Welded Steel Penstocks

The lower ends of the penstocks at Shasta Dam emerge from concrete anchors and plunge into the powerhouse.

Welded Steel Penstocks

United States Department of the Interior •

BUREAU OF RECLAMATION

Preface

THIS MONOGRAPH will assist designers in the solution of problems in design and construction of safe penstocks which may be fabricated in accordance with modern manufacturing procedures. Certain rules relative to materials, stresses, and tests might be considered unnecessarily conservative. Safety is of paramount importance, however, and penstocks designed and constructed according to these rules have given satisfactory service through years of operation.

Welded Steel Penstocks presents information concerning modern design and construction methods for pressure vessels applied to penstocks for hydroelectric powerplants. The data are based on some 40 years experience in penstock construction by the Bureau of Reclamation. During this period many of the largest penstocks in service today were designed and constructed.

Welded Steel Penstocks was first issued in 1949 under the authorship of P. J. Bier. Because of the continuing interest in penstock design, the monograph has been revised and updated to incorporate present day practice.

This edition now represents the contributions of many individuals in the Penstocks and Steel Pipe Section, Mechanical Branch, Division of Design, on the staff of the Chief Engineer, Denver, Colo.

This monograph is issued to assist designers in the solution of problems involved in the design and construction of safe and economical welded steel penstocks.

Because of the many requests for information concerning Bureau of Reclamation designed and built penstocks, a comprehensive bibliography has been added in the back.

Included in this publication is an informative abstract and list of descriptors, or keywords, and "identifiers." The abstract was prepared as part of the Bureau of Reclamation's program of indexing and retrieving the literature of water resources development. The descriptors were selected from the *Thesaurus of Descriptors*, which is the Bureau's standard for listings of keywords.

Other recently published Water Resources Technical Publications are listed on the inside back cover of this monograph.

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Report on Welded Steel Penstocks

Introduction

A penstock is the pressure conduit between the turbine scrollcase and the first open water upstream from the turbine. The open water can be a surge tank, river, canal, free-flow tunnel, or a reservoir. Penstocks should be as hydraulically efficient as practical to conserve available head, and structurally safe to prevent failure which would result in loss of life and property. Penstocks can be fabricated of many materials, but the strength and flexibility of steel make it best suited for the range of pressure fluctuations met in turbine operation.

The design and construction of pressure vessels, such as penstocks, are governed by appropriate codes which prescribe safe rules and practices to be followed. Until a special penstock code is formulated, steel penstocks should be constructed in accordance with the ASME Boiler and Pressure Vessel Code, Section VIII, Unfired Pressure Vessels, issued by the American Society of Mechanical Engineers, hereinafter referred to as the ASME code. This code is subject to periodic revision to keep it abreast of new developments in the design, materials, construction, and inspection of pressure vessels.

Present design standards and construction practices were developed gradually, following the advent of welded construction, and are the result of improvements in the manufacture of welding-quality steels, in welding processes and procedures, and in inspection and testing of welds.

LOCATION AND ARRANGEMENT

The location and arrangement of penstocks will be determined by the type of dam, location of intake and outlet works, relative location of dam and powerplant, and method of river diversion used during construction. At dams requiring tunnels for diversion of the river flow during construction, the penstocks may be placed in the tunnels after diversion has been discontinued and the intake of the tunnel has been plugged. This arrangement was used for the 30-foot lower Arizona and Nevada penstock and outlet headers at Hoover Dam as shown in figure 1, and for the 15-foot penstock header at Anderson Ranch Dam as shown in figure 2.

1
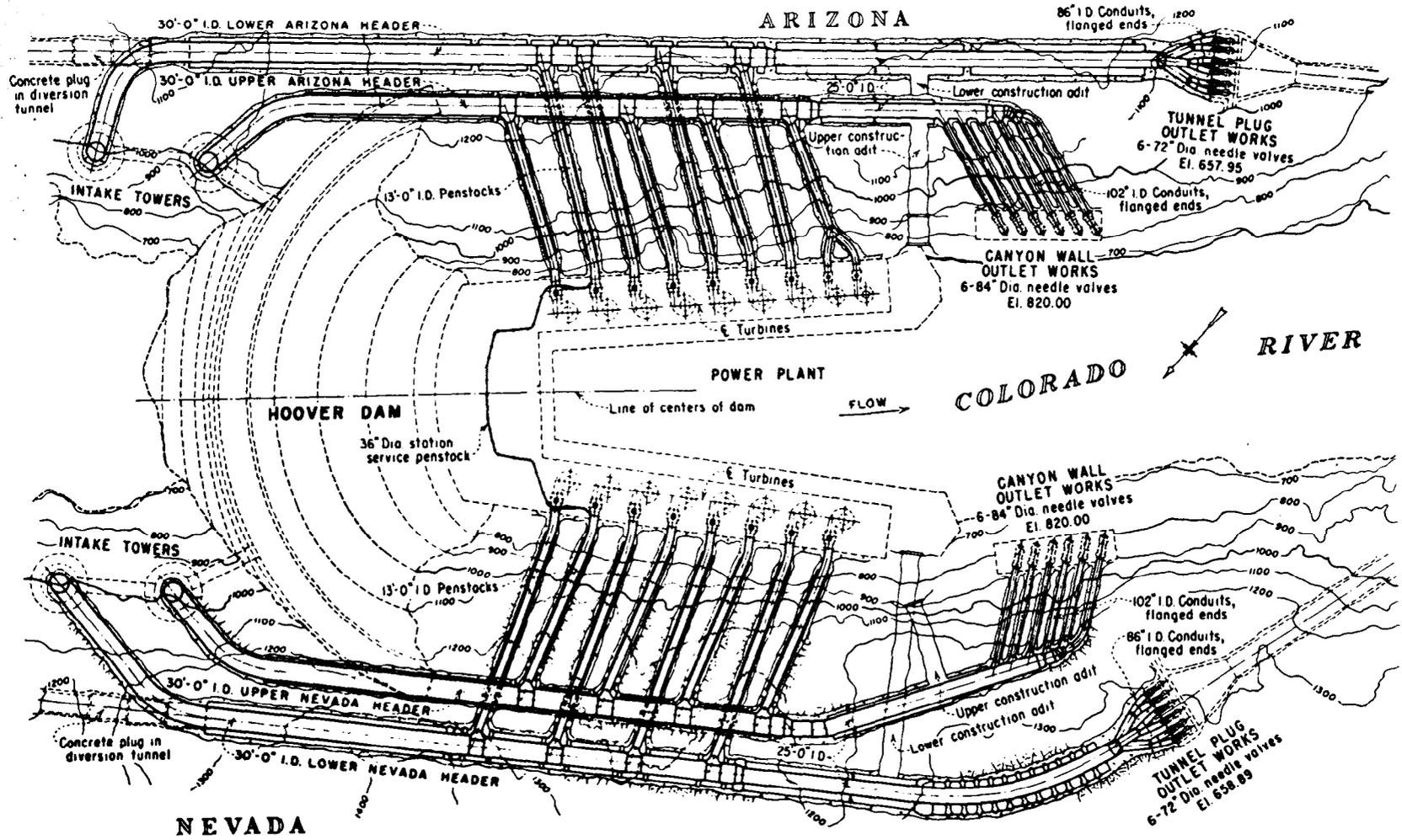
FIGURE 1.—*The 30-foot-diameter lower Arizona and Nevada penstock and outlet headers were installed in two of the diversion tunnels at Hoover Dam. The upper headers were installed in special penstock tunnels. The tunnels were not backfilled with the exception of the inclined portions leading from the intake towers.*

Outlet pipe No. 5
Outlet pipe No. 1
Penstock No. 3
Penstock No. 2
Penstock No. 1
SECTION
P.I. Horizontal bend
Sta. 19+06.03 - El. 3870

PLAN
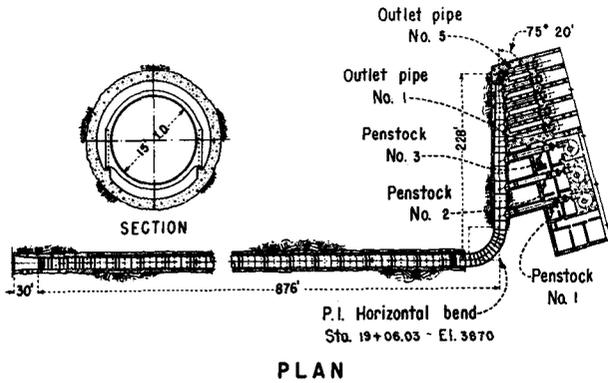
FIGURE 2.—*At Anderson Ranch Dam, the 15-foot-diameter penstock and outlet header were installed in the diversion tunnel, which was not backfilled.*

For low-head concrete dams, penstocks may be formed in the concrete of the dam. However, a steel lining is desirable to assure watertightness. In large concrete dams which have both transverse and longitudinal contraction joints, such as Glen Canyon Dam, steel penstocks are used to provide the required watertightness in the concrete and at the contraction joints. Figure 4 shows the 9-foot penstocks which are embedded in Kortes Dam.

Penstocks embedded in concrete dams, encased in concrete, or installed in tunnels backfilled with concrete may be designed to transmit some of the radial thrust due to internal water pressure to the surrounding concrete. More generally, such penstocks are designed to withstand the full internal pressure. In either case, the shell should be of sufficient thickness to provide the rigidity required during fabrication and handling, and to serve as a form for the concrete. Embedded or buried penstock shells also should be provided with adequate stiffeners or otherwise designed to withstand any anticipated external hydrostatic or grouting pressures. At Shasta Dam the upstream portions of the 15-foot penstocks are embedded in the dam, while the downstream portions are

PLAN   TYPICAL PENSTOCK PROFILE
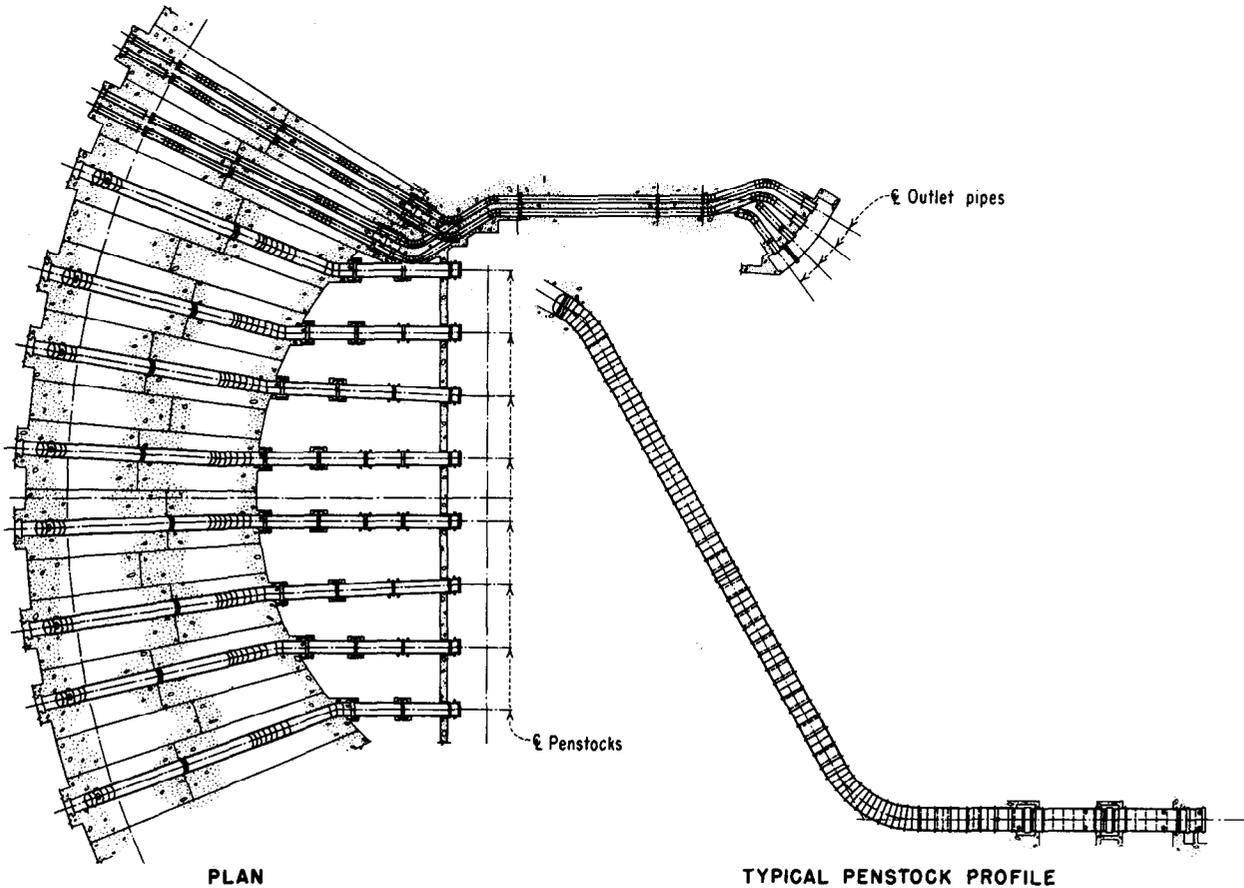
FIGURE 3.—*General plan and typical profile of 15'-0"-diameter penstocks at Glen Canyon Dam.*
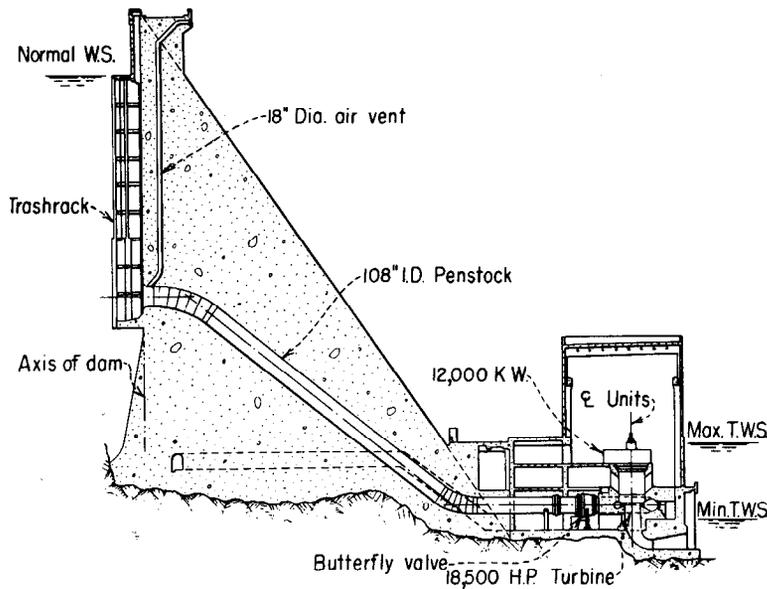
FIGURE 4.—*Mass concrete of Kortes Dam encased the 9-foot-diameter penstocks, which were installed as the concrete was placed.*
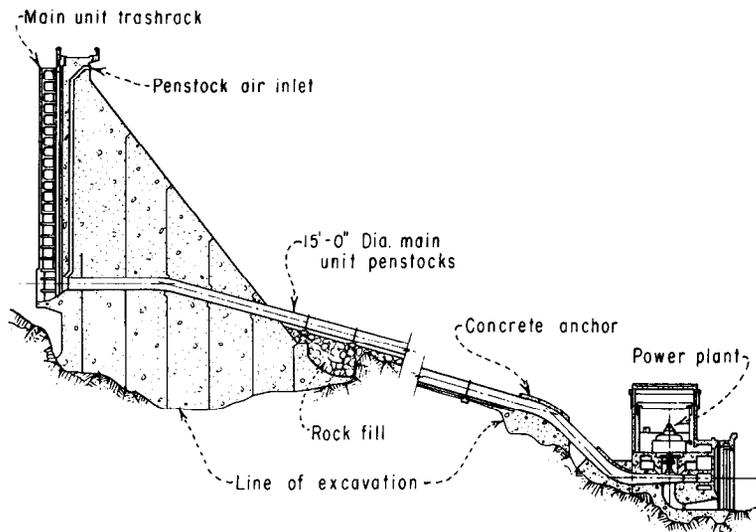
FIGURE 5.—*The 15-foot-diameter penstocks at Shasta Dam were embedded in the concrete of the dam at the upstream ends and were exposed above ground between dam and powerplant.*

exposed above ground, between the dam and the powerplant, as shown in figure 5. At other plants, the entire length of the penstock may be situated above ground, as in figure 6, which shows the 8-foot-diameter penstock at Marys Lake Powerplant.

When a powerplant has two or more turbines the question arises whether to use an individual penstock for each turbine or a single penstock with a header system to serve all units.

Considering only the economics of the penstock, the single penstock with a header system will usually be preferable; however, the cost of this item alone should not dictate the design. Flexibility of operation should be given consideration because with a single penstock system the inspection or repair of the penstock will require shutting down the entire plant. A single penstock with a header system requires complicated branch connections and a valve
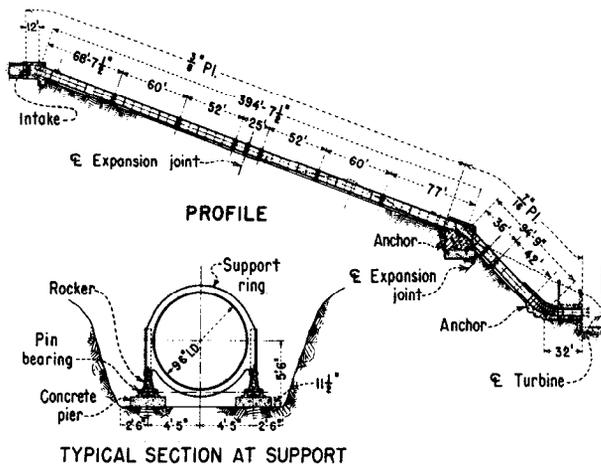
FIGURE 6.—*The full length of the 8-foot-diameter penstock at Marys Lake Powerplant lies above ground.*

to isolate each turbine. Also, the trashracks and bulkhead gates will be larger, resulting in heavier handling equipment. In concrete dams it is desirable to have all openings as small as possible. The decision as to the penstock arrangement must be made considering all factors of operation, design, and overall cost of the entire installation.

Proper location of the penstock intake is important. In most cases the intake is located at the upstream face of the dam, which provides short penstocks and facilitates operation of the intake gates. In some cases the penstock intake may be situated in an independent structure located in the reservoir, as at Hoover and Green Mountain Dams, where diversion tunnels or topographic conditions influenced the arrangement. Regardless of arrangement, the intake should be placed at an elevation sufficiently below low reservoir level and above the anticipated silt level to allow an uninterrupted flow of water under all conditions. Each intake opening is protected against floating matter by means of a trashrack structure and is controlled by suitable gates.

To prevent the development of a partial vacuum during certain operating conditions, penstock profiles from intake to turbine should, whenever possible, be laid on a continuous slope.

ECONOMIC STUDIES

A penstock is designed to carry water to a turbine with the least possible loss of head consistent with the overall economy of installation. An economic study will size a penstock from a monetary standpoint, but the final diameter should be determined from combined engineering and monetary considerations. An example would be an installation where the economic diameter would require the use of a surge tank for regulation, but a more economical overall installation might be obtained by using a penstock considerably larger than the economic diameter, resulting in the elimination of the surge tank.

Voetsch and Fresen (1)[1] present a method of determining the economic diameter of a penstock. Figure 7 was derived from their method, and figure 8 is an example of its use. Doolittle (2) presents a method for determining the economic diameters of long penstocks where it is economical to construct a penstock of varying diameters. This "step by step" method requires considerable time but should be considered for final design for long penstocks.

All the variables used in an economic study must be obtained from the most reliable source available, keeping in mind that an attempt is being made to predict the average values of all variables for the life of the project. Special attention must be given to the "plant factor", figure 7, as this item materially affects the calculations.

HEAD LOSSES IN PENSTOCKS

Hydraulic losses in a penstock reduce the effective head in proportion to the length of the penstock and approximately as the square of the water velocity. Accurate determination of these losses is not possible, but estimates can be made on the basis of data obtained from pipe flow tests in laboratories and full-scale installations.

[1] Numbers in parentheses refer to literature cited in section, "A Selected Bibliography and References", at the end of this monograph.

"K" = $\dfrac{K_s \, e \, f \, S_g \, e \, j \, b}{a \, r \, (1+n)}$ For penstocks

"K" = $\dfrac{K_s \, f \, S_g \, e \, j \, b}{a \, e \, r \, (1+n)}$ For pump lines

GRAPH B

VALUES OF "K"

DIAMETER MULTIPLIER "B"

GRAPH A

LOSS FACTOR f

PLANT FACTOR F

$\text{PLANT FACTOR } F = \dfrac{\text{AVERAGE ANNUAL LOAD}}{\text{RATED ANNUAL CAPACITY}}$

VALUES OF D'

DIAMETER MULTIPLIER "B"

VALUES OF D'

FLOW Q IN C.F.S. AT DESIGN HEAD
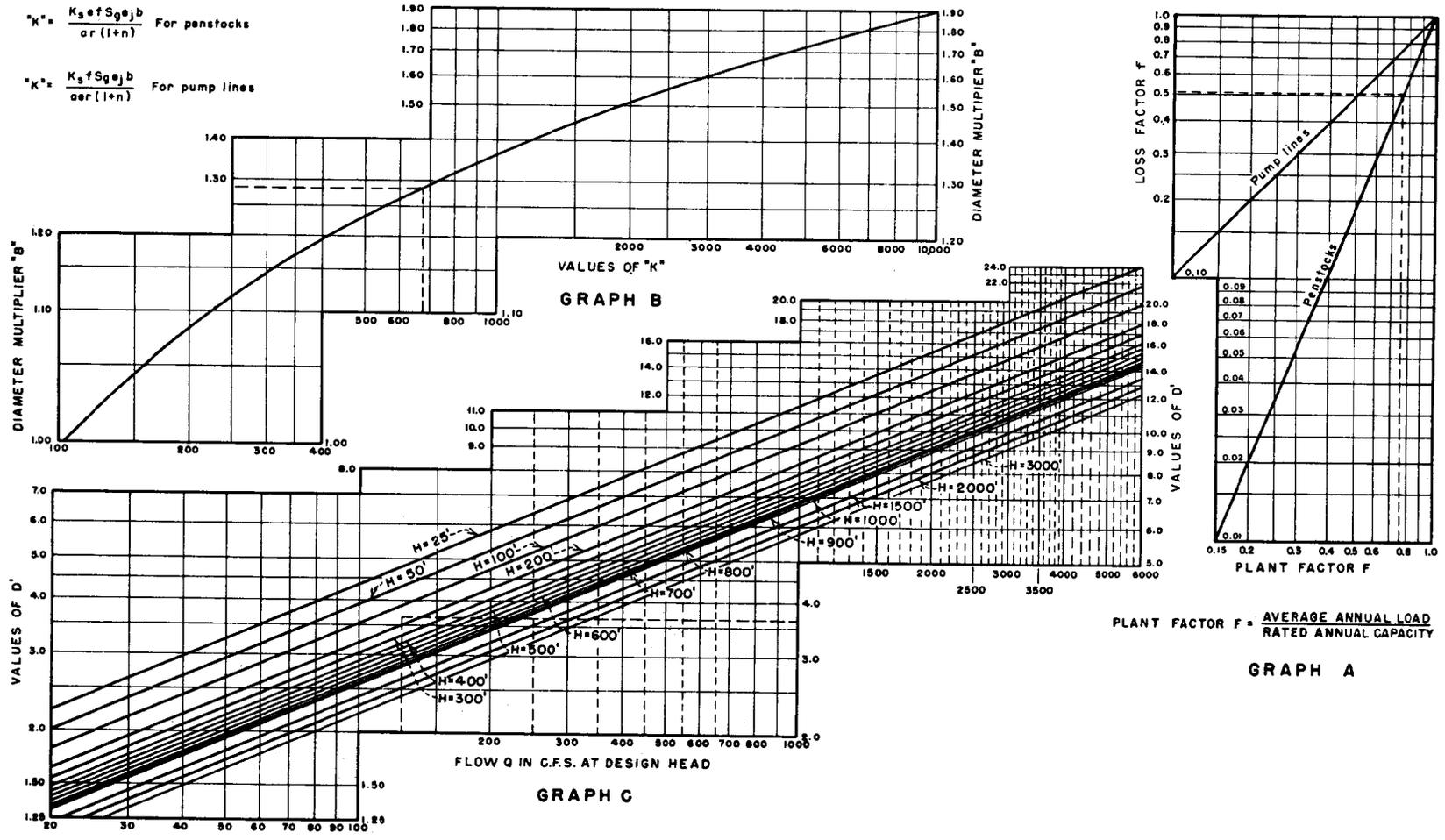
GRAPH C

FIGURE 7.—*Economic diameter of steel penstocks when plate thickness is a function of the head.*

NOTATION

a = Cost of pipe per lb., installed, dollars.
B = Diameter multiplier from Graph B.
b = Value of lost power in dollars per kwh.
D = Economic diameter in feet.
e = Overall plant efficiency.
e_j = Joint efficiency.
f = Loss factor from Graph A.

H = Weighted average head including water hammer. (based on design head)
K_s = Friction coefficient in Scobey's formula (0.34).
n = Ratio of overweight to weight of pipe shell.
Q = Flow in cubic feet per second. (at design head of turbine)
r = Ratio of annual cost to "a"(see explanation)
s_g = Allowable tension, p.s.i.
t = Weighted average plate thickness(at design head) for total length.
t_n = Average plate thickness for length L_n.

EXPLANATION AND EXAMPLE
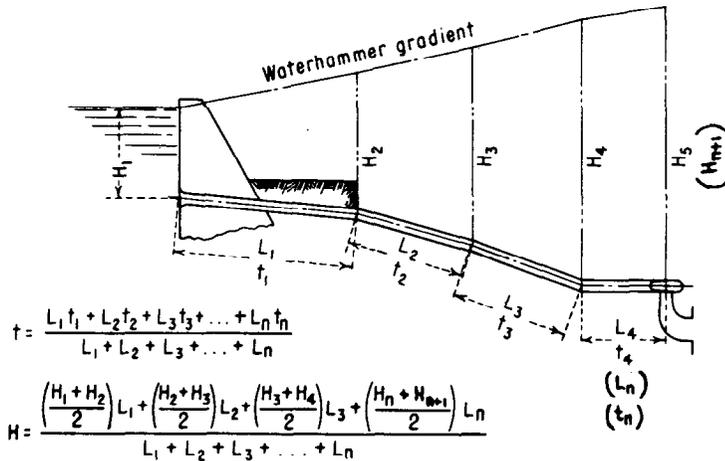
$$t = \frac{L_1 t_1 + L_2 t_2 + L_3 t_3 + \ldots + L_n t_n}{L_1 + L_2 + L_3 + \ldots + L_n}$$

$$H = \frac{\left(\frac{H_1+H_2}{2}\right)L_1 + \left(\frac{H_2+H_3}{2}\right)L_2 + \left(\frac{H_3+H_4}{2}\right)L_3 + \left(\frac{H_n+H_{n+1}}{2}\right)L_n}{L_1 + L_2 + L_3 + \ldots + L_n}$$

$$\% \, 0.\&M. = \frac{0.\&M. \, per \, foot}{Cost \, per \, foot \, of \, pipe}$$

0.&M. = Cost of maintaining interior and exterior surface area of pipe(inside surface area for embedded pipe, inside and outside surface area for exposed pipe) per year.

Depreciation = See Reclamation Manual, Vol. ⅤI Power, page 2,4,110.

r = Interest + Depreciation + % 0.&M.

Example for penstock

ASSUME

Dia. = 5'-0"   Avg. plate = $\frac{1}{4}$"
Value of power per kwh = $0.005 = b
Cost of steel pipe installed = $0.27/lb. = a
Plant factor (see Graph A)=0.75(f=0.510)
Interest = 3%   n = 0.15
Depreciation = 0.005135
0.&M. = $0.02 per sq. ft.

r = 0.03 + 0.005135 + 0.0119 = 0.0470

Q = 128 CFS

$L_1 = 225$; $L_2 = 150$; $L_3 = 150$; $L_4 = 100'$

$H_1 = 80'$; $H_2 = 120'$; $H_3 = 170$; $H_4 = 230'$; $H_5 = 240'$

$$H = \frac{\left(\frac{H_1+H_2}{2}\right)L_1 + \left(\frac{H_2+H_3}{2}\right)L_2 + \left(\frac{H_3+H_4}{2}\right)L_3 + \left(\frac{H_4+H_5}{2}\right)L_4}{L_1 + L_2 + L_3 + L_4} = 156.4'$$

$$0.\&M. = \frac{[(225 \times 15.71) + (400 \times 15.71 \times 2)]0.02}{625} = \$0.515$$

Weight / foot = 15.71 x 10.2 = 160.24 lbs.
Cost / foot = 160.24 x 0.27 = $43.26
% 0.&M. = $\frac{0.515}{43.26}$ = 0.0119

$$K = \frac{K_s \, ef \, s_g \, e_j{}^b}{ar(1+n)} = \frac{0.34 \times 0.85 \times 0.510 \times 15,000 \times 0.90 \times 0.005}{0.27 \times 0.0470 \times 1.15} = 680$$

B = 1.285 (from Graph B); D'= 3.7(from Graph C); Economic dia. = 1.285 x 3.7= 4.75 (use 4'-10" dia.)

NOTE: Calculated economic diameter should be very close to assumed diameter as it is in this example. The problem should be reworked until this condition exists. Depreciation is based on the accumulation of an annual sinking fund earning 3% interest required to replace 50% of the pipe in 45 years. The annual payment required is equal to 0.005135 times the first cost.

FIGURE 8.—*Economic diameter of steel penstocks when plate thickness is a function of the head.*

The various head losses which occur between reservoir and turbine are as follows:

1. Trashrack losses
2. Entrance losses
3. Losses due to pipe friction
4. Bend losses
5. Losses in valve and fittings.

Losses through trashracks at the intake vary according to the velocity of flow and may be taken as 0.1, 0.3, and 0.5 foot, respectively, for velocities of 1.0, 1.5, and 2.0 feet per second.

The magnitude of entrance losses depends upon the shape of the intake opening. A circular bellmouth entrance is considered to be the most efficient form of intake if its shape is properly proportioned. It may be formed in the concrete with or without a metal lining at the entrance. The most desirable entrance curve was determined experimentally from the shape formed by the contraction of a jet (vena contracta) flowing through a sharp-edged orifice. For a circular orifice, maximum contraction occurs at a distance of approximately one-half the diameter of the orifice. Losses in circular bellmouth entrances are estimated to be 0.05 to 0.1 of the velocity head. For square bellmouth entrances, the losses are estimated to be 0.2 of the velocity head.

Head losses in pipes because of friction vary considerably, depending upon velocity of flow, viscosity of the fluid, and condition of the inside surface of the pipe. Among the conventional pipe flow formulae used for the computation of head losses, the Scobey, Manning, and Hazen-Williams formulae are the most popular. For large steel pipe the Scobey formula is favored; for concrete pipe, the Manning formula; and for cast-iron pipe in waterworks, the Hazen-Williams formula.

The Scobey formula(3), derived from experiments on numerous steel pipe installations, is expressed as follows:

$$H_F = K_s \frac{V^{1.9}}{D^{1.1}} \dots\dots\dots\dots\dots\dots \quad (1)$$

in which

H_F = head loss due to friction in feet per 1,000 feet of pipe

K_s = loss coefficient, determined experimentally

V = velocity of flow in feet per second

D = diameter of pipe in feet.

Values for K_s vary for different types of pipe. For new continuous interior pipe unmarred by projections on the inside (as for butt-welded pipe) a value of 0.32 may be used. Scobey gives values of K_s for old pipe allowing for deterioration of the interior surface. To allow for deterioration a value of $K_s = 0.34$ is usually assumed in design for pipes whose interior is accessible for inspection and maintenance. Friction losses for this value of K_s, for pipes up to 10 feet in diameter, can be read from the chart, figure 9. For pipes too small to permit access for maintaining the interior coating a value of $K_s = 0.40$ is usually assumed.

Bend losses vary according to the shape of the bend and the condition of the inside surface. Mitered bends constructed from plate steel no doubt cause greater losses than smooth curvature bends formed in castings or concrete; however, there is no way to evaluate such effects since data on actual installations are very meager. Laboratory experiments on very small size bends with low Reynolds numbers are not applicable to large size bends with high Reynolds numbers. When water flows around a bend, eddies and secondary vortices result, and the effects continue for a considerable distance downstream from the bend. In sharp angle bends the secondary vortex motion may be reduced by guide vanes built into the bend.

Thoma's(4) formula is based on experiments made at the Munich Hydraulic Institute with 1.7-inch-diameter smooth brass bends having Reynolds numbers up to 225,000, as shown on the chart in figure 10, and is expressed as:

$$H_b = C \frac{V^2}{2g} \dots\dots\dots\dots\dots\dots\dots \quad (2)$$

where

H_b = bend loss in feet

C = experimental loss coefficient, for bend loss

V = velocity of flow in feet per second.

The losses shown in figure 10 vary according to the R/D ratio and the deflection angle of the bend. An R/D ratio of six results in the lowest head loss, although only a slight decrease is indicated for R/D ratios greater than

FRICTION LOSS - FEET PER 1000 FEET

FLOW IN C.F.S.
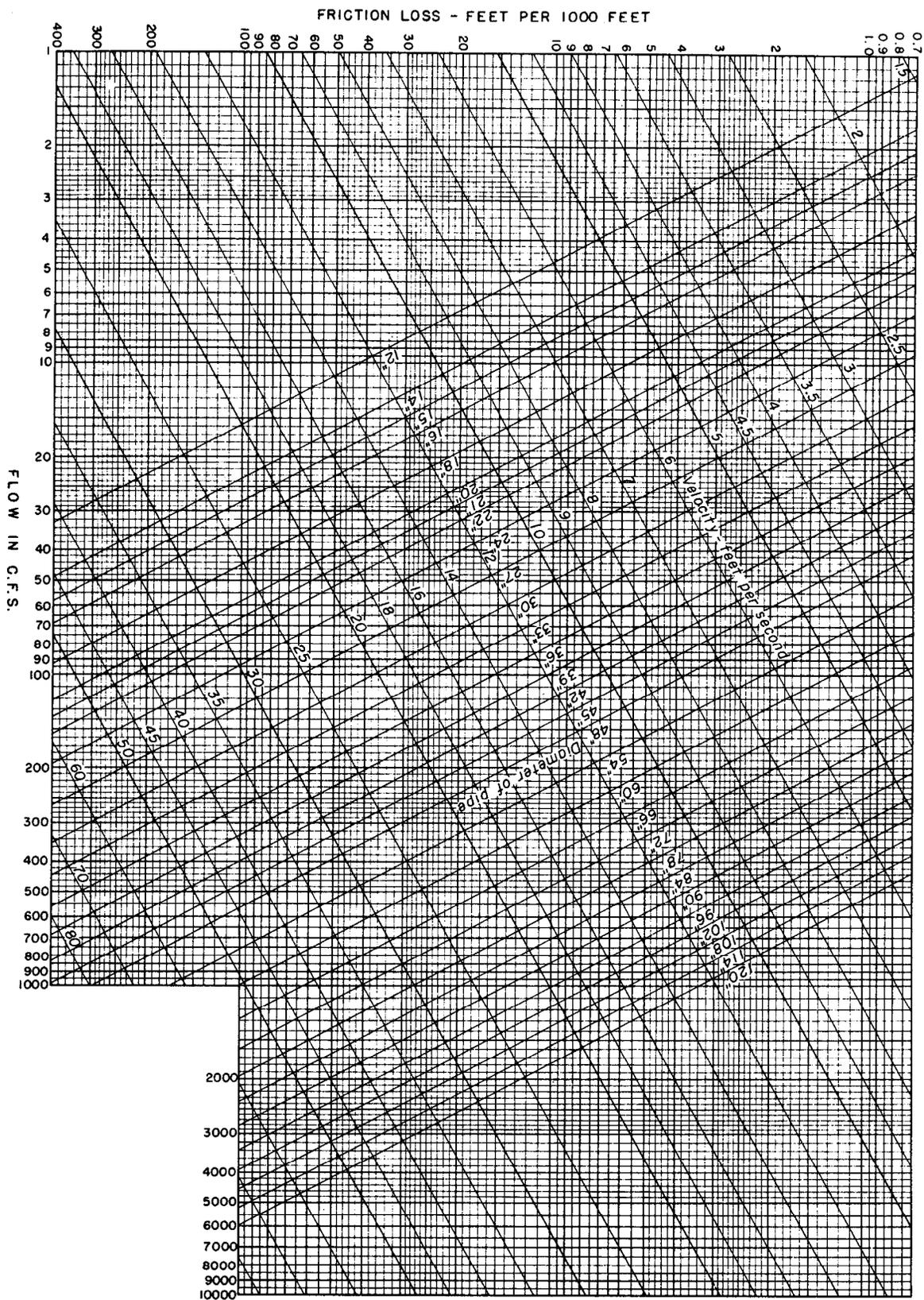
FIGURE 9—*Friction losses in welded steel pipe based on Scobey's formula for 6-year-old pipe and nonaggressive waters.*
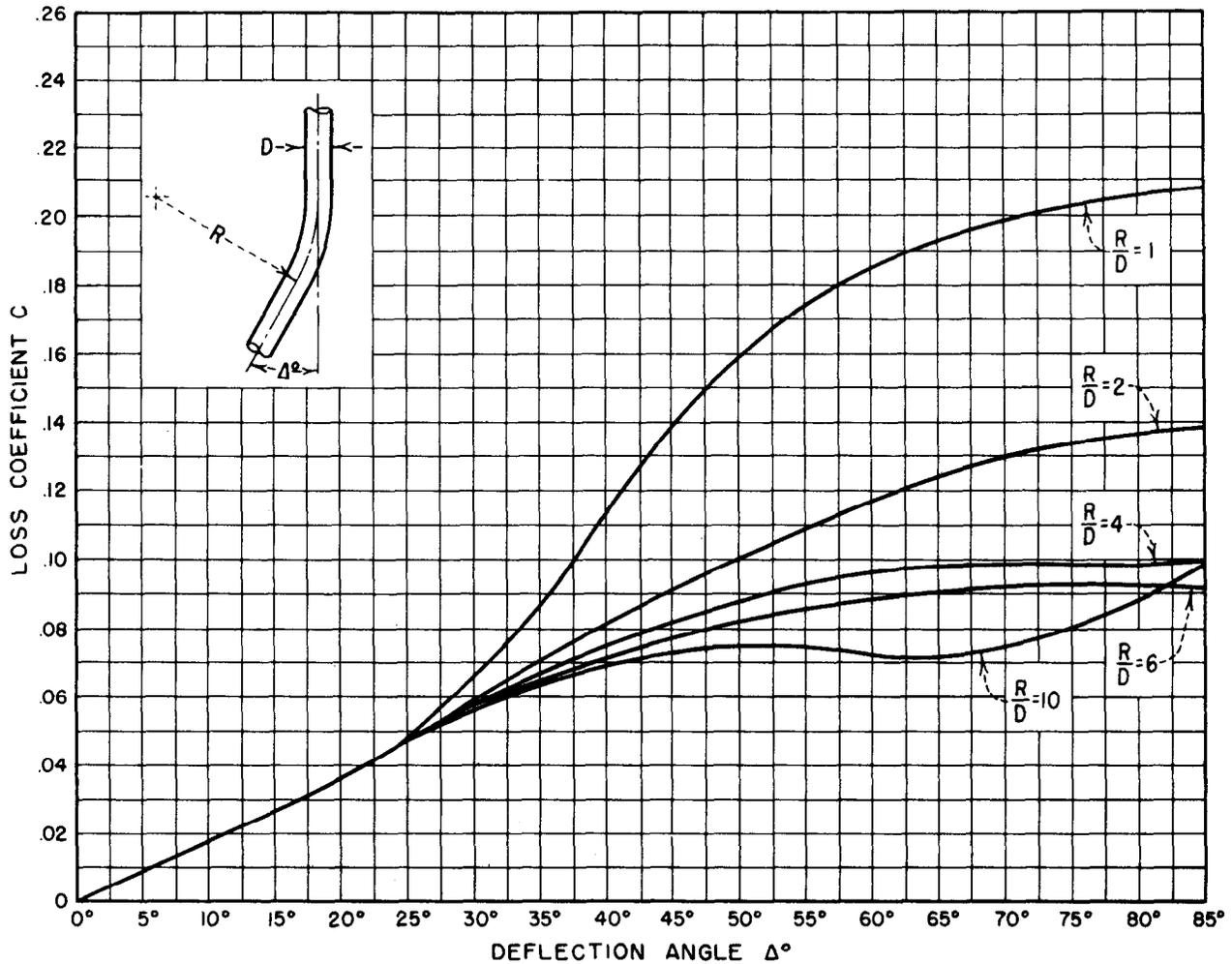
FIGURE 10.—*Losses for various values of $\frac{R}{D}$ ratios and deflection angles up to 90°.*

four. This relationship is also indicated by the curves of figure 11, which were plotted from experiments with 90° bends. As the fabrication cost of a bend increases with increasing radius and length, there appears to be no economic advantage in using R/D ratios greater than five.

Head losses in gates and valves vary according to their design, being expressed as:

$$H_g = K\frac{V^2}{2g} \quad\quad\quad\quad\quad (3)$$

in which K is an experimental loss coefficient whose magnitude depends upon the type and size of gate or valve and upon the percentage of opening. As gates or valves placed in penstocks are not throttled (this being accom-

plished by the wicket gates of the turbines), only the loss which occurs at the full open condition needs to be considered. According to experiments made at the University of Wisconsin(5) on gate valves of 1- to 12-inch diameter, the coefficient K in Equation (3) varies from 0.22 for the 1-inch valve to 0.065 for the 12-inch valve for full openings. For large gate valves an average value of 0.10 is recommended; for needle valves, 0.20; and for medium size butterfly valves with a ratio of leaf thickness to diameter of 0.2, a value of 0.26 may be used. For sphere valves having the same opening as the pipe there is no reduction in area, and the head loss is negligible.
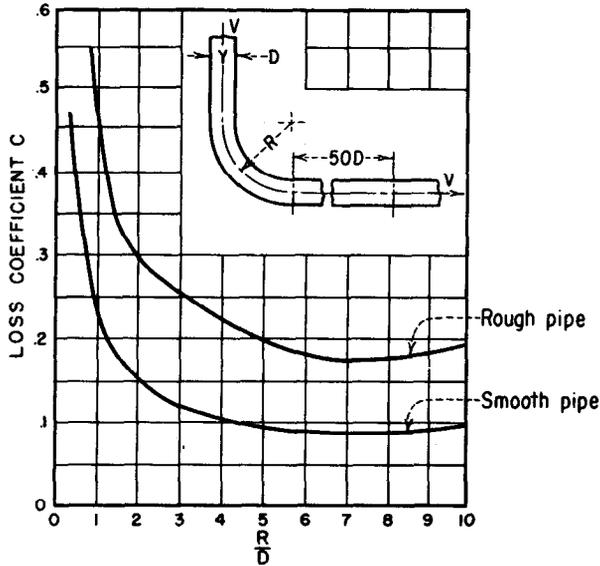
FIGURE 11.—Head losses in 90° pipe bends as determined for various $\frac{R}{D}$ ratios.

CURVE NO.	$\frac{d_o}{d}$	Δ
1	.349	90°
2	.349	60°
3	.581	90°
4	.581	60°

CURVE NO.	$\frac{d_o}{d}$	Δ	θ	L
5	.349	60°	13½°	2.7d_o
6	.581	90°	12½°	2.7d_o
7	.581	60°	13°	2.6d_o
8	.581	45°	13½°	2.0d_o

CURVE NO.	$\frac{d_o}{d}$	Δ
9	.349	60°
10	.581	90°
11	.581	60°

FIGURE 12.—Loss coefficients for divided flow through small tees and branch outlets as determined for various flow ratios $\frac{Q_a}{Q}$.

Fittings should be designed with smooth and streamlined interiors since these result in the least loss in head. Data available on losses in large fittings are meager. For smaller fittings, as used in municipal water systems, the American Water Works Association recommends the following values for loss coefficients, K; for reducers, 0.25 (using velocity at smaller end); for increasers, 0.25 of the change in velocity head; for right angle tees, 1.25; and for wyes, 1.00. These coefficients are average values and are subject to wide variation for different ratios between flow in main line and branch outlet. They also vary with different tapers, deflection angles, and streamlining. Model tests made on small tees and branch outlets at the Munich Hydraulic Institute show that, for fittings with tapered outlets and deflection angles smaller than 90° with rounded corners, losses are less than in fittings having cylindrical outlets, 90° deflections, and sharp corners. (See figure 12.) These tests served as a basis for the design of the branch connections for the Hoover Dam penstocks.

EFFECT OF WATER HAMMER

Rapid opening or closing of the turbine gates produce a pressure wave in the penstock called water hammer, the intensity of which is proportional to the speed of propagation of the pressure wave produced and the velocity of flow destroyed. Joukovsky's fundamental equation gives the maximum increase in head for closures in time less than 2L/a seconds:

$$\Delta H = \frac{av}{g} \quad \dots \dots \dots \dots \dots \dots \dots \quad (4)$$

in which
  ΔH = maximum increase in head
  a = velocity of pressure wave
  L = length of penstock from forebay to turbine gate
  v = velocity of flow destroyed
  g = acceleration due to gravity.

From this formula, which is based on the elastic water-hammer theory, Allievi, Gibson, Durand, Quick, and others developed independent equations for the solution of water-hammer problems (6).

In his notes published in 1903 and 1913, Allievi introduced the mathematical analysis of water hammer, while M. L. Bergeron, R. S. Quick, and R. W. Angus developed graphical solutions of water-hammer problems which are more convenient to use than the analytical methods. A comprehensive account of methods for the solution of water-hammer phenomena occurring in water conduits, including graphical methods, was published by Parmakian(7).

For individual penstocks of varying diameter, the pressure reflections at points of change in diameter complicate the problem. However, if the varying diameter is reduced to a penstock of equivalent uniform diameter, a close estimate can be made of the maximum pressure rise. For penstocks with branch pipes, it is necessary to consider the reflection of pressure waves from the branch pipes and dead ends in order to determine the true pressure rise due to velocity changes.

As the investment in penstocks is often considerable, they must be safeguarded against surges, accidental or otherwise. Surges of the instantaneous type may develop through resonance caused by rhythmic gate movements, or when the governor relief or stop valve is improperly adjusted. A parting and rejoining of the water column in the draft tube or a hasty priming at the headgate may also cause surge waves of the instantaneous or rapid type. Adjustments in the profile of a penstock may be necessary to prevent the development of a vacuum and water column separation during negative pressure surges. As water-hammer surges occurring under emergency conditions could jeopardize the safety of a penstock if they are not considered in the design, their magnitude should be determined and the shell thickness designed for the resultant total head. Stresses approaching yield-point values may be allowed. By using ductile materials in the penstock, excessive surge stresses may be absorbed by yielding without rupture of plates or welds. Design criteria for including the effects of water hammer in penstock and pump discharge line installations, as used by the Bureau of Reclamation, is shown in table 1.

Surge tanks are used for reduction of water hammer, regulation of flow, and improvement in turbine speed regulation. A surge tank may be considered as a branch pipe designed to absorb a portion of the pressure wave while the remainder travels upstream toward the forebay. When located near the powerhouse, it provides a reserve volume of water to meet sudden load demands until the water column in the upper portion of the penstock has time to accelerate.

When the length, diameter, and profile of the penstock have all been determined, considering local conditions and economic factors, the selection of a minimum closure time for the turbine gates will require a compromise between the allowable pressure variation in the penstock, the flywheel effect, and the permissible speed variation for given load changes on the unit.

With reaction turbines, synchronous relief valves, which open as the turbine gates close, may be used to reduce the pressure rise in the penstock. Reduction of pressure rise is proportional to the quantity of water released. As relief valves are usually designed to discharge only a portion of the flow, this portion is deducted from the total flow in computing the reduced velocity and the corresponding pressure rise.

Pressure Rise in Simple Conduits

With instantaneous gate closure, maximum pressure rise in penstocks of uniform diameter and plate thickness occurs at the gate; from there, it travels undiminished up the conduit to the intake or point of relief. For slower closures which take less than 2L/a seconds (L=length of penstock; a=velocity of pressure wave), the maximum pressure rise is transmitted undiminished along the conduit to a point where the remainder of the distance to the intake is equal to Ta/2 (T=time for full gate stroke), from which point to the intake the pressure rise diminishes uniformly to zero. With uniform gate closure equal to or greater than the critical time, 2L/a, the maximum pressure rise occurs at the gate, from which point it diminishes uniformly along the length of the penstock to zero at the intake. An anal-

TABLE 1.—*Basic conditions for including the effects of water hammer in the design of turbine penstocks.*
The basic conditions for including the effects of water hammer in the design of turbine penstock installations are
divided into normal and emergency conditions with suitable factors of safety assigned to each type of operation.

Normal conditions of operation		Emergency conditions of operation	Emergency conditions not to be considered as a basis for design

Normal conditions of operation

The basic conditions to be considered as normal operation are as follows:

1. The turbine penstock installation may be operated at any head between the maximum and minimum values of the forebay water surface elevation.

2. The turbine gates may be moved at any rate of speed by the action of the governor head up to a predetermined rate, or at a slower rate by manual control through the auxiliary relay valve.

3. The turbine may be operating at any gate position and be required to add or drop any or all of its load.

4. If the turbine penstock installation is equipped with any of the following pressure control devices it will be assumed that these devices are properly adjusted and function in the manner for which the equipment is designed:

  (a) Surge tanks
  (b) Relief valves
  (c) Governor control apparatus
  (d) Cushioning stroke device
  (e) Any other pressure control device.

5. Unless the actual turbine characteristics are known, the effective area through the turbine gates during the maximum rate of gate movement will be taken as a linear relation with respect to time.

6. The water hammer effects will be computed on the basis of governor head action for the governor rate which is actually set on the turbine for speed regulation. If the relay valve stops are adjusted to give a slower governor setting than that for which the governor is designed, this rate shall be determined prior to proceeding with the design of turbine penstock installation and later adhered to at the power plant so that an economical basis for designing the penstock, scroll case, etc., under normal operating conditions can be established.

7. In those instances where, due to a higher reservoir elevation, it is necessary to set the stops on the main relay valve for a slower rate of gate movement, the water hammer effects will be computed for this slower rate of gate movement also.

8. The reduction in head at various points along the penstock will be computed for the rate of gate opening which is actually set in the governor in those cases where it appears that the profile of the penstock is unfavorable. This minimum pressure will then be used as a basis for normal design of the penstock to insure that subatmospheric pressures will not cause a penstock failure due to collapse.

9. If a surge tank is present in the penstock system the upsurge in the surge tank will be computed for the maximum reservoir level condition for the rejection of the turbine flow which corresponds to the rated output of the generator during the gate transversing time which is actually set on the governor. Unless an overflow spillway is provided the top elevation of the surge tank will be determined by adding a freeboard of 20 per cent of the computed upsurge to the maximum height of water at the highest upsurge.

10. The downsurge in the surge tank will be computed for the minimum reservoir level condition for a load addition from speed-no-load to the full gate position during the gate traversing time which is actually set on the governor. The bottom of the surge tank will be located at a distance of 20 per cent of the computed downsurge below the lowest downsurge in the tank to safeguard against air entering the penstock.

11. The turbine, penstock, surge tank, and other pressure control devices will be designed to withstand the conditions of normal operation which are given above with a minimum factor of safety of 4 to 5 based on the ultimate bursting or collapsing strength.

Emergency conditions of operation

The basic conditions to be considered as emergency operation are as follows:

1. The turbine gates may be closed at any time by the action of the governor head, manual control knob with the main relay valve, or the emergency solenoid device.

2. The cushioning stroke will be assumed to be inoperative.

3. If a relief valve is present it will be assumed to be inoperative.

4. The gate traversing time will be taken as the minimum time for which the governor is designed.

5. The maximum head including water hammer at the turbine and along the length of the penstock will be computed for the maximum reservoir head condition for final part gate closure to the zero-gate position at the maximum governor rate in $2L$ seconds.
$$\overline{a}$$

6. If a surge tank is present in the penstock system the upsurge in the surge tank will be computed for the maximum reservoir head condition for the rejection of full gate turbine flow at the maximum rate for which the governor is designed. The downsurge in the surge tank will be computed for the minimum reservoir head condition for a full gate opening from the speed-no-load position at the maximum rate for which the governor is designed. In determining the top and bottom elevations of the surge tank, nothing will be added to the upsurges and downsurges for this emergency condition of operation.

7. The turbine, penstock, surge tank, relief valve, or other control devices will be designed to withstand the above emergency conditions of operation with a minimum factor of safety of 2 to 3 based on the ultimate bursting or collapsing strength.

Emergency conditions not to be considered as a basis for design

1. Other possible emergency conditions of operation are those during which certain pieces of control equipment are assumed to malfunction in the most unfavorable manner. The most severe emergency condition of operation which will yield the maximum head rise in a turbine penstock installation will occur from either of the two following conditions of operation:

  (a) Rapid closure of the turbine gates in less than $\frac{2L}{a}$ seconds, (L is the length of the penstock, and a is the wave velocity), when the flow of water in the penstock is a maximum. (The maximum head rise in feet due to this condition of operation is 100 to 125 times the water velocity in feet per second.)

  (b) Rhythmic opening and closing of the turbine gates when a complete cycle of gate operation is performed in $\frac{4L}{a}$ seconds. (Under extreme conditions the maximum head rise due to this condition of operation is twice the static head.)

Since these conditions of operation require a complete malfunctioning of the governor control apparatus at the most unfavorable moment, the probability of obtaining this type of operation is exceedingly remote. Hence, these conditions will not be used as a basis for design. However, after the design has been established from other considerations, it is desirable that the stresses in the turbine scroll case, penstock, and pressure control devices be not in excess of the ultimate bursting strength of the structures for these emergency conditions of operation.

ysis of pressure time curves shows that the maximum pressure rise is determined by the rate of change of velocity with respect to time. Maximum pressure rise will develop in a penstock when closure starts from some relatively small percentage of full stroke so that some finite velocity is cut off in a time equal to 2L/a seconds.

The governor traversing time is considered to be the time required for the governor to move the turbine gates from the rated capacity position to the speed-no-load position. As the rate of governor time is adjustable, it is important that a minimum permissible rate be specified if maximum pressure rise in the penstock is to be kept within design limits.

Water-hammer conditions should be determined for the unit operating at rated head and under maximum static head. The highest total head, consisting of static and water-hammer heads, should be used for computing plate thickness of the penstock.

R. S. Quick (6) simplified water-hammer computations by using a pipeline constant, K, and a time constant, N, in the equations (similar to Allievi's) which determine pressure rise, or water hammer, resulting from instantaneous closure. The chart in figure 13 shows the relative values of K and P (equal to $\frac{h}{h_{max}}$) for various values of N. Also included is a chart which shows the velocity, a, of the pressure wave in an elastic water column for various ratios of penstock diameter to thickness. Figure 14 gives only the maximum values of P for uniform gate motion and complete closure. It covers a range of closures from instantaneous to 50 intervals, and a range of values of K from 0.07 to 40, which includes the majority of practical cases. The nearly vertical curve shows the limiting value for maximum pressure rise at the end of the first time interval, 2L/a. Values of pressure rises to the left of this line attain their maximum values at the end of the first interval.

PIPE SHELL

As has been stated, penstocks should be designed to resist the total head consisting of

static and water-hammer heads. Working stresses which will assure safety under all expected operating conditions should be used. However, stresses approaching the yield point may be used in designing for emergency conditions. For penstocks supported on piers in open tunnels or above the ground, allowance should be made for temperature and beam stresses in addition to the stresses due to internal pressure. The diagram shown in figure 14 permits a quick determination of equivalent stresses if principal stresses are known. The diagram is based on the Hencky-Mises theory of failure, sometimes called the shear-distortion or shear-energy theory. The plate thickness should be proportioned on the basis of an allowable equivalent stress, which varies with the type of steel used. The ASME code gives maximum allowable tensile stresses for various types of steels.

The hoop tension, S, in a thin shell pipe, due to internal pressure is expressed as:

$$S = \frac{Dp}{2te} \quad \dots\dots\dots\dots\dots\dots\dots\dots \quad (5)$$

in which

D = inside diameter of pipe in inches
p = internal pressure in psi
t = plate thickness in inches
e = efficiency of joint.

Regardless of pressure, a minimum plate thickness is recommended for all large steel pipes to provide the rigidity required during fabrication and handling. For penstocks the desired minimum thickness for diameter, D, may be computed from the formula:

$$t_{min} = \frac{D+20}{400} \quad \dots\dots\dots\dots\dots\dots\dots \quad (6)$$

A thinner shell may in some cases be used if the penstock is provided with adequate stiffeners to prevent deformation during fabrication, handling, and installation.

Joint efficiencies for arc-welded pipe depend on the type of joint and the degree of examination of the longitudinal and circumferential joints. The ASME code stipulates a maximum allowable joint efficiency of 100 percent for double-welded butt joints completely radiographed, and of 70 percent if radiographic examination is omitted. Corresponding joint efficiencies for single-welded butt joints with

EXPLANATION

P = Pressure Rise as a proportion of h_{max}.
h = Pressure Rise or excess Head above normal, Feet.
h_{max} = Pressure Rise of instantaneous closure = aV_0/g, Feet.
g = Acceleration of Gravity, Feet per second, per second.
a = Velocity of Pressure Wave along Pipe, Feet per second - See graph.
V_0 = Velocity in Pipe near gate, corresponding to H_0 and Q_0, Feet per second.
H_0 = Initial Steady Head near gate, corresponding to V_0, Feet.
Q_0 = Initial Steady Flow in pipe prior to start of gate closure corresponding to H_0, Cu. Ft. per sec.
T = Time of gate closure travel, Seconds.
L = Length of pipe from gate to forebay or other point of relief, Feet.

Values of $P = \dfrac{h}{h_{max}} = \dfrac{gh}{aV_0}$

Values of Time Constant $N = \dfrac{aT}{2L}$

Region of Max. Rise at End of First Interval — Region of Max. Rise after End of First Interval

Limiting Line for Max. Pressure Rise, End of First Interval

Values of Pipe - Line Constant $K = \dfrac{aV_0}{2gH_0}$
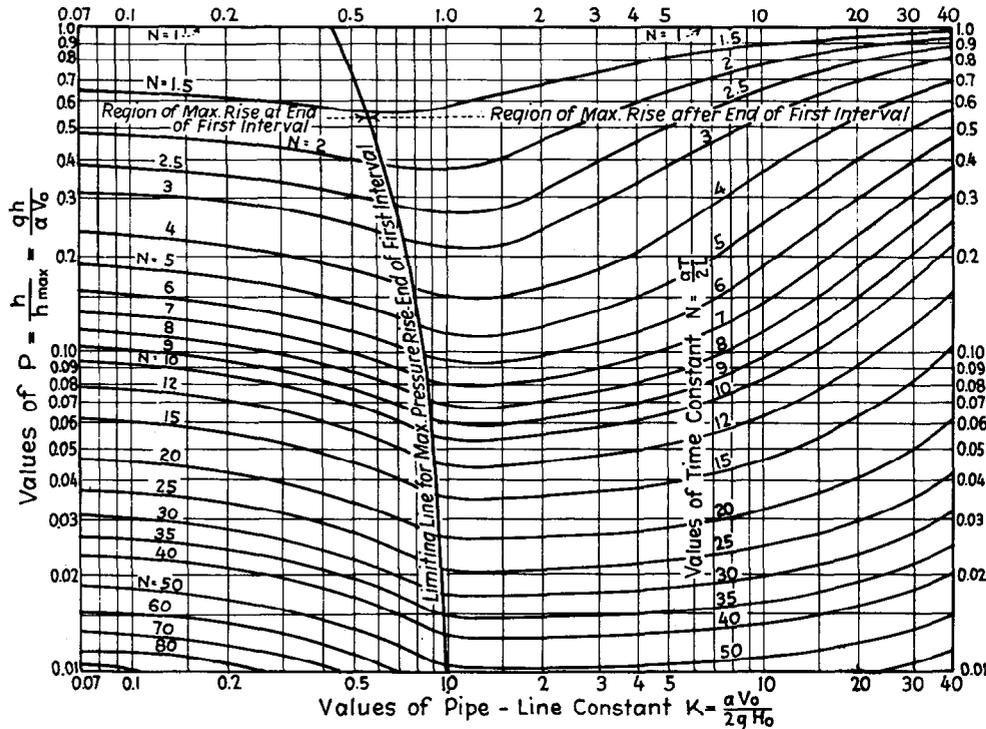
CHART SHOWING MAXIMUM PRESSURE RISE WITH UNIFORM GATE MOTION AND COMPLETE CLOSURE : BASED ON ELASTIC-WATER-COLUMN THEORY

NOTE: Ratio of Pressure Rise "h" to Initial Steady Head "H_0" determined from relation $2KP = h/H_0$

Velocity of Pressure Wave a in Ft. per Sec.

Ratio of Pipe Diameter to Thickness of Pipe Wall
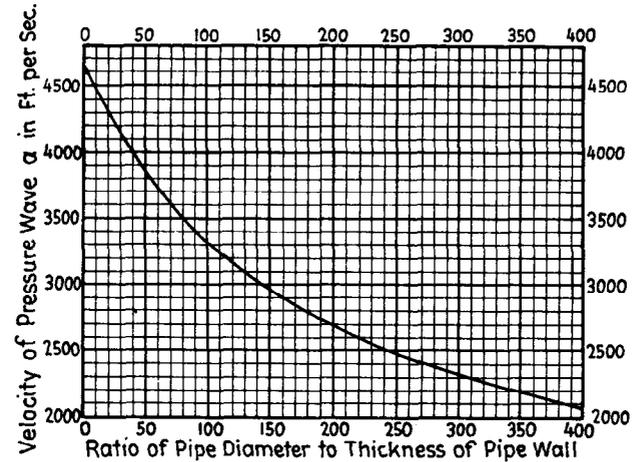
CHART SHOWING VELOCITY OF TRAVEL OF PRESSURE WAVE IN ELASTIC WATER COLUMN

Formula: $\quad a = \dfrac{4660}{\sqrt{1 + \dfrac{kd}{Eb}}} = \dfrac{4660}{\sqrt{1 + \dfrac{d}{100\,b}}}$

Where a = Velocity of Travel of Pressure Wave. Ft. per Sec.
k = Bulk Modulus of Elasticity of Water = 294,000 Lbs. per Sq. In.
E = Young's Modulus for Pipe Walls = 29,400,000 Lbs. per Sq. In. approx. for Steel.
b = Thickness of Pipe Walls, Inches.
d = Inside Diameter of Pipe, Inches.

FIGURE 13.—*Water-hammer values for uniform gate motion and complete closure.*
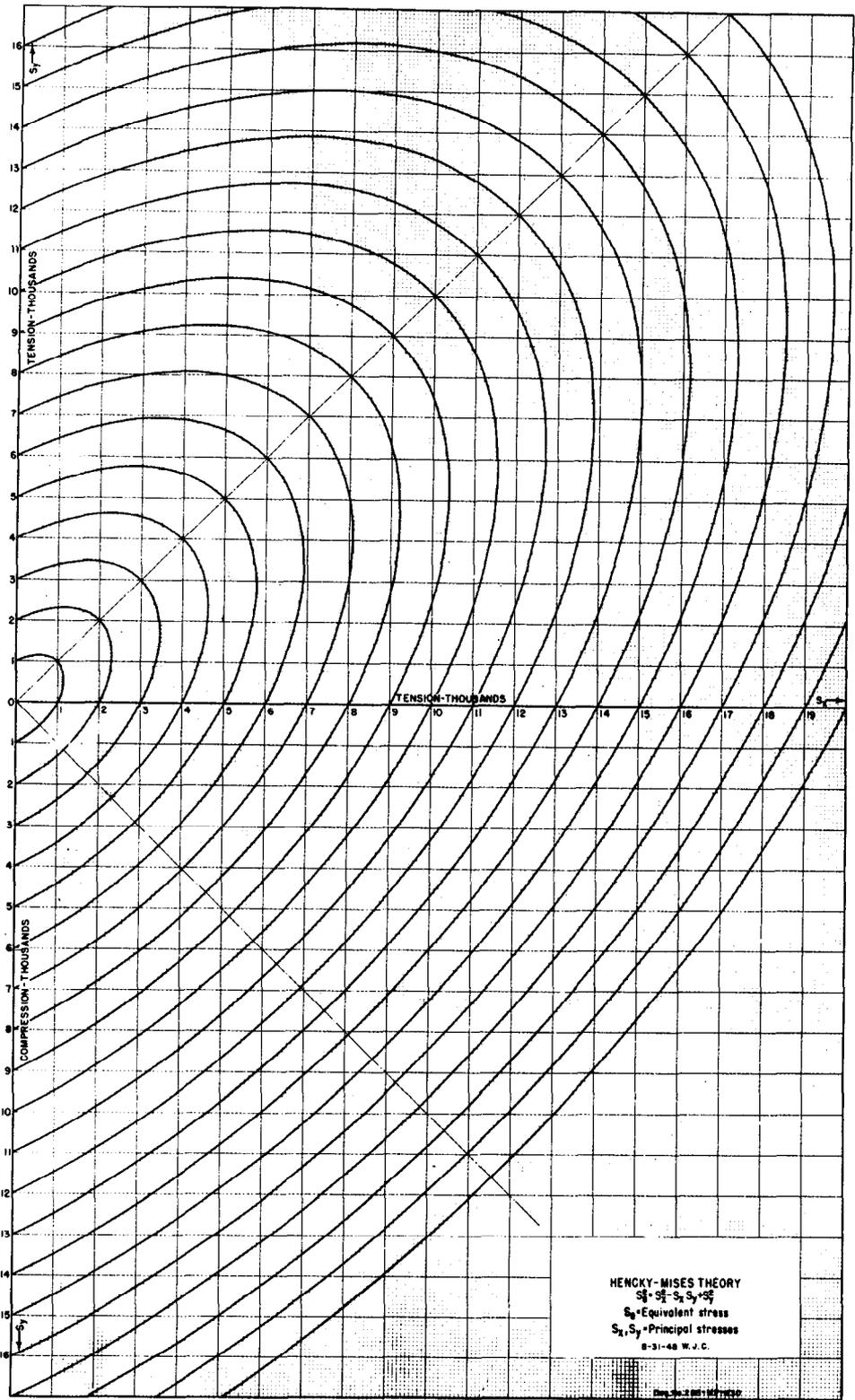
FIGURE 14.—*Equivalent stress diagram.*

backing strips are 90 and 65 percent, respectively. If radiographic spot examination is used, allowable joint efficiencies are 15 percent higher than for nonradiographed joints. Postweld heat treatment of welds is required if the wall thickness exceeds a specified minimum thickness. Joint efficiencies and radiographic inspection procedures used by the Bureau of Reclamation conform to the requirements of the ASME code.

Specifications issued by the Bureau of Reclamation for construction of penstocks usually require that they be welded by a rigidly controlled procedure using automatic welding machines, that the longitudinal and circumferential joints be radiographed, and that either the individual pipe sections or the entire installation be tested hydrostatically.

Since the head varies along the profile of a penstock in accordance with its elevation and pressure wave diagram, it is customary to plot the heads and stresses as shown in figure 15. The total head at each point along the profile can then be scaled off and the plate thickness computed accordingly.

Other stresses which must be considered in addition to hoop stresses are as follows:

Temperature stresses
Longitudinal stresses which accompany radial strain (related to Poisson's ratio)
Beam stresses.

Temperature Stresses

For a steel pipe fully restrained against movement, the unit stress per degree of temperature change is equal to the coefficient of expansion of the steel multiplied by its modulus of elasticity, or $0.0000065 \times 30 \times 10^6 =$ 195 psi per degree of temperature change. For a pipe having expansion joints and being free to move on supports, the longitudinal temperature stress is equal to the frictional resistance between supports and pipe plus the frictional resistance in the expansion joint. The resistance at supports varies according to the type of support and its condition. The following average values of coefficients of friction have been determined by tests:

Steel on concrete (cradle supports) .0.60
Steel on steel—rusty plates 0.50
Steel on steel—greased plates 0.25
Steel on steel—with two layers
of graphited asbestos sheets
between 0.25
Rocker supports—deteriorated......0.15

For expansion joints a frictional resistance of 500 pounds per linear foot of circumference was determined by test and may be used for the computation of longitudinal forces in a pipeline.

Longitudinal Stresses Caused by Radial Strain

Radial expansion of a steel pipe caused by internal pressure tends to cause longitudinal contraction (Poisson's ratio), with a corresponding longitudinal tensile stress equal to 0.303 of the hoop tension. This is true, provided the pipe is restrained longitudinally. This stress should be combined algebraically with other longitudinal stresses in order to determine the total longitudinal stress.

Beam Stresses

When a pipe rests on supports it acts as a beam. The beam load consists of the weight of the pipe itself plus the contained water. Beam stresses at points of support, particularly for longer spans, require special consideration. This matter is discussed in the section on design of supports.

Based on a preliminary design, various combinations of beam, temperature, and other stresses should be studied so as to determine the critical combination which will govern the final design. It may be necessary or desirable to reduce the distance between anchors for pipes without expansion joints to reduce the temperature stresses or to shorten the span between supports to reduce beam stresses. For penstocks buried in the ground, and for all other installations where the
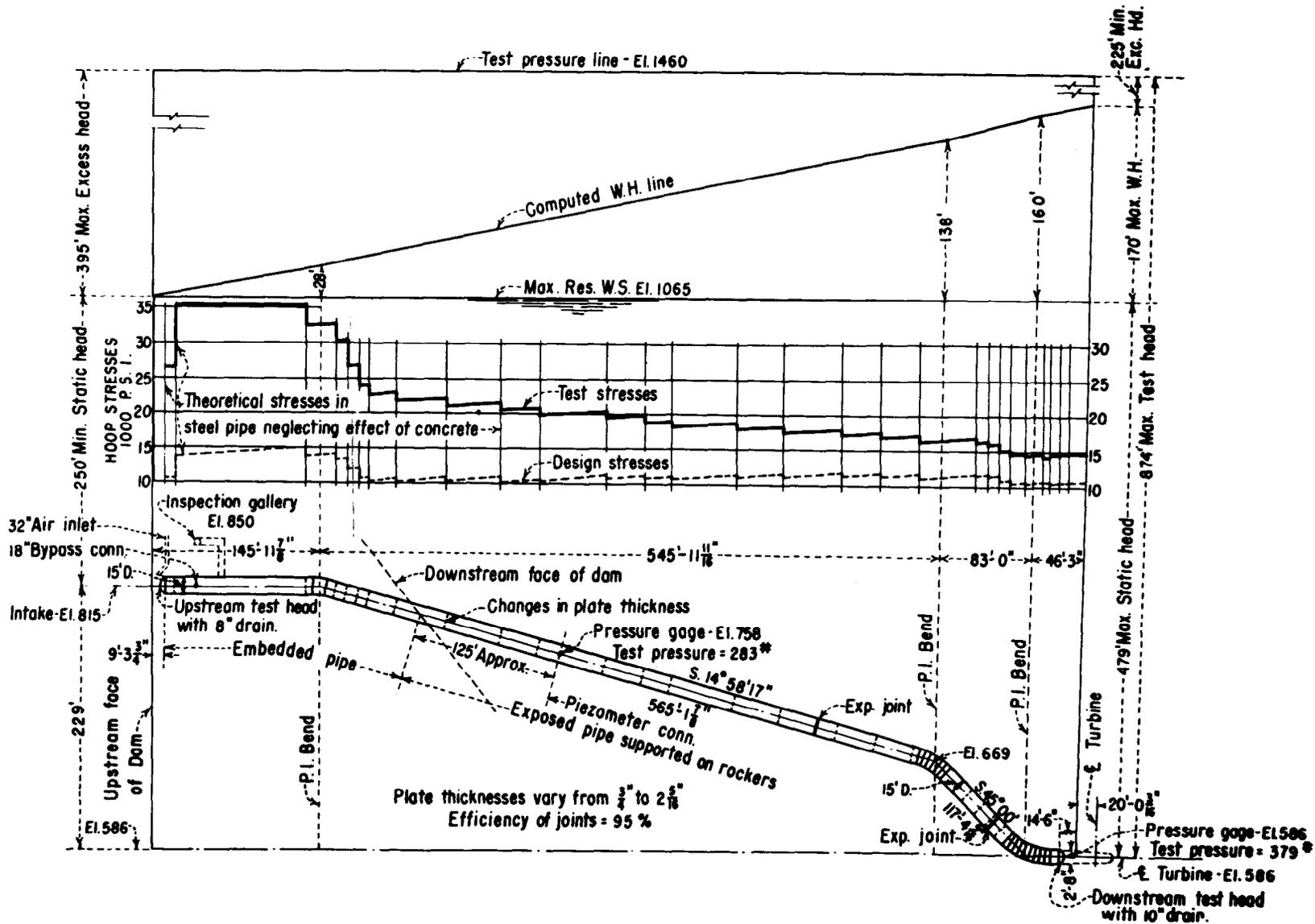
FIGURE 15.—A graphical illustration of heads and stresses determined for the hydrostatic testing of the Shasta penstocks.

temperature variation in the steel corresponds to the small temperature range of the water, expansion joints may be eliminated and all temperature stresses carried by the pipe shell. For a pipe without expansion joints and anchored at both ends in which the beam stresses are negligible, the longitudinal stresses may be kept within allowable limits by welding the last girth joint in the pipe at the mean temperature of the steel. A procedure similar to this was used for the penstock and outlet headers at Hoover Dam where expansion joints for the 30-foot pipe were not considered to be feasible. In this case it was desired to eliminate all longitudinal tension in the penstock because the pinned girth joints had an efficiency of only 60 percent in tension but 100 percent in compression. The lowest anticipated service temperature was 45° F. In order to reduce the length of a penstock section between anchors to that corresponding to a temperature of 45° F, mechanical prestressing by means of jacks applied at the periphery of the pipe was resorted to. A compressive force corresponding to the difference between the erection temperature and the lowest service temperature was applied. After welding the final closing joint the jacks were removed, leaving the penstock in compression. At 45° F. the longitudinal stress is then zero, and at higher temperatures the penstock is in compression.

SUPPORTS

Modern trend in design requires that steel pipes located in tunnels, above ground, or across gullies or streams be self-supporting. This is possible in most cases without an increase in plate thickness except adjacent to the supports of the longer spans. If the pipe is to function satisfactorily as a beam, deformation of the shell at the supports must be limited by use of properly designed stiffener rings or ring girders. A long pipeline with a number of supports forms a continuous beam except at the expansion joints, where its continuity is lost. Ring girders prevent large deformation of the pipe shell at the supports. Stresses may therefore be analyzed by the elastic theory of thin cylindrical shells (9). The shell will be mainly subjected to direct beam and hoop stresses, with loads being transmitted to support rings by shear. Because of the restraint imposed by a rigid ring girder or concrete anchor, secondary bending stresses occur in the pipe shell adjacent to the ring girder or anchor. Although this is only a local stress in the shell, which decreases rapidly with increasing distance from the stiffener, it should be added to the other longitudinal stresses. For a pipe fully restrained, the maximum secondary bending stress is:

$$S_b = 1.82 \frac{pr}{t} \quad\dots\dots\dots\dots\dots\dots\dots\dots\dots (7)$$

in which
   p = pressure in psi
   r = radius of pipe in inches
   t = plate thickness in inches.
This secondary bending stress decreases with any decrease in restraint.

If use of Equation (7) results in excessive longitudinal stresses, it may be necessary to increase the pipe shell thickness on each side of the stiffener ring for a minimum length of $3/q$, in which $q = 1.285/\sqrt{rt}$. At this distance from the stiffener ring, the magnitude of the secondary stresses becomes negligible. Secondary bending stresses at edges or corners of concrete anchors may be reduced by covering the pipe at these points with a plastic material, such as asbestos or cork sheeting, prior to concrete placement. This will also protect the edges or corners of the concrete against cracking or spalling.

Pipes designed in accordance with the preceding principles may be supported on long spans without intermediate stiffener rings. Figure 16 shows the 10-foot 3-inch Shoshone River siphon, which has a 150-foot span. The length of the span to be used on any particular job is usually a matter of economy. Very long spans, such as shown in figure 16, are economical only under certain conditions, as in the crossing of rivers or canyons where the construction of additional piers, which shorter spans would require, is not feasible.

If continuous pipelines with or without expansion joints are supported at a number of points, the bending moments at any point

FIGURE 16.—*The Shoshone River siphon crosses the river on a 150-foot span.*

along the pipe may be computed as in an ordinary continuous beam by using applicable beam formulae. Spans containing expansion joints should be made short enough that their bending moments will correspond to those of the other spans. Expansion joints should be placed at midspan where deflections of the two cantilevered portions of pipe are equal, thus preventing a twisting action in the joint.

A pipe can be designed to resist safely the bending and shear forces acting in a cross-sectional plane by several methods, as follows:

1. By sufficient stiffness in the shell itself
2. By continuous embedment of part of the periphery of the pipe
3. By individual support cradles or saddles

4. By stiffener rings which carry the load to concrete piers by means of support columns.

As the static pressure within a pipe varies from top to bottom, it tends to distort the circular shape of the shell. This is especially pronounced for thin-shelled large-diameter pipes under low head or partially filled. The weight of the pipe itself and the weight of backfill, if the pipe is covered, also cause distortion of the shell.

Depending on the method of support, stresses and deformations around the circumference of a filled pipe will assume various patterns as shown in figure 17. These diagrams indicate the best location for longitudinal
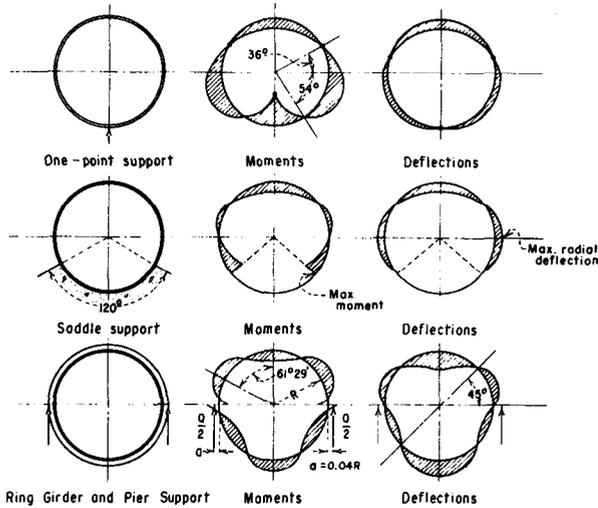
FIGURE 17.—*Moments and deflections developed in a pipe precisely full, using various types of supports.*

joints in pipe shell and joints in stiffeners to avoid points of highest stress or largest deformation. The saddle and the ring girder with column supports are widely used. The one-point support should not be used for a permanent installation. It is included merely to illustrate its flattening effect on an unstiffened pipe.

For the ring girder and column-type support, the support columns are attached to the ring girders eccentrically with respect to the centroidal axis of the ring section so as to reduce the maximum bending moment in the ring section. In computing the section modulus of the ring girder, a portion of the adjacent shell may be considered as acting with the girder. The total length of the shell thus acting is:

$$l = b + 1.56\sqrt{rt} \quad \dots \dots \dots \dots \dots \dots \dots \quad (8)$$

in which b is the width of the ring girder (see figure 18) and r and t are as defined in Equation (7). Essential formulae and coefficients for the computation of stresses in ring girders are given in figure 18. These formulae and coefficients were developed from the stiffener ring analysis for the Hoover Dam penstocks. The table gives stress coefficients, K1 to K6, inclusive, for various points around the circumference of the ring. These coefficients are to be inserted in the appropriate equations shown for the determination of direct stress, T, bending moment, M, and radial shear stress,

S, in the ring section. By adding the direct, bending, and tensile stresses in the ring due to internal pressure in the pipe, the total unit stress in the inner and outer fibers of the ring may be determined.

In installations subject to seismic disturbances, the severity of the earthquake shocks should be ascertained from local records and considered in the design of the supports. Unless the project is located close to a fault zone, a horizontal seismic coefficient of 0.1 to 0.2 of the gravity load is adequate for most areas in the United States. Stresses due to earthquake loads for various points along the periphery of the ring girder may be computed from the equations and stress coefficients given in figure 19. In determining the required section for a ring girder, stresses so computed should be added to the stresses caused by static loads.

A typical ring girder and column support designed for an 8-foot penstock with a span of 60 feet is shown in figure 20. The girder consists of two stiffener rings continuously welded to the pipe on both sides and tied together with diaphragm plates welded between the two rings. Two short columns consisting of wide-flange I-beams are welded between the rings to carry the load to the rockers by means of cast-steel bearing shoes. A typical rocker assembly is shown in figure 21. It consists of an 18-inch cast-steel rocker, a 3-1/2-inch steel pin with bronze bushings, and a cast-steel pin bearing. The rockers are kept in alinement by means of a steel tooth bolted to the side of each rocker and guided between two studs threaded into the bearing shoe of the support. The two pin bearings transmit the load to the concrete pier. After being positioned in accordance with a temperature chart so as to provide effective support for the range of temperature anticipated, they are grouted into the top of the pier as shown in figure 21. The adjustable steel angle yoke shown in figure 21 is used only during erection and grouting, after which time it is removed.

Thin-shelled pipes, when restrained longitudinally, are subjected to buckling stresses because of axial compression. The permissible span between supports is limited by the stress at which buckling or wrinkling will

STIFFENER RING COEFFICIENTS

Centroid of combined section ring

SEC. Y-Y
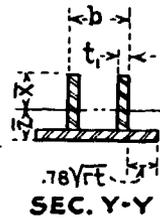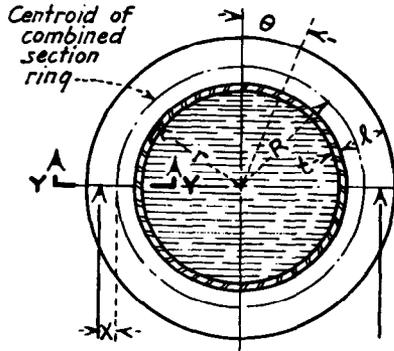
$$B = \frac{r}{R}\left(1 - \frac{2K}{qr}\right) - \frac{X}{R}$$

$$q = \frac{1.285}{\sqrt{rt}}$$

$$K = \frac{r}{L}\left[\frac{\mu L^2}{12 r^2} + \left(1-\mu^2\right)\left(1-\frac{Qs}{2Q}\right) + \frac{(2+\mu)L}{4qr^2}\right]$$

μ = Poisson's ratio = 0.3

$$C = \left(\frac{r}{R}-1\right)\left(1 - \frac{2K}{qr}\right) - \frac{X}{R}$$

$$N = Pr\left[b + \frac{2(1-\mu^2)}{q}\right]$$

A = Area combined ring section in square inches = $t\left(b + 1.56\sqrt{rt}\right) + 2\lambda t_1$
I = Moment of inertia of Section Y-Y.
L = Length of one span in inches.
M = Bending moment in the ring, inch pounds.
N = Tension due to internal pressure, on section of ring, in pounds.
P = Pressure head in pounds per square inch.
Qs = Weight of pipe shell in one span in pounds.
Q = Combined weight pipe shell and water in one span, in pounds.
S = Radial shear stress in stiffener ring, in pounds.
T = Direct stress in the ring exclusive of N, in pounds.
Total stress in outer fiber of ring = $\frac{T}{A} - M\frac{X}{I} + \frac{N}{A}$.
Total stress in inner fiber of ring = $\frac{T}{A} + M\frac{X}{I} + \frac{N}{A}$.

+ = Tension, - = Compression

$\theta°$	$T = Q\left(K_1 + B K_2\right)$		$M = Q\left(R K_3 + X K_4\right)$		$S = Q\left(K_5 + C K_6\right)$	
	K_1	K_2	K_3	K_4	K_5	K_6
0°	−.238732	+.318310	+.011267	−.068310	0	0
15°	−.241384	+.307464	+.008618	−.057464	+.019651	+.082385
30°	−.248415	+.275664	+.001585	−.025665	+.032380	+.159155
45°	−.257198	+.225079	−.007198	+.024921	+.032117	+.225079
60°	−.263704	+.159155	−.013704	+.090845	+.014417	+.275664
75°	−.263023	+.082385	−.013023	+.167616	−.022945	+.307463
90°−	−.250000	0	0	+.250000	−.079577	+.318310
90°+	+.250000	0	0	−.250000	−.079577	+.318310
105°	+.263023	−.082385	+.013023	−.167616	−.022945	+.307463
120°	+.263704	−.159155	+.013704	−.090845	+.014417	+.275664
135°	+.257198	−.225079	+.007198	−.024921	+.032117	+.225079
150°	+.248415	−.275664	−.001585	+.025665	+.032380	+.159155
165°	+.241384	−.307464	−.008618	+.057464	+.019651	+.082385
180°	+.238732	−.318310	−.011267	+.068310	0	0

+ = Tension, - = Compression

REFERENCE - BOULDER CANYON PROJECT - FINAL REPORTS, PART 5, - BULLETIN 5, TABLES NO'S. 3, 4 & 6.

FIGURE 18.—*Formulae and coefficients for the computation of stresses in ring girders as developed for stiffener ring analyses.*

θ°	M·nQ(RK₁+WK₂)		Tᵢ·nQ(K₃+BK₄)		S·nQ(K₅+CK₆)	
	K_1	K_2	K_3	K_4	K_5	K_6
0	0	0	0	0	+.079577	+.318310
15	-.019651	-.064705	-.019651	-.082385	+.066082	+.307464
30	-.032380	-.125000	-.032380	-.159155	+.027249	+.275664
45	-.032119	-.176776	-.032119	-.225079	-.032119	+.225079
60	-.014417	-.216506	-.014417	-.275664	-.104549	+.159155
75	+.022946	-.241482	+.022946	-.307464	-.180639	+.082385
90⁻	+.079577	-.250000	+.079577	-.318310	-.250000	0
90⁺	+.079577	+.250000	+.079577	-.318310	+.250000	0
105	+.022946	+.241482	+.022946	-.307464	+.180639	+.082385
120	-.014417	+.216506	-.014417	-.275664	+.104549	+.159155
135	-.032119	+.176776	-.032119	-.225079	+.032119	+.225079
150	-.032380	+.125000	-.032380	-.159155	-.027249	+.275664
165	-.019651	+.064705	-.019651	-.082385	-.066082	+.307464
180	0	0	0	0	-.079577	+.318310
195	+.019651	-.064705	+.019651	+.082385	-.066082	+.307464
210	+.032380	-.125000	+.032380	+.159155	-.027249	+.275664
225	+.032119	-.176776	+.032119	+.225079	+.032119	+.225079
240	+.014417	-.216506	+.014417	+.275664	+.104549	+.159155
255	-.022946	-.241482	-.022946	+.307464	+.180639	+.082385
270⁻	-.079577	-.250000	-.079577	+.318310	+.250000	0
270⁺	-.079577	+.250000	-.079577	+.318310	-.250000	0
285	-.022946	+.241482	-.022946	+.307464	-.180639	+.082385
300	+.014417	+.216506	+.014417	+.275664	-.104549	+.159155
315	+.032119	+.176776	+.032119	+.225079	-.032119	+.225079
330	+.032380	+.125000	+.032380	+.159155	+.027249	+.275664
345	+.019651	+.064705	+.019651	+.082385	+.066082	+.307464
360	0	0	0	0	+.079577	+.318310

SECTION F-F

$$B = \frac{r}{R}\left(1 - \frac{2K}{qr^2}\right) + \frac{\pi W}{4R} \qquad \left(-\frac{\pi}{2} < \theta < \frac{\pi}{2}\right)$$

$$q = \frac{1.285}{\sqrt{rt}}$$

$$B = \frac{r}{R}\left(1 - \frac{2K}{qr^2}\right) - \frac{\pi W}{4R} \qquad \left(\frac{\pi}{2} < \theta < \frac{3\pi}{2}\right)$$

μ = Poisson's ratio = 0.3

$$C = \left(\frac{r}{R}-1\right)\left(1 - \frac{2K}{qr^2}\right) + \frac{\pi W}{4R}$$

$$K = \frac{r}{L}\left[\frac{\mu L^2}{12r^2} + (1-\mu^2)\left(1 - \frac{Qs}{2Q}\right) + \frac{(2+\mu)L}{4qr^2}\right]$$

A = Area of combined section ring in square inches = $t(b+1.56\sqrt{rt})+2Lt_1$
L = Length of one span in inches
M = Bending moment in the ring, inch pounds
Qs = Weight of pipe shell in one span, pounds
Q = Combined weight of pipe shell and water in one span, pounds
n = Seismic coefficient
S = Radial shear stress in stiffener ring, pounds
T = Direct stress in stiffener ring, pounds

Total stress in outer fiber of ring = $\frac{T}{A} - M\frac{X}{I}$

Total stress in inner fiber of ring = $\frac{T}{A} + M\frac{Z}{I}$

$$T = T_i + \frac{n(Q-Qs)}{\pi L}\left[b + \frac{2}{q}(1-\mu^2)\right]$$

FIGURE 19.—*Formulae and coefficients for the computation of stresses in ring girders due to earthquake loads.*

occur. The theoretical buckling stresses in a cylindrical shell of perfect form is given by Timoshenko (12) and others as:

For buckling failure by the formation of a single wrinkle

$$S = \frac{E}{\sqrt{3(1-M^2)}}\left(\frac{t}{r}\right) \quad \dots\dots\dots\dots \quad (9)$$

EXPANSION JOINTS

For penstocks above ground, the temperature of the steel is influenced principally by the temperature of the water when filled, and by the temperature of the air when empty. If the pipe is exposed, its temperature will be affected by heat from the sun. In underground installations, the pipe temperature is affected by the temperature of the contained water and surrounding soil.

The main purpose of expansion joints is to permit longitudinal expansion of the pipe which results from temperature change. Expansion joints also serve as construction joints to adjust discrepancies in pipe lengths. Disregarding frictional resistance, the change in length of a pipe of length L per degree temperature change, is 0.0000065 L.

Among the several types of expansion joints in use, the sleeve type is the most popular for large steel pipes. Longitudinal movement is permitted by two closely fitting sleeves, one sliding in the other, with a stuffing box and packing to prevent leakage. A bolted packing gland is used to compress the packing which consists of long-fibre braided flax impregnated with a suitable lubricant.

A typical expansion joint of this type designed for an 8-foot penstock under 210-foot static head is shown in figure 22. The exterior surface of the inner sleeve is clad with chromium to prevent corrosion and insure free sliding in the joint. This type of joint

Support
ring

Dust
guard

Bearing
shoe

Bearing
plate

℄ Rocker support

TYPICAL ELEVATION

Symmetrical
about ℄

HALF SECTION

Support ring

Pipe shell

Drain holes
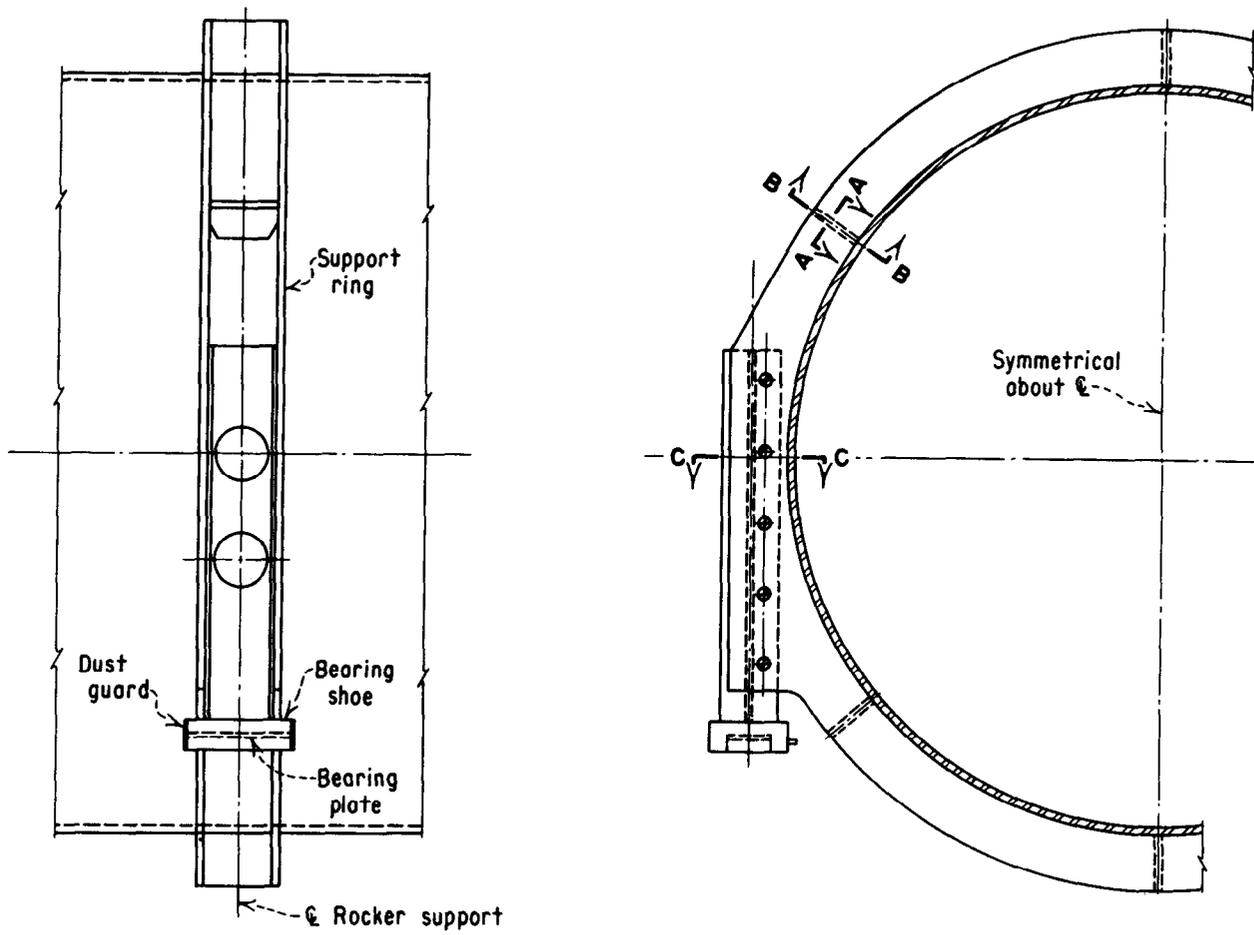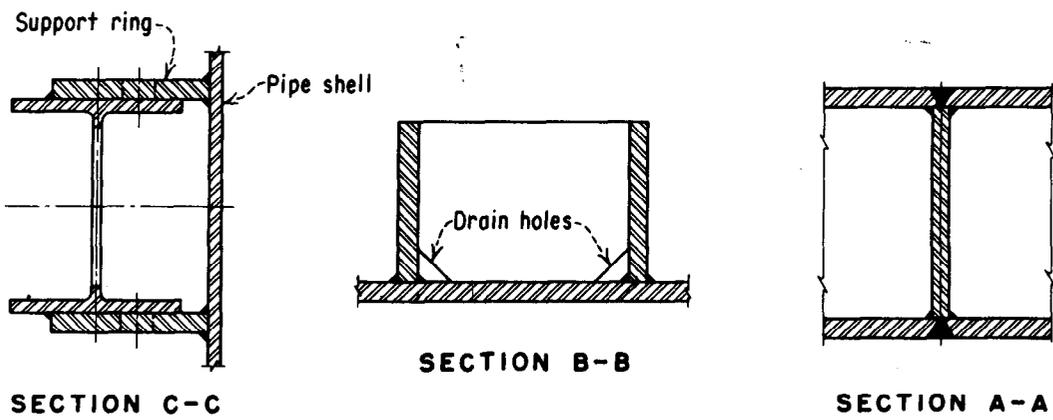
SECTION C-C

SECTION B-B

SECTION A-A

TYPICAL SECTIONS

FIGURE 20.—*Typical ring girder and column support.*

Lubricate pin 5 or 6 with graphite grease when assembling

FIELD NOTE: Bend 13 and 14 so that uphill adjusting bolt is normal to the uphill face of pier.

----Uphill face of pier
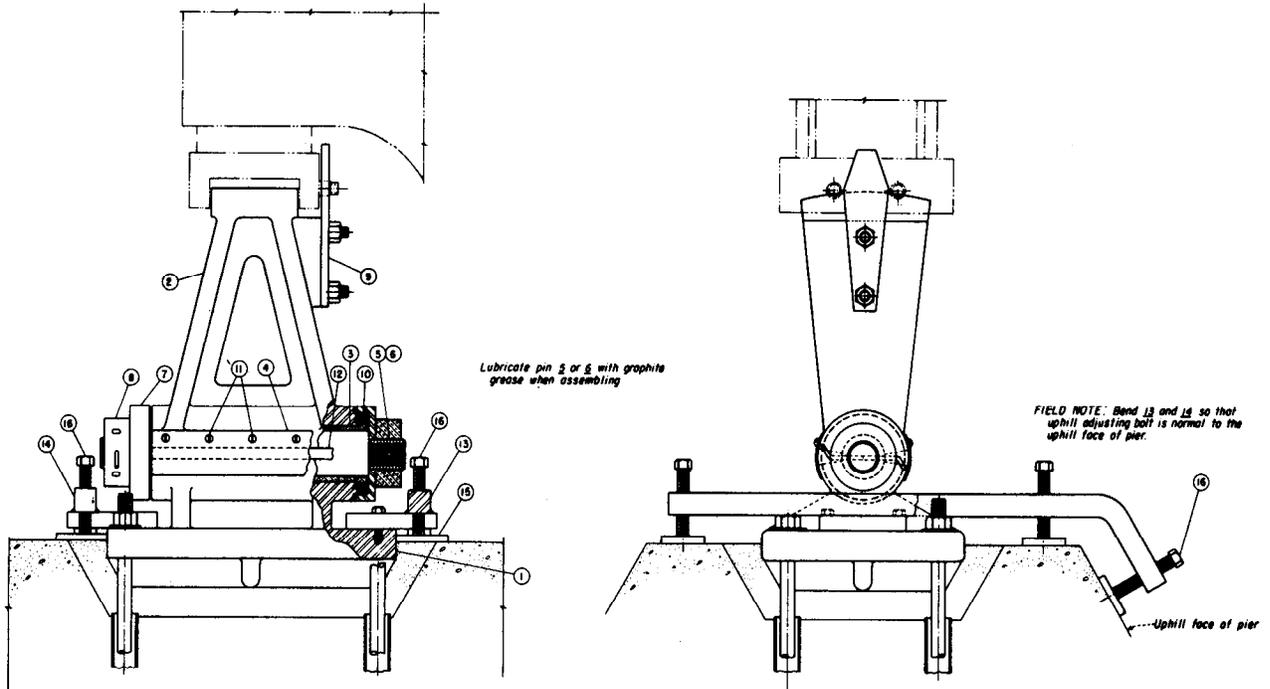
FIGURE 21.—*Typical rocker support. The angle yoke is used only for alinement during grouting.*

Packing gland----

----Retainer ring

----Outer sleeve

----Inner sleeve

-----Inside of pipe shell

----Packing rings
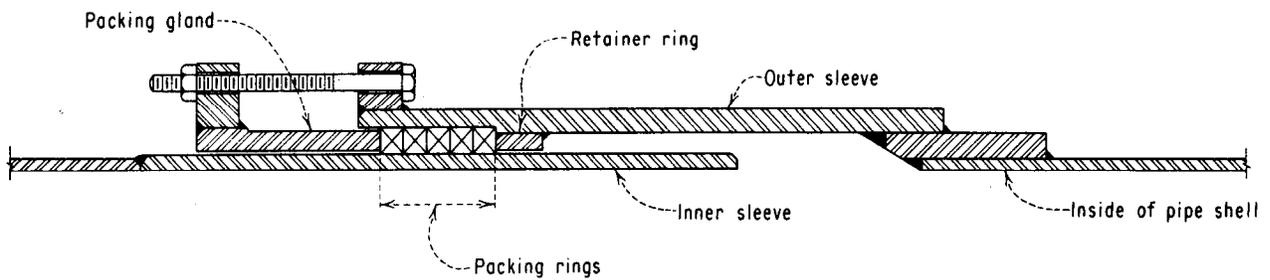
FIGURE 22.—*Typical sleeve-type expansion joint.*

----Stiffener ring

----Ladder rung

----Symmetrical about ₵

----Packing gland

Retainer ring---

----Outer sleeve

----Inside of pipe shell

----Inner sleeve

----Packing rings
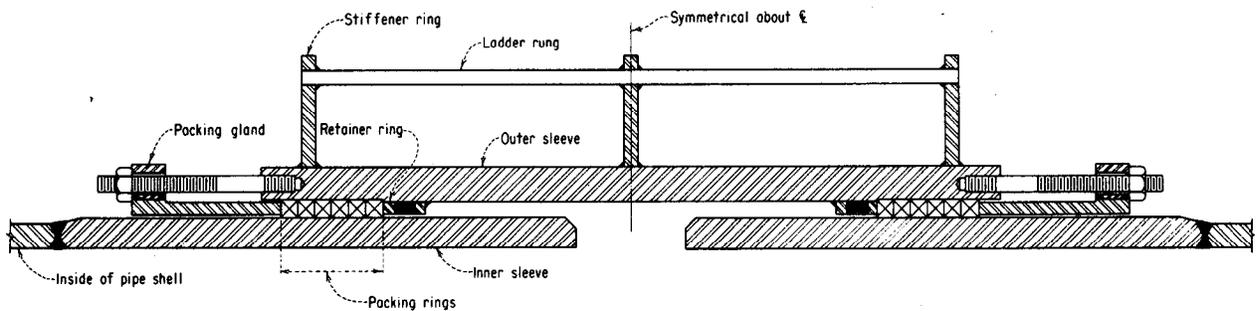
FIGURE 23.—*Flexible sleeve-type expansion joint with two stuffing boxes used to permit longitudinal temperature movement and transverse deflection.*

may also be designed with two stuffing boxes, as shown in figure 23, to permit longitudinal temperature movement and transverse deflection in the line. Such a flexible joint is desirable where a penstock passes through a construction joint separating the concrete masses of a dam and a powerhouse. As the dam is built on a foundation considered to be elastic, its downstream toe will deflect vertically with respect to the powerhouse when subjected to reservoir pressure.

Sleeve-type expansion joints have been used successfully for penstocks up to 22-foot-diameter, as at Davis Dam, and for heads up to 700 feet. This type of expansion joint can be used only on pipes which are accessible for tightening and replacing the packing. Also, the lubricated flax packing may lose its plasticity and water-sealing effect after a period of service. This is particularly true where the

pipeline is frequently empty and exposed to the direct rays of the sun.

The sleeve-type expansion joint must be well-fitted with reasonably close tolerances to insure watertightness under high heads. If it is deemed necessary to machine portions of the expansion joint in order to obtain watertightness, postweld heat treatment will assist in maintaining dimensional tolerances.

BENDS, BRANCH OUTLETS, AND WYES

Pipe Bends

Changes in direction of flow are accomplished by curved sections commonly called bends. Plate steel bends are made up of short
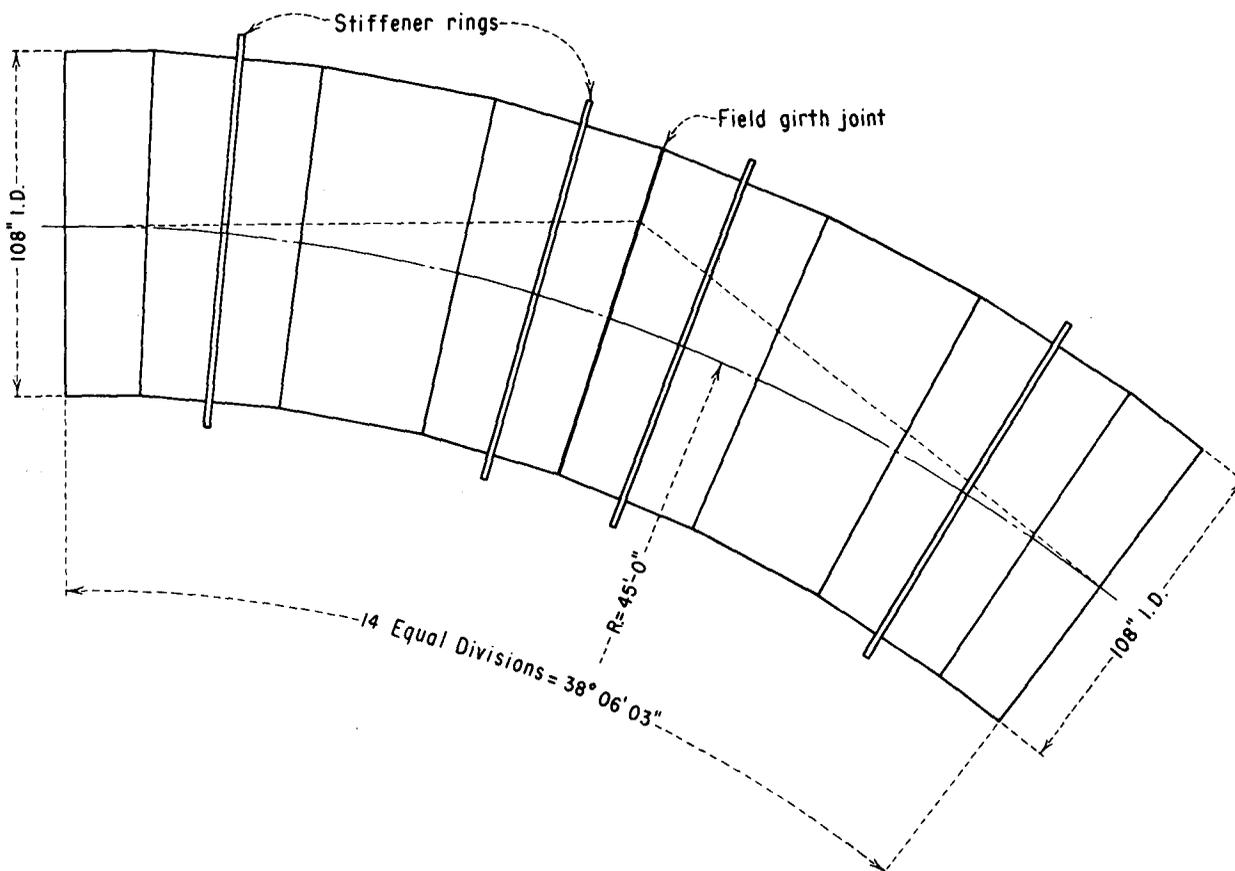
FIGURE 24.—Constant diameter bend with the radius of the bend five times the diameter.
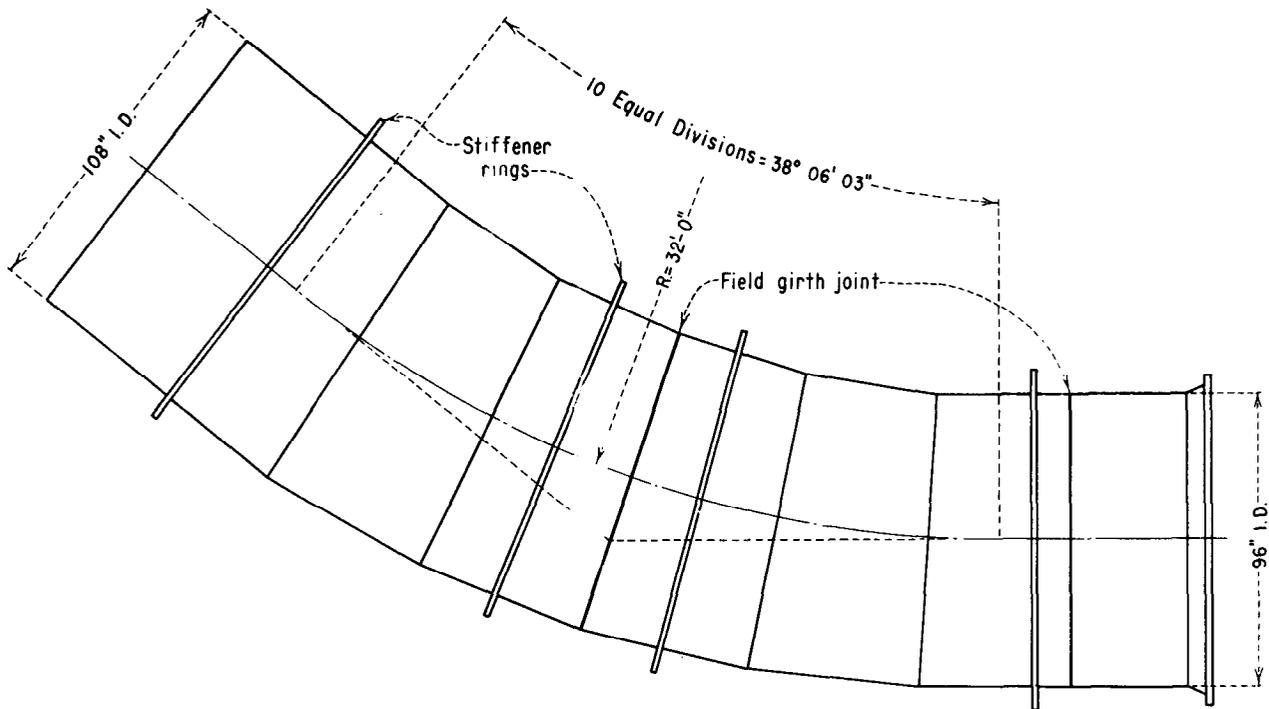
FIGURE 25.—*Bend reducing in diameter from 9 feet to 8 feet, the radius equal to four times the smaller diameter.*

segments of pipe with mitered ends. To conserve as much of the available head as possible, bends for penstocks should be made with large radii and small deflections between successive segments. Bend radii of three to five times the pipe diameter and deflection angles of 5° to 10° between segments are recommended. Bends may be designed with a constant diameter or with a different diameter at each end. Figure 24 shows a typical constant diameter bend with a total line deflection angle of about 38° designed for a 9-foot penstock. The bend radius is 45 feet, while the seven mitered segments have deflection angles of about 5-1/2° each. Figure 25 shows a reducing bend with a reduction in diameter from 9 feet to 8 feet.

Compound or combined bends, in which the plane of the bend is neither vertical nor horizontal, require certain trigonometric computations. Usually the plan angle and profile angles are known and it is required to determine the true angle in the plane of the bend and the bend rotations. These computations and applicable formulae are shown in figure 26.

Branch Outlets and Wyes

On some large penstocks, specially fabricated branch outlets and wyes are used for diverting water from the headers. The main considerations in the design of branch outlets and wyes are structural strength to withstand the internal pressure, and proper streamlining to reduce hydraulic losses.

Since outlet openings reduce the strength of the pipe at the opening, reinforcement must be provided to compensate for the removed material. As a general rule the reinforcement should be adequate to make the connection equal in strength to that of the pipe without the opening. Several branch outlets and wyes are illustrated in figures 27 and 28, respectively. These two figures show some of the fittings commonly used and the different methods of providing necessary reinforcement. The unsupported pressure areas in the pipe shells are shown by shading, and the distribution of load in the reinforcing members is indicated graphically. The right angle tees shown in figures 27a

FIGURE 26.—*Computation method for determining true pipe angle in a compound pipe bend.*
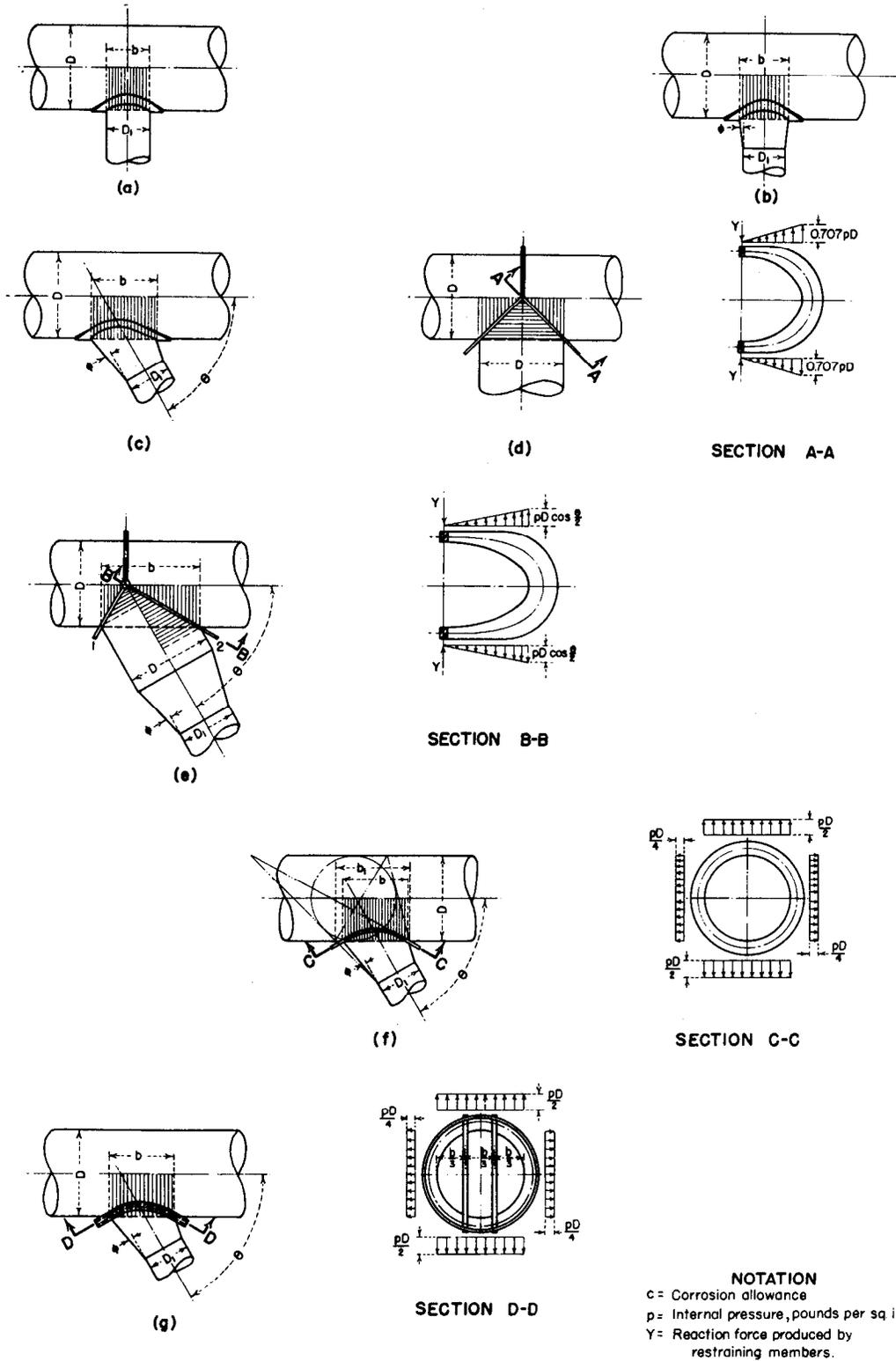
FIGURE 27.—*Loading diagrams for the development of reinforcement of branch outlets.*
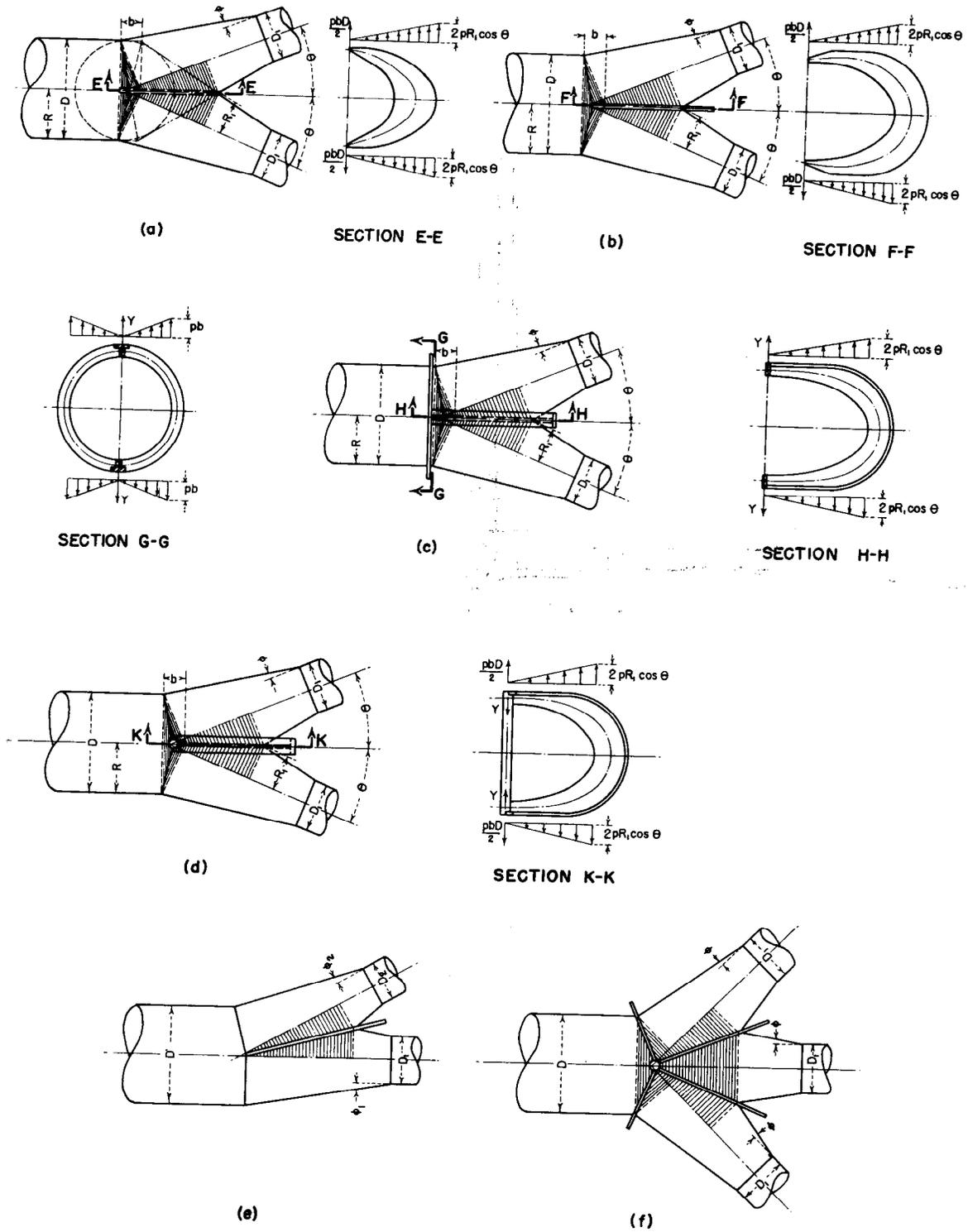
FIGURE 28.—*Loading diagrams for the development of reinforcement of wye branches in penstocks.*

and 27d are hydraulically inefficient and should be avoided whenever possible. The use of a frustum of a cone with convergence of 6° to 8°, as shown in figure 27b, reduces the branch loss to approximately one-third that of a cylindrical branch or outlet. Branch losses may also be reduced by joining the branch pipe to the main pipe at an angle less than 90° as shown in figure 27c. Ordinarily deflection angles, 0, of branch outlets, vary from 30° to 75°. However, difficulties are encountered in reinforcing branch outlets and wyes with deflection angles less than 45°. The hydraulic efficiency increases as the deflection angle decreases. Branch outlets of the type shown in figures 27f and 27g are not desirable when the diameter of the branch pipe is in excess of, say, three-fourths of the diameter of the header, since the curvature of the reinforcement becomes very sharp. In such cases the type of reinforcement shown in figure 27e is preferable. Branch outlets and wyes are usually designed so that the header and the branches are in the same plane.

Branch outlets as shown in figures 27a, 27b, and 27c may be reinforced with a simple curved plate designed to meet the requirements of the ASME code referred to previously. Branch outlets and wyes as shown in figures 27d to 27g inclusive, and in figures 28b to 28f inclusive, may be reinforced with one or more girders or by a combination of girders and tie rods. The type and size of reinforcement required depends on the pressure, the extent of the unsupported areas, and clearance restrictions. For branch outlets which intersect in a manner as shown in figures 27d and 27e, and for wyes as shown in figure 28f, three or four exterior horseshoe girders may be used, the ends of which are joined by welding. The welding of the ends of the horseshoes will be facilitated by use of a round bar at the junction. For wyes used in penstocks designed for low velocity flow, an internal horseshoe girder, sometimes called a splitter, as shown in figure 28a, may be used. Although the splitter is structurally efficient, it will cause disturbances if the flow is not equally divided between the branches.

Stress analysis of branch outlets and wyes is approximate only. Exact mathematical anal-

ysis based on the theory of elasticity becomes too involved to be of much practical value. In the approximate method, simplifying assumptions are made which give results considered sufficiently accurate for practical purposes. The reinforcement of a fitting should be proportioned to carry the unsupported loads, the areas of which are shown shaded in figures 27 and 28. The total load to be carried by the reinforcement is equal to the product of the internal pressure and the unsupported area projected to the plane of the fitting. A portion of the pipe shell is considered as acting monolithically with the girders as in the case of stiffener rings.

In the analysis of ring girder reinforcement of the type shown in figures 27f and 27g it is assumed that the curved girder acts as if it lay in one plane, that the loads in both directions are uniformly distributed, and that the ring is circular. The first of these assumptions is believed to be reasonably accurate because the ring girder is supported along its entire perimeter by the pipe shell and cannot be appreciably twisted or deflected laterally. The assumption of uniform load distribution is on the side of safety. In regard to the circularity of the ring, however, it should be noted that the ring girder is egg-shaped for branch outlets with small deflection angles, while for larger deflection angles the shape is more nearly circular and the assumption of circularity is more proper. The ring girder is statically indeterminate when used as this type of reinforcement and, for the stress analysis, the ring dimensions must first be assumed. Computations will be simplified by using a ring of constant cross section. Where tie rods are used, the deflections of the girder at the junctions with the tie rods must be calculated. These deflections, which are due to the loads in both directions, and the redundant tie-rod forces are then equated to the elongation of the tie rods and the forces in these obtained by solving the resulting equations. A symmetrical arrangement of the tie rods with respect to the centerline of the ring will simplify calculations. To provide additional strength at a branch outlet, the plate thickness of a portion of the header

PLAN

END ELEVATION

SIDE ELEVATION

SECTION A-A

SEC. G-G
DETAIL X

SEC. B-B

SEC. C-C

SEC. D-D   SEC. E-E   SEC. F-F
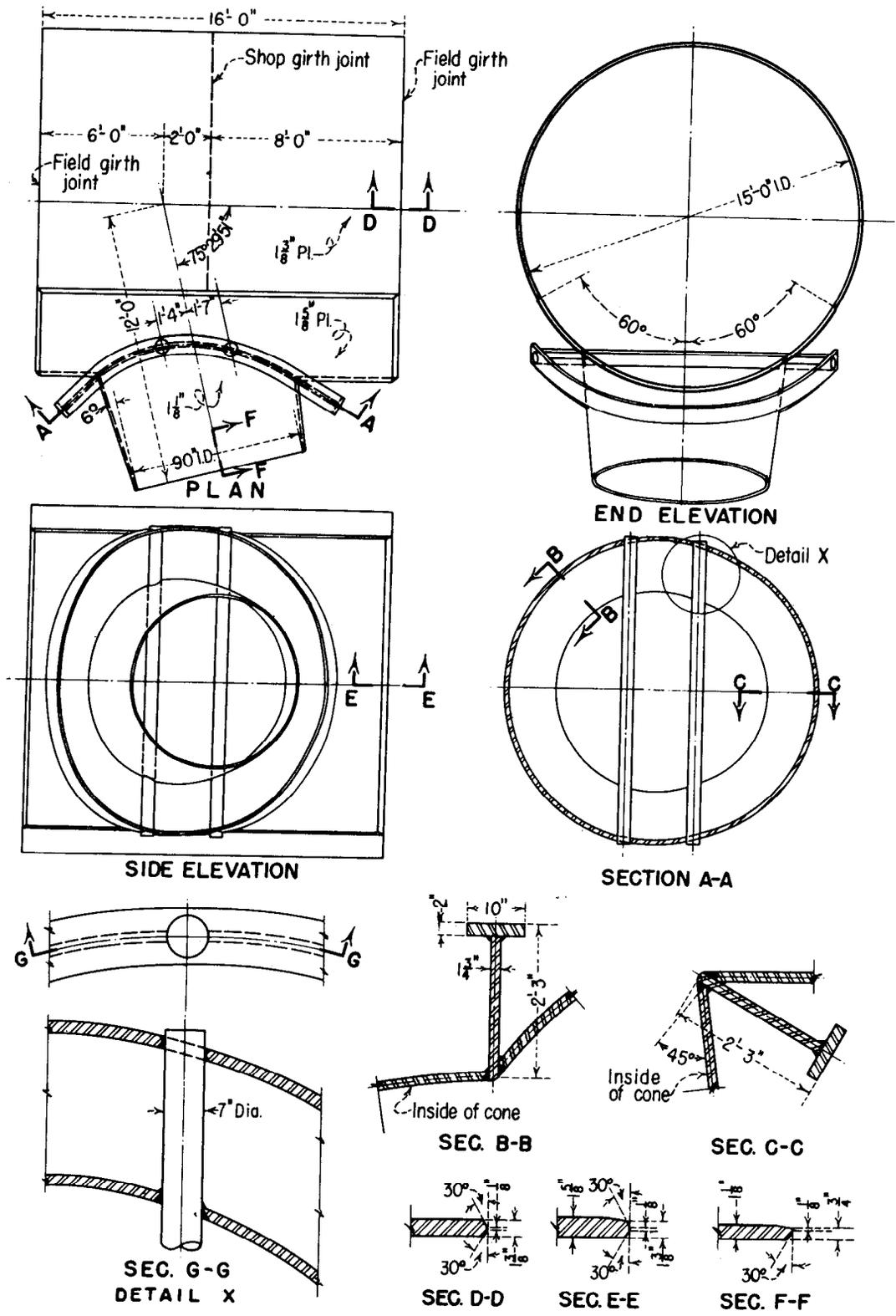
FIGURE 29.—*Typical internal and external reinforcement for a branch outlet.*

around the outlet openings and of the joining branch pipes is sometimes increased as shown in figure 29.

The reinforcement for the wyes shown in figures 28a and 28b are statically determinate. The bending and direct stresses for any section due to the loadings shown can be calculated without difficulty. The increase in the bending stress caused by the small radius of curvature at the throat of the member can be evaluated by using a correction factor in the bending formula for straight beams.

The method of analysis briefly discussed above has been used extensively in the design of special fittings. Branch connections are hydrostatically tested at pressures equal to 1½ times the design pressure. In some cases strain gage readings have been taken during the hydrostatic test and results of these tests lead to the conclusion that the approximate methods of analysis of the reinforcing members are satisfactory.

The design of branch outlets and wyes is discussed in a number of publications (13, 14, 15).

PENSTOCK ACCESSORIES

For Installation and Testing

Among the penstock accessories which should be given consideration in design are the following:

a. Temporary supports are generally required for penstocks embedded in dams or tunnels. They should be designed to carry only the empty pipe and to anchor the pipe to prevent flotation during placement of concrete. These supports remain in place and are concreted in with the pipe.

b. Standard dished test heads are used in hydrostatic pressure tests of the installed penstocks.

c. Piezometer connections for turbine performance tests, one type of which is shown in figure 30, should be placed in straight sections of pipe away from bends and branch outlets.
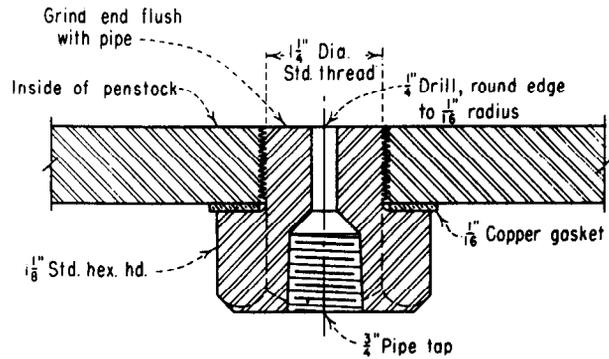
FIGURE 30.—*Installation of a piezometer connection in shell of penstock for turbine performance tests.*

For Operation and Maintenance

Proper maintenance of penstocks requires the following accessories:

a. Filling lines, which are used to fill the penstocks from the reservoir and place them under balanced pressure to facilitate opening of intake gates. They should be provided with suitable control valves.

b. Air inlets located below the intake gate admit air into the penstocks and prevent negative pressures during draining and admit air during emergency gate closure. In addition, air pipes or air valves should be installed at summits in the penstocks to release air during filling and admit air during draining.

c. Drains at the upstream and downstream ends of penstock to handle any water leaking past the intake gate.
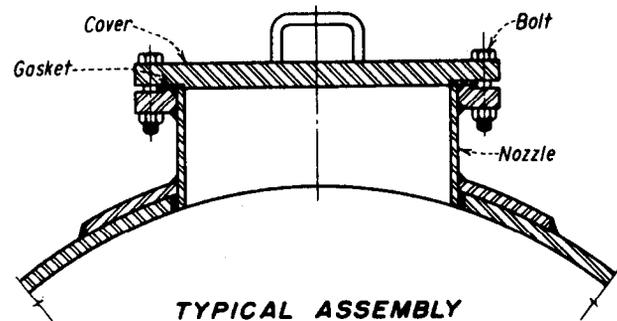
FIGURE 31.—*Typical manhole designed for inspection and maintenance.*

d. Manholes for inspection and maintenance work shall be located to facilitate ventilation and the entry of men and materials during inspection and maintenance work. One type is shown in figure 31.

e. Walkways and stairs are required for inspection and maintenance of large penstocks in open tunnels or above the ground.

f. Service connections are usually required to provide a source of water for the power-plant, dam, or municipal use.

g. The reinforcement of all openings in a penstock such as manholes, connections for drains, waterlines, filling lines, and air vents shall be designed in accordance with the ASME code.

DESIGN OF PIERS AND ANCHORS

General

Penstocks installed above ground or in open tunnels are usually supported on piers spaced from 20 to 60 feet apart. At bends, anchors are used to resist the forces which tend to cause displacement of the pipe. Sometimes anchors are also placed at intermediate points between bends. Piers and anchors are usually of reinforced concrete. As the sizes of piers and anchors are influenced by the type of soil on which they rest, information regarding the soil is required before a design can be prepared. It is preferable to construct piers and anchors on rock foundations wherever possible. Bases should be placed at elevations sufficiently below the frostline to protect the structures against frost damage.

Support Piers

Piers are designed to support the deadload of pipe and contained water and resist the longitudinal forces resulting from temperature change. Earthquake forces may be considered in the design in areas subject to seismic disturbances. For large thinshell pipes and pipes supported on high piers or bends, lateral windloads on the empty pipe may affect the design of the supports. The magnitude of longitudinal forces in a penstock provided with expansion joints is dependent on the methods of support between pipe and pier. Piers which carry the pipe directly on the concrete or on steel bearing plates are subjected to large longitudinal forces when axial movement of the pipe occurs. These longitudinal forces may be reduced by placing lubricated plates or other low friction factor material between pipe and pier or by use of rocker and roller supports. The friction coefficients recommended in each case have been mentioned in the section on design of pipe shell. After all the forces acting on the supports have been ascertained, the proportions of the pier may be determined from the moments and shear forces. Piers should be designed so that the resultant of the vertical load (including the weight of the pier and the prism of earth over the base) and the
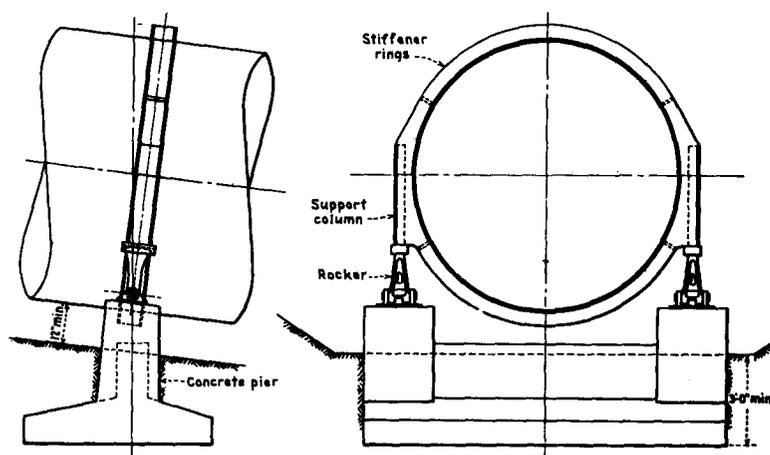
FIGURE 32.—*Typical monolithic pier construction for rocker supports.*

longitudinal and lateral forces will intersect the base within its middle third. Figure 32 shows a typical concrete pier designed for an 8-foot penstock supported on rockers. This pier is of monolithic construction with two grout recesses on top for the rocker bearings, which are fixed in place with anchor bolts.

Anchors

Freely supported penstocks must be anchored at bends, and sometimes at intermediate points, to prevent shifts in the pipeline during installation and to resist the forces which tend to cause displacement in a bent pipe under pressure. Displacement at bends may be caused by a combination of forces. They may result from temperature changes, or from gravitational, dynamic, or hydrostatic forces. The spacing of anchors in long tangent sections between bends depends primarily on the magnitude of the longitudinal forces in the line. For buried pipes the usual practice is to provide anchors only at horizontal bends with large deflections, using a sliding coefficient of 1:00, and at vertical bends where considerable uplift is expected. If, during installation, a long pipe section is exposed to the atmosphere for some time before backfilling, it should be anchored in place and should have one field girth joint between each pair of anchors left unwelded to permit free movement with changes in temperature. After the pipe has been backfilled and its temperature brought to normal, the open joints may be welded, coated, and backfilled.

Figure 33 shows a resolution of forces acting on an anchor under conditions of expansion and contraction. As has been stated in the case of piers, the resultant of these forces should intersect the base of the anchor within its middle third. The weight of the pipe and contained water and the weight of the concrete in the anchor itself are included in the combination of forces. The size of the anchor is also influenced by the sliding coefficient of friction between the concrete and the underlying soil. This sliding coefficient varies from 0.30 for clay to 0.75 for a good rock base.

Anchors may be of the type which encases the entire circumference of the penstock or they may be of the type which is in contact with only a lower segment of the circumference as shown in figure 34. This latter type of anchor may be constructed before the penstock is installed, in which case recesses are provided in the concrete for grouting the pipe and stiffener rings in place after installation. The stiffeners will assist in transferring the longitudinal forces from the pipe to the anchor.

MATERIALS

Steel Plates

The types of steels used by the Bureau of Reclamation for penstocks conform to the Standard Specifications of the American Society for Testing and Materials (ASTM) listed in the bibliography. Low carbon steels are considered to be the most satisfactory because of their favorable fabrication and welding characteristics and their high ductility. The steels most often used conform to ASTM Specifications A-285, Grade B or C or A-201, Grade B. Firebox quality grades are always used because of the more restricted limits on chemical ingredients and the more extensive testing required to assure a greater degree of uniformity of composition and homogeneity. The A-285 steel, which is a semikilled steel, is often specified where plate thicknesses are not greater than about 1 inch. For thicknesses exceeding 1 inch the A-201 steel, which is fully killed, is preferred as it has greater notch toughness than the A-285 steel which minimizes the danger of brittle failures. A-201 steel made to fine grain melting practice in accordance with ASTM Specifications A-300 has been used for special fittings such as branch outlets and wyes fabricated from thick plates. The use of the higher grade fine-grained steel in these fittings is considered justifiable to reduce the danger of brittle failures and because radiographic inspection of welds is often not feasible. Steel is currently available under the ASTM Designation A-201; however, revisions and consolidations of

PROFILE OF PENSTOCK

PROFILE
EXPANDING CONDITION

PROFILE
CONTRACTING CONDITION

PROFILE
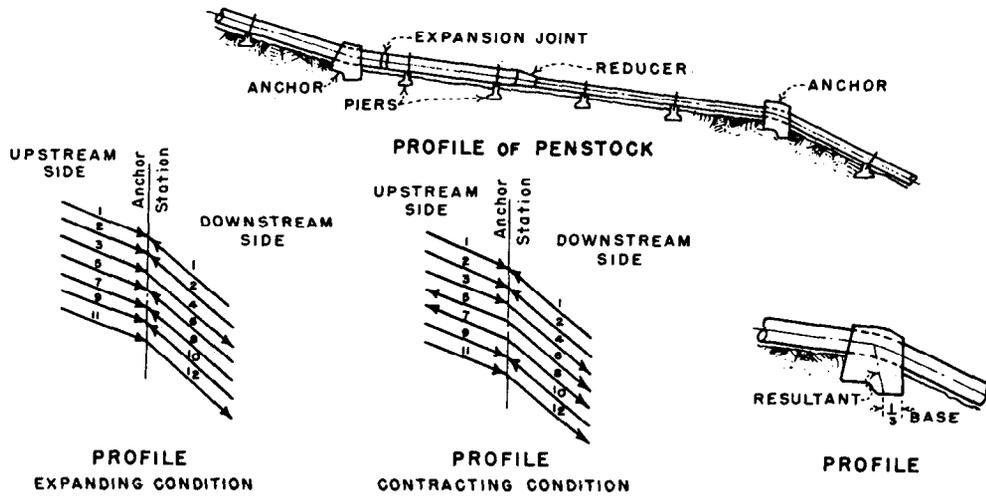
FORCES ON ANCHOR

(1) Hydrostatic force acting along axis of pipe on each side of bend
$$= w A H$$

(2) Dynamic force acting against outside of bend
$$= \frac{q w v}{g}$$

(3) Force due to dead weight of pipe from anchor uphill to expansion joint, tending to slide downhill over piers
$$= P \sin x$$

(4) Force due to dead weight of pipe from anchor downhill to expansion joint, tending to slide downhill over piers
$$= P' \sin y$$

(5) Sliding friction of pipe on piers due to expansion or contraction uphill from anchor
$$= f \cos x \left(P + W - \frac{P}{2} \right)$$

(6) Sliding friction of pipe on piers due to expansion or contraction downhill from anchor
$$= f \cos y \left(P' + W' - \frac{P'}{2} \right)$$

(7) Sliding friction of uphill expansion joint
$$= \frac{f' \pi (d + 2t)}{12}$$

(8) Sliding friction of downhill expansion joint
$$= \frac{f' \pi (d + 2t)}{12}$$

(9) Hydrostatic pressure on exposed end of pipe in uphill expansion joint
$$= \frac{w H \pi t (d+t)}{144} = w a H$$

(10) Hydrostatic pressure on exposed end of pipe in downhill expansion joint
$$= w a' H$$

(11) Longitudinal force due to reducer above anchor
$$= w H (A' - A)$$

(12) Longitudinal force due to reducer below anchor
$$= w H (A - A'')$$

DEFINITIONS OF SYMBOLS

f = coefficient of friction of pipe on piers.

f' = friction of expansion joint per lin. ft. of circumference = approx. 500 lb.

w = weight of water per cu. ft. = 62.4 lb.

A = cross sectional area of pipe in sq. ft. at anchor.

A' = cross sectional area of pipe above upper reducer, in sq. ft.

A'' = cross sectional area of pipe below lower reducer, in sq. ft.

H = maximum head at any point, including water hammer, in feet.

t = thickness of pipe shell, in inches.

q = flow in cubic feet per second.

v = velocity in feet per second.

g = acceleration due to gravity in feet per second = 32.16.

P = dead weight of pipe from anchor uphill to expansion joint, in pounds.

W = weight of water in pipe P.

P' = dead weight of pipe downhill from anchor to expansion joint, in pounds.

W' = weight of water in pipe P'

x = slope angle above anchor.

y = slope angle below anchor.

p = weight of pipe and contained water, from anchor to adjacent uphill pier, in pounds.

p' = weight of pipe and contained water, from anchor to adjacent downhill pier, in pounds.

d = inside diameter of pipe in inches.

a = cross sectional area of pipe shell at uphill expansion joint, in square feet.

a' = cross sectional area of pipe shell at downhill expansion joint, in square feet.

C = weight of anchor, in pounds.

FIGURE 33.—*Resolution of forces on pipe anchors.*
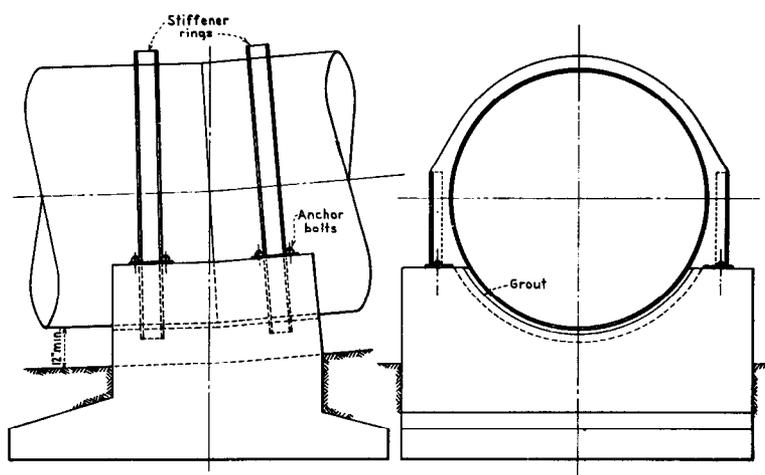
FIGURE 34.—*Typical concrete anchor.*

various specifications will eventually require that coarse grain steels be specified in accordance with ASTM A-515, and that fine grain steel be obtained under ASTM A-516. Both specifications include firebox quality material and the Grade 60 steel has tensile properties comparable to the ASTM A-201 Grade B steel.

A number of low alloy steels suitable for pressure vessel construction are available and approved by the code. However, the objections of greater cost and the fabricating difficulties previously experienced are gradually becoming reconciled and the utilization of low alloy steels is becoming more common under certain design conditions.

Flanges, Fittings, Valves, and Other Appurtenances

Flanges and welding nozzles or saddles are usually of forged or rolled steel, conforming to "Standard Specifications for Forged or Rolled-Steel Pipe Flanges, Forged Fittings, and Valves and Parts for High Temperature Service", ASTM A-105. Cast-steel flanges, if used in place of forged steel, should conform to the "Standard Specifications for Carbon-steel Castings Suitable for Fusion Welding for High Temperature Service", ASTM A-216.

Structural steel used for supports should conform to "Tentative Specification for Structural Steel", ASTM A-36. The bushings for the rocker pins are usually made from bridge bearing bronze, Class C, conforming to the "Standard Specification for Bronze Castings for Bridges and Turntables", ASTM B-22.

Rocker pin bushings may be of manganese bronze, Federal Specification QQ-B-726e. Flax packing used for expansion joints should conform to Federal Specification HH-P-106c. Sheet packing for gaskets should conform to Federal Specification HH-P-151e. Bolts, studs, and nuts used for pipe connections and expansion joints are usually Class B, conforming to Federal Specification FF-B-571a (1). For platforms and stair treads, galvanized steel grating which conforms to Federal Specification RR-G-66lb should be used.

FABRICATION

Structure and Arrangement

Penstocks may be fabricated in the manufacturer's shops or in a field fabricating plant near the dam, depending on the size of the pipe. Pipes up to approximately 12 feet in diameter may be shipped by rail. These are usually shop-fabricated in sections or laying lengths of from 20 to 40 feet. Fabrication of penstocks requires a variety of special machines and equipment, such as rolls, presses, flame-cutting tools, welding machines, testing, radiographic, and handling equipment. If the

plates are to be assembled into sections of pipe at a field fabricating plant, it is usually economical to prepare the plates and appurtenant fixtures in the manufacturer's home plant and ship them to the field plant for assembly. This preparation in home plant usually consists of cutting the plates to size, preparing the edges for welding, and rolling in two or more circular segments, the number of segments being dependent on the diameter of the penstock and length of the plates available. In some cases, it may be possible to completely fabricate pipe sections of large diameter in the contractor's home plant provided they can be shipped to the site of installation. Length of the sections is influenced by various factors such as the width of plates, loading economy, capacity of testing and field handling equipment, etc. Steel mills publish information concerning the availability of sheared plates of various thicknesses, widths, and lengths and the selection of plates conforming with mill limitations will provide an economic advantage.

If pressure tested in a testing machine, the length of the section will be limited by the length of the machine. Bends or sections exceeding the capacity of the testing machine may be tested by closing both ends with bulk-heads. This limitation should be considered in the design. If penstock sections are to be installed and welded under a separate contract, it will be necessary to predetermine their length.

Fabrication includes cutting the plates to exact dimensions, preparing the edges for welding, pressing and rolling the plates to the required radius, and welding the plates together. The type of edge preparation required depends on the welding procedure to be used. Shop joints are usually welded with semi-automatic or automatic welding machines, while field joints are manually welded. In shop welding, after the plates have been rolled, they are tack-welded into pipe courses, figure 35, then welded by automatic machines, figure 36. With the submerged arc-welding process, the joints are either machine welded on either side, and the opposite side welded manually, or are welded by one pass of the machine on both inside and outside. If a multiple-pass machine is used, the weld metal is deposited in successive layers, the number of layers depending on the thickness of the plates being welded. In the installation of a penstock in a tunnel, there may not be sufficient clearance between penstock and tunnel walls to back-
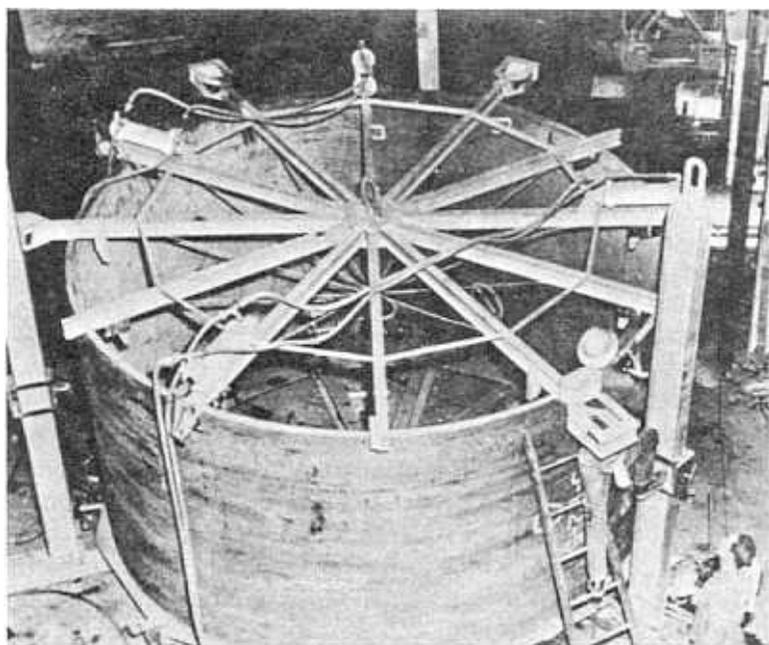
FIGURE 35.—*Shop fabrication of large diameter penstock.*
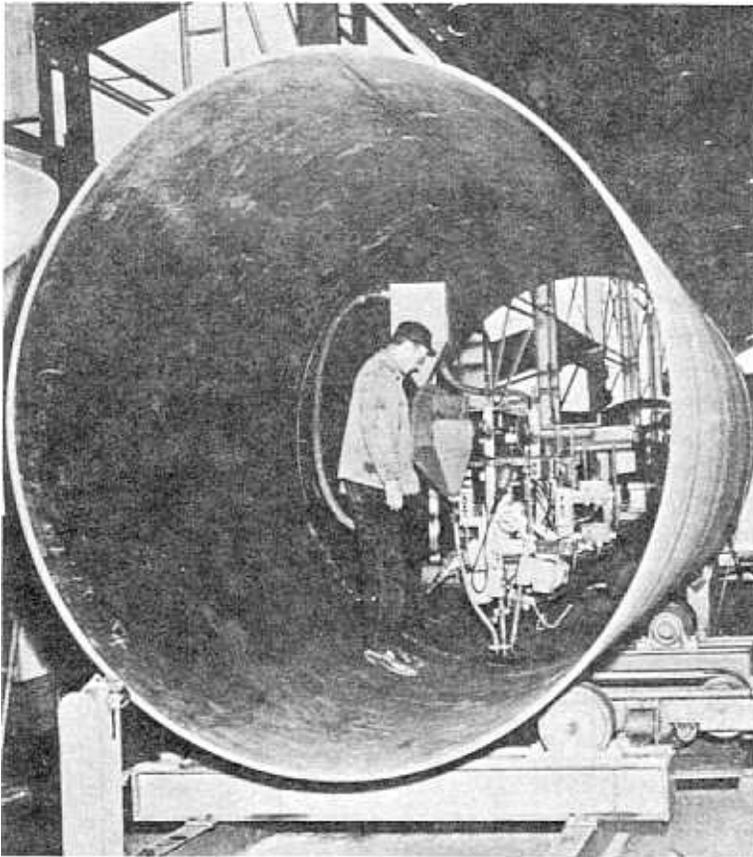
FIGURE 36.—*Shop joints are welded with automatic equipment.*

weld field girth joints from the outside. In such cases, it will be necessary to complete the weld from the inside using an outside backing strip.

Stiffener rings may be formed from bars or flame-cut from plates. The segments so produced are butt-welded into full rings, then welded to the pipe with fillet welds either manually or by machine.

Special rounding-out spiders are often used to aid fitup and welding work and to prevent distortions in the pipe. Makeup sections with excess laying lengths are provided for long tangents to provide for discrepancies in length between anchors. They are also useful between penstocks and turbines or control valves for the same purpose. Flanges for connection to turbines or valves should be faced after welding to the pipe to eliminate distortions due to welding. This facing operation will be facilitated by welding the flange to a short section of pipe, which may be provided with some excess length in order to adjust misalinements between pipe and flanges in the field.

Nondestructive Inspection of Welds

After completion, butt-welded joints are usually radiographed to detect defects in the welds. The same standards of inspection are applied to longitudinal and circumferential joints. If joints are fully radiographed the joint efficiency may be increased from 70 percent for nonradiographed joints to 100 percent. The corresponding reduction in plate thickness and weight is often sufficient to defray the cost of inspection. Weld defects may consist of slag inclusions, cracks, gas pockets, porosity, incomplete fusion, and undercutting. Cracks, incomplete fusion, and undercuts are

not acceptable but a certain amount of porosity, slag inclusions, and cavities may be acceptable if their size and distribution is such as not to impair the strength of the weld. Criteria for judging the acceptability of defects are given in the ASME code. Unacceptable defects are removed by chipping, machining, or flame gouging. After a defective area has been re-welded the repair is reradiographed to check its quality. Radiographic inspection always precedes post weld heat treatment and hydro-static testing.

The inspection may be performed by using either X-rays or gamma rays. Either method can be relied on to detect thickness differences of 2 percent if the proper technique is used. To enable the inspector to judge the accept-ability of radiographs it is essential that pene-trameters whose images appear on the film be placed on the work. Gamma rays yield pic-tures which show considerably less contrast than those produced with X-rays in steel up to about 1 inch thick. In steel from 1 inch to 2-1/2 inches thick there is little difference in sensitivity between the two methods. Above 3 inches the gamma rays are normally used exclusively. Formerly, radium was the only source for gamma rays. Now radioactive isotopes, whose cost is much lower, are avail-able. The most frequently used radioactive isotopes are iridium 192 and cobalt 60. The practical thickness range of steel for iridium is from 3/8 inch to 2-1/2 inches and for cobalt from 2 to 6 inches.

Equipment for radiographic inspection is usually portable. Figure 37 shows an X-ray machine used for the shop inspection of welds in a penstock section. Smaller compact units are available for field use. Radium and radio-active isotopes are contained in metal capsules, which are stored and transported in portable lead containers. Proper care must be taken to protect personnel from the dangerous ef-fects of X-rays and gamma rays.

Films 4-1/2 by 17 inches in size are gen-erally used which permit a 15-inch length of effective exposure, assuming an overlap of 1 inch at each end. Films of the slow-burning type are preferred to reduce fire hazards. All
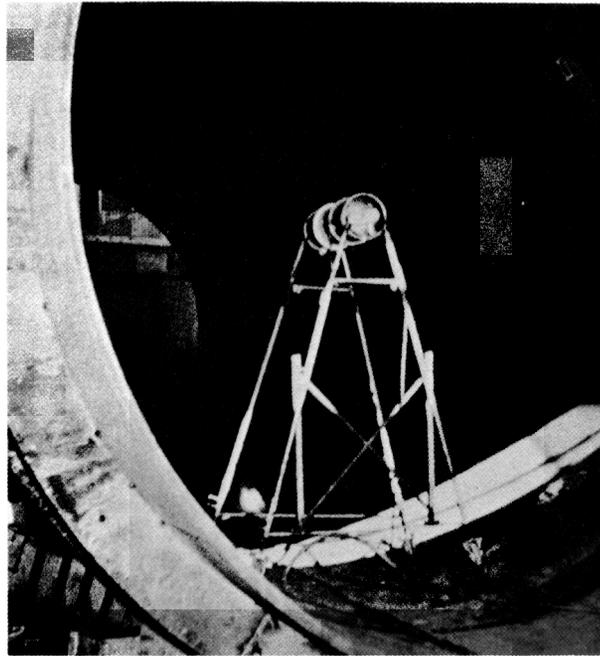
FIGURE 37.—*Radiographic inspection of welds is per-formed using portable X-ray equipment.*

films should be marked with identification numbers in accordance with a marking dia-gram of the penstock so that defects appearing on the radiographs may be accurately located. A complete set of radiographs for each job should be retained and kept on file for a period of 5 years.

For welds that cannot be satisfactorily in-spected by radiography, as for example some of the welds on branch outlets and wyes, other nondestructive methods of inspection can be used. Methods available are: magnetic par-ticle inspection, ultrasonic inspection, and var-ious methods using dye penetrants. Of these, magnetic particle inspection will only dis-close defects close to or extending to the surface. The ultrasonic method requires con-siderable experience on the part of the in-spector but it is suitable for detecting internal defects. Dye penetrants are suitable only for locating surface discontinuities. Table 2 lists some of the advantages, disadvantages, and limitations of some of the methods of non-destructive testing.

TABLE 2.—*Testing methods*

Feature	Hydrostatic test	Radiograph	Magnetic flux	Dye penetrant	Ultrasonic
Sensitivity	Only serious defects revealed.	Certain defects may remain undetected.	Primarily for surface flaws.	Only for surface flaws	Certain defects may be overlooked.
Erroneous indication ...	None	None	Possible	Remotely possible ...	Possible ....
Test coverage	Entire vessel	Mainly welded seams. Could cover other areas	Welded seams only	Welded seams only	Mainly welded seams. Could cover any area
Cost	Quite expensive	Expensive	Inexpensive	Inexpensive	Fairly expensive ......
Personal training	Some	Considerable	Fair amount	Small amount	Very well trained operator
Indication of type of defect	Limited	Definite	Definite	Definite	Limited mainly by experience and skill of operator
Indication of position of defect	Definite. Must be all the way through	Possible	Primarily surface ...	Surface only	Dependent on the experience and skill of operator
Time required	Considerable	Moderate	Small	Small	Moderate ....
Permanent record of test	None	Yes	None	None	None ....
Reliability test	Yes	No	No	No	No ....

Preheating and Postweld Heat Treatment

Preheating and postweld heat treatment are not always required, being used for the heavier plates and for steels of high carbon or alloy content. However, penstocks fabricated from plates thicker than 1 inch are usually benefited by preheating before welding. This reduces the cooling rate of the welds and reduces residual stresses in the completed section. Sections to be heat treated are heated in an enclosed furnace to a minimum temperature of 1,100° F and maintained at that temperature for 1 hour for each inch of plate thickness, then cooled in the furnace to 600° F at a rate not exceeding 500° F per hour divided by the maximum thickness in inches, then cooled in still air to normal temperature. Figure 38 shows a penstock section being moved into a

the normal contraction due to welding. Peening thus controls distortion and reduces the tendency to crack. It will be most effective if performed at dull red heat (16). At this temperature, the grain size will be refined and the ductility of the weld metal increased. If performed at temperatures below dull red heat, peening decreases ductility of the weld and may cause cracking. Because it is difficult to control the temperature at which peening should be done, the requirement for peening has been eliminated from many specifications.

INSTALLATION

Handling

Upon delivery of penstock sections at the specified delivery point, they are usually transported to the place of installation by truck
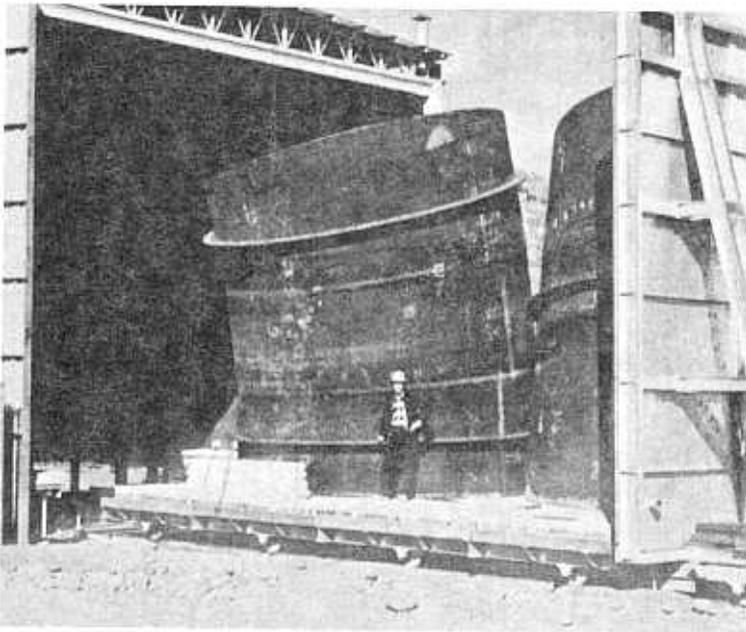
FIGURE 38.—*Postweld heat treatment is accomplished by heating in an enclosed furnace.*

postweld heat treating furnace. It is desirable that branch outlets be heat treated after fabrication so as to reduce residual stresses caused by welding.

Welding stresses may also be relieved by mechanical peening which spreads the weld and, if properly performed, counter-balances

or trailer (figure 39) and lifted in place by cableway, derrick, or other means. For installations in tunnels, special handling equipment consisting of trolleys and hoists is required. The installation work becomes complicated by limited clearances and ventilation requirements. As has been stated, the length
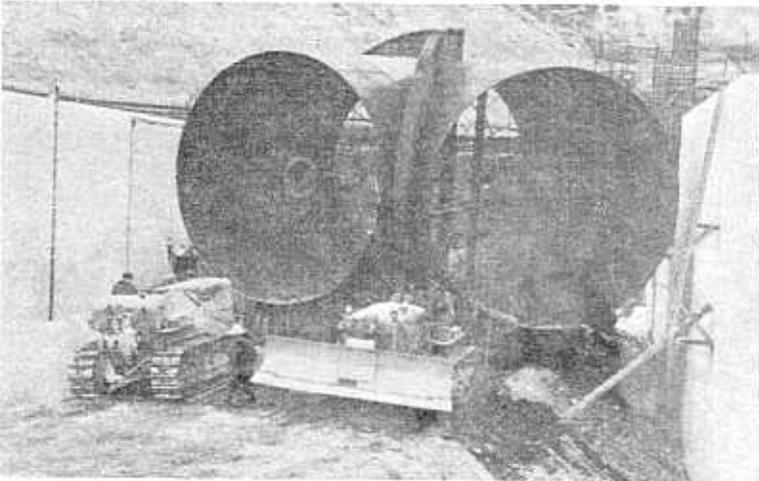
FIGURE 39.—*Transporting large wye-branch into place.*
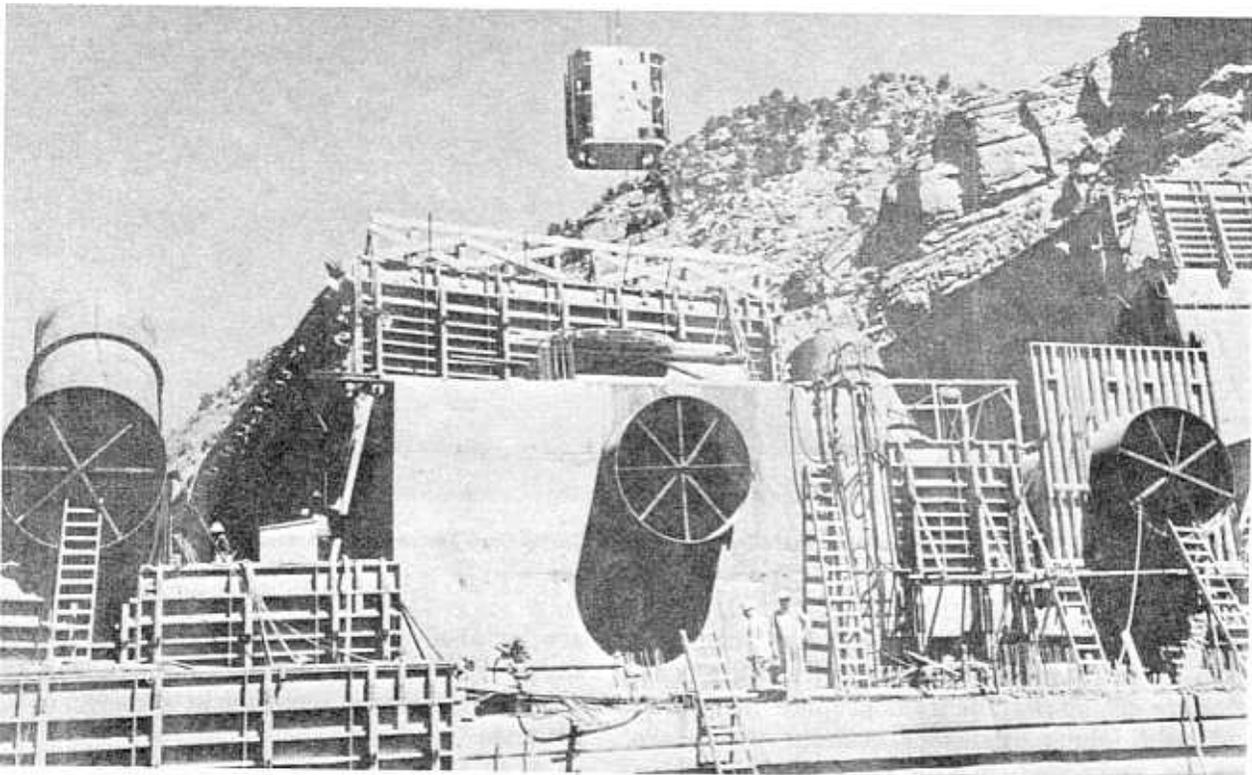
FIGURE 40.—*Penstocks being placed, showing temporary support.*

and weight of penstock sections are often dependent on the capacity of the transportation and handling equipment available. Figure 40 shows a penstock section with temporary supports in place, while figure 41 shows the installation work in progress at Flaming Gorge Dam.

Placing and Welding

After being set to line and grade on temporary supports, several pipe sections are first tack-welded together, then the joints are completed. If the specifications require radio-
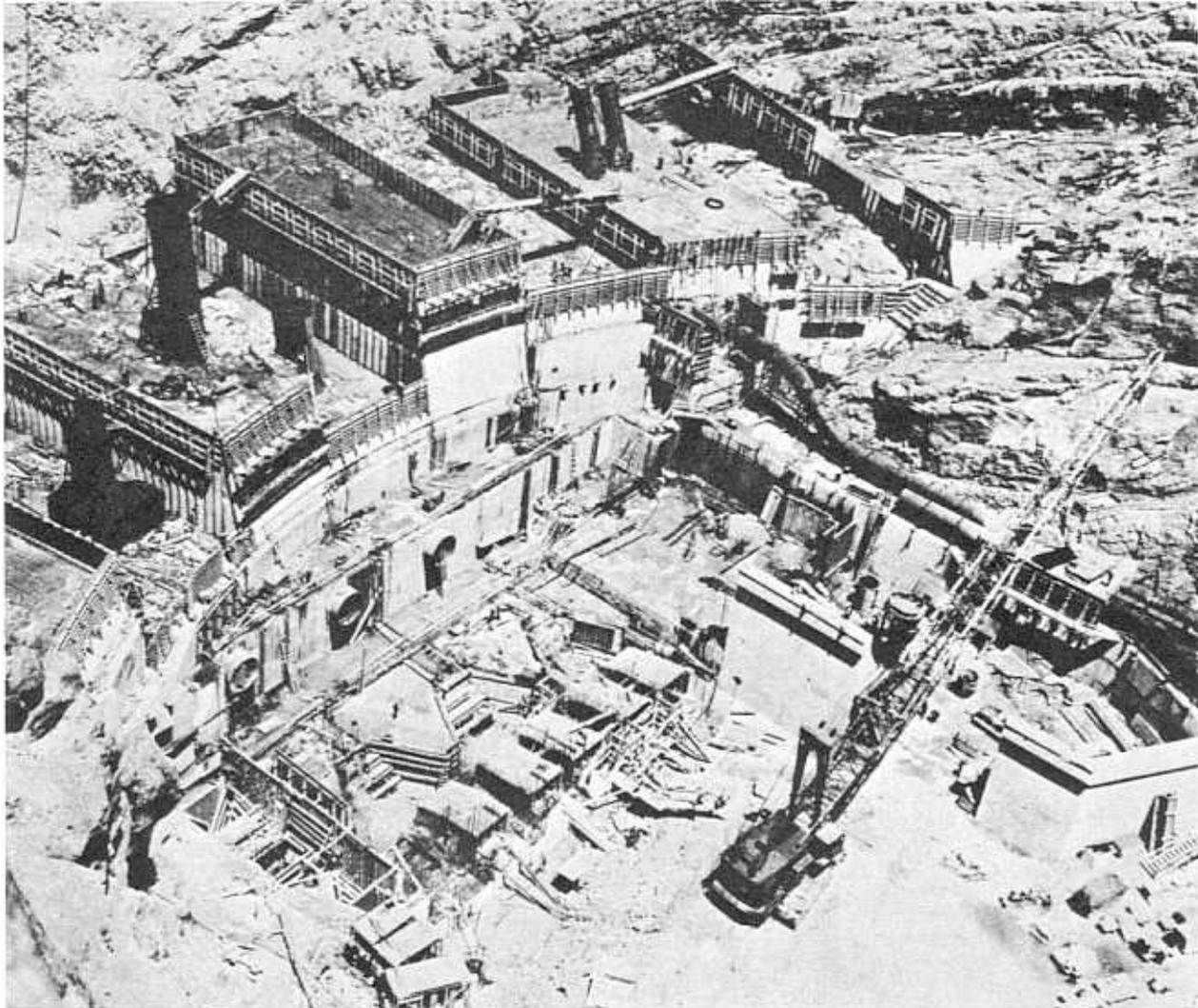
FIGURE 41.—*Penstocks installed in Flaming Gorge Dam are encased in mass concrete.*

graphic inspection, portable X-ray machines similar to those previously mentioned are used. Figure 37 shows the radiographic inspection of field joints. Penstock sections which are to be embedded in the dam are placed in position on temporary structural steel supports and welded into the line. Supports are anchored to the concrete to prevent displacement or flotation of the pipe during concrete placement. In Bureau of Reclamation dams, the concrete around penstocks is reinforced with sufficient steel to keep rim stresses around the opening within safe limits. When penstocks are installed in tunnels excavated through unstable rock, as at Parker Dam, the space around the penstock should be backfilled with concrete.

Hydrostatic Test

A proof hydrostatic test on the penstock after installation is most desirable. If the entire penstock cannot be tested hydrostatically, individual sections may be tested in the shop after they have been radiographed. Hy-

drostatic tests should be performed at a pressure sufficient to prove the adequacy of all plates and welds with the required margin of safety. The following formula may be used to determine the test pressure to be applied:

$$p = \frac{3st}{D} \quad\quad\quad\quad (10)$$

in which

p = test pressure in psi

t = minimum plate thickness in section to be tested in inches

D = internal diameter of penstock in inches

s = allowable hoop stress.

The test pressure so determined will produce in the pipe a hoop stress approximately 1-1/2 times the allowable stress. The pressure should be applied three times, being increased and decreased slowly at a uniform rate. The test pressure should be held for a length of time sufficient for the inspection of all plates, joints, and connections to detect leaks or signs of failure. It is desirable that the pressure test be performed when the pipe and water have a temperature of not less than 60° F. The penstock should be vented at high points during filling to prevent the formation of air pockets. Hammer impact tests are not recommended, as such tests do not represent operating conditions and are considered unduly severe. Objectionable defects disclosed during the pressure test should be repaired by welding, the section radiographed again, and retested.

SPECIFICATIONS AND WELDING CONTROL

Specifications

Specifications for fabrication and installation of penstocks should provide for the requirements as discussed in the above paragraphs. Specifications should be accompanied by drawings showing the general layout of the penstocks with sufficient design details to enable bidders to prepare estimates and shop drawings. In addition, the specifications should include fabrication, material, test, and erection requirements, and other essential information.

Welding Control

To insure compliance with specifications, it is necessary that all welding operations in shop and field be subjected to rigid inspection by qualified welding inspectors. Before commencement of welding, qualification of welding procedures and of operators' performance is usually required in accordance with applicable codes. For penstocks the Bureau stipulates qualification tests required by the ASME code. Repetition of tests is mandatory whenever essential variables in materials or procedures are made. The welding of test plates and the testing of specimens should be done in the presence of welding inspectors representing the purchaser. The test plates are welded in the same manner and with the same technique and electrodes as used in production welding.

Weld Tests

A number of different tests are specified in the ASME code for the qualification of procedures and welders. For groove welds, tests in direct tension are required in the procedure qualification to measure the tensile strength of the joints. Guided bend tests are used in both procedure and performance qualification tests to check for the degree of soundness and ductility. For fillet welds, fracture tests are used to detect cracks, incomplete fusion, slag inclusions and gas pockets. Etch tests are also required for fillet welds and are sometimes used on butt welds to give a clear definition of the structure of the weld.

CORROSION CONTROL FOR PENSTOCKS

Steel penstocks should be protected against corrosion by coatings appropriate to the exposure. Provision may also be made for the installation of cathodic protection in some circumstances.

The difficulty inherent in repair or replacement of penstock linings and coatings, and the

large direct and indirect costs associated therewith, dictates selection of the most durable materials which are appropriate to the particular exposure. Coal-tar enamel, with an estimated service life in excess of 50 years, stands out among current protective coatings with long, well-established service records. When suitable, such a material should therefore be specified. Frequently, however, conditions will preclude its use and selection must be made from a variety of coatings which, although costing nearly as much or more to apply, can be expected to last only about 20 years and to require more maintenance. Outstanding among such coatings are those based on vinyl resins. Lately, coal-tar epoxy paints are also gaining acceptability and showing strong promise of lengthy service, and a variety of other resin combinations are showing up well in early evaluations.

Penstocks are generally lined with coal-tar enamel, if buried or embedded, or if they will be operating continuously so as to eliminate exposure of the enamel to temperature extremes. Shop-spun enamel linings are preferable to hand applied linings for straight pipe sections because of the low hydraulic friction afforded by the glassy smooth surfaces. When above-ground penstocks will be empty at times, the selection should be a vinyl resin or other paint which is less sensitive to temperature.

The exterior of buried penstocks may also be protected by coal-tar enamel. The enamel, which protects the metal from corrosive soil constituents, must in turn be protected against soil stresses and backfill damage by a bonded felt wrapping, or by an embedded glass mat reinforcement plus the felt wrapping. Penstocks exposed above ground to atmospheric weathering may receive a synthetic resin primer and topcoats of aluminum or other material selected to furnish the required protection and also achieve any decorative effects desired.

Preparation of surfaces to receive the preceding coatings must remove all contaminants and should provide a somewhat roughened surface. Sandblasting to base metal is the only practical method known to insure this result for coatings to be in immersion or buried exposure.

Cathodic protection should rarely be required for properly coated buried penstocks. However, in special circumstances, such as when the coating may have been damaged during backfilling, the subsequent use of cathodic protection might be indicated. Provision for this eventuality may be made at small cost during construction by insuring the electrical continuity of the piping (bonding across any couplings) and attaching any necessary testing and power leads.

A Selected Bibliography and References

1. *Economic Diameter of Steel Penstocks*, by Charles Voetsch and M. H. Fresen, Transactions, ASCE, vol. 103, 1938, Paper No. 1982.
2. *A Method for the Economic Design of Penstocks*, by H. L. Doolittle, Transactions, ASME, vol. 46, 1924, Paper No. 1946.
3. *The Flow of Water in Riveted Steel and Analogous Pipes*, by Fred Scobey, U.S. Department of Agricultural Technical Bulletin No. 150, January 1930.
4. *Hydraulic Losses in Pipe Fittings*, by D. Thoma, Transactions of the Munich Hydraulic Institute, Bulletin No. 3, Translated by ASME, 1934.
5. *Loss of Head in Valves*, by Charles Ives Corp and Rolland O. Rouble, University of Wisconsin Engineering Series V.IX, No. 1, 1922.
6. *Comparison and Limitations of Waterhammer Theories*, by R. S. Quick, Mechanical Engineering, May 1927.
7. *Waterhammer Analysis*, by John Parmakian, Prentice-Hall, Inc., New York, 1955.
8. *Design of Large Pipe Lines*, by Herman Schorer, Transactions, ASCE, 1933, Paper No. 1829.
9. *Penstock Analysis and Stiffener Design*, Boulder Canyon Project Final Reports, Bulletin No. 5, Part V, Bureau of Reclamation, Denver, Colo.
10. *Strength of Materials*, by S. Timoshenko, Part II, D. Van Nostrand Co., Inc., New York.
11. *Theory of Plates and Shells*, by S. Timoshenko, McGraw-Hill Book Co., Inc., New York.
12. *Theory of Elastic Stability*, by S. Timoshenko, McGraw-Hill Book Co., Inc., New York.
13. *Design of Fabricated Plate Steel Tees, Laterals and Wyes of Large Diameter for the Pressure Aqueduct of the Boston Metropolitan District Water Supply Commission*, by C. J. Grinter and E. B. Cobb, Journal of Boston Society of Civil Engineers, vol. 28, No. 2, April 1941.
14. *Stress Analysis of Wye Branches*, by F. O. Ruud, Engineering Monograph No. 32, Bureau of Reclamation, Denver, Colo.
15. *Design of Wye Branches for Steel Pipe*, by H. S. Swanson, H. J. Chapton, H. J. Wilkinson, C. L. King, and E. D. Nelson, Journal of the American Water Works Association, vol. 47, No. 6, June 1955.
16. *Welding Metallurgy*, by G. E. Linnert, American Welding Society, New York.

Codes and Standards

1. ASME Boiler and Pressure Vessel Code, Sections VIII and IX.
2. AWWA Standard for Coal-tar Enamel Protective Coatings for Steel Water Pipe, AWWA C-203.
3. AWWA Standard Specifications for Cement-mortar Protective Coating for Steel Water Pipe, AWWA C-205.
4. ASTM A-285, Standard Specifications for Low and Intermediate Tensile Strength Carbon Steel Plates of Flange and Fire Box Qualities.
5. ASTM A-201, Standard Specifications for Carbon-silicon Steel Plates of Intermediate Tensile Ranges for Fusion-welded Boilers and Other Pressure Vessels.
6. ASTM A-300, Tentative Specifications for Steel Plates for Pressure Vessels for Service at Low Temperatures.
7. ASTM A-105, Standard Specifications for Forged or Rolled-steel Pipe Flanges, Forged Fittings, and Valves and Parts for High Temperature Service.
8. ASTM A-216, Standard Specifications for Carbon-steel Castings Suitable for Fusion Welding for High Temperature Service.
9. ASTM A-36, Tentative Specification for Structural Steel.
10. ASTM B-22, Standard Specification for Bronze Castings for Bridges and Turntables.
11. ASTM A-515, Standard Specification for Carbon Steel Plates of Intermediate Tensile Strength (coarse grained).
12. ASTM A-516, Standard Specification for Carbon Steel Plates of Intermediate Tensile Strength (fine grained).
13. ASTM A-517, Standard Specification for High strength Quenched and Tempered Alloy Steel Plate (fully killed, fine grained).

Federal Specifications:

QQ–B–726e	—Bronze castings, manganese and aluminum-manganese.
HH–P–106c	—Packing; flax or hemp.
HH–P–46c	—Packing, asbestos, sheet, compressed.
FF–B–571a(1)	—Bolts; nuts; studs; and tap-rivets (and material for same).
RR–G–661b	—Grating, metal.
HH–P–151e	—Packing; rubber-sheet, cloth-insert.

WELDED PLATE STEEL PENSTOCKS

(WELDED STEEL PENSTOCKS)

DAM OR POWER PLANT	PROJECT	SPEC NO. AND DATE	TYPE	DIA.	MAX. STATIC HEAD	TURBINE RATED HEAD	GATE CLOSURE TRAVEL TIME SECONDS	MAX. WATER HAMMER	ELEVATION ₵ INTAKE	ELEVATION ₵ TURBINE	NO. & H.P. OF UNITS	₵ RATED FLOW CU.FT/SEC.	NO. OF PIPES AND TOTAL LENGTH	PLATE THICKNESS MIN.	PLATE THICKNESS MAX.	TYPE AND SIZE OF SUPPORT RINGS	SUPPORTS TYPE	SUPPORTS SPACING	SIZE OF FILLING LINE	NO. OF EXP. JOINTS	SHOP TEST HYDROSTATIC PRESSURE	FIELD TEST HYDROSTATIC PRESSURE	ESTIMATED WEIGHT, LBS.	F.O.B. SHOP	DEL'V'D	INSTALLATION	TOTAL	NOTES
HOOVER	BOULDER CANYON ARIZ-NEV-CALIF.	534 JUNE,1932	IN TUNNEL	30' 25' 13' 10'2&10' 8'4"	590'	475'	5	37'	890	637	15-115,000 2-55,000	2530	4-4718' 4-2014' 16-5590' 12-1906' 12-238'	1½ 5/16 3/4 7/16	2⅝ 3⅛ 1⅜ 15/16	T-SHAPE RINGS	FIXED COLUMNS	25'-4"	30' STATION SERVICE PENST.	NONE	1½ x WORKING PRESSURE	1½ x WORKING PRESSURE	89,000,000 PLATES 82,000,000 FABRICATED	—	—	—	.153	TURBINES ARE PROVIDED WITH PRESSURE RELIEF VALVES PENSTOCKS WERE PRE-STRESSED.
HYRUM (WELLSVILLE PUMP PLANT)	HYRUM UTAH	646-0 JAN.,1935	IN OPEN	36" O.D.	48'	45'	—	—	4602	4591	1-—	45	1-453'	3/8	3/8	NONE	NONE	—	—	ONE	75 #/▫	—	37,600	.0635	—	—	—	
SEMINOE	KENDRICK WYOMING	975-D OCT.,1937	EMBEDDED IN CONCRETE	120"	211'	171'	3	15'	6181	6147	3-15,000	900	3-301	½	9/16	NONE	STEEL SADDLE (See notes)	—	10"	ONE	36,000 # D	76 #/▫ TO 92 #/▫	373,000	.059	.069	.022	.091	STEEL SADDLE DESIGNED BY CONTRACTOR
GRAND COULEE	COLUMBIA BASIN WASHINGTON	760 JAN.,1938	EMBEDDED IN CONCRETE	18'	352'	325'	4	95'	1041	938	18-150,000	4600	18-5266'	3/4	1½	NONE	NONE	—	24"	NONE	300 #/▫ & 1901 #/▫	—	16,540,000	.088	.094	.020	.114	
MARSHALL FORD	COLORADO RIVER TEXAS	42 FEB.,1938	EMBEDDED IN CONCRETE	16'	212'	—	—	—	560	502	3-30,000	—	3-471'	3/8	1"	NONE	NONE	—	10"	ONE	175 #/▫	—	PURCHASED BY LOWER COLORADO RIVER AUTHORITY, AUSTIN, TEXAS					
ELEPHANT BUTTE	RIO GRANDE NEW MEX.-TEX.	1217-D MAY,1939	EMBEDDED IN CONCRETE	73' 96' 103'	195'	156'	4	57'	4265	4212	3-11,500	630	6-790' 3-129' 3-131'	3/8 ½ 1/2	1/2 3/4	NONE	NONE	—	15"	NONE	160 #/▫	—	455,000	.063	.065	—	—	INSTALLED BY GOVT.
PARKER	PARKER DAM POWER CALIF.-ARIZONA	898 APR.,1940	EMBEDDED IN CONCRETE	22'	80'	80'	5	45'	393	375	4-40,000	4860	4-661'	5/8	1⅛	NONE	NONE	—	24"	NONE	—	—	2,200,000				.109	
SHASTA	CENTRAL VALLEY CALIFORNIA	941 JAN.,1941	EMBEDDED & IN OPEN	15'	479'	330'	4	180'	815	586	5-103,000	3200	5-4395'	3/4	2⅛	2 RINGS 1½ x 11"	ROCKER	60'	18"	TWO	44,000 # D	1½ x DESIGN PRESSURE	13,548,000	.116	.123	.020	.145	
GREEN MOUNTAIN	COLO-BIG THOMPSON COLORADO	964 JAN.,1941	IN TUNNEL	102"	254'	203'	4	197'	7708	7696	2-15,000	850	2-1675'	11/16	1¼	2 RINGS 1½ x 5"	SLIDING COLUMNS	40'	8"	ONE	44,000 # D	—	1,875,000			.042	.157	
HYRUM (WELLSVILLE PUMP PLANT)	HYRUM UTAH	1575-D OCT.,1941	IN TUNNEL	54"	81'	—	—	—	4597	4591	—	—	1-453'	3/16	3/8	2 RINGS ½ x 3"	135° SADDLE	30'&39'	—	THREE	200 #/▫	—	16,000	.305	.320	INSTALLED BY GOVT.		
ANDERSON RANCH	BOISE IDAHO	1316 JUNE,1946	EMBEDDED & IS TO 72"	15 TO 72	324'	245'	10	123'	3870	3862	3-18,500	755	6-1812'	3/8	⅝	2 RINGS 1½ x 10'	FIXED COLUMN	35'-0"	6" & 12"	THREE	—	285 #/▫	3,300,000 3,214,300	.201 .243	.207 .259	.0625 .0625	.2695 .3215	SCHEDULE NO.1 SCHEDULE NO.2
ESTES PARK	COLO-BIG THOMPSON COLORADO	1347 JUNE,1946	IN OPEN & BURIED	78"	558'	482'	5	—	8014	7477	3-21,000	442	3-11950'	3/8	1¾	2 RINGS ½ x 5"	ROCKER	60'-0"	—	SIX	40,000 # D	—	6,300,000	.11	.112	.045	.157	100% RELIEF VALVES
MARY'S LAKE	COLO-BIG THOMPSON COLORADO	1347 JUNE,1946	IN OPEN	96"	211'	210'	5	—	8232	8031	1-11,200	550	1-533'	3/8	3/8	2 RINGS ½ x 6"	ROCKER	VARIES	—	ONE	40,000 # D	—	240,000	.16	.173	.045	.218	100% RELIEF VALVE
HEART MOUNTAIN	SHOSHONE WYOMING	1441 AUG.,1946	BURIED	60" 69"	271'	265'	3	218'	5087	4964	1-8,300	340	1-243'	½	1"	NONE	NONE	—	—	NONE	40,000 # D	—	95,000	.11	.123	.12	.263	
JORDAN NARROWS PUMPING PLANT	PROVO RIVER UTAH	1906 AUG.,1946	BURIED	48"	315'	150'	—	—	4780	4490	1-1,000	65	1-548'	3/16	3/8	NONE	NONE	—	—	NONE	40,000 # D	—	158,500	.16	.173	.11	.283	
DAVIS	DAVIS DAM ARIZ.-NEV.	2104 MAR.,1948	EMBEDDED IN CONCRETE	22'	134'	125'	4	34'	548	513	5-60,000	5000	5-942'	3/4	1"	NONE	NONE	—	24"	ONE	—	—	3,125,400	—	.299	—	—	
KORTES	MO. RIVER BASIN WYOMING	1611 JAN.,1947	EMBEDDED IN CONCRETE	97" 108"	224'	200'	3	75'	6040	5942	3-18,500	940	3-700'	3/8	¾	1 RING 3/8 x 9" @ 5'-0" cra.	STRUCTURAL	16'-2½	—	THREE	40,000 # D	—	488,000	.16	.173	.05	.223	
HUNGRY HORSE	HUNGRY HORSE MONTANA	2278 JULY,1948	EMBEDDED IN CONCRETE	13'-6"	485'	400'	4	85'	3319	3080	4-105,000	2720	2-452' 2-448'	3/8	1¾	NONE	NONE	—	—	FOUR	40,000 # D	—	2,953,000	.237	—	.08	.317	
BOYSEN	MO. RIVER BASIN WYOMING	2621 MAY,1949	EMBEDDED IN CONCRETE	15'-0" 10'-6"	121'	96'	5	93'	4664.37	4619	2-10,500	1190	1-382' 2-300'	1⅜ ⅝	1" ⅝	2 RINGS 1½ x 10½	STRUCTURAL	—	—	TWO	40,000 # D	—	700,500	.196	.212	.120	.332	
CANYON FERRY	MO. RIVER BASIN WYOMING	2577 MAY,1949	EMBEDDED IN CONCRETE	13'-6"	160'	125'	4	35'	3705.97	3648	3-25,500	1970	3-494'	5/8	¾	NONE	NONE	—	12"	ONE	—	—	564,500	.16	.173	.08	.253	
ANGOSTURA	MO. RIVER BASIN SOUTH DAKOTA	3131 AUG.,1950	EMBEDDED AND IN OPEN	54"	124'	109'	3	89'	3123.77	3072.00	1-1675	168	1-104.5'	3/8	3/8	2 RINGS ½ x 4"	SLIDING SUPPORT	1 ONLY	—	TWO	40,000 # D	—						
FALCON	INTERNATIONAL BOUNDARY & WATER COMMISSION	0-4305 OCT.,1950	EMBEDDED AND BURIED	22' 13'	100' 100'	100' 100'	5 5	54' 54'	223.35 231.50	186.00 186.00	4-14,750 4-14,750	1530 1530	1-1438' 4-2006'	3/8	¾	NONE	NONE	—	—	NONE	—	—	1,558,000 2,106,000	— —	.153 .153	— .082	.235	MEXICAN SIDE AMERICAN SIDE
BLUE MESA	COLO. RIVER STORAGE COLORADO	5735 MAR.,1962	IN TUNNEL	16'-0"	358'	280'	8	96'	7519.40	7161.00	2-41,500	1540	2-1296'	3/4	1¾	NONE	—	—	—	—	45,000 # D	—	2,700,000	—	—	—	.333	STEEL LINER
MORROW POINT	COLO. RIVER STORAGE COLORADO	5915 APR.,1963	IN TUNNEL	13'-6"	405'	347'	8	72'	7080.00	6755.00	2-83,000	2,480	2-954'	7/16	1¾	NONE	—	—	12"	—	45,000 # D	—	1,472,000	—	—	—	.412	STEEL LINER

☀ Per Turbine Unit

WELDED PLATE STEEL PENSTOCKS

(WELDED STEEL PENSTOCKS)

DAM OR POWER PLANT	PROJECT	SPEC NO. AND DATE	TYPE	DIA.	MAX STATIC HEAD	TURBINE RATED HEAD	GATE CLOSURE TRAVEL TIME SECONDS	MAX WATER HAMMER	ELEVATION @ INTAKE	ELEVATION @ TURBINE	NO. & H.P. OF UNITS	RATED FLOW CU.FT/SEC	NO. OF PIPES AND TOTAL LENGTH	PLATE THICKNESS MIN	PLATE THICKNESS MAX	TYPE AND SIZE OF RING	SUPPORTS TYPE	SPEC. RING	SIZE OF FILLING LINE	NO. OF EXP. JOINTS	SHOP TEST HYDROSTATIC PRESSURE	FIELD TEST HYDROSTATIC PRESSURE	ESTIMATED WEIGHT LBS.	ACTUAL COST PER LB. SHOP	DELV'D	INSTALL	TOTAL	NOTES
SAN LUIS	CENTRAL VALLEY CALIFORNIA	5855 DEC. 1962	CONCRETE ENCASEMENT & ANCHOR	17'-6"	344	307	190-210	95'	213.68	202.00	9-66,800	2,150	4-4673'	1⅜"	1⅞"	NONE	—	—	—	—	45,000T D	—	15,258,700	—	—	—	.273	STEEL LINER
YELLOWTAIL	MISSOURI RIVER BASIN MONTANA	5525 APR. 1961	CONCRETE ENCASEMENT	12'-0"	482	440	5	83'	3450.00	3178.00	4-86,500	2,040	4-1592'	⅞"	1⅛"	NONE	—	—	12"	—	45,000T D	—	2,212,000	—	—	—	.352	
POLE HILL	COLO.-BIG THOMPSON COLORADO	3451 JUNE 1951	IN OPEN & BURIED	96"	838	825	5	205'	7408.30	6575.00	1-47,500	990	1-2039'	⅜"	1⅛"	2 RINGS ⅞"x7"	ROCKER	50'-0"	—	FIVE		LEAKAGE TEST-200%	2,065,000	—	—	—	.253	100% RELIEF VALVE
FLATIRON	COLO.-BIG THOMPSON COLORADO	3451 JUNE 1951	IN OPEN & BURIED	72"	1132	1,045	5	260'	6516.40	5457.00	2-48,000	405	2-9925'	⅜"	1½"	2 RINGS ⅞"x6"	ROCKER	60'-0"	—	29		LEAKAGE TEST-200%	10,100,000	—	—	—	.257	100% RELIEF VALVE
FOLSOM	CENTRAL VALLEY	3511 SEPT. 1951	EMBEDDED & IN OPEN	15'-6"	340	300	6	90'	307.00	153.50	3-74,000	2530	3-1675'	⅞"	1¼"	2 RINGS 1½"x10"	ROCKER	50'-0"	14"	SIX	—	—	2,203,200	—	.190	.113	.303	
ALCOVA	KENDRICK WYOMING	3564 NOV. 1951	EMBEDDED IN CONCRETE	10'-0"	164	154	5	39'	5337.58	5335.50	2-26,500	1780	1-925'	⅝"	1⅛"	NONE	NONE	—	8"	TWO	—	—	900,000	—	·	—	.320	LINER
EKLUTNA	EKLUTNA ALASKA	3621 FEB. 1952	EMBEDDED IN CONCRETE	90" 83" 75"	850	800	3	291'	716.67	24.00	2-25,000	660	1-1464'	⅜"	1⅜"	NONE	NONE	—	—	—	46,000T D	—	1,482,000	.192	—	.26	.452	DRESSER COUPLED PIPE
PALISADES	PALISADES IDAHO	— DEC. —	EMBEDDED IN CONCRETE	26'-0"	239	190	8	107'	5387.49	5382.00	4-39,300	1780	1-1643'	⅜"	1⅝"	NONE	NONE	—	—	FOUR	—	225%	1,290,000 2,220,000	—	.385	.15	.473	LINER MANIFOLD
CHANDLER	YAKIMA WASHINGTON	3950 JUNE 1953	EMBEDDED AND BURIED	120"	115'	116'	3	0'	543.62	507.00	2-6,500	735	9-486'	⅜"	⅞"	NONE	NONE	—	—	—	—	—	270,000	—	—	—	.365	
GLENDO	MO. RIVER BASIN WYOMING	— NOV. FEB. —	EMBEDDED IN CONCRETE	21'-0"	168	104	5	32'	4635.00	4497.75	2-16,750	1670	1-393'	⅝"	1⅜"	NONE	NONE	—	—	—	130%/a"	1664,000 470,000	—	.285	.149	0.176	.431 .325	PENSTOCK & OUTLET PIPE SURGE TANK
ROZA	YAKIMA WASHINGTON	4553 DEC. 1955	BURIED	10'-0"	154	156	3	30'	1174.18	1033.00	1-10,000	1,180	1-448'	⅜"	⅞"	NONE	NONE	—	—	—	—	—	400,000	—	—	—	.325	INCLUDES 477' OF 8' WATER LINE
WANSHIP GATEWAY	WEBER BASIN UTAH	4768 OCT. 1956	BURIED BURIED	54" 54"	118' 145	116' 140	3 3	67' 144	5800.00 4929.94	5590.00 4804.00	1-2,000 2-2,900	179 215	1-33' 1-388'	⅜" ⅜"	⅜" ⅜"	NONE NONE	NONE NONE	—	—	—	150%/b" 150%/b"	—	11,500 294,000	—	—	—	.475 .278	SCHEDULE NO. 1 SCHEDULE NO. 2
HELENA VALLEY	MO. RIVER BASIN MONTANA	— DEC. —	IN OPEN & EMBEDDED	10'-0"	150'	120'	5	115'	3699.97	3636.75	2-5,800	500	1-501'	⅜"	⅝"	2 RINGS 1"x8"	ROCKER	60'-0"	—	TWO	150%/b"	LEAKAGE TEST-125%	375,000	322	.341	—	—	
FREMONT CANYON	MO. RIVER BASIN WYOMING	4901 JAN. 1957	IN TUNNEL	10'-9"	353	300	5	80'	5672.00	5497.00	2-33,500	1,160	2-777'	⅝"	⅞"	NONE	NONE	—	—	—	40,000T D	—	1,250,000				.60	INCLUDES SURGE TANK COMPLETE WITH ALL APPURTENANCES
TRINITY	CENTRAL VALLEY	4824 FEB. 1957	IN TUNNEL	16'-0"	445	375	8	225'	1945.30	—	2-86,800	1,800	1-1726'	1½"	2"	2 RINGS 1½"x10"	SLIDING SUPPORT	40'-0"	12"	—	—	LEAKAGE TEST-250%	7,800,000	—	.33	.07	.40	DRESSER COUPLED PIPE
GLEN CANYON	COLO. RIVER STORAGE ARIZONA-UTAH	4825 MAR. 1957	EMBEDDED & BURIED	15'-0"	560	460	5	135'	3470.00	3140.00	8-108,500	3,600	8-548'	¾"	1⅜"	NONE	SLIDING	60'-0"	20"	EIGHT	43,000T D	—	16,700,000	—	.19	.14	.33	
YANKEE	YANKEE THAILAND	—	EMBEDDED, IN OPEN & BURIED	4.5M	428	328	5	67'	207.00M	13935M	2-97,000 6-FUTURE	3,066	8-533 M	⅜"	1⅝"	NONE	SLIDING & ROCKER	—	20"	FOUR	—	—	3,504,300					TAKEN OVER BY ENGINEERING CONSULTANTS INC.
BIG THOMPSON	COLO. BIG THOMPSON COLORADO	4990 MAY 1957	BURIED	74"	181	178'	5	65'	5445.12	5275.00	1-6500	380	1-33'	⅜"	⅜"	NONE	NONE	—	—	—	200%/b"	—	165,000	—	—	—	.33	
FLAMING GORGE	COLO. RIVER STORAGE UTAH-WYOMING	5045 JUNE 1958	EMBEDDED IN CONCRETE	10'-0" 9'-0"	444	365	5	83'	5850.00	5601.00	3-50,000	1,420	3-1003	1"	1⅜"	—	—	—	16"	—	46,000T D	—	1,160,000	—	—	—	.307	
GREEN SPRINGS	ROGUE RIVER BASIN OREGON	4908 JUNE 1957	BURIED	49" 34½	1987	1800	100	198'	4330.00	2420.00	1-23,500	128	1-8964	¼"	1⅝"	—	—	—	—	—	46,000T D	—	3,250,000	—	—	—	.33	DIA. VARIES FROM 49" TO 34½"
UPPER MOLINA	COLLBRAN COLORADO	5161 APR. 1959	BURIED	36" 30"	2686	2490	120	269'	9535.20	7111.00	1-12,000	50	1-12,885	⅜"	1⅛"	—	—	—	—	—	43,000T D	—	3,480,000	—	—	—	.234	DIA. VARIES FROM 36" TO 30"
LOWER MOLINA	COLLBRAN COLORADO	5161 APR. 1959	BURIED	36" 30"	1610	1400	120	161'	7076.20	5461.00	1-6800	50	1-24984	⅜"	¾"	—	—	—	—	—	43,000T D	—	3,341,000	—	—	—	.234	DIA. VARIES FROM 36" TO 30"
SPRING CREEK	CENTRAL VALLEY	5381 SEPT. 1960	IN OPEN AND BURIED	8'-10½"	639	582	10	280	928.97	511	2-108,000	2036	2-2730'	⅝"	1⅛"	2 RINGS 1"x7½"	SLIDING AND FIXED	60'-0"	—	—	46,000T D	—	6,370,000				.212	
CLEAR CREEK	CENTRAL VALLEY	5334 JUNE 1960	IN OPEN AND BURIED	8'-10½"	707	514	10	360	1700.96	1195	2-93,500	1940	2-4414'	1⅛"	1⅜"	2 RINGS 1"x9"	SLIDING AND FIXED	60'-0"	—	—	46,000T D	—	7,500,000				.203	
TRINITY	CENTRAL VALLEY	5334 JUNE 1960	BURIED	12'-0"	483	330	10	203	2100	1904	2-70,000	1940	2-365'	—	1½"	—	—	—	—	—	46,000T D	—	7,000,000				.203	

☆ Per Turbine Unit

WELDED STEEL PENSTOCKS

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Abstract

This monograph assists in the design and construction of welded steel penstocks which must be hydraulically efficient to conserve head and structurally safe to prevent failures. It gives up-to-date information on penstocks for hydroelectric plants based on 40 years of Bureau of Reclamation experience in powerplant design and construction. Until a special penstock code is adopted, steel penstock practice is governed by appropriate codes that apply to pressure vessels. Present Bureau design standards and construction practices were developed gradually, and result from improvements in welding quality steels, in welding processes, and in the inspection and testing of welds. This monograph is based on studies first published in 1946 and revised twice since then. It contains sections on location and arrangement; economic studies, head losses, and effect of water hammer; pipe stresses, supports, and expansion joints; bends, branch outlets, and wyes; penstock accessories, piers and anchors, and materials; fabrication, installation, specifications, and welding and corrosion control for penstocks. A new selected bibliography of 16 references has been added.

DESCRIPTORS—Arc welding/casings/codes/corrosion control/dam design/flanges/head losses/hydraulic turbines/hydroelectric powerplants/*penstocks/*pressure conduits/*pressure vessels/safety factors/stresses/ *welded joints/tunnel linings/field tests.

IDENTIFIERS—hydraulic design/American Water Works Association/American Society for Testing & Materials/Federal Specifications/*Welded Steel Penstocks.

Mission of the Bureau of Reclamation

The Bureau of Reclamation of the U.S. Department of the Interior is responsible for the development and conservation of the Nation's water resources in the Western United States.

The Bureau's original purpose "to provide for the reclamation of arid and semiarid lands in the West" today covers a wide range of interrelated functions. These include providing municipal and industrial water supplies; hydroelectric power generation; irrigation water for agriculture; water quality improvement; flood control; river navigation; river regulation and control; fish and wildlife enhancement; outdoor recreation; and research on water-related design, construction, materials, atmospheric management, and wind and solar power.

Bureau programs most frequently are the result of close cooperation with the U.S. Congress, other Federal agencies, States, local governments, academic institutions, water-user organizations, and other concerned groups.