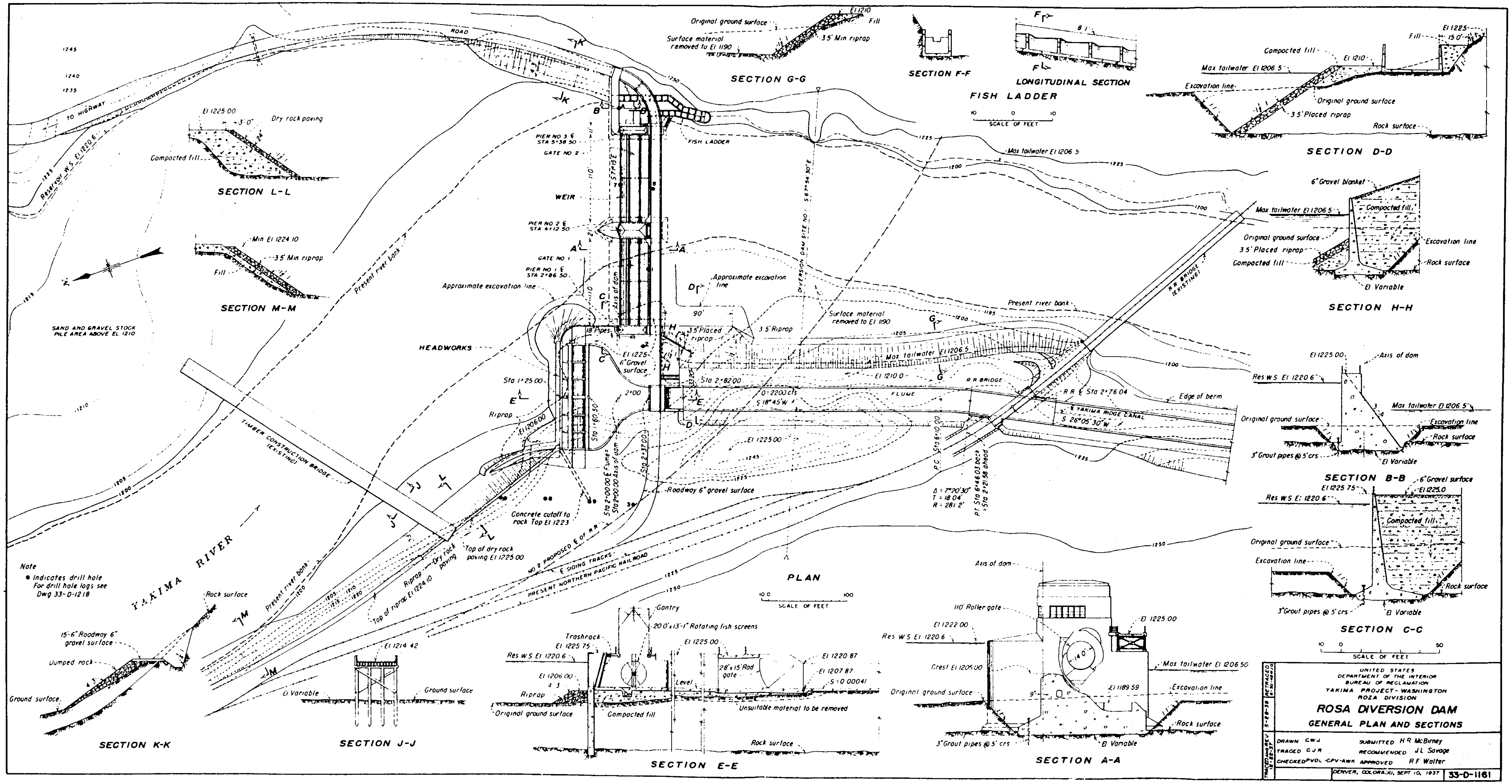
.27 In the design of all types of fish screens, the velocity of approach is important. In the case of mechanical screens, the approach velocity should not exceed 1.5 feet per second in order to keep the fish from being held against the screens by the force of the current. Low velocities are also necessary where electric screens are used to avoid having the fish carried helplessly through the electrified zone. To maintain screening efficiency for louver diverters the velocity of approach should be from 2.5 to 4.0 feet per second to create optimum turbulence.

FISH BYPASS CHANNEL

.28 Another important feature in connection with the design of fish screens is the fish bypass channel. When the small fish are traveling downstream and are stopped by the screens, it is believed that they will not turn back but will find a way through or past the screens, or destroy themselves trying. Therefore, a suitable downstream bypass should be provided to allow an alternative route for the fish when stopped by the screens to return to the main stream.

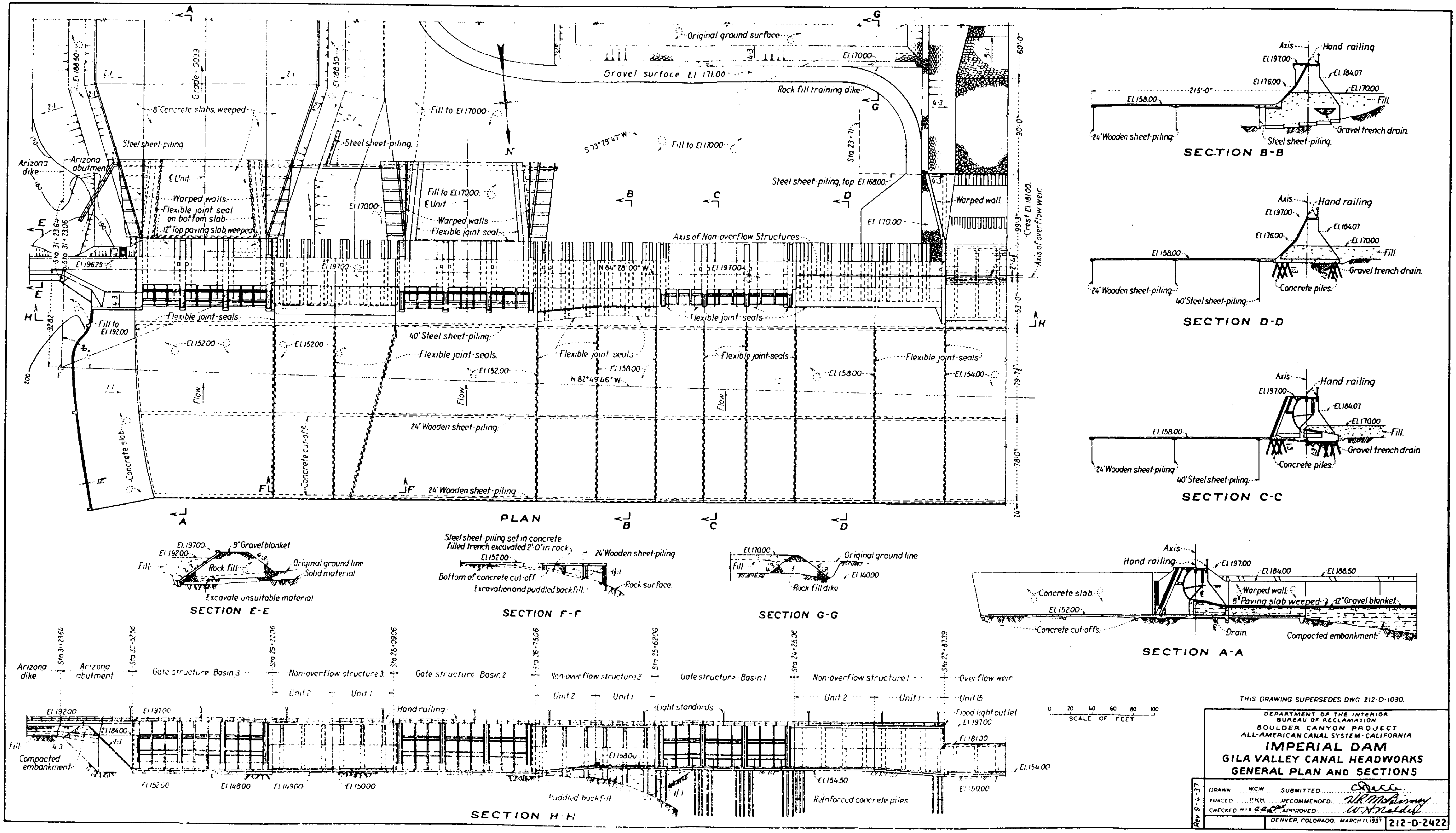
ROSA DIVERSION DAM--GENERAL PLAN AND SECTIONS

Fig. 1 Par. 4.8



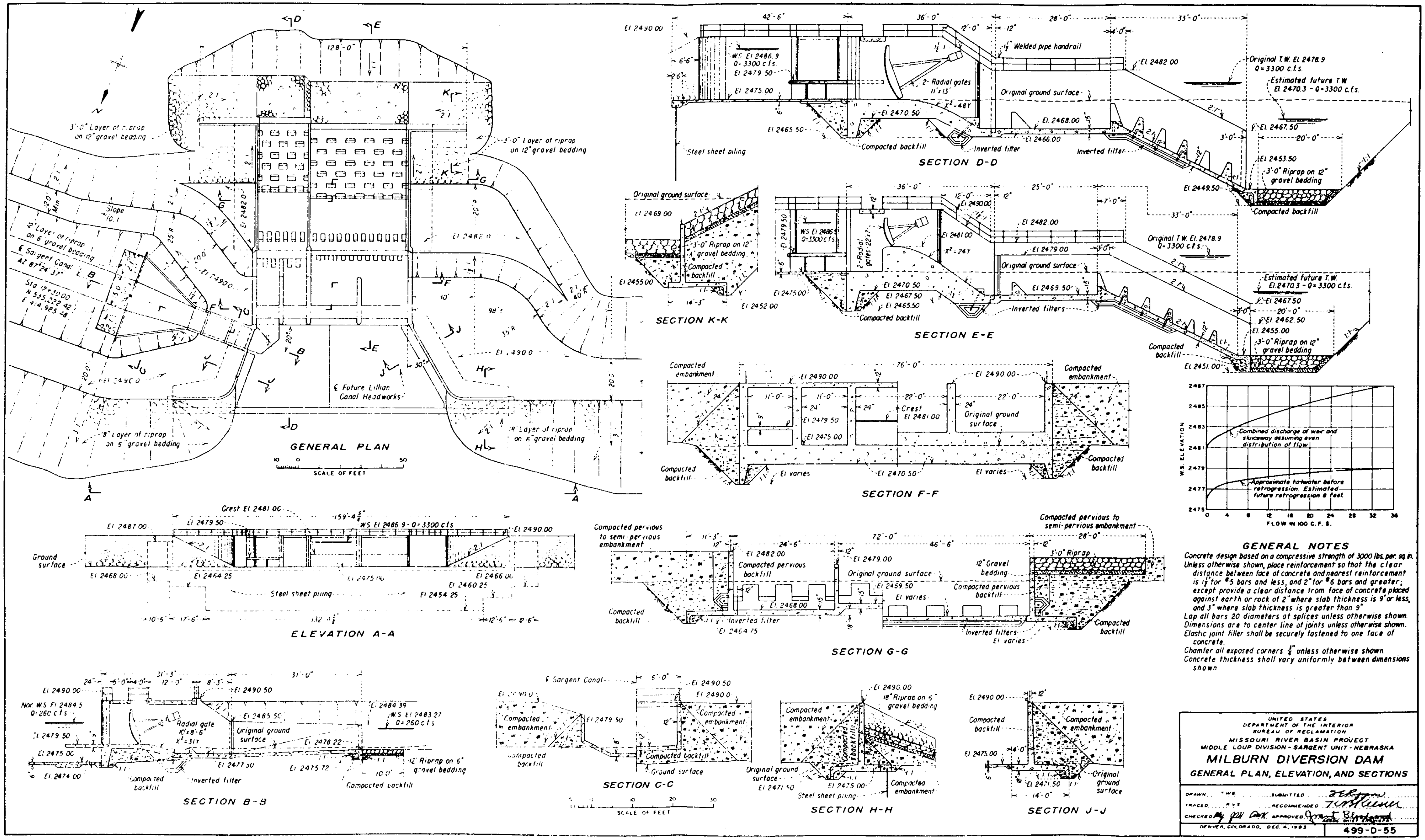
GILA VALLEY CANAL HEADWORKS, IMPERIAL DAM--GENERAL PLAN AND SECTIONS

Fig. 2 Par. 4.9



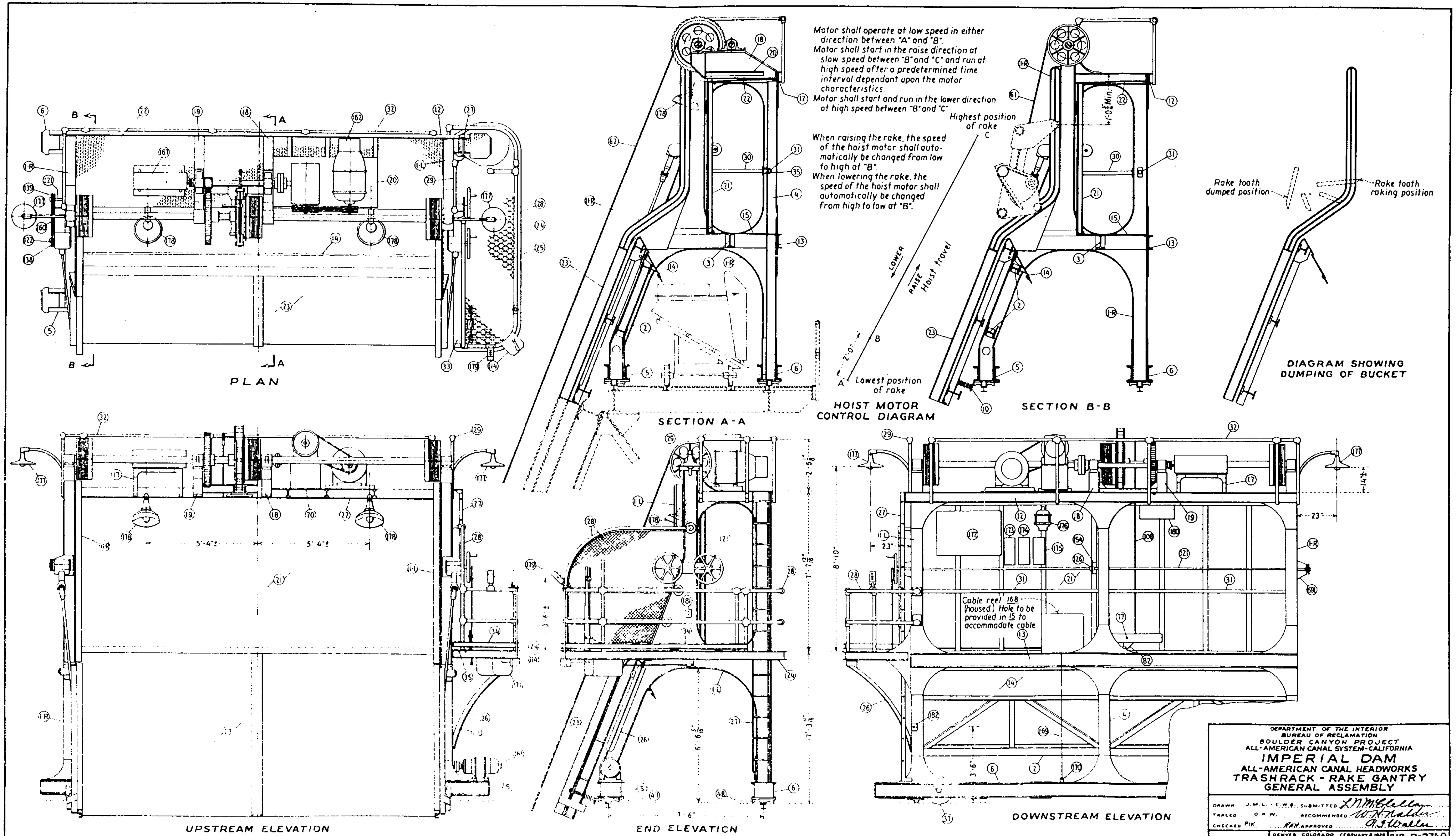
MILBURN DIVERSION DAM--GENERAL PLAN, ELEVATION, AND SECTIONS

Fig. 3 Par. 4.13A



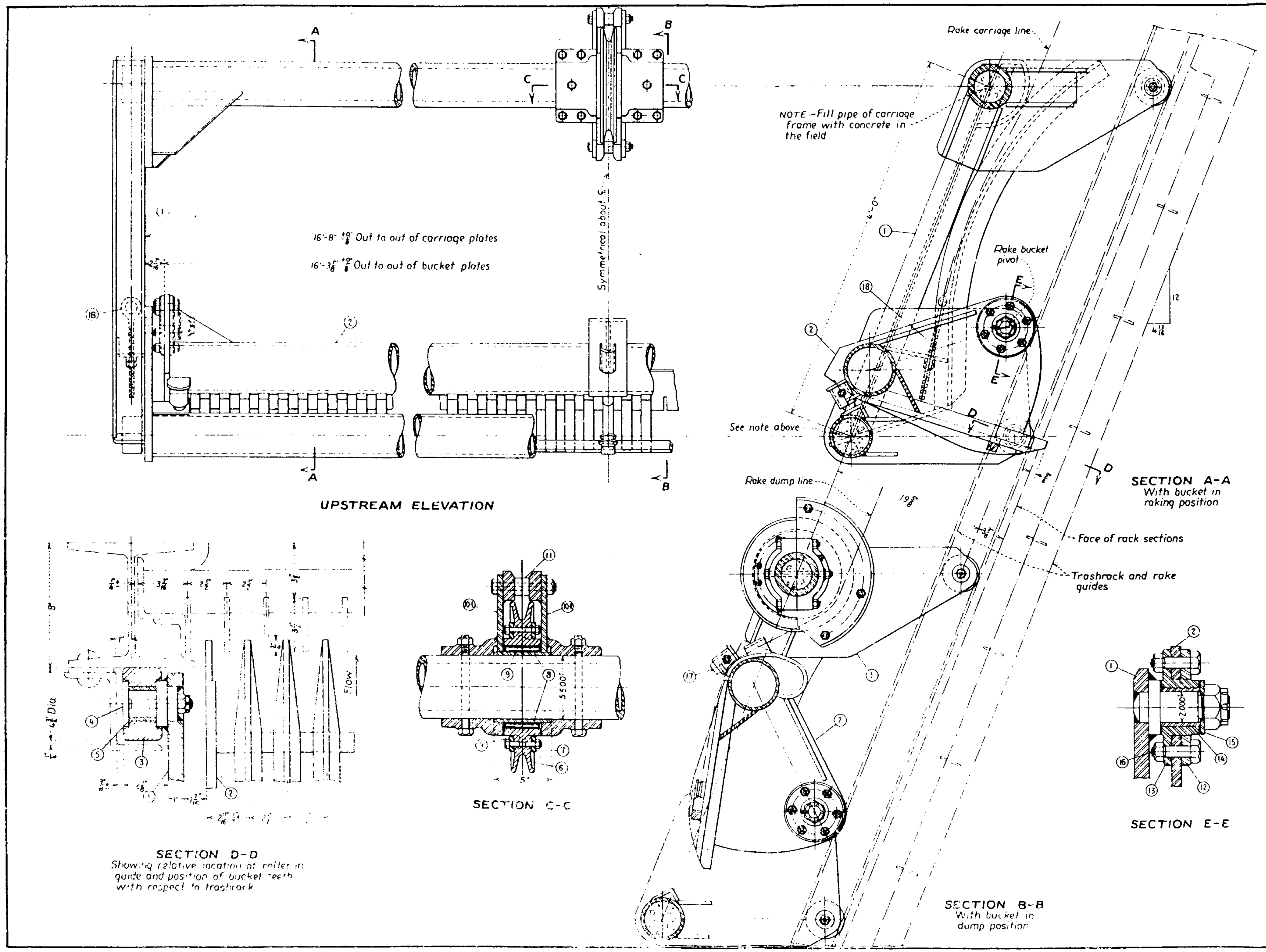
TRASHRACK RAKE GANTRY, GENERAL ASSEMBLY--ALL-AMERICAN CANAL HEADWORKS

Fig. 4 Par. 4.20



TRASHRACK RAKE ASSEMBLIES, LIST OF PARTS--ALL-AMERICAN CANAL HEADWORKS

Fig. 5 Par. 4.20



LIST OF PARTS

PART No.	DESCRIPTION	MATERIAL	No. REQ'D	DRAWING No.
1	Carriage	Steel	1	212-D-3757
2	Bucket	Steel	1	212-D-3758
3	Roller	Semisteel	4	212-D-3759
4	Roller cap	S.A.E. 1045 Steel	4	" "
5	Bushing	Steel	4	" "
6	Split sheave	Semisteel	1	" "
7	Clamp ring	Steel	2	" "
8	Split bushing	Steel	1	" "
9	Roller bearing	Steel	41	" "
10-R	Sheave guide	Steel	1	" "
10-L	Sheave guide	Steel	1	" "
11	Spacer	Steel	3	" "
12	Flange	Steel	2	" "
13	Bucket bearing	Steel	2	" "
14	Bushing	Steel	2	" "
15	Washer	Steel	2	" "
16	Special bolt	Bolt steel	12	" "
17	Rubber bumper	Rubber	2	" "
18	Eye bolt	Steel	2	" "

LIST OF DRAWINGS

ASSEMBLIES - LIST OF PARTS 212-D-3756
CARRIAGE 212-D-3757
BUCKET 212-D-3758
ROLLER - SHEAVE - GUIDE 212-D-3759

REFERENCE DRAWINGS

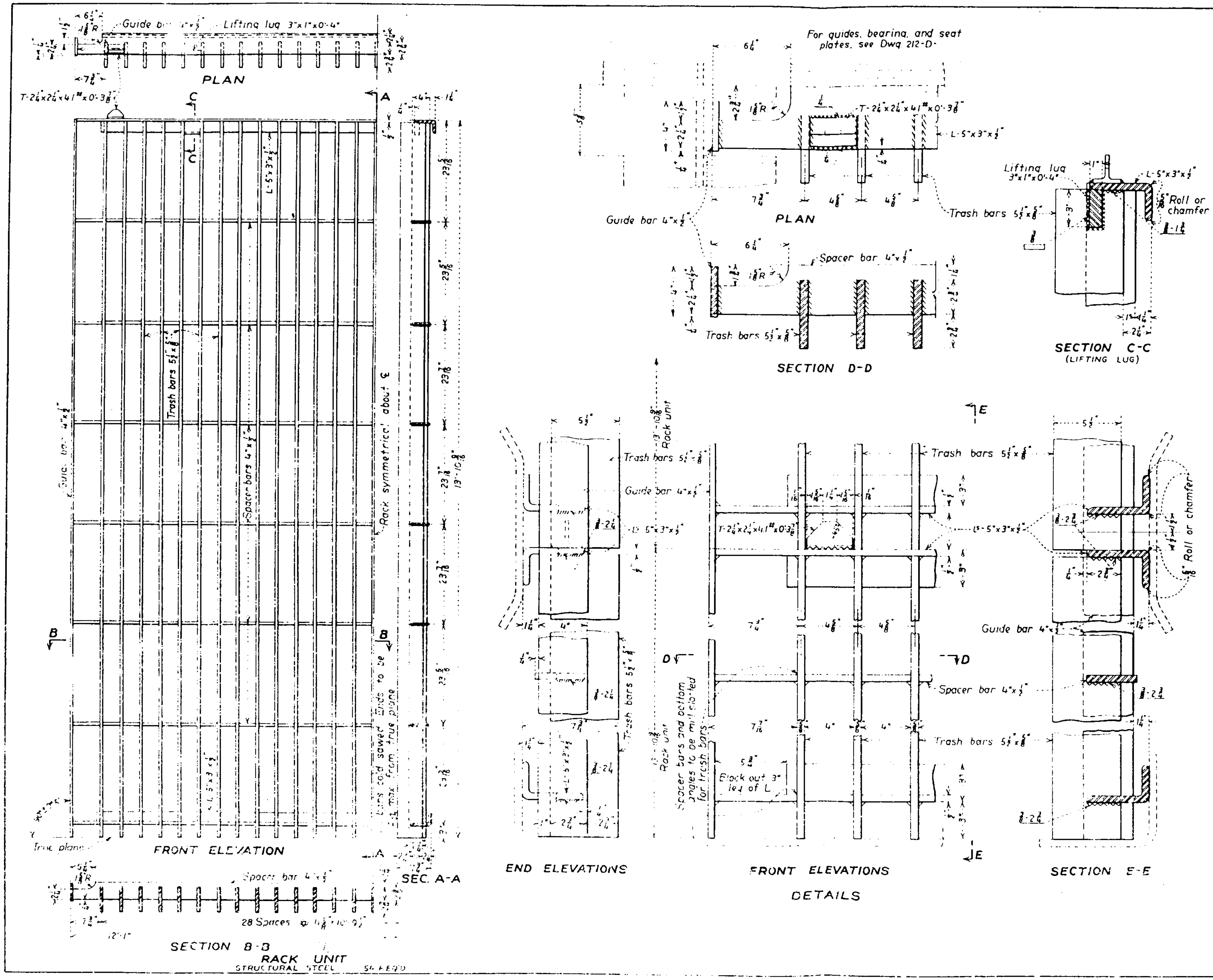
TRASHRACK STRUCTURE 212-D-2366
TRASHRACK 212-D-2420
TRASHRACK-RAKE GANTRY 212-D-3739
WELDING SYMBOLS X-D-1357

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
ALL-AMERICAN CANAL SYSTEM-CALIFORNIA
IMPERIAL DAM
ALL-AMERICAN CANAL HEADWORKS
TRASHRACK - RAKE
ASSEMBLIES - LIST OF PARTS

DRAWN JML SUBMITTED L.N. McClellan
TRACED EWC RECOMMENDED W.H. Nalder
CHECKED FK APPROVED J.F. Walter
DENVER, COLORADO FEB. 8, 1939 SHEET 1 OF 4 212-D-3756

TRASHRACK METALWORK, RACK UNIT, LIST OF PARTS--GILA VALLEY CANAL HEADWORKS

Fig. 6 Par. 4.20



LIST OF PARTS

PART NO.	DESCRIPTION	MATERIAL	NO. REQ'D.	DRAWING NO.
1	Rack unit	Structural steel	※ 18	212-D-1597
2	Bottom guide - Basin 1	" "	6	212-D-1598
3	Bottom guide - Basins 2 and 3	" "	12	"
4	Top guide - Basins 1, 2 and 3	" "	18	"
5R	Bottom guide - Basin 1	" "	3	"
5L	Bottom guide - Basin 1	" "	3	"
6R	Bottom guide - Basins 2 and 3	" "	6	"
6L	Bottom guide - Basins 2 and 3	" "	6	"
7R	Top guide - Basins 1, 2 and 3	" "	9	"
7L	Top guide - Basins 1, 2 and 3	" "	9	"
8	Anchor plate	" "	18	212-D-1599
9	Anchor plate	" "	18	"
10	Bearing plate	" "	27	"
11	Seat plate - Basins 1, 2, and 3	" "	27	"
12R	Seat plate - Basins 2 and 3	" "	18	"
12L	Seat plate - Basins 2 and 3	" "	18	"
13	Support beam	" "	9	"
14	Splice plate	" "	18	"
15	Splice plate	" "	30	"
16	Splice plate	" "	18	"
17	Splice plate	" "	18	"
18	Filler plate	" "	18	"
19	Filler plate	" "	18	"
20	Filler plate	" "	18	"
21	Filler plate	" "	30	"
22	Anchor bolt	Bolt steel	325	"
23	1"x3" Mach. bolt with hex. nut & wrot washer	" "	60	No detail
24	1"x3½" Mach. bolt with hex. nut & wrot washer	" "	195	" "
25	1"x3½" Bolt with hex. nut and wrot washer	S.A.E. 3140 steel	145	" "
26	1"x1¾" Hex. head cap screw with wrot washer	Bolt steel	72	" "
27	1"x1¾" Hex. head cap screw with wrot washer	" "	144	" "
28	1x2½" Csk. head bolt with hex. nut and lock "	" "	72	" "
29	Splice plate	Structural steel	18	212-D-1599

※ Rack units for Basin 3 only

DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BOULDER CANYON PROJECT
ALL-AMERICAN CANAL SYSTEM-CALIFORNIA

IMPERIAL DAM
GILA VALLEY CANAL HEADWORKS
TRASHRACK METALWORK
RACK UNIT-LIST OF PARTS

DRAWN J.E.G.		SUBMITTED	C. M. Day
TRACED C.R.G.-M.E.B.		RECOMMENDED	[signature]
CHECKED P.I.K.		APPROVED	[signature]

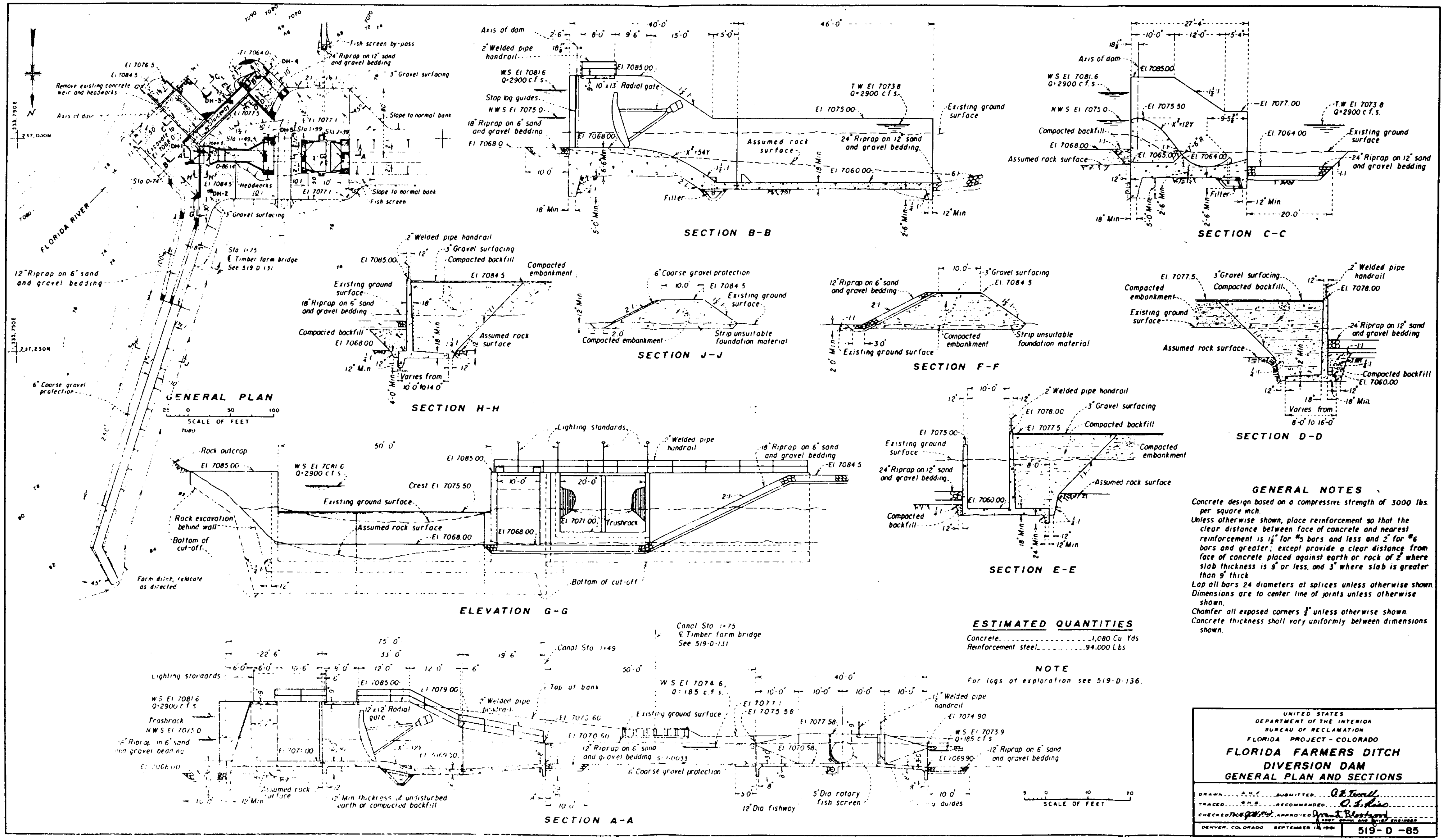
9-15-36

DENVER, COLORADO, JULY 8, 1936 SHEET 2 OF 4 — 212-D-1597

DC-3-5 - 12/8/67 (Supersedes 1/67-1)

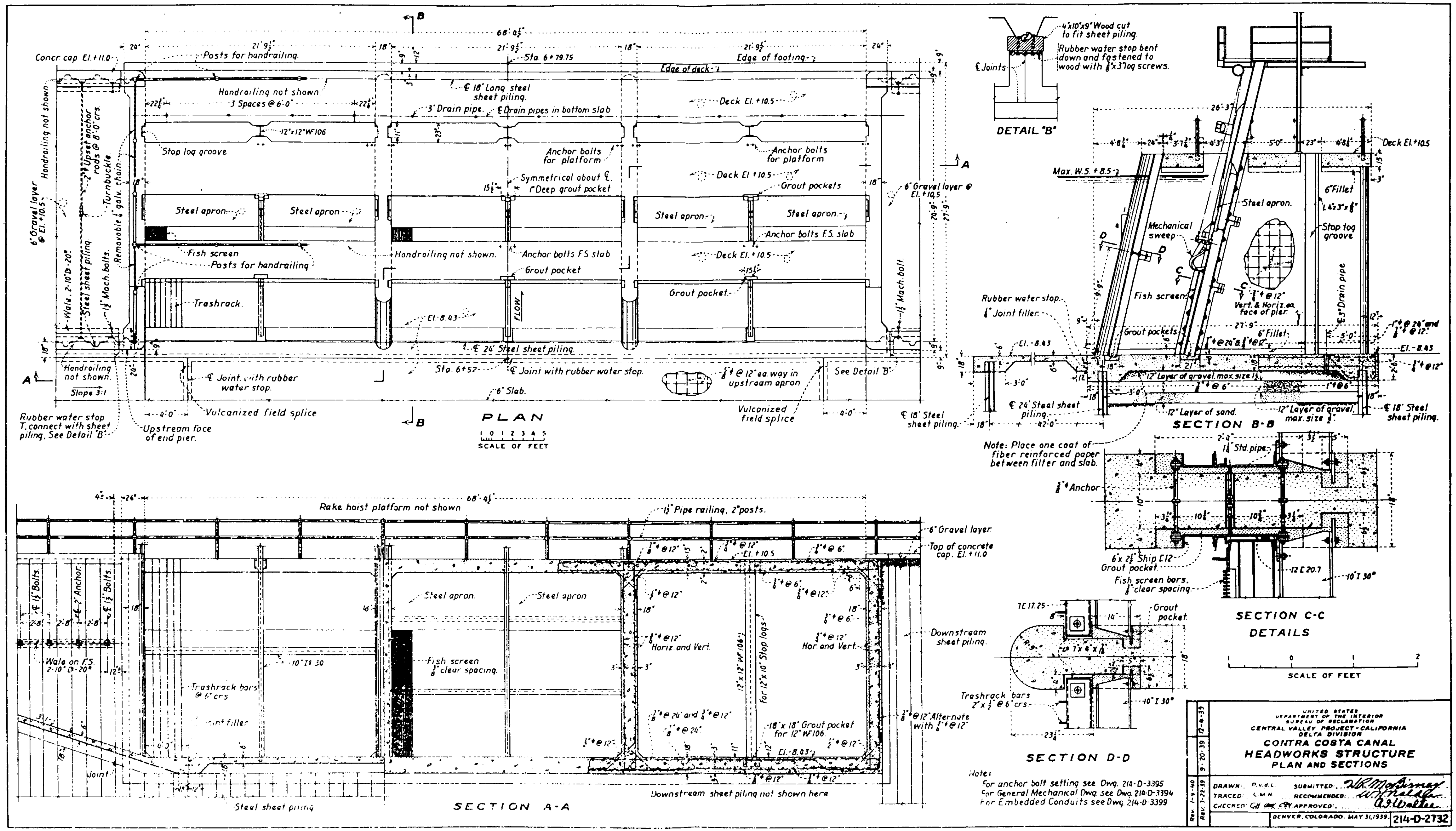
FLORIDA FARMERS DITCH DIVERSION DAM--PLAN AND SECTIONS

Fig. 7 Par. 4.23A



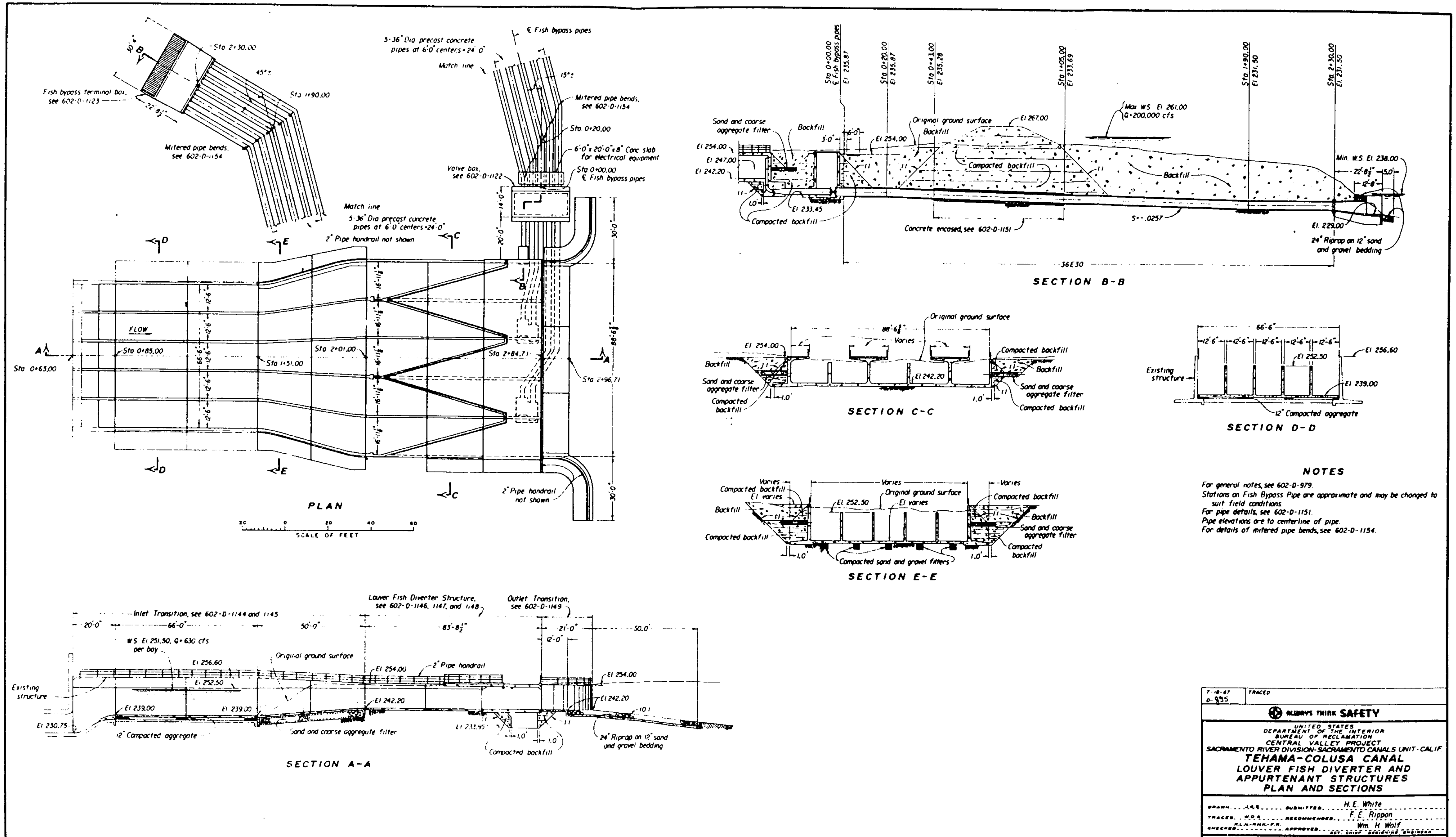
HEADWORKS STRUCTURE, PLAN AND SECTIONS--CONTRA COSTA CANAL

Fig. 8 Par. 4.24



TEHAMA-COLUSA CANAL LOUVER FISH DIVERTER AND APPURTENANT STRUCTURES--PLAN AND SECTIONS

Fig. 9 Par. 4.25



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.1 The following paragraphs deal with commonly used canal and lateral structures except water measurement structures, cross-drainage structures, protective structures, and bridges, which are described in following chapters. General structural and hydraulic design criteria relating to canal and lateral structures are given in Chapter 2. In the structural drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated. INTRODUCTION

SIPHONS

.2 A siphon (sometimes called an inverted siphon, sag pipe, or sag line) is a closed conduit designed to run full, and usually under pressure, that transports canal and lateral water by gravity under railroads, roads, various types of drainage channels, and depressions. DEFINITION

.3 The shape and number of barrels will be governed by the local conditions and economy. The shape of the sections most commonly used are circular and rectangular, although trapezoidal and other shapes have been used for specific requirements. BARREL TYPES

A. Precast concrete pressure pipe and monolithic concrete pipes are the most common types of circular siphon barrels used for heads up to about 125 feet. Plate steel, monolithic concrete with steel liner, precast concrete with steel cylinder, pretensioned concrete, prestressed concrete, asbestos cement, and cast iron pipe are usually used for higher heads. Precast concrete pressure pipe is normally used for diameters up to about 96 inches; however, larger sizes have been used in some areas. Monolithic concrete pipe is generally used for diameters larger than 96 inches and in some areas for smaller sizes. Chapter 2 gives allowable stresses for monolithic circular barrels under bursting heads. Figures 1 and 2 of this chapter show the layout and details for a siphon comprised of 96-inch-diameter precast concrete pressure pipe. Figure 3 shows an inlet transition with check. Figure 4 shows a 210-inch-diameter siphon, and Figure 5 shows the reinforcement details for the siphon barrel. Ultimate load analysis is generally used for the design of precast and monolithic concrete pipe. Circular Sections

B. Rectangular single- or multiple-barrel box sections are often used for short siphons when the head is 30 feet or less on the inside top of the box. Figure 6 shows a box siphon with concrete-lined canal. A trapezoidal box siphon is used occasionally for short siphons with little or no sump when it is important to keep the head loss to a minimum. Box Sections

.4 Normally, the head loss through a siphon should include friction losses in transitions and barrel, inlet convergence and outlet divergence losses, bend losses, and check loss when a check is installed in the transition. A 10-percent safety factor is usually added to all losses. Figure 7 shows a procedure for determining head losses for concrete siphons: (For design details, see Chapter 2.) For long siphons where conditions may result in operation when the inlet is not sealed, the design should be checked with Figure 12 in Chapter 2 to insure satisfactory operation. Although available head and economy usually determine the barrel size, where sand or other abrasive material is carried by the water it is desirable to limit the velocity to about 10 feet per second. For minimum hydraulic loss, it is desirable to provide a seal of $1.5\Delta h_v$ (3 inches minimum) at the inlet headwall measured from the upstream water surface and no submergence at the outlet headwall. HEAD LOSS

5.5

SIPHONS--Continued

APPURTENANCES .5 The following appurtenances are required at siphons as indicated:

Blowoff

A. A blowoff should be installed at or near the low point of a siphon if it is impracticable to empty the siphon by pumping from the lower end. In some instances it has been considered economical to break into 24-inch-diameter and smaller siphons instead of providing blowoffs for emergency draining. Figure 8 shows details of a simple blowoff structure.

Protective Features

B. Protective features shall be provided for all siphons (Class A, B, and C exposure) over 30 inches in diameter. Refer to Chapter 3, Safety Design Standards, of Design Standards No. 1 for specific requirements.

TUNNELS

GENERAL .6 Where a canal would be in deep cut or on unstable slopes, a tunnel can often be justified as an alternative to an open-canal location on the basis of economy, low maintenance cost, and safety of operation. A tunnel may also be the most economical means for carrying water from one watershed to another. Such tunnels are usually free-flow conduits lined with concrete and designed for a maximum water depth equal to about 0.82 times the tunnel height, with a minimum freeboard of 18 inches. If tunnel excavation is by blasting methods, a horseshoe section is generally used for free-flow tunnels and a circular section is used for tunnels operating under hydrostatic head, such as pressure tunnels in power systems. If tunnel excavation is by boring machines, the tunnel section is circular and differs in detail from the circular section constructed by blasting methods. Figures 9 through 13 show tunnel section details.

The size of most tunnels is determined by an economic study based on capacity required, construction cost, and value of head loss. Accessibility of the ends of the tunnel, adits, and shafts as well as geological and ground-water data are required to determine the design and feasibility of a tunnel. Ample disposal areas should be provided for tunnel excavation. In long tunnels, the possibility of adits at saddles or canyons and shafts at low cover points should be considered for their construction advantages. Permanent access should be provided in all tunnels for inspection and maintenance, and station markers should be provided at about 1,000-foot intervals in long tunnels.

PORTALS .7 Tunnel portals should usually be located where the depth of cover over the tunnel is twice the tunnel diameter or 20 feet minimum in rock, and three times the diameter or 30 feet minimum in common material.

The slopes of the portal cuts should be flat enough to avoid excessive raveling of unstable bank material. Portals should not normally be located in natural surface drainage channels. Cut-and-cover sections may extend beyond the portal where the stability of the material is doubtful. Transitioning of the flow from open canal or flume to a free-flow tunnel is normally made in two steps:

TUNNELS--Continued

(1) From open canal or flume to a rectangular section at the tunnel portal; and PORTALS (Cont'd.)

(2) From the portal to the tunnel section. (See Figure 14.)

.8 Tunnels may very in shape and size, but usually a minimum finished height of 6 or 7 feet, depending on tunnel length, is used to allow for use of standard equipment and working room during construction. Cross sections larger than the minimum should be determined by capacity requirements and economic studies. Where horizontal curves are necessary in a tunnel, the radius should be adequate to allow for unrestricted movement of construction equipment. Figure 15 gives dimensional formulae for tunnel portal transitions. DIMENSIONS

.9 Normally, the head loss through a tunnel should include friction losses in transitions and in the tunnel sections, inlet convergence and outlet divergence losses, and bend losses. For design details, see Chapter 2. Where sand or other abrasive material is carried by the water, it is desirable to limit the velocity to about 10 feet per second. For clear water transportation, the only limitation of velocity in a free-flow tunnel is a safe margin from critical velocity. (See Figure 16.) HEAD LOSS

.10 Ordinarily, a tunnel will require some type of support over a portion of its length to hold the walls and roof until lining can be placed. Where supports are required, steel ribs with timber lagging and blocking are generally used, and their shape and placement should be designated on the drawings. The size of the steel rib supports depends on the size of the tunnel and the characteristics of the ground to be supported. Since the kind of material to be encountered is not definitely known until the excavation is made, it is not practicable to designate the size or spacing of the rib supports until this time. In competent rock, no supports should be required. If the rock is shattered or crumbly, extremely heavy pressures may be exerted, which will require close spacing of heavy supports and the probable use of invert struts. When soft ground or very unstable material is encountered, the use of tunnel-liner plates may be required to restrain the material from moving into the excavation. Rock bolts have been used successfully for supporting incompetent rock under some conditions. Where their use is feasible, rock bolts with supporting members are more economical than steel rib supports with lagging and blocking. SUPPORTS

.11 For the purpose of establishing minimum excavation and discouraging excessive overbreak during construction, two lines are usually shown on the typical tunnel sections. These two lines are referred to as the "A" line and the "B" line. The "A" line is the line within which no unexcavated material of any kind, no timbering, and no metallic or other supports for the sides, roof or floor of the tunnel shall be permitted to remain; except that for the steel rib supported section for the machine bored tunnel the structural steel rib support may protrude inside the "A" line. The "B" line is the limit to which payment will be made. "A" AND "B" LINES

.12 High-pressure grouting, when employed, is used in an attempt to solidify shattered or unstable rock, to cut off or minimize the flow of ground water in the rock surrounding the tunnel, or to fill voids behind the concrete lining where high external hydrostatic pressure exists. Low-pressure or backfill grouting is employed generally in pressure tunnels in an attempt to provide the maximum amount of contact between the rock and the concrete lining. It may also be employed in free-flow tunnels in areas of incompetent rock. Ground-water pressure on the tunnel lining may be controlled by weep holes through the lining or by a drain under the invert if suitable drain outlets can GROUTING AND DRAINING

5.13

TUNNELS--Continued

GROUTING AND DRAINING (Cont'd.)

be provided. Where permanency of the ground-water pressure is doubtful and leakage from the tunnel is objectionable, flap valves on drain outlets in the tunnel have been employed. Grouting in areas in which invert drains of any type are installed for permanent relief will probably plug the drains and should therefore not be contemplated.

PROTECTIVE FEATURES

.13 See discussion of protective features for siphons in Subparagraph 5.5B. This discussion applies also to tunnels.

CHECKS

FUNCTION, ARRANGEMENT, AND TYPES OF CONTROL

.14 Checks are used to control the flow beyond the structure or to maintain a certain water depth above the structure. Provisions should be made for overflow on all checks. Checks may be a separate structure or combined with the inlet to siphons, drops, or chutes. The combination is often desirable for economy and to prevent racing and scouring upstream of siphons or drops.

Check structures generally use radial gates, slide gates, stoplogs, or some combination of these to control the amount of water passing the structure.

Radial Gates

A. Radial gates are generally used in large structures and may be provided with hand-lift or motor-operated hoists, depending on the size and weight. Gate widths should not exceed 25 feet for most structures, as several small gates are generally more economical and easier to operate than the larger gates. Deflection of walls and piers should be small to prevent binding the gates. Figures 17 and 18 show radial-gate check structures.

Slide Gates

B. One or more slide gates are often used for smaller structures. Normally these are hand-operated gates, but experience has shown that many times these gates are later converted to motor operation in conjunction with some form of automatic controls. Therefore, it is important that the gates selected are sturdy enough to be motorized without rebuilding and strengthening the frame. Figures 19 and 20 show small slide-gate check structures. The total fall in water surface through the check on Figure 20 is limited to a maximum of 18 inches.

Stop Planks

C. Stop planks are sometimes used in checks with capacities less than 50 cfs where operational changes are infrequent or where it is unlikely that automatic controls will be subsequently desired. Stop planks should not be used in openings greater than 5 feet wide or with water depths over 6 feet. The guides should be vertical where the distance from the floor to the bottom of the platform is less than 6 feet. For greater depths, the guides should be sloped 1:4 downstream from bottom to top of the stop plank guides.

The bottom of the stop plank guides or slide gates may be placed at the upstream end of the structure if the capacity is less than 100 cfs, but must be placed at least 2 feet from the upstream end of the structure for larger capacities. See Figures 21 and 22 for stop plank grooves and structural liners. Figure 23 is a design chart for stop planks.

Protective Features

D. A 2-foot minimum width platform can be used if the height above the downstream check floor is less than 3.5 feet, but if it is 3.5 feet or more the minimum platform width should be 3.0 feet. A downstream guardrail is required on the platform for heights above the floor of 3.5 feet or over, and both an upstream and a downstream guardrail are required if the height is 5.0 feet or more. For additional safety provision requirements see Chapter 3, Safety Design Standards, of Design Standards No. 1.

CHECKS--Continued

.15 A loss equal to 0.5 times the difference in velocity head between the check opening and the upstream canal section usually is adequate, but more accurate computations may be required where head is critical. Use 0.1 foot minimum loss for isolated checks in small canals and 0.05 foot in large canals where check structures are provided with streamlined transitions. About 3.5 feet per second is the maximum velocity through check structures using stop planks, owing to difficulty in operation, whereas a velocity of 5 feet per second is not objectional through most structures using gates. HEAD LOSS

FLUMES, CHUTES AND DROPS

.16 Flumes are used to convey water along steep hillsides, across depressions, and where restricted right-of-way or other reasons makes the construction of canal banks undesirable or impracticable. Flumes set on the ground are called bench flumes, and those supported above the ground, elevated flumes. Rectangular concrete flumes are the most common, and concrete footings with concrete or steel supports are generally used for elevated flumes. FLUMES

A. The width to height ratio of a flume section should be set by economy of construction costs including excavation and fill, unless limited by local conditions such as right-of-way. Backfill against the walls will also be determined by local conditions. On rocky hillsides it is usually good practice to backfill the side adjacent to the hill to protect the wall against rocks. After backfill conditions are known, drainage and sliding resistance must be provided as needed. The flume walls should be designed for the loadings discussed in Chapter 2, and the induced moment and shear in the bottom slab provided for. The walls of an elevated flume are usually treated as beams to carry the load between supports. However, where the width exceeds 20 feet it may be more economical to use an "ell" wall section separated from the floor by a joint and to carry the center section loads between supports in the floor slab. Flume Section

The flume should be provided with rubber waterstop joints every 25 feet. These joints can be butt joints except that elastic filler joints should be added from the PC to the PT of curves and adjacent to transitions. Bell joints are not recommended.

B. Hydraulic losses in a flume should be computed the same as for a siphon, except that the 10 percent excess loss is not provided. The velocity in a flume should be low enough to avoid approaching critical depth at structure irregularities or at the maximum grade allowed by construction tolerances with an assumed "n" = 0.011. See Figures 5 and 6 of Chapter 1 for similar "Recommended maximum design velocity" in concrete-lined canals. Hydraulics

Freeboard in a flume should be correlated with that in the adjacent canal; it should be set to overtop before the canal banks or be higher than the adjacent banks, whichever will give the least damage should overtopping occur. For flume sections designed for velocities higher than critical, see "Chutes" in the following paragraphs.

.17 Chutes and inclined drops are structures commonly used to convey water to a lower elevation. When not more than 15 feet of fall in the energy gradient occurs at the structure, it is generally called an inclined drop, and when more than 15 feet of fall in the energy gradient is to be dissipated by the structure, it is generally called a chute. Chutes and drops are usually rectangular in cross section; however, trapezoidal cross sections have occasionally been used for small capacities in an attempt to effect construction CHUTES AND INCLINED DROPS

5.17A

FLUMES, CHUTES AND DROPS--Continued

CHUTES AND INCLINED DROPS (Cont'd.)

economy when the structure was located in thorough cut. Figure 24 shows the nomenclature, symbols, and other data referred to herein for rectangular chutes and inclined drops. Figures 25 and 26 show a typical rectangular chute and stilling pool structure.

Inlet

A. Check structures are often combined with the inlets of both chutes and drops. The checks in such cases are utilized as a control to prevent racing of the water upstream from the inlet, in addition to their usual function of raising the water surface to permit diversion through upstream turnouts during periods of partial discharge. When no check structure is required near the inlet to a chute or drop and the canal upstream does not have a hard-surface lining, the inlet to the chute or drop must be designed to provide a control section which will prevent racing upstream and scouring of the canal. The inlet must be designed so that the full capacity can be discharged into the chute or drop with normal depth in the canal. The inlet should be symmetrical about the centerline and, whenever possible, sufficiently distant from horizontal bends upstream as to limit undesirable wave action due to unsymmetrical flow. The length of the percolation path along the inlet structure, as computed by Lane's weighted creep method, should be such that the phreatic line does not intersect the top of the chute channel or stilling pool walls. For construction reasons several cutoff walls, properly spaced, one of which is the upstream cutoff as described in Chapter 2, are usually preferable to one deep cutoff to provide the required length of percolation path.

Chute Channel and Tapered Transitions

B. The chute channel, as shown on Figure 25, will usually consist of a length of channel with its grade following the general configuration of the original ground surface, and a short steep section leading to the stilling pool. The slope of the steep section should not be steeper than 1-1/2:1 and not flatter than 3:1; a slope of 2:1 is preferred. An economic width of chute channel should be determined after the grade of the chute channel has been selected. For chutes, a spreading transition will usually be required between the narrow chute channel and the stilling pool, and a trajectory will be required between the flatter and steeper portions of the channel. For inclined drops (see Figure 24), it is generally practicable to make the chute channel the same width as the stilling pool, with a vertical curve or trajectory between the slope of the inlet and the chute channel slope. For the purpose of designing a rectangular chute the depths should be computed along the chute channel to the beginning of the trajectory or spreading transition using Manning's "n" of 0.010, by a stepwise procedure such as illustrated in King's Handbook of Hydraulics. Inlet structure losses should be neglected for this computation. The sine of the chute channel slope angle, not the tangent, should be used in these computations.

The cotangent of the angle of contraction or expansion in developed plan of each side of a tapered chute transition should not be less than 3.375F. The mean of the values of F at the beginning and end of the tapered transition may ordinarily be used. If the taper is for the most part on the trajectory, K for the trajectory should be used. If a considerable part of the taper is not on the trajectory, the taper angle and widths for several points should be computed using the applicable K value and a chord drawn for the taper which approximates the theoretical curve.

FLUMES, CHUTES AND DROPS--Continued

Chute Channel and Tapered Transitions (Cont'd.)

α = Coriolis (velocity distribution) coefficient (if not evaluated, α may be considered equal to unity),

d = depth normal to floor profile (water area divided by water surface width),

F = Froude number = $V/\sqrt{(1-K)(gd \cos\theta)/\alpha}$,

K = trajectory acceleration factor defined in Subparagraph C below. (if Subparagraph C is not used to detail the trajectory, $K = \alpha V^2/(gR \cos\theta)$),

n = 0.010 on concrete (n = 0.014 only for computing K on a ski jump),

R = radius of curvature of floor profile (+ for trajectory, - for ski jump),

V = longitudinal velocity (Q divided by water area), and

θ = angle of inclination of floor profile.

Where practicable, the spreading transition should start at the beginning of the trajectory, and end at the beginning of the stilling pool.

Trajectory

C. The trajectory may be determined from the following equations:

$$Y = X \tan\theta_0 + \frac{(\tan\theta_L - \tan\theta_0)\,X^2}{2L_t} \qquad (1)$$

$$K = \frac{(\tan\theta_L - \tan\theta_0)\,2h_v \cos^2\theta_0}{L_t} \qquad (2)$$

where

X = horizontal length from origin to a point on the trajectory,

Y = vertical fall from origin to point X on the trajectory,

L_t = horizontal length from origin to end of the trajectory,

θ_0 = angle of inclination of chute channel at the origin of the trajectory,

θ_L = angle of inclination of chute channel at the end of the trajectory,

$h_v = \frac{V^2}{2g}$ computed at origin of trajectory using Manning's "n" = 0.010, and

K = 0.5 or less. (Vertical curvature reduces hydrostatic pressure on the chute channel floor; limiting K to 0.5 or less assures positive pressures on the floor.)

A convenient value of L_t can be selected using Equation (2) above, which will limit K to 0.5 or less. Values of Y can then be computed. The trajectory should end at or upstream of the intersection of the chute channel walls with the stilling pool walls.

5.17D

FLUMES, CHUTES AND DROPS--Continued

Trajectory (Cont'd.)

The above trajectory equations may also be applied to a ski jump by considering Y to be the vertical fall or rise from the origin to point X on the upward trajectory, and by assigning proper algebraic signs to θ_0 and θ_L. Also, Manning's "n" = 0.014 should be used for computing K on a ski jump.

Chute Wall Height and Freeboard

D. When determining wall heights along the chute channel to the beginning of the trajectory or spread transition, depths should be computed using Manning's "n" = 0.014 and a stepwise procedure as illustrated in King's Handbook of Hydraulics. The critical depth, d_c, should be computed using the maximum capacity and the chute channel width. Minimum freeboard, Fb_c, should be added to the depths computed above, or to 0.4 d_c, whichever is greater, to determine minimum chute channel wall heights.

Depths and freeboard are normal to the chute channel slope. Air entrainment, which will occur with velocities exceeding 30 feet per second, is allowed for by using a depth not less than 0.4 d_c. Horizontal curves in a chute channel, if necessary, will require additional freeboard to provide for superelevation of water surface. Superelevation of the floor may also be provided. The minimum freeboard, Fb_c, for a chute channel is:

Capacity, Q(cfs)	Fb_c (inches)
100 or less	12
101 to 500	15
501 to 1000	18
Over 1000	24

Stilling Pool

E. The design of stilling pools for chutes and inclined drops is discussed in Paragraphs 5.20 through 5.23.

VERTICAL DROPS

.18 Vertical drops are often used in canals and laterals to dissipate a few feet of energy. The maximum vertical drop in water surface is about 3 feet for Q = 70 cfs or less, and 1.5 feet for larger Q for this type of structure, except where hard-surface lining or paving is provided downstream from the structure. Figure 27 shows a typical vertical check drop. Vertical drops of 8 feet in water surface have been used with satisfactory operation in concrete-lined canals; however, the maximum drop should normally be about 6 feet. A check structure should normally be combined with the vertical drop to prevent drawdown and scour upstream from the structure.

Dimensions

A. Some of the pool dimensions are very important for satisfactory operation. The downstream water surface must be lower than 0.6 times the upstream critical depth to permit the jet to plunge and thus prevent excessive waves downstream from the structure. Figure 28 shows the controlling dimensions, and the designs should use a pool length within 6 inches of those determined by this information, except for capacities of less than 100 cfs where some additional length may not cause objectionable waves. Percolation may require extra length or cutoffs on this type of structure, but the extra length is not desirable in the pool section.

CLOSED-CONDUIT DROPS

.19 Different types of closed-conduit drops are used in canals and laterals. The type generally used consists of an inlet transition with check, a reinforced concrete pipe, and an outlet transition or baffled outlet. This structure can provide a road crossing as well as an energy dissipator. Figure 29 shows two types of pipe drops.

Type 1 pipe drop is generally used where the pipe can be sumped sufficiently to create a hydraulic jump for dissipating excess energy in the pipe at its lowest elevation or upstream thereof. Pressure surges and air entrainment

FLUMES, CHUTES AND DROPS--Continued

CLOSED-CONDUIT DROPS (Cont'd.)

should be considered in the design, and a long downslope reach of pipe should be checked with Figure 12 of Chapter 2 to insure satisfactory operation. Design of the outlet is similar to that for a comparable siphon. The capacity of a Type 1 structure for continuous operation is usually based on a velocity of 5 feet per second with pipe flowing full.

Type 2 pipe drop is generally employed where it would be impracticable to sump the pipe as required for Type 1. Excess energy dissipation must then be accomplished at the outlet with either a baffled outlet as shown or a stilling pool. Velocities up to 50 feet per second have been allowed in the downslope reach of pipe, provided that the size of pipe is sufficiently large to limit the velocity to 12 feet per second when the pipe is flowing full.

STILLING POOLS

GENERAL

.20 Stilling pools are usually required to dissipate the excess energy of the water at the downstream ends of chutes and inclined drops. They are designed to produce a hydraulic jump and to contain the jump to the extent that turbulence of the water at the outlet will not cause erosion damage to the downstream canal or channel. The following aids to design are applicable to stilling pools where the Froude number $\left(F_1 = \frac{V_1}{\sqrt{gd_1}}\right)$ is between 4.5 and 15. Special consideration and in some cases model studies are required for pools having Froude numbers outside of this range. The discharge per foot of width is usually limited to a maximum of 200 cfs for stilling pools used in canals and wasteways. Flared pools are sometime employed and require special design consideration when used.

For inclined drop structures (where the vertical fall in energy gradient is less than 15 feet) the table in Figure 18 of Chapter 2 or the following formula may be used to determine the depth at the downstream end of the hydraulic jump:

$$d_2 = \frac{-d_1}{2} + \sqrt{\frac{2V_1^2 d_1}{g} + \frac{d_1^2}{4}} \qquad (3)$$

"F" is the vertical fall from the normal upstream energy gradient to the minimum downstream energy gradient. The minimum downstream energy gradient is usually determined as follows:

A. When the stilling pool discharges into an uncontrolled channel, a control must be provided by the outlet structure and critical depth at the control section should be used to determine the downstream energy gradient.

B. When the stilling pool discharges into a controlled channel, nonerodible or with downstream control, the channel depth as computed by reducing the assumed "n" value for the channel by 20 percent should be used for determining the minimum downstream energy gradient for full design capacity.

For chute structures with constant width stilling pool, Figure 19 of Chapter 2 or Equation (3) above may be used to determine d_2. The downstream energy gradient should be determined as in Subparagraph A above. The upstream energy gradient should be the computed depth plus the velocity head in the chute at the intersection of the chute floor and the assumed invert of the stilling pool. Several trial computations are usually necessary to obtain confirmation of d_2 and pool invert elevation as required by the downstream energy gradient.

STILLING POOLS--Continued

LENGTH AND FREEBOARD .21 Where use of the stilling pool is to be intermittent and for short duration of flow, such as in most wasteways or structures carrying floodwater, the minimum pool length should be $3d_2$. Where uninterrupted use or long duration of flow is expected, the minimum pool length should be $4d_2$. The pool length should be measured as shown on Figure 24. No portion of an outlet transition should be included in the minimum pool lengths given above. The curve shown on Figure 24 may be used as a guide for determining freeboard in stilling pools. This freeboard is considered to be the height above the normal downstream energy gradient.

CHUTE AND POOL BLOCKS .22 Chute and pool blocks should be provided to break up jet flow and stabilize the hydraulic jump in a stilling pool. The usual size, shape, spacing, and location of chute and pool blocks are shown on Figure 24.

OUTLET .23 The cross-sectional water area at the downstream end of the outlet structure should be equal to that of the canal downstream, or should be sufficient to produce a safe noneroding velocity for the channel. The length and shape of an outlet transition should be set to provide the water surface angles and hydraulic requirements described in Chapter 2, except that no loss is assumed for the transition.

BAFFLED APRONS

GENERAL .24 Baffled aprons are particularly useful for dissipating energy where the downstream control may change, but they are also suitable for other locations. When considered for use in canals or laterals where the downstream channel is controlled, cost comparisons should first be made with other types of drop structures such as rectangular inclined drops, pipe drops, and chutes. Figure 30 shows structures designed for about 36 cfs per foot of width adjusted to provide partial blocks against the wall. This illustration shows a notch control on the inlet to reduce upstream scour. When the crest of the apron is higher than the upstream channel, a slot should be provided through the crest for drainage. The inlet may be level if the upstream channel is sufficiently stable to prevent erosion.

DIMENSIONS .25 The following steps are suggested as guides to be used in setting the dimensions of baffled aprons:

(1) Set the longitudinal slope of the floor and side walls at 2:1.

(2) Approximate width of structure should be set by the relation,

$$w = \frac{Q}{q}$$

where

w = width,
Q = maximum total discharge, and
q = allowable discharge per foot of width

(3) Allowable q may be as much as 60 cfs per foot when the total discharge is over 1,000 cfs. See table of dimensions for suggested ranges of discharge per foot for various total discharges.

(4) Baffle blocks:

a. The first row of baffles should be placed so that the upstream face of the base is at the downstream end of the circular curve and no more than 12 inches in elevation below the crest.

BAFFLED APRONS--Continued

DIMENSIONS (Cont'd.)

b. Baffle block height, H, should be 0.9 of critical depth, d_c, to nearest inch.

c. Baffle block widths and spaces should be equal, and should be between H and 1-1/2 H. Partial blocks, width 1/3 H to 2/3 H should be placed against the sidewalls in row 1, 3, 5, 7, etc. Alternate rows of baffle blocks should be staggered so that each block is below a space in the row above. Adjust structure width found in step (2) above so that convenient baffle block widths can be used.

d. The slope distance between rows of baffle blocks should be 2 H, but no greater than 6 feet.

(5) The entrance velocity at the beginning of the chute should not exceed the critical velocity,

$$V_c = \sqrt[3]{gq}$$

(6) A minimum of four rows of baffle blocks should be used. The baffled apron should be extended so that the top of at least one row of baffle blocks will be below the bottom grade of the outlet channel as established by the applicable paragraph below. The apron should be extended below the last row of blocks a distance equal to the clear space between blocks.

a. The channel grade may be controlled by a downstream structure, by geologic formation, or by being on a stable slope for the capacity in question. A slope of 0.0018 will usually be stable for storm water flows, but for canal water the assumed slope should be no steeper than that of a canal in the same material.

b. Cross drainage storm flows may discharge on slopes of erodable material where no downstream controls exist or are provided. In such locations the amount of slope material that can be transported by one design storm should be computed. Also, the downstream channel, after the design storm, that will result from the eroded material being removed above a slope of 0.0018 should be computed. The bottom row of blocks should be set below this computed channel invert elevation. When scour has occurred, the apron and walls should be extended to provide for future storms by exposing the reinforcement in the end of structure and bonding new extension to the original installation.

(7) Gravel or riprap should be provided on each side of the structure from the top of the slope to the downstream channel. This protection above the maximum downstream water level is to prevent splash erosion. Below this water level the protection is required to prevent erosion by side rollers. Wing walls (see Figure 30) hold the slope protection in place. Channel protection below the structure should be in accordance with Paragraph 2.32. Rockfill at the bottom of the apron is usually unnecessary.

(8) The baffle blocks are usually constructed with their upstream faces normal to the chute floor. The longitudinal thickness of the baffle blocks at the top may vary from 8 inches for the smaller structures to 10 inches for the larger structures.

(9) Suggested height of the walls to provide adequate freeboard is three times the baffle pier height measured normal to the chute floor. It is

5.26

BAFFLED APRONS--Continued

DIMENSIONS (Cont'd.)

generally not feasible to set the freeboard for these structures to contain all of the spray and splash.

(10) The slope stability or tendency of the baffled apron to slide down the 2:1 slope should be checked. This is particularly important in channels where erosion may remove the support at the downstream toe of the apron. In model studies performed on baffled aprons, piezometer readings on the baffle blocks have indicated an average net water pressure on the blocks in the downstream direction of between 4 and 5 feet of water. This is equivalent to a force of between 250 and 310 pounds per square foot of block area in the downstream direction parallel to the apron slope.

Following is a tabulation of baffled apron dimensions:

Capacity (cfs)	*Discharge per foot of chute width (cfs)	Chute width (feet)	d_c (feet)
0 to 39	5 to 10	4	0.92 to 1.46
40 to 99	10 to 15	4 to 7	1.46 to 1.91
100 to 189	15 to 20	7 to 9.5	1.91 to 2.32
190 to 459	20 to 30	9.5 to 15	2.32 to 3.04
460 to 999	30 to 50	15 to 20	3.04 to 4.27
1,000 and up	50 to 60	20 and up	4.27 to 4.82

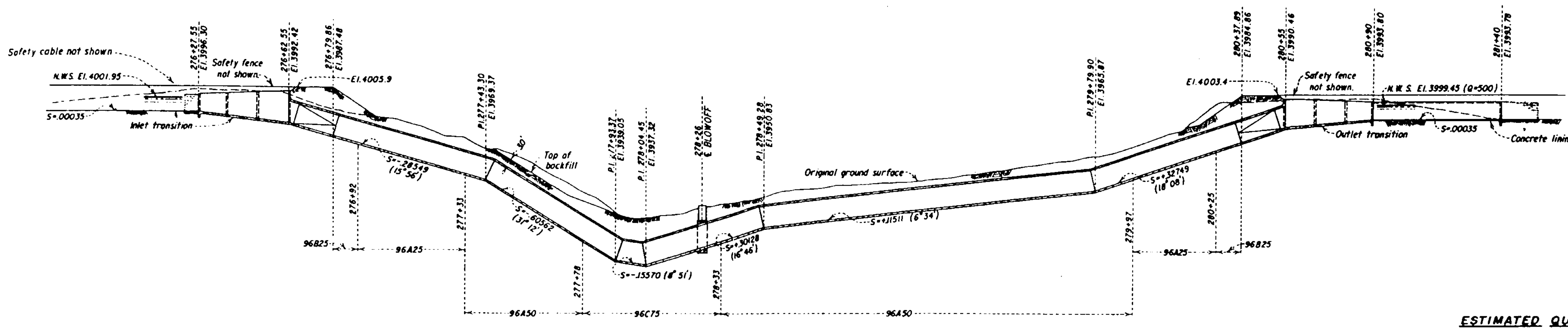
*Discharge per foot should be increased proportionally to capacity within range indicated

TURNOUTS

GENERAL .26 Turnouts divert water from a main water supply channel to a smaller channel or a farm irrigation ditch. The large turnouts are usually designed as open channels with a bridge, while the smaller structures usually have a covered conduit. Radial or slide gates are generally used to control the amount of water diverted through the turnouts. The structure must be large enough and set low enough to carry the required flow from a checked water surface in the main channel. Water measurement is usually required at turnouts; therefore, constant-head orifice structures, open flowmeters, line meters, Parshall flumes, or weirs are often combined with turnout structures. Except for the inlet, a closed-conduit turnout is usually the same type of structure as an inverted siphon or a closed-conduit drop.

TWO MEDICINE SIPHON--PRECAST CONCRETE PIPE BARREL

Fig. 1 Par. 5.3A



PROFILE

SCALE OF FEET

ESTIMATED QUANTITIES

Concrete .... 157 Cu. Yds.
Reinforcement steel .... 26,100 Lbs.
Precast concrete pressure pipe:
- 96A25 .... 78 Lin. Ft.
- 96B25 .... 21 Lin. Ft.
- 96A50 .... 217 Lin. Ft.
- 96C75 .... 59 Lin. Ft.

NOTES

Stations and elevations refer to invert, unless otherwise shown.
Stations and elevations of pipe bends, and of changes in classes of pipe, are approximate and may be adjusted as directed to fit actual laying length of pipe.
All pipe bends shall be encased mitered bends.
Place entire structure on a foundation of undisturbed earth or thoroughly compacted fill.
For details of pipe, see Table 1, of the specifications.
Pipe symbols 96A25, 96A50, etc., shown on profile represent classes of pipe; for explanation, see related paragraphs.
For logs of geologic exploration, see 2-D-80.

REFERENCE DRAWINGS

GENERAL PLAN AND CANAL SECTIONS 2-D-79
TRANSITIONS 2-D-84
BLOWOFF 2-D-86
BENDS FOR PRECAST CONCRETE PRESSURE PIPE 2-D-87
PIPE EARTHWORK DETAILS 2-D-88
SAFETY FENCE AND SAFETY CABLE 2-D-89

7-31-67 D-837	AS BUILT BY 604, LTR. 7-6-67

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BLACKFEET INDIAN IRRIGATION PROJECT—MONTANA
TWO MEDICINE CANAL—STA. 276+27.55
TWO MEDICINE SIPHON
PRECAST CONCRETE PIPE BARREL

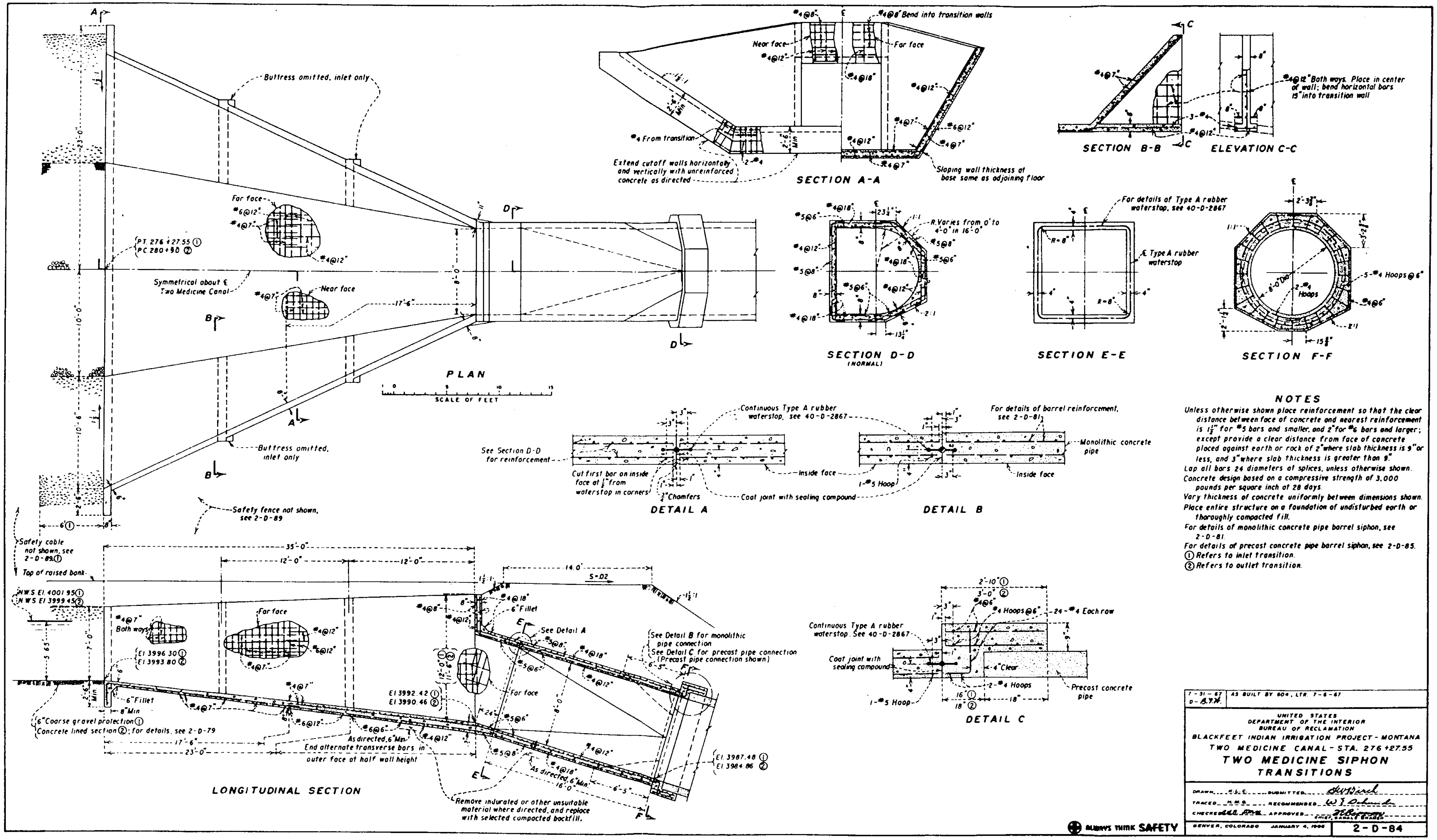
DRAWN K.L.C. SUBMITTED H.W. Bird
TRACED J.W.M. RECOMMENDED W.T. Schmidt
CHECKED G.M.A. APPROVED [illegible] CHIEF, CANALS BRANCH

ALWAYS THINK SAFETY

DENVER COLORADO JAN. 4, 1966	2-D-85

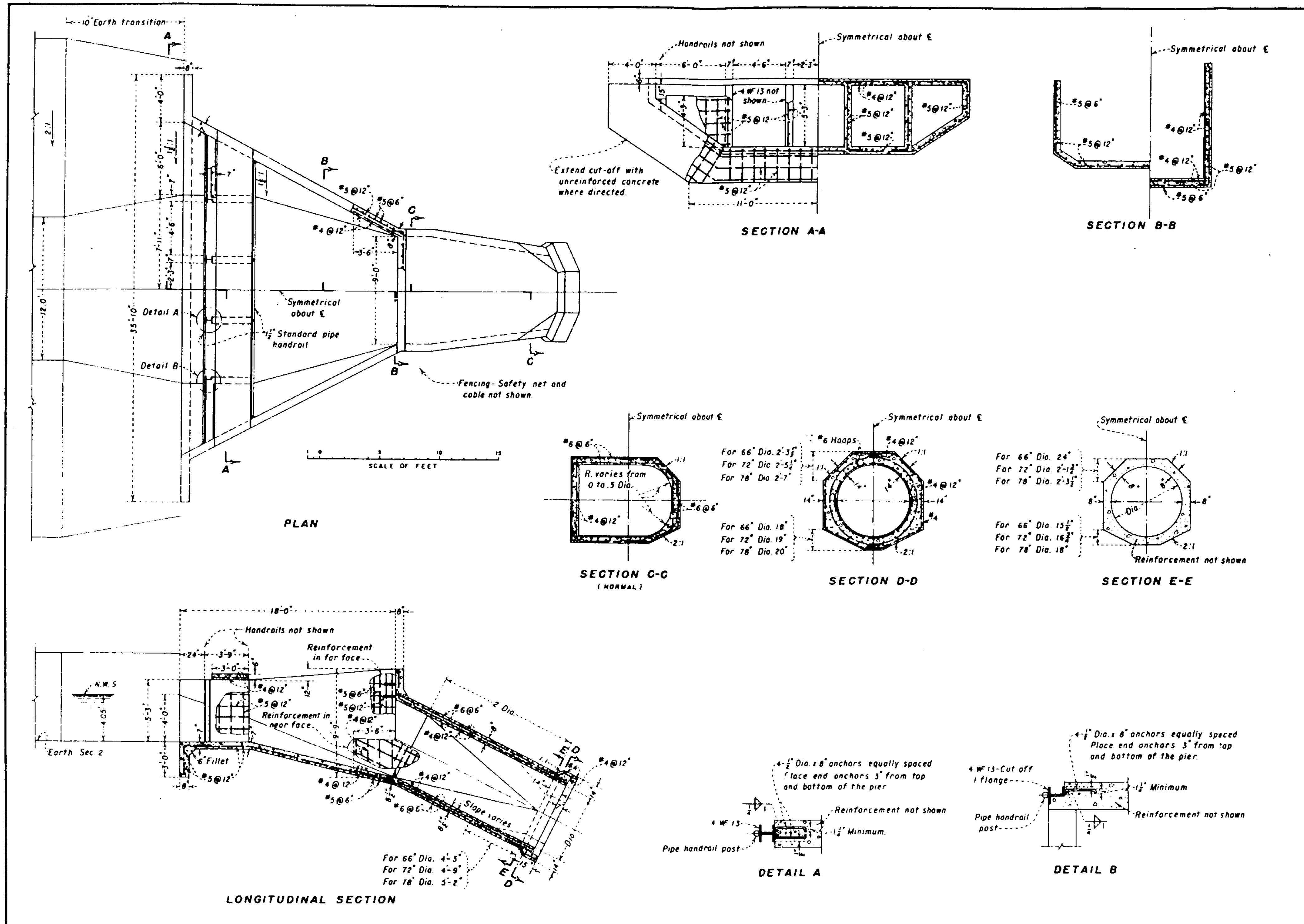
TWO MEDICINE SIPHON--TRANSITIONS

Fig. 2 Par. 5.3A



TYPICAL INLET TRANSITION WITH CHECK--66-, 72-, AND 78-INCH PIPE DIAMETERS

Fig. 3 Par. 5.3A



ESTIMATED QUANTITIES

SIZE	CONCRETE	REINFORCEMENT
66"	27 Cu. yds.	5100 Lbs.
72"	29 Cu. yds.	5500 Lbs.
78"	31 Cu. yds.	5900 Lbs.

NOTES

Unless otherwise shown, place reinforcement so that the clear distance between outside face of concrete and nearest reinforcement is 1½" inches, except that where concrete is placed against earth or rock the minimum clear distance shall be 2 inches.

Lap all bars 24 diameters at splices.

Monolithic concrete design based on a compressive strength of 3000 pounds per square inch at 28 days.

REFERENCE DRAWINGS

PIPE HANDRAIL POST CONNECTION DETAIL 304-D-154
HANDRAIL DETAILS 40-D-4315
FENCING-SAFETY NET AND CABLE-
TYPICAL DETAILS 304-D-150

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
EMERY COUNTY PROJECT-UTAH

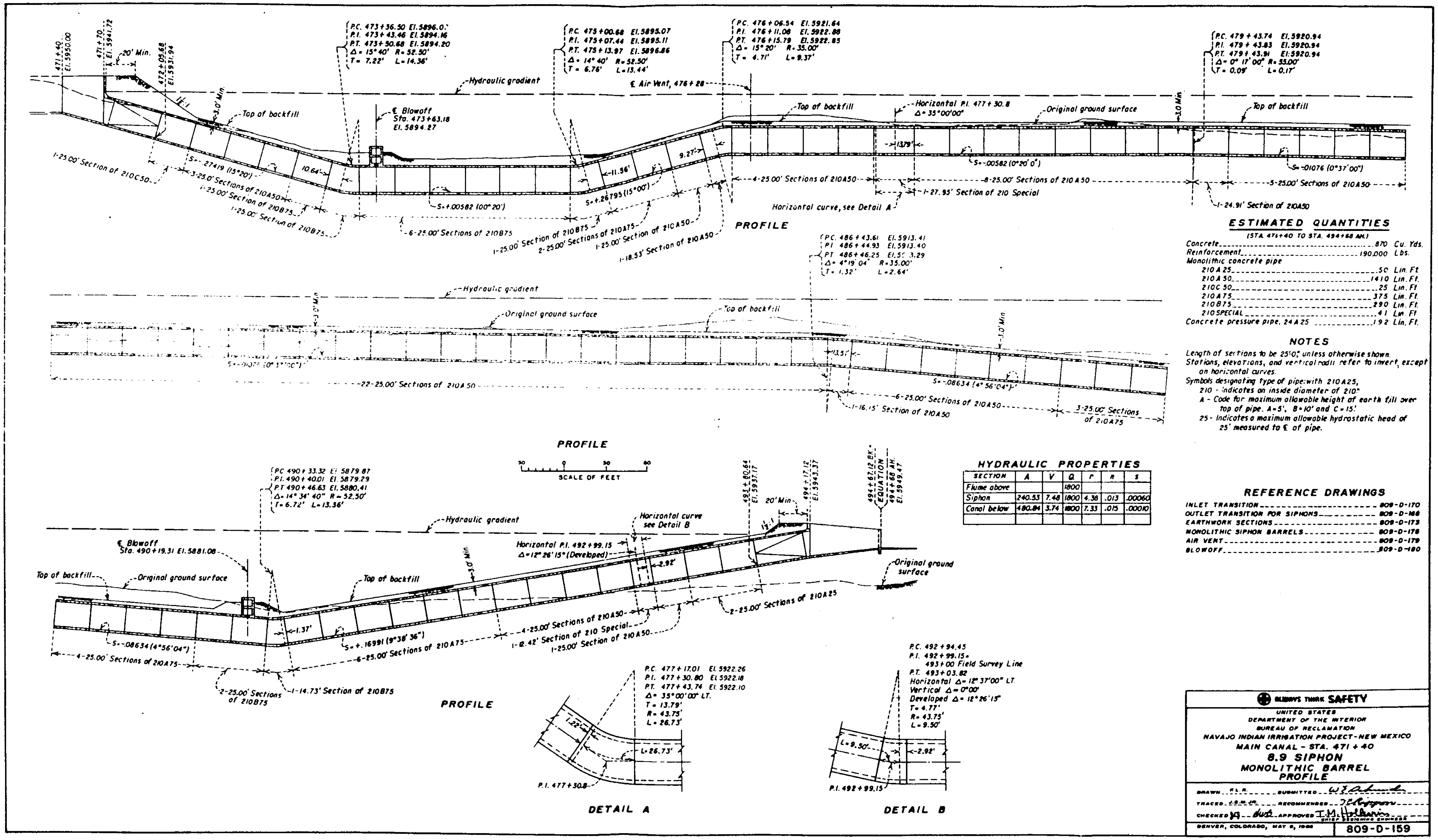
COTTONWOOD CREEK-HUNTINGTON CANAL
INLET TRANSITION WITH CHECK
66", 72" AND 78" PIPE DIAMETERS

DRAWN... H.J.W. ... SUBMITTED ... W. T. [signature]
TRACED ... W.A.W. ... RECOMMENDED ... A. H. Terrell
CHECKED ... W.W. ... APPROVED ... [signature]
CHIEF DESIGNING ENGINEER

DENVER, COLORADO. AUGUST 16, 1963 | 304-D-126

8.9 SIPHON, MAIN CANAL, NAVAJO INDIAN IRRIGATION PROJECT--MONOLITHIC BARREL PROFILE

Fig. 4 Par. 5.3A



HYDRAULIC PROPERTIES

SECTION	A	V	Q	r	n	s
Flume above			1800			
Siphon	240.53	7.48	1800	4.36	.013	.00060
Canal below	480.84	3.74	1800	7.33	.015	.00010

ESTIMATED QUANTITIES

(STA. 471+40 TO STA. 494+68 AH.)

Concrete	870	Cu. Yds.
Reinforcement	190,000	Lbs.
Monolithic concrete pipe		
210 A 25	50	Lin. Ft
210 A 50	1410	Lin. Ft.
210 C 50	25	Lin. Ft
210 A 75	375	Lin. Ft.
210 B 75	290	Lin. Ft.
210 SPECIAL	41	Lin. Ft
Concrete pressure pipe, 24 A 25	192	Lin. Ft.

NOTES

Length of sections to be 25'-0" unless otherwise shown.
Stations, elevations, and vertical radii refer to invert, except on horizontal curves.
Symbols designating type of pipe: with 210A25,
210 - indicates an inside diameter of 210"
A - Code for maximum allowable height of earth fill over top of pipe. A = 5', B = 10' and C = 15'.
25 - Indicates a maximum allowable hydrostatic head of 25' measured to ℄ of pipe.

REFERENCE DRAWINGS

INLET TRANSITION	809-D-170
OUTLET TRANSITION FOR SIPHONS	809-D-168
EARTHWORK SECTIONS	809-D-173
MONOLITHIC SIPHON BARRELS	809-D-178
AIR VENT	809-D-179
BLOWOFF	809-D-180

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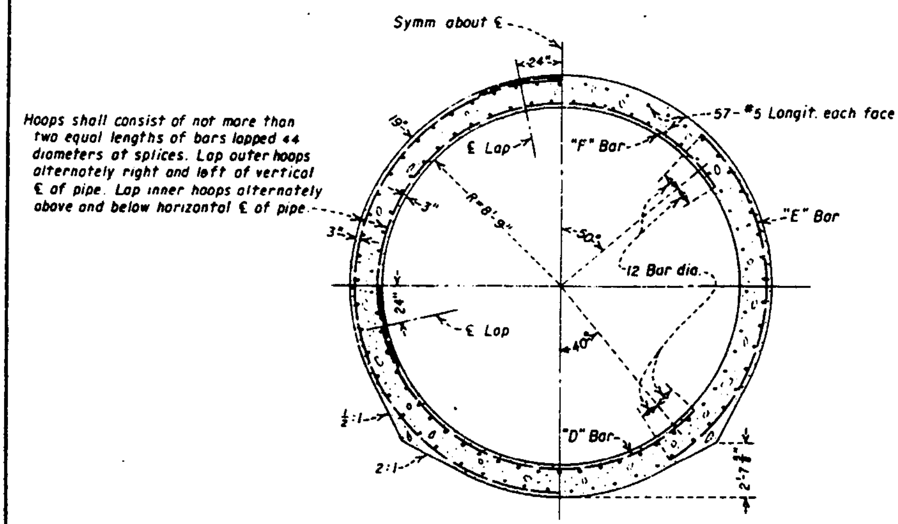
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
NAVAJO INDIAN IRRIGATION PROJECT-NEW MEXICO
MAIN CANAL - STA. 471 + 40
8.9 SIPHON
MONOLITHIC BARREL
PROFILE

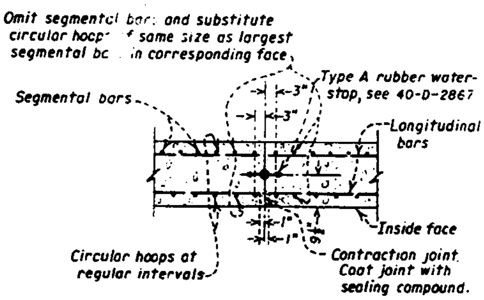
DRAWN E.L.R. SUBMITTED W.E. Ahrens
TRACED J.B.M. JR. RECOMMENDED J.E. Rippon
CHECKED JQ WA APPROVED I.M. Holtzman, CHIEF DESIGNING ENGINEER
DENVER, COLORADO, MAY 5, 1966 — 809-D-159

TYPICAL MONOLITHIC SIPHON BARRELS--HEADS 0 TO 100 FEET Fig. 5 Par. 5.3A

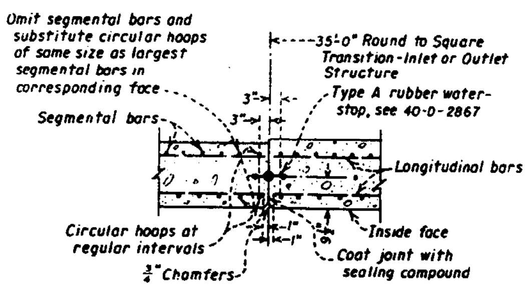


CIRCULAR HOOPS SEGMENTAL BARS

TYPICAL SECTION



CONTRACTION JOINT DETAIL



JOINT DETAIL
CONNECTION TO STRUCTURES

NOTES

Concrete design based on a compressive strength of 3000 lbs. per sq. inch at 28 days.
All dimensions to reinforcement are to centers of bars
Segmental bars to be placed midway between circular hoops
At bends place tranverse reinforcement in a fanlike manner so that the spacing is not exceeded at outside of curve.
Pipe classes are shown on the profiles as 210A50, 210B75, etc.
A, B, C and D designate allowable earth cover of 5, 10, 15 and 20 feet respectively; 25, 50, 75, etc. designate allowable heads in feet to ℄ of pipe.
Reinforcement bars shall conform to A.S.T.M. A432.

CIRCULAR REINFORCEMENT-TWO LAYERS					
CLASS	INNER REINFORCEMENT			OUTER REINF.	
	HOOP	SEGM. "D"	SEGM. "F"	HOOP	SEGM. "E"
210A25	#7@12"	#7@12"	#4@12"	#6@12"	#6@12"
A50	#8@10"	#6@10"	#4@10"	#6@5"	—
A75	#9@6½"	—	—	#7@5½"	—
A100	#10@5½"	—	—	#8@5½"	—
210B25	#8@12"	#8@12"	#4@12"	#7@12"	#6@12"
B50	#9@10"	#8@10"	#4@10"	#8@10"	#4@10"
B75	#11@10"	#4@10"	—	#7@5½"	—
B100	#10@5½"	—	—	#8@5½"	—
210C25	#8@10"	#8@10"	#4@10"	#7@12"	#7@12"
C50	#9@11"	#9@11"	#4@11"	#8@11"	#6@11"
C75	#11@10"	#6@10"	—	#8@6"	—
C100	#10@5½"	—	—	#8@5½"	—
210D25	#9@10½"	#9@10½"	—	#8@11"	#6@11"
D50	#10@11"	#9@11"	—	#8@11"	#7@11"
210 Spec.	#8@10"	#8@10"	#4@10"	#6@5"	—

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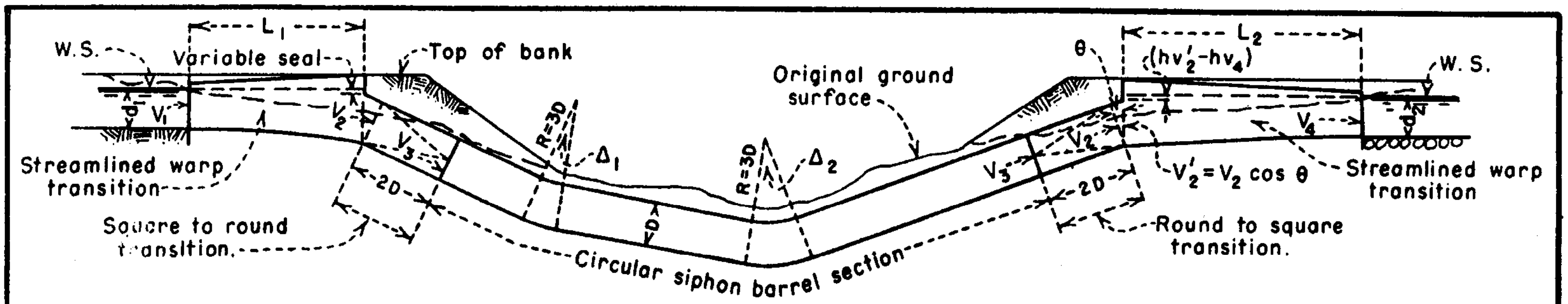
12-27-66 D JGB — INNER REINF. FOR 210A50 CHANGED FROM #8@12" TO #8@10". DELETED WELDING OF REINFORCEMENT.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
NAVAJO INDIAN IRRIGATION PROJECT-NEW MEXICO
MAIN CANAL
STA. 112+94 TO 128+00 AND 471+40 TO 712+50
MONOLITHIC SIPHON BARRELS
HEADS 0 TO 100 FEET

DRAWN A.W.D. SUBMITTED W. J. Schmidt
TRACED J.S.M. JR. RECOMMENDED
CHECKED R.S.C. McG. APPROVED T. M. Hollearin CHIEF DESIGNING ENGINEER
DENVER, COLORADO, MAY 9, 1966 — 809-D-175



PROFILE-TYPICAL SIPHON

EXAMPLE:

DIMENSIONS AND HYDRAULIC PROPERTIES

CANAL END OF INLET AND OUTLET TRANSITIONS

Base Width = 25.0'
Side slopes = $1\frac{1}{2}$:1
$d_1 = d_2 = 10.00'$
Q = 1000
A = 400.00
$V_1 = V_4 = 2.50$, $hv_1 = hv_4 = .097$
r = 6.55
n = .014
s = .00005-

SQUARE OPENING OF CLOSED TRANSITIONS

Size = 11.0' x 11.0'
Q = 1000
A = 121.00
$V_2 = 8.26$, $hv_2 = 1.061$
r = 2.75
n = .014
s = .00157

CIRCULAR SIPHON BARREL

Dia. = 11.0'
Q = 1000
A = 95.03
$V_3 = 10.52$, $hv_3 = 1.721'$
r = 2.75
n = .013 (Steel form finish or equal)
s = .00220
n = .014
s = .00255

COMPUTATION OF HEAD LOSSES

Inlet Open Transition (Friction)	$= 45'(L_1)\left(\frac{.00005+.00157}{2}\right)$	= .036'
Inlet Open Transition (Convergence)	$=^{*}.1\,[1.061\,(hv_2) - .097\,(hv_1)]$	= .096'
Inlet Closed Transition (Convergence)	$= .1\,[1.721\,(hv_3) - 1.061\,(hv_2)]$	= .066'
Closed Transitions (Friction)	$= 2 \times 22'\left(\frac{.00157+.00255}{2}\right)$	= .091'
Circular Barrel (Friction)	$= 160' \times .00220$	= .352'
Barrel Bend $\Delta_1 = 15°$	$=^{**}.027 \times 1.721\,(hv_3)$	= .046'
Barrel Bend $\Delta_2 = 30°$	$=^{**}.058 \times 1.721\,(hv_3)$	= .100'
Outlet Closed Transition (Divergence)	$= .2\,[1.721(hv_3) - 1.061(hv_2)]$	= .132'
Outlet Open Transition (Divergence)	$=^{*}.2\,[1.061\,(hv_2') - .097\,(hv_4)]$	= .193'
Outlet Open Transition (Friction)	$= 60'(L_2)\left(\frac{.00005+.00157}{2}\right)$	= .049'
Total Loss (Energy Gradient)		=1.161'
Add 10% ± for excess capacity		= .116'
Total Head required		=1.277'

*Coefficient used for computation of head loss in open transition should conform with type of transition selected.
**See Dwg. 106-D-44.
THIS DRAWING SUPERSEDES DWG 103-D-47

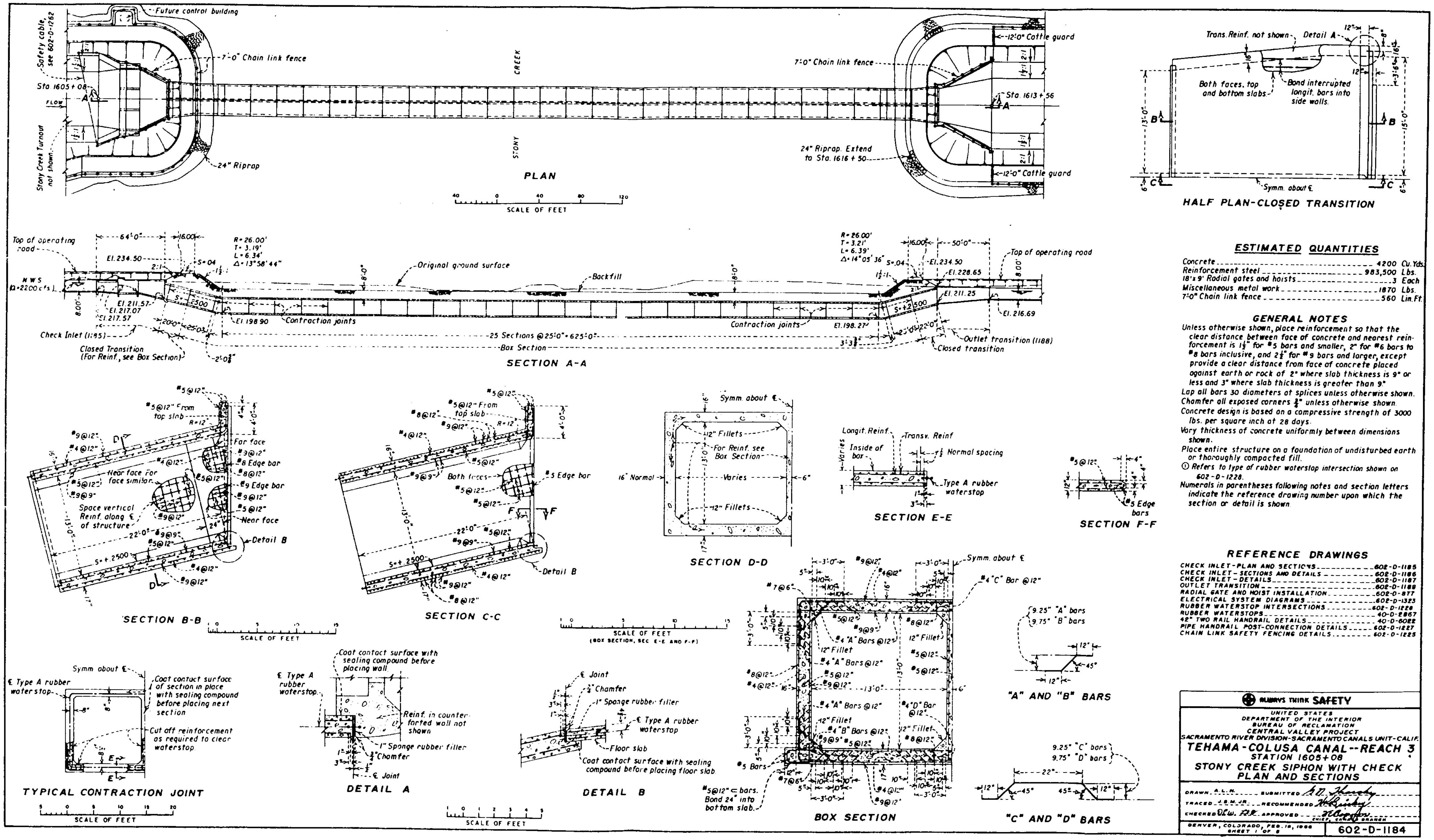
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DESIGN DATA

HEAD LOSS DETERMINATION FOR MONOLITHIC CONCRETE SIPHONS

DRAWN H.K.B. SUBMITTED [signature]
TRACED W.W.B. RECOMMENDED [signature]
CHECKED [initials] APPROVED [signature]
ASST. COMM. & CHIEF ENGINEER
DENVER, COLO. OCT. 10, 1960 103-D-633

STONY CREEK SIPHON WITH CHECK--PLAN AND SECTIONS

Fig. 6 Par. 5.3B



TYPICAL BLOWOFF FOR MONOLITHIC CONCRETE PIPE

Fig. 8 Par. 5.5A

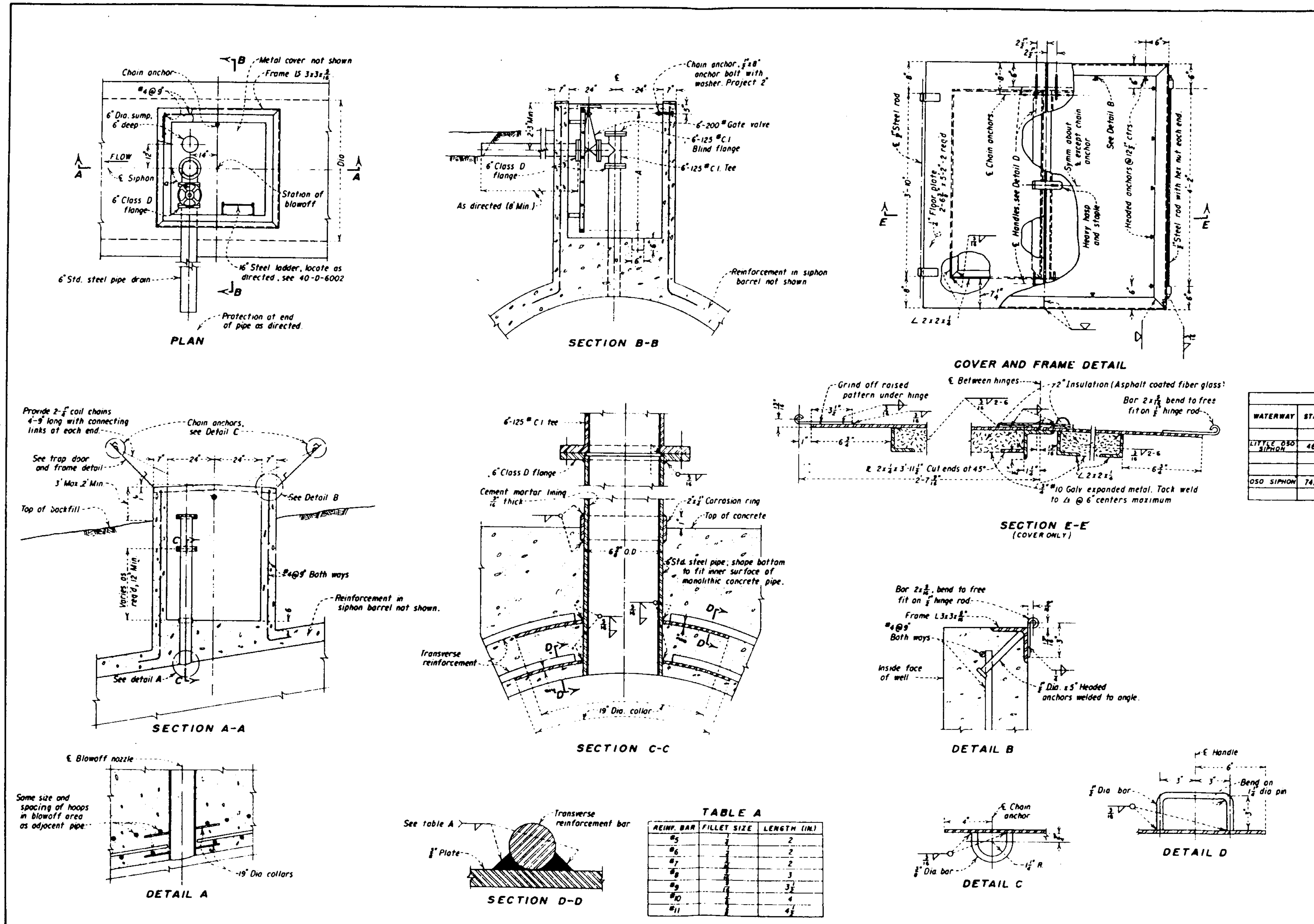


TABLE A

REINF. BAR	FILLET SIZE	LENGTH (IN.)
#5	[illegible]	2
#6	[illegible]	2
#7	[illegible]	2
#8	[illegible]	3
#9	[illegible]	3½
#10	[illegible]	4
#11	[illegible]	4½

DATA						EST. QUANTITIES		
WATERWAY	STATION	DIA.	θ	A (FT.)		CONC.	REINF. STEEL	MISC. METAL
LITTLE OSO SIPHON	465+40	96"	225°	8		4.5	450	550
OSO SIPHON	742+46	96"	225°	18		6.5	650	640

NOTES

Concrete design based on a compressive strength of 3,000 lbs. per sq. in. at 28 days.
Lap all bars 24 diameters at splices.
Place reinforcement in center of slab, unless otherwise shown.

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

SAN JUAN – CHAMA PROJECT, COLORADO – NEW MEXICO

LITTLE OSO AND OSO SIPHONS

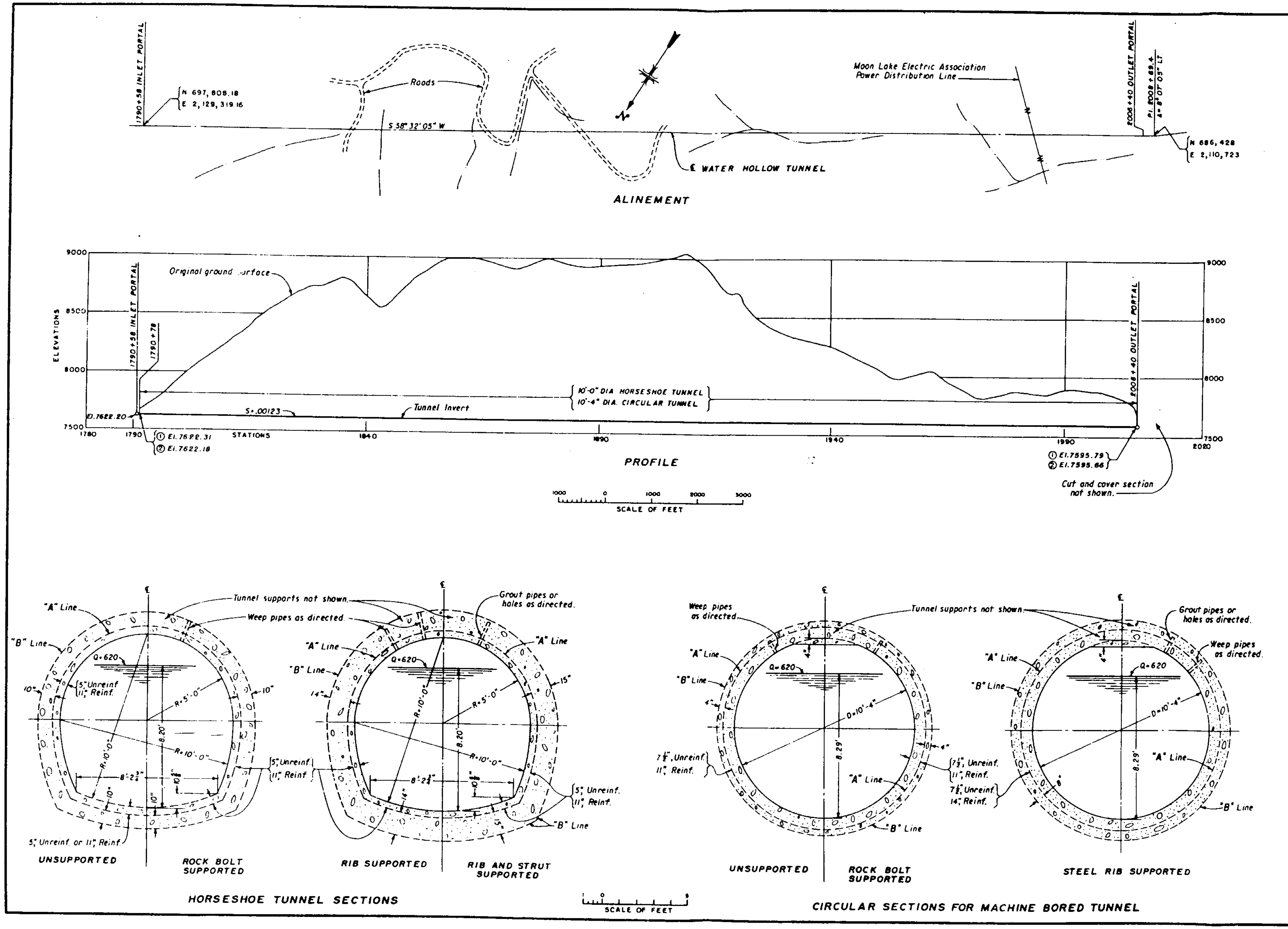
BLOWOFF
FOR MONOLITHIC CONCRETE PIPE

DRAWN ... SUBMITTED ...
TRACED ... RECOMMENDED ...
CHECKED ... APPROVED ... CHIEF CANALS BRANCH

DENVER, COLORADO AUG. 22, 1968 | 465-D-301

WATER HOLLOW TUNNEL--ALINEMENT, PROFILE AND SECTIONS

Fig. 9 Par. 5.6



EXCAVATION AND CONCRETE QUANTITIES TO "B" LINE

(Cubic yards per linear foot)

TUNNEL SECTION	EXC.	CONC.
HORSESHOE - 8" TO "A" LINE		
Unsupported	4.768	1.696
Rock bolt supported	4.768	1.696
Steel rib supported	5.282	2.210
Steel rib and strut supported	5.414	2.343
HORSESHOE - 11" TO "A" LINE		
Unsupported	5.548	2.477
Rock bolt supported	5.548	2.477
Steel rib supported	6.101	3.030
Steel rib and strut supported	6.244	3.172
CIRCULAR - 7½" TO "A" LINE		
Unsupported	4.365	1.289
Rock bolt supported	4.365	1.289
Steel rib supported	4.853	1.777
CIRCULAR - 11" TO "A" LINE		
Unsupported	4.791	1.715
Rock bolt supported	4.791	1.715
CIRCULAR - 14" TO "A" LINE		
Steel rib supported	5.701	2.626

HYDRAULIC PROPERTIES

SECTION	A	V	Q	r	n	s	D	d
Horseshoe	73.32	8.46	620	3.07	.013	.00123	10.00	8.20
Circular	72.10	8.60	620	3.14	.013	.00123	10.33	8.29

NOTES

Unreinforced tunnel sections shown, for details of reinforced tunnel sections, see 66-D-121 and -122.
Concrete design based on a compressive strength of 3,000 lbs. per sq. inch at 28 days.
Stations and elevations shown on profile refer to invert, unless otherwise shown.
Utah Coordinate System, Central Zone.
① Refers to elevation for horseshoe tunnel.
② Refers to elevation for circular tunnel.

REFERENCE DRAWINGS

INLET PORTAL 66-D-121
CUT AND COVER OUTLET TRANSITION 66-D-122
TUNNEL STATION MARKER DETAILS 66-D-124
PIEZOMETER AND FLOATWELL INSTALLATIONS 66-D-125
TYPICAL SUPPORTS FOR HORSESHOE TUNNEL 66-D-224
INLET AND OUTLET PORTAL AREAS 66-D-286
TYPICAL SUPPORTS FOR MACHINE BORED TUNNEL 66-D-258
OUTLET PORTAL AND CUT AND COVER SECTIONS 66-D-287

12-4-67 D- Jo8	TRACED. CHANGED ELEVATIONS OF TUNNEL IN ACCORDANCE WITH SUPP. NOTICE NO. 1
10-9-67 D- J.O.S.	DIMENSION FROM A TO B LINE CHANGED TO 8" FOR RIB SUPPORTED CIRCULAR SECTION

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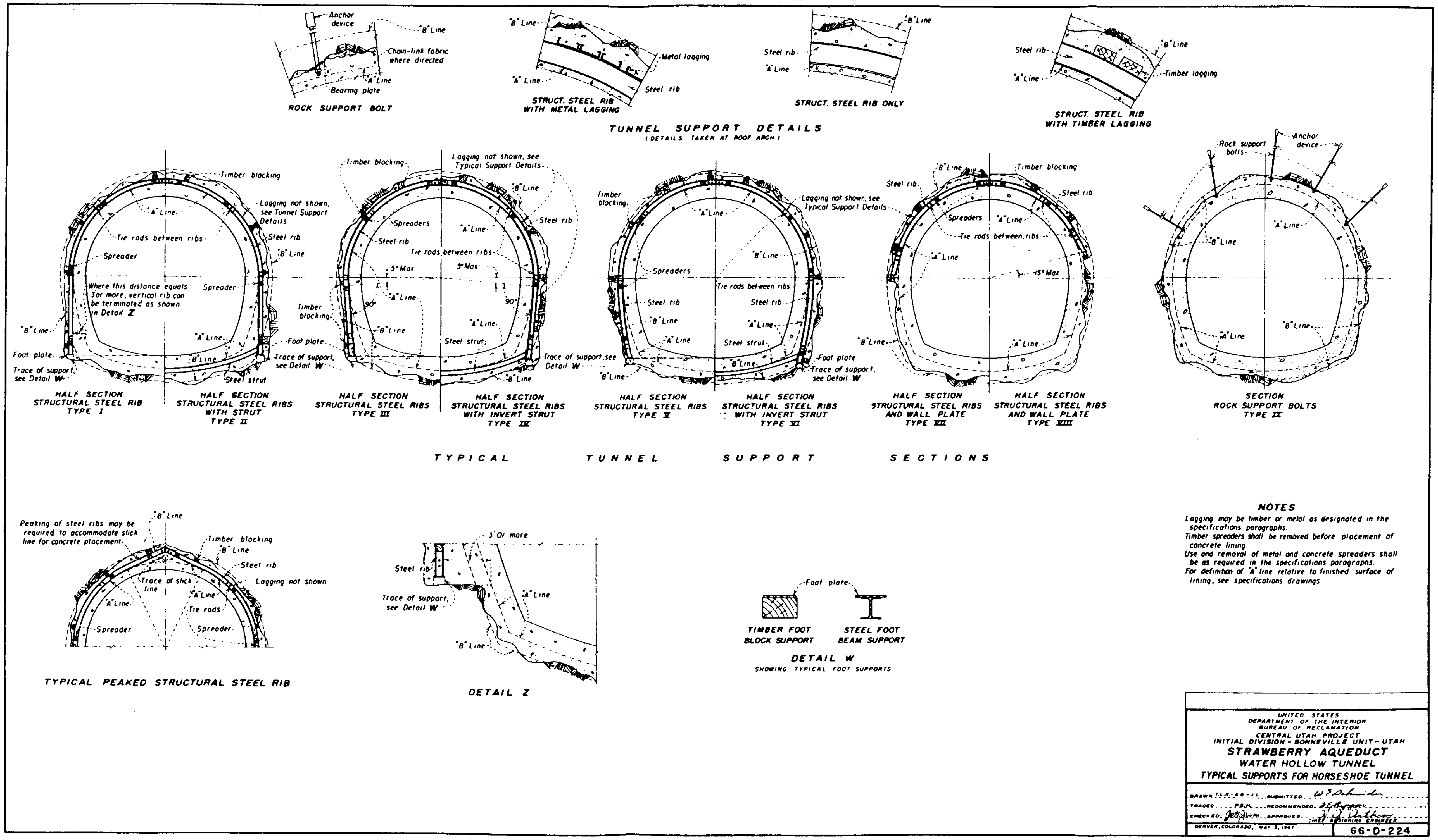
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL UTAH PROJECT
INITIAL DIVISION-BONNEVILLE UNIT-UTAH
STRAWBERRY AQUEDUCT
WATER HOLLOW TUNNEL
ALINEMENT, PROFILE AND SECTIONS

DRAWN F.L.R. SUBMITTED W. E. Schneider
TRACED J.S.M.JR. RECOMMENDED F. E. Rippon
CHECKED A.M.B.-J.O.S. APPROVED H. G. Arthur
CHIEF DESIGNING ENGINEER
DENVER, COLORADO, MAY 5, 1967
66-D-120

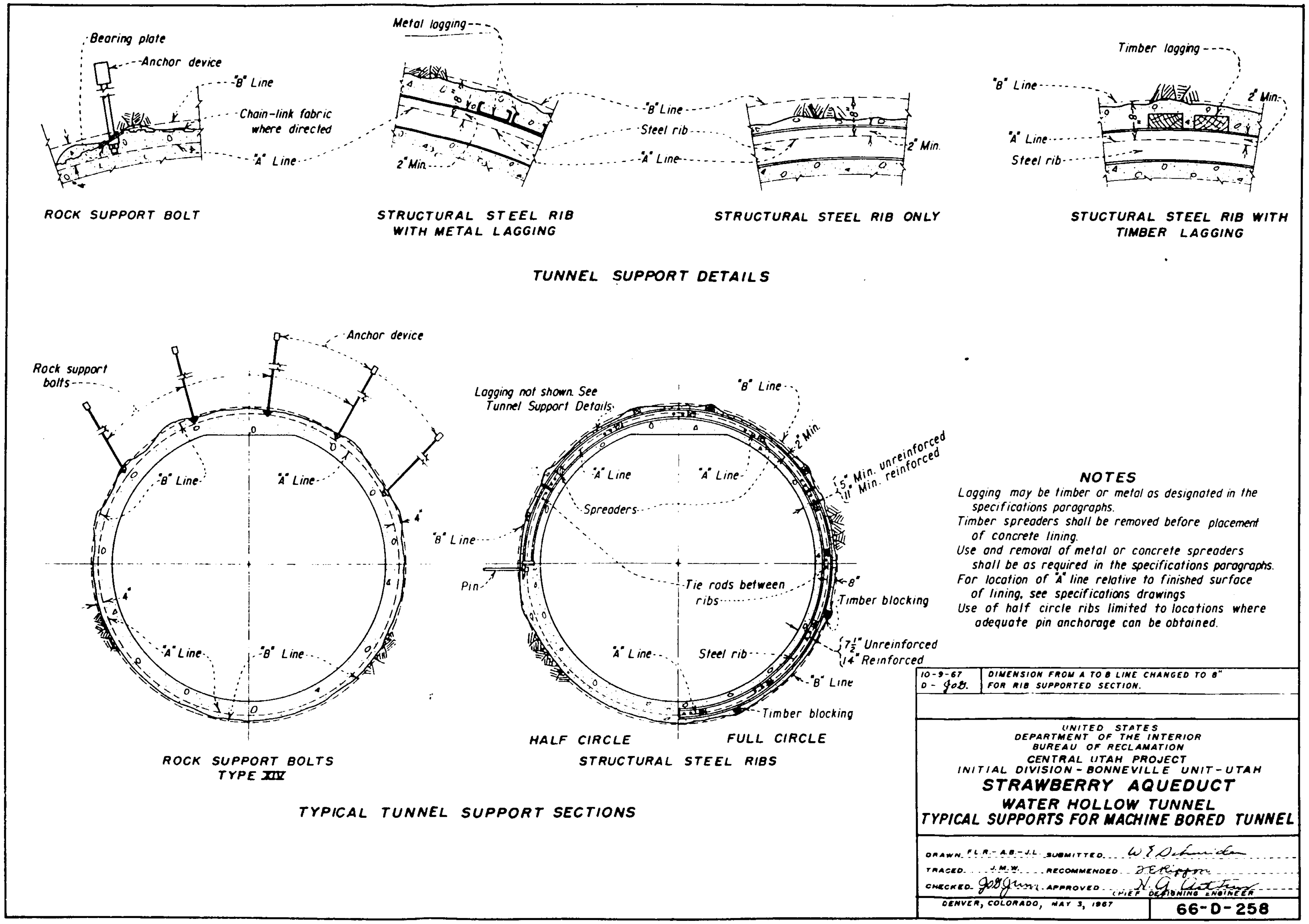
WATER HOLLOW TUNNEL--TYPICAL SUPPORTS FOR HORSESHOE TUNNEL

Fig. 10 Par. 5.6



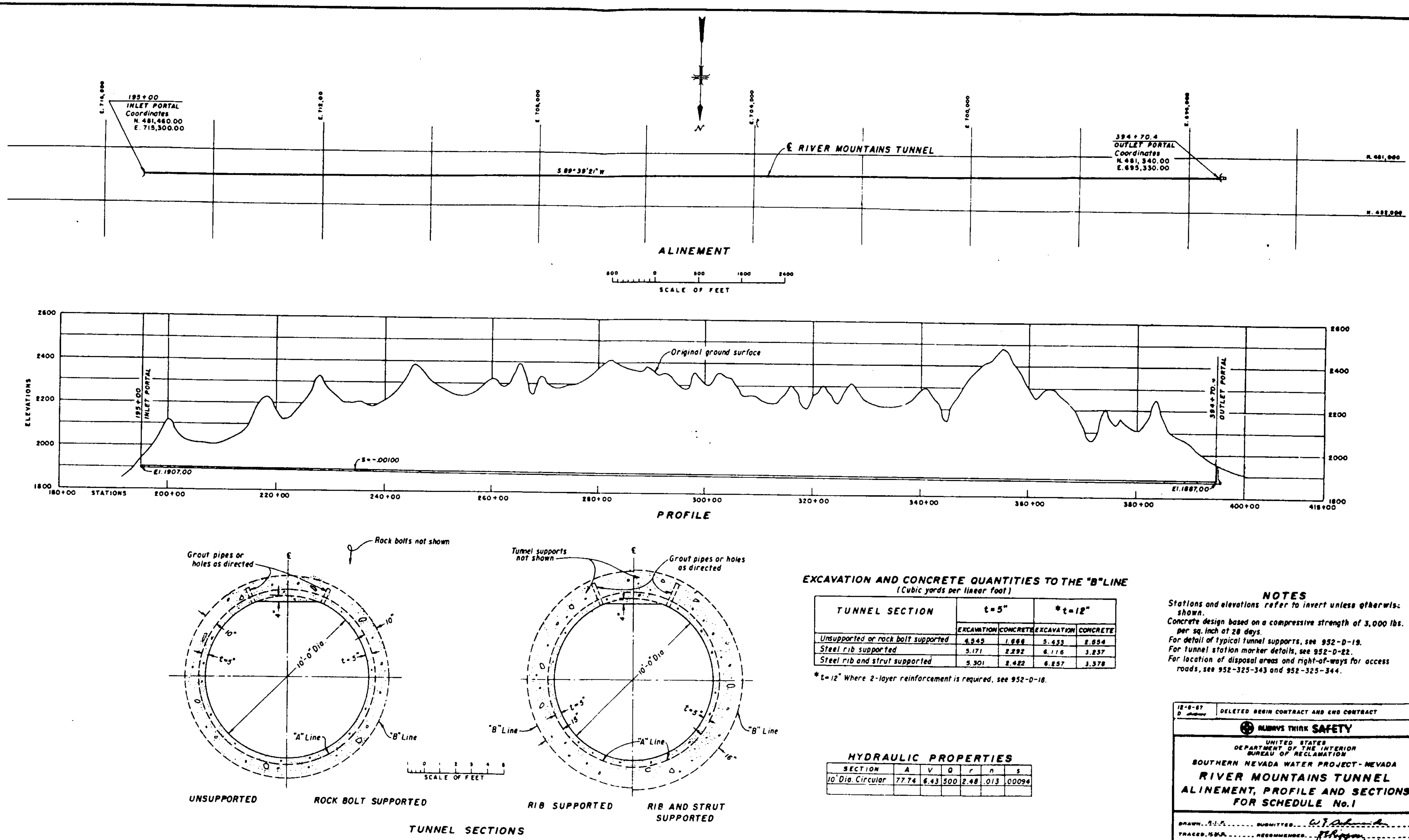
WATER HOLLOW TUNNEL--TYPICAL SUPPORTS FOR MACHINE-BORED TUNNEL

Fig. 11 Par. 5.6



RIVER MOUNTAINS TUNNEL--ALINEMENT, PROFILE, AND SECTIONS

Fig. 12 Par. 5.6



EXCAVATION AND CONCRETE QUANTITIES TO THE "B" LINE
(Cubic yards per linear foot)

TUNNEL SECTION	t=5"		*t=12"	
	EXCAVATION	CONCRETE	EXCAVATION	CONCRETE
Unsupported or rock bolt supported	4.545	1.666	5.433	2.654
Steel rib supported	5.171	2.292	6.116	3.237
Steel rib and strut supported	5.301	2.422	6.257	3.378

*t=12" Where 2-layer reinforcement is required, see 952-D-18.

HYDRAULIC PROPERTIES

SECTION	A	V	Q	r	n	s
10' Dia. Circular	77.74	6.43	500	2.48	.013	.00094

NOTES

Stations and elevations refer to invert unless otherwise shown.
Concrete design based on a compressive strength of 3,000 lbs. per sq. inch at 28 days.
For detail of typical tunnel supports, see 952-D-19.
For tunnel station marker details, see 952-D-22.
For location of disposal areas and right-of-ways for access roads, see 952-325-343 and 952-325-344.

12-8-67 D — DELETED BEGIN CONTRACT AND END CONTRACT

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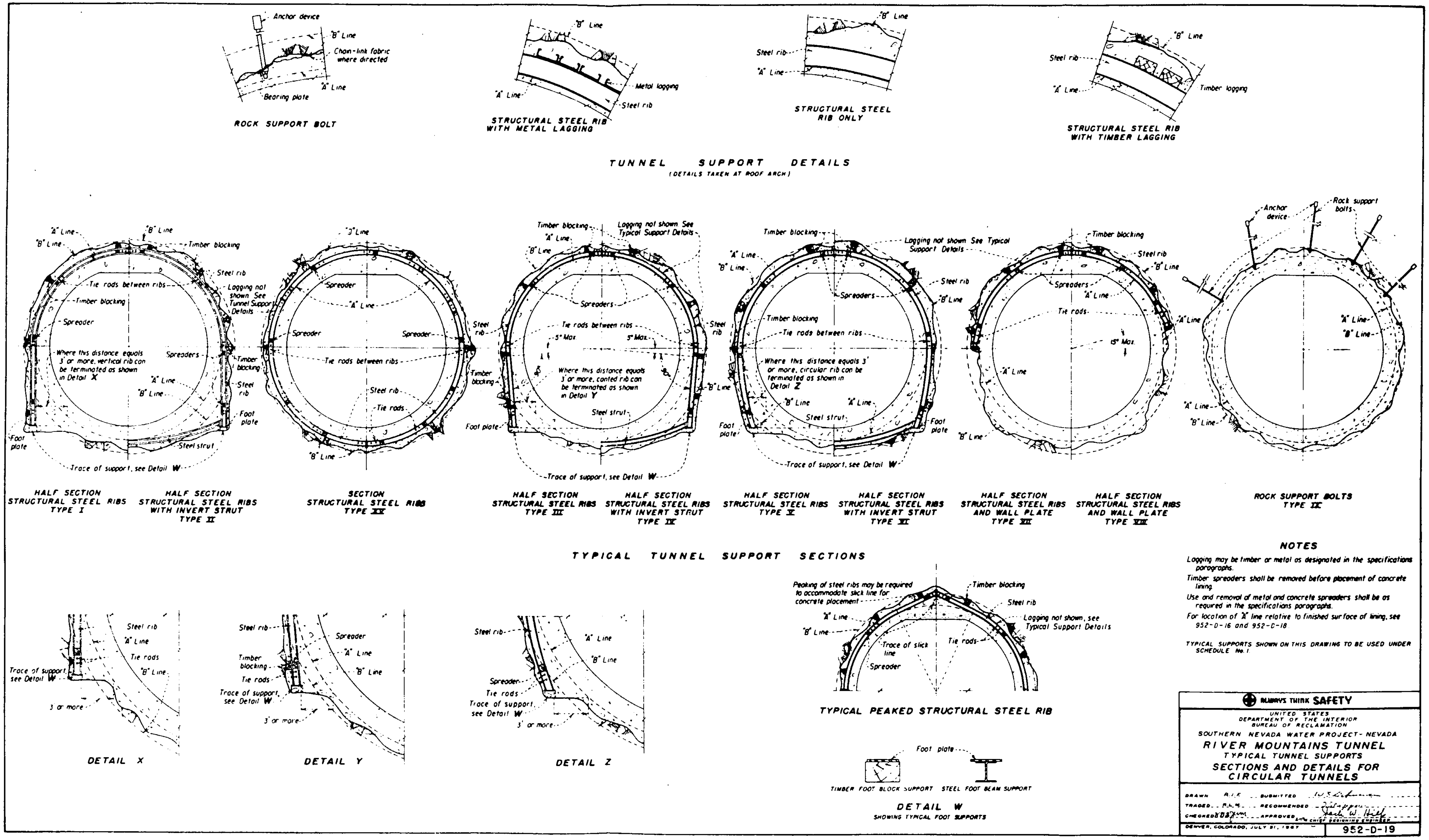
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
SOUTHERN NEVADA WATER PROJECT-NEVADA
RIVER MOUNTAINS TUNNEL
ALINEMENT, PROFILE AND SECTIONS
FOR SCHEDULE No. 1

DRAWN R.L.F. SUBMITTED W. J. [illegible]
TRACED [illegible] RECOMMENDED [illegible]
CHECKED [illegible] APPROVED Jack W. Hill
DENVER, COLORADO, JULY 31, 19[illegible] 952-D-16

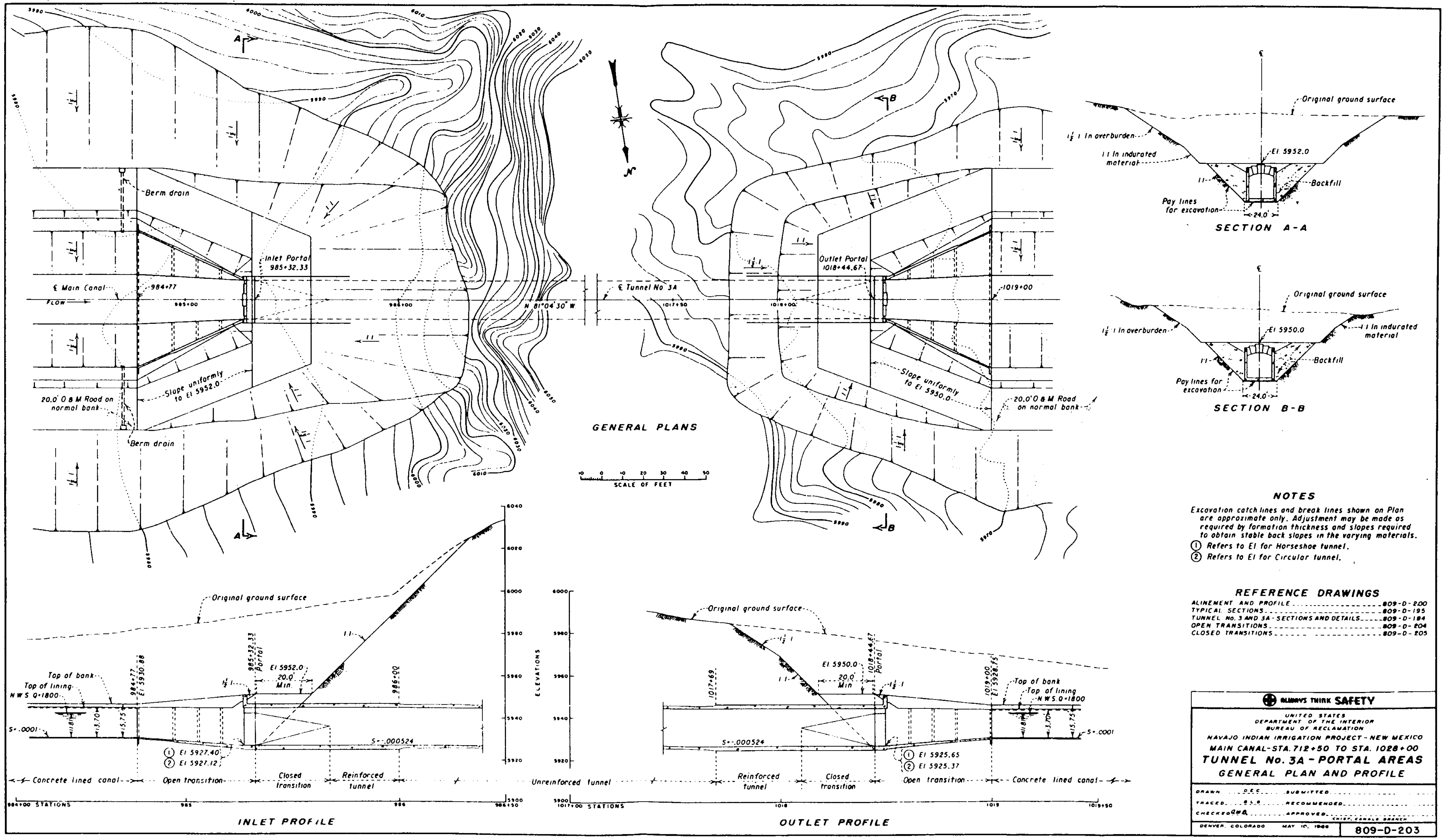
RIVER MOUNTAINS TUNNEL--SECTIONS AND DETAILS FOR CIRCULAR TUNNELS

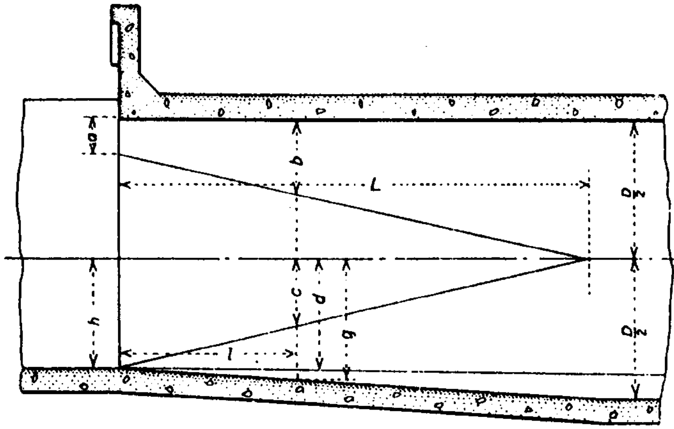
Fig. 13 Par. 5.6



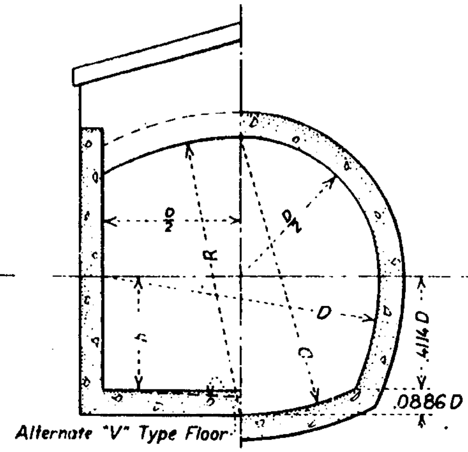
TUNNEL NO. 3-A PORTAL AREAS, MAIN CANAL, NAVAJO INDIAN IRRIGATION PROJECT--GENERAL PLAN AND PROFILE

Fig. 14 Par. 5.7

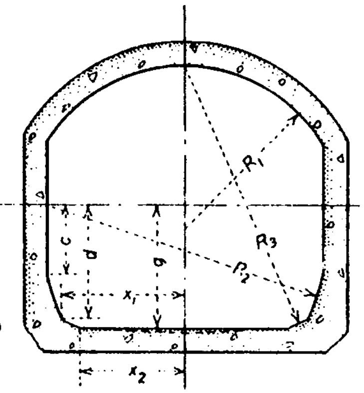




LONGITUDINAL SECTION - PORTAL TRANSITION



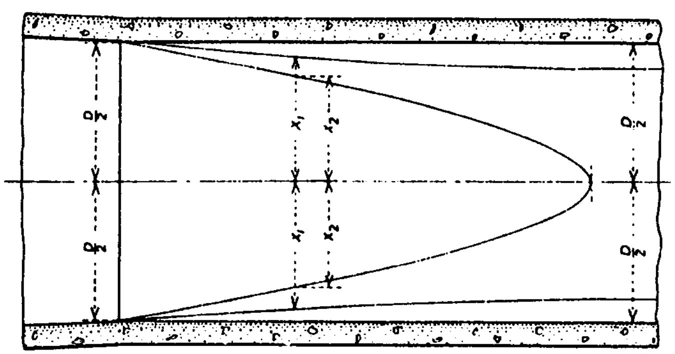
HALF SECTION - PORTAL   HALF SECTION - NORMAL TUNNEL



SECTION - DISTANCE "l" FROM PORTAL

EXPLANATION

- R = Radius of Top Arch at Portal.
- D = Diameter of Horseshoe Section Tunnel
- a = Rise of Arch at Portal.
- b = Rise of Arch at any distance "l" from Portal.
- c, d, g = Vertical distance of Traces shown from Tunnel Axis at any distance "l" from Portal.
- h = Vertical distance between Floor and Tunnel Axis at Portal.
- L = Length of Portal Transition.
- l = Distance from Portal to Section Considered.
- R_1 = Radius of Top Arch at any Point "l" from Portal.
- R_2 = Radius of Side Arc " " " "
- R_3 = Radius of Floor Arc " " " "
- x_1, x_2 = Horizontal distance of Traces shown from Tunnel Axis at any distance "l" from Portal



FLOOR PLAN - PORTAL TRANSITION

FORMULAE

$a = R - \frac{1}{2}\sqrt{4R^2 - D^2}$ $R_1 = \frac{4b^2 + D^2}{8b}$

$b = a + (\frac{D}{2} - a)\frac{l}{L}$ $R_2 = \sqrt{D^2 + c^2}$

$c = (\frac{L - l}{L})h$ $R_3 = \sqrt{(\frac{D}{2} + d)^2 + x_1^2}$

$d = g - \frac{l}{L}(0.0886D)$ $x_1 = \sqrt{R_2^2 - d^2} - \frac{1}{2}D$

$g = h + (\frac{D}{2} - h)\frac{l}{L}$ $x_2 = \sqrt{R_3^2 - (g + \frac{D}{2})^2}$

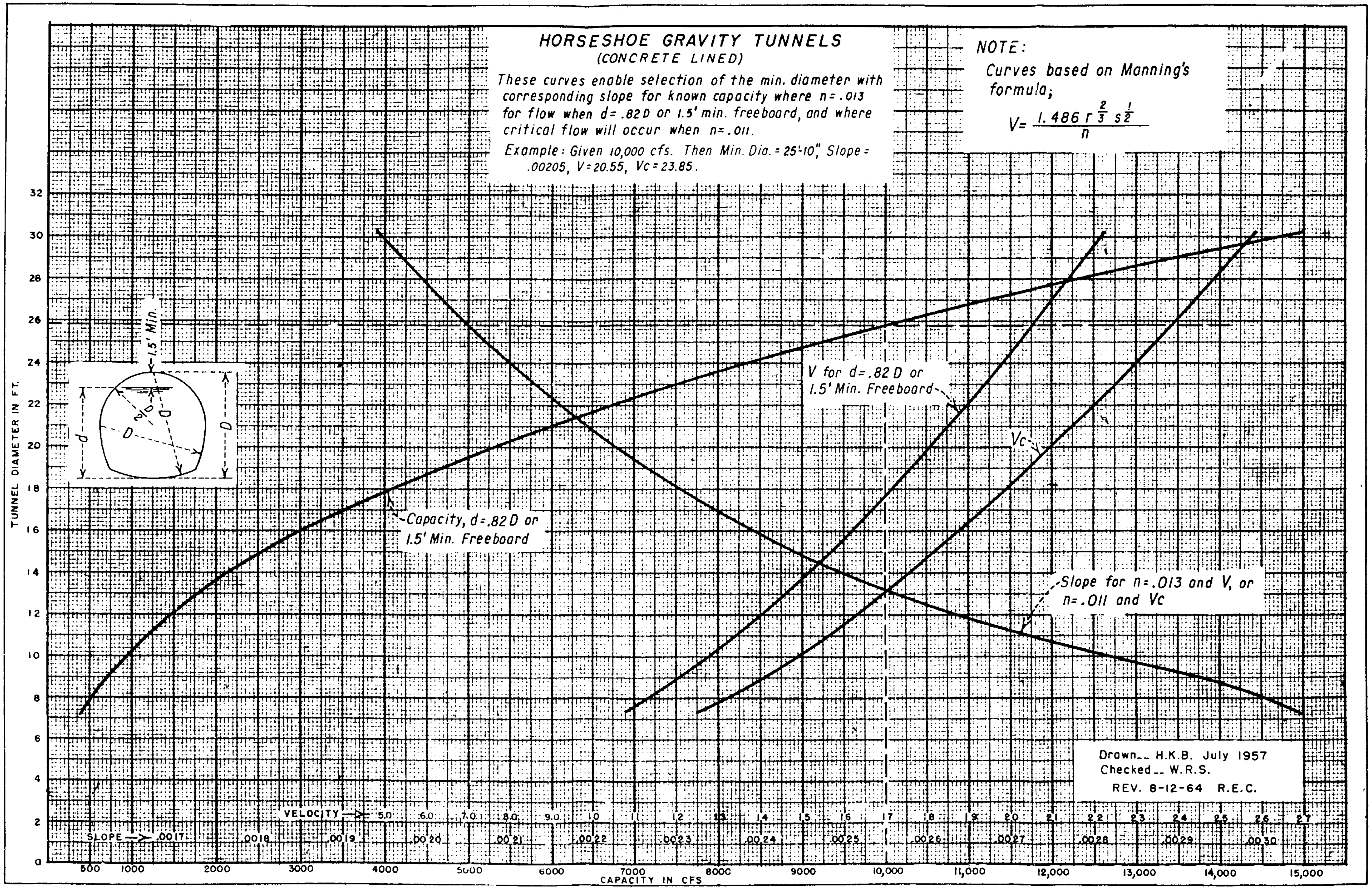
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DENVER OFFICE

TUNNEL PORTAL TRANSITION
DIMENSION FORMULAE

DRAWN: H.G.C.	SUBMITTED: W. R. McBirney
TRACED: T.W.C.	RECOMMENDED: J. L. Savage
CHECKED: ABR	APPROVED: A. J. Kaltz
DENVER, COLO., JULY 17, 1931	40-D-2011

HORSESHOE GRAVITY TUNNELS (CONCRETE LINED)--CAPACITY CURVES

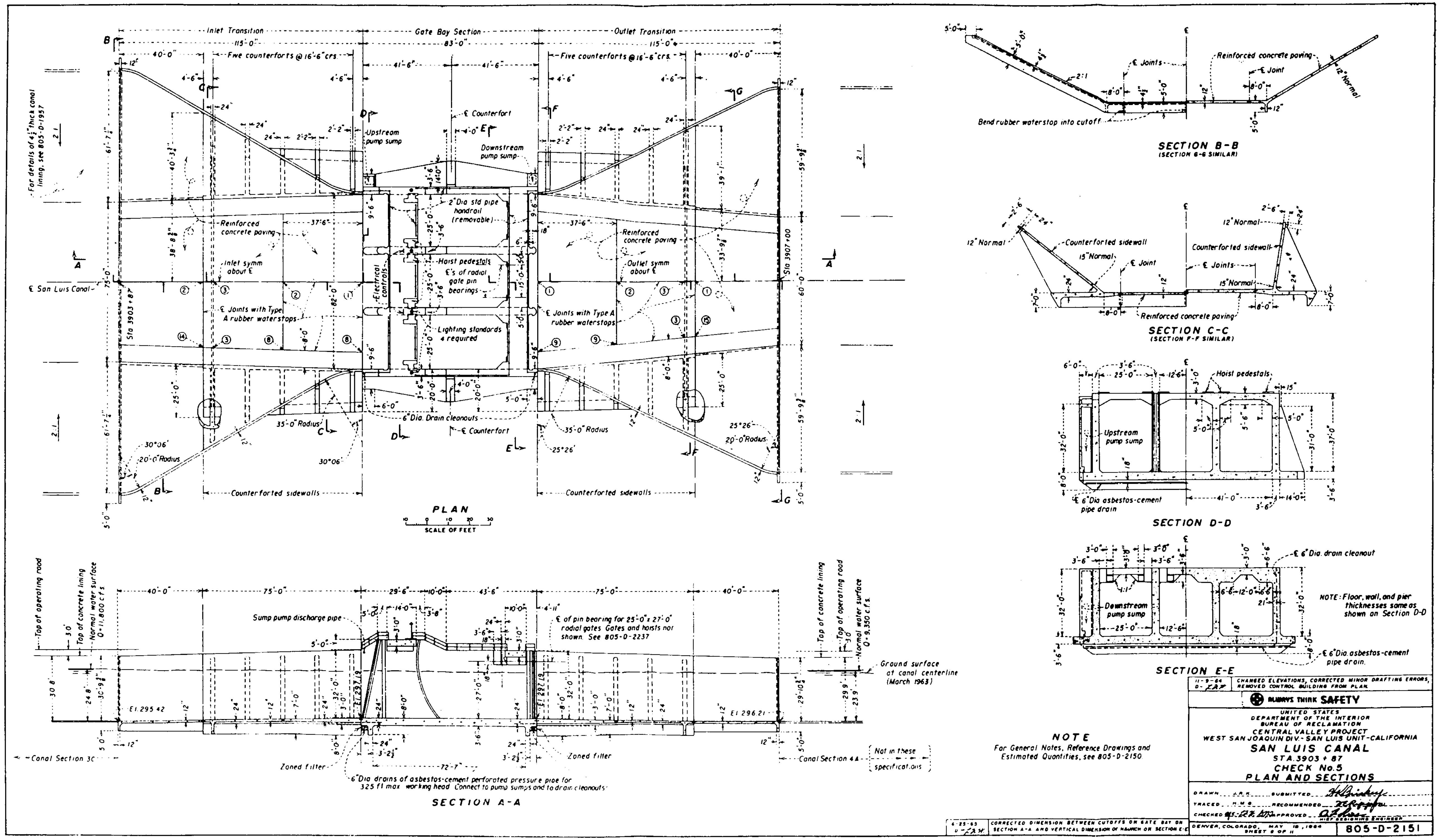
Fig. 16 Par. 5.9



103-D-637

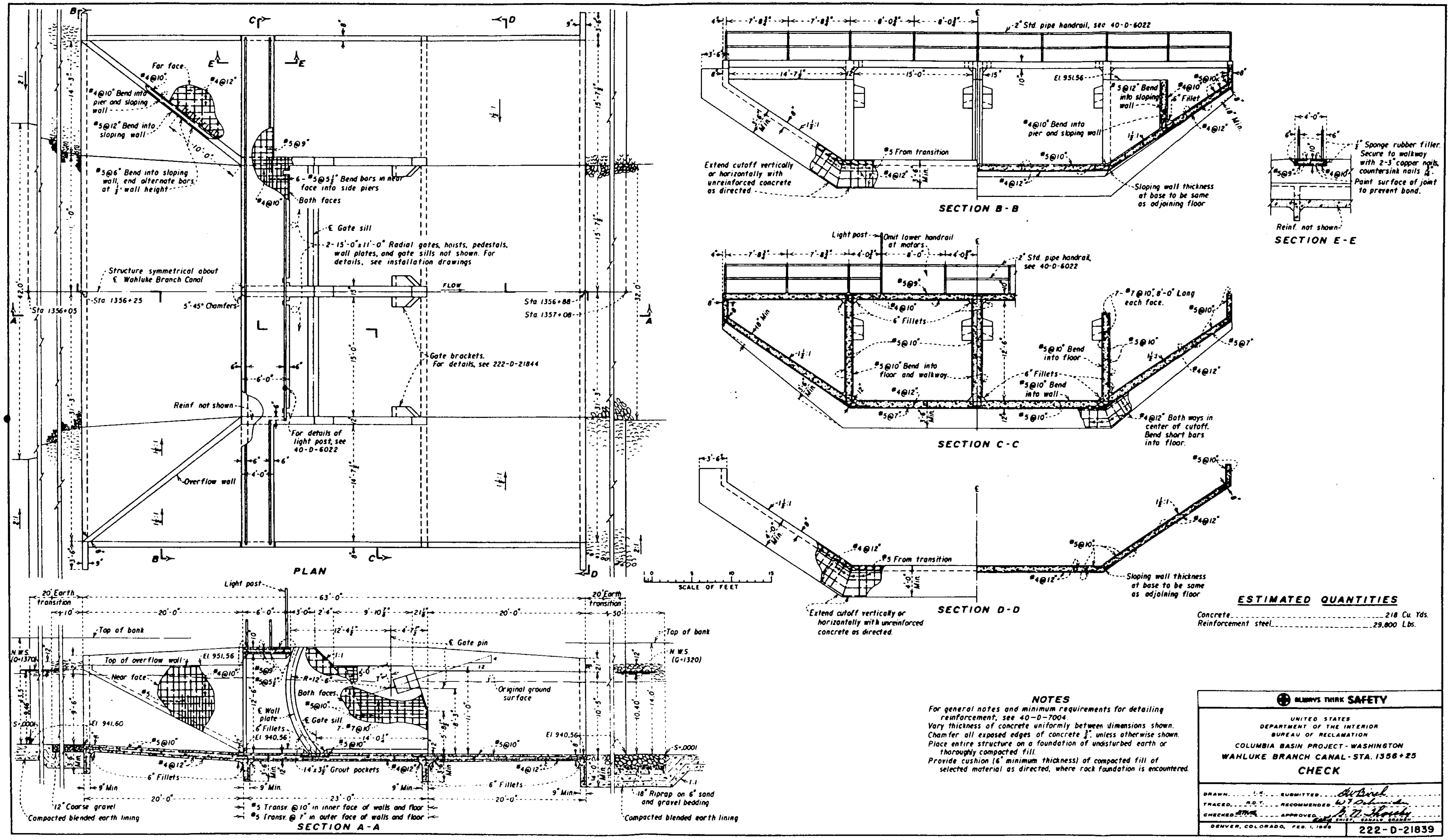
CHECK NO. 5, SAN LUIS CANAL--PLAN AND SECTIONS

Fig. 17 Par. 5.14A



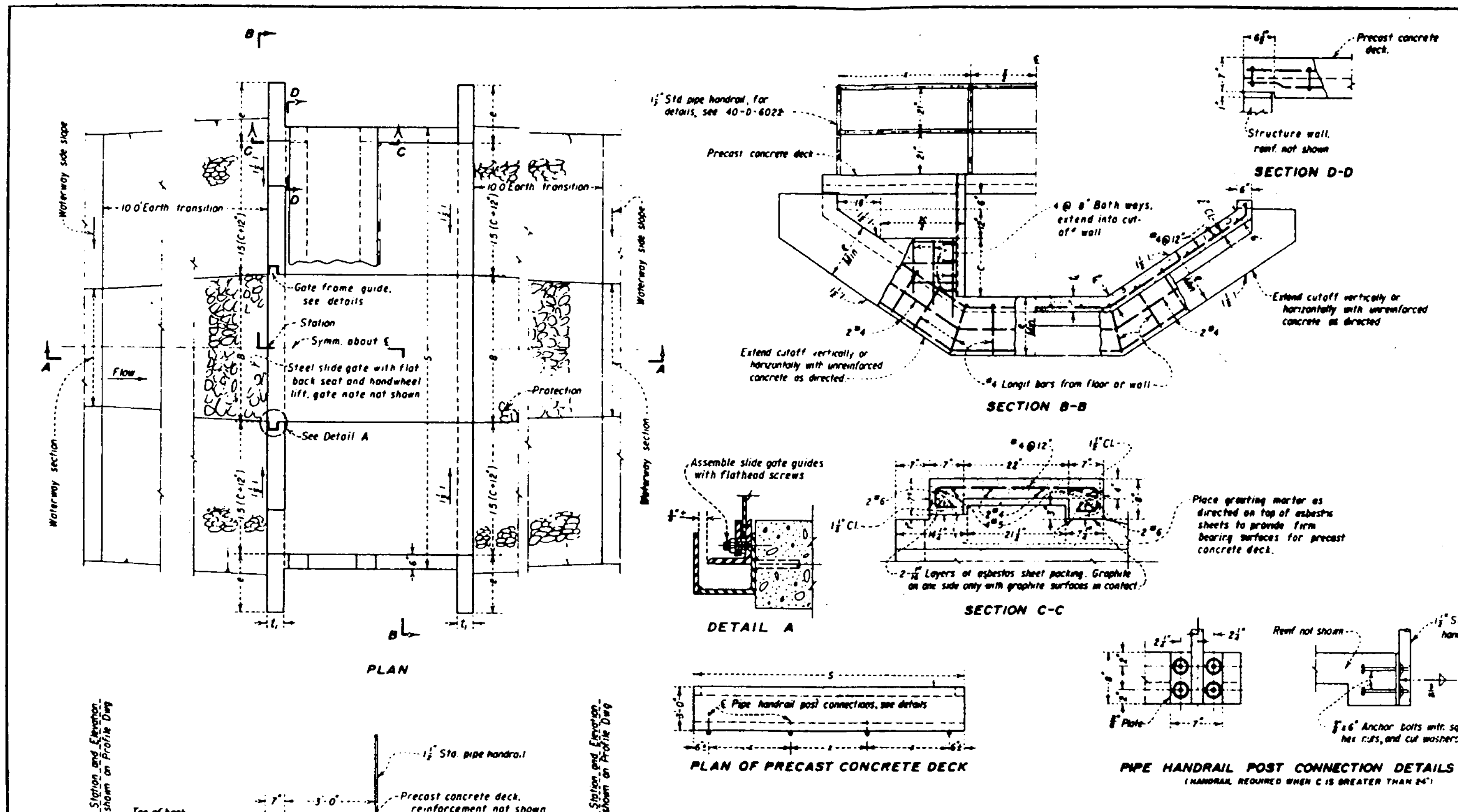
WAHLUKE BRANCH CANAL STATION 1356 + 25--CHECK

Fig. 18 Par. 5.14A



CHECKS--TYPE 10 A G

Fig. 19 Par. 5.14B



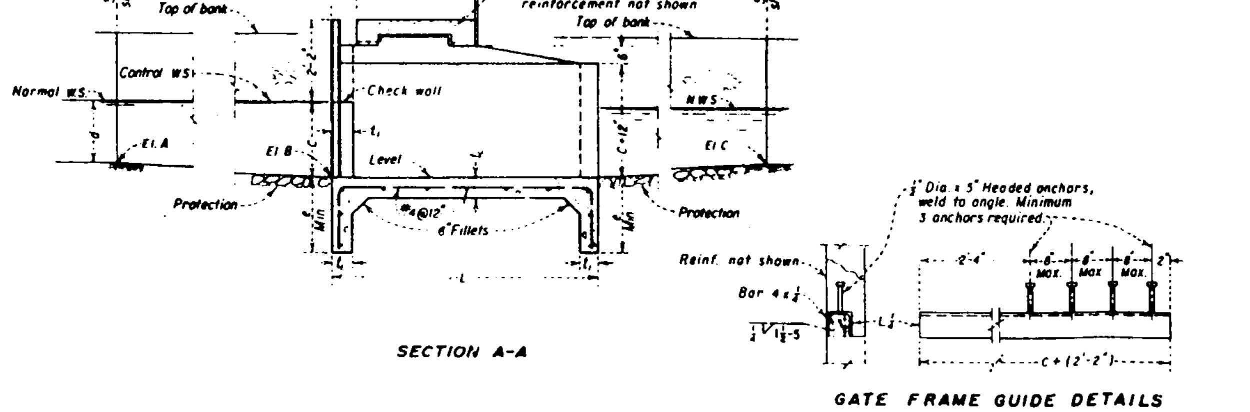


TABLE OF LOCATIONS AND ELEVATIONS

WATERWAY	STATION	STR. NO.	NOR. Q	ELEVATIONS A	ELEVATIONS B	ELEVATIONS C	REMARKS
WB38A	15+37	8	36	866.20	865.50	866.01	
WB38A	36+20	7	31	865.19	864.99	865.28	
WB38A	76+10	7	31	862.52	862.02	862.32	
WB38A	120+35	4	22	859.64	859.14	859.62	
WB38A	171+86	3	16	856.58	856.18	856.37	
WB38A	196+35	3	13	854.88	854.48	854.76	
WB38A	215+15	3	10	853.18	852.58	853.07	
WB38C	14+80	16	75	807.30	806.60	807.10	
WB38C	34+60	16	75	806.51	805.81	806.30	
WB38C	101+24.6	16	70	803.64	802.94	803.53	
WB38C	145+50	16	61	802.21	801.41	802.30	
WB38C	232+95	15	42	798.51	797.61	798.19	
WB38D	38+25	3	21	789.62	789.42	789.63	
WB38E	2+53	3	15	768.61	768.31	768.50	
WB38E	21+25	2	13	767.21	767.21	767.22	
WB38E	42+42	2	10	765.55	765.35	765.36	
WB38F	0+90	7	30	731.32	730.82	731.32	

STR. NO.	MAX. Q	SLIDE GATE (WIDTH x HEIGHT)*	SLIDE GATE FRM. HT.	STANDARD DIMENSIONS B	C	L	S	X		e	t_1	t_2	ESTIMATED QUANTITIES CONC. CU. YDS.	REIN. STEEL LBS.	MISC. METAL LBS.	REMARKS
1	10	36"x12"	5'	3'-0"	12"	4'-6"	10'-0"	—		24"	6"	6"	2.5	260	65	
2	15	36"x15"	6'	3'-0"	18"	4'-6"	11'-6"	—		24"	6"	6"	2.8	290	80	
3	21	36"x21"	6'	3'-0"	24"	5'-0"	13'-0"	—		24"	6"	6"	3.5	370	90	
4	26	36"x27"	6'	3'-0"	2'-6"	6'-0"	14'-6"	6'-9"		2'-6"	6"	8"	4.8	500	220	
5	21	48"x15"	6'	4'-0"	18"	4'-6"	12'-0"	—		24"	6"	6"	3.1	330	80	
6	28	48"x21"	6'	4'-0"	24"	5'-0"	14'-0"	—		24"	6"	6"	3.3	350	90	
7	35	48"x27"	6'	4'-0"	2'-6"	6'-0"	15'-6"	7'-3"		2'-6"	6"	8"	5.0	530	225	
8	42	48"x33"	6'	4'-0"	3'-0"	7'-0"	17'-0"	8'-0"		2'-6"	7"	8"	5.8	610	240	
9	35	60"x21"	6'	5'-0"	24"	5'-0"	15'-0"	—		24"	6"	6"	3.7	390	90	
10	43	60"x27"	6'	5'-0"	2'-6"	6'-0"	16'-6"	7'-9"		2'-6"	6"	8"	5.4	570	230	
11	52	60"x33"	6'	5'-0"	3'-0"	7'-0"	18'-0"	5'-8"		2'-6"	7"	8"	6.3	660	245	
12	61	60"x39"	6'	5'-0"	3'-6"	7'-6"	19'-6"	6'-2"		2'-6"	7"	8"	7.1	750	280	
13	42	72"x21"	6'	6'-0"	24"	5'-0"	16'-0"	—		24"	6"	6"	4.1	430	90	
14	53	72"x27"	6'	6'-0"	2'-6"	6'-0"	17'-6"	5'-6"		2'-6"	6"	8"	5.8	610	250	
15	63	72"x33"	6'	6'-0"	3'-0"	7'-0"	19'-0"	6'-0"		2'-6"	7"	8"	6.6	700	265	
16	80	84"x33"	6'	7'-0"	3'-6"	8'-0"	21'-6"	6'-10"		2'-6"	7"	8"	8.6	900	300	

*When a gate with specified height is not available, a gate with next greater available height shall be used.

NOTES

For general notes and minimum requirements for detailing reinforcement, see 40-D-7004.
Thickness of concrete to vary uniformly between dimensions shown.
Transverse bars to be continuous in walls and floor.
Place grouting mortar as directed on top of asbestos sheets to provide firm bearing surfaces for precast deck.
Set Elevation B so that top of check wall is at Control Water Surface Elevation.
Handrail required when C ≥ 24"
Gate frame height measured above ℄ of gate opening

REFERENCE DRAWINGS

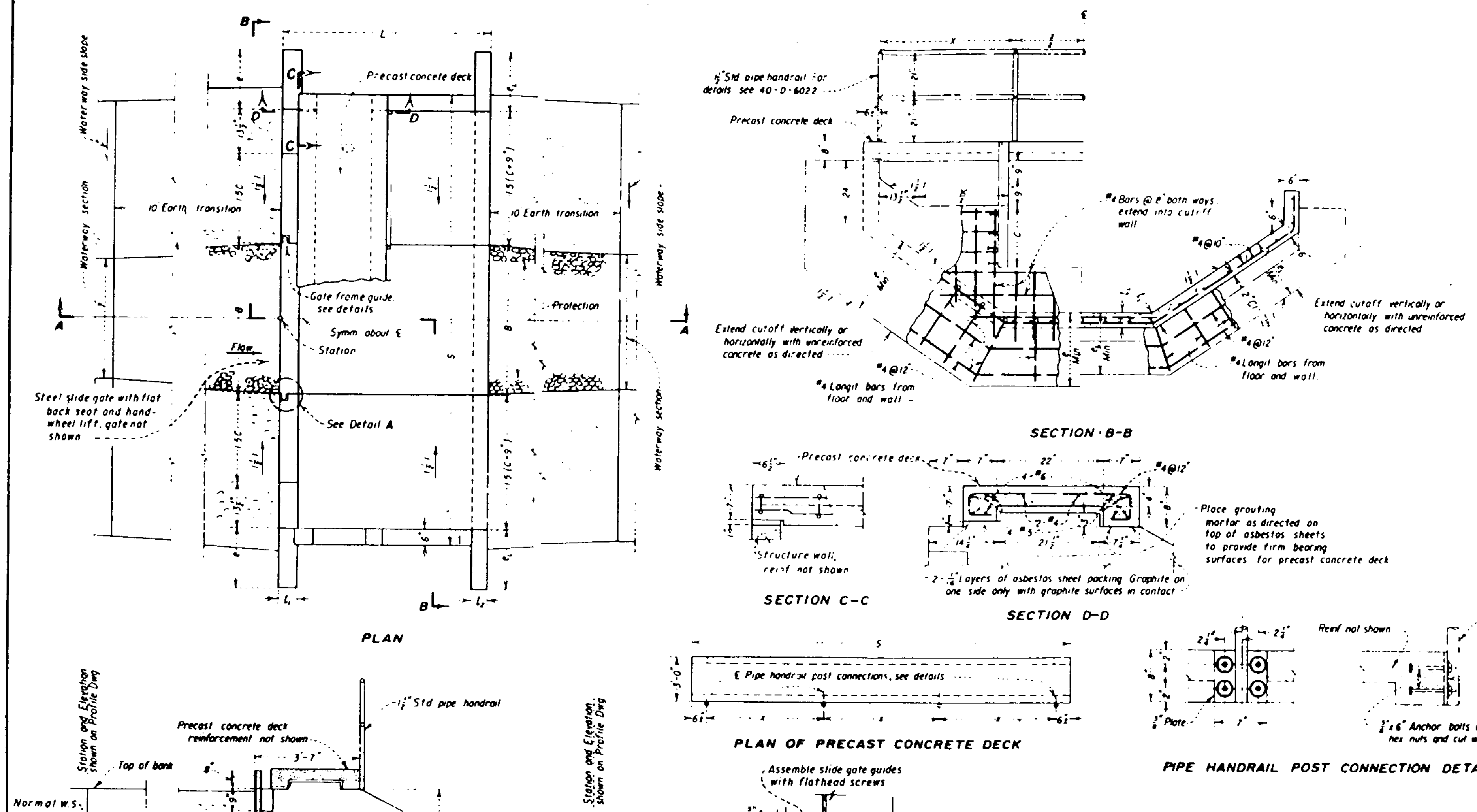
TYPICAL PROTECTION DETAILS 222-D-22211

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
WAHLUKE BRANCH CANAL LATERALS - BLOCK 25
CHECKS
TYPE 10 AG

DRAWN E.L.W. SUBMITTED ....
TRACED D.J.P. RECOMMENDED ....
CHECKED ..... APPROVED .... CHIEF, CANALS BRANCH
DENVER, COLORADO MARCH 21, 1967 222-D-22195

CHECKS--TYPE 13 A G

Fig. 20 Par. 5.14B



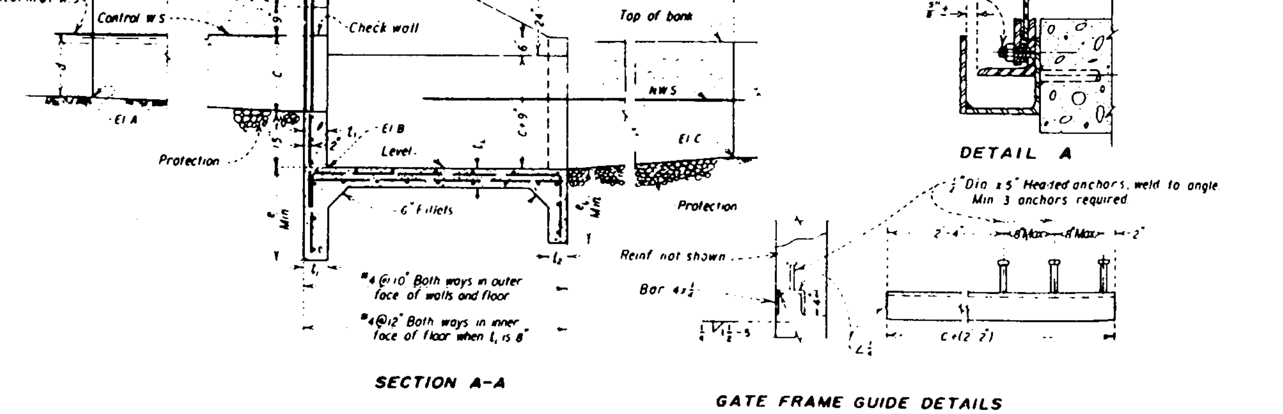


TABLE OF LOCATIONS AND ELEVATIONS

WATERWAY	STATION	STR NO	MAX Q	ELEVATIONS A	ELEVATIONS B	ELEVATIONS C	REMARKS
WB38	156+75	6	14	647.48	645.48	646.17	
WB38A	243+50	2	7	849.90	848.00	848.47	
WB38B	34+30	4	21	864.10	862.00	863.10	
WB38B	60+53.7	4	18	860.71	858.61	860.29	
WB38B	88+30	4	18	858.65	856.35	857.63	
WB38B	116+30	5	14	854.97	853.67	853.87	
WB38D	10+15	6	24	791.91	790.31	791.29	

STR No.	MAX Q	SLIDE GATE (WIDTH x HEIGHT)*	SLIDE GATE FRM HT	B	C	L	S	X	e	t_1	t_2	e_1	t_3	ESTIMATED QUANTITIES CONC CU YDS	RE. STEEL LBS	MISC METAL LBS	REMARKS
1	10	36"x12"	5'	3'-0	15	6'-6"	10'-0"	4'-6	24"	8"	6"	24"	6"	4.1	490	160	
2	15	36"x18"	6'	3'-0"	21"	6'-6"	11'-6	5'-3	24	8	6"	24	6	4.4	520	180	
3	21	36"x24"	6	3'-0	2'-3"	7'-0"	13'-0"	6'-0	24"	8"	6"	24"	7"	4.8	570	200	
4	26	36"x30"	6'	3'-0	2'-9	7'-6"	14'-6	6'-9	2'-6"	8"	8"	24"	7"	5.6	670	225	
5	21	48"x18"	6'	4'-0"	21"	7'-0"	12'-6"	5'-9	24"	8"	6"	24"	6"	4.8	570	180	
6	28	48"x24"	6'	4'-0	2'-3"	7'-6"	14'-0"	6'-6	24"	8"	6"	24"	7"	5.9	700	210	
7	35	48"x30"	6'	4'-0"	2'-9"	8'-0"	15'-6"	7'-3"	2'-6"	8"	8"	24"	7"	6.9	820	220	
8	42	48"x36"	6'	4'-0	3'-3"	8'-6"	17'-0"	8'-0	2'-6"	9"	8"	2'-6	8"	8.2	980	235	
9	35	60"x24"	6'	5'-0"	2'-3"	7'-6"	15'-0"	7'-0	24"	8"	6"	24"	7"	6.2	740	210	
10	43	60"x30"	6'	5'-0"	2'-9	8'-0"	16'-6"	7'-9	2'-6"	8"	8"	24	7"	7.2	860	230	
11	52	60"x36"	6'	5'-0"	3'-3	8'-6"	18'-0"	5'-8	2'-6"	9"	8"	2'-6"	8"	8.7	1040	260	
12	42	72"x24"	6'	6'-0"	2'-3"	7'-6"	16'-0"	7'-6	24"	8"	6"	24"	7"	6.5	770	215	
13	53	72"x30"	6'	6'-0	2'-9"	8'-0"	17'-6"	5'-6	2'-6"	9"	8"	24"	7"	7.6	900	250	
14	63	72"x36"	6'	6'-0	3'-3	8'-6"	19'-0"	6'-0"	2'-6"	9"	8"	2'-6	8"	9.2	1100	265	

* When a gate with specified height is not available, a gate with next greater available height shall be used

NOTES

For general notes and minimum requirements for detailing reinforcement, see 40-D-7004.
Outer face transverse bars to be continuous in walls and floors
Thickness of concrete to vary uniformly between dimensions shown
Place grouting motar as directed on top of asbestos sheets to provide firm bearing surfaces for precast deck
Set Elevation B so that top of check wall is at Control Water Surface Elevation
Gate frame height measured from ℄ of gate opening

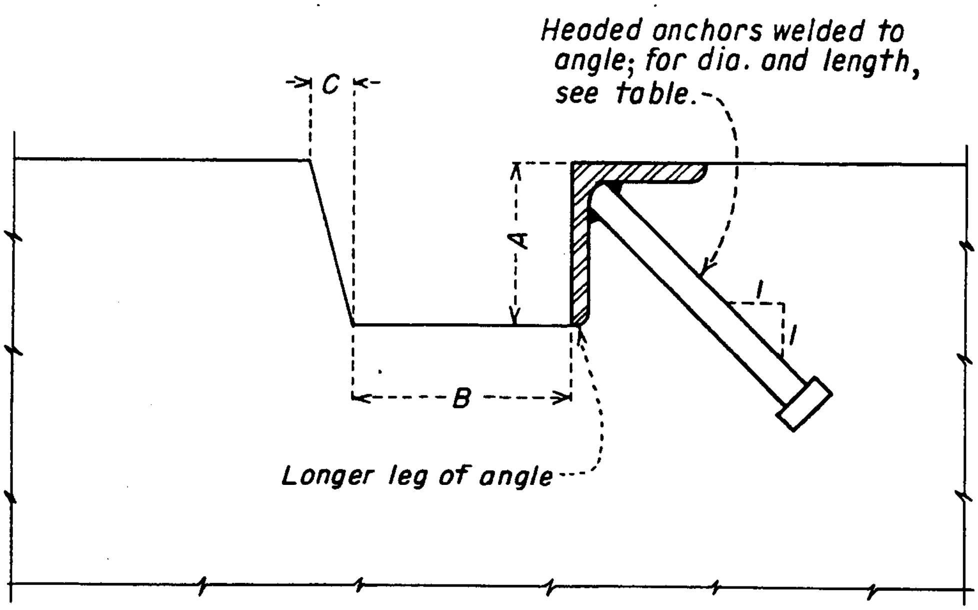
REFERENCE DRAWINGS

TYPICAL PROTECTION DETAILS 222-D-22211

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT - WASHINGTON
WAHLUKE BRANCH CANAL LATERALS - BLOCK 25
CHECKS
TYPE 13AG

DRAWN ...	SUBMITTED ...
TRACED ...	RECOMMENDED ...
CHECKED ...	APPROVED ... CHIEF, CANALS BRANCH
DENVER, COLORADO, MARCH 21, 1957	222-D-22196

STOP PLANK GROOVES--STANDARD DESIGNS Fig. 21 Par. 5.14C



STOP PLANK THICKNESS	DIMENSIONS			ANGLE FOR GROOVE	WEIGHT PER FT.	ANCHOR
	A	B	C			
2"	$2\frac{1}{2}$"	3"	$\frac{1}{2}$"	$2\frac{1}{2} \times 2 \times \frac{1}{4}$	3.62	$\frac{3}{8}$" Dia. x 4"
3"	3"	4"	$\frac{3}{4}$"	$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5.60	$\frac{1}{2}$" Dia. x 5"
4"	3"	5"	$\frac{3}{4}$"	$3 \times 2\frac{1}{2} \times \frac{5}{16}$	5.60	$\frac{1}{2}$" Dia. x 5"
6"	4"	7"	1"	$4 \times 3 \times \frac{3}{8}$	8.50	$\frac{1}{2}$" Dia. x 5"
8"	5"	9"	$1\frac{1}{4}$"	$5 \times 3\frac{1}{2} \times \frac{3}{8}$	10.40	$\frac{5}{8}$" Dia. x $6\frac{1}{2}$"

THIS DRAWING SUPERSEDES DWG. 40-D-2945.

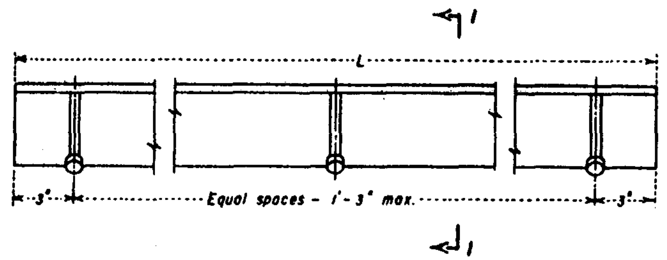
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS
STOP PLANK GROOVES

DRAWN E.A.A. SUBMITTED
TRACED E.M.W. RECOMMENDED
CHECKED APPROVED
ACT. CHIEF ENGINEER
DENVER, COLO. 10-28-53 40-D-5326

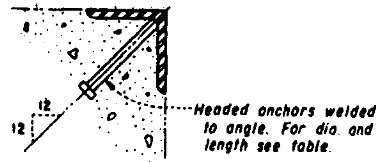
REV. 6-29-67

Fig. 22 Par. 5.14C

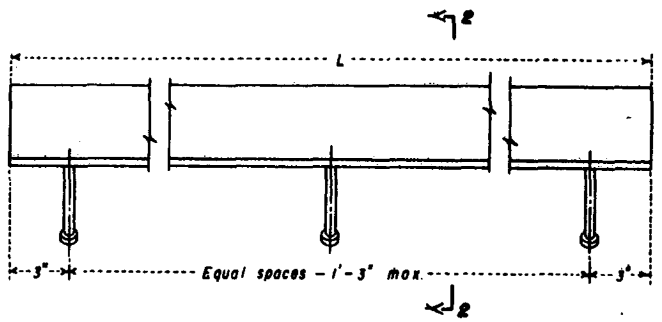
STRUCTURAL STEEL LINERS—STANDARD DESIGNS



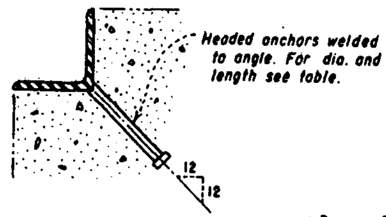
ANGLE LINER
TYPE "A"



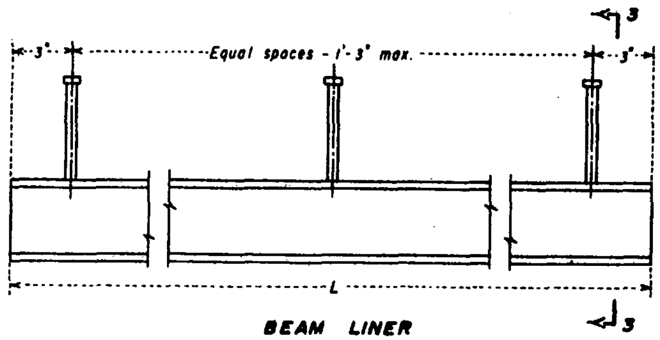
SECTION 1-1



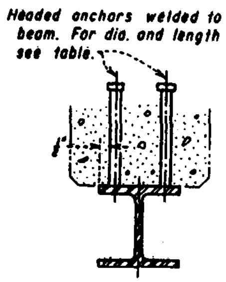
ANGLE LINER
TYPE "B"



SECTION 2-2



BEAM LINER



SECTION 3-3

ANGLE LINER
TYPE "A" AND TYPE "B"

ANGLE	ANCHOR
2½ x 2 x ¼	⅜" Dia. x 4"
2½ x 2½ x ¼	⅜" Dia. x 4"
3 x 2½ x 5/16	½" Dia. x 5"
3 x 3 x 5/16	½" Dia. x 5"
3½ x 3½ x 5/16	½" Dia. x 5"
4 x 3 x ⅜	½" Dia. x 5"
4 x 4 x ⅜	½" Dia. x 5"
5 x 3 x ⅜	⅝" Dia. x 6½"
5 x 3½ x ⅜	⅝" Dia. x 6½"
5 x 5 x ⅜	⅝" Dia. x 6½"
6 x 4 x ⅜	⅝" Dia. x 6½"
6 x 6 x ⅜	⅝" Dia. x 6½"

BEAM LINER

* BEAM	ANCHOR
4 WF 13.0	½" Dia. x 5"
5 WF 18.5	⅝" Dia. x 6½"
6 WF 25.0	⅝" Dia. x 6½"
8 WF 31.0	¾" Dia. x 8"
10 WF 49.0	¾" Dia. x 8"

*Beams of the same nominal size and having equal or slightly greater weight may be substituted for the wide flange beams shown in the table.

NOTE
Dimensions "L" and sizes of liners given in the purchase order or specifications.

THIS DRAWING SUPERSEDES DWG. 40-D-5295

ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
STANDARD DESIGNS

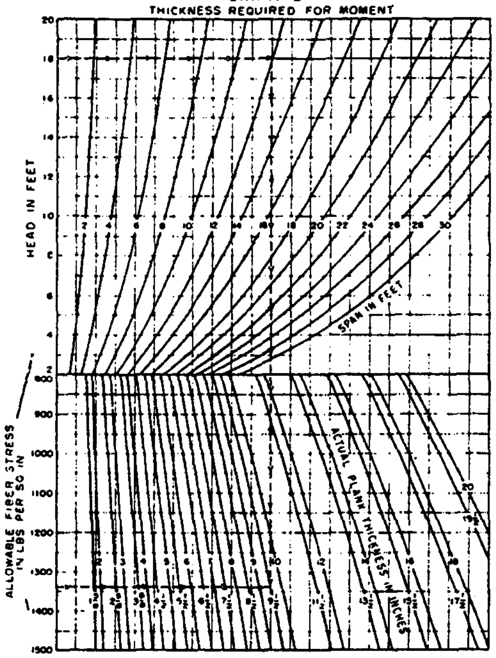
STRUCTURAL STEEL LINERS

DRAWN ..... SUBMITTED .....
TRACED ..... RECOMMENDED .....
CHECKED ..... APPROVED .....
DENVER, COLORADO MAY 31, 1963 40-D-5907

STOP PLANKS (WOOD)--DESIGN CHARTS

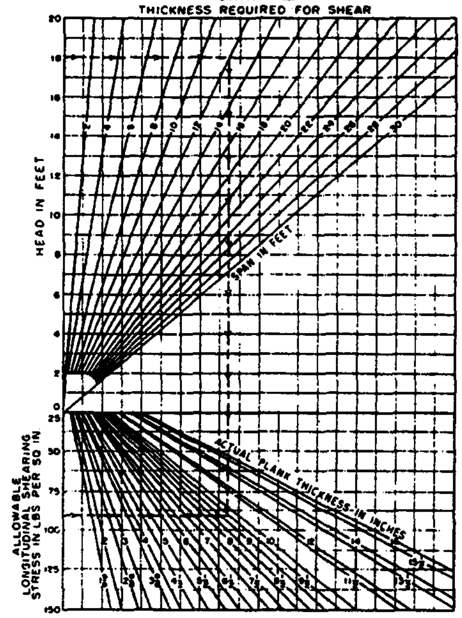
Fig. 23 Par. 5.14C

CHART I
THICKNESS REQUIRED FOR MOMENT



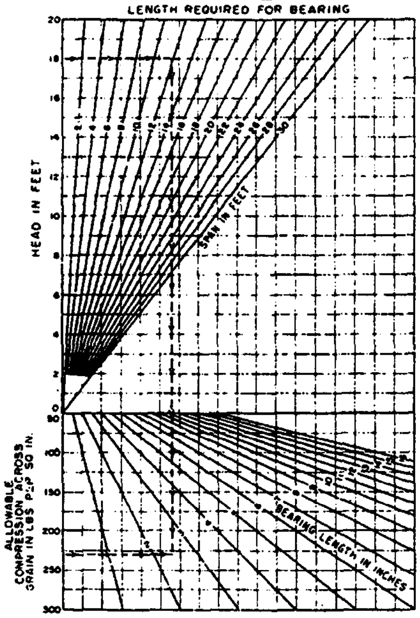
EXAMPLE Find the thickness req'd for a head of 18', a span of 12', and an allowable fiber stress of 1340 lbs per sq inch. ANS. 9½" Min

CHART II
THICKNESS REQUIRED FOR SHEAR



EXAMPLE Find the thickness req'd for a head of 18', a span of 12', and an allowable stress of 90 lbs. per sq.in. ANS. 9½" Min.

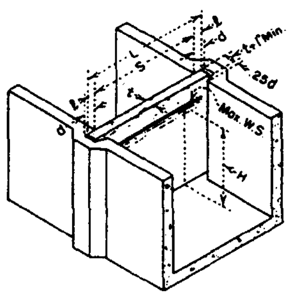
CHART III
LENGTH REQUIRED FOR BEARING



EXAMPLE Find bearing length req'd for a head of 18', a span of 12', and an allowable stress of 230 lbs. per sq inch. ANS. 2½" Min

ALLOWABLE STRESSES

Species	Extreme Fiber	Longitudinal Shear	Compression across grain
Douglas fir, coast region.	1340	90	230
Douglas fir, coast region, close-grained.	1420	90	240
Douglas fir, inland region.	1200	90	220
Douglas fir, inland region, close-grained.	1280	90	235
Douglas fir, Rocky Mountain region.	980	85	190
Douglas fir, dense, all regions	1550	105	265
Pine, Idaho white, No white, Sugar, Ponderosa	800	85	175
Pine, Norway	960	85	210
Pine, longleaf and shortleaf, southern.	1330	110	230
Pine, longleaf and shortleaf, southern, dense	1550	130	265
Redwood	1065	70	175
Redwood, close-grained	1140	70	185
Spruce, red, white and Sitka	980	85	175



EXPLANATION

H = Head in feet.
L = Length of stop plank $S+\frac{1}{6}(d+\ell)$ feet min
S = Span in feet.
d = Depth of stop plank groove in inches.
ℓ = Length, in inches, req'd for bearing. See Chart III
t = Thickness, in inches, req'd for moment or shear.
See Chart I for thickness req'd for moment
See Chart II for thickness req'd for shear.

EXAMPLE

Find the dimensions required for wooden stop planks for a head of 18 feet and a clear span of 12 feet using Douglas Fir, coast region, lumber.

From Table of Allowable Working Stresses.

- Moment - Extreme fiber in bending..........1340 p.s.i.
- Shear - Longitudinal shear..................90 p.s.i.
- Bearing - Compression across grain..........230 p.s.i.

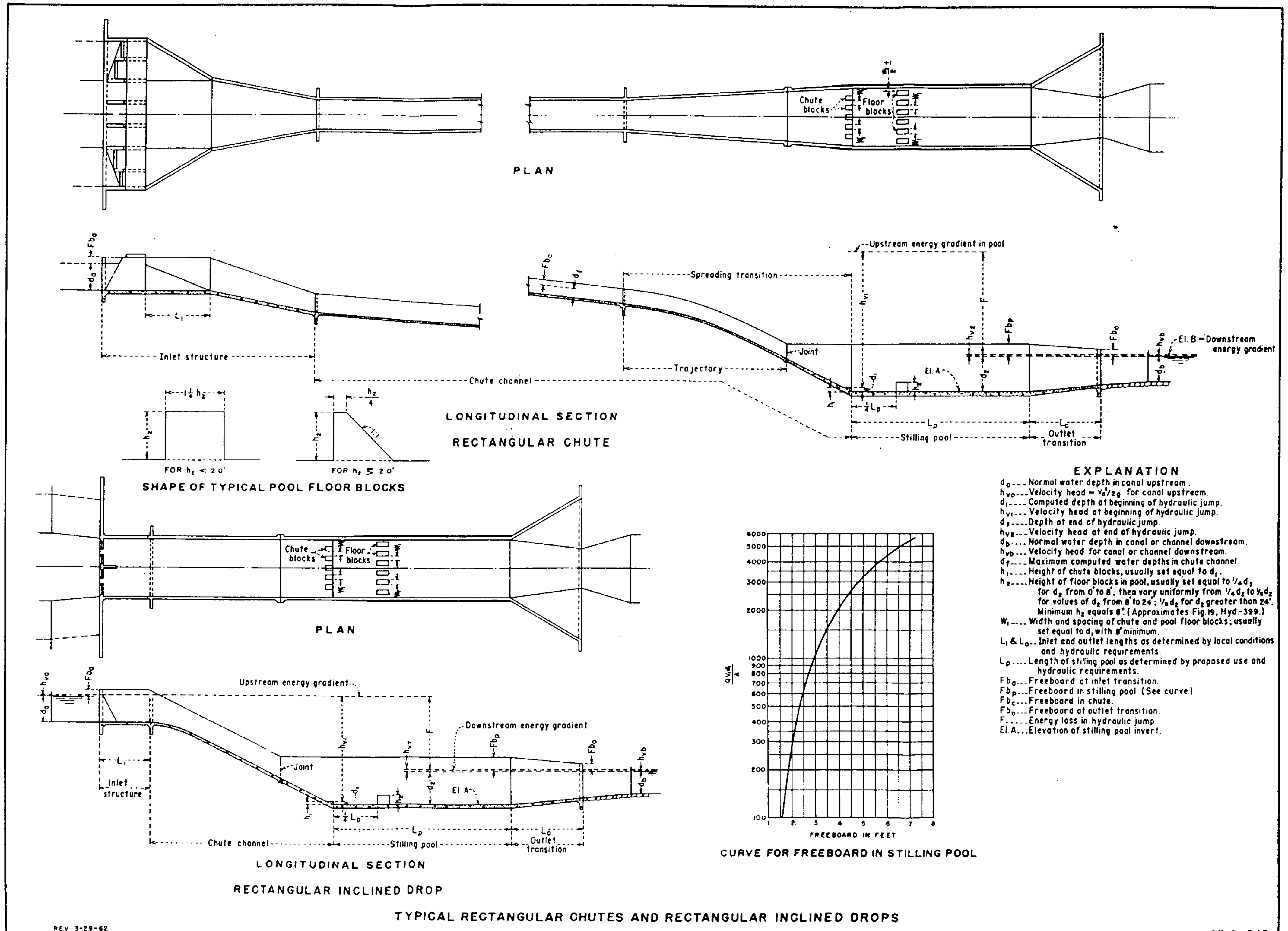
ANS.

- From Chart I Thickness req'd. for moment = 9 1/2" Min.
- From Chart II Thickness req'd. for shear = 9 1/2" Min.
- From Chart III Length req'd. for bearing = 2 1/2" Min.

X-D-1305

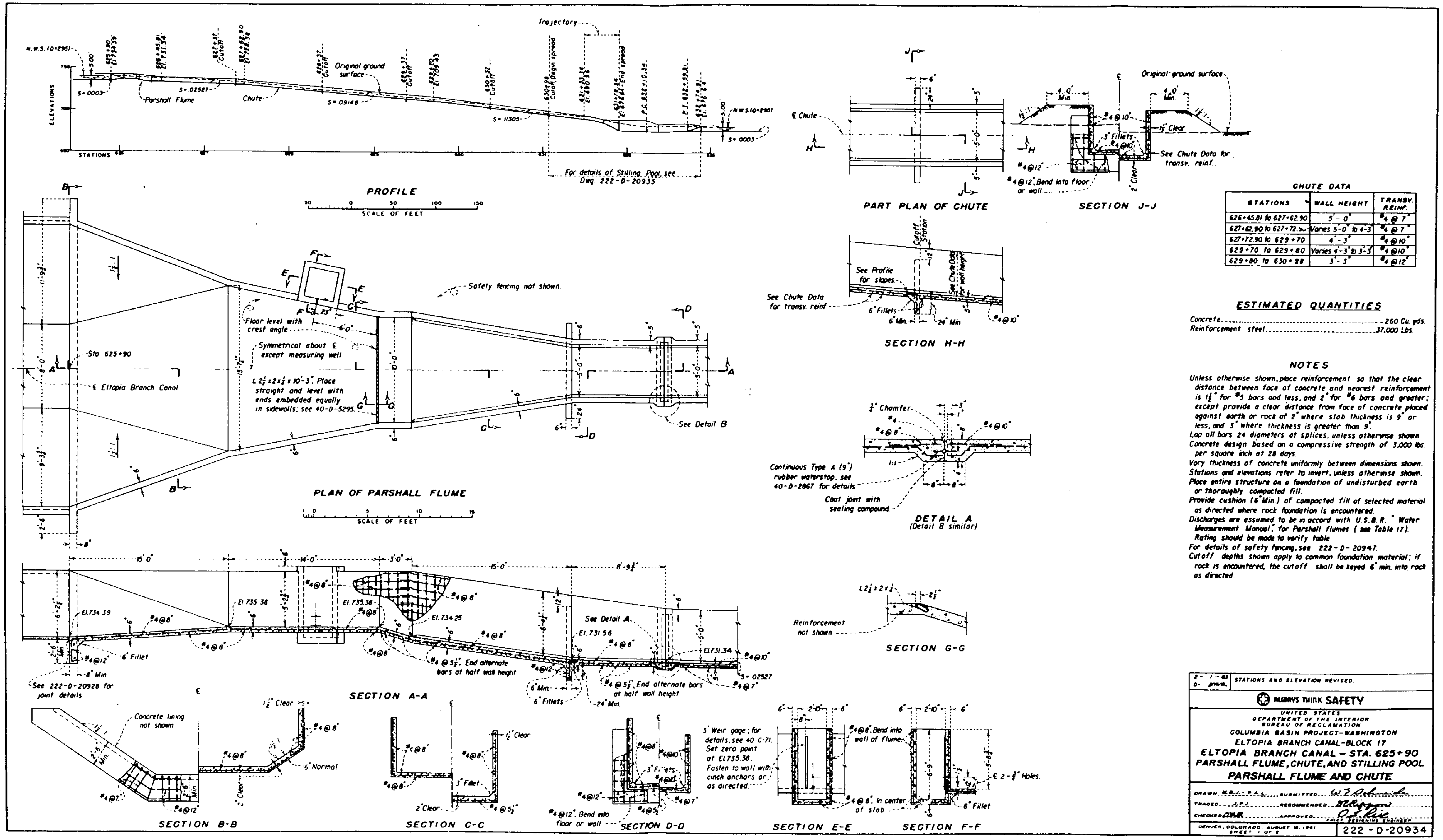
TYPICAL RECTANGULAR CHUTES AND RECTANGULAR INCLINED DROPS

Fig. 24 Par. 5.17



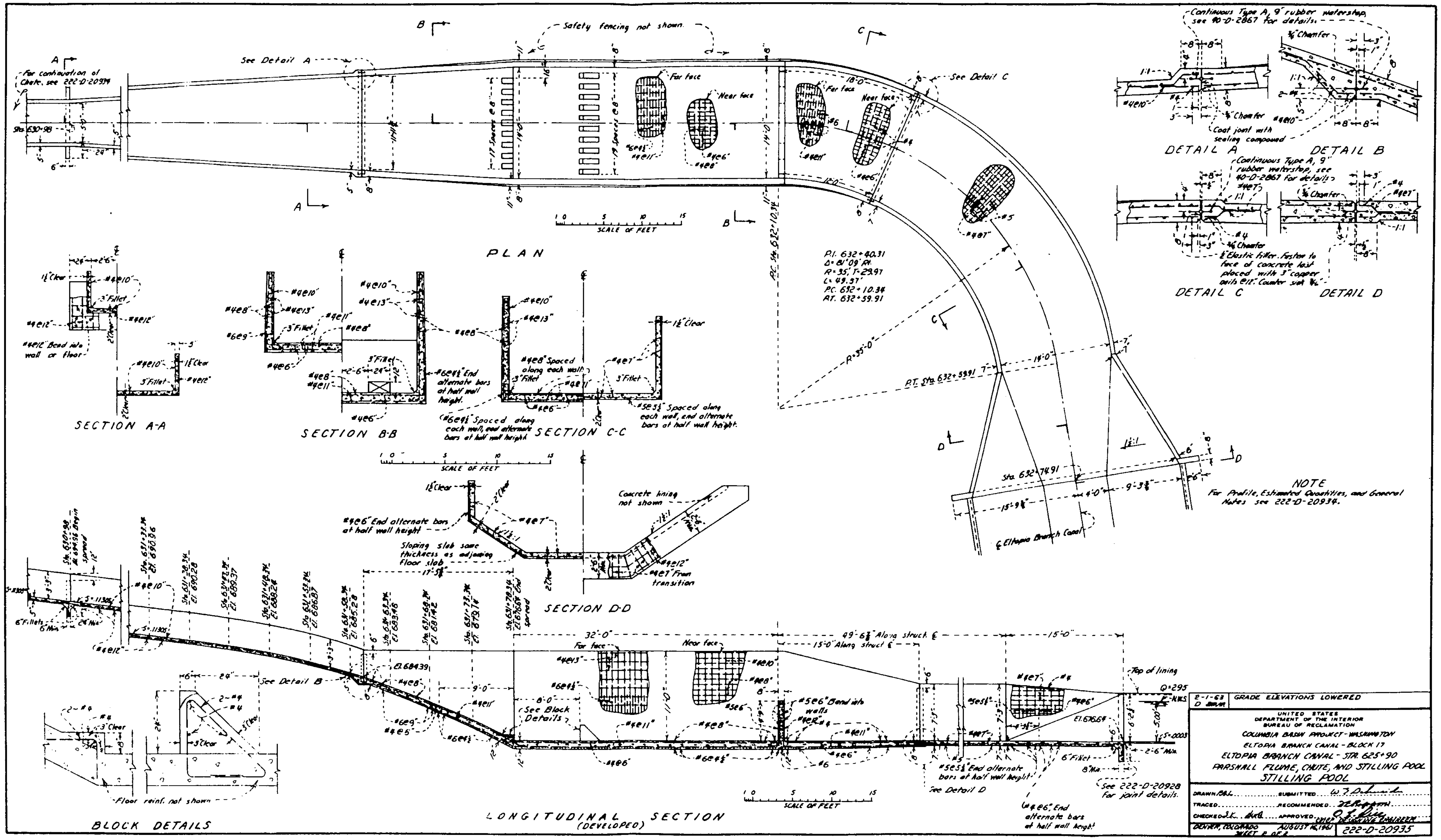
PARSHALL FLUME AND CHUTE--ELTOPIA BRANCH CANAL

Fig. 25 Par. 5.17



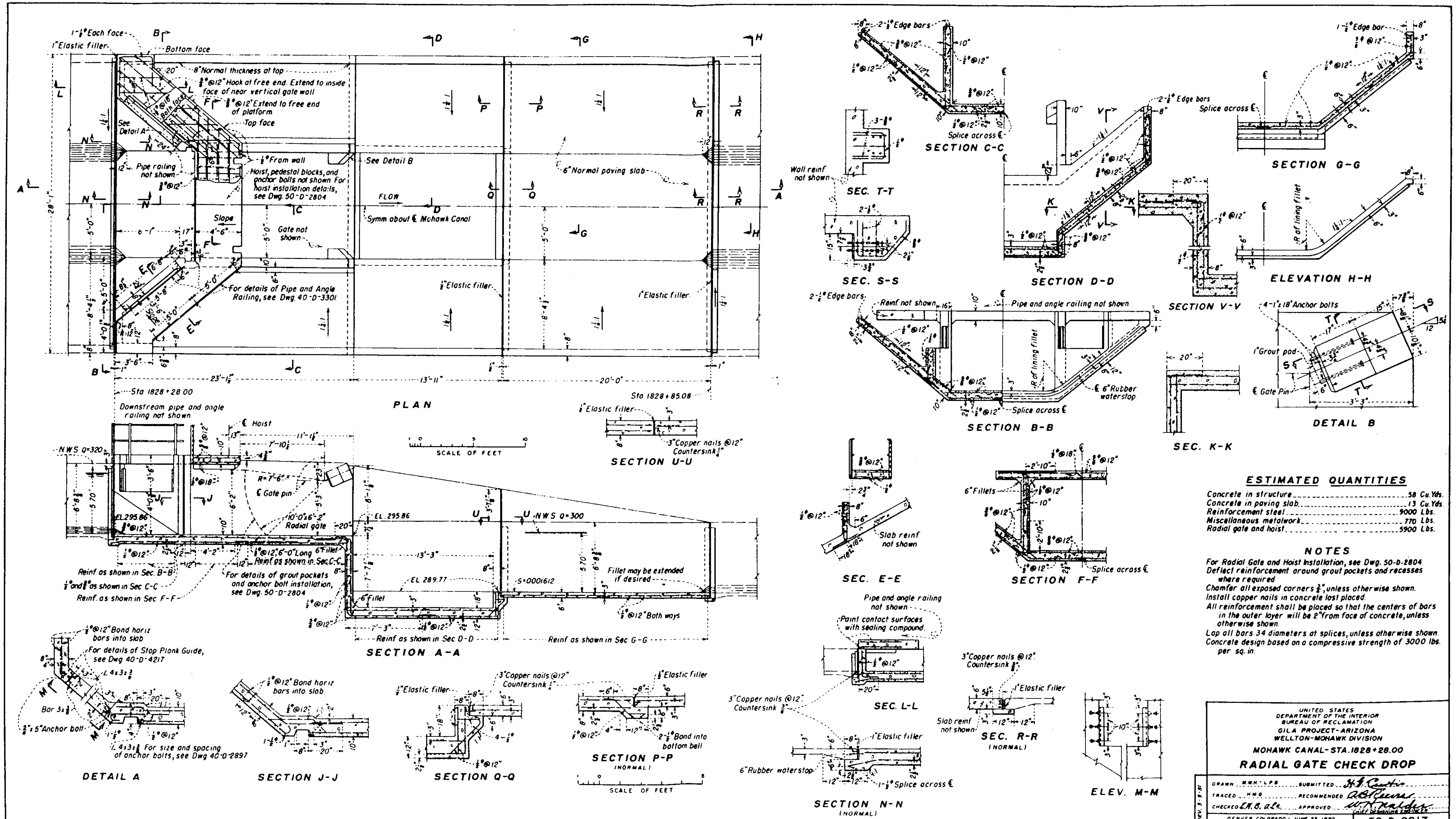
STILLING POOL--ELTOPIA BRANCH CANAL

Fig. 26 Par. 5.17

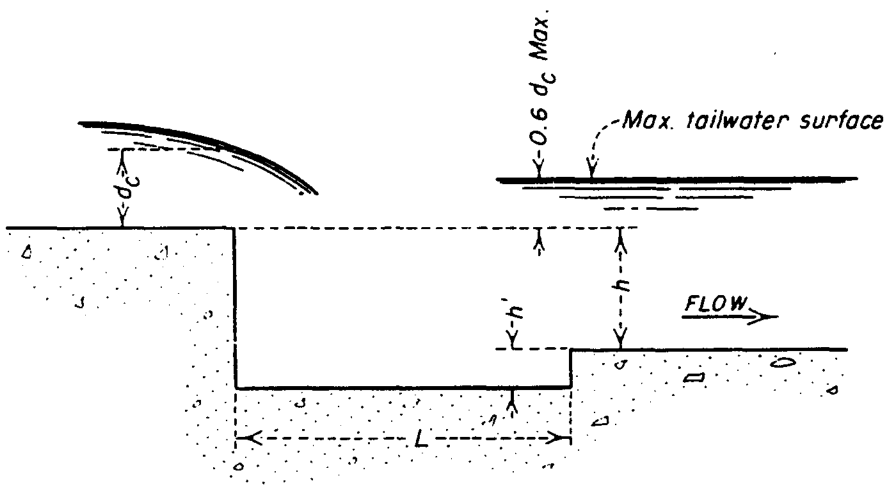


RADIAL GATE CHECK DROP--MOHAWK CANAL

Fig. 27 Par. 5.18



STILLING POOL DIMENSIONS FOR VERTICAL DROPS Fig. 28 Par. 5.18A



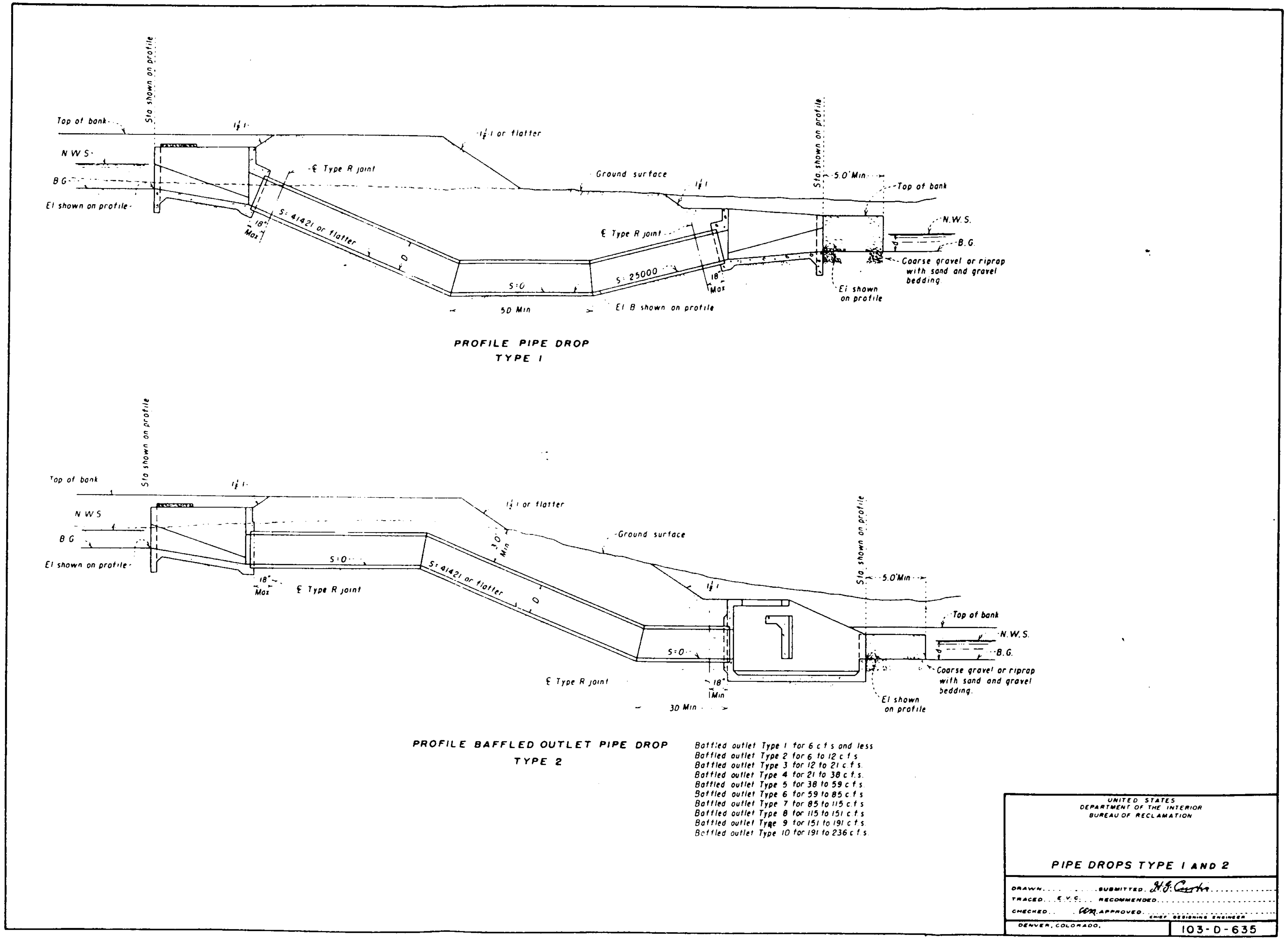
$$L = \left[2.5 + 1.1\frac{d_c}{h} + 0.7\left(\frac{d_c}{h}\right)^3\right]\sqrt{h\,d_c} \qquad h' = \frac{d_c}{2}$$

STILLING POOL DIMENSIONS FOR VERTICAL DROPS

103-D-641

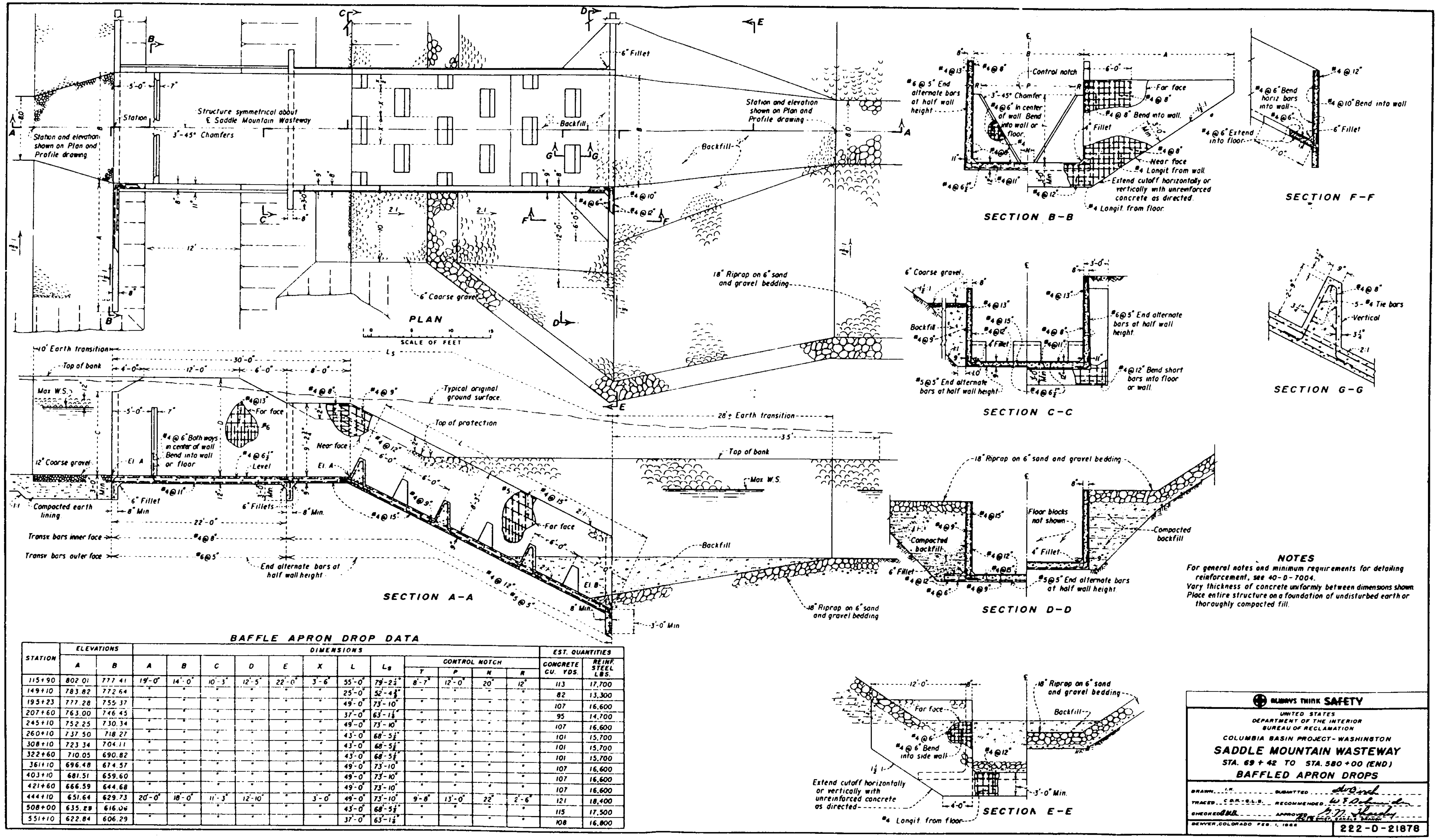
PIPE DROPS TYPES 1 AND 2

Fig. 29 Par. 5.19



BAFFLED APRON DROPS--SADDLE MOUNTAIN WASTEWAY

Fig. 30 Par. 5.24



BAFFLE APRON DROP DATA

STATION	ELEVATIONS		DIMENSIONS								CONTROL NOTCH				EST. QUANTITIES	
	A	B	A	B	C	D	E	X	L	Ls	T	P	N	R	CONCRETE CU. YDS.	REINF. STEEL LBS.
115+90	802.01	777.41	19'-0"	14'-0"	10'-3"	12'-5"	22'-0"	3'-6"	55'-0"	79'-2 1/4"	8'-7"	12'-0"	20"	12"	113	17,700
149+10	783.82	772.64		"	"	"	"	"	25'-0"	52'-4 3/8"	"	"	"	"	82	13,300
195+23	777.28	755.37	"	"	"	"	"	"	49'-0"	73'-10"	"	"	"	"	107	16,600
207+60	763.00	746.45	"	"	"	"	"	"	37'-0"	63'-1 1/8"	"	"	"	"	95	14,700
245+10	752.25	730.34	"	"	"	"	"	"	49'-0"	73'-10"	"	"	"	"	107	16,600
260+10	737.50	718.27	"	"	"	"	"	"	43'-0"	68'-5 1/2"	"	"	"	"	101	15,700
308+10	723.34	704.11	"	"	"	"	"	"	43'-0"	68'-5 1/2"	"	"	"	"	101	15,700
322+60	710.05	690.82	"	"	"	"	"	"	43'-0"	68'-5 1/2"	"	"	"	"	101	15,700
361+10	696.48	674.57	"	"	"	"	"	"	49'-0"	73'-10"	"	"	"	"	107	16,600
403+10	681.51	659.60	"	"	"	"	"	"	49'-0"	73'-10"	"	"	"	"	107	16,600
421+60	666.59	644.68	"	"	"	"	"	"	49'-0"	73'-10"	"	"	"	"	107	16,600
444+10	651.64	629.73	20'-0"	18'-0"	11'-3"	12'-10"	"	3'-0"	49'-0"	73'-10"	9'-8"	13'-0"	22"	2'-6"	121	18,400
508+00	635.29	616.06	"	"	"	"	"	"	43'-0"	68'-5 1/2"	"	"	"	"	115	17,500
551+10	622.84	606.29	"	"	"	"	"	"	37'-0"	63'-1 1/8"	"	"	"	"	108	16,800

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INTRODUCTION

.1 This chapter contains design information for the more commonly used water measurement structures in canals and laterals. They are constant-head orifice structures, open flowmeters, line meters, Parshall flumes, and weirs. General structural and hydraulic design criteria relating to water measurement structures are given in Chapter 2. In the structural drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated.

CONSTANT-HEAD ORIFICE

.2 Constant-head orifice structures are often used in turnouts where sufficient head is available between canal and lateral or farm delivery water surfaces. Two gates are required for satisfactory operation. The first gate controls the area of the orifice and the second gate controls the water depth below the orifice. A constant difference in head on the two sides of the orifice gate is maintained by adjusting the downstream gate. The amount of water passing is determined by the size of the orifice set by the first gate with a definite differential head. This type of structure may also be used for water measurement on an in-line structure. These structures, called orifice measuring devices, may be operated by varying the differential head if properly calibrated. Figure 1 shows a constant-head orifice turnout structure of a type that has been constructed at many locations. The following criteria are for capacities up to 30 cfs with 0.2-foot differential head. Special consideration should be given for greater capacities and other differential heads.

Hydraulics

A. A value of C = 0.70 should be used for the orifice gate with angle-iron reinforcement at the bottom of the gate leaf. A value of C = 0.65 should be used where the angle-iron reinforcement is not provided. A value of C = 0.75 should be used for the turnout or second gate. When the orifice gate is set for maximum capacity, the bottom of the gate leaf must extend below the breast wall a distance equal to or greater than the nominal thickness of the breast wall. Where accuracies better than ± 7 percent are required, careful field ratings of the turnout must be made.

For good accuracy with discharges as shown in the table or computed by the orifice equation, the upstream submergence must be equal to or greater than the gate opening for maximum capacity. Less submergence reduces the accuracy and calibration is required. The top of the turnout gate opening must be below the water surface in the measuring box by at least 1.78 times the velocity head of the full pipe plus 3 inches.

The above seals are required for accurate water measurement. Where the constant-head orifice is used as a turnout with closed conduit, the conduit and outlet should be treated the same as a siphon or pipe drop depending on the local conditions. An air vent as shown on Figure 1 must be provided wherever the delivery water surface at any flow is such that a vacuum can develop downstream of the control gate. Dimensions for a constant-head orifice are shown in Figure 2.

Dimensions

B. For accurate measurement, a level floor must be provided in front of the orifice gate, of a length equal to or greater than the height of the orifice gate opening for full capacity. The inside minimum length of the measuring box should be at least 2-1/4 times the orifice gate opening for maximum capacity or 1-3/4 times the wall opening, whichever is larger, for turnouts with maximum capacities up to 10 cfs, and 2-3/4 times the height of the orifice gate opening at maximum capacity for structures with maximum capacities above 10 cfs. The inlet walls should be parallel unless extra width is needed at the inlet cutoff to prevent it from being a control in which case extra width may be obtained by flaring the walls, customarily 8:1.

The distance between the inlet cutoff and orifice gate should be a minimum of 1-1/2 times the difference in elevation between the invert at the

6.3

Dimensions (Cont'd.)

cutoff and at the orifice gate. In an earth canal, the top of the sloping inlet walls should intersect the canal sideslope at or a few inches above normal water surface. The inlet walls are usually sloped steeper than the canal sideslope and set back into the bank so they will not be out in the canal if it is widened at the bottom during cleaning or reshaping. The following tabulation shows recommended inlet wall slopes corresponding to various canal sideslopes.

Canal sideslope	Inlet wall slope
2-1/2:1	2:1
2:1	1-1/2:1
1-1/2:1	1-1/2:1 set into the bank 12 to 24 inches depending on size of canal and local conditions

OPEN FLOW-METERS

.3 Open flowmeters are propeller devices generally used at the discharge ends of closed conduits where rate of flow and totalization recording are desired, or where a minimum head loss is required. Water carrying considerable sediment or trash can cause trouble with these meters. Therefore, trashracks are sometimes provided at the upstream end of the conduit to prevent fouling of the meter. Open flowmeters may be attached to the outlet transition headwall on turnouts, siphons, and similar structures. The outlet transition must be designed to force the conduit to flow full at the meter location. The desirable velocity in the conduit is from 1 to 8 feet per second for accurate measurement with the open flowmeter. Most meters operate satisfactorily with velocities down to 1/2 foot per second. Also, the accuracy of the meter is not affected by velocities higher than 8 feet per second, but the life of the standard meter bearings is reduced to the extent that excessive repair and maintenance will result. A special heavy-duty meter can be obtained for higher velocities, in which case velocities up to 13 feet per second have been allowed. A minimum of 6 diameters of straight, level conduit must be provided immediately upstream from the meter location unless straightening vanes are used. Figure 3 shows a typical turnout with an open flowmeter in the outlet transition.

LINE METERS

.4 Line meters are propeller devices which operate on the same principle as open flowmeters. They are generally installed in a pipeline with a well to the ground surface to protect the recorder. Hydraulic losses are minor for meters and can usually be neglected.

PARSHALL FLUMES

.5 Parshall flumes can be designed for measurement of small to large flows; Parshall's tables were prepared for capacities up to 3, 300 cfs through a 50-foot-wide flume. Since the velocity through the structure is higher than that in the adjacent channel, waterborne sand and silt will not deposit in the structure to affect the accuracy, which is ordinarily within 5 percent.

Dimensions

A. Measurement of discharge through a Parshall flume is based on coefficients determined by experiments. The hydraulic design procedure must follow the rules and dimensions in Colorado State College Bulletins No. 386 or No. 426-A "Parshall Flumes of Large Size" and No. 423 "The Parshall Measuring Flume." Parshall's tables can be used where the structure is designed with a converging upstream section and level floor, a downward sloping throat, and an upward sloping section diverging downstream with all dimensions in accordance with the above bulletins. The Bureau's Water Measurement Manual also contains discharge tables for Parshall flumes. Modified Parshall flumes have often been

6.6

Dimensions (Cont'd.)

used without the upward sloping section diverging downstream, where an outlet transition or stilling pool has been provided. Calibration of these flumes is required to determine their performance.

A straight channel of sufficient length to provide uniform flow must be provided upstream of the Parshall flume. If the structure is to be placed upstream of a reach of canal where the bottom may build up due to sediment or where a checked water surface affects the flow, care must be taken to prevent submergence which could make the structure inoperative. Figure 4 shows details of a 185-cfs Parshall flume, and Figure 5 shows details of a modified Parshall flume and drop.

WEIRS

.6 The use of weirs is limited to locations where adequate head is available. Although other types of weirs have been developed, the Cipolletti and rectangular weirs are generally used. The Bureau's Hydraulic and Excavation Tables and the Manual for Measurement of Irrigation Water contain tables giving the discharge for various weirs. The accuracy is reduced if the upstream pool is not maintained, if the head on the weir is greater than one-third the weir length, or if the depth over the weir is less than 0. 2 foot. The velocity of approach should be small, or corrections for the approach velocity and other factors made as described in the reference tables. The weir crest should be sharp edged, and the stream over the weir should have free fall with admission of air under the stream.

Division Box and Adjustable Weir

A. Movable and adjustable Cipolletti weirs, which can be easily removed from the structure or adjusted vertically within a frame, provide a means of control in addition to measurement, and are often used in division boxes on lateral systems. Figure 6 shows two types of pipe division boxes. Figure 7 shows division boxes for an open lateral. Figure 8 shows the adjustable weir.

Weir Box

B. Figure 9 shows the weir box. This structure is for a maximum discharge of 5 cfs. Designs are also available for discharges to 10 cfs.

CONSTANT HEAD ORIFICE TURNOUTS--TYPE 4A

Fig. 1 Par. 6.2

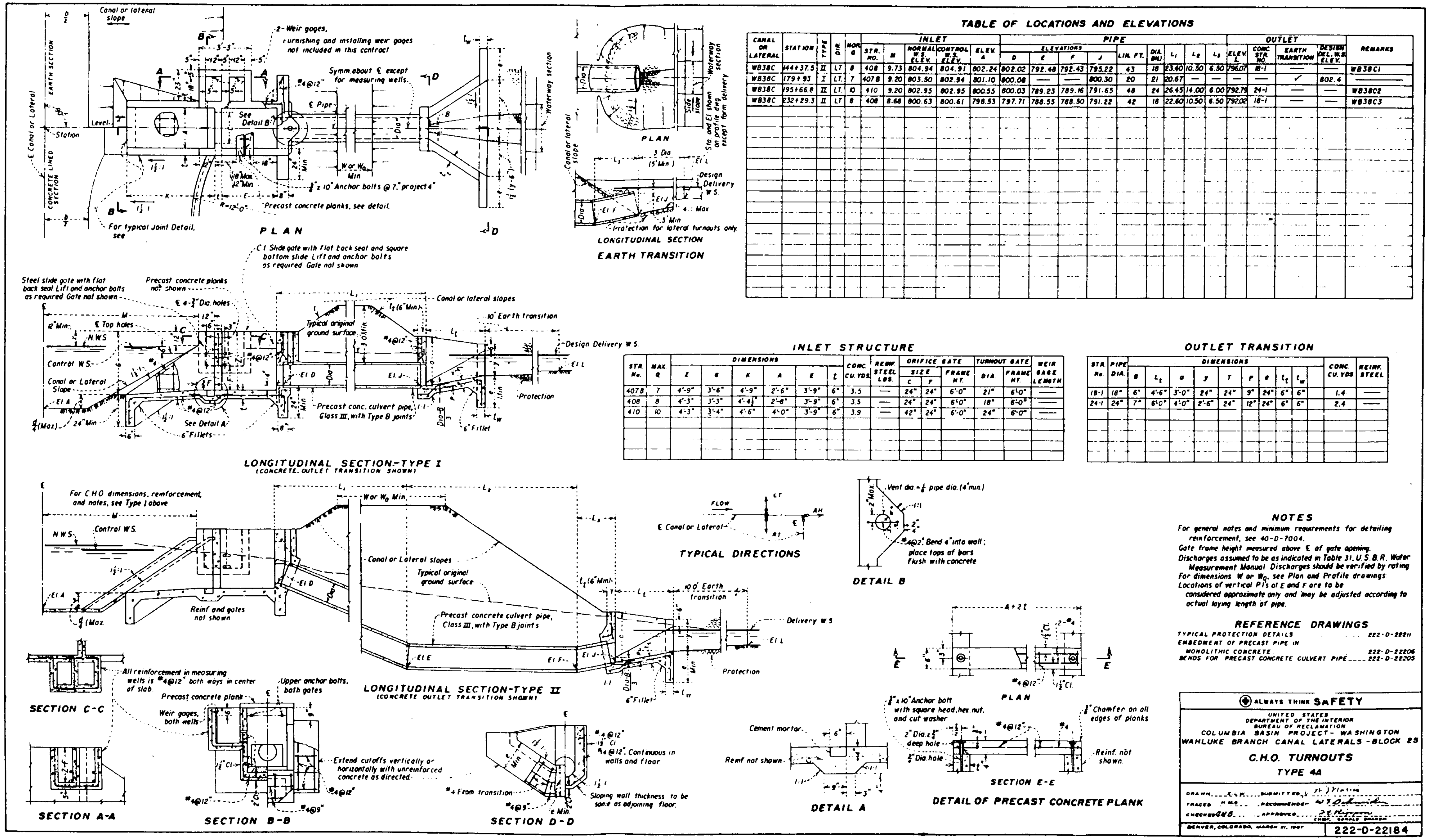


TABLE OF LOCATIONS AND ELEVATIONS

CANAL OR LATERAL	STATION	TYPE	DIR.	NOR. Q	INLET: STR. NO.	INLET: M	INLET: NORMAL W.S. ELEV.	INLET: CONTROL W.S. ELEV.	INLET: ELEV. A	PIPE ELEVATIONS: D	PIPE ELEVATIONS: E	PIPE ELEVATIONS: F	PIPE ELEVATIONS: J	PIPE: LIN. FT.	PIPE: DIA. (IN.)	PIPE: L_1	PIPE: L_2	PIPE: L_3	OUTLET: ELEV. L	OUTLET: CONC. STR. NO.	OUTLET: EARTH TRANSITION	OUTLET: DESIGN DEL. W.S. ELEV.	REMARKS
WB38C	144+37.5	II	LT.	8	408	9.73	804.94	804.91	802.24	802.02	792.48	792.43	795.22	43	18	23.40	10.50	6.50	796.07	18-1	—	—	WB38C1
WB38C	179+93	I	LT.	7	407B	9.20	803.50	802.94	801.10	800.08	—	—	800.30	20	21	20.67	—	—	—	—	✓	802.4	—
WB38C	195+66.8	II	LT.	10	410	9.20	802.95	802.95	800.55	800.03	789.23	789.16	791.65	48	24	26.45	14.00	6.00	792.79	24-1	—	—	WB38C2
WB38C	232+29.3	II	LT	8	408	8.68	800.63	800.61	798.53	797.71	788.55	788.50	791.22	42	18	22.60	10.50	6.50	792.02	18-1	—	—	WB38C3

INLET STRUCTURE

STR. No.	MAX. Q	DIMENSIONS: Z	DIMENSIONS: G	DIMENSIONS: K	DIMENSIONS: A	DIMENSIONS: E	DIMENSIONS: t	CONC. CU. YDS.	REINF. STEEL LBS.	ORIFICE GATE SIZE: C	ORIFICE GATE SIZE: F	ORIFICE GATE: FRAME HT.	TURNOUT GATE: DIA.	TURNOUT GATE: FRAME HT.	WEIR GAGE LENGTH
407B	7	4'-9"	3'-6"	4'-9"	2'-6"	3'-9"	6"	3.5	—	24"	24"	6'-0"	21"	6'-0"	—
408	8	4'-3"	3'-3"	4'-4½"	2'-8"	3'-9"	6"	3.5	—	24"	24"	6'-0"	18"	6'-0"	—
410	10	4'-3"	3'-4"	4'-6"	4'-0"	3'-9"	6"	3.9	—	42"	24"	6'-0"	24"	6'-0"	—

OUTLET TRANSITION

STR. No.	PIPE DIA.	DIMENSIONS: B	DIMENSIONS: L_t	DIMENSIONS: a	DIMENSIONS: y	DIMENSIONS: T	DIMENSIONS: F	DIMENSIONS: e	DIMENSIONS: t_t	DIMENSIONS: t_w		CONC. CU. YDS.	REINF. STEEL
18-1	18"	6"	4'-6"	3'-0"	24"	24"	9"	24"	6"	6"		1.4	—
24-1	24"	7"	6'-0"	4'-0"	2'-6"	24"	12"	24"	6"	6"		2.4	—

NOTES

For general notes and minimum requirements for detailing reinforcement, see 40-D-7004.
Gate frame height measured above ℄ of gate opening.
Discharges assumed to be as indicated in Table 31, U.S.B.R. Water Measurement Manual Discharges should be verified by rating
For dimensions W or W_0, see Plan and Profile drawings.
Locations of vertical PI's at E and F are to be considered approximate only and may be adjusted according to actual laying length of pipe.

REFERENCE DRAWINGS

TYPICAL PROTECTION DETAILS 222-D-22211
EMBEDMENT OF PRECAST PIPE IN MONOLITHIC CONCRETE 222-D-22206
BENDS FOR PRECAST CONCRETE CULVERT PIPE 222-D-22205

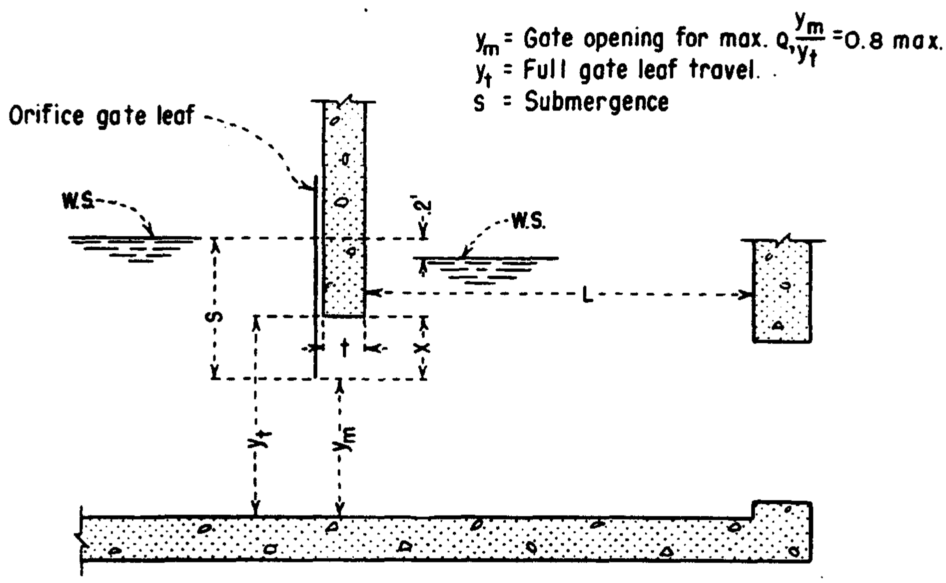
ALWAYS THINK SAFETY

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
COLUMBIA BASIN PROJECT- WASHINGTON
WAHLUKE BRANCH CANAL LATERALS -BLOCK 25

C.H.O. TURNOUTS
TYPE 4A

DRAWN ...C.L.K.... SUBMITTED ...
TRACED ...H.M.O.... RECOMMENDED ...
CHECKED ...G.W.B.... APPROVED ... CHIEF, CANALS BRANCH

DENVER, COLORADO, MARCH 21, 1967 | 222-D-22184



"X" must be equal to or greater than "t" for max. Q.
"S" is equal to or greater than "y_m" for good accuracy.
For Q up to 10 cfs, L must be at least $2\frac{1}{4}$ y_m or $1\frac{3}{4}$ y_t, whichever is greater. (3'-6" minimum)
For Q above 10 cfs, L = $2\frac{3}{4}$ y_m minimum.

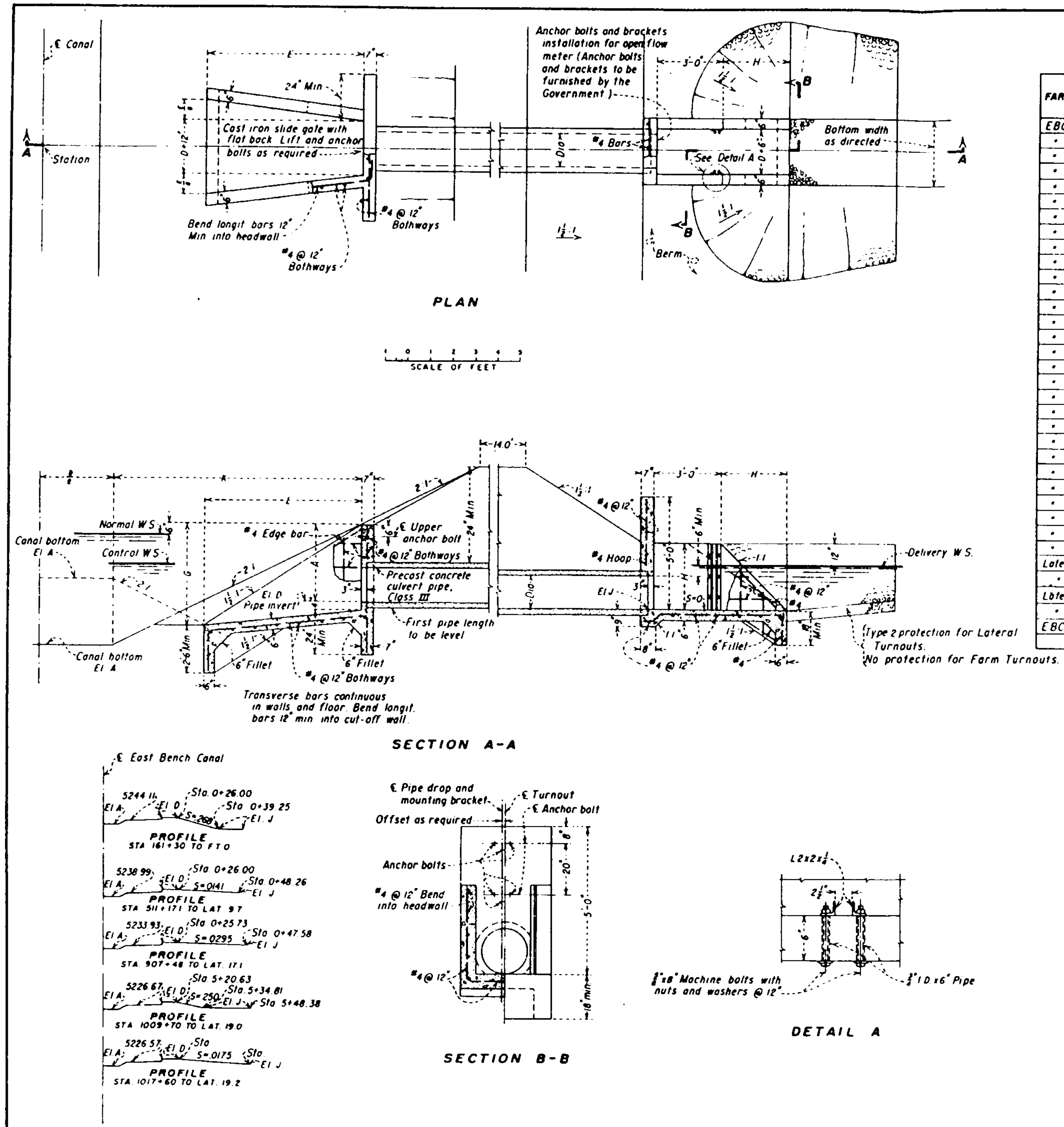
DIMENSIONS FOR A CONSTANT-HEAD ORIFICE

REV. 3-29-62
OCT. 31, 1960

103-D-644

TURNOUT WITH OPEN FLOW METER--EAST BENCH CANAL AND LATERALS

Fig. 3 Par. 6.3



TABLE

FARM & LATERAL TURNOUTS	T.O. TO LAT. OR FARM	ELEVATIONS			INLET STRUCTURE DIMENSIONS				PIPE					OUTLET STRUCTURE		Q	DEL. W.S.
		NWS	CWS	INVERT A	A	K	E	G	DIA.	L	El. D	SLOPE	El. J	MEASURING DEVICE	DIM. H		
EBC Sta. 161+30 F.T.O. Lt.	Farm	5249.28	5247.88	5242.65	4'-0"	14'-3 1/2"	8'-6"	5'-8"	18"	24'-0"	5245.78	.0261 .009	5245.44	Fixed Weir See 699-D-114		5	5246.1
• • 177+00 • •	•	5249.08	47.88	42.45	4'-0"	•	•	•	18"	26'-0"	45.58	Level	45.58	Open flow meter	3'-0"	5	47.57
• • 210+50 • •	•	48.66	47.88	42.03	4'-0"	•	•	•	•	24'-0"	45.16	•	45.16	• • •	3'-6"	5	47.57
• • 251+99 • •	•	48.14	47.88	41.51	•	•	•	•	•	26'-0"	44.64	•	44.64	• • •	•	7.5	47.38
• • 273+30 • •	•	47.88	•	41.25	•	•	•	•	•	24'-0"	44.38	•	44.38	• • •	4'-0"	•	•
• • 330+00 • •	•	47.16	46.75	40.53	•	•	•	•	•	•	43.66	•	43.66	• • •	3'-6"	•	46.24
• • 338+60 • •	•	47.06	•	40.43	•	•	•	•	•	•	43.56	•	43.56	• • •	4'-0"	5	46.44
• • 362+75 • •	•	46.75	•	40.12	•	•	•	•	•	•	43.25	•	43.25	• • •	•	•	46.43
• • 376+90.7 • •	•	45.81	44.16	39.18	4'-6"	•	•	•	•	26'-0"	41.81	•	41.81	• • •	3'-0"	•	43.86
• • 397+00 • •	•	45.56	•	38.93	•	•	•	•	•	•	41.56	•	41.56	• • •	3'-6"	•	43.83
• • 418+80 • •	•	45.29	44.16	38.66	4'-0"	•	•	•	•	•	41.79	•	41.79	• • •	3'-0"	•	•
• • 496+07 • •	•	44.35	•	37.72	•	•	•	•	•	24'-0"	40.85	•	40.85	• • •	4'-0"	7.5	43.7
• • 547+08 • •	•	43.71	42.62	37.08	•	•	•	•	•	•	40.21	•	40.21	• • •	3'-0"	5	42.3
• • 571+65.2 • •	•	43.40	•	36.77	•	•	•	•	•	•	39.90	•	39.90	• • •	3'-6"	•	•
• • 714+00 • •	•	41.59	40.71	35.30	•	13'-7"	•	•	•	•	38.09	•	38.09	• • •	•	•	40.4
• • 737+42 • •	•	41.28	•	34.99	3'-[illegible]"	•	•	•	•	•	38.28	•	38.28	• • •	•	•	•
• • 781+00 • •	•	40.72	•	34.43	•	•	•	•	•	•	37.72	•	37.72	• • •	•	•	40.41
• • 881+75 • •	•	39.44	38.61	33.15	4'-0"	•	•	•	•	•	35.94	•	35.94	• • •	•	•	38.3
• • 1038+75 • •	•	31.47	31.13	25.18	3'-6"	•	•	•	•	•	28.47	•	28.47	• • •	•	•	30.8
• • 1044+00 • •	•	31.40	•	25.11	•	•	•	•	•	•	28.40	•	28.40	• • •	•	•	•
• • 195+98 L.T.O. •	Lat. 3.7	48.84	47.88	42.21	7'-1 1/2"	14'-3 1/2"	11'-3"	7'-6"	•	30'-0"	42.21	•	42.21	• • •	3'-0"	•	44.2
• • 226+02 • •	Lat. 4.3	48.46	•	41.83	4'-0"	•	8'-6"	5'-8"	•	24'-0"	44.96	•	44.96	• • •	3'-6"	7.5	47.28
• • 310+31 • •	Lat. 5.9	47.42	46.75	40.79	4'-3"	•	•	•	24"	•	43.67	•	43.67	• • •	•	15.	46.25
• • 511+171 • •	Lat. 9.7	44.16	44.16	37.53	5'-3"	•	•	•	18"	•	39.41	.014	39.10	Division Box, see 699-D-103		7.5	43.6
• • 634+15 • •	Lat. 12.0	42.62	42.62	35.99	3'-3"	•	•	•	•	•	39.87	Level	39.87	Open flow meter	3'-0"	•	41.92
• • 907+48 • •	Lat. 17.1	39.10	38.61	32.81	5'-3"	13'-7"	8'-6"	5'-8"	24"	•	34.35	.0295	33.70	Division Box, see 699-D-104		10.4	38.2
• • 929+97 • •	Lat. 17.5	38.81	38.76	32.51	4'-0"	•	•	•	•	30'-0"	35.31	Level	35.31	Open flow meter	3'-6"	15.7	37.8
• • 1009+50 • •	Lat. 19.0	31.84	31.13	25.55	•	•	•	•	•	30'-0"	28.34	.150 Level	24.80	• • •	•	10.4	27.3
• • 1017+60 • •	Lat. 19.2	31.74	31.13	25.45	C.H.O. See 699-D-295				30"	24'-0"	27.32	Level	27.32	Transition 30-2, see 699-D-94		18.4	30.32
Lateral 5.9 3+11.29 F.T.O. Rt.	Farm	46.12	46.1	44.52	3'-0"	6'-10 1/2"	4'-5 1/4"	3'-4"	18"	16'-0"	43.62	Level	43.62	Open flow meter	3'-0"	7.5	45.6
• • 3+41.24 • •	•	•	•	•	•	•	•	•	•	•	•	•	•	• • •	•	•	•
Lateral 19.2 3+50 • Lt.	•	30.17	—	—	—	—	See 699-D-106				—	—	26.17	• • •	4'-6"	5	5229.87
• • • • Rt.	•	•	—	—	—	—	See 699-D-106				—	—	•	• • •	•	•	•
EBC Sta. 579+00 F.T.O. Lt.	•	43.31	42.62	36.68	4'-0"	14'-3 1/2"	8'-6"	5'-8"	18"	24'-0"	39.80	Level	39.80	• • •	3'-6"	5	5242.3

NOTE

For typical protection details see 699-D-112.

GENERAL NOTES

Monolithic concrete design based on a compressive strength of 3000 lbs. per sq. inch.
Dimensions are to center line of joints, unless otherwise shown.
For embedment details see 699-D-113.

2-8-62	AT STA. 1017+60 EBC THE TURNOUT STRUCTURE IS REPLACED BY A CONSTANT HEAD ORIFICE STRUCTURE.

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
MISSOURI RIVER BASIN PROJECT
THREE FORKS DIVISION-EAST BENCH UNIT-MONTANA

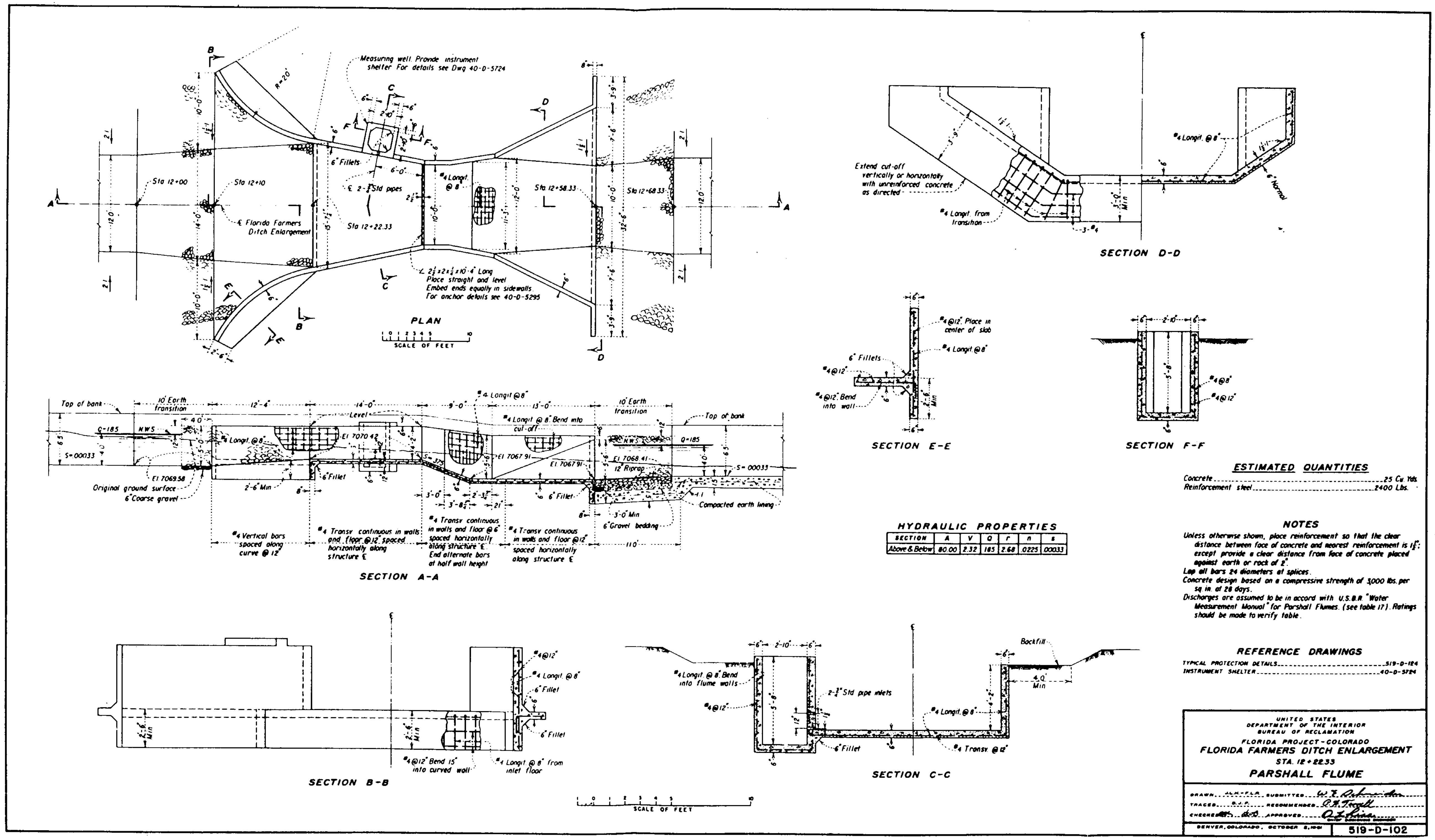
EAST BENCH CANAL AND LATERALS
TURNOUTS

DRAWN C.S.E. SUBMITTED
TRACED W.W.S. RECOMMENDED
CHECKED APPROVED
CHIEF DESIGNING ENGINEER

DENVER, COLORADO, MARCH 13, 1961 | 699-D-100

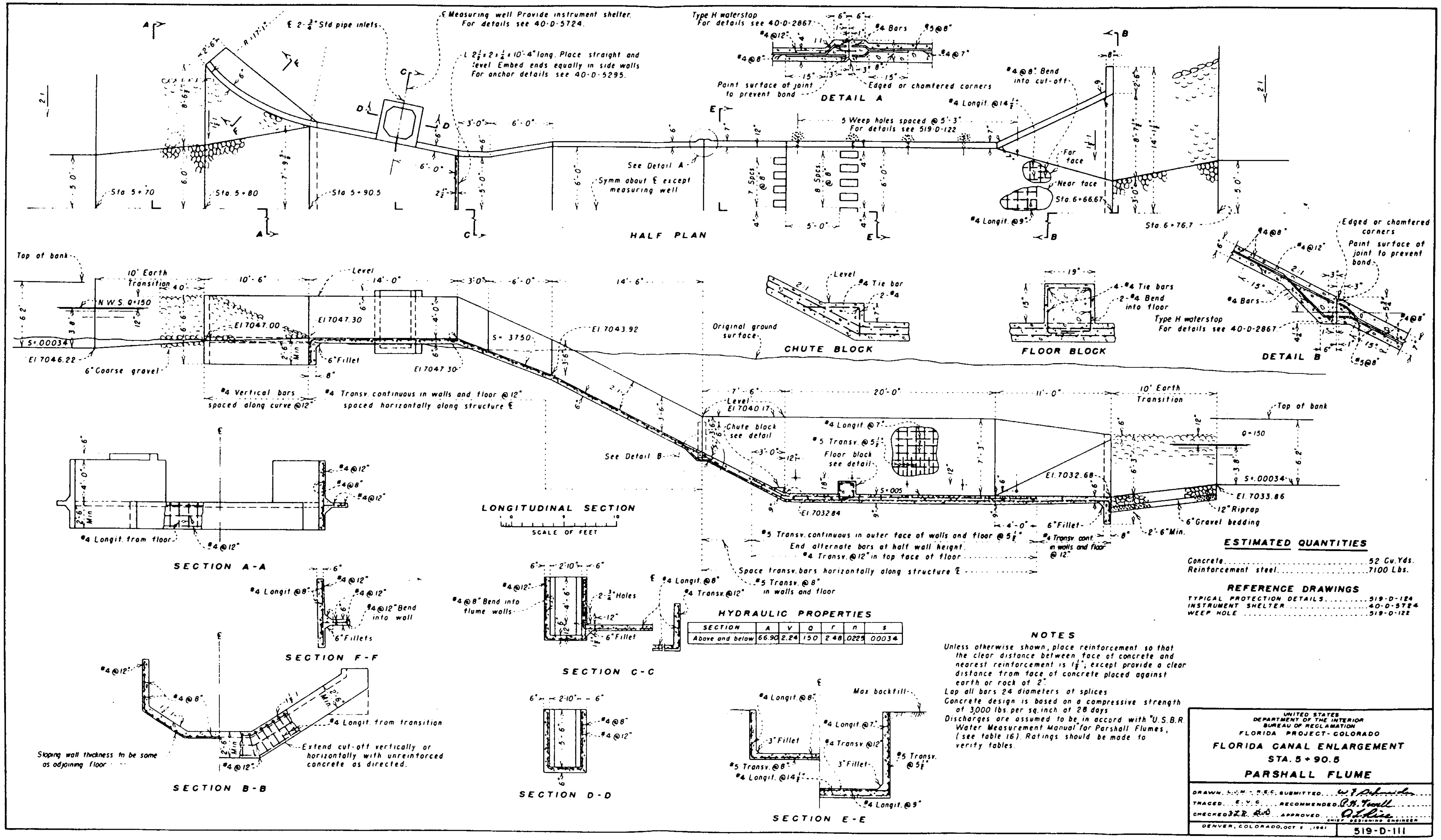
PARSHALL FLUME--FLORIDA FARMERS DITCH, STATION 12 + 22.33

Fig. 4 Par. 6.5A



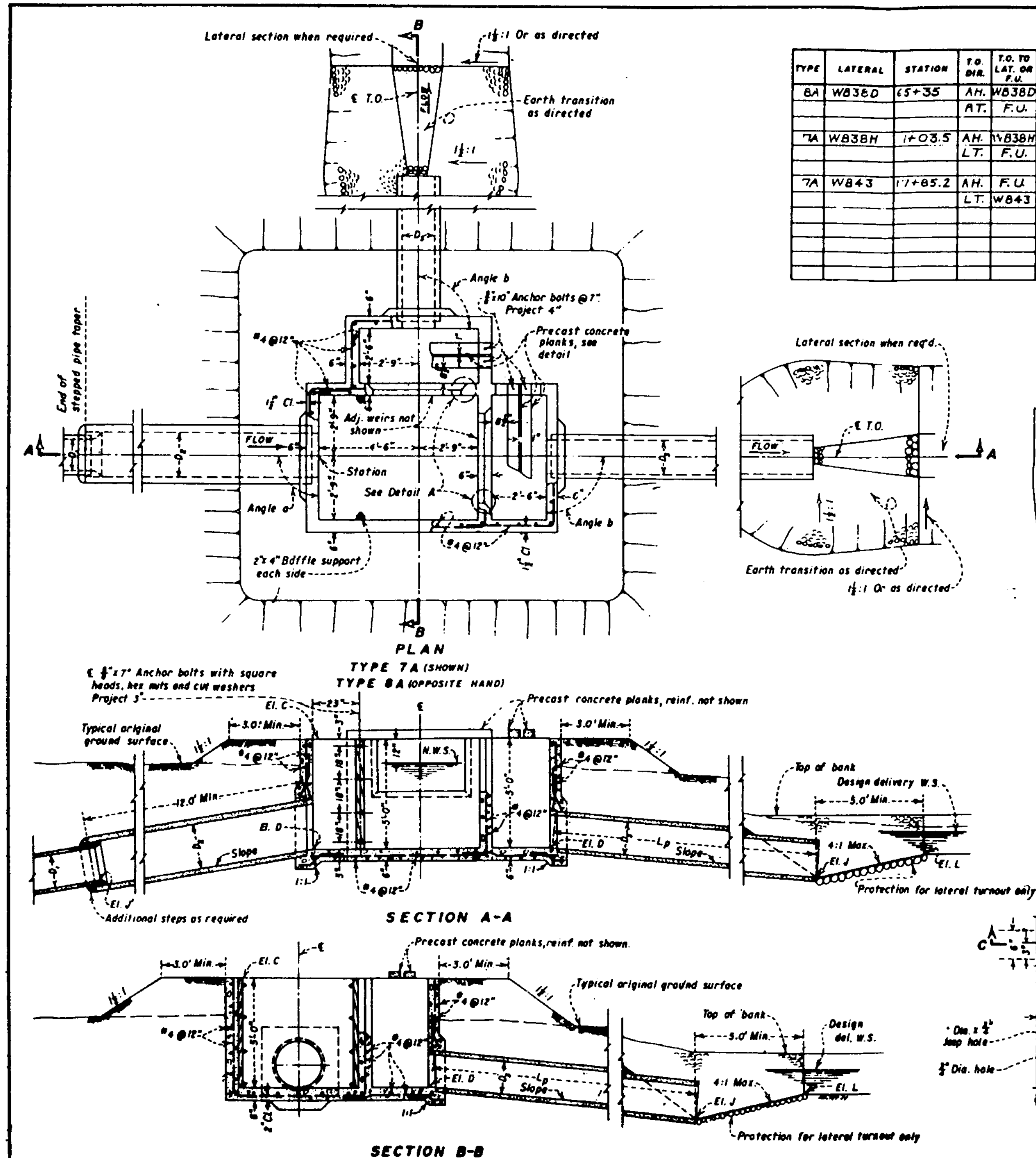
MODIFIED PARSHALL FLUME AND DROP--FLORIDA CANAL ENLARGEMENT, STATION 5+90.5

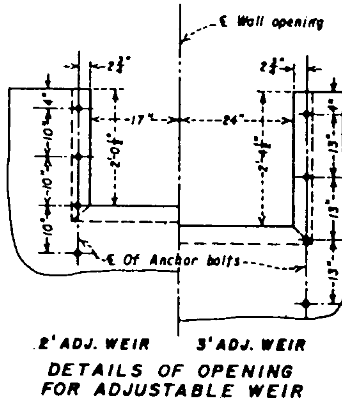
Fig. 5 Par. 6.5A



PIPE DIVISION BOXES--TYPES 7A AND 8A

Fig. 6 Par. 6.6A





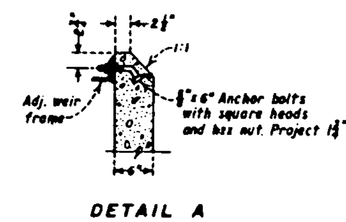


TABLE OF LOCATIONS AND ELEVATIONS

TYPE	LATERAL	STATION	T.O. DIR.	T.O. TO LAT. OR F.U.	Q	INLET PIPE D_1	INLET PIPE D_2	INLET PIPE SLOPE	INLET PIPE ANGLE a	ELEVATIONS	C	NORMAL W.S.	D	J	L	DESIGN DEL. W.S.	OUTLET PIPE D_3	OUTLET PIPE L_p	OUTLET PIPE SLOPE	OUTLET PIPE ANGLE b	ADJ. WEIR (W)	REMARKS
8A	W838D	65+35	AH.	W838D	7	30"	42"		90°		788.62	787.62	783.62	781.21	782.64	—	30"	16'	-.151	90°	3'	(4)
			RT.	F.U.	7								783.62	784.00		786.64	21"	20'	+.019		3'	
7A	W838H	1+03.5	AH.	W838H	7	24"	30"		90°		700.63	699.63	695.63	695.43	696.28	—	21"	13'	-.015	90°	3'	(4)
			LT.	F.U.	7								695.63	696.75		698.80	21"	20'	+.056		3'	
7A	W843	17+85.2	AH.	F.U.	7	15"	30"		90°		896.60	895.60	891.60	892.50		894.70	21"	12'	+.075	90°	3'	
			LT.	W843	7								891.60	—		—	21"	—	—	89°14'	3'	(1)

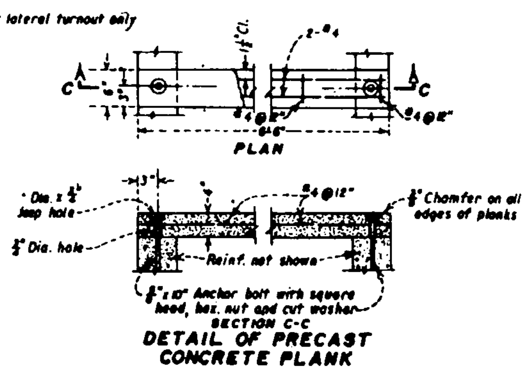
(1) Connect to road crossing
(2) Connect to pipe line
(3) Connect to pipe drop
(4) 10' Earth transition

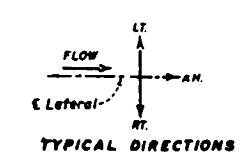
ESTIMATED QUANTITIES

Concrete.................6.5 Cu. Yds.
Reinforcement steel.................540 Lbs.

NOTES

For general notes and minimum requirements for detailing reinforcement, see 40-D-7004.
Extend cutoff and wingwalls vertically and horizontally with unreinforced concrete as directed.
All pipe is precast concrete culvert pipe, Class II with Type B joints, unless otherwise shown





REFERENCE DRAWINGS

ADJUSTABLE WEIR W=2'-0" AND 3'-0".................222-D-14596
EMBEDMENT OF PRECAST PIPE IN MONOLITHIC CONCRETE.................222-D-22206
TYPICAL PROTECTION DETAILS.................222-D-22211
STEPPED PIPE TAPERS.................222-D-22207

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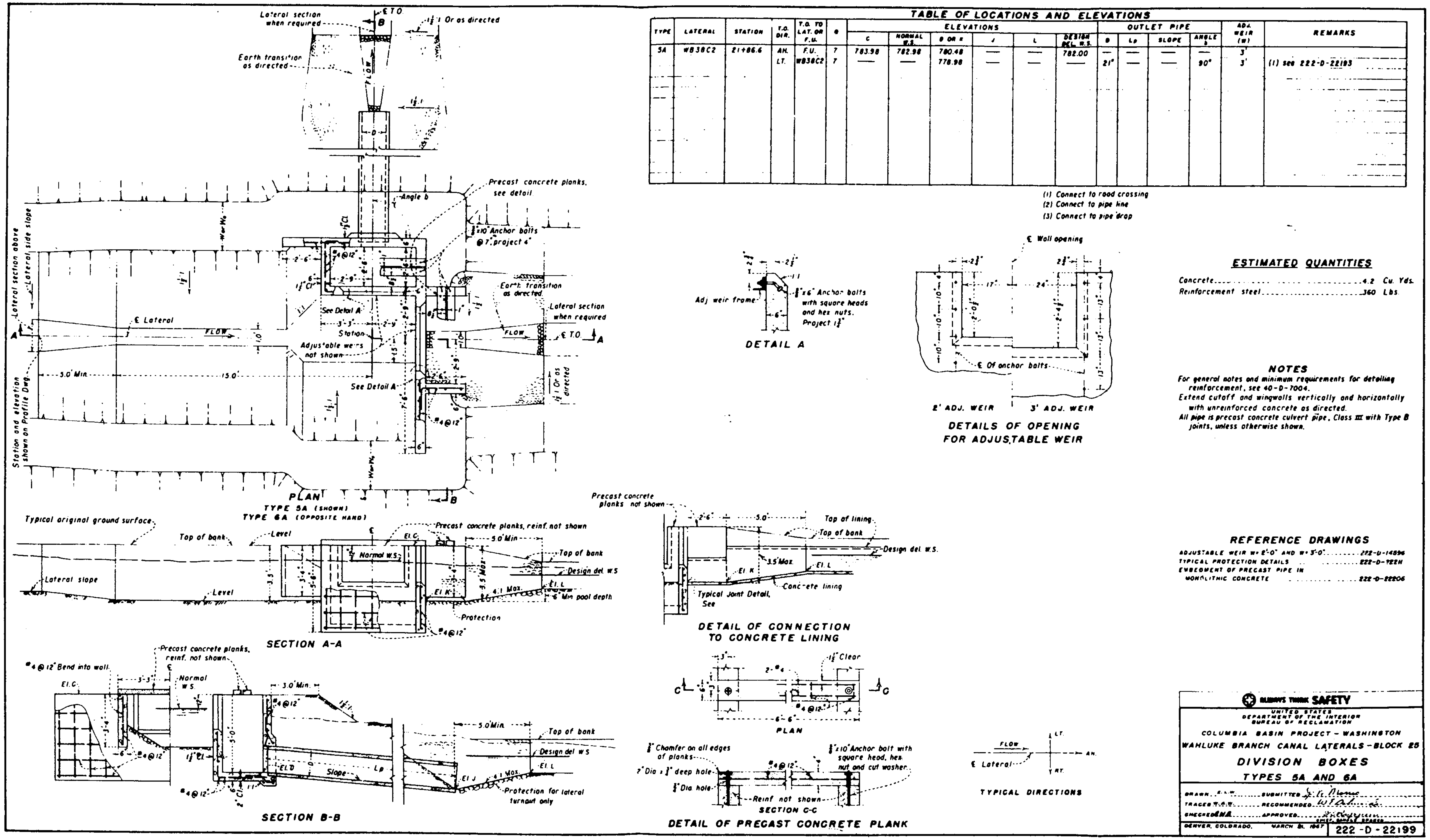
COLUMBIA BASIN PROJECT - WASHINGTON
WAHLUKE BRANCH CANAL LATERALS-BLOCK 25

PIPE DIVISION BOXES
TYPES 7A AND 8A

DRAWN S.H.B. SUBMITTED J. N. Morris
TRACED ... RECOMMENDED W. T. ...
CHECKED ... APPROVED ...

DENVER, COLORADO, MARCH 21, 1957 | 222-D-22200

DIVISION BOXES--TYPES 5A AND 6A



TYPE	LATERAL	STATION	T.O. DIR.	T.O. TO LAT. OR F.U.	θ	C	NORMAL W.S.	B OR K	J	L	DESIGN DEL. W.S.	D	Lp	SLOPE	ANGLE b	ADJ. WEIR (W)	REMARKS
5A	WB38C2	21+86.6	AH.	F.U.	7	783.98	782.98	780.48	—	—	782.00	—	—	—	—	3'	
			LT.	WB38C2	7	—	—	778.98	—	—	—	21"	—	—	90°	3'	(1) see 222-D-22193

ADJUSTABLE WEIR--2- AND 3-FOOT WIDTHS

Fig. 8 Par. 6.6A

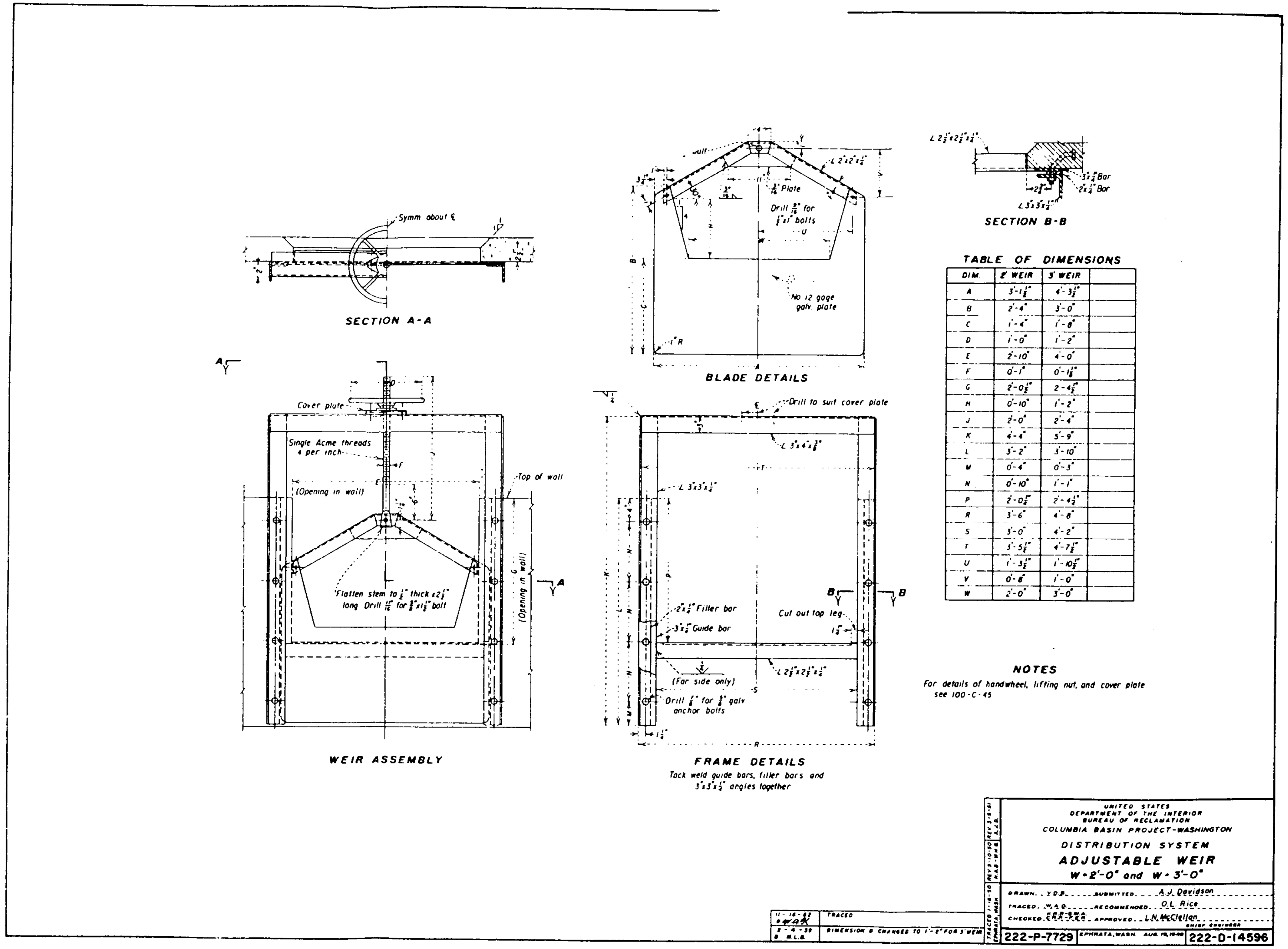


TABLE OF DIMENSIONS

DIM.	2' WEIR	3' WEIR	
A	3'-1 1/2"	4'-3 1/2"	
B	2'-4"	3'-0"	
C	1'-4"	1'-8"	
D	1'-0"	1'-2"	
E	2'-10"	4'-0"	
F	0'-1"	0'-1 1/8"	
G	2'-0 1/2"	2-4 1/2"	
H	0'-10"	1'-2"	
J	2'-0"	2'-4"	
K	4'-4"	5'-9"	
L	3'-2"	3'-10"	
M	0'-4"	0'-3"	
N	0'-10"	1'-1"	
P	2'-0 1/4"	2'-4 1/4"	
R	3'-6"	4'-8"	
S	3'-0"	4'-2"	
T	3'-5 1/2"	4'-7 1/2"	
U	1'-3 1/2"	1'-10 1/2"	
V	0'-8"	1'-0"	
W	2'-0"	3'-0"	

WEIR BOX WITH METAL BAFFLE ASSEMBLY--MAXIMUM DISCHARGE = 5 CFS

Fig. 9 Par. 6.6B

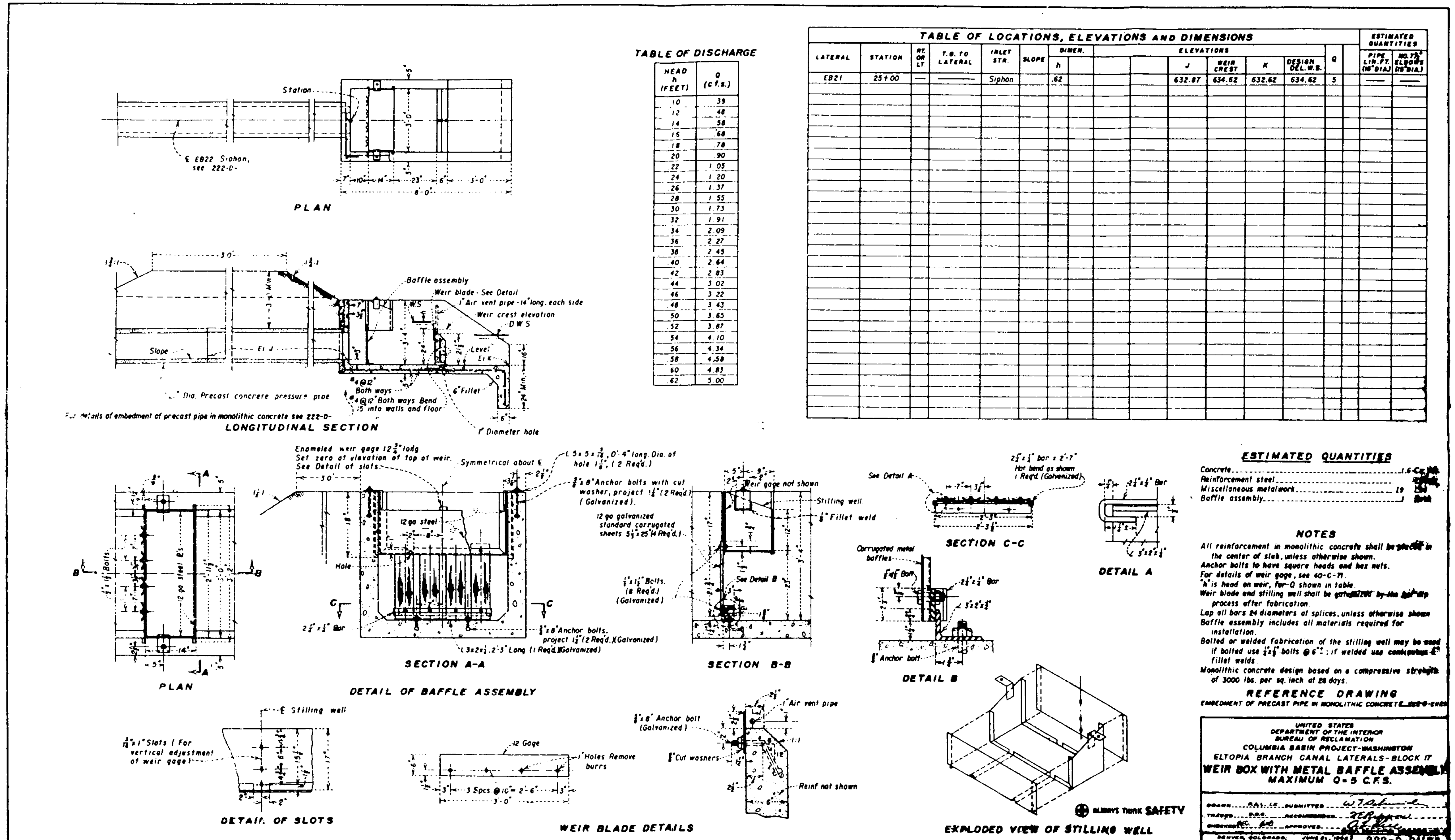


TABLE OF DISCHARGE

HEAD h (FEET)	Q (c.f.s.)
.10	.39
.12	.48
.14	.58
.15	.68
.18	.78
.20	.90
.22	1.05
.24	1.20
.26	1.37
.28	1.55
.30	1.73
.32	1.91
.34	2.09
.36	2.27
.38	2.45
.40	2.64
.42	2.83
.44	3.02
.46	3.22
.48	3.43
.50	3.65
.52	3.87
.54	4.10
.56	4.34
.58	4.58
.60	4.83
.62	5.00

TABLE OF LOCATIONS, ELEVATIONS AND DIMENSIONS

LATERAL	STATION	RT. OR LT.	T.B. TO LATERAL	INLET STR.	SLOPE	DIMEN. h	DIMEN.	ELEVATIONS	ELEVATIONS	ELEVATIONS J	ELEVATIONS WEIR CREST	ELEVATIONS K	ELEVATIONS DESIGN DEL. W.S.	Q		ESTIMATED QUANTITIES PIPE LIN. FT. (18" DIA.)	ESTIMATED QUANTITIES NO. ELBOWS (18" DIA.)
EB21	25+00	—	—	Siphon		.62				632.87	634.62	632.62	634.62	5		—	—

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7.2	Culverts
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7.2B	Outlet
7.3	Overchutes
7.4	Drain Inlets
7.5	Spillways
7.5A	Overflow Spillway
7.5B	Radial-gate Spillway
7.5C	Siphon Spillway
7.6	Wasteways

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2	Overchute--Putah South Canal, Station 1534+80	7.3	413-D-576
3	Irrigation Crossings--Florida Farmers Ditch Enlargement	7.3	519-D-118
4	Wasteway Plan and Sections--Osborne Canal, Station 443+00	7.5	468-D-330
5	Wasteway Inlet Details--Osborne Canal, Station 443+00	7.5	468-D-331
6	Typical Low-head Siphon Spillway--Design Data	7.5C	103-D-46

7.1

INTRODUCTION

.1 This chapter contains design information for structures that convey storm or drainage water under, over, or into canals; also structures that provide controlled discharge of excess water from the canal for safety of operation. General structural and hydraulic design criteria relating to cross drainage and protective structures are given in Chapter 2. In the structural drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated.

CULVERTS

.2 Culverts are structures that carry flood or drainage water under canals, laterals, roads and railroads. Various shapes and materials have been used in culverts, but the most common are single- or multiple-barrel precast concrete pipe and rectangular monolithic concrete conduits. Adequate cutoffs, at least one on each side of the canal, should be provided to prevent percolating water from following the conduit and causing washouts. Where the culvert barrel passes under an earth canal, the top of the barrel should be at least 24 inches below the canal invert. Where the culvert barrel passes under a lined canal, a clear distance of 3 inches should be allowed between the barrel and the lining. In determining the minimum size of a culvert carrying floodwater, the quantity of flow which will be produced by a 25-year frequency flood, divided by a velocity of 10 feet per second, should be used to obtain the area required, except that if a baffled outlet is provided a velocity of 12 feet per second may be used. With the minimum size so determined, selection of the magnitude of storm for which the culvert is designed should take into consideration the local hazards involved such as the effect of ponding above the canal, the possibility of damage from water discharged from the culvert, and the overall damaging effect of possible structure failure. A minimum culvert diameter of 24 inches is usually set for use on a project. The size will depend on the damage that plugging might cause and on the type of debris expected from the drainage area. Figure 1 shows typical pipe culverts with transitions.

Inlet

A. Inlets may consist of the plain end of the culvert pipe, a straight or bent headwall, or various types of concrete transitions. The inlet should be determined on the basis of existing conditions. It must permit the design flow to enter the culvert with adequate freeboard for canal protection. This freeboard is normally 2 feet or more for the design flood. The inlet invert should be located near the existing ground surface or at the bottom grade of an inlet channel, if provided, to prevent degradation above the culvert. If the conduit slope is steep and the outlet water surface low, the capacity of the culvert will be controlled by the inlet. In this case the depth of ponding above the canal will be determined from the control at the inlet cutoff or the headwall, whichever governs. If the downstream water surface is high enough to make the conduit flow full, the hydraulics to determine the capacity and pondage should be computed on the basis of a submerged conduit with proper loss coefficient for the existing conditions.

Outlet

B. The outlet should be designed to protect the structure against local scour. The safety of the outlet is also dependent on the stability of the downstream channel. This waterway must resist gullying or the culvert may fail regardless of the outlet type. Baffled outlets or other types of energy dissipators should be used where it is required that adjacent facilities be protected from local scour. A level length of pipe equal to at least three pipe diameters is desirable at the inlet to baffled outlet structures; however, slopes up to 15° have been permitted. Baffled outlets may be objectionable if the culvert transports debris that might plug the structure.

OVERCHUTES

.3 An overchute is a structure that carries flood or drainage water across and above the canal prism. An overchute should be used where the cross drainage gradient is sufficiently high to provide freeboard over the canal water

7.4

OVER-CHUTES (Cont'd.)

surface without excessive ponding at the inlet. Rectangular concrete flume sections are generally used for large flows, while closed conduits are often used for small flows. Transverse ribs are sometimes provided on the flume floor of large structures to retain an insulating earth layer which will reduce the induced temperature stresses in the concrete floor and piers. Figure 2 shows an overchute with a stilling pool. Figure 3 shows a steel pipe crossing. The type of energy dissipator is selected for least cost to meet the requirements for the structure. The outlet must be designed to adequately control local scour. The downstream channel must be sufficiently stable to avoid erosion which might endanger the structure.

DRAIN INLETS

.4 Drain inlets are used to introduce small amounts of drainage water into the canal when an economical means of crossing the canal is not available. In general, it is more desirable and safer to carry flood and drainage water across the canal in culverts, in overchutes, or over canal siphons instead of permitting it to enter the canal, because of the operating problems, cleaning costs and other costs which are introduced by permitting storm and drainage water to enter the canal.

The maximum drainage inlet capacity at any point should not exceed 10 percent of the capacity of the canal unless evacuation facilities are provided immediately upstream from the point of storm water entry. In case large flows are taken into the canal, and a waste outlet is provided immediately upstream, some storm water will pass on down the canal because of the rise in water surface necessary to operate the waste facilities. This increased flow, plus all other water admitted to the canal during any given storm, must be evacuated before the accumulation endangers the banks or structures. The maximum allowable rise due to this increased flow is usually equal to one-half the lining freeboard or one-fourth the bank freeboard, whichever is the least. The point of maximum water surface rise will be upstream of a structure such as a siphon, and this will usually locate the wasteway. (See Paragraph 7.6.)

Water may be emptied into the canal through a rectangular concrete flume section or a closed conduit. In earth canals, a flume section usually requires a chute and pool to prevent eroding or damaging the canal, whereas a pipe can often discharge a small amount of water near the canal water surface without appreciable damage to the canal.

SPILLWAYS

.5 Spillways are provided on canals to automatically remove some water from the canal when the water surface exceeds a set elevation. The spillway capacity should be adequate to empty flood and unused irrigation water and thus prevent damage to the canal and appurtenant structures. If a spillway is used as protection for a powerplant or similar installation, it may be required to pass the entire capacity of the canal. Functionally, a spillway differs from a wasteway in that, while it may possibly discharge the entire flow, it cannot be used to empty the canal. The most common canal spillways are side-channel overflow spillways, siphon spillways, and automatic radial-gate spillways. Wasteways and spillways are often combined to provide a means of emptying the canal as well as controlling the water depth for protection. This also permits the use of a common outlet channel. Figures 4 and 5 show a combined overflow spillway and wasteway.

Overflow Spillway

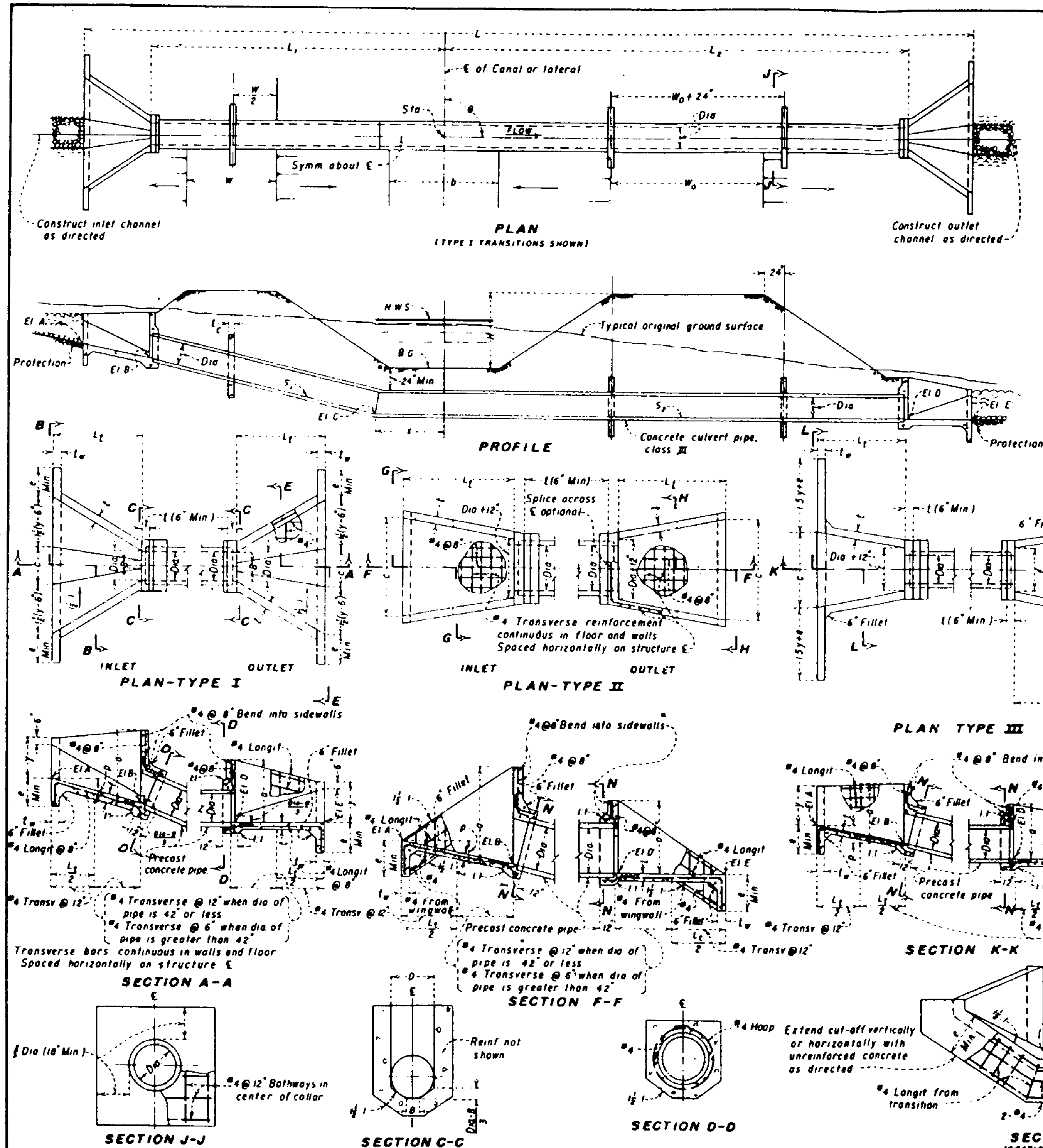
A. Overflow spillways may be used to empty weeds, trash, ice, and other floating objects, as well as excess water, from the canal. They are desirable where the exact amount of water to be emptied is not known, because a small additional head over a long overflow crest will considerably increase the discharge. However, the long overflow crest may be objectionable at some locations. Usually a wasteway gate can be combined with an overflow spillway without appreciable added expense for the structure.

7.5B

B. Spillways using radial gates which will automatically open or close with the rise or fall of the water surface in the canal are sometimes employed to advantage where an overflow or siphon spillway is not suitable. Radial-gate Spillway

C. Siphon spillways are generally constructed where overflow spillways are not feasible. The spillway capacity must be known, as additional head will not appreciably increase the capacity after the siphon has primed. Although two identical siphon spillways may not operate the same, Figure 6 presents the best available design data for this type of structure. Sharper bends or other changes may reduce the capacity considerably. The throat and upper portion of the lower leg should be designed to resist external atmospheric pressure in addition to other loads. An adjustable slotted opening should be provided at the water surface in the hood on the upper leg to obtain control of the flow through the siphon spillway for a wide range of discharges and prevent slugging when the siphon primes. See Hydraulic Laboratory Report No. Hyd-335 for model studies of several types of adjustable slotted openings. Siphon Spillways

.6 Wasteways are usually designed for manual or automatic operation. Normally, they will empty the canal, but they can be operated to remove any part of the flow. Wasteways may be combined with spillways or designed as separate structures. Automatic spillway gates may be installed to drain the canal and thus operate as a wasteway; they can be set to hold the canal water surface nearly constant as well as to discharge the canal capacity, but they are usually expensive and require considerable maintenance. Automatic radial gates may be electrically or hydraulically controlled. Manually operated slide gates in conjunction with overflow spillways are generally used for small capacity wasteways. Wasteways and spillways should be provided as required to adequately protect the canal system. Where feasible, natural drainage channels should be utilized for wasting canal water. Retention ponds and long, expensive wasteway channels have been provided in some cases on large canals to give the desired protection. See also Paragraph 7.4. WASTEWAYS



STATION ℄ WATERWAY	DIA	θ	b	H	W	W_0	TYPE OF TRANSITIONS INLET	TYPE OF TRANSITIONS OUTLET	ELEVATIONS B.G.	ELEVATIONS A	ELEVATIONS B	ELEVATIONS C	ELEVATIONS D	ELEVATIONS E	S_1	S_2	X	L_1	L_2	L	LIN. FT. PIPE	REMARKS
832+62.7	48"	90°	6'	5.0'	4'	10'	48-I-A	48-I-B	1487.39	1485.39	1483.39	1480.70	1480.45	1480.39	-.1740	-.005	5.0'	21.50	46.00	91.50	68'	(1)
917+20	48"	90°	5'	4.8'	4'	10'	48-I-A	48-I-B	1482.06	1479.50	1477.50	1475.64	1475.46	1475.40	-.1151	-.005	4.5'	20.66	31.75	76.41	52'	
1027+50.3	36"	90°	5'	4.4'	4'	10'	36-I-A	36-I-B	1471.77	1471.17	1469.67	1466.00	1462.94	1462.89	-.237	-.069	3.0'	18.50	41.50	78.00	60'	(1)

TABLE OF DIMENSIONS AND EST. QUANTITIES-TYPE I

STR. No.	DIA	a	y	e	L_t	B	c	p	t	t_w	CONC (CU. YDS.)	REINF. (LBS.)
48-I-A	48"	7'-0"	4'-6"	2'-6"	12'-0"	5"	4'-0"	24"	7"	8"	9.2	830
48-I-B	48"	6'-0"	5'-0"	2'-6"	12'-0"	15"	4'-0"	0	7"	8"	8.3	750
36-I-A	36"	4'-6"	3'-0"	2'-6"	9'-0"	11"	3'-0"	18"	6"	8"	3.5	320
36-I-B	36"	4'-6"	3'-6"	2'-6"	9'-0"	11"	3'-0"	0	6"	8"	3.7	340

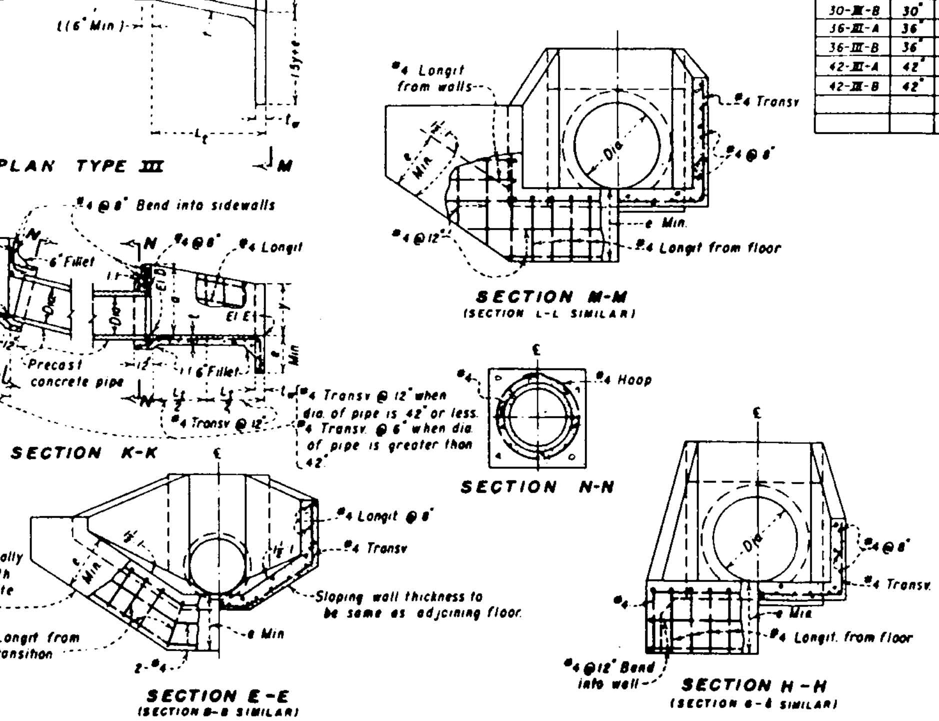
(1) Provisions have been made for a farm head ditch crossing.

TABLE OF DIMENSIONS AND EST. QUANTITIES-TYPE II

STR. No.	DIA.	a	e	L_t	c	p	t	t_w	CONC. (CU. YDS.)	REINF. (LBS.)
24-II-A	24"	5'-0"	24"	6'-0"	6'-0"	12"	5"	6"	1.7	190
24-II-B	24"	4'-0"	24"	6'-0"	6'-0"	0	5"	6"	1.5	170
30-II-A	30"	5'-11"	2'-6"	7'-0"	7'-6"	15"	6"	8"	2.6	240
30-II-B	30"	4'-8"	2'-6"	7'-0"	7'-6"	0	5"	8"	2.2	230
36-II-A	36"	6'-10"	2'-6"	8'-0"	9'-0"	18"	7"	8"	3.9	350
36-II-B	36"	5'-4"	2'-6"	8'-0"	9'-0"	0	5"	8"	2.9	300
42-II-A	42"	7'-6"	2'-6"	9'-0"	10'-6"	18"	7"	8"	4.8	460
42-II-B	42"	6'-0"	2'-6"	9'-0"	10'-6"	0	6"	8"	4.1	400

TABLE OF DIMENSIONS AND EST. QUANTITIES TYPE III

STR. No.	DIA.	a	y	e	L_t	c	p	t	t_w	CONC. (CU. YDS.)	REINF. (LBS.)
24-III-A	24"	5'-0"	2'-6"	24"	6'-0"	6'-0"	12"	5"	6"	2.5	250
24-III-B	24"	4'-0"	2'-6"	24"	6'-0"	6'-0"	0	5"	6"	2.4	240
30-III-A	30"	5'-11"	3'-0"	2'-6"	7'-6"	7'-6"	15"	6"	8"	3.8	360
30-III-B	30"	4'-8"	3'-0"	2'-6"	7'-6"	7'-6"	0	5"	8"	3.5	320
36-III-A	36"	6'-10"	3'-6"	2'-6"	9'-0"	9'-0"	18"	7"	8"	6.1	580
36-III-B	36"	5'-4"	3'-6"	2'-6"	9'-0"	9'-0"	0	5"	8"	4.5	450
42-III-A	42"	7'-6"	4'-0"	2'-6"	10'-6"	10'-6"	18"	7"	8"	7.5	680
42-III-B	42"	6'-0"	4'-0"	2'-6"	10'-6"	10'-6"	0	6"	8"	6.3	600



REFERENCE DRAWINGS

EMBEDMENT OF PRECAST PIPE IN MONOLITHIC CONCRETE 222-D-21337
BENDS FOR PRECAST CONCRETE CULVERT PIPE 222-D-21332
TYPICAL PROTECTION DETAILS 222-D-21336

NOTES

Unless otherwise shown, place reinforcement so that the clear distance between face of concrete and nearest reinforcement is $1\frac{1}{2}$ inches; except provide a clear distance from face of concrete placed against earth or rock of two inches.
Lap all bars 24 diameters at splices
Monolithic concrete design based on a compressive strength of 3,000 lbs. per sq. inch at 28 days.
Collar thickness t_c, 6" when Dia ≦ 39", 8" when Dia > 39"

3-26-64 D-3LR.	REVISED STRUCTURE ELEVATIONS

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UNITED STATES
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COLUMBIA BASIN PROJECT - WASHINGTON

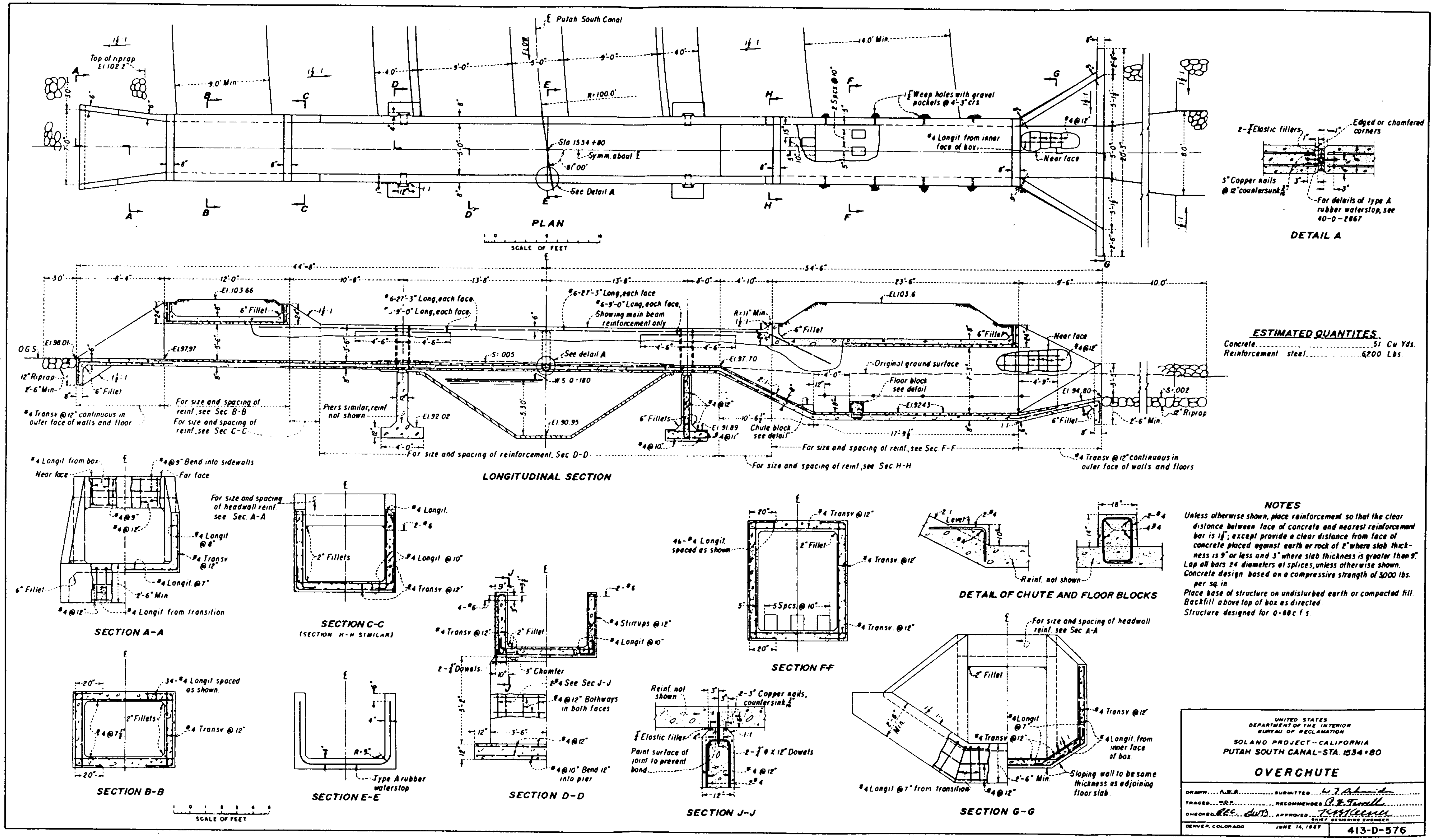
WEST CANAL LATERALS - BLOCK 81 - PART I

CONCRETE PIPE CULVERTS

DRAWN ... H.E.G. ... SUBMITTED ... W.Z. [signature]
TRACED ... L.A.C. ... RECOMMENDED ... [signature]
CHECKED ... [signature] ... APPROVED ... [signature]
CHIEF DESIGNING ENGINEER
DENVER, COLORADO. NOV. 7, 1963 222-D-21331

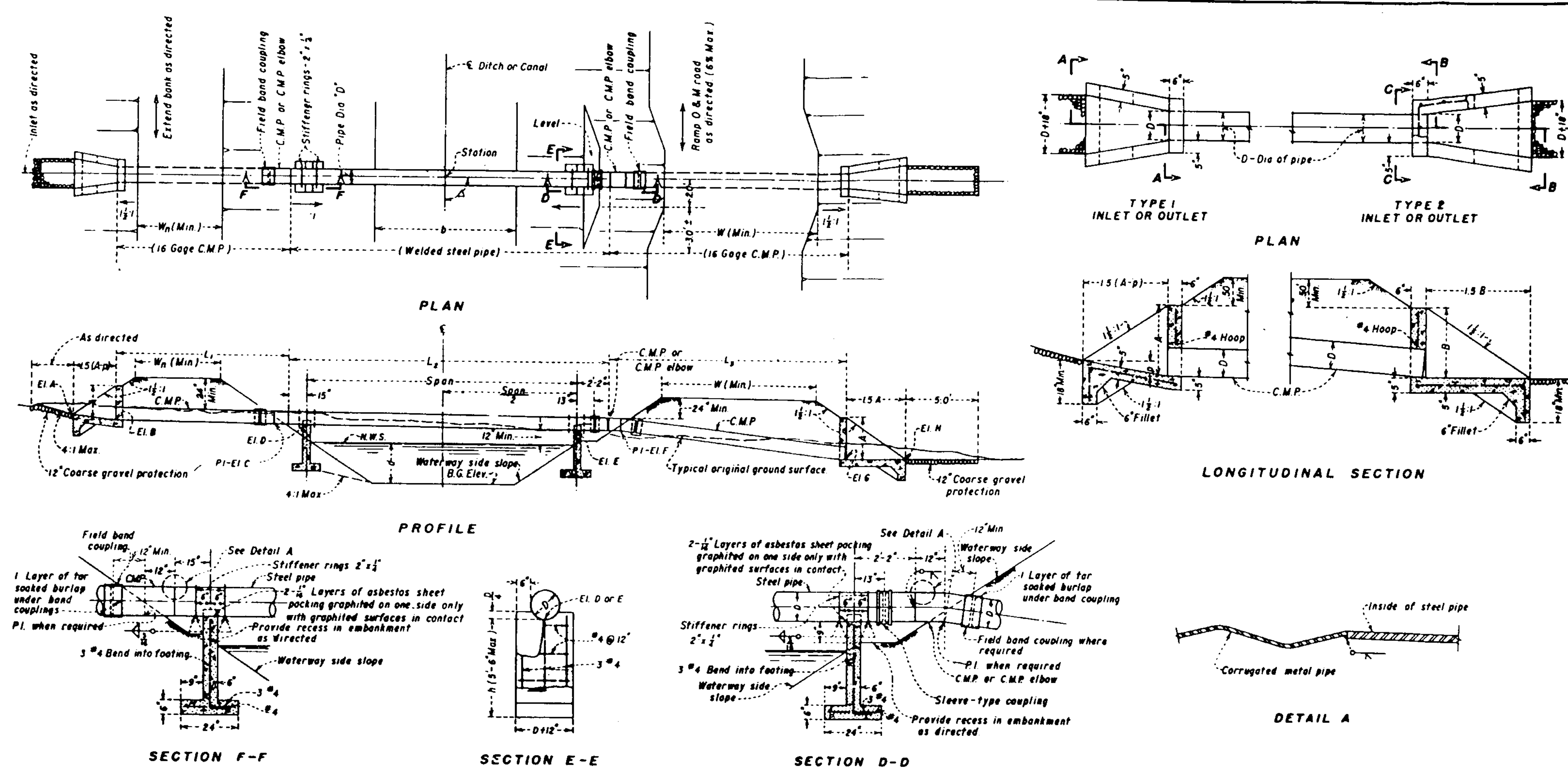
OVERCHUTE--PUTAH SOUTH CANAL, STATION 1534+80

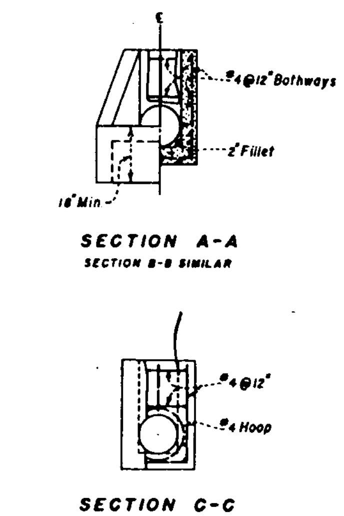
Fig. 2 Par. 7.3



IRRIGATION CROSSINGS--FLORIDA FARMERS DITCH ENLARGEMENT

Fig. 3 Par. 7.3





NOTES

All reinforcement in monolithic concrete shall be placed in center of slab.
Lap all bars 24 diameters at splices.
Backfill above top of pipe as directed, 2.0' Min.
Monolithic concrete design based on a compressive strength of 3000 lbs. per sq. in. at 28 days.

For dimensions b, d, W, Wn see Plan and Profile drawings.

STA.	PIPE DIA. "D"	ELEVATIONS B.G.	A	B	C	D	E	F	G	H	LENGTHS SPAN	L_1	L_2	L_3	h	Δ	TRANSITIONS INLET TYPE	OUTLET TYPE	TRANSITION DIMENSIONS TYPE 1 A*	P	TYPE 2 B*	ESTIMATED QUANTITIES CONC.	REINF. STEEL	16 GAGE C.M.P. 8"	12"	18"	WELDED STEEL PIPE 12 GAGE 8"	10 GAGE 12"	18"
		FLORIDA FARMER'S DITCH ENLARGEMENT																											
151+85	12"	7052.25	7059.51	7058.76	7058.68	7058.67	7058.53	7058.51	7058.40	7058.40	28'-6"	18.1'	31.92'	25.1'	4'-6"	90°	1	2	2'-6"	9"	2'-6"	1.3	125		41'			31'-11"	
157+79	12"	7052.05	7057.30	7057.30	7057.20	7057.19	7057.05	7057.03	7055.48	7055.48	28'-6"	16.1'	31.92'	22.1'	3'-6"	90°	2	2			2'-6"	1.3	125		38'			31'-11"	
161+00	12"	7049.97	7057.12	7056.37	7056.28	7056.27	7056.13	7056.11	7056.00	7056.00	28'-6"	20.1'	31.92'	22.1'	4'-6"	90°	1	2	2'-6"	9"	2'-6"	1.3	125		42'			31'-11"	
173+85	12"	7049.55	7056.72	7055.97	7055.87	7055.86	7055.72	7055.70	7055.60	7055.60	28'-6"	20.1'	31.92'	22.1'	4'-6"	90°	1	2	2'-6"	9"	2'-6"	1.3	125		42'			31'-11"	
183+70	12"	7049.22	7055.50	7055.50	7055.41	7055.40	7055.26	7055.24	7055.13	7055.13	28'-6"	18.1'	31.92'	22.1'	4'-6"	90°	2	2			2'-6"	1.4	135		40'			31'-11"	
186+75	12"	7049.12	7056.57	7055.82	7054.27	7054.26	7054.12	7054.10	7053.10	7053.10	28'-6"	19.9'	31.92'	22.1'	3'-6"	90°	1	2	2'-6"	9"	2'-6"	1.3	125		42'			31'-11"	
194+20	12"	7048.87	7054.11	7054.11	7054.03	7054.02	7053.88	7053.86	7052.87	7052.87	28'-6"	16.1'	31.92'	20.1'	3'-6"	90°	2	2			2'-6"	1.3	125		36'			31'-11"	
201+80	12"	7048.63	7054.73	7054.73	7053.78	7053.77	7053.63	7053.61	7050.32	7050.32	28'-6"	20.1'	31.92'	24.0'	3'-6"	90°	2	2			2'-6"	1.3	125		44'			31'-11"	
		FLORIDA FARMER'S DITCH RELOCATION																											
3+20	12"	7044.51	7048.88	7048.88	7048.81	7048.80	7048.71	7048.69	7047.80	7047.80	18'-0"	14.1'	21.42'	20.1'	3'-6"	90°	2	2			2'-6"	1.3	125		34'			21'-5"	
			FLORIDA CANAL ENLARGEMENT																										
19+63	12"	7027.45	7032.85	7032.85	7032.76	7032.75	7032.61	7032.59	7032.50	7032.50	26'-0"	20.1'	29.42'	16.1'	3'-6"	90°	2	2			2'-6"	1.3	125		38'			29'-5"	
** 96+70 90' LT.	12"	6941.02	6944.68	6944.68	6944.60	6944.59	6944.50	6944.48	6941.50	6941.50	18'-0"	16.1'	21.42'	21.0'	3'-6"	90°	2	2			2'-6"	1.3	125		34'			21'-5"	

** Elevations and lengths are approximate only and may be changed by the construction engineer.

* D+18" Min.

2-15-62 D.W.?	CHANGED IRRIG. X-ING. STA 204+40 TO 201+80 ADDED X-ING LT. STA. 96+70

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

FLORIDA PROJECT - COLORADO

FLORIDA FARMERS DITCH ENLARGEMENT AND FLORIDA CANAL ENLARGEMENT

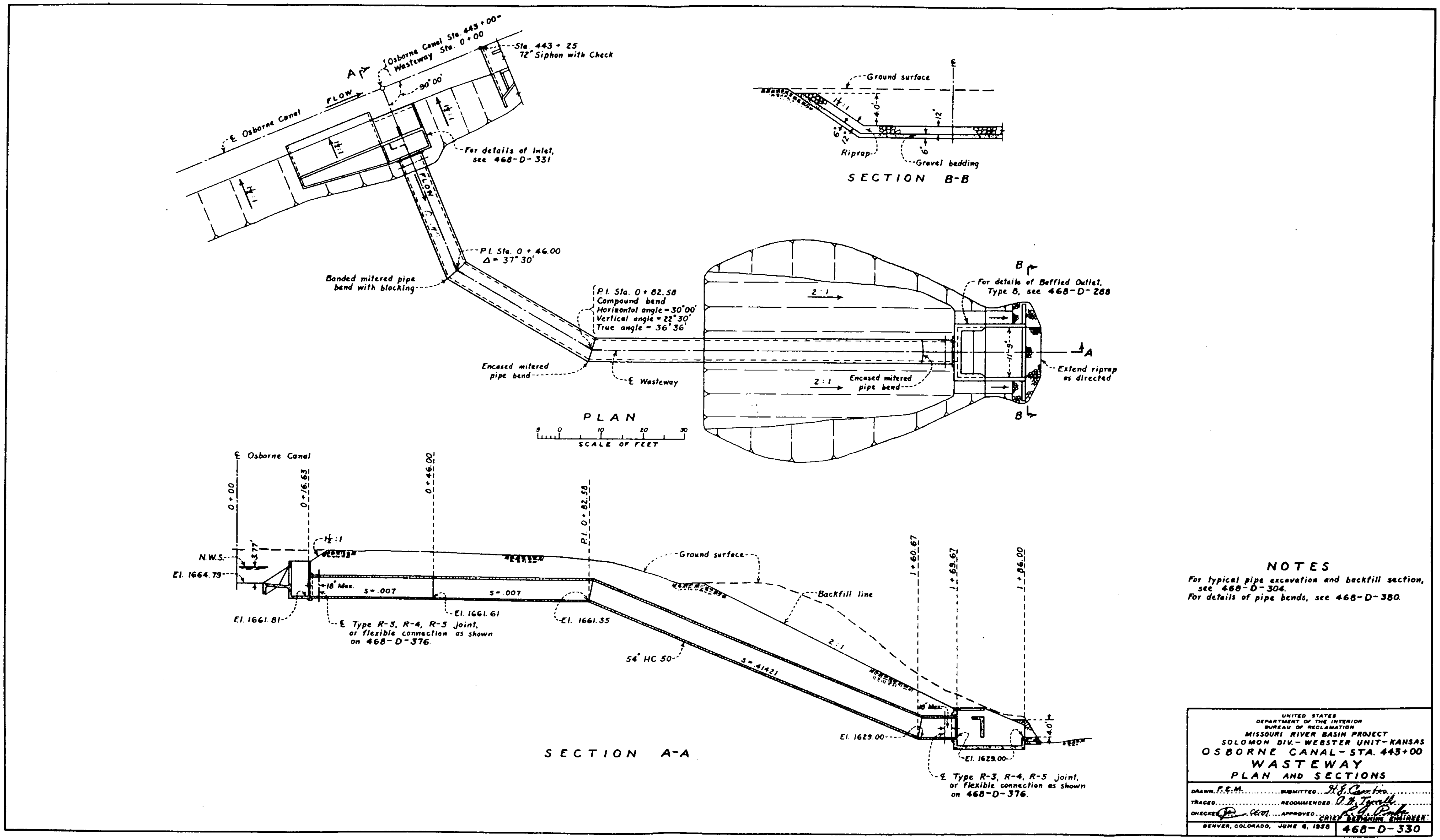
IRRIGATION CROSSINGS

DRAWN L.J.M.-R.E.C. SUBMITTED W. T. [illegible]
TRACED R.C.C. RECOMMENDED P. H. Terrell
CHECKED [illegible] APPROVED [illegible]

DENVER, COLORADO. OCT. 2, 1961 — 519-D-118

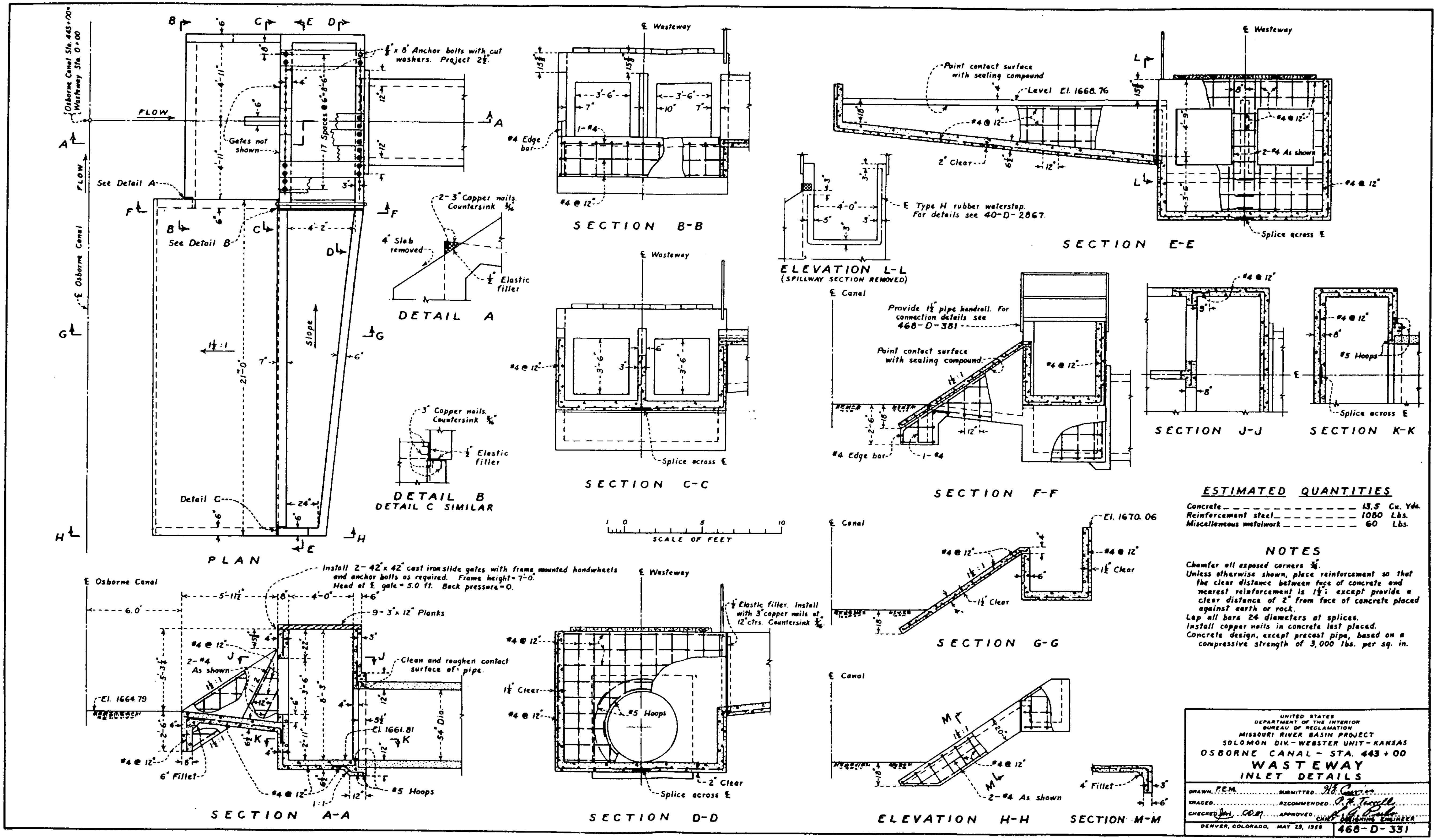
WASTEWAY PLAN AND SECTIONS--OSBORNE CANAL, STATION 443+00

Fig. 4 Par. 7.5



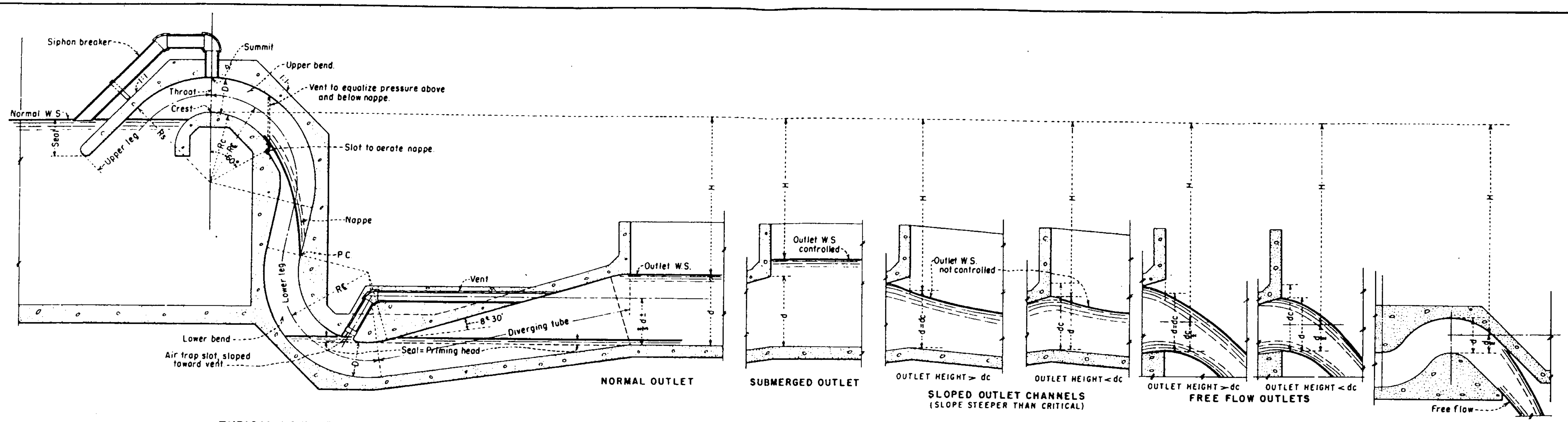
WASTEWAY INLET DETAILS--OSBORNE CANAL, STATION 443+00

Fig. 5 Par. 7.5



TYPICAL LOW-HEAD SIPHON SPILLWAY--DESIGN DATA

Fig. 6 Par. 7.5C



TYPICAL LOW-HEAD SIPHON SPILLWAY

(Maximum head = atmospheric pressure equivalent at site)

DESIGN DATA

$$q = CD\sqrt{2gH} \leqq Rc\sqrt{0.7h(2g)} \times \log_e \frac{Rs}{Rc} = \text{(Vortex flow} \sim \text{Max } q)$$

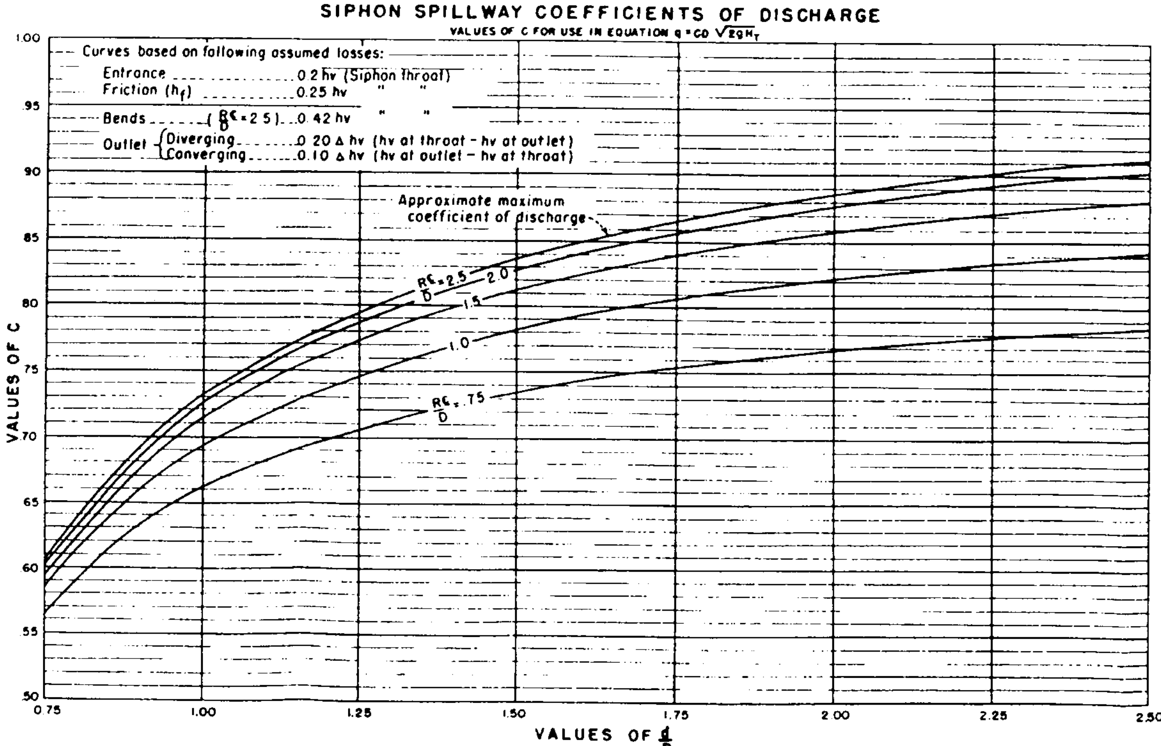
q = Cubic ft per sec per ft of crest width
Rc = Radius of crest at throat in ft.
Rs = Radius of summit at throat in ft
D = Throat size in ft.
d = Depth of water at outlet in ft
h = Atmospheric pressure at site in ft of water
H = Available head = (See outlet details.)
C = Discharge coefficient based on $\frac{d}{D}$ and $\frac{R_{\mathfrak{C}}}{D}$
g = 32.16

SECTION	DESIGN	REASONS	REMARKS
Upper leg	A well proportioned transition providing a gradually contracting area. Inlet area should be 2 or 3 times the area of the throat	Provides accelerating velocity. Reduces entrance losses.	Entrance loss for form shown suggested as 0.2 Δhv.
Throat	Size depends upon required capacity. Recommended ratio of R℄ to D is 2.5. For forming with timber D is 2'-0" min.; metal forms left in place may be used for less than 2'-0". Recommended width of throat = 2D.	Keeps bend losses low	Wide crests and shallow depths reduce priming time.
Upper bend	Recommended ratio of R℄ to D is 2.5. Aerating slot is set at 60° from crest. Pipe vent, set in sidewalls, connects slot to summit area	Keeps bend losses low. Slot prevents nappe from clinging to crest. Vent insures nappe trajectory.	Area in upper and lower bend is same as area at throat
Lower bend	PC is set to contact nappe. P.C. is a point on trajectory of nappe as determined from desired priming head at crest. Recommended ratio of R℄ to D is 2.5. Top of siphon at vertical section is set at elevation of outlet bottom. Air trap elevation is elevation of outlet bottom plus desired priming head. Vent area = throat area ÷ 48 (approx.)	Provides seal and exhausts air from upper bend. Provides seal of lower bend and insures creation of partial vacuum. Vent exhausts entrained air with negligible loss of discharge.	Vent decreases priming time, increases priming head slightly
Diverging tube	Outlet end set at desired elevation. Length of tube is set by 8°30' angle of divergence. Vent outlet set at ⅓ depth (±) of outlet at max. capacity.	Provides transition. Increases discharge.	Discharge loss for form shown suggested as 0.2 Δhv.
Siphon breaker	A pipe vent set at summit with inlet end set at or slightly below normal water surface. Area = throat area ÷ 24 (Min.)	To break siphonic action slowly; prevents severe vibration.	If practicable, adjustable inlet on siphon breaker is desirable.
Seal at intake	Lip of upper leg is submerged an arbitrary distance, but should be set deep enough to prevent excessive draw down.	Prevents "gulping" of air, and provides more normal action of siphon breaker.	

Note: Design recommendations given above are presumed to give minimum losses according to experiments. Deviation therefrom should include consideration of additional losses.

SIPHON SPILLWAY COEFFICIENTS OF DISCHARGE

VALUES OF C FOR USE IN EQUATION $q = CD\sqrt{2gH_T}$



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"A Study of Low-Head Siphons" by T.G. Owen - 1939.
U.S.B.R. Data on Model Tests, Wasteway No. 2 (Yakima Project) 1942.
A.S.C.E. Transactions, - 1934, Page 986.
A.S.C.E. Transactions, 1939, Page 1789.
"Siphon Spillways" by A.H. Naylor - 1935.

RECOMMENDED DESIGN PROCEDURE

1. Determine crest Elevation. (Canal W.S. EL. + Freeboard desired)
2. Determine head available (crest EL. - W.S. EL. at outlet.)
3. Select preliminary values for D and R℄ and solve for max. q using Vortex flow formula: $q = Rc\sqrt{0.7h(2g)}\,\log_e \frac{Rs}{Rc}$ (Recommend D = 2'-0" Min. and $\frac{R_{\mathfrak{C}}}{D} = 2.5$)
4. Select values of d and H and obtain value of C from graph. Solve equation $q = CD\sqrt{2gH}$. (q must not be greater than value obtained in (3)).
5. Solve for required crest width = $\frac{Q}{q}$. Select crest width and number of siphons to be used for total Q and determine q for this crest width.
6. Repeat (3) and (4) until $q = CD\sqrt{2gH} \cong Rc\sqrt{0.7h(2g)}\,\log_e \frac{Rs}{Rc}$.
7. Select priming head by determining the canal W.S. EL. at which operation is desired. If priming head is less than $\frac{D}{3}$ locate P.C. of lower bend on trajectory of nappe.
8. Determine size and inlet EL. of siphon breaker. (Area of siphon breaker = Area of siphon throat ÷ 24).
9. Select and design priming aids to be used (Slot to aerate nappe, Vent to equalize pressure above and below nappe, and vent at lower bend).

THIS DRAWING SUPERSEDES DWG. 40-D-3363

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
BRANCH OF DESIGN AND CONSTRUCTION
DESIGN DATA
TYPICAL LOW-HEAD SIPHON SPILLWAY

DRAWN: M.M.S. SUBMITTED: A.B. Reeves
TRACED: M.M.S. RECOMMENDED: H.G. Curtis
CHECKED: [illegible] APPROVED: [illegible]
DENVER, COLORADO - JUNE 15, 1949
103-D-46

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6	Typical Deliveries From 4-Inch Outlets	8.4	103-D-1067

8.1 INTRODUCTION

.1 In pipe distribution systems, buried closed conduits are used for conveyance of irrigation water to the delivery point on the farmer's land. Normally, pipelines are placed adjacent to roads and may be installed both up and down slope if the pipe can be kept below the hydraulic gradient. There is no necessity to contour as in the case of a canal. Hydraulic head sufficient to serve farm deliveries, including friction loss in farm laterals, is carried in the pipeline as pressure head. Each delivery is usually provided with a meter for measuring the rate of flow and for totalizing the amount of water delivered. Design criteria for individual open (or limited pressure) and closed (or full pressure) systems have been developed. Either may be adapted to use for pump or gravity service areas.

The closed (or full pressure) system may be one of two types. One type gives the farmer only a few feet of head so he can put in a pressure pipe system or an open ditch system to distribute water on his land. The other type assures the farmer sufficient head (78 feet or more) to distribute water on the land by sprinklers.

General structural and hydraulic design criteria relating to pipe distribution systems are given in Chapter 2. In the structural drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated.

OPEN OR LIMITED PRESSURE SYSTEM

.2 The open or limited pressure system utilizes low-head pipe with open baffle or gate-controlled stands located at irregular intervals along the pipeline (see Figure 1). Equal spacing of stands may cause serious line surges at partial capacity flow. The baffle serves the same purpose as a check and its top elevation should be set so that static water level will provide sufficient head to all farm deliveries between the stand and the next upstream baffle when the reach is operating at zero flow beyond the baffle. The top of the stand should be set to provide minimum freeboard above water surface for maximum design capacity flowing over the baffle as a weir. The weir may operate either submerged or free fall; however, the latter condition may result in considerable air entrainment and thus require air vents from the pipeline. All pipe is designed to withstand the head measured from the hydraulic gradient (for maximum demand) to the centerline of the pipeline, and is sized to provide full water supply to the farm units. To limit pressure on delivery valves, to permit the use of low-head pipe, and to make the stands more accessible and less costly, the baffle height is usually limited to less than 20 feet. This type system should only be used when a closed or full pressure system is not practical.

Operational Control in Open Pipe System

A. Operational control for an open pipe system must be from the head of each lateral, branch line, and farm delivery. Care must be exercised, in making changes in demand on the system, to prevent overtopping of stands or waste through overflows installed in the stands. Meters and throttling valves should be placed at the heads of all laterals, sublaterals, and farm deliveries, to enable the operators to perform the necessary regulation. Pipe subject to freezing must be drained during the nonoperating season. Wasteways are usually required at or near the end of each line.

CLOSED OR FULL PRESSURE SYSTEM

.3 The closed or full pressure system uses medium- to high-head pipe, depending on the topography and the farm delivery head required. Normally, closed systems operate from the water surface elevation occurring at the intake of the distribution network or from a head created by pumping. All pipe is designed to withstand pressures measured from the static head at the intake plus adequate allowances for water hammer that results from valve closures. Where topography is steeply sloping, it may be economical to install pressure-reducing valves or a water surface controlled open tank in the main lines to

8.3A

CLOSED OR FULL PRESSURE SYSTEM (Cont'd.)

drop the hydraulic gradients. If pressure regulators are used, the downhill system must have some type of protection such as a relief valve to protect the lower system from regulator valve malfunction. An overflow must be provided from the open tank to bypass the flow into a natural watercourse or into the discharge line downstream in the event the tank controls fail to operate properly. Failure to provide this protection may result in bursting of the pipe. Pipelines are sized to provide for the total governing farm deliveries during periods of maximum demand.

Operational Requirements

A. Operational control for a closed system is exercised at the individaul farm delivery, where flow is adjusted by means of a throttling valve. Since the drop in head across this regulating valve may be appreciable when a delivery is operated during periods of minimum system demand, cavitation will result below the valve unless some type of protection is provided. Figures 2 and 3 show installations that have proved successful for differential heads up to 100 feet across the valve. Valve adjustments on farm deliveries should be made slowly so that excessive pressure surges in the lateral pipe system will be avoided. In general, the closed system is easier to operate and requires fewer operators than the open system. For deliveries or sublaterals in a pipe system providing sprinkler pressures, pressure reducing valves may be installed where the line pressure at the delivery exceeds 100 pounds per square inch under maximum operating heads.

FIELD DATA REQUIRED FOR DESIGN

.4 Field data required for the design of a pipe distribution system are much the same as for an open lateral system. Other design considerations are given in the following paragraphs.

Headgate Diversion Requirements

A. System headgate diversion requirements should be computed as for an open canal and lateral system, except that:

(1) Water transmission losses are small and are negligible compared to those in an open canal.

(2) In a given area, net acreages to be irrigated by a pipe distribution system will be greater than those for an open lateral system by the portion of the farm ownerships normally deducted for canal and lateral rights-of-way.

Farm Delivery Capacities

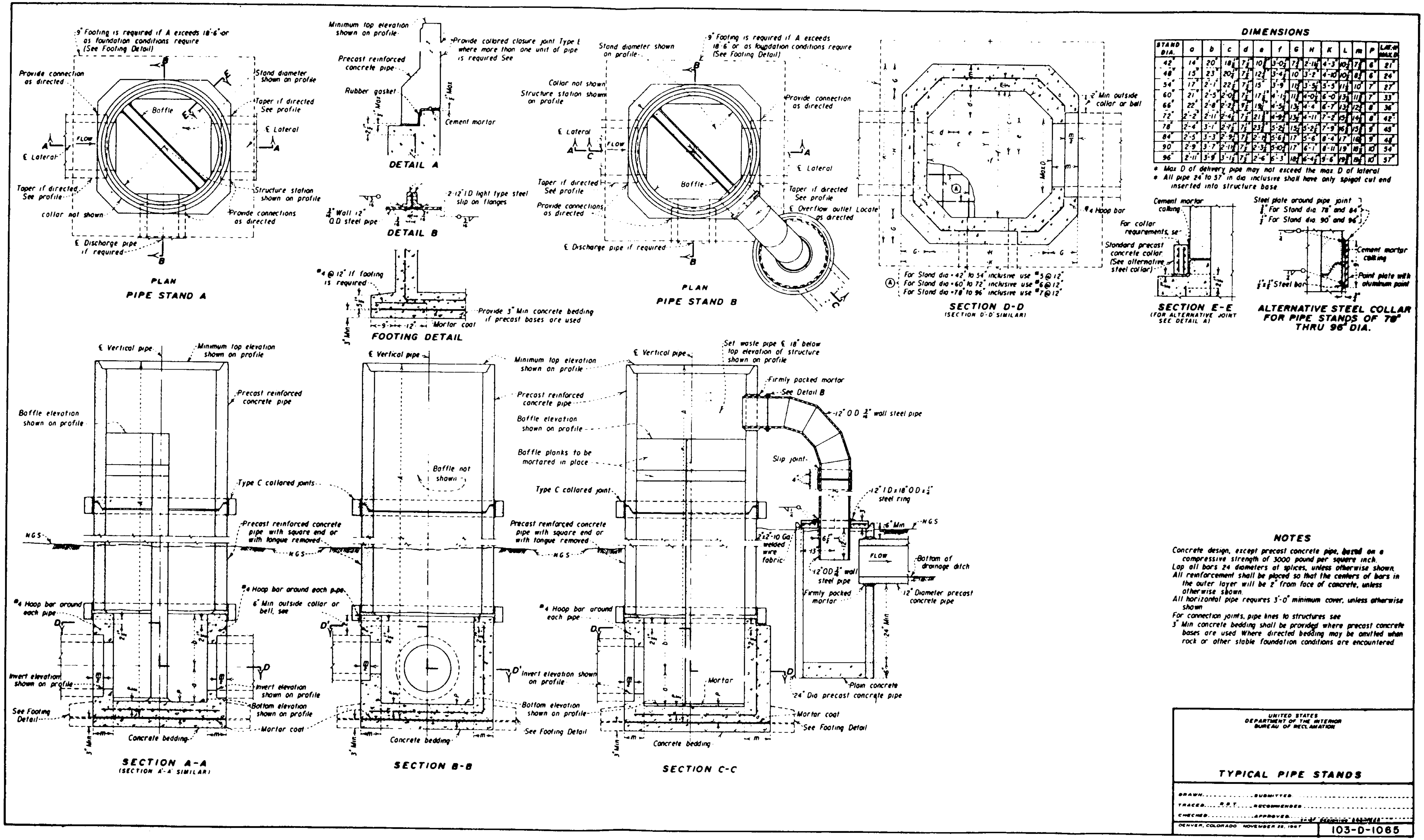
B. Farm delivery capacity requirement of a pipe system is computed the same as for a canal system because each must furnish the turnout the required amount of water during the period of maximum system demand.

(1) The minimum theoretical pipeline capacity must deliver farm headgate requirements with continuous 24-hour operation of the turnout throughout the maximum demand period. In order to reduce farm labor costs, it is customary (where the soil infiltration rates permit) to select the farm delivery capacity so as to deliver full requirements to several farmers on a turn or "rotation" basis. Criteria developed on this basis usually employ a "sliding" time requirement scale so that small acreages require less time per irrigation than larger holdings. Details of the farm delivery rotation plan should be worked out in cooperation with irrigation district officials or with an irrigation agronomist.

(2) The construction cost increases more rapidly for pipe systems than for open laterals when the capacity increases; therefore, the rotation plan for deliveries is used more extensively than demand irrigation for pipe distribution systems.

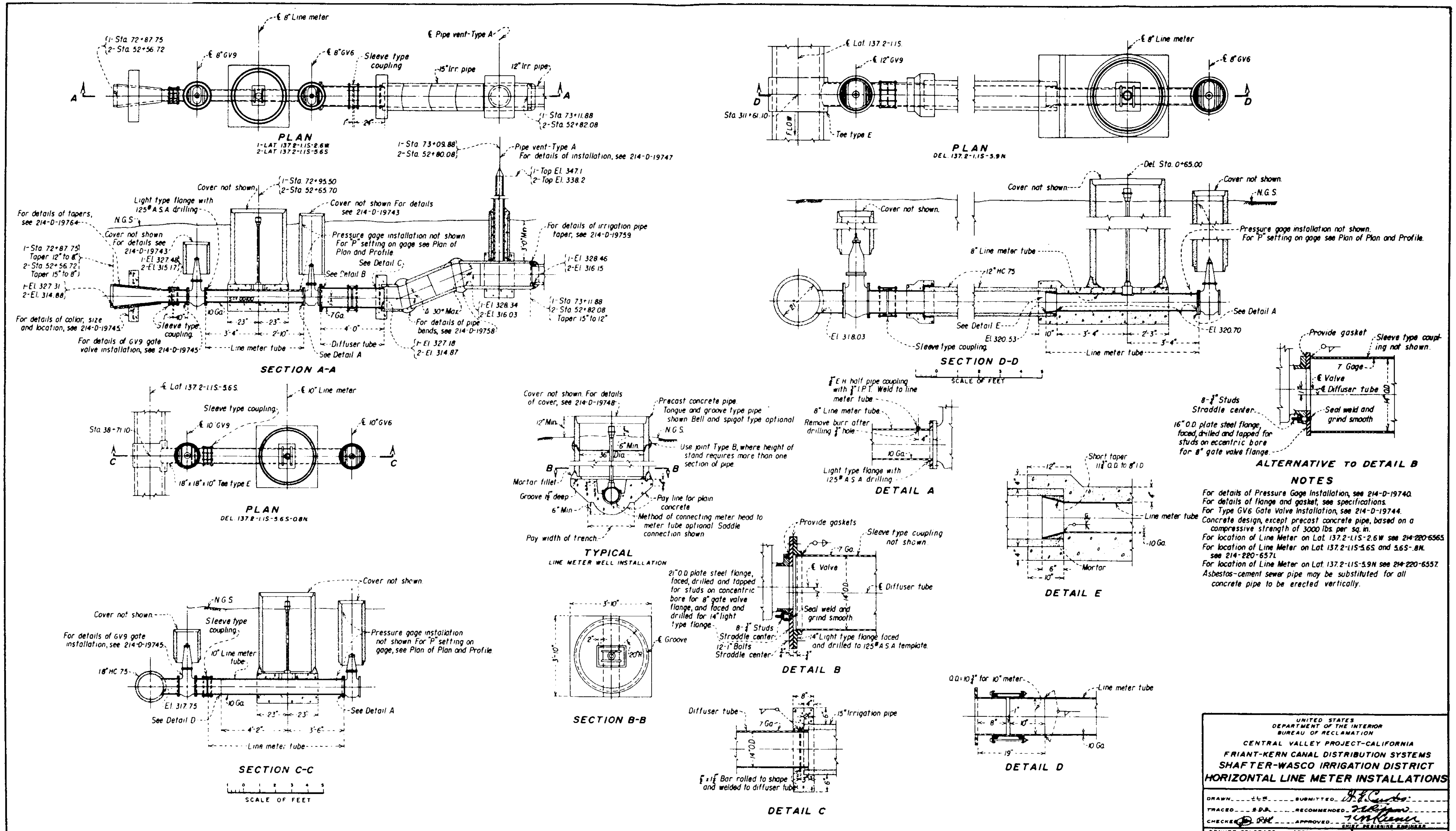
TYPICAL PIPE STANDS

Fig. 1 Par. 8.2



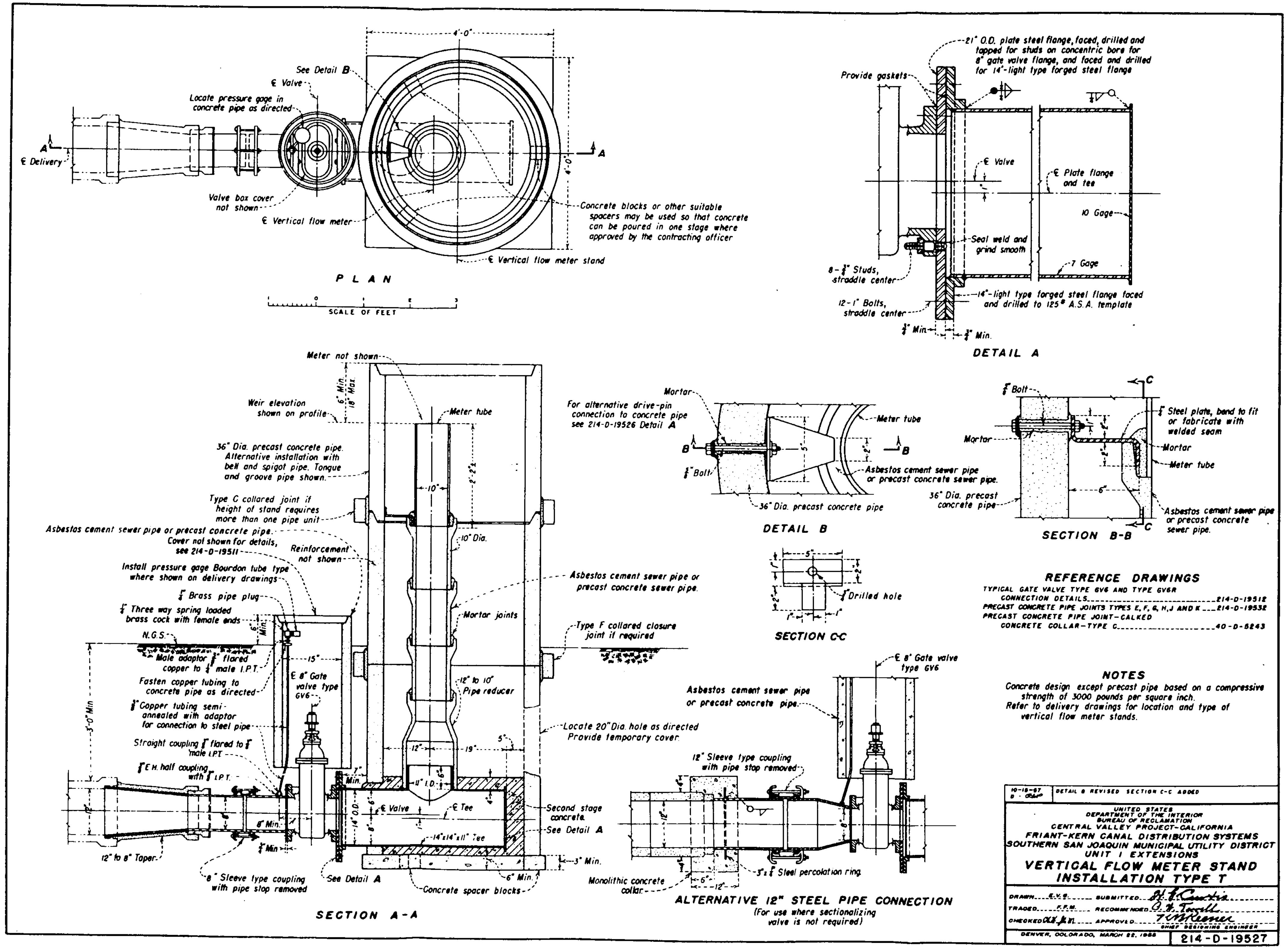
HORIZONTAL LINE METER INSTALLATIONS--SHAFTER-WASCO IRRIGATION DISTRICT--
FRIANT-KERN CANAL DISTRIBUTION SYSTEMS

Fig. 2 Par. 8.3



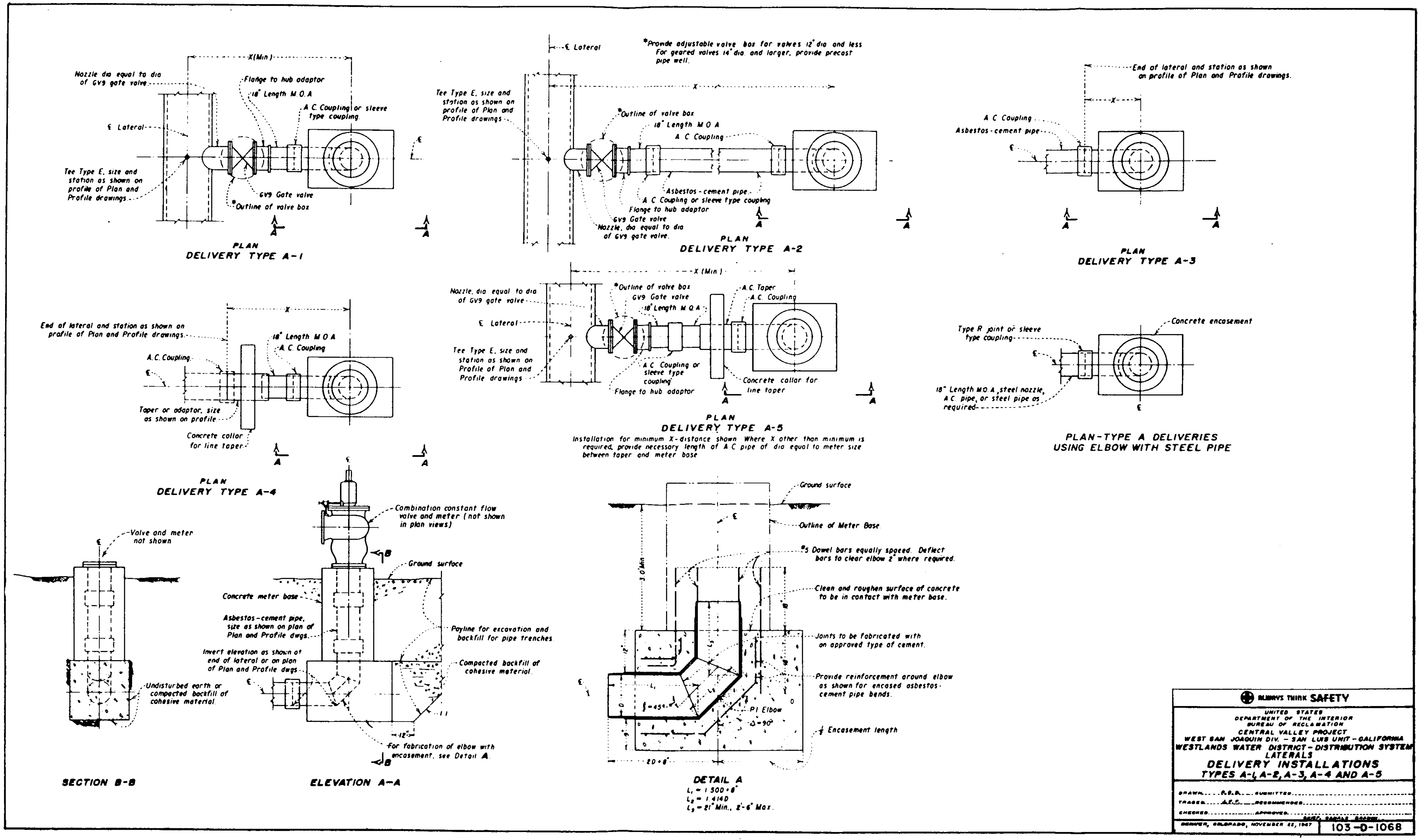
VERTICAL FLOW METER STAND, INSTALLATION TYPE T--UNIT 1 EXTENSION,
SOUTHERN SAN JOAQUIN MUNICIPAL UTILITY DISTRICT--
FRIANT-KERN CANAL DISTRIBUTION SYSTEMS

Fig. 3 Par. 8.3



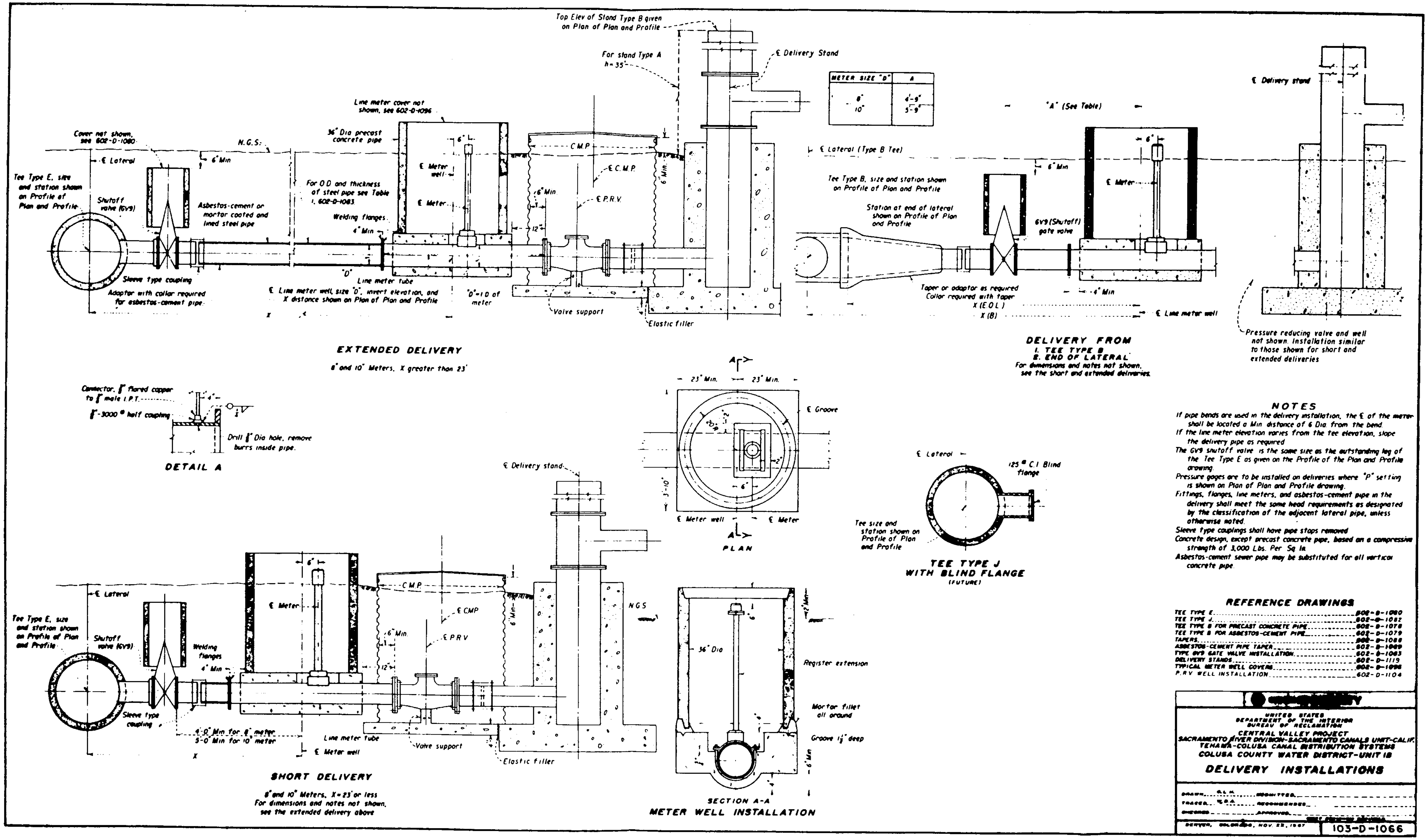
DELIVERY INSTALLATIONS, TYPES A-1, A-2, A-3, A-4, AND A-5--WESTLANDS WATER DISTRICT DISTRIBUTION SYSTEM

Fig. 4 Par. 8.4



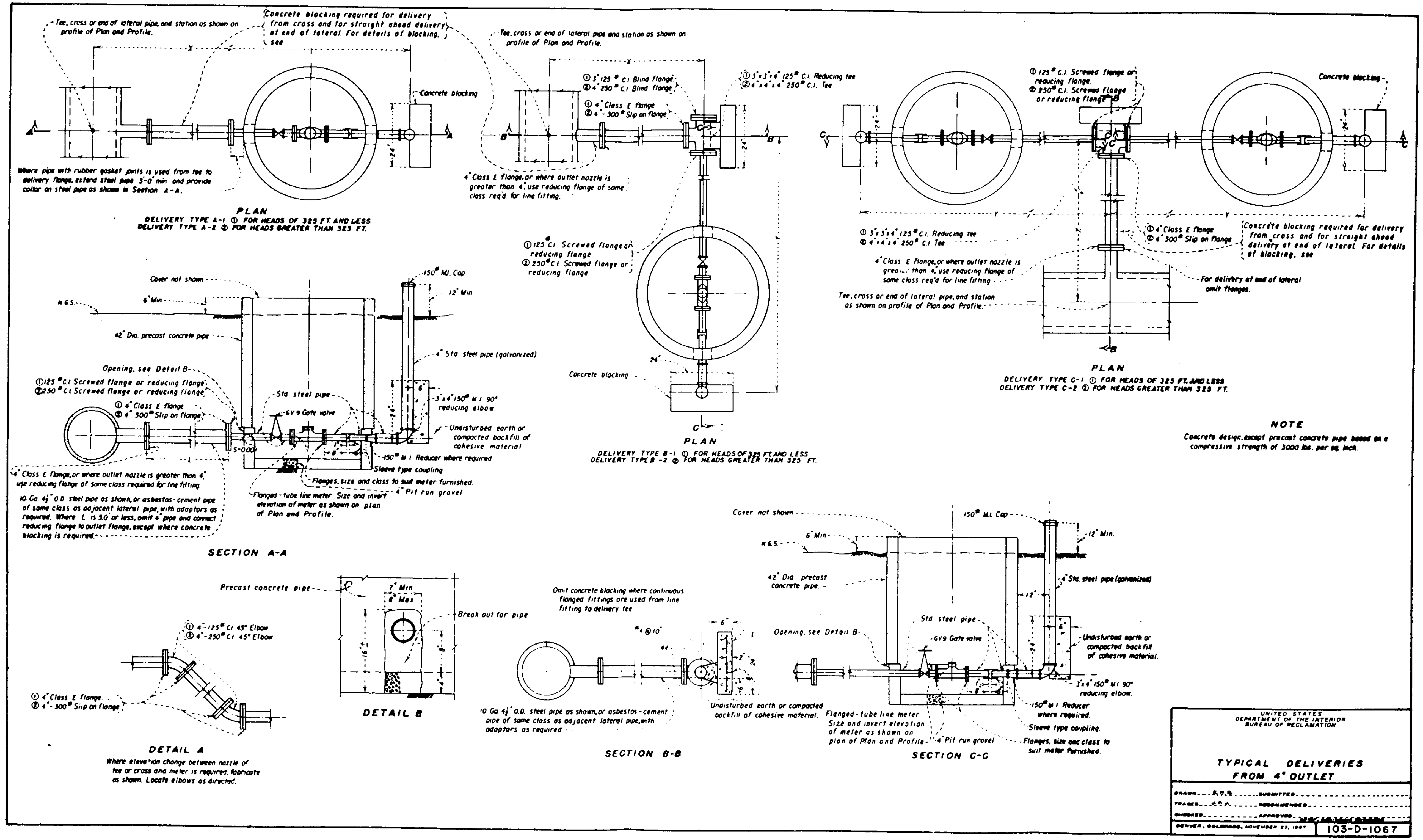
DELIVERY INSTALLATIONS--COLUSA COUNTY WATER DISTRICT, UNIT 1B

Fig. 5 Par. 8.4



TYPICAL DELIVERIES FROM 4-INCH OUTLETS

Fig. 6 Par. 8.4



8.4C

C. Unless modified by an overall plan of operation adopted by the irrigation district, farm delivery heads required for a pipe distribution system should be as follows: Farm Delivery Heads

(1) Where a farm unit is served by an open farm ditch, locate the delivery at the high point of the land as for a delivery from a canal system. Provide a minimum net head (after delivery losses have been deducted) sufficient to permit effective flow in the farm ditch. Frequently, minimum net head is assumed to be from 1 to 2 feet above ground surface at the point of delivery.

(2) Where the farmer's delivery system is to be pipe with gravity outlets, the minimum net delivery water surface elevation should be computed to provide not less than 2 feet above highest ground surface elevation in the area served plus an appropriate head allowance to compensate for the friction loss in the farmer's pipelines. A minimum of 5 feet above the adjacent ground surface is frequently provided.

(3) Where the farmers intend to utilize sprinkler irrigation, delivery pressures ranging from 35 to 100 pounds per square inch are generally desired. These pressures may be provided at the farm turnout either by gravity or by a pumping plant in the system. However, when the desired sprinkler pressures are not provided at the farm turnout, the farmers will have to provide them with their own pumping plants. Open-pit, wet-well-type pumping plants are recommended. Direct connection of the farmers' pumping plants to the lateral pipelines is not desired as it would require the entire lateral system to be designed for the water-hammer heads imposed.

Figure 4, is an example of a delivery that requires the farmer to install an open structure downstream of the combination constant-flow valve and meter, if he wants to use a booster pump. Figure 5 is an example of a delivery that provides a limited amount of head (height of delivery stand) to the farmer. If the farmer wants additional head, he can add a booster pump downstream of the stand. The pressure-reducing valve (P. R. V.) before the stand provides constant head and prevents the water from overflowing the top of the stand. Figure 6 is an example of a delivery that provides full sprinkler pressure to the farmer.

.5 The type of pipe used in distribution systems depends on the head, cover, and local conditions. TYPE OF PIPE

A. Unreinforced concrete pipe with rubber-gasketed joints may be used in diameters up to 30 inches for heads up to 20 feet. Unreinforced Concrete Irrigation Pipe

B. Except where the low-head unreinforced concrete pipe previously mentioned is suitable, reinforced concrete pressure pipe with rubber-gasket joints is normally used for heads up to 125 feet. Standard designs are available for pipe 12 to 108 inches in diameter with pressure heads from 25 to 125 feet. Special designs are usually prepared for concrete pressure pipe larger than 108 inches in diameter. Concrete Pressure Pipe

C. Prestressed pipe, pretensioned pipe, steel pipe, or specially designed concrete cylinder pipe is normally used when heads exceed the maximum allowed for concrete pressure pipe. (Note: Prestressed pipe is concrete pipe with spirally wound reinforcement prestressed to produce an initial compressive stress in the concrete; and pretensioned pipe consists of a Other Concrete and Steel Pipe

8.5D

Other Concrete and Steel Pipe (Cont'd.)

steel cylinder wrapped with reinforcement to provide an initial compressive stress in the cylinder, with a concrete lining and an external concrete coating.)

Asbestos-cement Pipe

D. Asbestos-cement pipe is used optionally in sizes 36-inch diameter and smaller, and it may be competitive with ordinary reinforced concrete pipe, pretensioned pipe, prestressed pipe, and steel pipe.

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GENERAL CONSIDERATIONS

.1 In general, county and highway (State or Interstate) bridges for canal, reservoir, and lateral crossings conform to the current issue of the "Specifications for Highway Bridges," published by the American Association of State Highway Officials. Loadings for bridges for farm and operating roads are adapted from those specifications. Railroad bridges conform to the current issues of the "Specifications for Concrete and Reinforced Concrete Bridges and Other Structures" and "Specifications for Steel Railway Bridges," respectively, of the American Railway Engineering Association. In the structure drawings selected for illustration, there may be instances in which current design practices differ in some respects from those illustrated. SPECIFICATIONS

.2 For Bureau work, seven classes of bridges are recognized. They are temporary bridges, farm bridges, operating bridges, access bridges, county bridges, highway bridges, and railroad bridges. Operating bridges are maintained by the Bureau. Other bridges are, in general, maintained by the proper agencies or companies. CLASSES OF BRIDGES

.3 Operating and farm bridges usually cross canals and laterals at right angles and have the top of the bridge floor as low as practicable or at the level of the canal bank. County, highway, and railroad bridges are placed on existing alinements and grades when possible. Reduction of skews of bridges by realinement of roads, highways, and railroads is generally impractical, but it can often be accomplished when required by adjusting the alinement of the canal. ALINEMENT AND GRADES

.4 The width of roadway for the various classes of bridges is the distance between bottoms of curbs or the slab width when no curbs are used. WIDTH OF ROADWAYS AND LOADINGS

A. The width and loading for a temporary bridge are generally specified to meet particular requirements. Temporary Bridges

B. For farm bridges, the minimum width and load are 16 feet and H15 loading, respectively; but heavier design loads should be used where unusually heavy loads are anticipated. Wider roadways and low railings are sometimes required for farm equipment such as combines. Thirty-two-foot roadway widths with standard railing will allow passage of combines, though in special cases roadway widths of 50 feet have been provided. Designs based on an H15 loading are generally sufficient for various types of farm equipment. Farm Bridges

C. For operating bridges, a roadway width of not less than 12 feet should be used for one-way traffic and 20 feet for two-way traffic, and wider widths where required for passage of large equipment. The minimum design loading is H15 and the heaviest HS20, the latter being the heaviest loading on primary highways. Heavy equipment loading diagrams are given in Figure 1. Operating Bridges

D. For county and highway bridges, the width of roadways and loadings vary with the importance of the roads; and while there is considerable difference in the requirements specified by local authorities in various parts of the country, the widths are as specified by local authorities or are the minimum in accordance with the AASHO Specifications. County and Highway Bridges

E. Access bridge widths and loading vary with the requirements for passage of large equipment. Usually the loading requirements are special for the structure. Some examples of loading diagrams are shown on Figures 1 and 16. Access Bridges

9.4F

GENERAL CONSIDERATIONS--Continued

Railroad Bridges

F. Railroad bridges may have single or double tracks as determined by right-of-way requirements. Loadings depend on the nature of the equipment used by the railroads, with an E loading in accordance with that used by the railroad.

GENERAL DESIGN CONSIDERATIONS

.5 The number of spans for a bridge should be determined by comparative estimates of costs or by comparison with structures of similar type previously constructed. For channels with high velocity of water (15 feet per second or more), such as chutes and spillways, or for canals operated in winter where ice must be considered, bridges spanning from bank to bank without piers in the channel section should be used. Otherwise piers or pile bents may be used within the canal section. Generally, bridges over canals and laterals have spread foundations. Pile foundations should be used where the soil is not adequate to support a spread footing and in wasteways or natural drainage channels where erosion may be expected, or where the ground-water table would make construction of spread foundations expensive.

TYPES OF BRIDGES

.6 The seven classes of bridges are constructed of timber, reinforced concrete, structural steel, or a combination of these materials. Temporary bridges are generally constructed of untreated timber. Operating and farm bridges are constructed of reinforced concrete on concrete foundations. County bridges and State highway bridges are constructed of reinforced concrete, structural steel, or a combination of these materials. Railroad bridges are generally reinforced concrete and steel structures. Since design criteria for bridges constructed of similar materials are very similar, it is convenient to group the various classes of bridges into three types: timber bridges; concrete and steel highway bridges; and concrete and steel railroad bridges.

TEMPORARY TIMBER BRIDGES

GENERAL

.7 Temporary bridges will generally be maintained by the Bureau. Untreated timber bridges with plank floors may be used in lieu of bridges with laminated floors. For temporary bridges with multiple spans, intermediate supports should be timber bents or timber pile bents. Temporary timber bridges should be designed in accordance with the general design requirements shown in Figure 2. Standard bridge designs are available for standard H5 through H20 highway loadings. In general, single spans should be used up to 34 feet. If a bridge is longer than this, multiple spans should be used. Expansion joints are not required for timber bridges.

MATERIAL

.8 Lumber is to be Douglas Fir, coast region; Southern Yellow Pine; or Longleaf Yellow Pine. Floor planks, laminated floor, caps, and stringers are "dense construction" if Douglas Fir, "prime structural" if Longleaf Yellow Pine, and "dense structural" if Southern Yellow Pine. All other timbers are "construction" for Douglas Fir; and "No. 1 structural" and "dense No. 1" for Longleaf Yellow Pine and Southern Yellow Pine, respectively. Native lumber of structural quality is sometime used for railing, curbs, and bridging.

Timbers are surfaced as follows:

Floor plank, floor laminations, curbs, and headers	S4S
Stringers and caps	S2E
Sills, caps, and scuppers	S2S
Railing, railing posts, and clearance markers	S4S
All other lumber	Rough

TEMPORARY TIMBER BRIDGES--Continued

Even where untreated timber is used in temporary bridges, bulkheads, cleats, and sills in contact with earth may be treated for longer life if warranted. In regions infested by termites, contact surfaces and ends of all untreated lumber should be coated with wood preservative. MATERIAL (Cont'd.)

.9 Metal railings 2 feet 8 inches high are to be provided on the temporary timber bridges with surfaced laminated floors. Bridges with plank floor and untreated timber usually have timber railings 3 feet 6 inches high, but may have a low timber railing. RAILINGS

CONCRETE AND STEEL HIGHWAY BRIDGES

.10 Concrete and steel highway bridges must meet requirements of the agency for which the bridge was designed. These requirements vary considerably for different agencies such as State Highway and County Highway Departments, but in general follow the AASHO Standard Specifications for Highway Bridges. Owing to the variations it has not been practicable to establish standard designs. GENERAL

.11 Experience has shown that certain types of bridges tend to be most suitable or economical for certain span lengths, but the engineer may prepare economic studies, including the erection requirements, in the final selection of the type to be constructed. In general, for bridges of more than two spans, continuous-type construction offers the most economy, through development of negative moment stresses and permitting fewer required bearings and expansion joints--a desirable feature from the standpoint of maintenance. SELECTION OF TYPE OF BRIDGE

It is essential for the safety of continuous-span bridges that they be founded on good foundation material such that the abutments and piers will be free of settlement. However, it is not essential that rock foundation be available, as spread footings with low bearing pressures may also be satisfactory. Generally, clay foundations that are dry in the original state and will become wet after a canal is built are not acceptable for continuous structures.

Continuous-type multiple-span bridges may be built without expansion joints by expanding against the earth as shown in Figure 3. If overall length requires expansion joints for movement, fixed bearings and expansion bearings with metal expansion joints in roadway slabs at points of movement are provided. Multiple-span bridges may also be a series of simple spans with sponge-rubber filler joints between spans.

In general, the types of bridge construction are as follows:

A. Concrete slab spans are generally constructed as simple spans of about 40 feet maximum length with a maximum skew of the crossing not exceeding 15°. Expansion joints are not required for a one- or two-span crossing. For the two-span bridge, the positive steel extends through the joint with sponge-rubber filler material extending into the slab. See detail A, Figure 4. Spans are cambered by adjusting forms for dead load deflections. A typical two-span slab bridge is shown in Figure 5. Simple Concrete Slab Spans

B. Haunched concrete slab spans are constructed for bridges of three or more continuous spans only where the foundation will be free of settlement. Maximum span length for this type of structure is approximately 60 feet. Maximum skew of the crossing should not exceed 15°. A typical three-span haunched concrete slab bridge is shown in Figure 6. Continuous Haunched Concrete Slab Spans

9.11C

CONCRETE AND STEEL HIGHWAY BRIDGES--Continued

Precast Prestressed Slab Spans With Voids

C. Precast prestressed slabs with voids are used only for simple span construction. Lengths of precast units are limited by handling weight, resulting in maximum spans of approximately 50 feet. Bridges may be single or multiple span, with sponge-rubber filler provided at piers as illustrated in Figures 7 and 8.

Cast-in-place Concrete Beam Spans

D. Cast-in-place monolithic concrete beam and roadway slab spans are used as single- or multiple-span bridges with span lengths of approximately 40 to 70 feet. Figure 9 illustrates the use of two single spans in a bridge with elastomeric bearing pads and sponge-rubber joint between the spans. For economy, two-span bridges are designed as simple spans, while three or more spans may be designed as continuous if foundation material permits. The degree of skew of the crossing is not limited in this type of construction. Continuous structures may or may not provide for expansion as illustrated by Figures 3 and 10.

Rolled Steel Beam Spans

E. Rolled steel beam spans with concrete roadway slabs are used for simple or continuous multispan bridges. Economical designs may be obtained by use of welded cover plates to beams and using composite design. Figure 11 illustrates a steel beam composite design structure. On multispan bridges expansion is provided at one or more locations, depending on the length of the structure, by rocker or roller bearings and metal roadway expansion joints. If camber exceeds the tolerances provided in the specifications, the beams are cambered an amount equal to the calculated deflection due to the total dead load. A typical multispan steel beam bridge is illustrated in Figure 12.

Prestressed or Post-tensioned Beams

F. Prestressed concrete beams with concrete roadway slabs are used for spans from 30 to 100 feet. Four standard types of beams are provided by the Joint Committee AASHO on Bridges and Structures and the Prestressed Concrete Institute. These beams are generally only used for simple span construction, and in multispan bridges sponge-rubber filler is provided between spans. Elastomeric pads or sliding plate expansion joints are used on shorter spans while rocker or roller bearings are used on the longer spans. Provisions must be made for the beam camber resulting from prestressing. Examples of these types of structures are shown in Figures 13, 14, and 15.

Plate Girder Spans

G. In general, where spans exceed 60 to 80 feet, welded plate girders are economical. Use of composite design results in additional economy and less depth of structure. The maximum span lengths of girders built are in excess of 400 feet, which allows the engineer a great many selections in structure types. The choice of simple, cantilever, or continuous design should be determined by comparative estimates, as no general rule for the determination of the most economical type has been found. The substructure conditions may be a deciding factor in choice of superstructure. Spans should be provided with field splices of members as governed by the limitations of transportation by railway or highway and by erecting conditions. All spans are provided with expansion bearings of rocker or roller type, and the deck slab is provided with metal expansion joints at the ends of the continuous structure. All spans are cambered for the total calculated dead-load deflection whenever it exceeds the tolerance provided by the specifications. In addition, deflections due to live loads must be within specification tolerances. Examples of plate girder construction are shown in Figures 16, 17, and 18.

Truss and Arch Spans

H. Spans in excess of 120 feet may economically be constructed using trusses, depending on loading, clearance, and terrain. In highway construction intermediate expansion joints in the roadway slab are provided

CONCRETE AND STEEL HIGHWAY BRIDGES--Continued

Truss and Arch Spans (Cont'd.)

in spans over 300 feet. All spans are provided with expansion bearings and a metal expansion joint in the deck at the expansion end of simple or continuous trusses. All spans are cambered for the dead load of the structure. Trusses can be simple, cantilever, or continuous. Other types of structures may be considered such as a tied arch or a two-hinged arch, with the most economical design based on comparative estimates. Examples of truss and arch construction are shown in Figures 19, 20, and 21.

Rigid Frame Spans

I. Where studies indicate that it is economical, rigid frame construction may be used. An example is the concrete bridge shown in Figure 22.

CONCRETE AND STEEL RAILROAD BRIDGES

GENERAL

.12 Open timber decks have often been used for railroad bridges, as they are generally economical in first cost. However, such decks require special deck framing and do not permit the use of in-line modern mechanized track-laying operations as do ballasted decks. Ballasted decks are also less expensive to maintain. Therefore, modern railroad requirements, particularly on main lines, may require that ballasted decks be used. Closed timber deck or ballasted deck is necessary on overpasses for safety reasons. Ballasted track may be supported on either concrete or treated timber decks. The concrete deck may be either constructed of precast concrete units which are readily placed by railway cranes or cast in place. Owing to the large dead load of ballast and timber or concrete decks, bridges of such construction are generally more expensive than bridges with open decks. With railroad approval the use of composite design for some railroad girder bridges, to utilize fully the cast-in-place deck, has produced an economical ballasted deck girder design. As piers for railroad bridges are massive, their number should be held to a minimum for multiple-span bridges on canal crossings, particularly when loss of head in the canal is important. However, crossing a large canal with a single span is usually prohibitive in cost.

SELECTION OF TYPE OF BRIDGE

.13 For economy of construction, the type of bridge varies with the length of span. The engineer may prepare economic studies, including the erection requirements, in the final selection of the type of structure to be constructed.

Rolled Steel Beam Spans

A. Steel beams are used for open or ballasted deck with spans up to approximately 50 feet. Composite design with cast-in-place deck would allow longer spans and more economy, but would require specific railroad approval. Two steel beams connected by steel diaphragms are generally required under each rail. Simple-span bridges are used rather than continuous spans, for with the heavy live load inherent in all railway construction, the uplift forces would require use of expensive anchor bearings on continuous spans. Expansion bearings of sliding plate type are used at the expansion end of simple spans. Bridges of this type are shown in Figures 23 and 24.

Steel Girder Spans

B. Steel girders are generally used for open or ballasted deck bridges of spans 45 to 120 feet. Center-to-center girder spacing for deck bridges varies from 6 feet 6 inches for short spans to 8 feet for the longer spans. For through plate girder spans the spacing of girders is determined by clearance specifications. Through plate girders should be avoided if possible for economical reasons, except when vertical waterway clearances do not permit other types of structure. Through plate girder bridges are illustrated in Figures 25, 26, and 27. Girder spans are provided with rocker- or roller-type expansion bearings. Ballasted deck

9.13C

CONCRETE AND STEEL RAILROAD BRIDGES--Continued

Steel Girder Spans (Cont'd.)

bridges provide expansion devices to retain ballast. Girders for railway bridges are usually shop fabricated in one piece and usually can be erected by heavy railroad cranes without the need of falsework. Plate girders more than 90 feet in length are cambered equal to the calculated deflection produced by the dead load only. Deck-type steel girder bridges are illustrated in Figures 28, 29, and 30.

Prestressed Concrete Box Girder Spans

C. Prestressed concrete box girders are used for ballasted track bridges. These units are erected requiring no falsework, thus reducing the time the railroad is out of operation. The spans are provided with elastomeric pads for expansion and rotation due to deflection. Dowels of sufficient size are provided at fixed end to transfer the longitudinal loading. The maximum span length of this type of span is determined by the capacity of the equipment erecting the box girder. A prestressed concrete box girder of this type is shown in Figure 31.

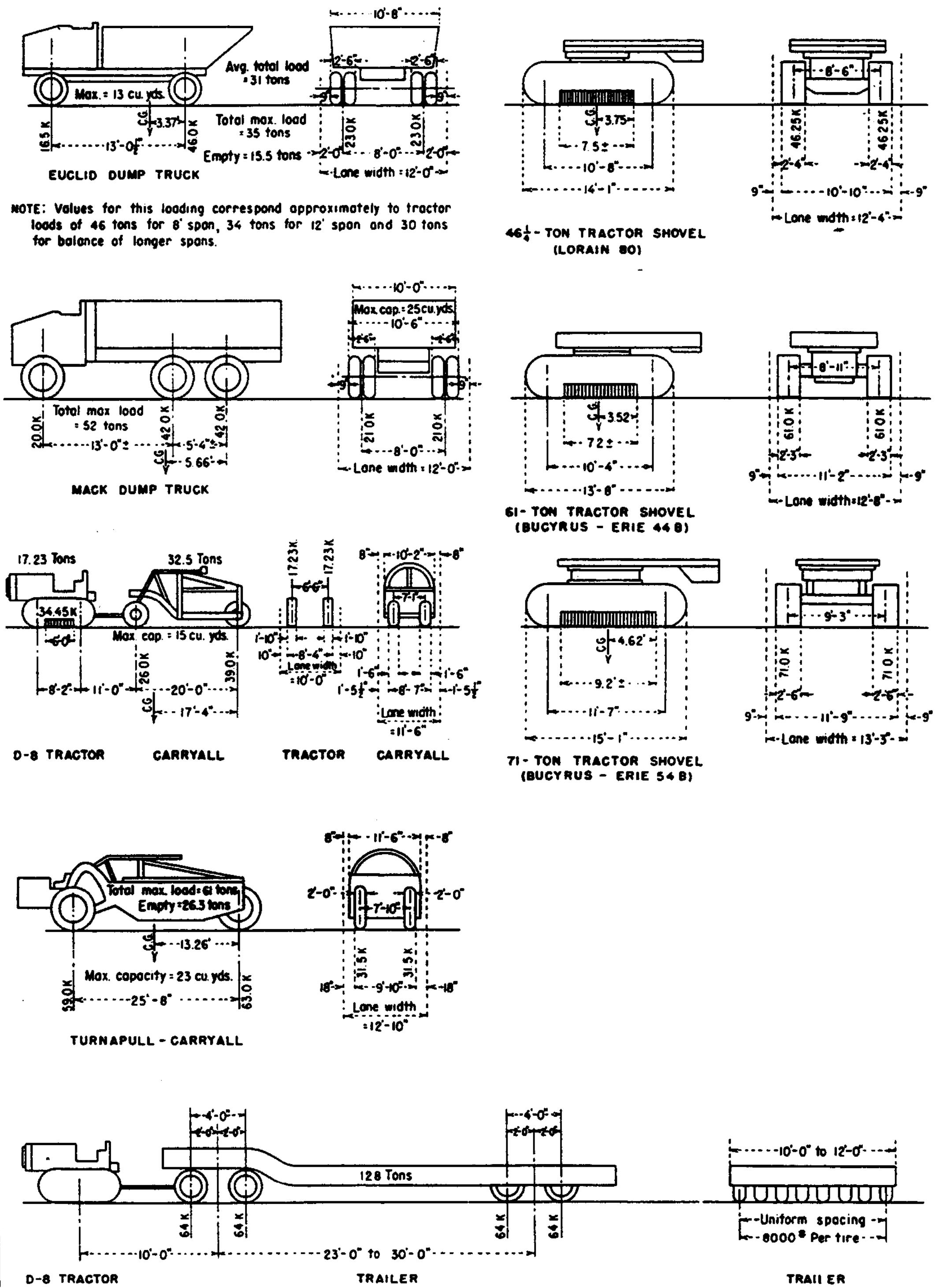
Truss Spans

D. Bridges for spans above 120 feet are seldom used for canal crossings, but are used for river crossings in connection with railway relocations and to accommodate reservoir construction. Truss spans exceeding 300 feet require special provisions for expansion in the floor system. Truss design requires special consideration of local conditions dictating their use. All trusses are cambered as required by specifications. An example of a truss railway bridge is shown in Figure 30.

Reinforced Concrete Trestle Spans

E. Reinforced concrete trestle bridges are usually composed of a series of ballasted short spans of precast concrete slabs approximately 22 feet in maximum length, supported on pile bents. The bents usually consist of three or more piles with a cast-in-place reinforced concrete cap. The piles may be either precast reinforced concrete, concrete-filled tubular steel, or steel H piles. Expansion is provided by open joints with asphalt planks to prevent ballast from entering the opening. The slabs are placed on premolded joint material. Reinforced concrete trestle spans are also used for approach spans for a larger bridge. An example using the reinforced concrete trestle is shown in Figure 28.

HEAVY EQUIPMENT LOADING DIAGRAMS Fig. 1 Par. 9.4C

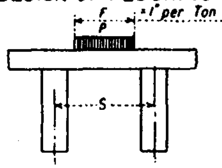


HEAVY EQUIPMENT LOADING DIAGRAMS

4-17-47-F.J.W. FORMERLY X-D-3715 103-D-230

Fig. 2 Par. 9.7 TIMBER BRIDGES--FLOOR, STRINGERS, AND BENTS

DESIGN OF FLOORING



It is assumed that the extreme fiber stress due to moment will govern; for which stress the moment is calculated by the following formula —

$M = P/8\,(2s - F)$ F ≥ s

P = Wheel Load

F = Width of Wheel

s = Spacing of Beams

Effective width: width of planks for plank floor, 15 inches for laminated floor

DESIGN OF STRINGERS FOR BENDING

Distribute the concentrated wheel loads according to formula below, which will give the fraction of concentrated wheel load to be carried by one beam

P = Wheel Load

P_1 = Load per Beam

c_1 = Coefficient of Distribution

s = Distance c. to c. of Beams in Feet

$P_1 = P \times s/c_1$

TABLE OF VALUES FOR C_1

TYPE OF FLOOR	ONE LANE C_1	ONE LANE MAX SPAC.	TWO LANES C_1	TWO LANES MAX SPAC.
Plank floor	4.0	4.0'	3.75	3.75'
4 in. strip floor or 4 in. blocks on subplanks	4.5	4.5'	4.0	4.0'
6 in. strip or more	5.0	5.0'	4.25	4.25'

DESIGN OF STRINGERS FOR BEARING

Live load carried by one stringer

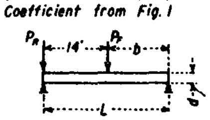
$= P_R \times (1 - F/4s) + P_F \times c_2 \times \frac{b}{L}$

P_R = Rear wheel P_F = Front wheel

F = Width of Wheel

s = Distance c. to c. of Beams

c_2 = Coefficient from Fig. 1



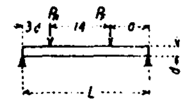
DESIGN OF STRINGERS FOR HORIZONTAL SHEAR

Neglect all loads within a distance from the supports equal to depth of beam

Heaviest concentrated load to be placed at a distance from the support equal to three times the depth of the beam

Distribute wheel loads according to curves of Fig 1, which will give the fraction of wheel load to be carried by one beam

Use only 90 per cent of the live load shear when calculating the horizontal shear stress



P_R = Rear wheel, P_F = Front wheel

$P_{R1} = c_2 P_R$ $P_{F1} = c_2 P_F$

c_2 for P_{R1} and P_{F1} are different

w = Uniform dead load in lbs. per lin. ft

Dead Load Shear = $S_D = w(L/2 - d)$

Live Load Shear = $S_L = P_{R1}(L - 3d) \div L + P_{F1}a \div L$

Total Shear = $S = S_D + 0.9\,S_L$

Unit Horizontal Shear = $1.5 \times S \div bd$

d = Depth of Beam

b = Width of Beam

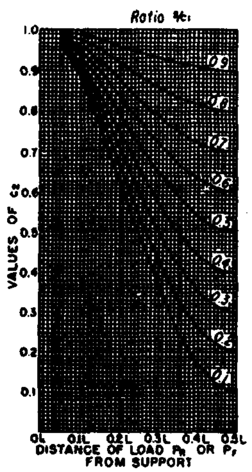


FIGURE 1

BENTS

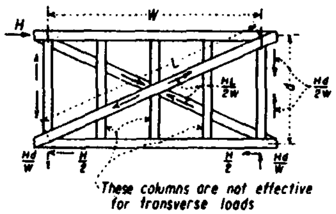


FIGURE 2

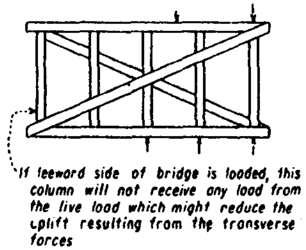


FIGURE 3

ASSUMPTIONS FOR WIND LOADS

(1) For one or two span bridges, the deck will carry all the wind loads to the abutments

(2) For three spans or more, design the bent for a wind load of 30 lb. per sq. ft. on the unloaded bridge, and 15 lb per sq ft on the loaded bridge with a live load wind of 100 lb per lin. ft of bridge.

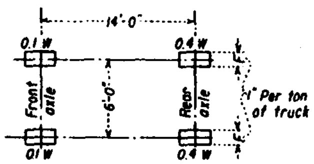
(3) For stress assumptions in bent see Fig 2 and 3

DETAILS

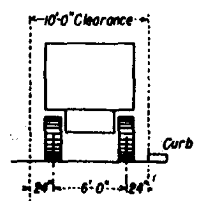
(1) Ring connectors shall be used where bolts are not strong enough, otherwise 3/4" ϕ bolted connections

(2) Where the length of bracing exceeds 32' due to width of bent, two panels of bracing shall be used with even number of post spaces

LOADING DIAGRAM



W = Total weight of loaded truck



UNIT STRESSES

BENDING........................1700 LBS PER SQ. IN

COMPRESSION (COLUMNS).......1300 LBS PER SQ. IN.

BEARING........................455 LBS PER SQ. IN.

HORIZONTAL SHEAR............145 LBS PER SQ. IN.

THIS DRAWING SUPERSEDES DWG. X-D-1296

8-11-54 D L.H.B.	CHANGE IN UNIT STRESSES PER A.A.S.H.O. 1953

UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DESIGN DATA

TIMBER BRIDGES

DESIGN

FLOOR, STRINGERS AND BENTS

DRAWN....J.M.E.........SUBMITTED....[signature]

TRACED....C.E.M.........RECOMMENDED....[signature]

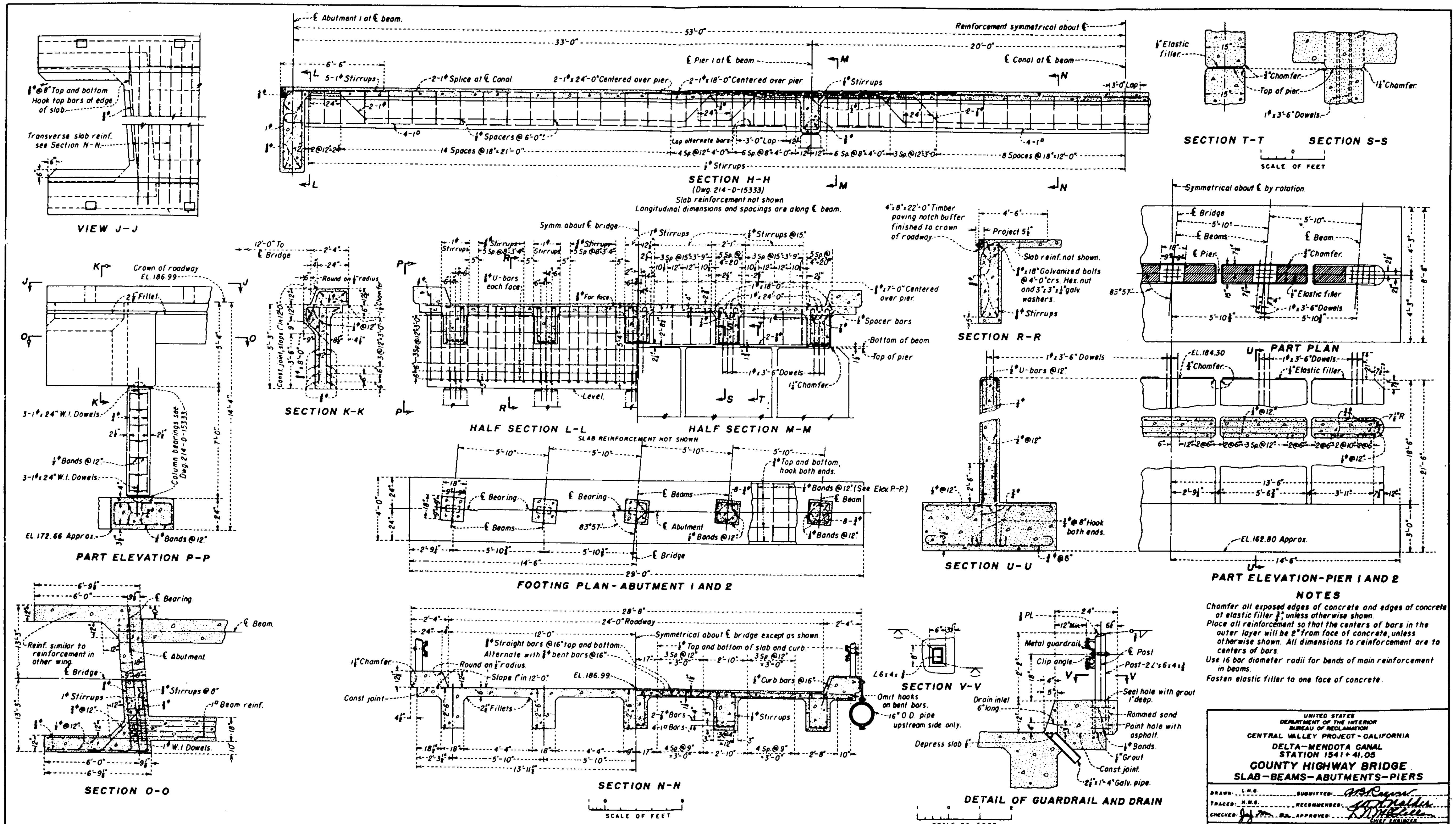
CHECKED....SCC.........APPROVED....[signature]

CHIEF DESIGNING ENGINEER

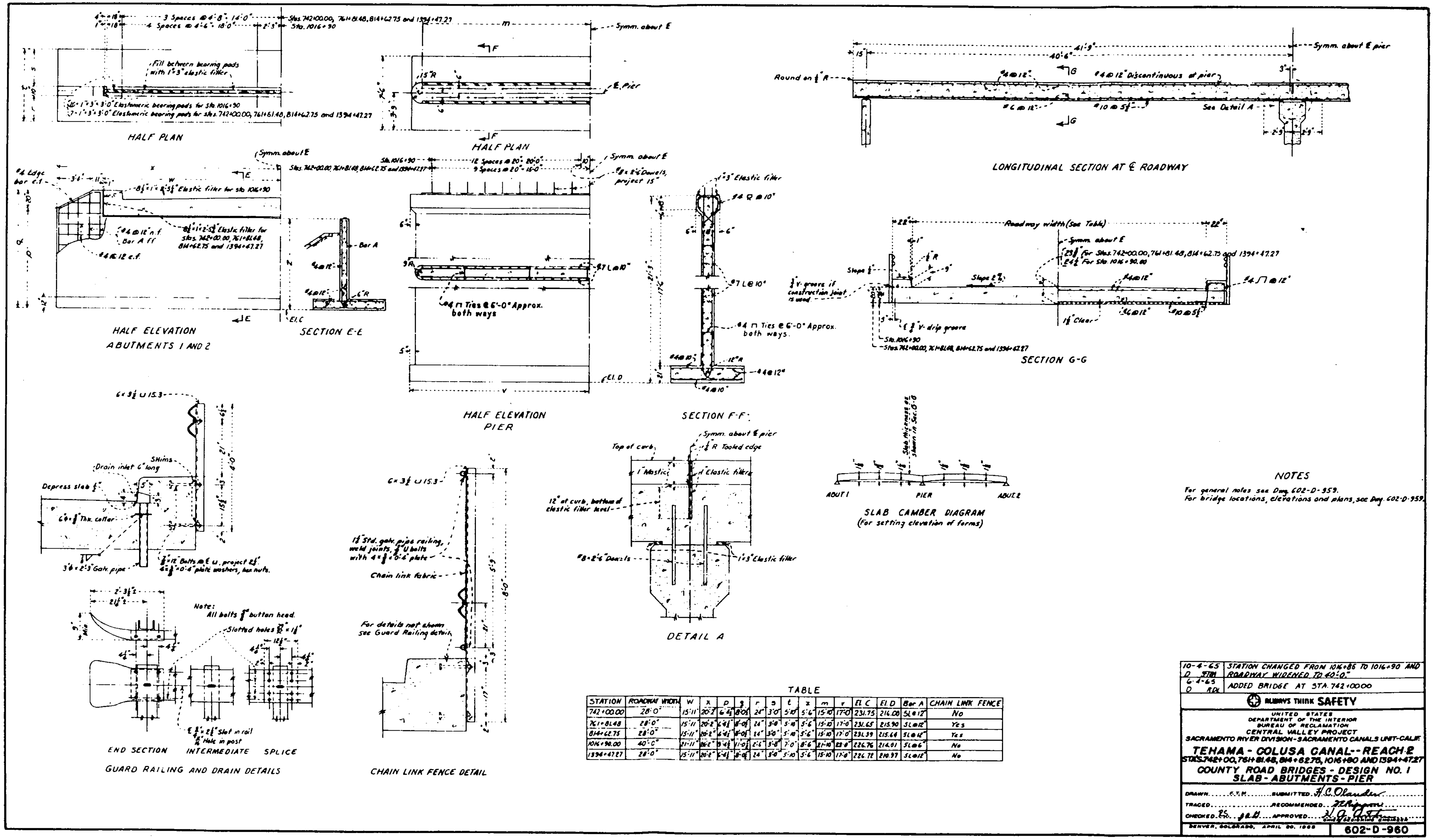
DENVER, COLORADO, MAY 3, 1949 — 103-D-23

COUNTY HIGHWAY BRIDGE--SLAB, BEAMS, ABUTMENTS, PIERS--DELTA-MENDOTA CANAL

Fig. 3 Par. 9.11



COUNTY ROAD BRIDGES--SLAB, ABUTMENTS, PIER--TEHAMA-COLUSA CANAL, REACH 2 Fig. 4 Par. 9.11A

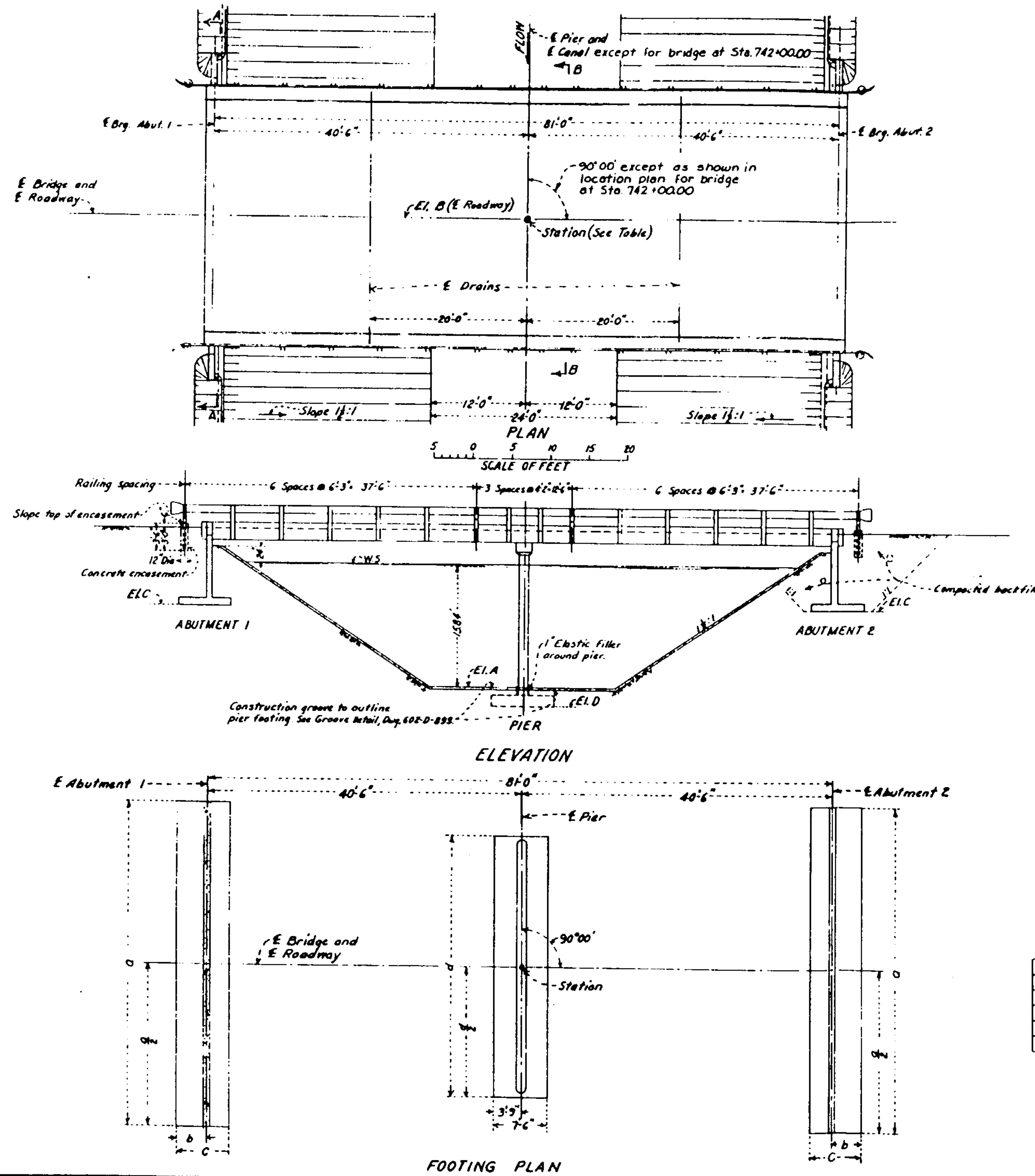


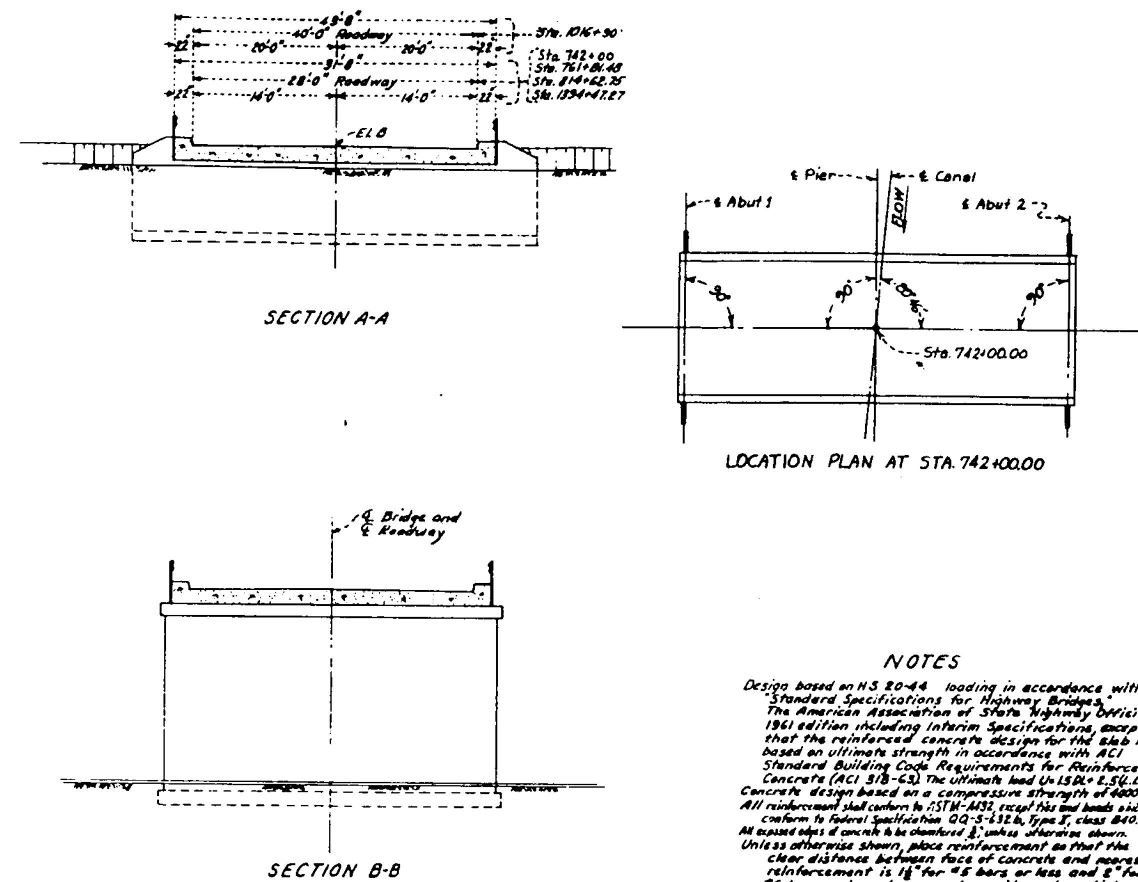
TABLE

STATION	ROADWAY WIDTH	w	x	p	q	r	s	t	z	m	v	El C	El D	Bar A	CHAIN LINK FENCE
742+00.00	28'-0"	15'-11"	20'-2"	6'-4½"	8'-0½"	24"	3'-0"	5'-10"	5'-6"	15'-10"	17'-0"	231.75	216.00	5L@12"	No
761+81.48	28'-0"	15'-11"	20'-2"	6'-4½"	8'-0½"	24"	3'-0"	5'-10"	5'-6"	15'-10"	17'-0"	231.65	215.90	5L@12"	Yes
814+62.75	28'-0"	15'-11"	20'-2"	6'-4½"	8'-0½"	24"	3'-0"	5'-10"	5'-6"	15'-10"	17'-0"	231.39	215.64	5L@12"	Yes
1016+90.00	40'-0"	21'-11"	26'-2"	8'-4½"	11'-0½"	2'-6"	3'-8"	7'-0"	8'-6"	21'-10"	23'-0"	226.76	214.01	5L@6"	No
1394+47.27	28'-0"	15'-11"	20'-2"	6'-4½"	8'-0½"	24"	3'-0"	5'-10"	5'-6"	15'-10"	17'-0"	226.72	210.97	5L@12"	No

COUNTY ROAD BRIDGES--PLAN, ELEVATION, SECTIONS--TEHAMA-COLUSA CANAL, REACH 2

Fig. 5 Par. 9.11A





NOTES

Design based on HS 20-44 loading in accordance with "Standard Specifications for Highway Bridges," The American Association of State Highway Officials, 1961 edition including Interim Specifications, except that the reinforced concrete design for the slab is based on ultimate strength in accordance with ACI Standard Building Code Requirements for Reinforced Concrete (ACI 318-63). The ultimate load U=1.5DL+2.5(L.L+I).
Concrete design based on a compressive strength of 4000 psi..
All reinforcement shall conform to ASTM-A432, except ties and bends which shall conform to Federal Specification QQ-S-632b, Type II, class B40.
All exposed edges of concrete to be chamfered 3/4" unless otherwise shown.
Unless otherwise shown, place reinforcement so that the clear distance between face of concrete and nearest reinforcement is 1 1/2" for #5 bars or less and 2" for #6 bars and greater, except provide a clear distance from face of concrete placed against earth or rock of 2" where thickness is 9" or less and 3" where thickness is greater than 9".
All dimensions to reinforcement are to centers of bars.
Lap bars 30 diameters unless otherwise shown.
n.f. f.f. and e.f. indicate near face, far face, and each face respectively.
Width of 1" elastic filler around pier to be sufficient to extend from 1" above canal lining to the top of pier footing.

TABLE

STATION	El. A	El. B	El. C	El. D	a	b	c	d
761+81.48	218.86	239.18	231.65	215.90	40'-4"	3'-5"	5'-0"	34'-0"
814+62.75	218.60	238.92	231.39	215.64	40'-4"	3'-5"	5'-0"	34'-0"
1016+90.00	216.93	237.37	226.76	214.01	55'-6"	4'-1"	7'-0"	46'-0"
1394+47.27	219.93	234.25	226.72	218.97	40'-4"	3'-5"	5'-0"	34'-0"
742+00.00	218.96	239.28	231.75	216.00	40'-4"	3'-5"	5'-0"	34'-0"

10-4-65 D SDM	STATION CHANGED FROM 1016+85 TO 1016+90 AND ROADWAY WIDENED TO 40'-0"
6-8-65 D RDL	ADDED BRIDGE AT STA. 742+00.00

ALWAYS THINK SAFETY

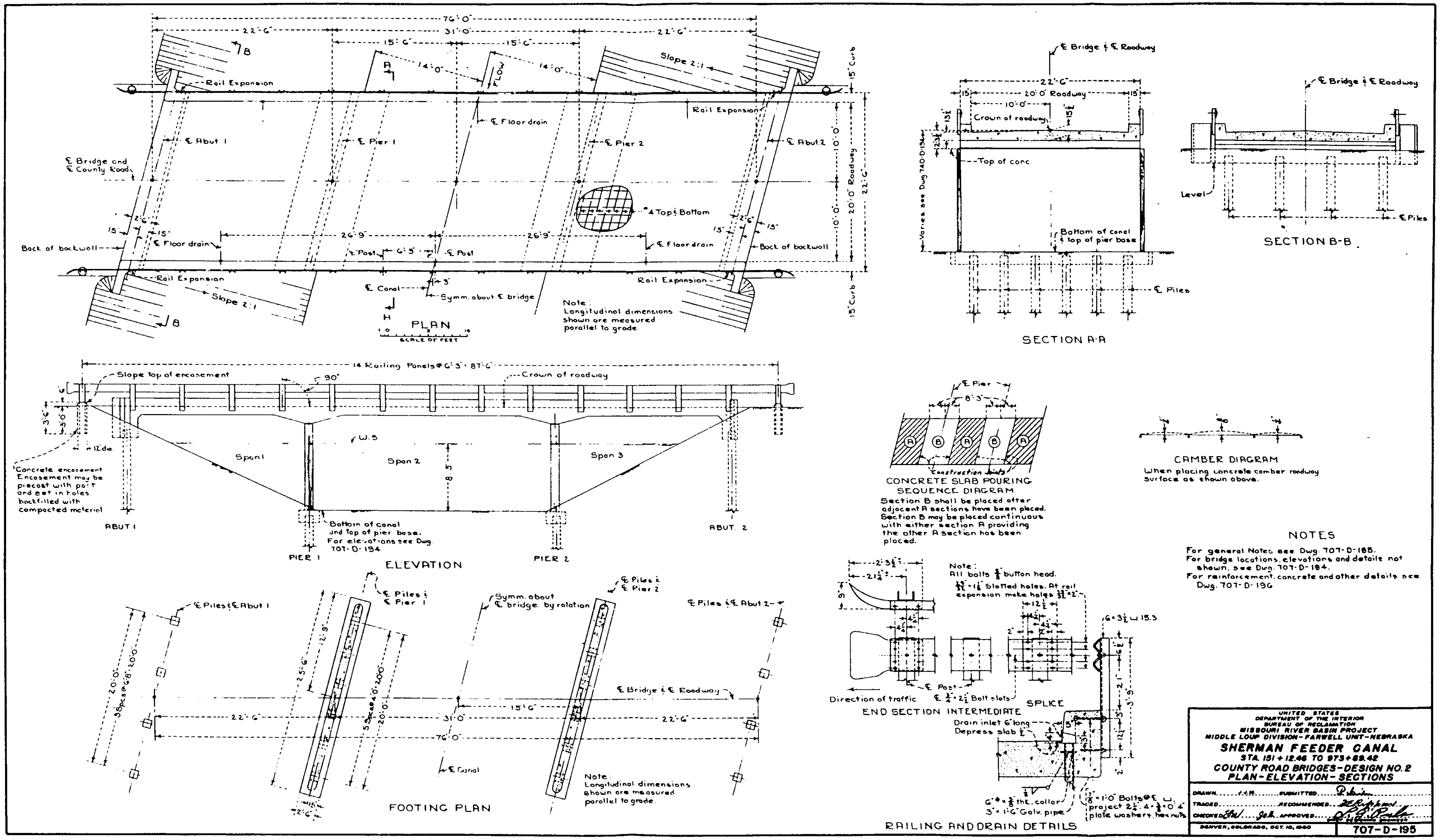
UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
CENTRAL VALLEY PROJECT
SACRAMENTO RIVER DIVISION-SACRAMENTO CANALS UNIT-CALIF.

TEHAMA - COLUSA CANAL--REACH 2
STAS. 742+00, 761+81.48, 814+62.75, 1016+90 AND 1394+47.27
COUNTY ROAD BRIDGES-DESIGN NO.1
PLAN-ELEVATION-SECTIONS

DRAWN F.T.M. SUBMITTED H.C. Olander
TRACED RECOMMENDED
CHECKED J.E.H. APPROVED
CHIEF DESIGNING ENGINEER
DENVER, COLORADO. APRIL 20, 1965 602-D-959

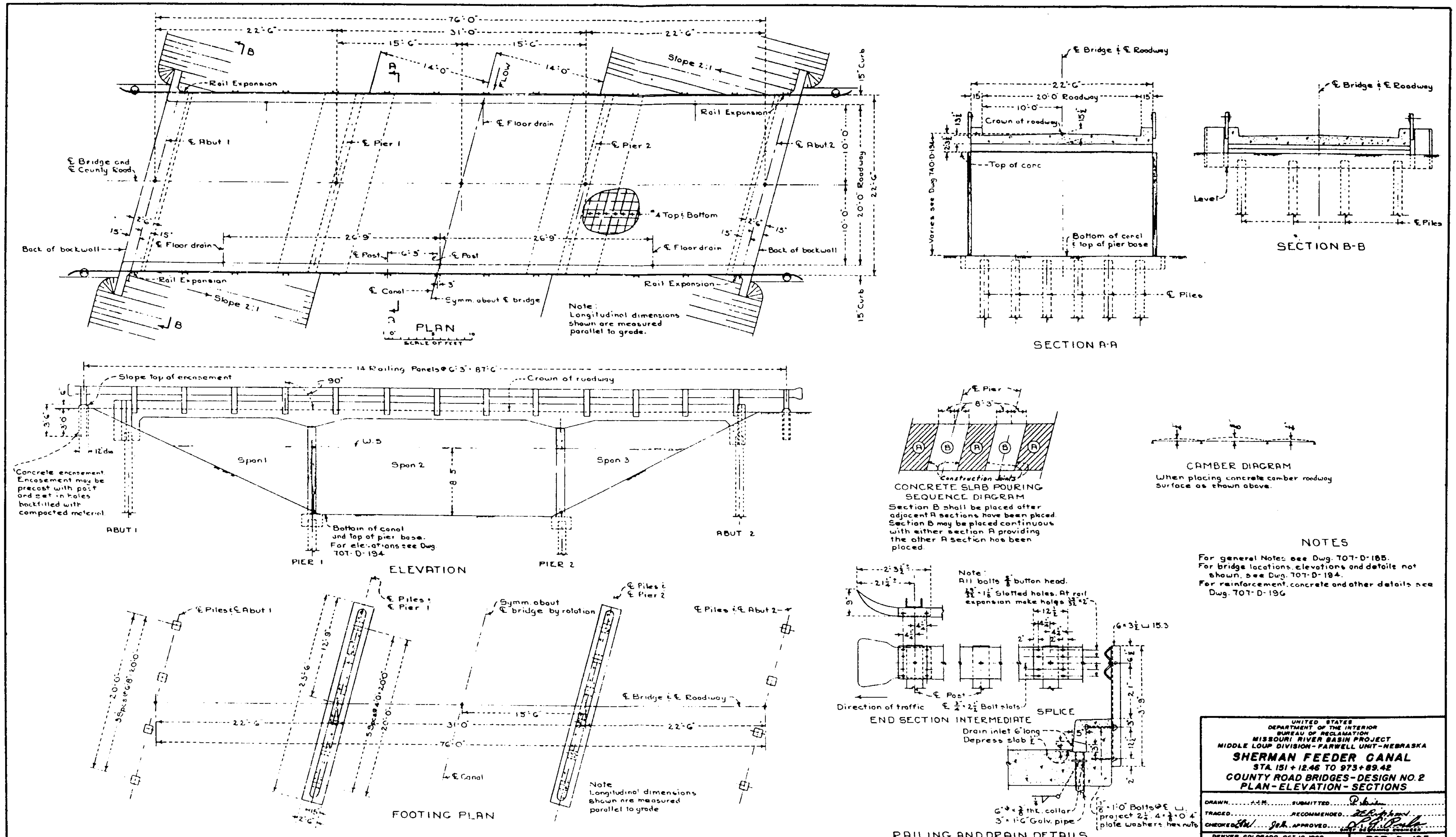
COUNTY ROAD BRIDGES--PLAN, ELEVATION, SECTIONS--SHERMAN FEEDER CANAL

Fig. 6 Par. 9.11B



COUNTY ROAD BRIDGES--PLAN, ELEVATION, SECTIONS--SHERMAN FEEDER CANAL

Fig. 6 Par. 9.11B



CONCRETE BRIDGES--PLAN, ELEVATION, SECTIONS--EXTENSION OF WELTON-MOHAWK MAIN OUTLET DRAIN

Fig. 7 Par. 9.11C

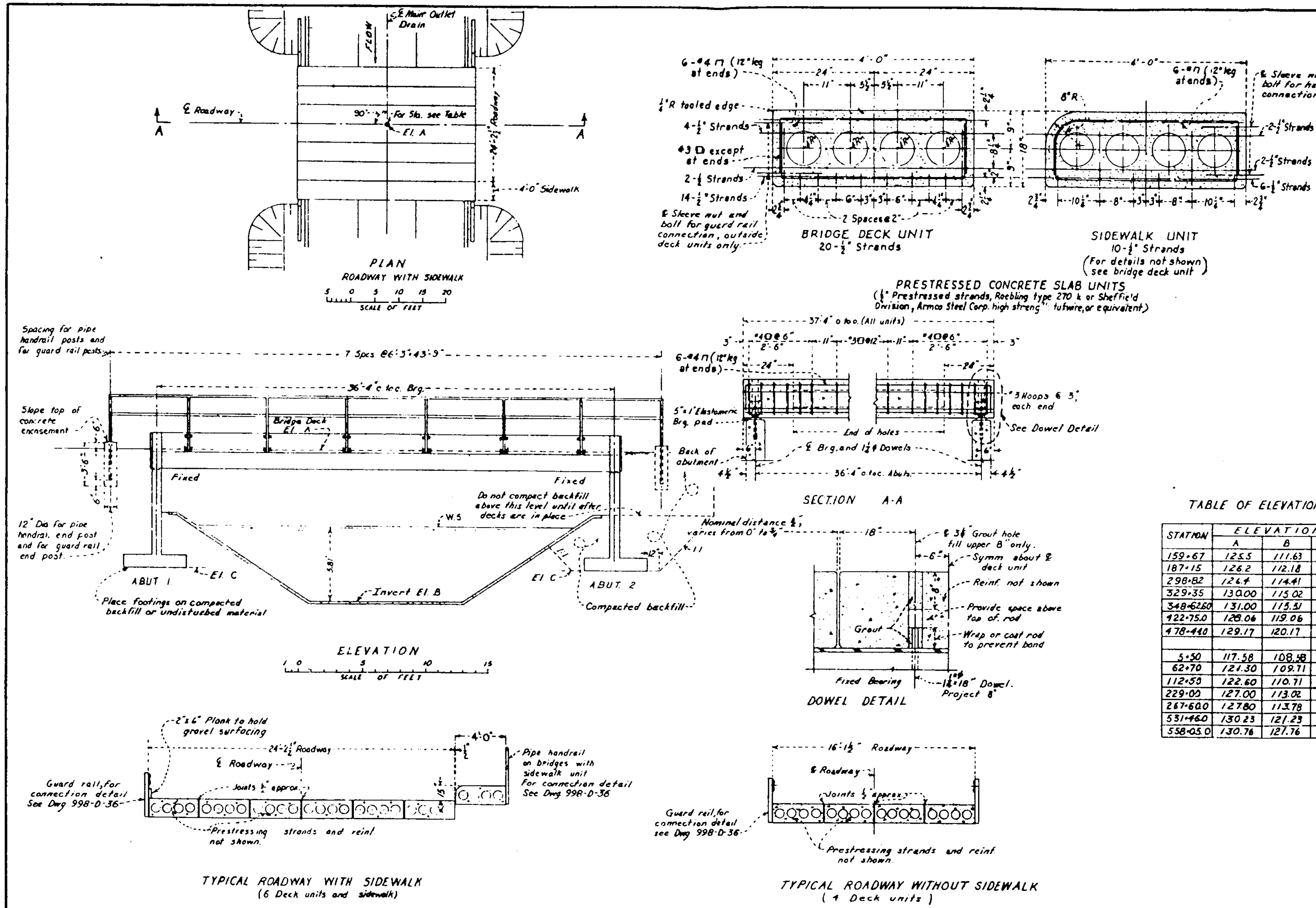


TABLE OF QUANTITIES

STATION	NUMBER OF SLAB UNITS		CONC. IN ABUTS.	REINF. IN ABUTS	WT. OF GUARD RAIL POSTS	WT. OF PIPE HANDRAIL
	DECK	SIDEWALK				
159+67	6	—				
187+15	6	—				
298+82	6	1				
329+35	6	1				
348+62.60	6	1				
422+75.0	6	1	17 CY	2400 Lbs	450 Lbs.	400 Lbs.
478+44.0	6	—	15	2100	900	—
5+50	4	—				
62+70	4	—				
112+50	4	—				
229+00	4	—				
267+60.0	4	—				
531+46.0	4	—	12	1600	900	—
558+05.0	4	—	12	1600	900	—

TABLE OF ELEVATIONS

STATION	ELEVATION		
	A	B	C
159+67	125.5	111.63	114.0
187+15	126.2	112.18	114.7
298+82	126.4	114.41	116.9
329+35	130.00	115.02	117.5
348+62.60	131.00	115.51	118.5
422+75.0	128.06	119.06	121.56
478+44.0	129.17	120.17	122.67
5+50	117.58	108.58	110.53
62+70	121.30	109.71	111.80
112+50	122.60	110.71	113.10
229+00	127.00	113.02	115.50
267+60.0	127.80	113.78	116.30
531+46.0	130.23	121.23	123.73
558+05.0	130.76	121.76	124.26

NOTES

Bridges designed for H20-44 loading in accordance with "Standard Specifications for Highway Bridges." The American Association of State Highway Officials 1961 Edition and subsequent revisions.

Concrete design based on a compressive strength of 5000 psi for pre stressed slabs and 3000 psi for abutments.

Unless otherwise shown, all exposed edges of concrete to be chamfered ¾".

For general notes pertaining to reinforcement, see Dwg 998-D-52

REFERENCE DRAWINGS

RAILING	998-D-36
ABUTMENTS	998-D-52

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UNITED STATES
DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION
DELIVERY OF WATER TO MEXICO
YUMA AREA - ARIZONA

EXTENSION OF WELLTON - MOHAWK MAIN OUTLET DRAIN
CONCRETE BRIDGES
PLAN - ELEVATION - SECTIONS

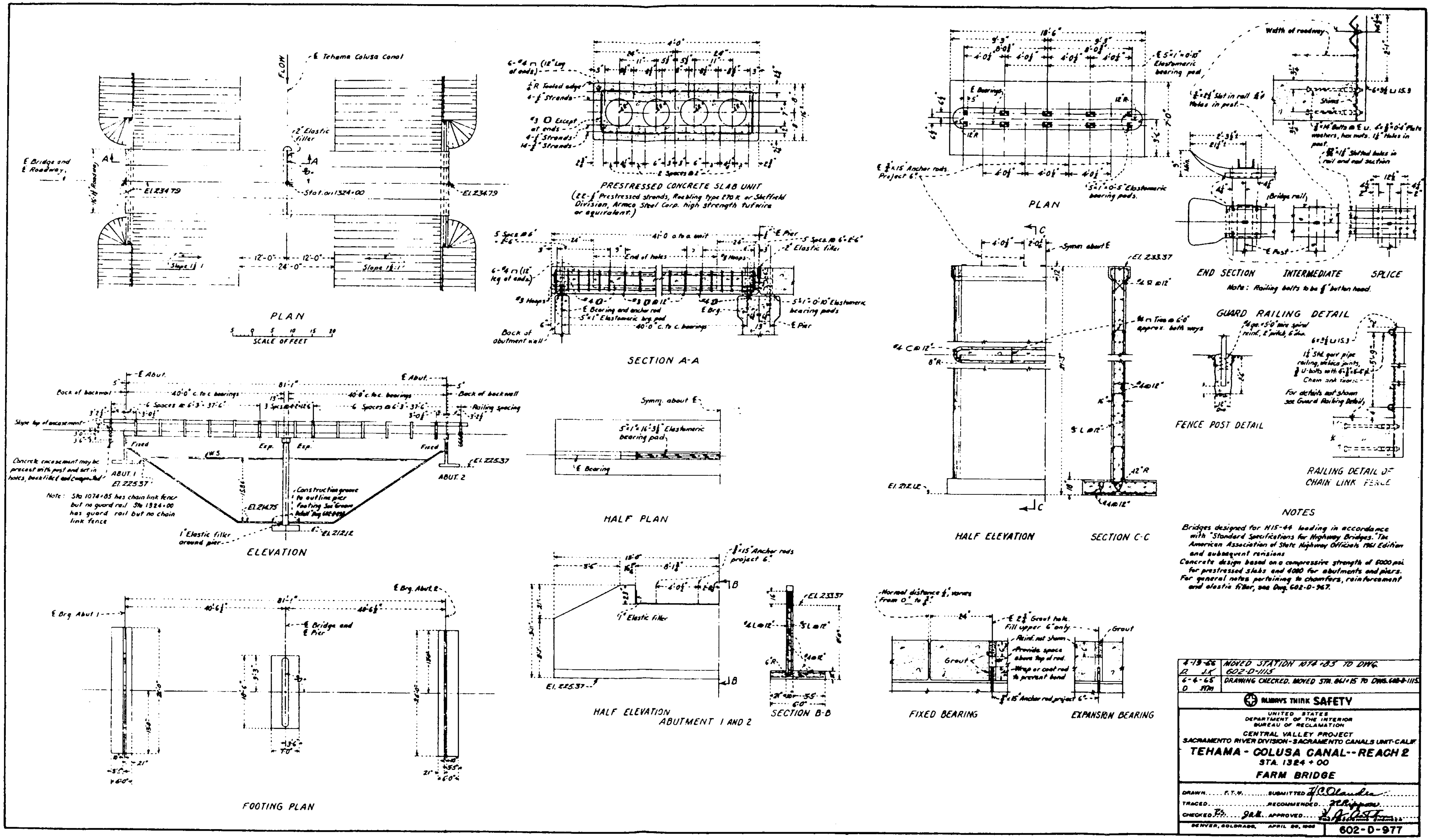
DRAWN W.L.L.:GFW SUBMITTED K.C. Olander
TRACED .... RECOMMENDED P.H. Terrell
CHECKED H.B.D. APPROVED H.G. Arthur CHIEF DESIGNING ENGINEER

DENVER, COLORADO JUNE 11, 1965 998-D-35

9-29-66 D.	AS BUILT BY 303, LTR. 3-8-66
9-27-65 D.	REVISED ELEVATIONS, CHANGED STA. 3+00 TO 5+50. CORRECTED DOWEL SIZE IN SEC. A-A
8-5-65 D.	CHANGED ELEVATIONS AND DOWEL DETAIL
7-20-65 D.	Added Roadway, Dowel detail, and Table of Elevations. Changed table of quantities, and minor changes.

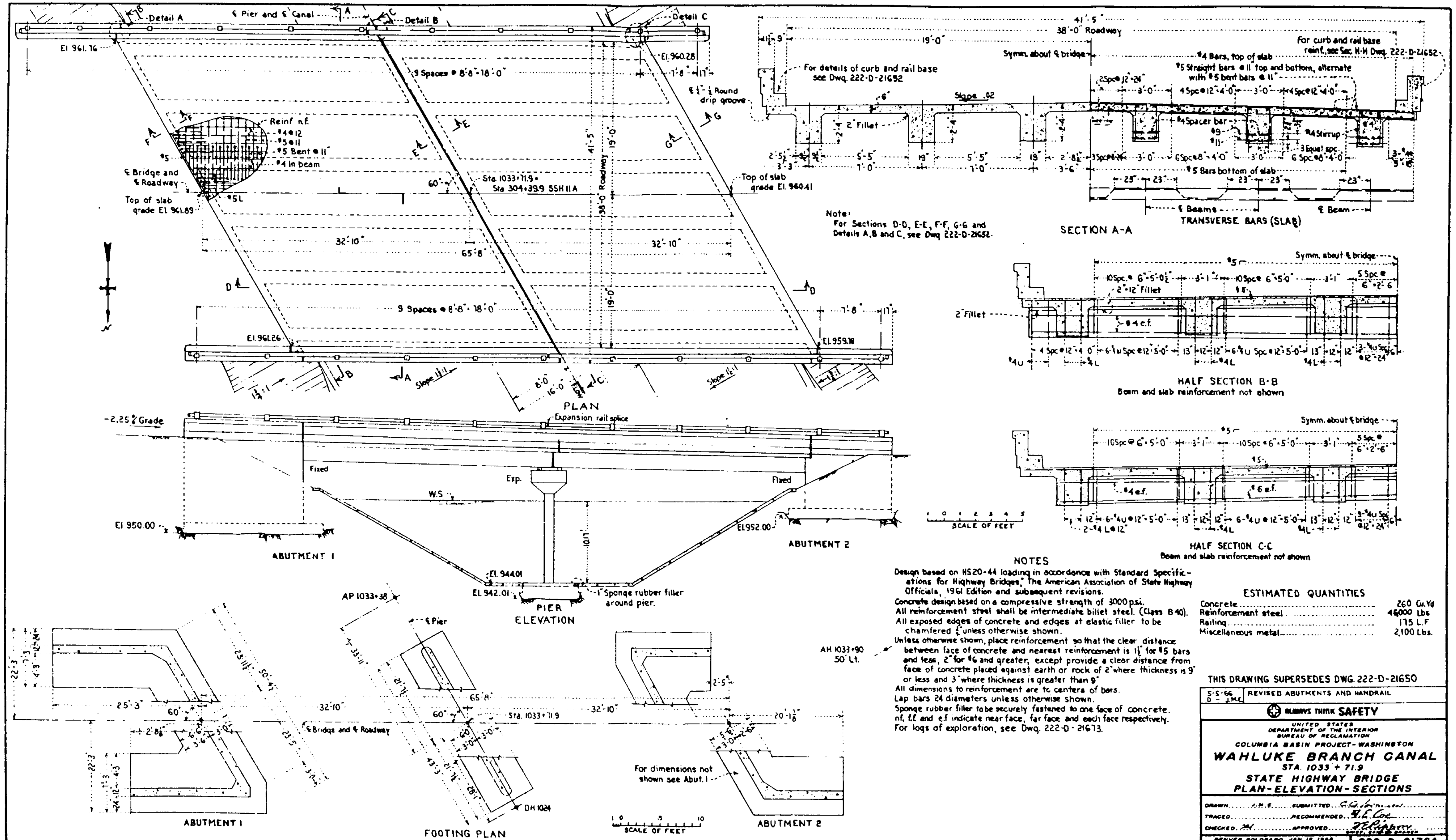
FARM BRIDGE--TEHAMA-COLUSA CANAL, REACH 2

Fig. 8 Par. 9.11C



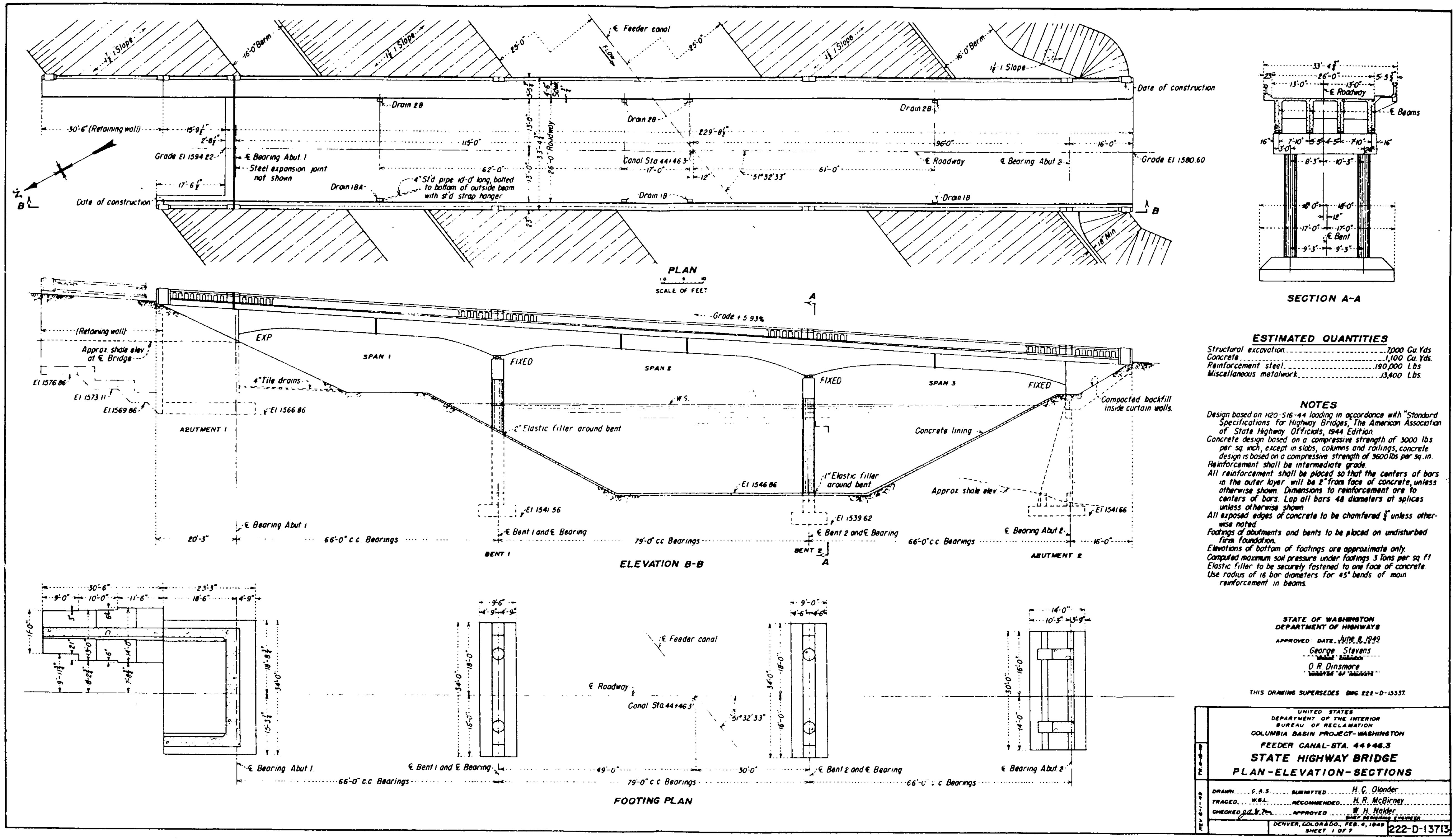
STATE HIGHWAY BRIDGE--PLAN, ELEVATION, SECTIONS--WAHLUKE BRANCH CANAL

Fig. 9 Par. 9.11D



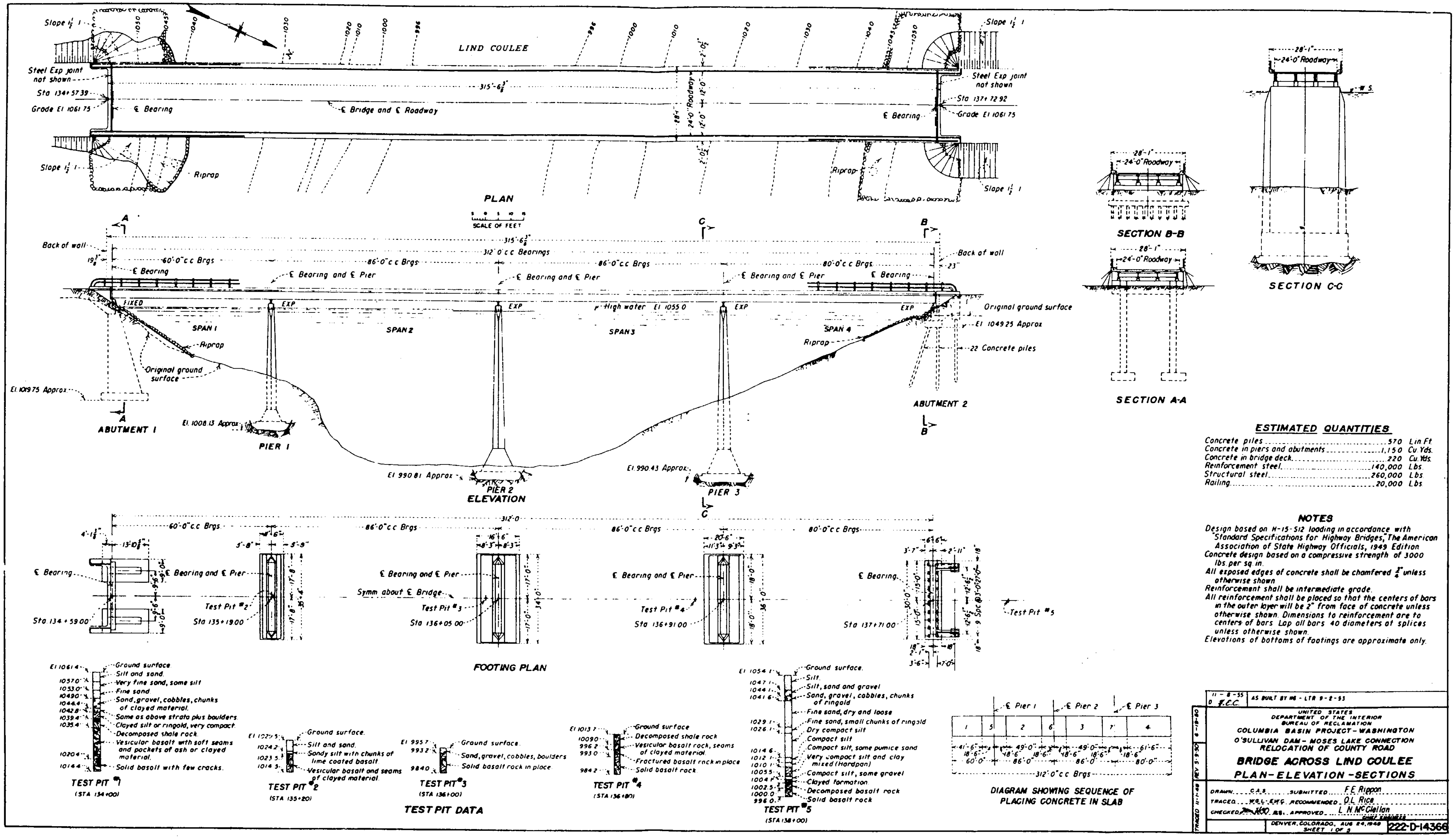
STATE HIGHWAY BRIDGE--PLAN, ELEVATION, SECTIONS--FEEDER CANAL, COLUMBIA BASIN

Fig. 10 Par. 9.11D



BRIDGE ACROSS LIND COULEE--PLAN, ELEVATION, SECTIONS--O'SULLIVAN DAM-MOSES LAKE CONNECTION

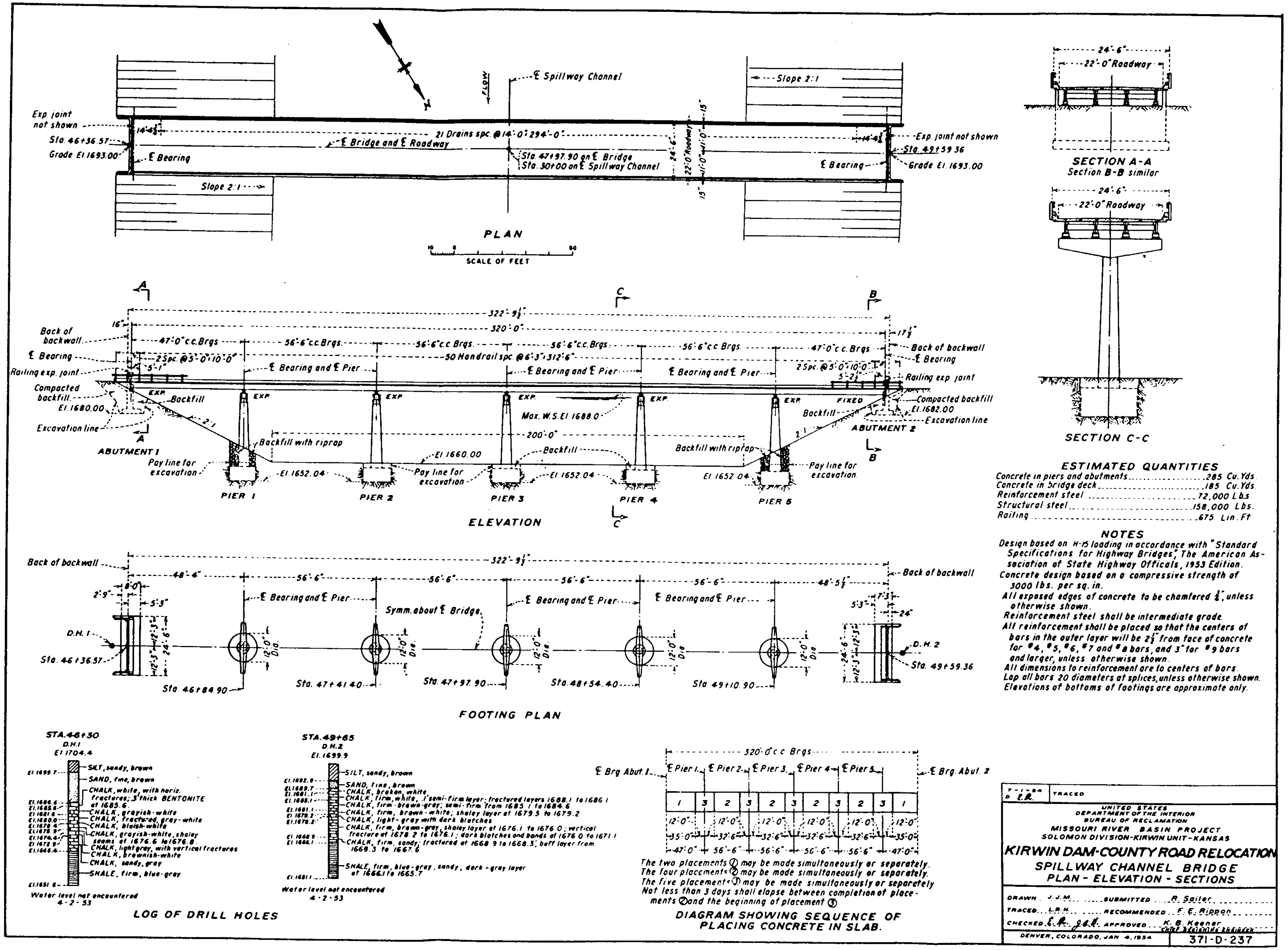
Fig. 11 Par. 9.11E



DS-3-5 - 12/8/67

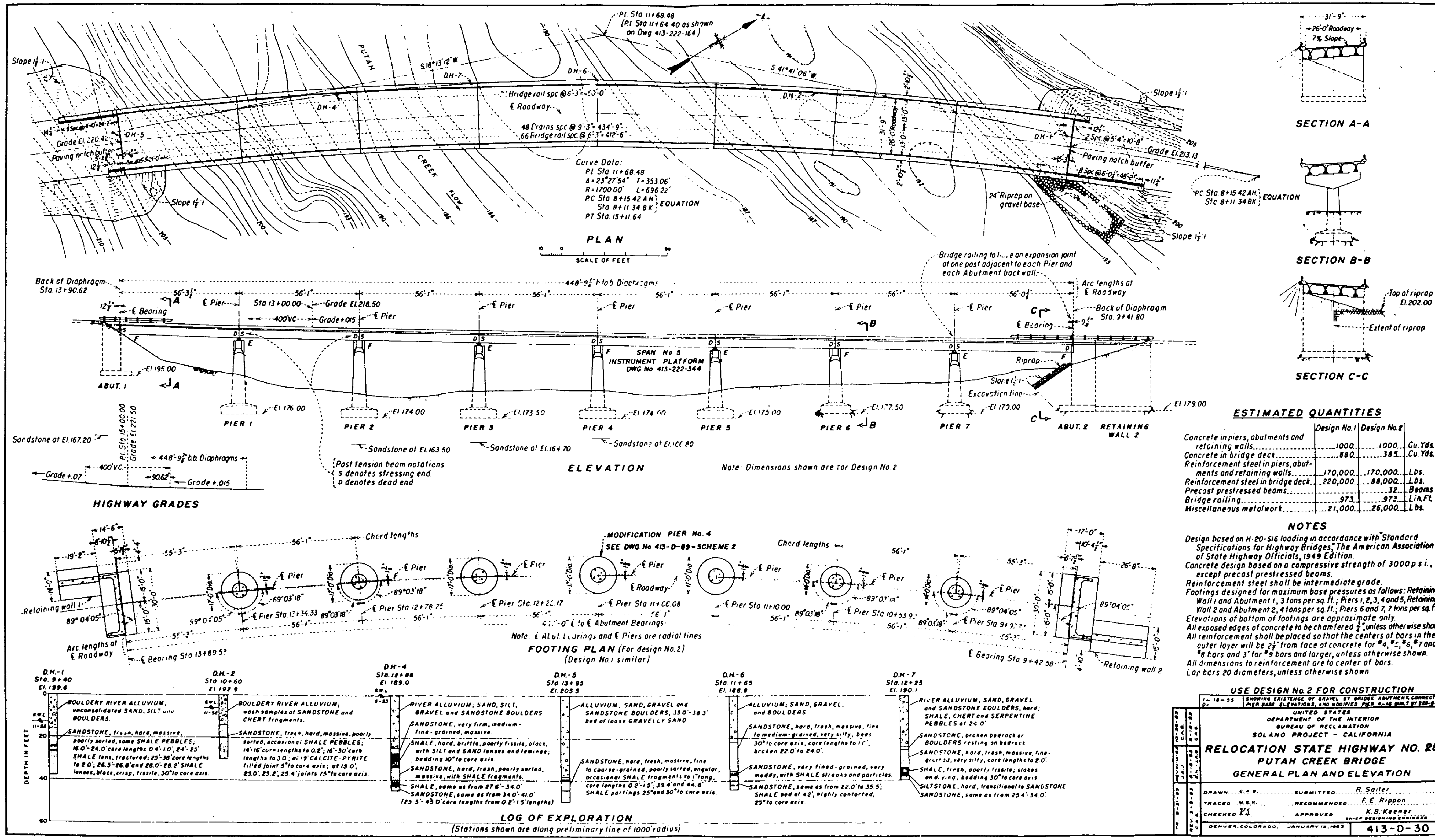
SPILLWAY CHANNEL BRIDGE--PLAN, ELEVATION, SECTIONS--KIRWIN DAM, COUNTY ROAD RELOCATION

Fig. 12 Par. 9.11E



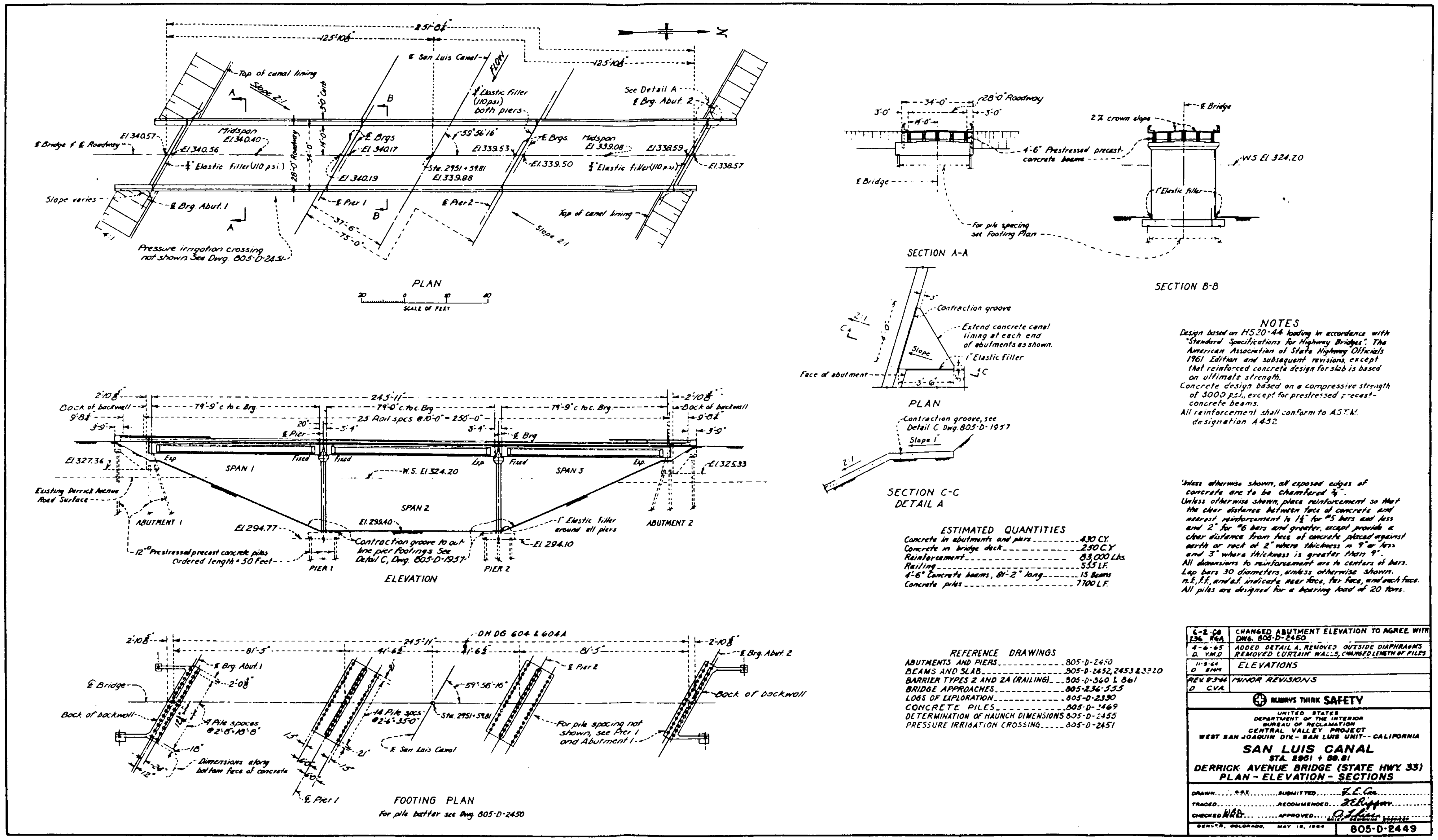
PUTAH CREEK BRIDGE--GENERAL PLAN AND ELEVATION--RELOCATION STATE HIGHWAY NO. 28, CALIFORNIA

Fig. 13 Par. 9.11F

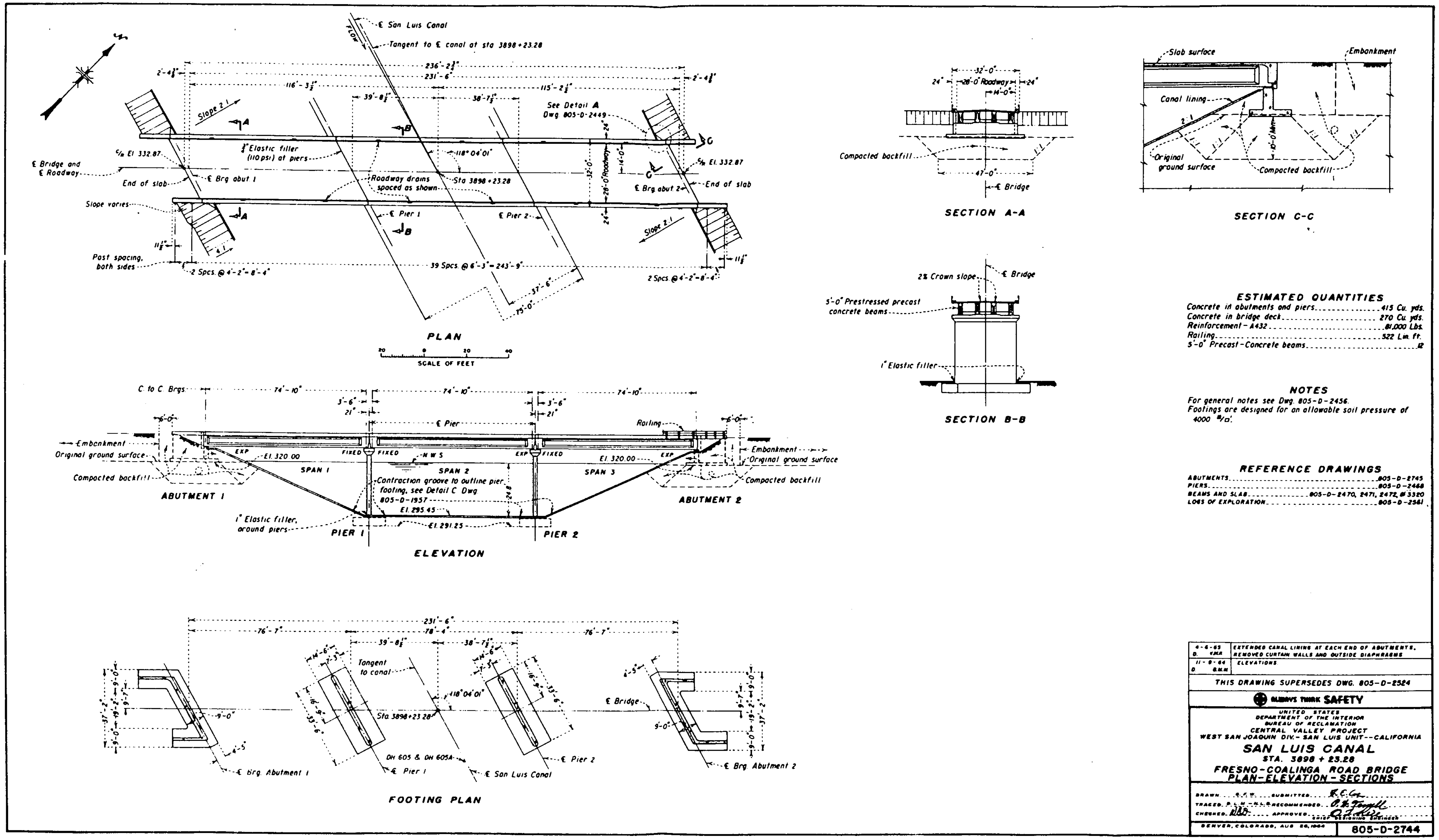


DERRICK AVENUE BRIDGE--PLAN, ELEVATION, SECTIONS--SAN LUIS CANAL

Fig. 14 Par. 9.11F

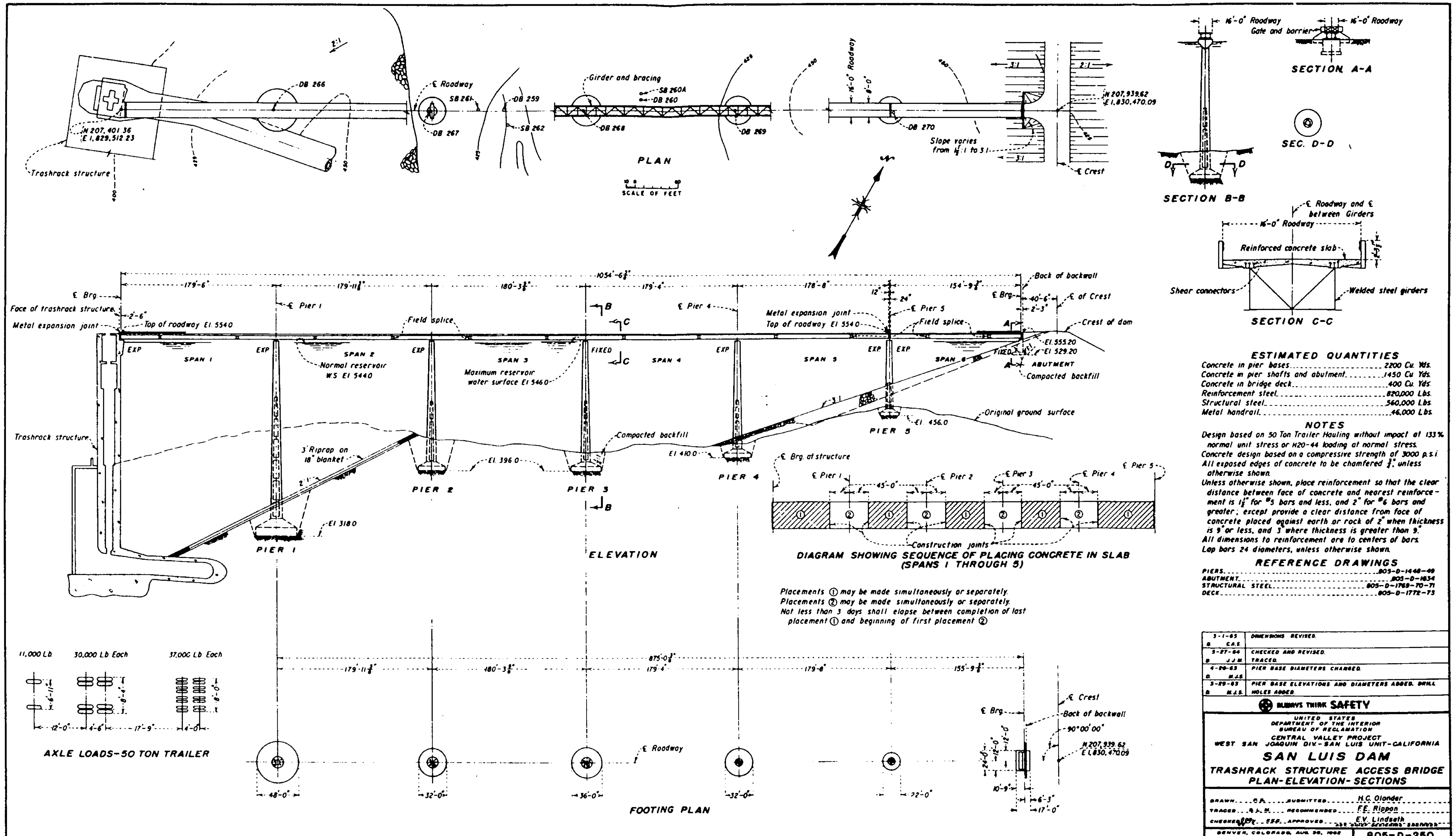


FRESNO-COALINGA ROAD BRIDGE--PLAN, ELEVATION, SECTIONS--SAN LUIS CANAL Fig. 15 Par. 9.11F



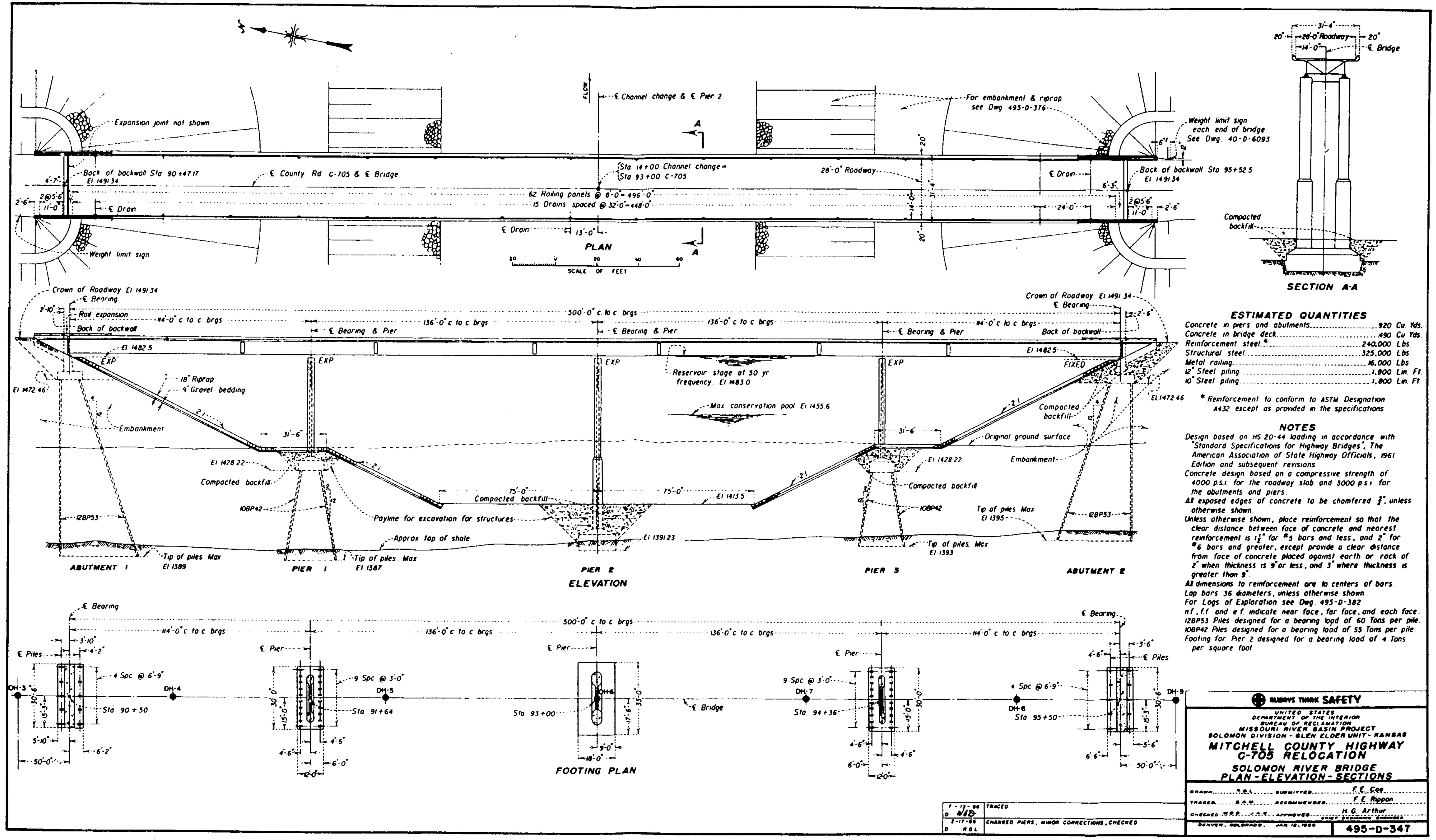
TRASHRACK STRUCTURE ACCESS BRIDGE--PLAN, ELEVATION, SECTIONS--SAN LUIS DAM

Fig. 16 Par. 9.11G



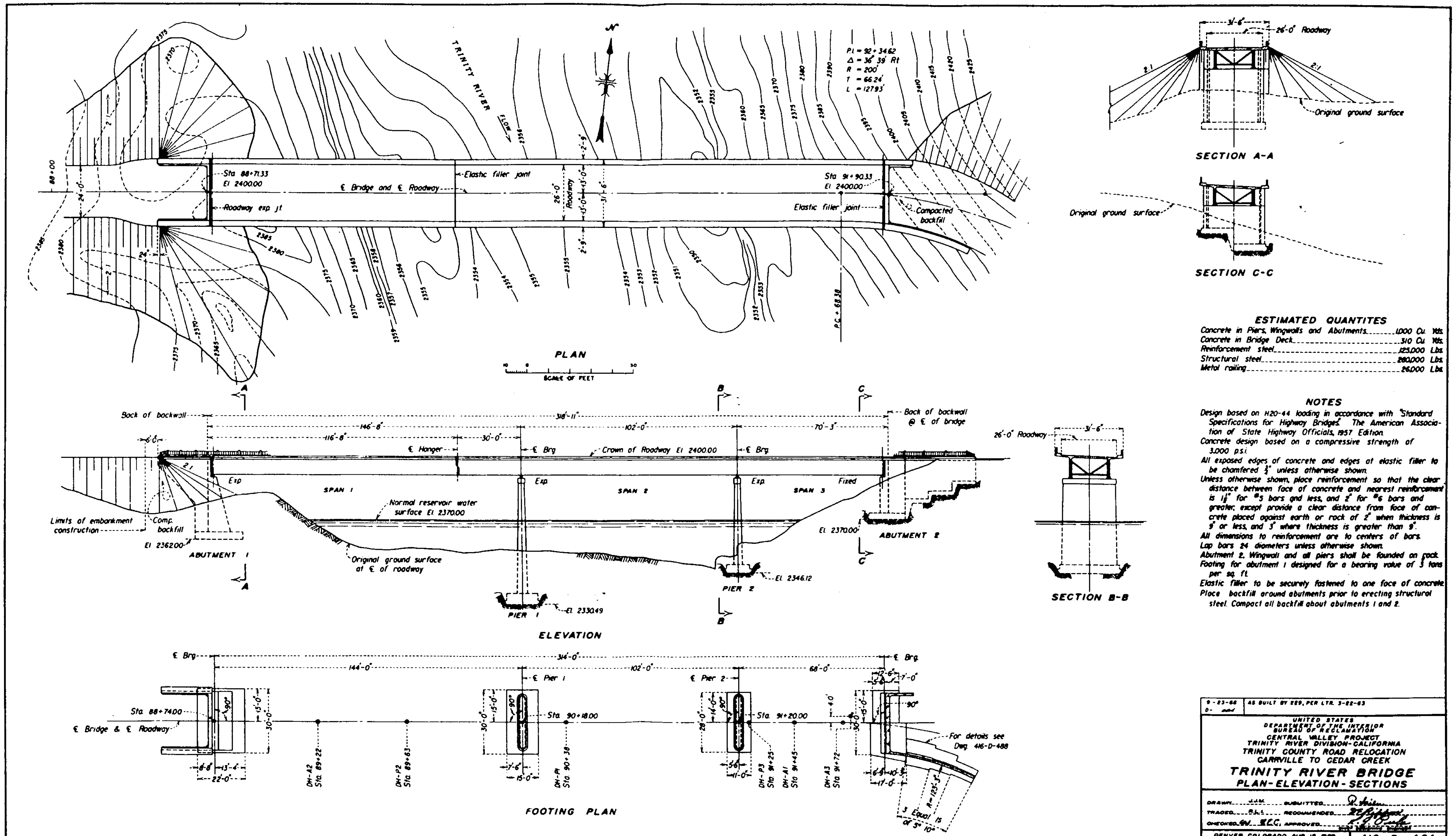
SOLOMON RIVER BRIDGE--PLAN, ELEVATION, SECTIONS--MITCHELL COUNTY HIGHWAY C-705 RELOCATION, KANSAS

Fig. 17 Par. 9.11G



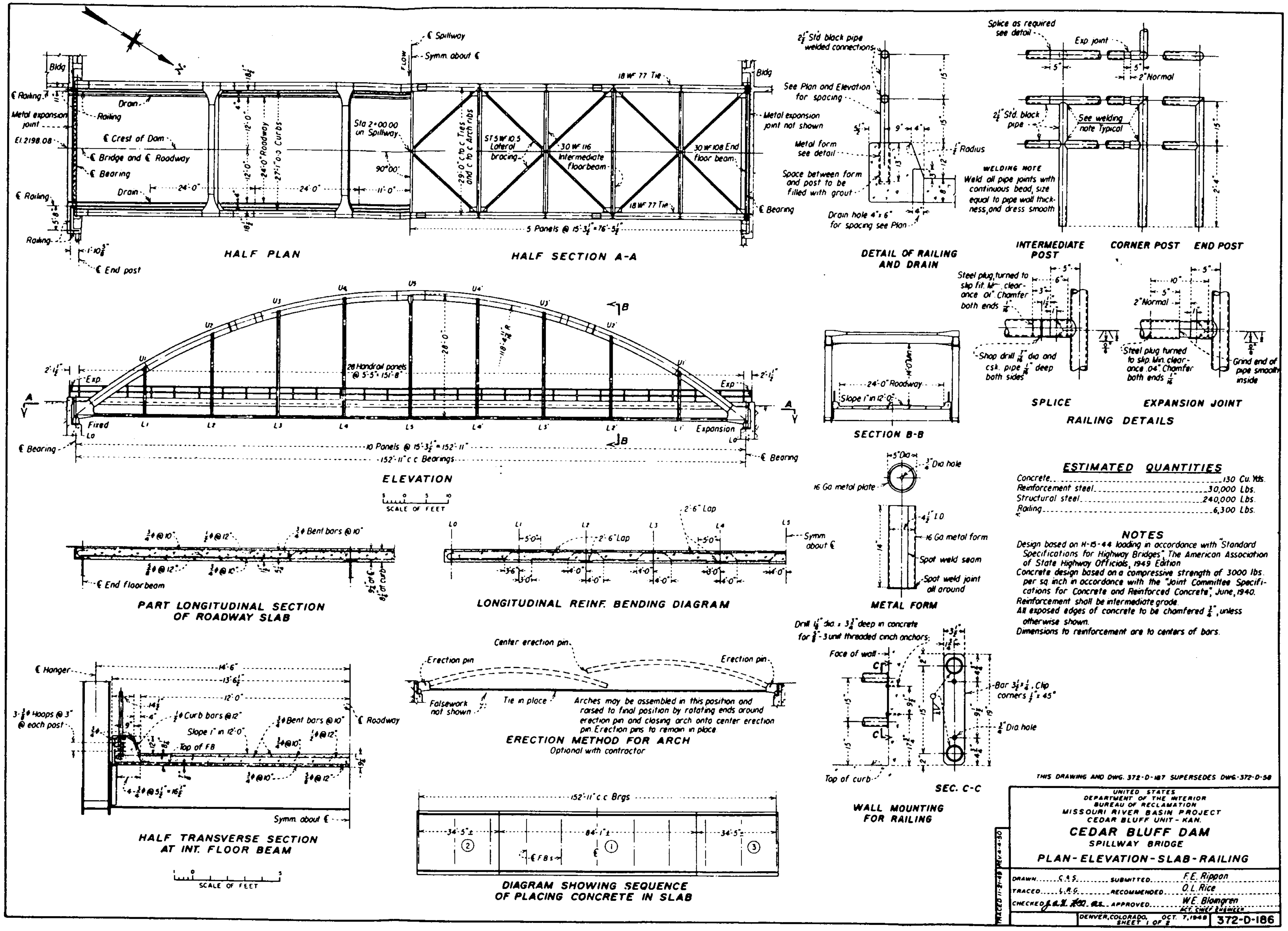
TRINITY RIVER BRIDGE--PLAN, ELEVATION, SECTIONS--TRINITY COUNTY ROAD RELOCATION CARRVILLE TO CEDAR CREEK

Fig. 18 Par. 9.11G



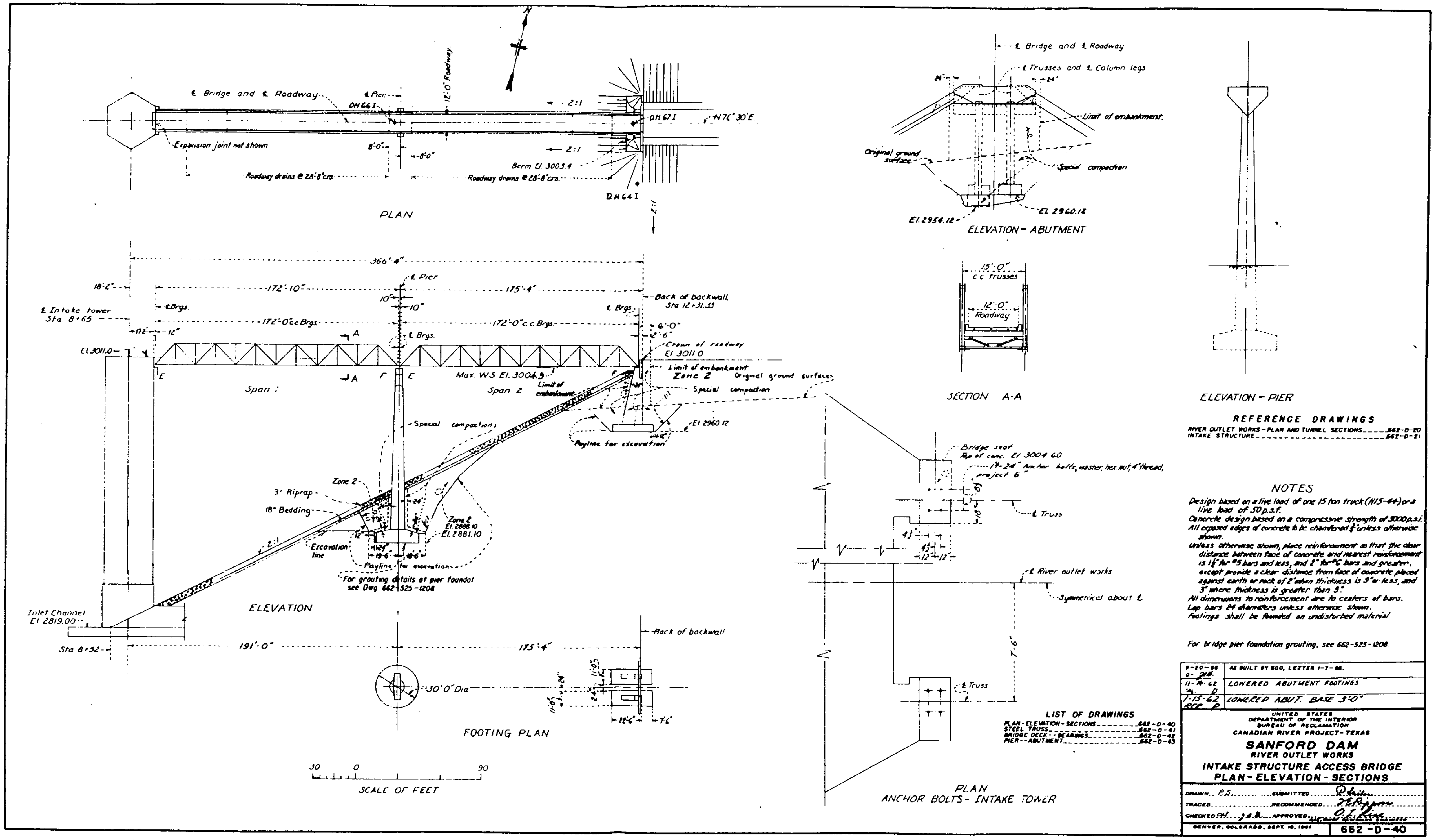
SPILLWAY BRIDGE--PLAN, ELEVATION, SLAB, RAILING--CEDAR BLUFF DAM

Fig. 19 Par. 9.11H



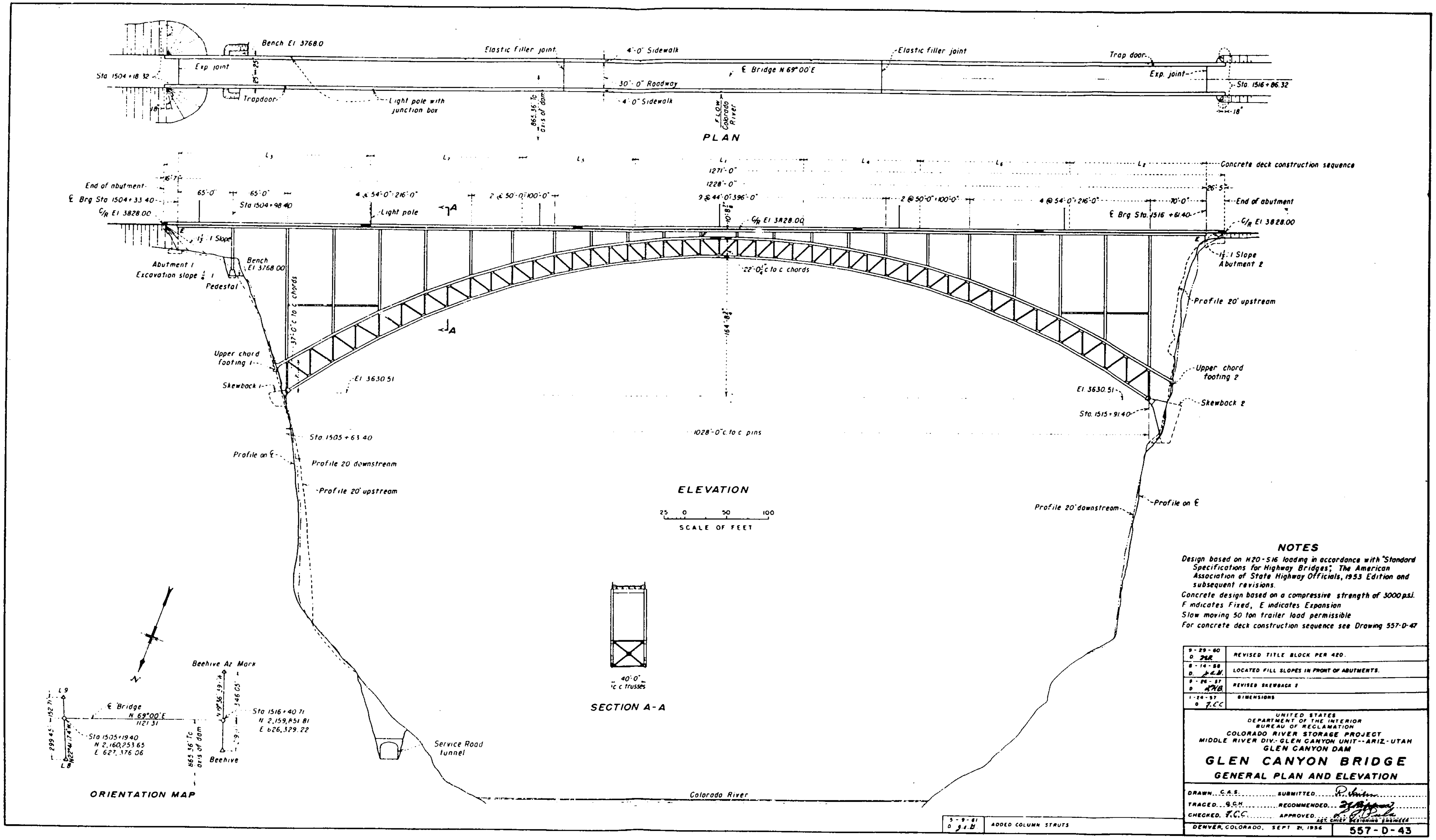
INTAKE STRUCTURE ACCESS BRIDGE--PLAN, ELEVATION, SECTIONS--SANFORD DAM, RIVER OUTLET WORKS

Fig. 20 Par. 9.11H



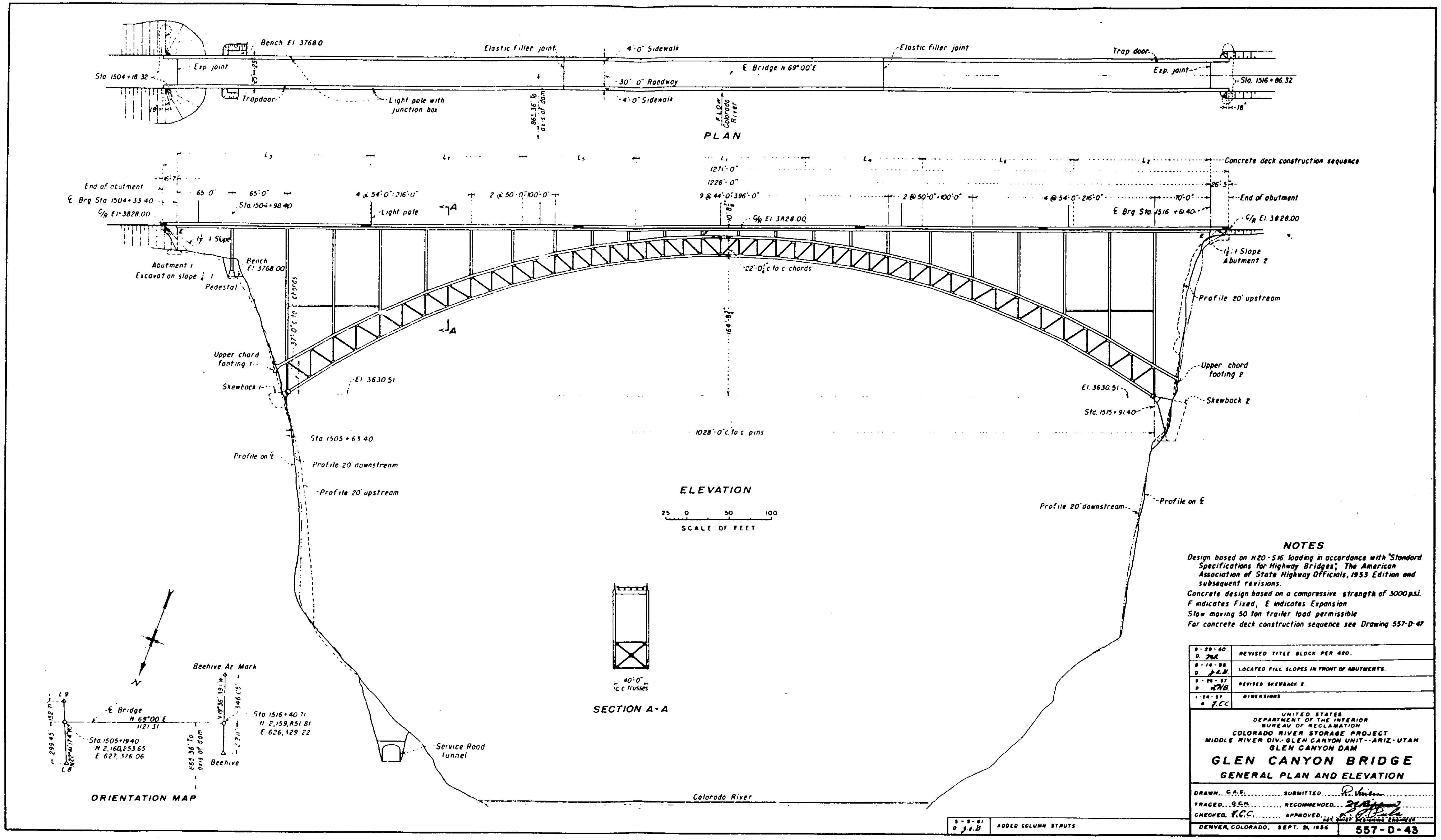
GLEN CANYON BRIDGE--GENERAL PLAN AND ELEVATION--GLEN CANYON DAM

Fig. 21 Par. 9.11H



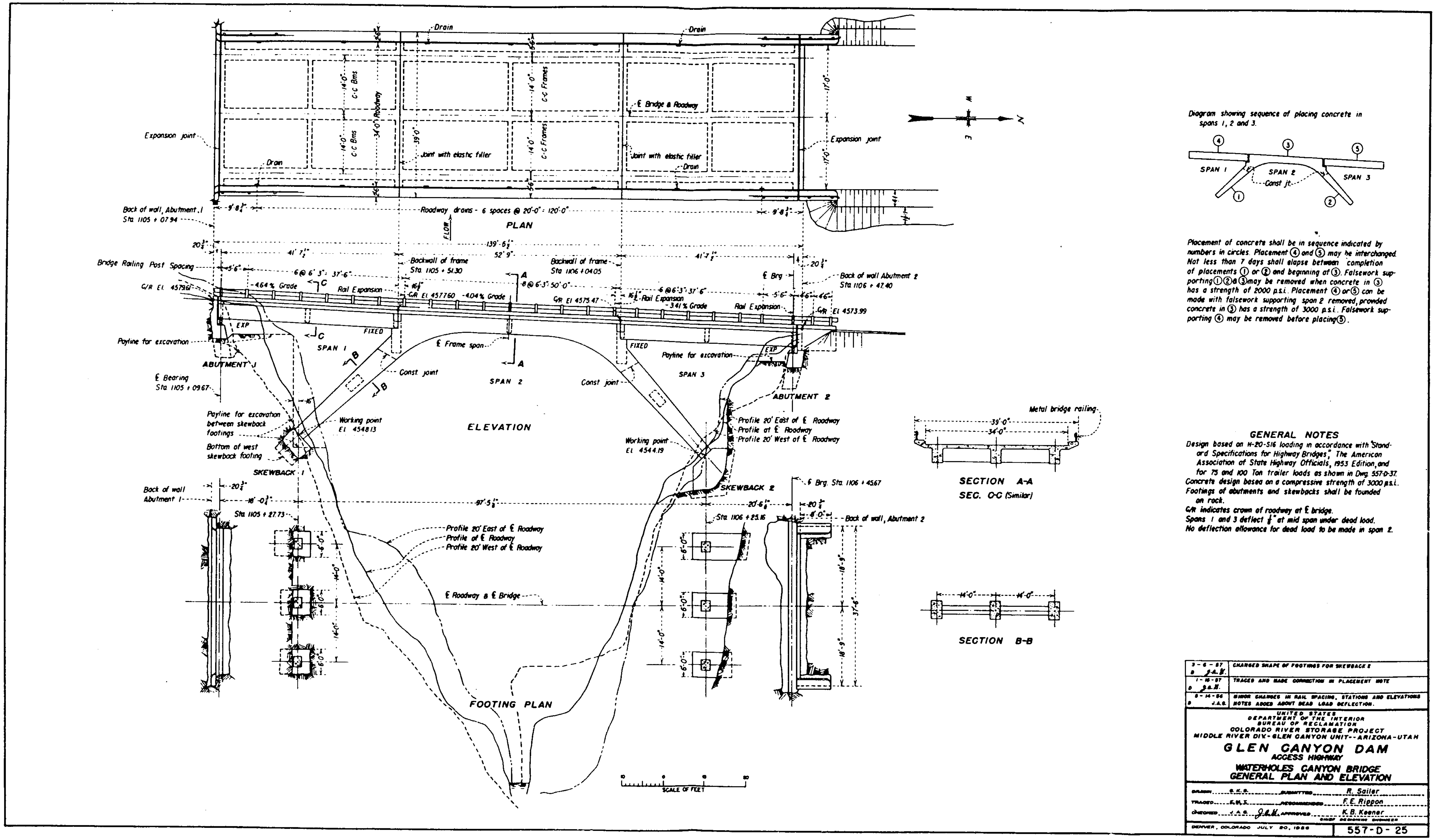
GLEN CANYON BRIDGE--GENERAL PLAN AND ELEVATION--GLEN CANYON DAM

Fig. 21 Par. 9.11H



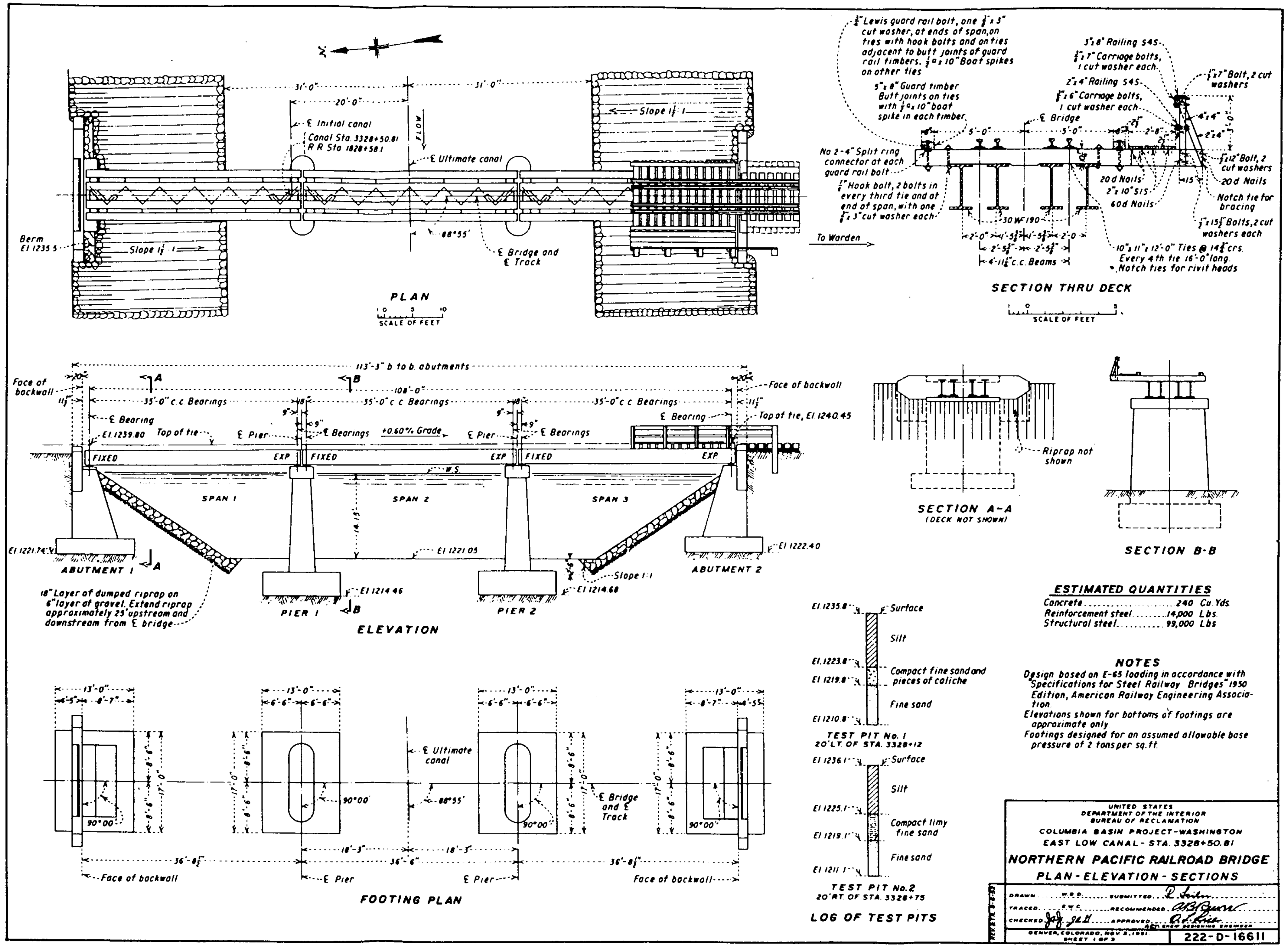
WATERHOLES CANYON BRIDGE--GENERAL PLAN AND ELEVATION--GLEN CANYON DAM ACCESS HIGHWAY

Fig. 22 Par. 9.11J



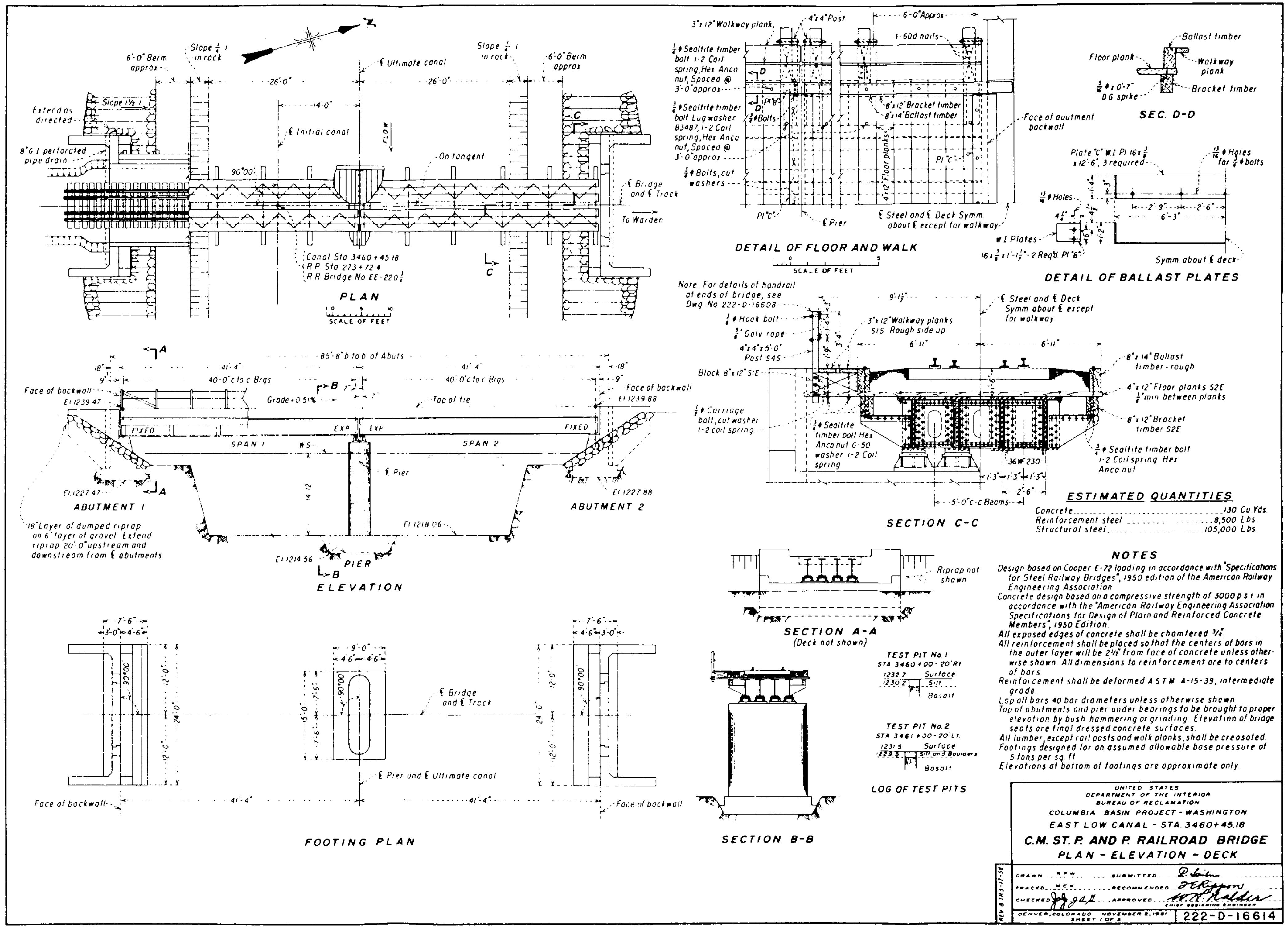
NORTHERN PACIFIC RAILROAD BRIDGE--PLAN, ELEVATION, SECTIONS--EAST LOW CANAL

Fig. 23 Par. 9.13A



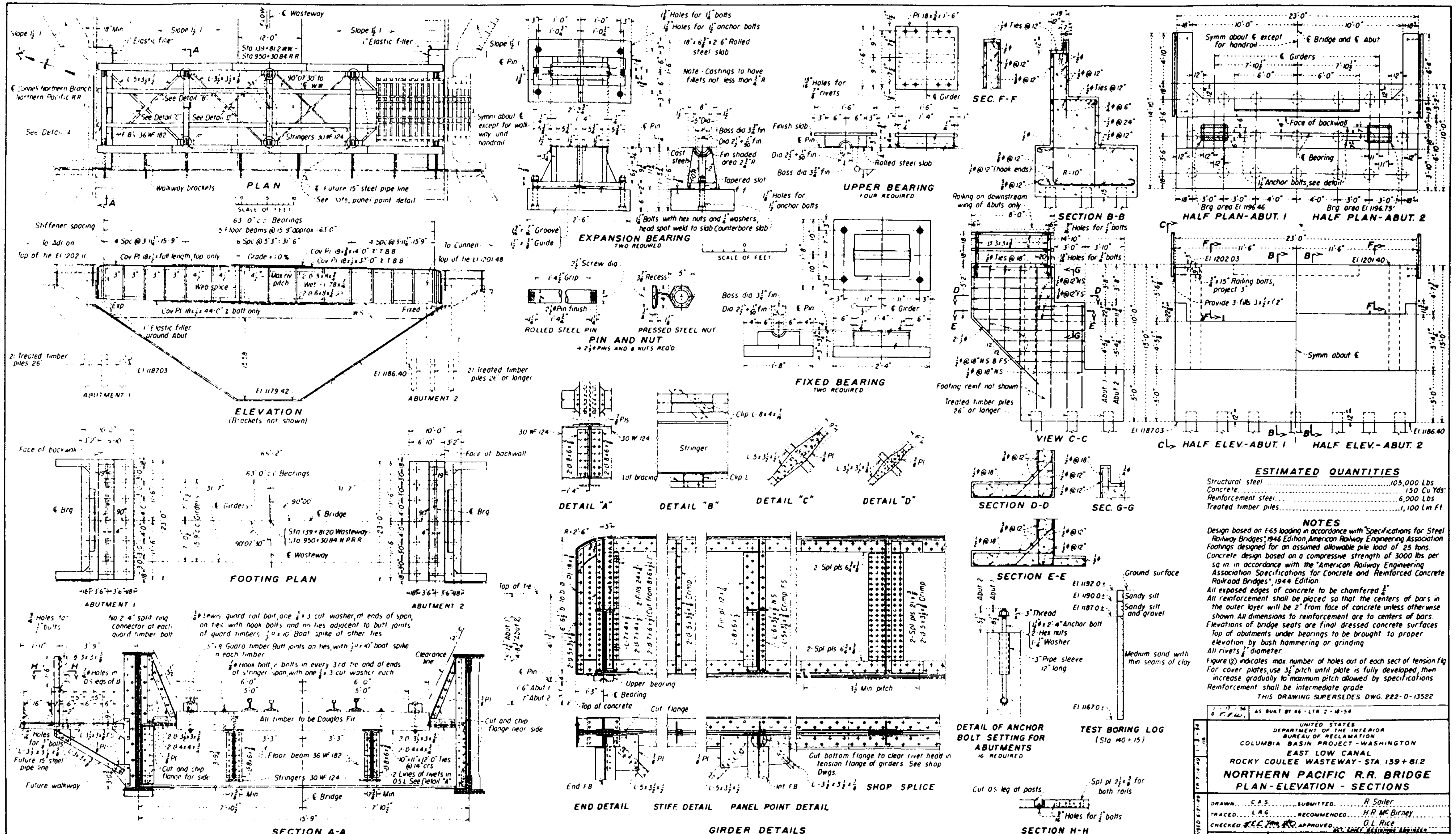
C. M. ST. P. AND P. RAILROAD BRIDGE--PLAN, ELEVATION, DECK--EAST LOW CANAL

Fig. 24 Par. 9.13A



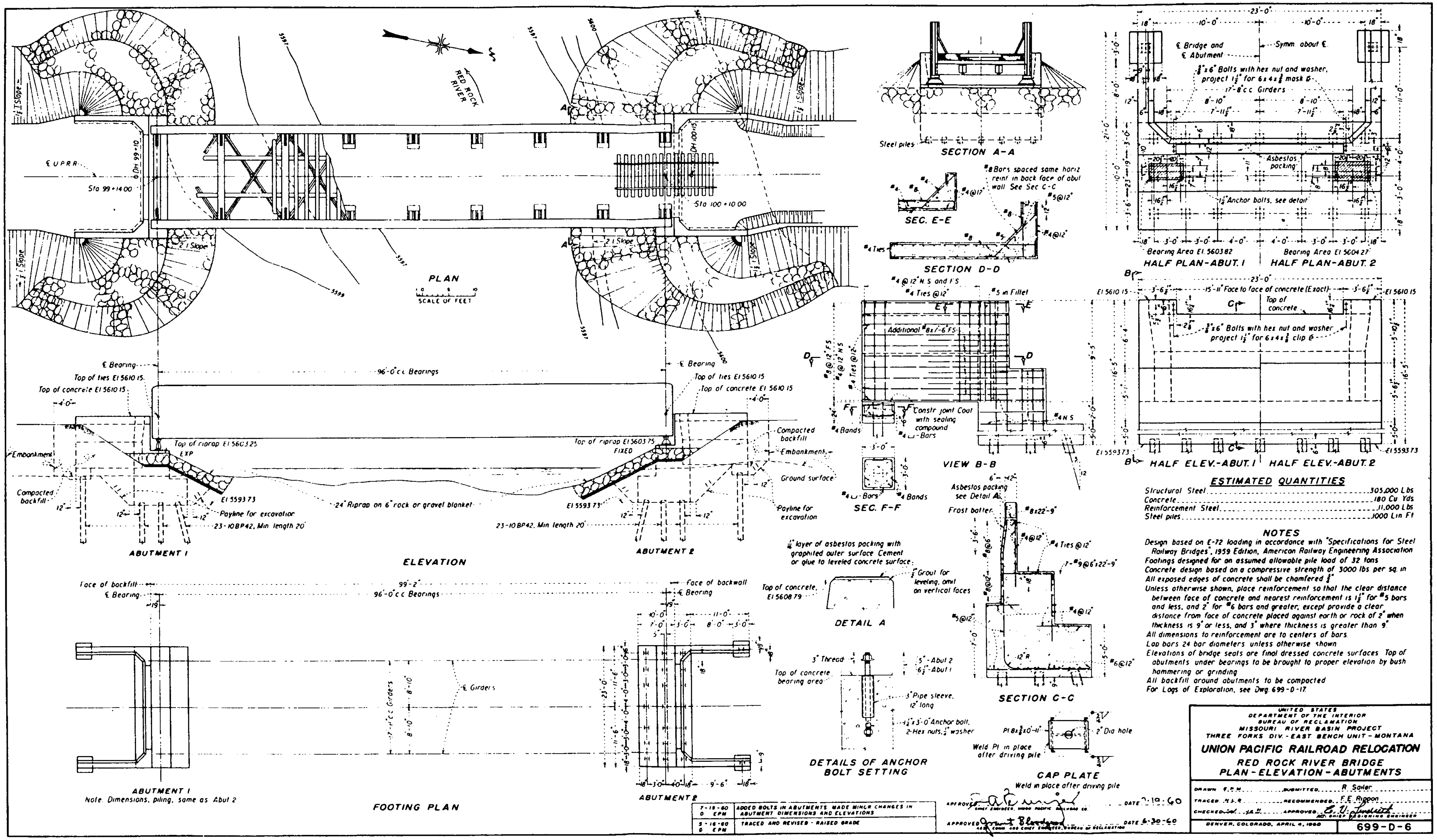
NORTHERN PACIFIC RAILROAD BRIDGE--PLAN, ELEVATION, SECTIONS--EAST LOW CANAL, ROCKY COULEE WASTEWAY

Fig. 25 Par. 9.13B



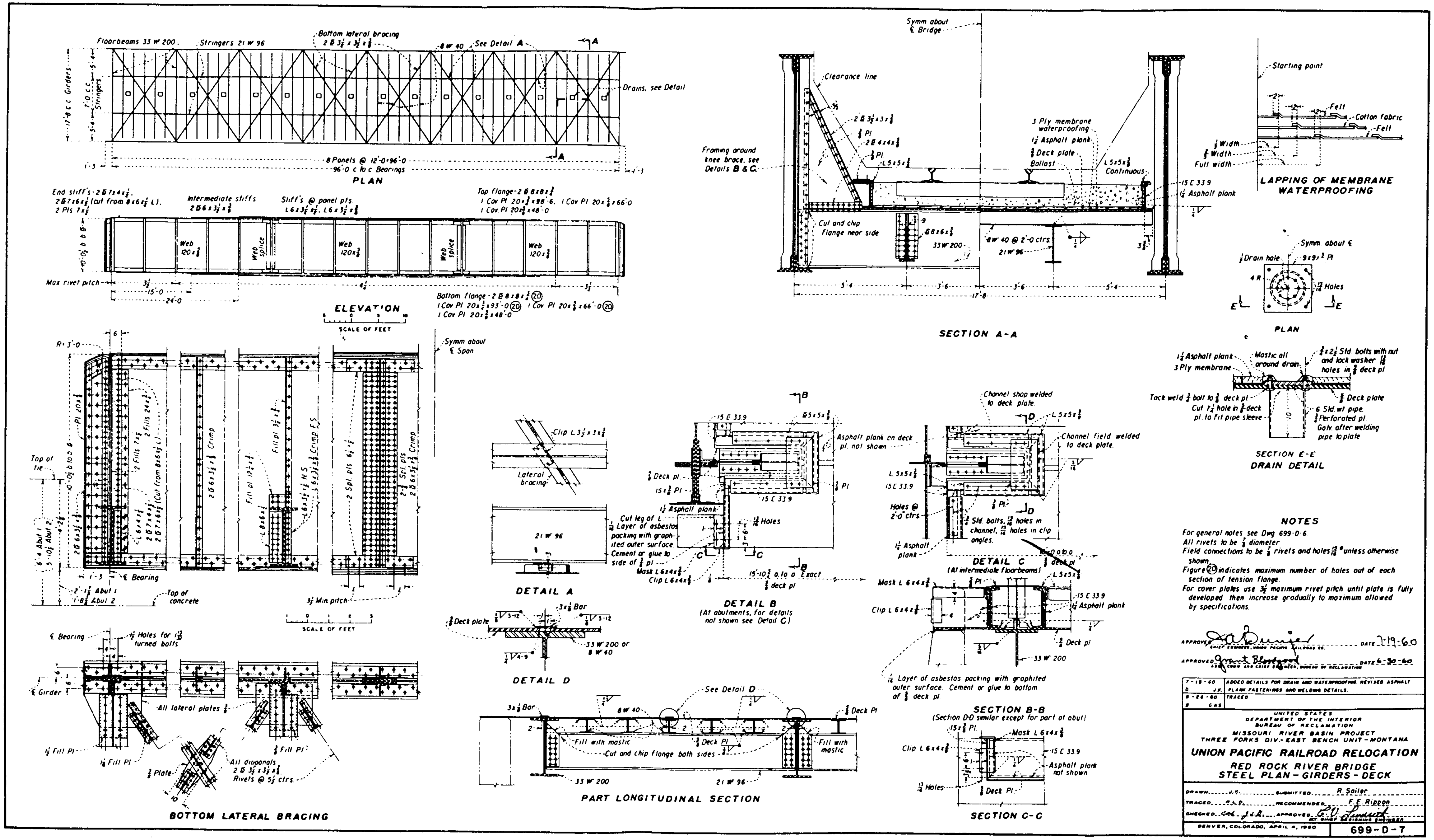
UNION PACIFIC RAILROAD RELOCATION, RED ROCK RIVER BRIDGE--PLAN, ELEVATION, ABUTMENTS--EAST BENCH UNIT

Fig. 26 Par. 9.13B



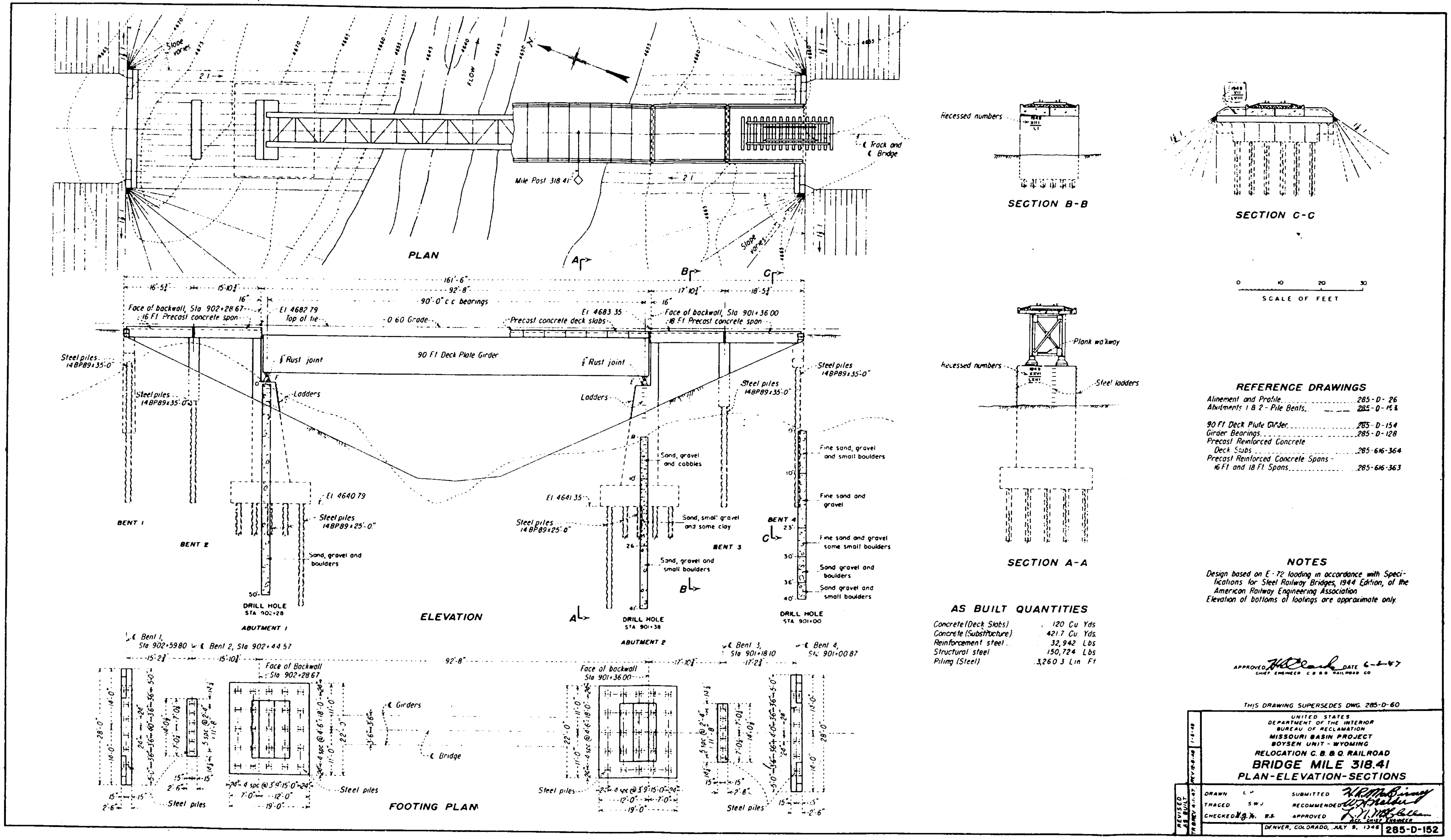
UNION PACIFIC RAILROAD RELOCATION, RED ROCK RIVER BRIDGE--STEEL PLAN, GIRDERS, DECK--EAST BENCH UNIT

Fig. 27 Par. 9.13B



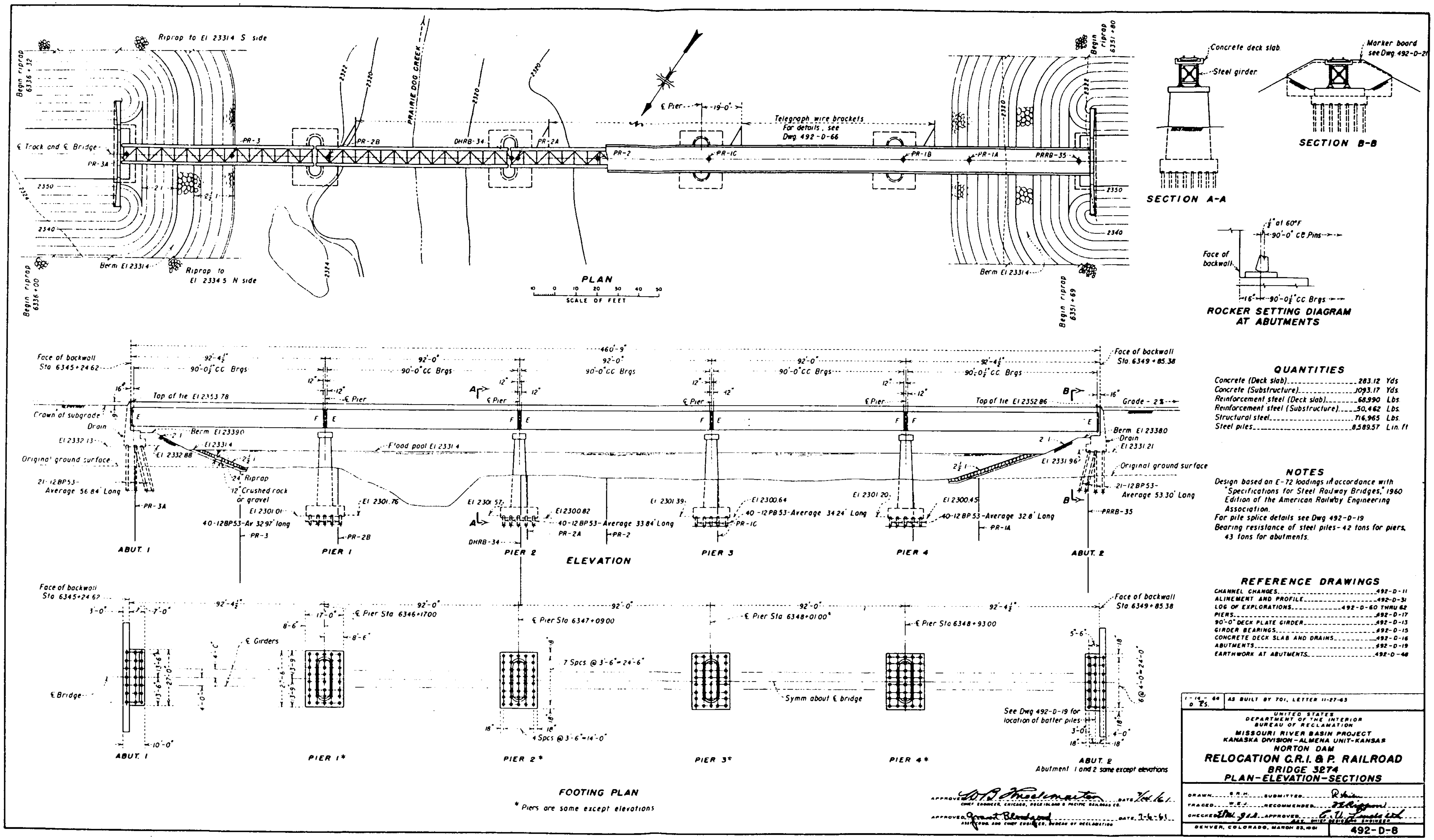
RELOCATION C. B. & Q. RAILROAD, BRIDGE MILE 318.41--PLAN, ELEVATION, SECTIONS--BOYSEN UNIT

Fig. 28 Par. 9.13B



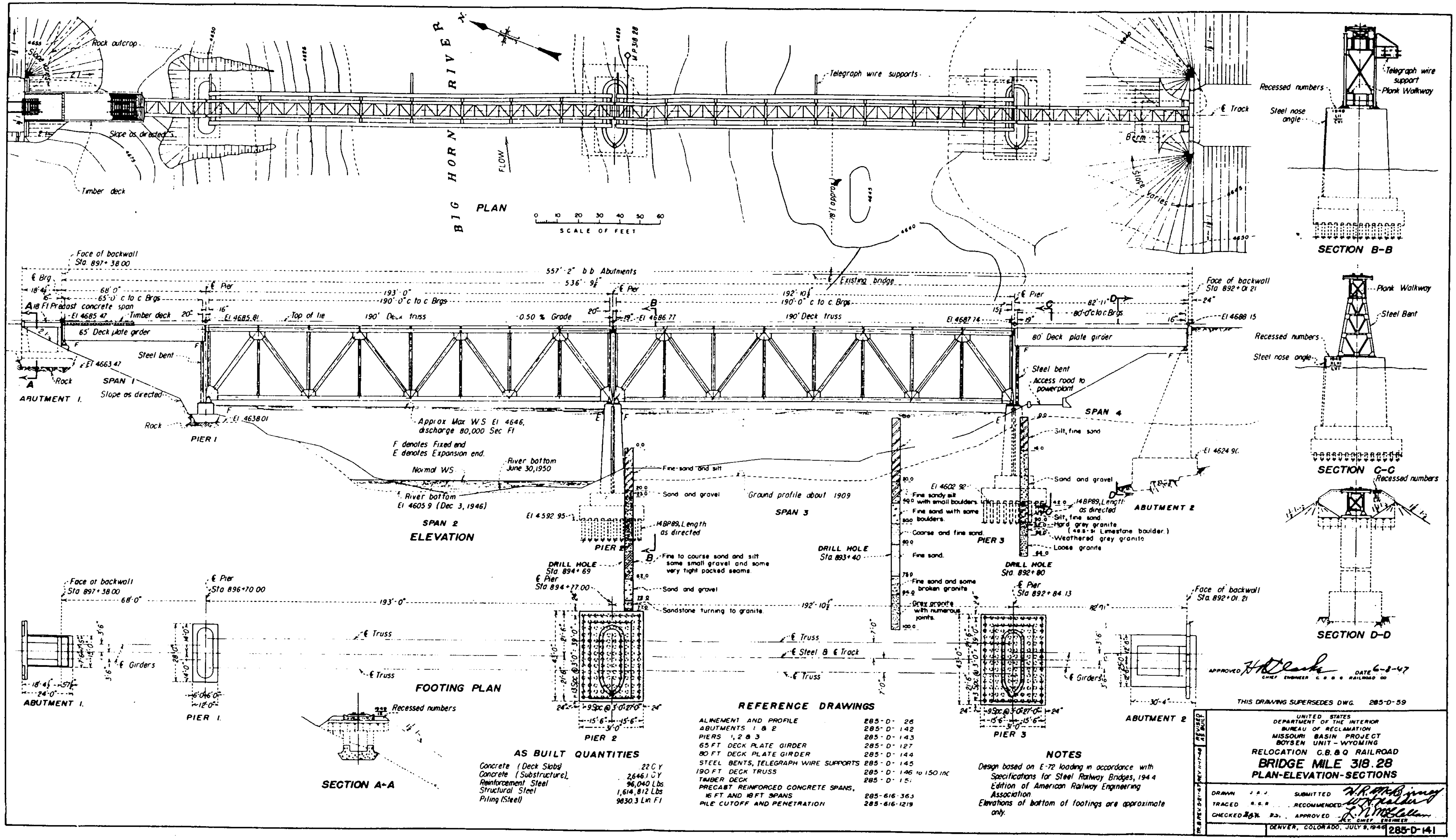
RELOCATION C.R.I. & P. RAILROAD, BRIDGE 3274--PLAN, ELEVATION, SECTIONS--NORTON DAM

Fig. 29 Par. 9.13B



RELOCATION C. B. & Q. RAILROAD, BRIDGE MILE 318.28--PLAN, ELEVATION, SECTIONS--BOYSEN UNIT

Fig. 30 Par. 9.13B



SOUTHERN PACIFIC RAILROAD CROSSINGS FOR CANAL AND EQUALIZING FLOODWAY, SAN LUIS CANAL--PRESTRESSED BOX GIRDERS

Fig. 31 Par. 9.13C