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CONTENTS

Page

Page

LIST OF FIGURES

PREFACE

Much of the material contained in this monograph was prepared originally to aid in the training of new employees. Consequently, emphasis was placed on particular types of model studies, techniques, and equipment which have been useful in the Bureau of Reclamation Hydraulic Laboratory. The reader may find that some techniques of particular interest to him are mentioned briefly or not at all. Deficiencies may be offset by the emphasis on basic principles in the chapters on similitude, general features of model design, construction and operation, and field studies.

The presentation stresses the necessity for understanding the viewpoint of the designer of hydraulic structures and equipment, and thus gives prominence to the conviction that the laboratory worker must appreciate the criteria and limitations which affect the designer's work. Familiarity with design considerations is essential also to the proper conduct of field studies.

INTRODUCTION

The Hydraulic Laboratory of the Bureau of Reclamation is equipped, staffed, and operated to study, by analytical methods, scale models, and field investigations, problems which arise in connection with Bureau of Reclamation projects. In the civil engineering field the studies may include dam spillways, outlet works for dams, canal chutes, drops, irrigation distribution systems, diversion works, sediment control works, and open channel features. In the field of mechanical engineering are studies of gates, valves, piping systems, penstocks, siphons, turbines, pumps, and closed conduit features.

The laboratory is primarily a service organization and most of the work is done at the request of the design department. Preliminary designs are usually submitted to the laboratory with the request that specified features be studied from the standpoint of hydraulic performance. In some instances the request is in general rather than specific terms. In either case, the studies, especially when a model is involved, are conducted in such a way that the maximum amount of pertinent information is obtained.

Model studies are conducted according to established rules of hydraulic similitude whereby there is a definite known relationship between performance of the model and that of the corresponding field structure. Comparison of model and prototype has clearly demonstrated that, with few exceptions, there is a correspondence of behavior within, and usually well beyond, expected limitations. In many cases agreement between model and prototype performance has exceeded expectations.[1] In some cases where agreement at first appeared to be lacking, it was found that failure to recognize or interpret model results correctly was cause for the disagreement. Correspondence between the model and prototype has been especially complete for overfall spillway crests, valves, gates, outlet features, and energy dissipators. It is customary to provide calibration curves based on model results in lieu of field calibration. Energy dissipators, including stilling basins and buckets of various types, designed on the basis of model findings, have been successfully operated in substantial agreement with model indications. River improvement plans of tremendous magnitude have worked out successfully according to predictions based on model tests. The high efficiencies and smooth operating characteristics of the large modern turbines and pumps can also be attributed to model experiments. In practically all cases it will be found that improvements indicated by models have been substantiated when the prototype structures have been constructed.

Origin and Development of the Laboratory

Hydraulic model testing had its inception in the Bureau of Reclamation in August 1930 when a small staff of engineers, carpenters, and laborers began work in the Hydraulic Laboratory of the Colorado Agricultural Experiment Station, Fort Collins, Colorado. Other Bureau hydraulic laboratories have since been established and operated in various places as a particular need arose. During the clement seasons of 1931 to 1936, a laboratory was operated on the South Canal of the Uncompahgre Project near Montrose, Colorado, where a head of 50 feet and a discharge of 200 second feet of water were available. This laboratory was used to test large models of the side-channel spillways for Hoover Dam, a complete model of the Imperial Dam and its appurtenant works, and a model of the Grand Coulee Dam spillway bucket. During the period 1934 to 1937, the Hydraulic Laboratory was maintained in the basement of the Old Customhouse in Denver, Colorado, where many of the smaller structures designed by the Bureau

[1] "Conformity Between Model and Prototype in Hydraulic Structures," A Symposium, Transactions, ASCE, 1944, Vol. 109, pp. 3-193.

during that period were studied. In 1937, when the addition to the New Customhouse in Denver was completed, the equipment in the Old Customhouse was moved to the New Customhouse where a small but convenient laboratory was constructed. The Fort Collins laboratory building was expanded to about four times its original size in 1935 to meet the ever-increasing load of assignments. But, with the establishment of the laboratory in the New Customhouse, personnel from the Fort Collins laboratory were gradually absorbed. By the Fall of 1938, the Bureau had completely discontinued its operation at the Fort Collins laboratory. In 1939, laboratory facilities were installed in the Arizona Canyon wall outlet house at Hoover Dam to utilize the head of 350 feet and discharge of 200 second feet. This laboratory was operated for several months in 1940 and 1941, and again in 1945, the program being geared to the studies made in the Denver laboratory on smaller models. In the latter part of 1946, the Hydraulic Laboratory was moved from the New Customhouse to its present home in Building 56 of the Denver Federal Center. This is by far the most desirable location and the best-equipped laboratory to date. In addition to the excellent laboratory building, there is outdoor space available for large river models.

LABORATORY EQUIPMENT

General Arrangement of the Laboratory

The Bureau's Hydraulic Laboratory at the Denver Federal Center is housed in a factory-type building which was constructed and used during World War II for the manufacture of munitions. The laboratory floor space, exclusive of offices, shops, and storage, embraces an area of 53,000 square feet, obstructed only by 10-inch steel columns on 30-foot centers. Seventy percent of this area has 25 feet of head room; the remainder, 13 feet. Figure 1 shows the arrangement of the laboratory.

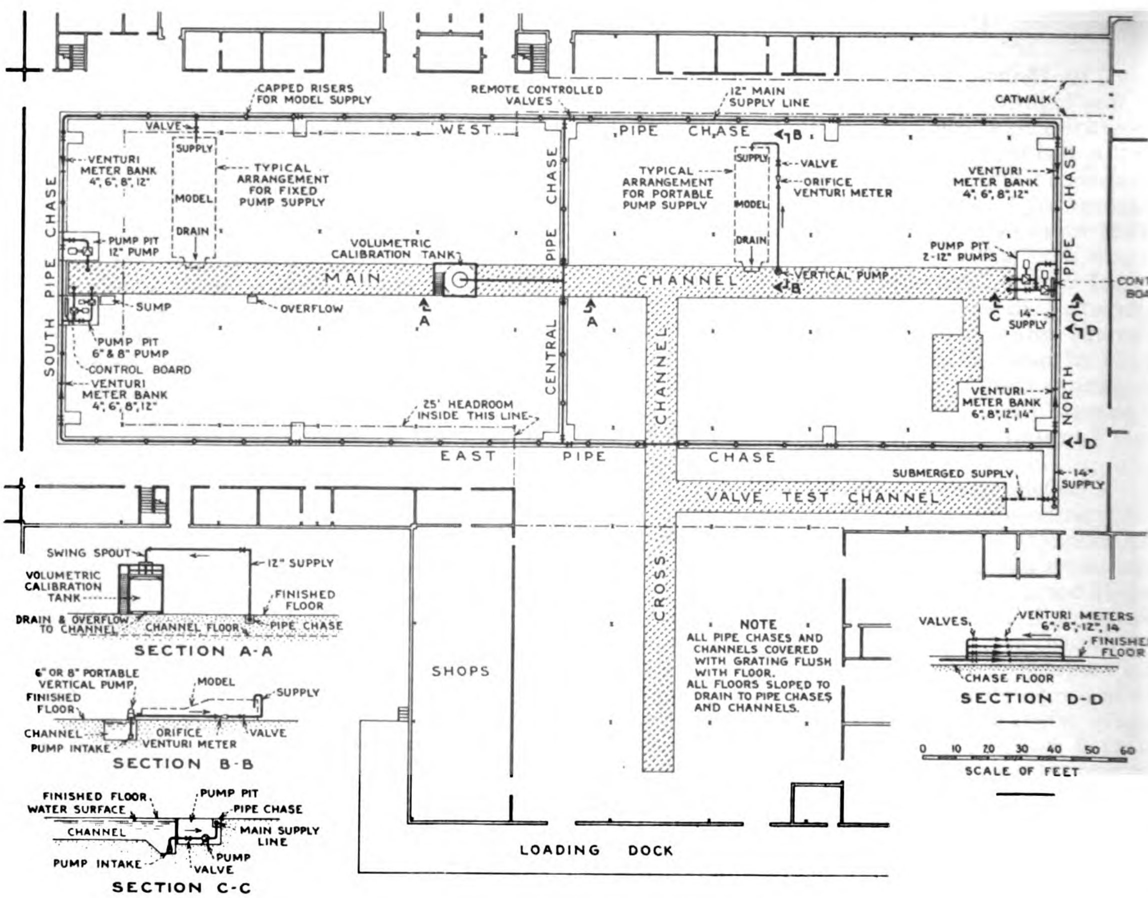


Figure 1. Floor plan of the Hydraulic Laboratory

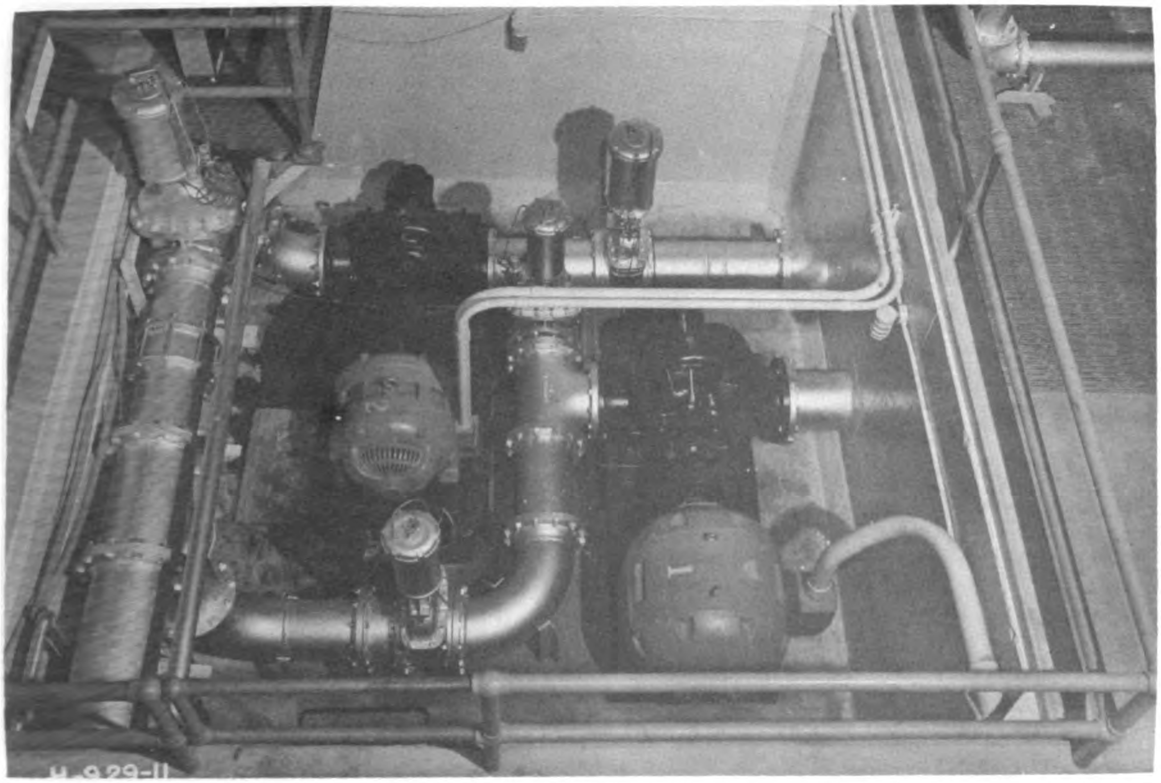


Figure 2. North pump pit. These two pumps have a capacity of 12 cfs each and may be connected in series or parallel, or operated separately

Facilities for Circulating Water

1. Fixed equipment.--The large water channels which traverse the laboratory are 9-1/2 feet wide and vary from 3 to 8 feet deep. These constitute the sump or reservoir for the entire laboratory. Pump pits located at the ends of the main sump channel contain three 12-inch horizontal pumps, each driven by a 100-horsepower electric motor. Hydraulically operated gate valves and the piping in the north pump pit (Figure 2) enable the two pumps to be operated in series or in parallel. The smaller channel around the perimeter of the main portion of the laboratory (Figure 1) is a pipe chase which contains the permanent measuring equipment, supply piping, and valves. The laboratory floors slope gently toward the chases and channels for drainage. The main channels and pipe chases are covered with steel grating set flush with the floor. Permanent measuring equipment consists of four banks of Venturi meters ranging from 4 to 14 inches in diameter, arranged so that each bank serves a quarter of the laboratory. (See Figures 1 and 3.) Piping throughout the chases is principally 12-inch standard pipe with tee connections and vertical risers at 15-foot intervals. To connect a model into the system, light temporary pipe is employed between the tee and the model.

Flow in the permanent line is regulated with hydraulically controlled gate valves. These are located below the floor line, except for some of the valves on the Venturi meter lines. To reduce excess bulk and weight, the operating cylinders for these valves were specially designed and were manufactured in the laboratory shops. Figure 4 shows a cross section of one of the cylinders. The cylinders are operated by Denver city water pressure (approximately 70 psi) and controlled by four-way pilot valves. The pilot valves for the Venturi meter lines are hand operated. These valves, which are located on the control boards (Figure 6), can be moved to the right, to the left, or to a neutral position, thus opening or closing the main gate valves on the Venturi lines, or holding them in any intermediate position. It is possible to observe the positions of the valves on the Venturi meter lines from an indicator on the gage board, Figure 6. As a gate valve moves from open to closed position, or

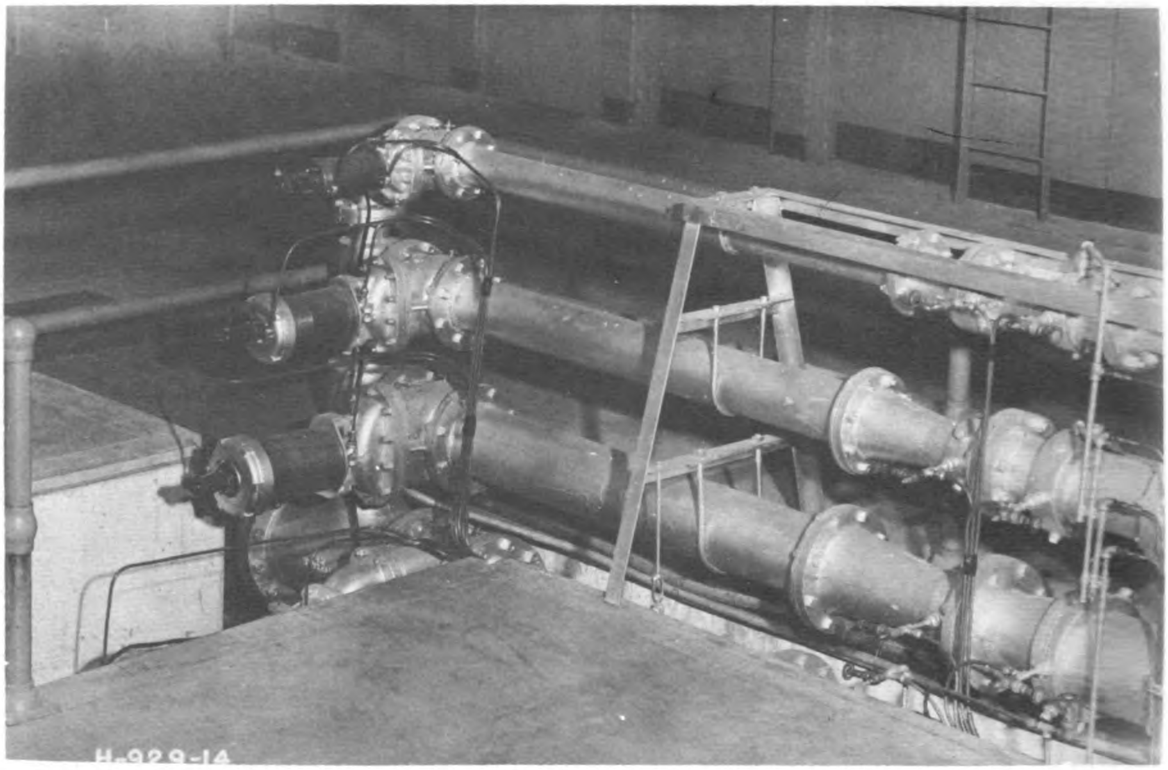


Figure 3. Typical bank of Venturi meters with hydraulically controlled throttling valves downstream

vice versa, a twisted square rod, actuated through the piston in each hydraulic valve cylinder, causes a Selsyn generator mounted on the cylinder (Figure 3) to turn through approximately one revolution. A corresponding Selsyn motor on the gage board is thereby energized to revolve the same amount. A positioning dial on the control board indicates the percentage of opening of each Venturi meter valve.

The remaining gate valves in the main circulation loop have solenoid-operated four-way valves attached to the hydraulic cylinders (Figure 5). The solenoid valves are electrically connected to the boards and operated from there by microswitches. These gate valves are operated either open or closed, there being no control for intermediate positions. Individual signal lights on the boards indicate whether the valves on the main line are open or closed.

The two main control boards also contain specially designed pot-type mercury manometers for indicating differential head across the Venturi meters, Figure 6. These gages were specially designed in the laboratory, because no commercial manometer containing all of the desired features could be purchased. Figure 7 shows front and side views of one of the gages. All parts which may come in contact with the mercury are of stainless steel or glass to prevent amalgamation with the mercury. The mercury is contained in the stainless steel pot at the bottom and rear of the gage. The reading glass consists of 1/2-inch inside diameter heavy-wall tubing sufficiently large to minimize capillarity and to increase the damping effect of the connecting tubing. The height of mercury in the reading glass is obtained by reading the rack and pinion vernier gage to the right of the glass, which is accurate to 0.001 of a foot. Parallax in reading is eliminated by leveling the eye with two lines on a finder equipped with a small light that illuminates the manometer glass and reading scale in the vicinity of the finder. The remainder of the gage consists of many small parts. One of these is a needle valve at the top of the glass tube which closes when the mercury rises sufficiently to float the needle. The purpose of the needle valve is to prevent loss of mercury should the gage be operated improperly.

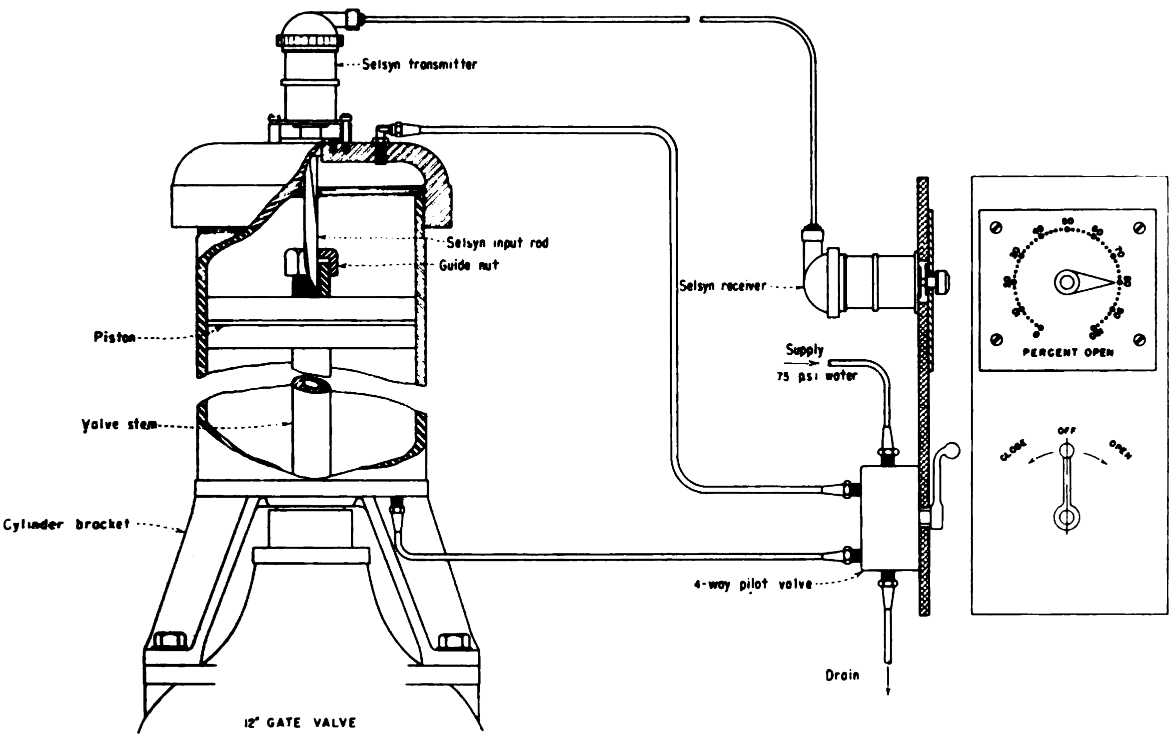


Figure 4. Typical hydraulic regulating valve assembly

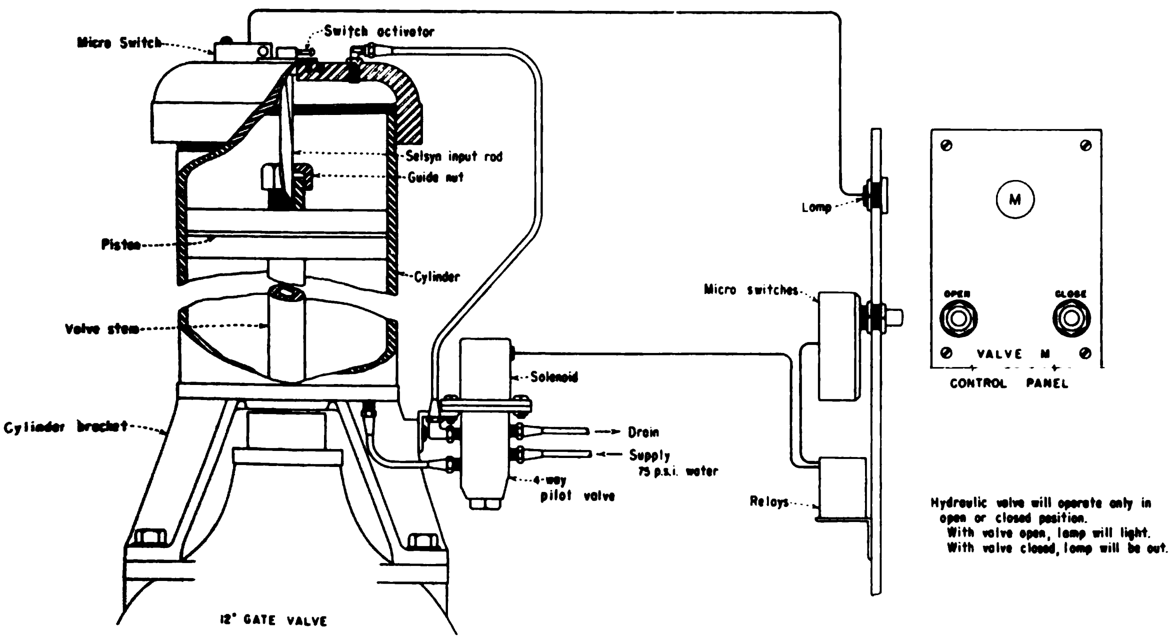


Figure 5. Typical solenoid-operated hydraulic valve assembly

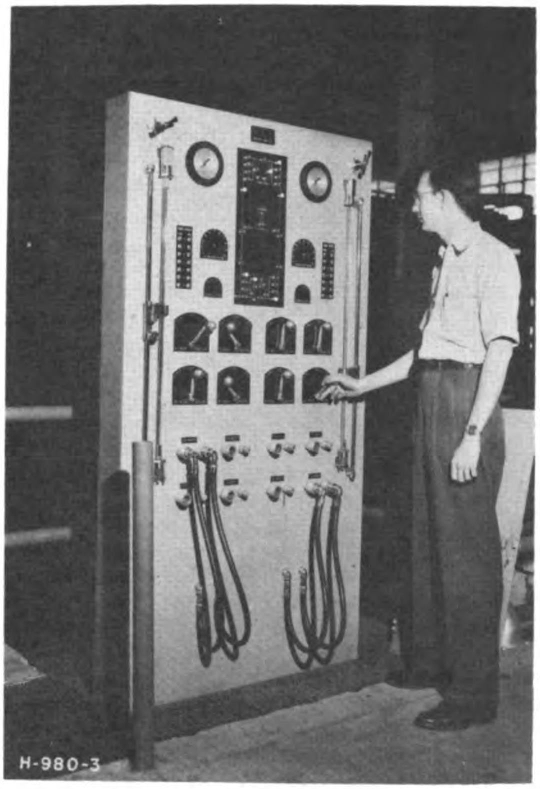


Figure 6. Laboratory control board

The unusual feature of the gage is a specially designed valve mounted near the top of the unit which makes it possible to bleed the air from the gage toward the Venturi meters at any time. On most gages of this type the bleeding is done in the reverse direction and the existing pressure in the Venturi line is relied on to establish flow. The special valve eliminates the need for three valves: when open, it balances the pressures on both legs of the manometer so that mercury will not be lost; at the same time it allows water from the city mains to flow under steady pressure from the gage to the Venturi meters through the two connecting lines. The advantage of this feature is that the Venturi meters can be bled at any time during a run without having to close the valve to develop sufficient pressure. The gage cannot be bled improperly, and it is impossible to lose mercury.

One gage is provided for each Venturi meter bank, because only one meter is used at any time. The pressure taps for each meter are connected through a pair of 3/8-inch-diameter copper tubes to the gage board. The end of each of these lines is fitted with one-half of a slip-connector (Figure 6) similar to those used for temporary connections on small air lines. Two flexible lines originating at the manometer gage are each fitted with one-half of a slip connection. Thus, it is a simple matter to connect the manometer gage to any one Venturi meter by plugging the two slip-fittings from the manometer gage into the proper pair of connectors on the panel. These slip-connectors are built with check valves in both directions so that on disconnection the gage lines remain full of water. On completion of a connection at the panel, both check valves in the connectors are automatically forced open so that flow in either direction is then possible. This arrangement has an advantage over the usual one in that it eliminates hand valves which may leak and cause erroneous manometer readings.

2. Portable equipment.--One of the most versatile pieces of equipment in the laboratory is a portable pump unit (Figure 8), equipped with an individual flow measuring device, which can be easily transported and assembled over any sump channel. It consists of a vertical turbine-type pump, a standard section of lightweight pipe containing a flow straightener, and an 8-inch combination orifice Venturi meter developed specially for this application. The meter (Figure 9) has a ring seal which automatically seals the orifice plate in place when the pump is set in motion. The seal is originally spring loaded, but pressure from the pump completes the seal. To change orifices or use the device as a Venturi meter it is merely necessary to shut down the pump, lift the orifice plate from the slot, replace it with the one desired, and restart the pump. There are no bolts or clamps to loosen, and it is not necessary to drain the water from the line when changing orifice plates. The unit has the advantage of being usable as a Venturi meter for large discharges, where losses are important, or as an orifice meter for intermediate and smaller flows where losses are of little concern. The outstanding advantage of this meter is its portability. The laboratory has seven 8-inch and six 6-inch pump units of this type. The 8-inch units have a capacity of 5 second feet each; the 6-inch units handle 2 second feet each. Each unit is equipped with a portable mercury manometer gage.

3. Calibration apparatus.--All meters in the laboratory are calibrated and checked in place at regular intervals by using the volumetric calibration tank (Figures 1 and

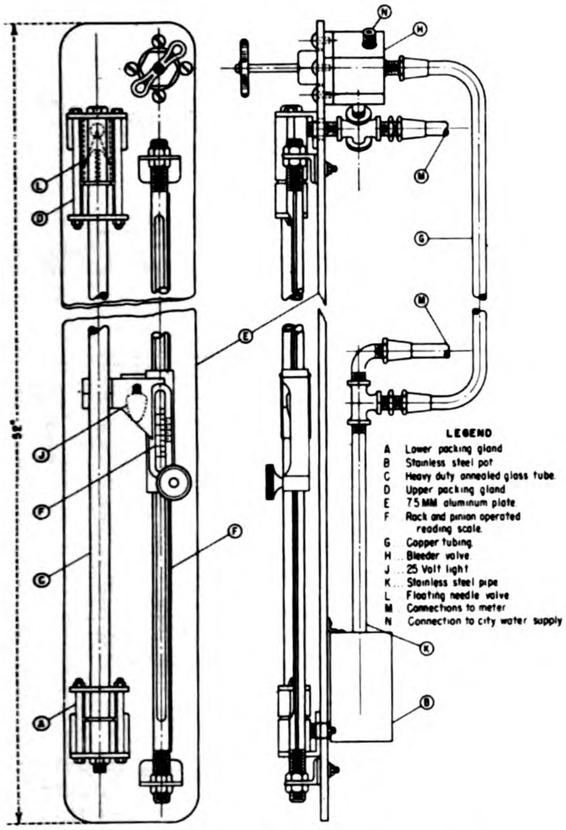


Figure 7. Mercury manometer

10) in the center of the laboratory. The main tank, which has a volume of approximately 700 cubic feet, contains a similar but smaller tank of about 60 cubic feet in capacity. For discharges of 2 second feet and less the small tank suffices; for larger discharges both tanks are used. These tanks resemble chemist's pipettes with large bodies and small necks. Details are shown in Figure 11. By means of a swing spout the inflowing water may be diverted into the small tank or the large tank, or it may be bypassed back to the reservoir. To make a calibration, a steady flow is established through a Venturi meter with the swing spout in the waste position. The swing spout is then shifted by a pneumatic jack to one of the tanks until the tank is filled to some point in the neck, after which the spout is returned to its original position. During the same period a record is made of the time required to fill the tank and a set of readings is taken from the mercury manometer gage accompanying the Venturi meter being rated. The volume of water in the calibration tank is determined from a graph which shows the volume of the tank for any hook gage reading and temperature. The pilot valves on the tanks are interlocked so that it is impossible to fill the large tank with the valve in the small tank closed. If this were possible, there would be a tendency to float the small tank which would probably result in distortion of both tanks.

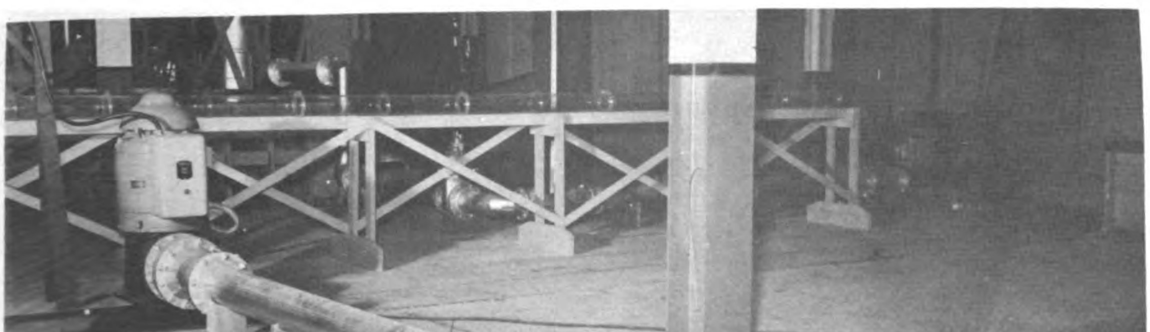


Figure 8. Vertical portable pump unit and meter

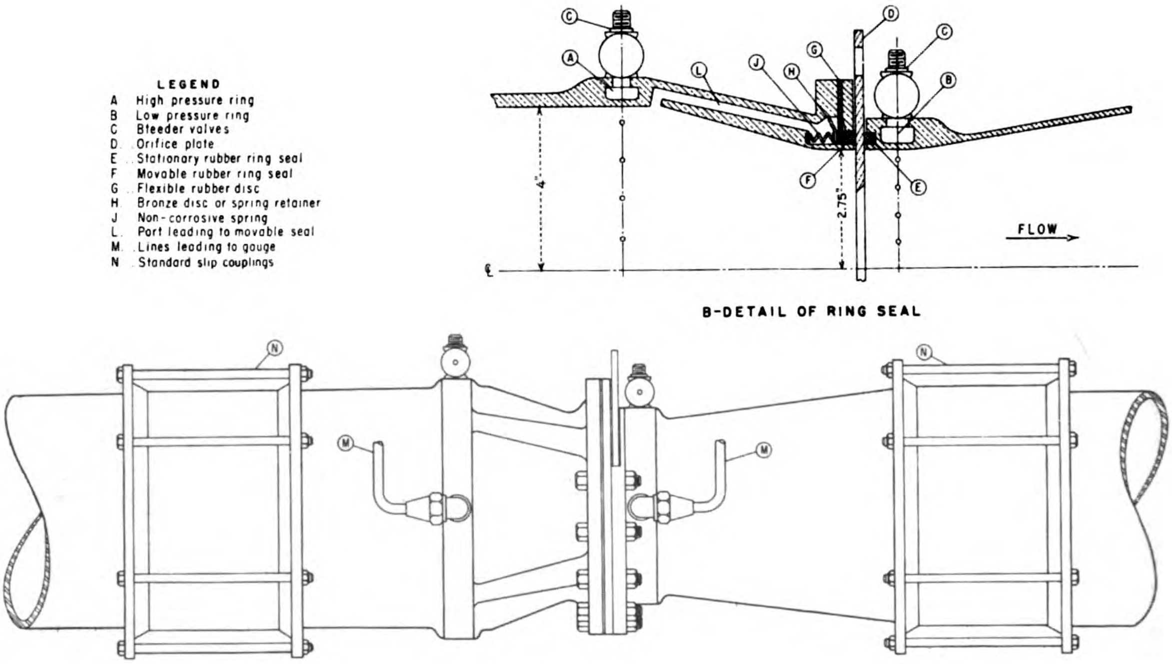


Figure 9. Venturi orifice meter

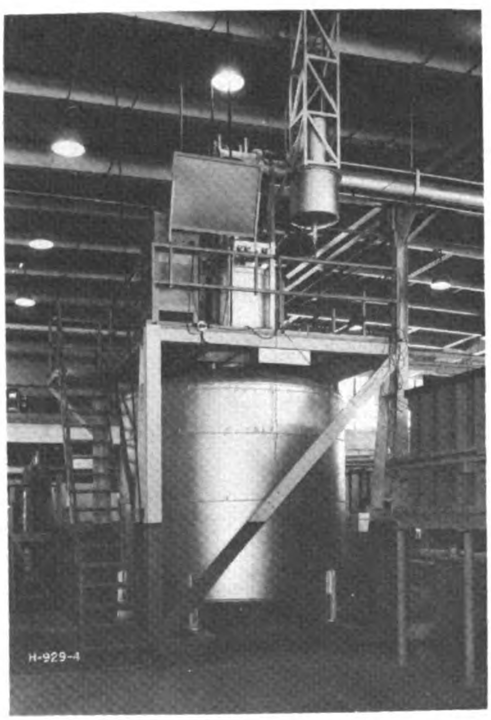


Figure 10. Laboratory calibration equipment

The time interval of filling is determined by means of an electronic counter (Figure 12) controlled by a 100 cps tuning fork. The figure shows the external features consisting of 20 neon lamps, a magnetic counter, tank selector, and a reset switch. Prior to each calibration run the counter is set at zero position by actuating the reset switch. The timer is started and stopped automatically by motion of the swing spout. The difference between the final and initial readings on the magnetic counter gives the number of seconds elapsed. The bank of 10 lights on the left shows the additional fraction of a second to tenths, and the bank of 10 lights on the right gives the fraction of a second to hundredths. Thus, the timer can be read to 1/100 of a second.

An independent pipette tank, shown to the extreme right in Figure 10 and in detail in Figure 11, was used for the initial calibration of the large volumetric tanks. The independent pipette consists of two compartments, one having a volume of 1.18 cubic feet and the other a volume of approximately 7.36 cubic feet. Skimming weirs in this tank assure accurate volumetric measurement. The main tank was calibrated by repeatedly filling the independent tank with water and emptying it into the larger one, at the same time keeping an accurate record of water temperatures. Prior to calibration of the large tank, the independent pipette tank was removed from

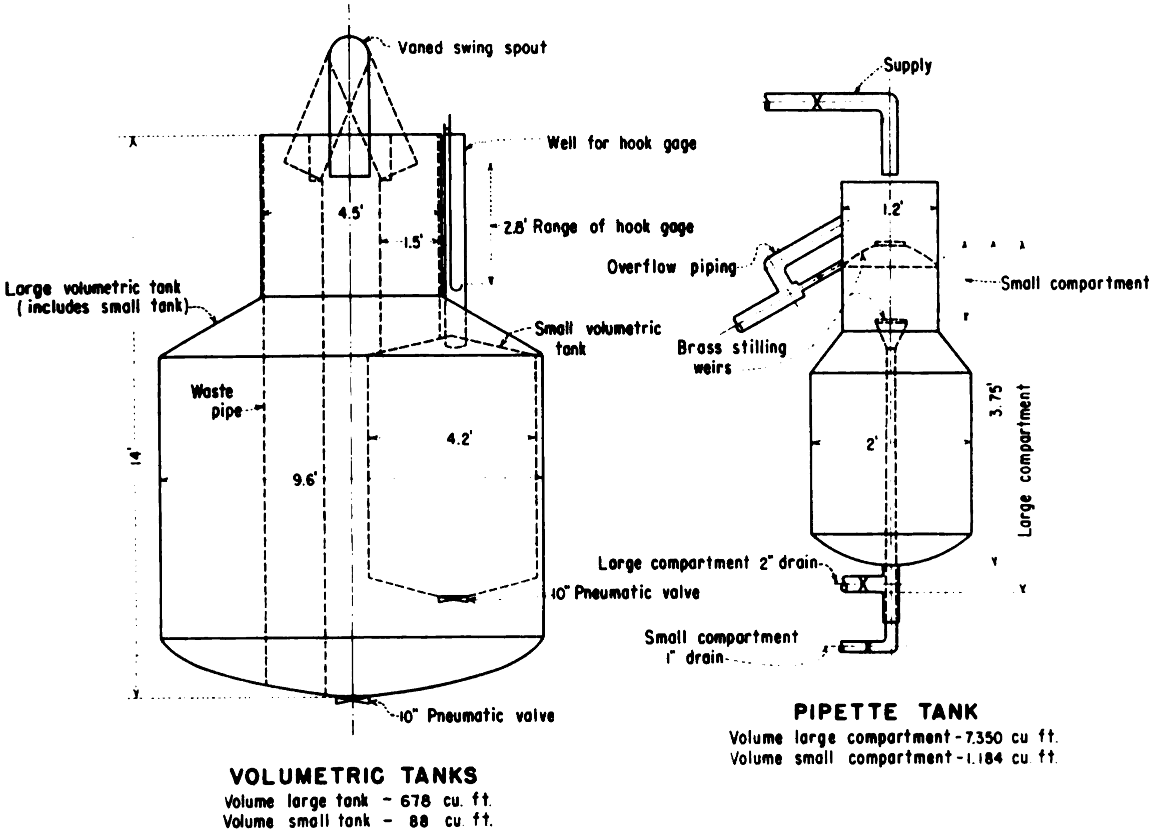


Figure 11. Details of calibration tanks

the position shown and calibrated by weight using a sensitive 500-pound scale.

Accuracy is maintained with these relatively small-capacity tanks by the following measures: First, the small tank, or both tanks, is always filled to the neck, thus improving the accuracy of water surface measurement from which the volume is obtained directly; second, special care is exercised in determining volumes of tanks at various water temperatures as the volumes change slightly with the temperature; third, pneumatically operated diverting equipment insures uniform speed of movement of the swing spout; fourth, the position of spout can have no influence on the flow through it; and fifth, the timing device is extremely accurate and dependable. To obtain the desired accuracy the discharge is limited to about 12 cfs.

Special Testing Facilities

1. Glass-walled flume.--A glass-paneled steel flume forms a permanent part of the laboratory test equipment (Figure 13). This flume is particularly useful for studying sectional models of spillways and aprons or buckets. Also, studies of wave forces on embankments, dikes, or riprap slopes may be performed using the flume as a tank. Other problems in which the flume is useful are: flow over or under gates of all kinds; studies of spillway crest shapes; and investigations of erosion tendencies on hydraulic structures of all kinds. The glass panels allow visual inspection of the action of suspended or bed load in sedimentation studies. With minor modifications and additions the flume may be used for almost any problem involving flowing or still water.

The flume is 80 feet long and consists of panels 10 feet long and 8 feet high anchored to a concrete floor. Panels are interchangeable and either nine tempered glass windows 1/2 inch thick or a single steel plate may be installed in any panel. The floor is a reinforced concrete slab containing drains and holes for anchor bolts. The floor slab rests on the laboratory floor and is sufficiently wide to provide support for steel buttresses located at the ends of each panel.

The headbox, used to quiet the inflowing water before it passes into the flume proper, is 10 feet high (2 feet higher than

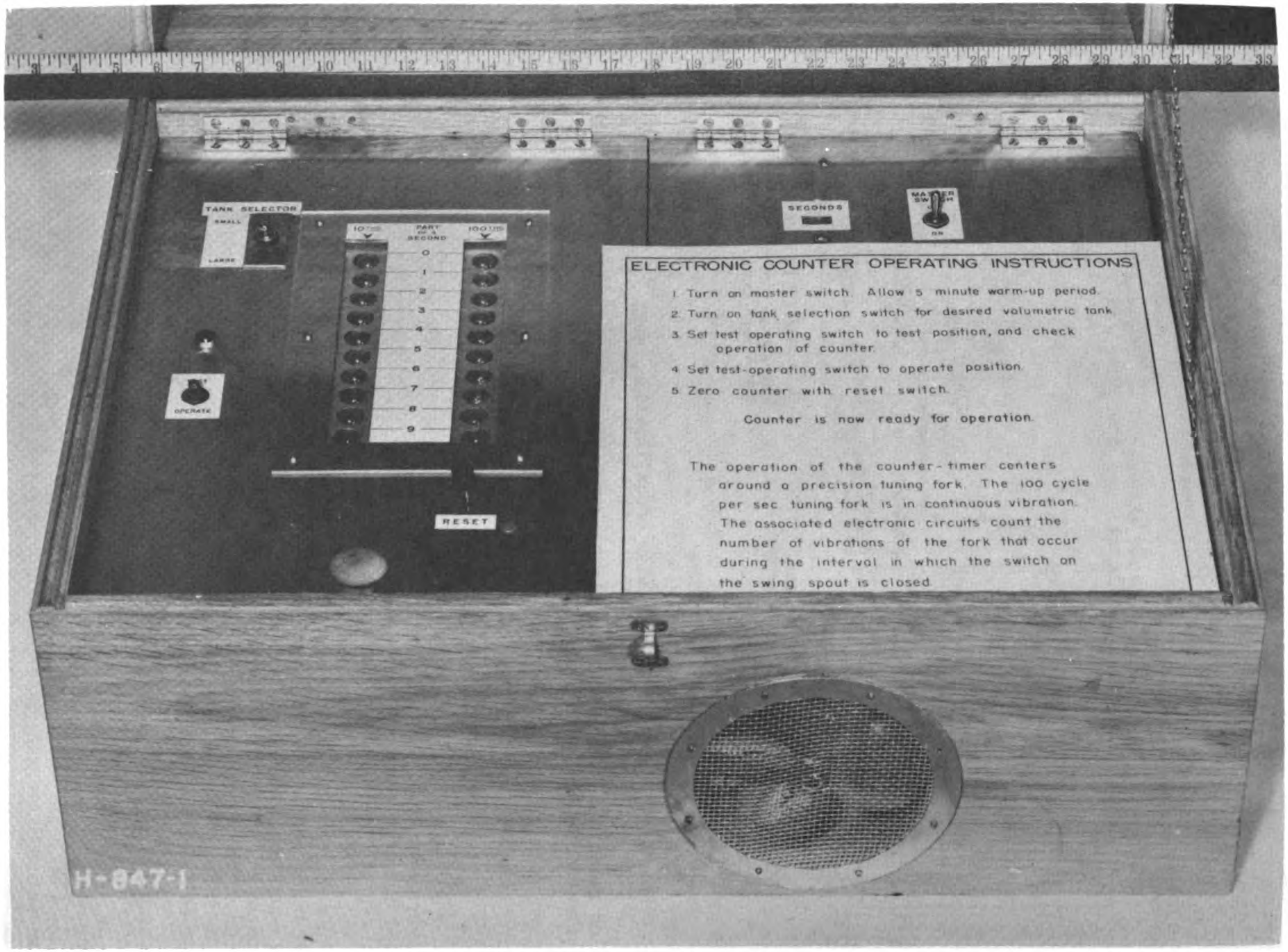


Figure 12. Electric counter for timing the filling of calibration tanks

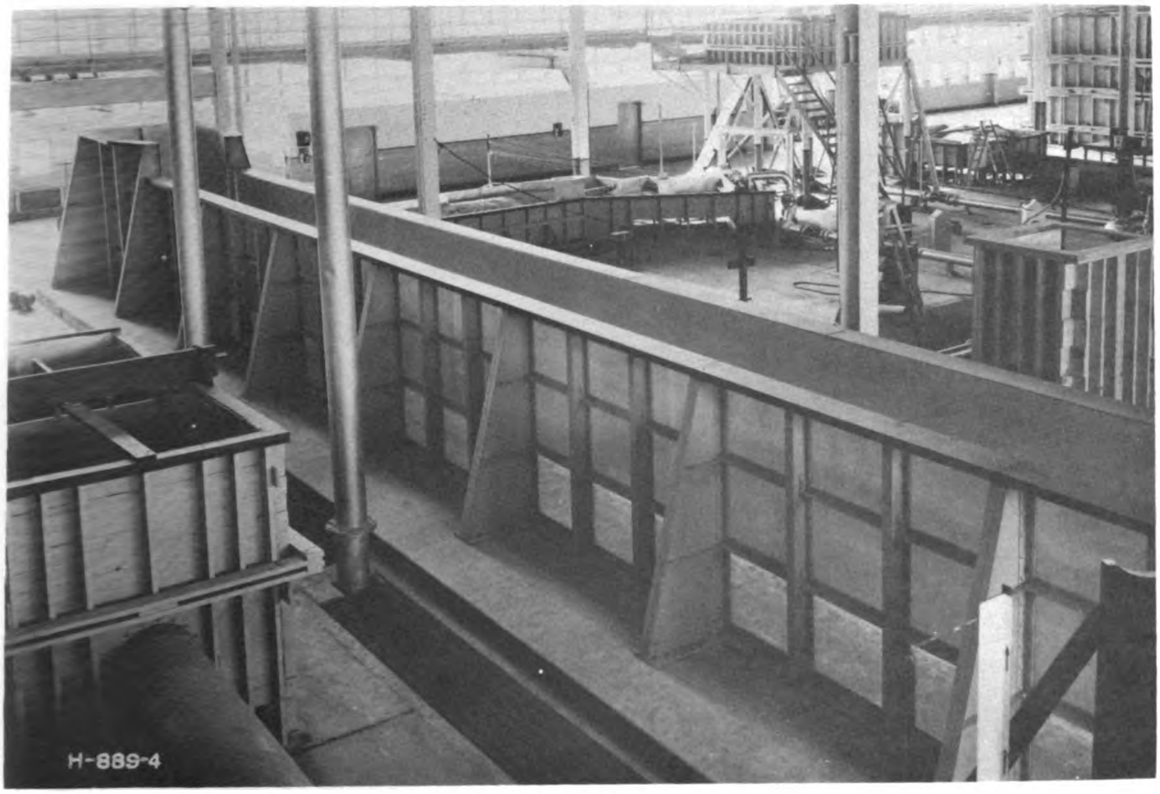


Figure 13. Glass-sided test flume

the flume) to allow for surges and head loss through the flow-straightening baffle. The baffle consists of sheets of corrugated steel 15 inches wide placed horizontally and 1 inch apart. Water passing through the baffle undergoes a series of expansions and contractions between the corrugations and enters the test flume smoothly and uniformly with very little head loss.

The water surface in the flume can be regulated by various types of tailgates at the downstream end of the flume. These gates are interchangeable and the tailgate used depends on the type of model test. The flume is so located in the laboratory that discharges of over 30 second feet may be introduced into the headbox.

2. Air-testing equipment.--In addition to the hydraulic equipment previously described, the laboratory has air-testing equipment consisting of two centrifugal blowers, each having a capacity of approximately 3,000 cfm at 2 psi; also related apparatus by which various closed conduit problems, such as those involving valves, gates, and pipe lines, can be solved quickly by using air as the fluid instead of water. In air testing, it is possible to construct models of plaster of paris or wood and to complete the testing before similar hydraulic models, which must be constructed of more durable material, are out of the shop. The air model is limited to closed conduit work, but where applicable, the results have closely checked those obtained with corresponding hydraulic models. The blowers are driven at a constant speed, and the total air flow is measured by sharp-edged intake or discharge orifices. Figure 14 shows a blower set up to test an intake structure. In this case the measuring orifice is located on the discharge side of the blower. A discussion on model testing with air will be found under the subject "Closed Conduit Flow."

3. Fluid polariscope.--Another interesting piece of laboratory equipment, which can be used for demonstration purposes or for testing of tentative designs of hydraulic structures, is the fluid polariscope. The apparatus shown in Figure 15 consists of two 12-inch polaroid lenses and two quarter-wave plates of the same dimension. The plates are set in a supporting frame as shown, in the following order from front to back: polaroid lens, quarter-wave plate, model to be tested, quarter-wave plate,

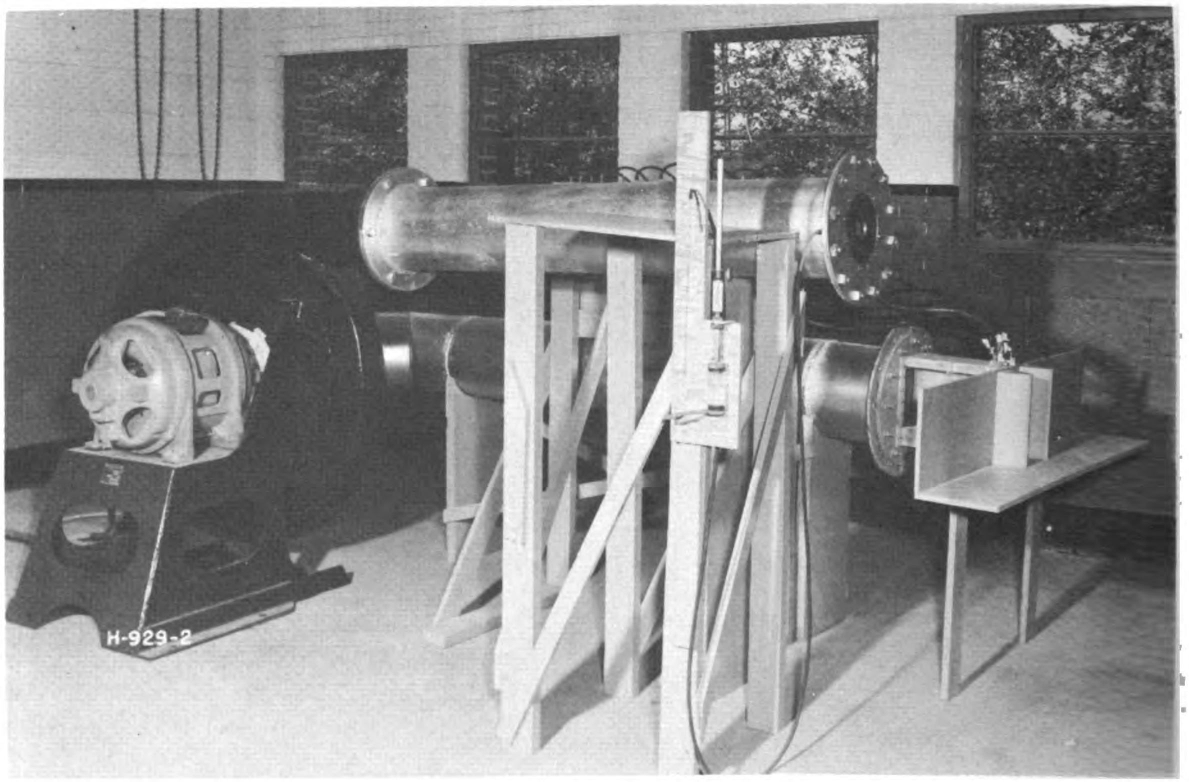


Figure 14. Laboratory blower for testing with air as the fluid

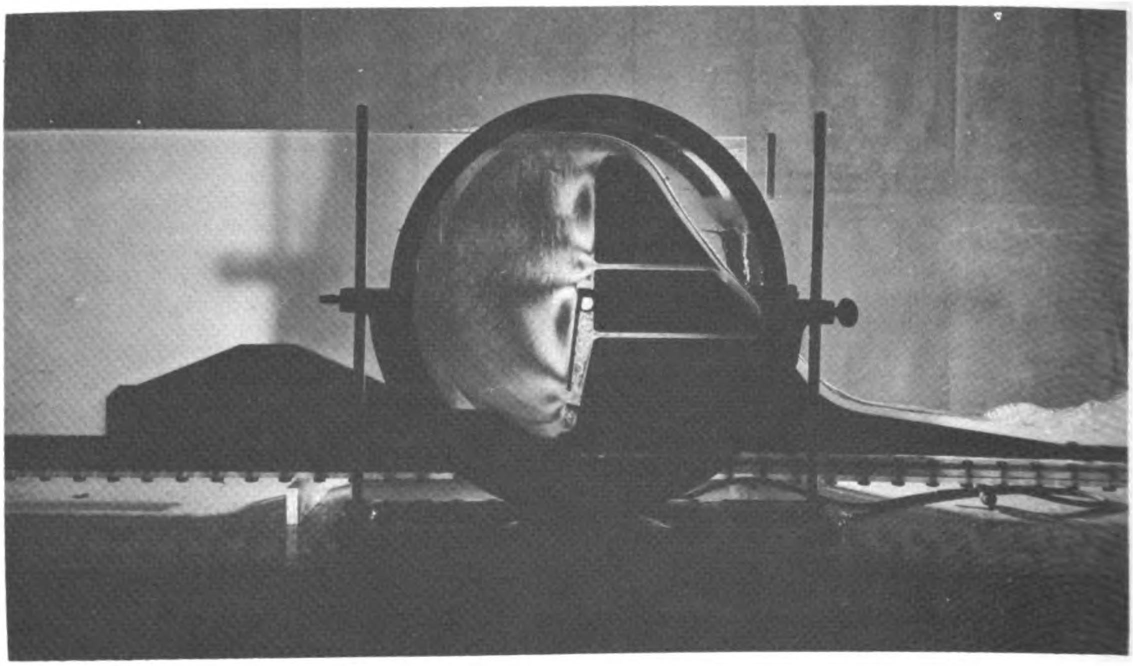


Figure 15. Fluid polariscope

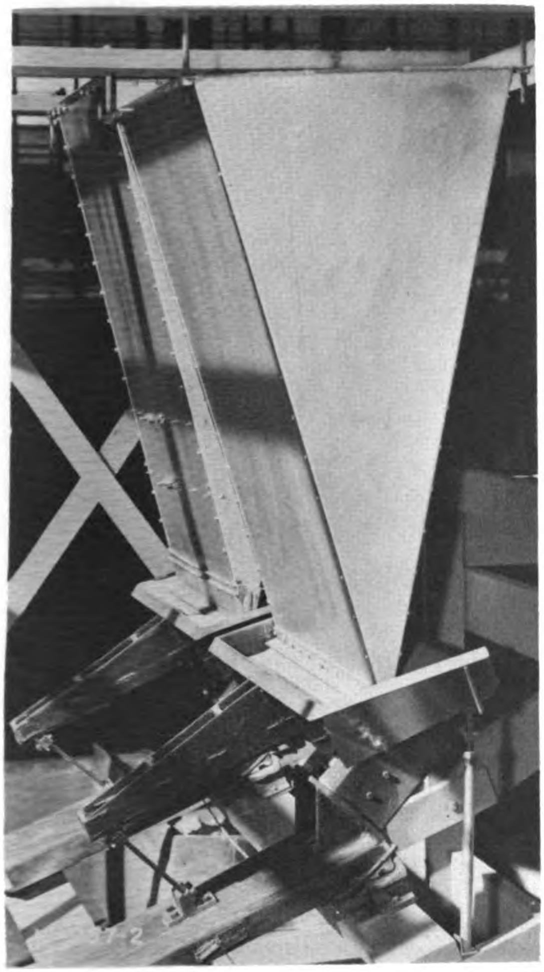


Figure 16. Apparatus for uniform feeding of sediment to movable-bed models

polaroid lens, opal-glass diffuser plate, and source of light. The models to be tested are necessarily two-dimensional, and are encased in a transparent closed vessel usually about one-half inch in width. The vessel, which contains an inlet and outlet, is connected in series with a miniature variable-speed centrifugal pump and a reservoir. The reservoir is filled with a mixture of distilled water, 1.5 percent of a special pure white magnesium bentonite, and 0.01 percent of sodium pyrophosphate. The bentonite particles consist of platelets so small that there is no tendency to settle out of the suspension or rise toward the surface. They move exactly as the fluid does, apparently having no appreciable inertia in themselves. The bentonite solution becomes bifringent through the action of fluid shear and so modifies the light passing through the stream that colored bands or "fringes" are visible. These connect points where the same velocity gradient prevails.

This type of hydraulic model testing lends itself best to preliminary designs in which flow conditions are questionable. A model can be constructed and tested in a few hours and much time and money can be saved by eliminating questionable designs. Should the design show merit but require further investigation, an air or hydraulic model can be constructed and tested.

4. Sediment feeding apparatus.--A special device for providing a uniform supply of sediment to a movable bed model is shown in Figure 16. A sheet metal hopper is mounted so the opening in the bottom is just above a pan on a vibratory-type feeder. The feeder is a trough or pan mounted on flexible leaf springs supported in a frame and vibrated at 60 cycles per second by an electromagnet. The magnet, which is energized by pulsating current, pulls the trough sharply down and back, then when the current is broken, the leaf springs return the pan up and forward to its original position. The return is not as sharp as the downpull and the resulting motion causes material on the pan to move forward even when the pan is sloped upward.

The feeder is equipped with a separate control box which contains the operating switch, a rheostat for controlling speed of flow of material, and an electronic valve that converts alternating current into sharp pulsating current. The pulsating current activates the magnet intermittently and thus causes the vibratory motion of the feeder. Sediment that has a mean size smaller than 0.50 mm must be dried before it is fed. Larger sediment will feed satisfactorily when damp.

INSTRUMENTATION

General Considerations

Since the object of every model investigation is a carefully planned series of measurements, the required instruments comprise an essential feature of the laboratory equipment. The function of each conventional instrument is briefly discussed without attempting to recommend particular instruments. Recommendations will be made in the discussions of different types of studies in subsequent sections. Many studies involve measurement of an unsteady quantity, such as a varying water surface or fluctuating pressure, which is beyond the scope of ordinary instruments. In such cases special instruments must be devised. The considerations involved in developing these instruments will be described in some detail.

Discharge Measurements

Laboratory instruments for measuring discharge consist principally of standard weirs, Venturi meters, orifices, and flow nozzles. Volumetric and weighing tanks may be included, although these usually serve as calibration equipment by which the accuracy of the standard meters can be checked. Weirs can be constructed by following the recommendations in any good textbook on hydraulics. Venturi meters, being more difficult to build, usually are purchased from one of the nationally known meter companies. Orifice meters and flow nozzles can be purchased commercially from the same sources; however, they can be constructed in the laboratory by adhering to the specifications.[2] Intake orifices[3] and discharge orifices[4] are also useful measuring devices in a laboratory. Other discharge measuring devices which can be used successfully in a laboratory are the elbow meter,[5] the Parshall flume,[6] the adjustable orifice gate,[2] the propeller-type flowmeter,[2] and the current meter.[6] Any of the above flowmetering devices is sufficiently accurate for laboratory use if it can be calibrated in place by suitable weighing or volumetric tanks. If this is not possible, it is advisable to rely on commercial meters or laboratory-constructed meters of standardized designs.

Measurement of Water Surface

The elevation of a water surface is usually measured with a hook or point gage operated directly over the surface or in a stilling-well arranged to dampen surface fluctuations. These gages may be used to indicate differences in elevation or absolute elevation. By use of special mountings they are adaptable to use under a variety of conditions. A point gage may be mounted on rails and thus cover a large area. Float gages connected to an indicating mechanism will reveal minor changes in water surface elevation. Hook and point gages shown in Figure 17A are operated through rack-and-pinion arrangements and are available commercially in various lengths and weights.

Velocity Measurements

Instruments for measuring velocities are as numerous as the conditions under which measurements are required. Instruments appropriate for moderate or high velocities are not accurate at low velocities, and those designed for low velocities are too fragile to withstand high velocities. Some instruments are more suitable for open conduits than for closed conduits.

The instrument shown as A on Figure 17B is known as a Bentzel tube and is designed for measuring velocities ranging from 0.4 to 3.5 feet per second. With the nose of the tube submerged in the flow, a vacuum pump is used to evacuate all air from the two legs. This establishes flow through the gage, up the leading leg and down the trailing leg. Incorporated in the leading leg is a conical glass tube containing a rubber float having a specific weight slightly greater than that of the liquid. The float rises or falls with a change of velocity and the glass tube is graduated to read directly in feet per second.

The most common velocity measuring devices are the Pitot tubes, some of which are pictured in Figure 17B. Tube B consists of a single kinetic leg and is used principally for measuring air velocities. Pitot tubes C, D, and E, of the modified Prandtl type, have both kinetic and static legs, and are designed for a coefficient of unity. The tubes, which are geometrically similar, vary in size according to the requirements of the work. This type is widely used because it works well over a large range of velocities. Because of its shape, it is principally applicable to open channel measurements.

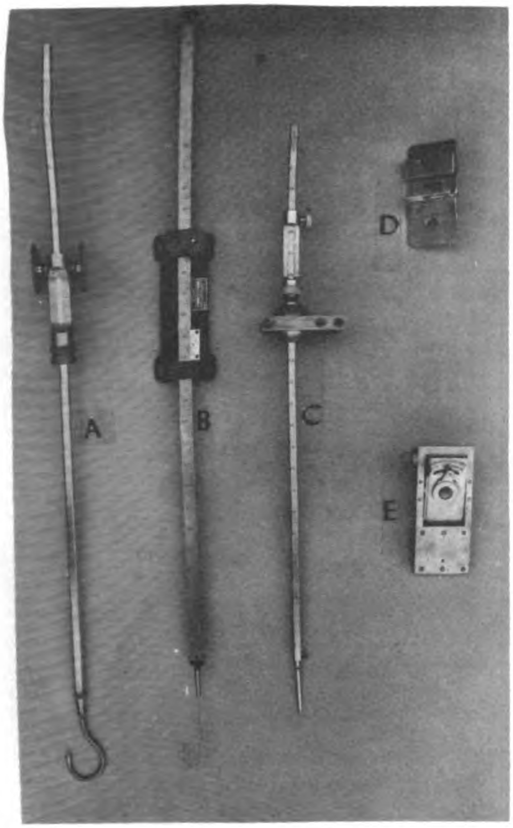
2 "Fluid Meters, Their Theory and Application," published by the ASME, Fourth Edition, p. 38.

3 Marks, Lionel S., "Square-edged Inlet and Discharge Orifices for Measuring Air Volumes in the Testing of Fans and Blowers," Transactions, ASME, AER-58-7, p. 593.

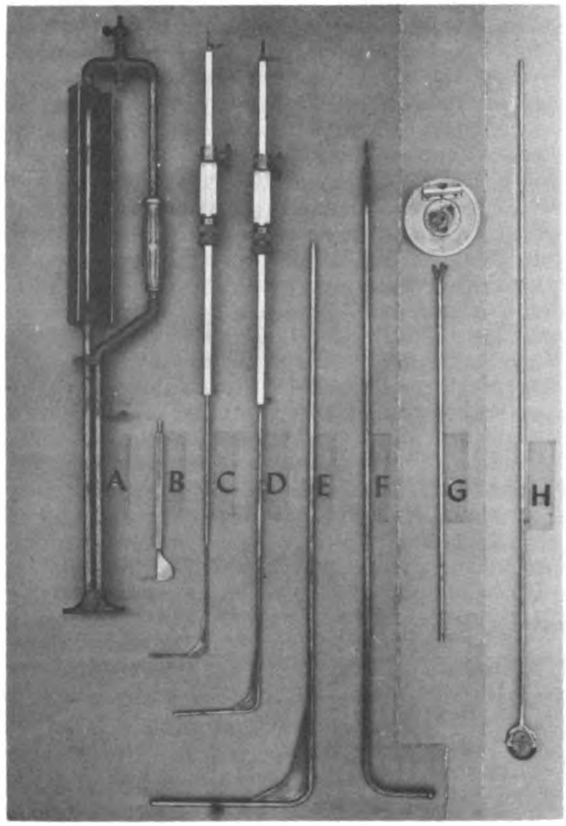
4 O'Brien, M. P., and Folson, R. G., "Modified I. & A. Orifice with Free Discharge," Transactions, ASME, R.P.-59-1, p. 61.

5 "The Use of an Elbow in a Pipeline for Determining the Rate of Flow in the Pipe," Engineering Experiment Station Bulletin No. 289, University of Illinois, Urbana, Illinois.

6 "Manual for Measurement of Irrigation Water," United States Department of the Interior, Bureau of Reclamation publication, October 1946.



A. Point gages, hook gages and mounting devices



B. Instruments for measuring velocity

Figure 17. Laboratory instruments for measuring water-surface elevations and velocities

Tube F is a ball-nose Pitot tube which is used occasionally where its size does not materially interfere with the flow. The nose is made in the form of a sphere, except for the shaft on the downstream side. A circular port centered on the upstream side of the sphere serves as the kinetic leg and an annular slot, cut at an angle of 68° with the horizontal shaft, leads to the static connection. This tube has an advantage in that it registers approximately 1.5 times the theoretical velocity head over the range of velocities for which it was designed.

Tube G is a Pitot cylinder suited for measuring medium and low velocities in closed conduit work. It consists of a circular rod supported at both sides of the conduit through packing glands. Extension rods fit into the lower end of this Pitot tube so that it extends completely through the conduit. Usually packing glands are provided 90° apart in the conduit; thus it is possible to make two velocity traverses, one normal to the other. Tube G has three ports in a plane normal to the centerline of the shaft. The center or kinetic port is directed into the stream of flow by balancing the pressures on the two outer ports. This is a special purpose Pitot tube constructed for a specifically narrow range of velocities. For the design range the tube registers approximately 1.33 times the theoretical velocity head. The coefficients vary with the velocity for spherical and cylindrical Pitot tubes as the pressure distribution on these shapes changes with the Reynolds number. For this reason, spherical and cylindrical tubes must be used only within the calibrated range.

The instrument shown as H on Figure 17B is a miniature current meter used frequently in open channel work for measuring velocities ranging from 0.1 to 6.0 feet per second. Each revolution of the propeller is registered by an electrical impulse which can be transmitted to a headset, chronograph, or oscillograph.

Three methods of mounting Pitot tubes and point gages for open channel work are illustrated on Figure 17A. The simplest and most widely used mount is shown as D. Where it is desired to record actual angles in Pitot tube work, a tilting head such as shown as E of Figure 17A is available. This head with micrometer screw not only indicates the vertical angle of tilt but allows some rotation of the Pitot tube. All heads are designed to operate along structural aluminum channels.

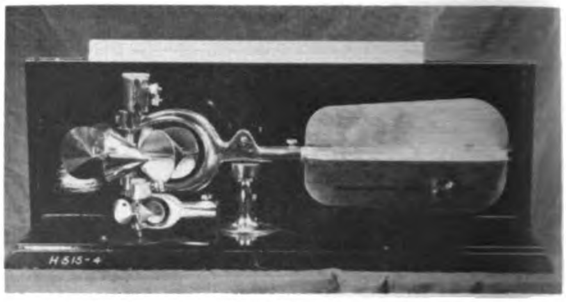
Price current meters of standard or pygmy sizes may be used to determine velocities in large open channel models. A standard and a pygmy Price current meter are shown in Figure 18A.

It may be necessary to design a velocity measuring device to cover an unusual range or occurrence of velocities such as are experienced in a tidal estuary. Such a device is the pendulum meter (Figure 18B) developed in the laboratory for measuring continually changing and reversing velocities. It consists of a sphere mounted on one end of a pivoted rod hung in the flow. The force of the current pushes the sphere downstream and the needle at the upper end of the rod simultaneously indicates the momentary velocity on the calibrated scale. The instrument can be adapted to any range of velocities by changing the length of the pendulum or by substituting spheres of different diameters, or both.

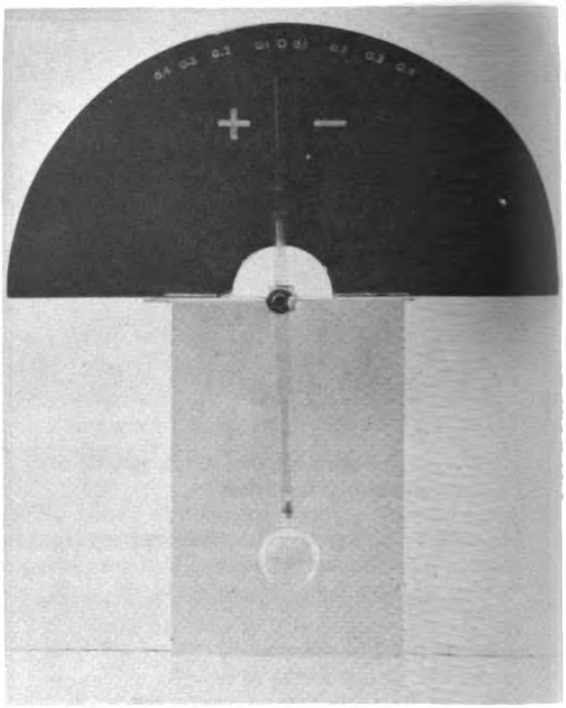
Other special devices for measuring velocities have been developed. Sometimes photographs can be used to determine velocities. Submerged or floating particles such as powdered aluminum or confetti can be photographed and appear on the print as lines or streaks. If the exact shutter speed of the camera is known and a length scale is included in the photograph, velocities at various points can be computed.

Pressure Measurements

The simplest device for measuring positive water pressures is the single-leg manometer. Some of these are shown in the center of Figure 19. Each glass tube is connected by flexible tubing to a piezometer tap located at the point on a model where a pressure measurement is desired. The pressure is then read directly in feet of water above the point. This type of manometer is not suitable for measuring subatmospheric pressures. With tubes as small as those shown in Figure 19, a wetting agent, such as a few drops of aerosol, is inserted in each tube to minimize surface tension. The water in these tubes is often colored by adding a trace of fluorescein to improve definition of the meniscus. The U-tube shown in Figure 19 is a simple device which can be used to measure subatmospheric as well as positive pressures. The magnitude of the



A. Standard and pygmy Price cur-



B. Pendulum velocity meter

Figure 18. Velocity meters

pressure to be measured determines the gage liquid. Water and mercury are the most common liquids.

Piezometers sometimes give false readings for no readily apparent reason; therefore, conclusions as to the pressures should not be made from a single piezometer.[7] Piezometers should be installed in groups

[7] Allen, C. M., and Hooper, L. J., "Piezometer Investigations," Transactions, ASME, 1932, Vol. 54, p. 1.

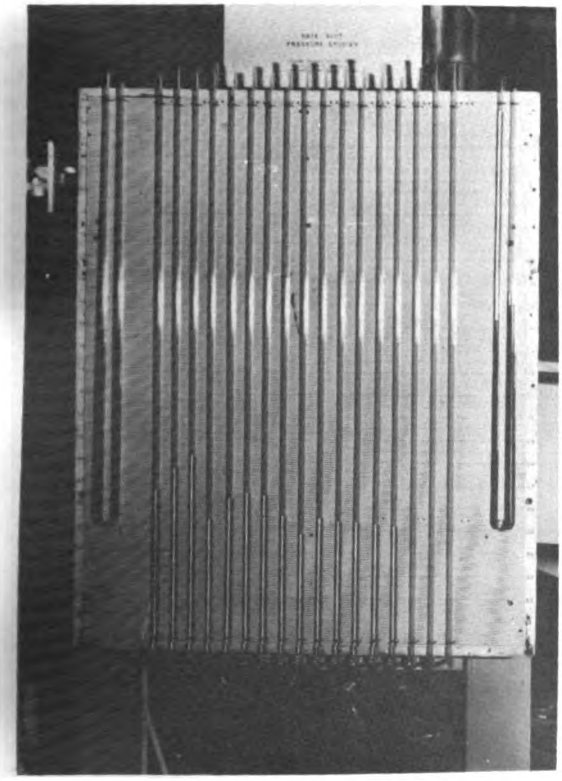


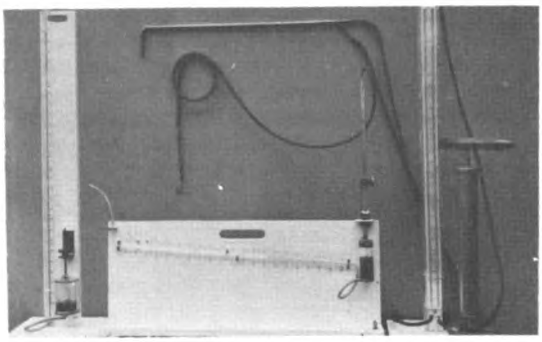
Figure 19. Measurement of pressures with manometers

and connected to suitable indicating manometers that may all be viewed simultaneously. Any single pressure which appears out of line may be checked immediately. The practice of using one manometer and connecting it in turn to different piezometer taps is to be avoided because faulty readings are difficult to detect. Conclusions as to pressures should be based on average values of repeated piezometer readings, the number of which depends on the accuracy desired. Piezometers of the same group should be read simultaneously to insure the proper relation between adjacent piezometers. Simultaneous readings may be obtained by pinching off all connecting tubes at the same instant with a mechanical device or by photographing the manometer board. The latter method is preferred because the manometer readings may be checked for errors, if necessary, after the model has been disassembled.

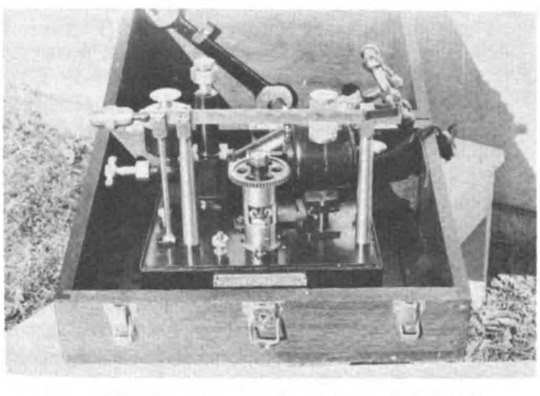
The gage on the left, Figure 20A, is a pot-type manometer in which a reservoir is substituted for one leg of a U-type manometer. The gage is designed to read single or differential pressures directly on one scale. When water is the manometer liquid, the use of the gage is limited to measurement of positive or negative air pressures or negative water pressures. For positive water pressures, mercury is used as the manometer liquid. For the smaller positive water pressures encountered in the laboratory, however, simple single-leg water tubes are preferred.

The center gage in Figure 20A is a pot-type sloping manometer used in air model testing. The sloping tube magnifies the movement 10 times and the movable pot enables measurement over a greater range than would be possible with the inclined tube alone. A leveling bubble on the frame makes it possible to set the tube on the proper slope, which is 10 to 1. A trace of aerosol is necessary in the water gage solution to decrease surface tension.

The gage on the right in Figure 20A is a Pitot tube manometer. The Pitot tube is connected as shown and the nose is immersed in water. A vacuum pump is then



A. Pot-type pressure manometer and pitot tube gage



B. Fluid pressure scale for weighing pressure up to 300 psi

Figure 20. Laboratory instruments for measuring pressure

used to lower the pressure in the manometer, thus causing the water surface to rise in both legs of the gage. A needle valve in a small reservoir behind the gage is then opened for an instant allowing a few drops of aerosol to enter each leg of the manometer for the purpose of reducing surface tension. In this case the differential pressure, or velocity head, is the only measurement desired, and it can be read directly by means of the sliding scale.

For measurement of positive water pressures up to 100 feet of water, mercury U-tubes or pot gages, are usually employed. For higher pressures a fluid-pressure scale (Figure 20B) is available. This is an adaptation from a dead-weight testing machine in which the force of fluid pressure against a piston is actually weighed. Movement of the piston actuates a system of levers. Loading the main lever arm until a balance is obtained weighs the pressure. The motor slowly revolves the piston and thus minimizes static friction between the piston and the walls of the cylinder. Pressures as great as 300 psi can be measured with this instrument.

For pressures greater than the capacity of the fluid-pressure scale, Bourdon-type indicator gages are used. These gages are not suitable for accurate hydraulic model testing because they are difficult to maintain in accurate working condition if moved often.

Measurement of Time

A high degree of accuracy in time measurement is particularly important in calibrating laboratory equipment, and sometimes in determining time intervals for model study. A stop watch, accurate to about one-fifth of a second, may be used for most work. Where greater accuracy is required, an electronic counter (Figure 12) enables measurement to 1/100 of a second. If even greater accuracy is desired, a tuning fork incorporated in an oscillograph serves as a timing mechanism and enables the recording of time to one one-thousandth of a second.

Special Instrumentation

There is a growing application of electronics in the development of special instruments for use in hydraulic investigations, because such apparatus enables observation of hydraulic phenomena which were not previously measurable. The design, construction, and operation of electronic devices require the services of specialists.

Need for special instrumentation in the Hydraulic Laboratory and in the field arises when transient or unsteady phenomena are involved. In the case of hydraulic models, measurements may include velocities, pressures, water depth, wave characteristics, and amplitudes of vibrations: in field measurements, all of the foregoing may be involved; also, salinity determinations, sedimentation studies, and geophysical surveys. Laboratory controls for regulation of water level, of gate and valve openings, and of motor speeds are often required. Sometimes very highly specialized laboratory apparatus such as a magnetostriction oscillator is required in studies of cavitation erosion.

1. Design of instruments.--Specifications commonly require that the calibration of instruments be reproducible within 1 percent of their full range. To meet that standard, the process of development cannot be rushed. Requests for particular instruments should, therefore, be made as soon as definite need is apparent. Design procedure usually consists of choosing or developing basic components such as pick-up devices, recorders, coupling circuits, or amplifiers that are best suited to the purpose. In some cases the problem may be solved by adaptation of an instrument previously developed or commercially available. For example, to test for vibration in a radial gate under investigation in the laboratory, it was only necessary to suspend the gate from the armature of a pressure cell. A high degree of sensitivity to vibration can be obtained in this manner.

The pick-up is the part of the equipment that conditions the quantity being measured for recording, and its design is of utmost importance in instrumentation. Selection of the type of pick-up is based on a consideration of frequency, sensitivity, and physical requirements. Close attention must be given to the frequency of the phenomenon whose characteristics are to be recorded. An example is found in a vibrometer designed to measure the amplitude and frequency of a vibration. Above a particular frequency, the meter responds quite satisfactorily, but below the natural frequency of the instrument, it acts as an accelerometer. In order that the moving parts of any pick-up follow the applied variations, momentum must be kept to a minimum, or the natural frequency of the moving parts must be high compared to the measured frequency. This assures a negligible time lag between signal and response. Proper sensitivity in the pick-up is essential. An instrument may respond to the desired quantity, but because of low sensitivity, important data will not be emphasized enough to bring out the true characteristics. Physically, the pick-up should be built strong enough to withstand considerable abuse without losing its calibration. Many times, instruments designed and built for the laboratory are taken to the field for measurements on the prototype. A weak, poorly designed pick-up will give questionable results. For example, the pressure-cell diaphragm mounting is designed to withstand many times the maximum pressure it may be expected to measure. There is a balance point in all pick-up devices at which there will be a small sacrifice of sensitivity and response in favor of ruggedness.

The choice of recording instrument is, in most cases, determined by the frequency of the measured variable. Water stage recorders are of both pen and ink and pencil types. These are used for recording diurnal variations. Variations from 0 to 60 cycles per second may be easily recorded using the sensitive galvanometers of a recording oscillograph. Frequencies up to 4,500 cycles per second may be recorded on less sensitive galvanometers. Phenomena of much higher frequencies may be recorded by photographing the trace on the screen of a cathode-ray oscilloscope. The physical requirements are not a major consideration in this case, since a recorder is nearly always a precision instrument and will be given the necessary care to insure its proper operation. Also, it is generally operated under favorable conditions.

Coupling circuits used to connect the pick-up to the recorder may be mechanical, electrical, or electronic. In any case, they operate to amplify, attenuate, or match the output of the pick-up to the recorder. The chief requirement placed on this component is that of adequate frequency response. Mechanical linkage may be used as in the case of some types of recording thermometers. Adjustments must be provided for calibration and for zeroing the equipment. If electrical or electronic linkage is used, the recorder must have a current-sensitive coil to give the deflections. This consists of a small coil within a strong magnetic field. Impedance matching is of paramount importance in the design of this linkage, because it prevents false indications or distortion of the records.

If electronic amplifiers are employed in the coupling circuits, adequate fidelity, stability, and control are the essential elements. These are attained by use of proper decoupling circuits, carefully designed filter circuits throughout, negative feed-back circuits, and quality parts. Wherever possible, commercially available components are used to facilitate servicing the equipment. Some equipment requires special wave-shaping circuits, such as those used in the electronic steel detector or in telemetering circuits. Most of the variations studied and recorded in hydraulic testing take place within the low frequency range of 0 to 30 cycles per second. An electronic amplifier for these frequencies is difficult to build from standard parts. It is common practice, therefore, to use a much higher frequency as a carrier and modulate it with the low frequency pick-up signal. This requires an electronic oscillator of adequate frequency, stability and power out-put characteristics. The pick-up is part of a bridge arrangement in which the off-balance out-put voltage is proportional to the variable to be measured. The out-put voltage is amplified and presented to a phase-sensitive network in order that the bridge unbalance in either direction may be detected. Certain types of pick-ups generate their own voltage, such as the geophone used in geological investigations of dam sites. In this case, the amplifiers used are designed to match the impedance of the geophone, to provide adequate amplification, and to operate galvanometers in the recording oscillograph.

Some types of instrumentation may be unnecessarily complicated by overextensive use of electronics. Other components that will accomplish the same results and simplify the instrument may be substituted. For example, the associated circuits required with the pressure-cell have been reduced to two transformers and two copper-oxide rectifiers with a simple filter arrangement. The electronic amplifiers originally used have been eliminated. Only an electronic oscillator and its amplifier are required to supply the carrier frequency to the pressure-cell circuits. Since a carrier current is supplied to the pressure cell, its low frequency response is unlimited, and can be calibrated under static conditions. The upper limit of response is governed by the carrier frequency and by the natural frequency of the moving parts. Twenty-five percent of the carrier frequency or of the natural frequency of the pick-up, whichever is lower, is considered as the upper limit of response. The carrier frequency in general use in hydraulic testing is 1,000 cycles per second. The natural frequency of the moving parts within the pressure cell is almost always higher than the frequency of the pressure variations encountered, except when cavitation is occurring. Special components are required for this condition. One of the chief difficulties to be overcome in instrumentation work is that caused by the effects of temperature variations. If the instrument is not properly developed, temperature effects may approach the magnitude of the quantity being measured. These effects must either be compensated in the circuits, or made exceedingly small, by choosing the proper components in construction.

2. Examples of special instrumentation. -- To illustrate the type of electronic work performed in the laboratory, a few examples are given.

Measurement of transient pressures that result from the operation of a valve, gate, or pump was solved by development of a pressure cell and necessary associated circuits. The pressure cell is of the variable inductance type, as shown in Figure 21A. It consists of two diaphragms having their centers connected by a rod. Mounted on the rod is a small soft-iron laminated armature oriented in such a way that it is mid-way between two coils. The coils, which are nearly identical, have laminated cores and approximately 300 turns of wire. They are connected into a bridge arrangement so that at balance and with atmospheric pressure applied, there is zero out-put from the circuit. With hydraulic pressure applied to one of the diaphragms, the armature is moved a proportional amount, thereby increasing one air gap and decreasing the air gap for the other coil. The bridge circuit is unbalanced by an amount that is proportional to the applied pressure. The unbalance voltage is then rectified and applied to a recording instrument. The circuit is so designed that it differentiates between positive and negative pressures. Energizing potential for the bridge is supplied from an electronic oscillator at a frequency of 2,000 cycles per second. The frequency is high enough to permit study of phenomena having frequency components as high as 500 cycles per second. As the bridge has sufficient out-put for recording directly on an oscillograph, amplifiers are unnecessary. Figure 21B shows the equipment used for recording pressure variations on the miter gates on the dry dock at Grand Coulee Dam.

A salinity meter developed in the laboratory is used to study the distribution of ocean salt in the Sacramento-San Joaquin Rivers during the seasonal period of low water flow. Control of saline water is mandatory because the quality of irrigation water must be kept within specified limits at all times. The meter measures the conductivity of the river water and records a value that is proportional to the concentration of total dissolved solids. A river stage recorder has been adapted to this use by addition of a self-balancing bridge, one leg of which is the conductivity cell that continuously samples the river water. A second conductivity cell filled with a standard solution, sealed and insulated electrically from ground, is used in the bridge circuit for temperature compensation. The unbalance voltage amplifier is at the left in Figure 22 and the bridge chassis, mounted on the stage recorder, is at the right. The conductivity cells, not shown, are connected to the bridge chassis. The plotting mechanism is a Stevens A-35 recorder.

An electronic analogue was constructed to represent the electrical equivalent of the rivers and sloughs in a portion of the Sacramento-San Joaquin Delta in the Central Valley, California. All the factors which operate to determine the division of flow among the various channels, shown on the chart of Figure 23, have been represented electrically. The purpose of the electrical model, or analogue, is to facilitate computation of the combined tidal and stream flows in the various natural and proposed artificial channels. The need for computation arises in connection with the evaluation of the capacity of various artificial channels to transfer water from the Sacramento River to the San Joaquin River, and thus avoid expensive diversion dams or pumping plants. In particular, the analogue permits ready evaluation of the possibilities of increasing the transfer by tidal pumping, that is, by manipulation of a gate to capture favorable tidal flow. The corresponding electrical and hydraulic quantities are shown in the following table:

Hydraulic	Electrical
Elevation of water surface (head)	Voltage
Discharge	Current
Tidal oscillation in water surface	Alternating voltage
Inertia due to mass	Inductance
Frictional resistance of channel	Electrical resistance
Storage of channel	Capacitance

The ratios interrelating the analogous hydraulic and electrical quantities were chosen to permit easy transference. For example, one milliampere of current corresponds to 10,000 second feet. The special feature of this analogue is an electronic circuit shown in Figure 23 which gives an electrical resistance that changes with current in the same nonlinear fashion that the channel resistance changes with discharge.

3. Operation of instruments. --After the equipment has been designed, constructed, calibrated, and tested for accuracy and stability, preliminary tests are made on the model for which it was intended. Study of the test records in collaboration with those who requested the apparatus may suggest instrumental refinements that would provide more significant data. After such refinements are made, the instrument is ready for use. Although operation of most instruments is relatively simple, it is commonly assigned to instrumentation specialists. This assures good records and fosters suggestions for further development and simplification. An example of this is found in the assembly and operation of a field truck for measurement purposes. This truck was designed and built to provide mobile facilities required in hydraulic, geological, and structural field tests and surveys. It contains all the power equipment and instrumentation required for such tests and could be aptly termed a mobile testing laboratory. The complexity of the equipment requires that the operator be experienced in its use and capable of the necessary service and maintenance.

Other items of special equipment or instrumentation for use in laboratory and field studies will be discussed in connection with their specific application.

HYDRAULIC SIMILITUDE

Laws of Similitude

Hydraulic Laboratory practice is based on the application of a set of relationships which are known as the laws of hydraulic similitude. These laws, which are developed from the basic relations of fluid mechanics, express the interrelations of the various fluid parameters such as velocity, pressure, shear, etc., under similar boundary conditions. A knowledge of hydraulic similitude is essential to an enlightened use of hydraulic models.

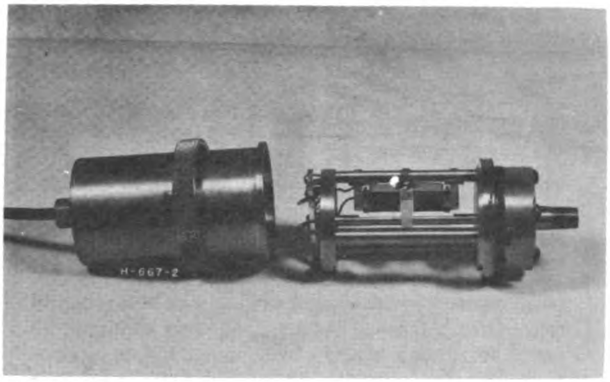
Similitude, as applied to hydraulic models, goes considerably beyond the superficial aspects of geometric similarity with which it is sometimes erroneously identified. Similitude can be defined as a known and usually limited correspondence between the behavior of a model and that of its prototype, with or without geometric similarity. The correspondence is usually limited because it is impracticable to construct and operate a model in which all the conditions required for complete similitude (geometric, kinematic, and dynamic similarity) are satisfied, even though these conditions are known. The term "similitude" should be qualified to indicate the general limits of correspondence, or, one might speak of several similitudes, each of which has a definite set of limitations.

The application of similitude in hydraulic model testing is based on recognition of the fact that although complete similitude is impracticable, there are several imperfect similitudes which can be exploited as required. Experience has demonstrated that any given problem can be simplified into the interplay of two major forces. It is therefore possible to develop the pertinent similitude by theoretical means. Each similitude consists of a set of transference ratios which may be applied to model findings in predicting prototype behavior.

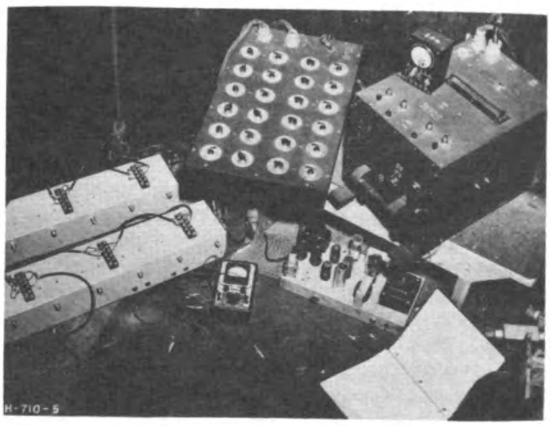
In addition to similitudes having an analytical basis, there are others which are developed, by experiment with the model, for application in special cases, such as the prediction of changes in bed configuration in erodible channels. Although attempts are being made to develop a more rational basis for transference of experimental results in such cases, the current procedure is based primarily on empirical relations. In other cases, the only basis for predicting prototype behavior from model experiments is one compounded of experience and hydraulic sense. The techniques which have been mastered by engineers experienced in this field permit them, by adjusting and applying model results in a nonformalized manner, to arrive at reasonably accurate predictions of prototype action. In general, the value of a model depends on the degree of accuracy with which it demonstrates that phase of the behavior of its prototype being investigated.

The laws of similitude play an increasingly important role in present-day applications of fluid mechanics. The mechanical engineer predicts the performance of his turbines, pumps, gates, and valves by model testing; the ship designer studies his models in the towing basin, or water tunnel, to reveal their behavior; the aeronautical engineer has his wind tunnel for study of the aerodynamic properties of aircraft design; and the civil engineer, by testing models of hydraulic structures, can predict flow characteristics of the prototypes or develop new design procedures.

The laws of similitude for flowing liquids and gases are relationships derived from Newton's second law of motion. (Force = mass x acceleration.) Application of these laws, however, is difficult and requires techniques and procedures developed through experience with models and field structures. Many treatises have been written on the sub-



A. Variable inductance diaphragm-type pressure cell



B. Power supply oscillator AC bridge and rectifiers for above pressure cell

Figure 21. Electronic instruments for measuring pressure

ject, but the majority of these do not state the limitations clearly and are quite involved for the average engineer reader. An attempt is made here to develop the similitude relationships as clearly and as logically as possible from a practical standpoint.

Geometric Similarity

Two objects or systems are geometrically similar if the ratios of all corresponding linear dimensions are equal. This is independent of motion of any kind and involves only similarity in form. Similarity of length can be expressed as follows:

$$\frac{L_m}{L_p} = L_r$$

where L_m and L_p are corresponding linear dimensions in model and prototype, respectively, and L_r is the length ratio or simply scale ratio.

Then

$$\text{Area ratio } A_r = \frac{L^2{}_m}{L^2{}_p} = L^2{}_r$$

$$\text{Volume ratio } V_r = \frac{L^3{}_m}{L^3{}_p} = L^3{}_r$$

and the hydraulic radius ratio

$$R_r = \frac{A_m\, P_p}{P_m\, A_p} = \frac{L^2{}_m\, L_p}{L_m L^2{}_p} = \frac{L_m}{L_p} = L_r$$

where P represents wetted perimeter. The scale ratio will then read 1/50, 1/100, etc.

Kinematic Similarity and Dynamic Similarity

Kinematic similarity is similarity of motion. When the ratios of the components of velocity at homologous points in two related or geometrically similar systems are equal, the two states of motion are kinematically similar, and the paths of homologous particles will also be geometrically similar.

Dynamic similarity between two geometrically and kinematically similar systems requires that the ratios of all homologous forces (including the force of inertia) in the two systems be the same. This follows Newton's second law of motion which can be written:

$$Ma = \text{vector sum } F_p + F_g + F_v + F_t + F_e \quad (1)$$

where

Ma = mass reaction to the forces acting, considered herein as the inertial force

F_p = pressure force connected with, or resulting from, the motion

F_g = force imposed on the liquid mass by gravity, as represented by its weight

F_v = viscous shear force

F_t = force due to surface tension, and

F_e = forces produced by elastic compression of the fluid

For over-all similarity in two fluid motion occurrences, the ratio of the inertia forces, model to prototype, must equal the

Figure 22. Instrument for continuous recording of salinity in Sacramento-San Joaquin Delta, Central Valley, California

ratio of the vector sum of the active forces, model to prototype.

Or

$$\frac{M_m a_m}{M_p a_p} = \frac{(F_p+F_g+F_v+F_t+F_e)m}{(F_p+F_g+F_v+F_t+F_e)p} \qquad (1a)$$

where the subscripts m and p represent model and prototype, respectively. Perfect similitude requires in addition that

$$\frac{M_m a_m}{M_p a_p} = \frac{(F_p)m}{(F_p)_p} = \frac{(F_g)m}{(F_g)_p} =$$

$$\frac{(F_v)m}{(F_v)_p} = \frac{(F_t)m}{(F_t)_p} = \frac{(F_e)m}{(F_e)_p} \qquad (1b)$$

All but one of these ratios may be regarded as independent, that one being fully determined once the others are established. The pressure ratio is usually regarded as the dependent quantity, and therefore it does not play a controlling part in the following discussion of similitude techniques.

No model fluid is known which has the requisite viscosity, surface tension, and elastic modulus to satisfy the conditions of Equation 1b. Moreover, hidden in the term for viscous forces is the effect of boundary roughness which is equally difficult to adjust for complete similitude.

Experience has shown that it is not so difficult to apply Equation 1a, as it would at first appear, because one or more of the forces may not act in the flow occurrence in question, while some may act only to a negligible amount or may be related to the most prominent force. In fact, for all practical purposes, a particular state of fluid motion can usually be simulated in the model by considering only one of the forces on the right of Equation 1a. In at least 90 percent of all hydraulic model studies, the forces connected with surface tension and elastic compression are relatively small and can be neglected safely. From a practical standpoint, a particular fluid motion can be represented in a model by considering that

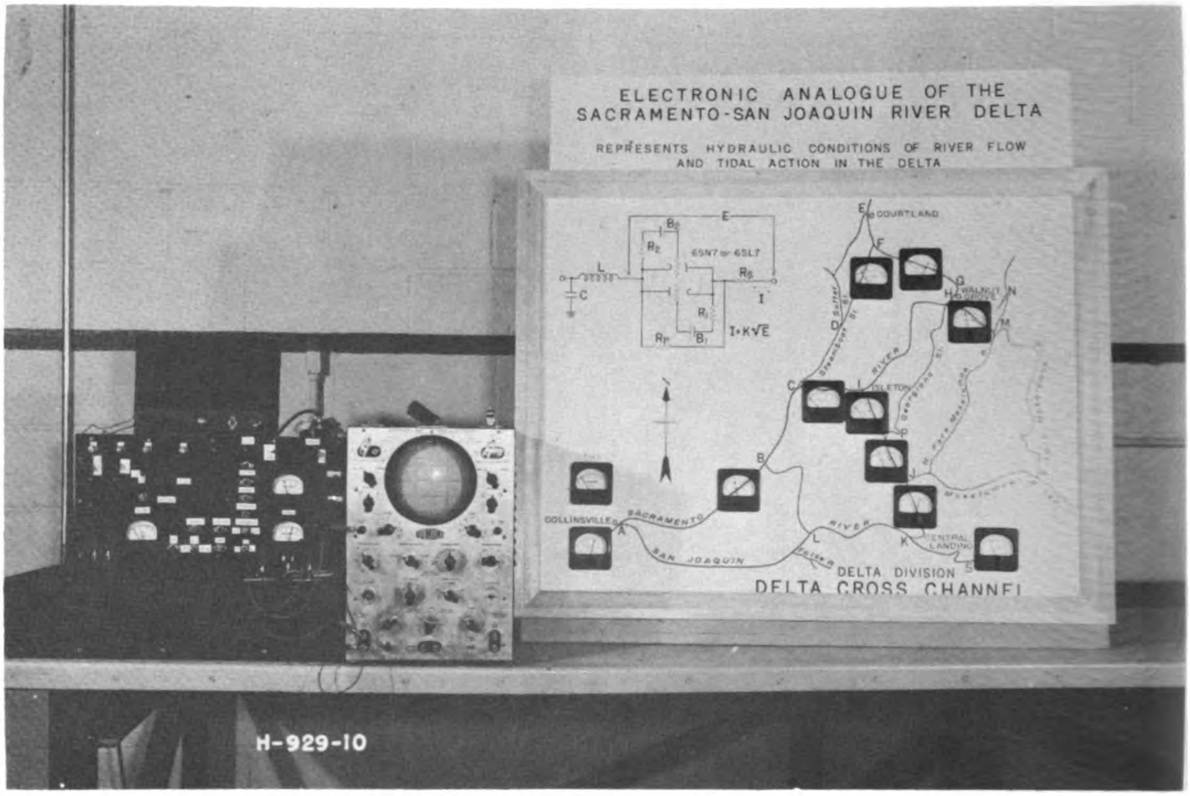


Figure 23. Electronic analogue of the channels and sloughs in the Sacramento-San Joaquin Delta, Central Valley, California

either gravity forces or viscous forces predominate.

The Froude Law

When gravitational effects predominate, a practical basis for similitude can be established by equating the ratio of gravitational forces to that of inertia forces and neglecting the other forces in Equation 1a. Before doing so, the inertia force of Equation 1 will be expressed in dimensional terms.

$$\text{Since mass} = \frac{W}{g} \text{ or } \frac{\gamma L^3}{g}$$

where γ is the specific weight of the liquid, and

$$\text{acceleration} = \frac{L}{t^2},$$

$$Ma = \frac{\gamma L^3 L}{gt^2} = \frac{\gamma L^4}{gt^2}$$

Substituting ρ for $\frac{\gamma}{g}$ and V for $\frac{L}{t}$

$$Ma = \rho L^2 V^2$$

where ρ is density of fluid, L is length, and V is velocity.

Dimensionally, the gravity force is $F_g = \gamma L^3$. Considering only the gravity force on the right of Equation 1,

$$Ma = F_g = \gamma L^3$$

or

$$\rho L^2 V^2 = \gamma L^3$$

Substituting ρg for γ in the right-hand member

$$\rho L^2 V^2 = \rho g L^3$$

Expressing this in model prototype ratios as suggested in Equation 1a:

$$\frac{L_m^2 V_m^2}{L_p^2 V_p^2} = \frac{g_m L_m^3}{g_p L_p^3}$$

or

$$L_r^2 V_r^2 = g_r L_r^3$$

and

$$\frac{V_r^2}{g_r L_r} = 1, \text{ or } \frac{V_r}{\sqrt{g_r L_r}} = 1 \qquad (2)$$

The expression $\frac{V}{\sqrt{gL}}$ applied to either the model or the prototype, is known as the Froude number. The equality of the number in model and prototype as expressed by Equation 2 is known as the Froude law.

The Reynolds Law

When viscous forces predominate, the basis for similitude is obtained by equating the ratio of viscous forces to that of inertia forces and neglecting the other forces in Equation 1a. By definition[8] $F_V = \mu LV$ where μ is the coefficient of viscosity of the fluid. From Equation 1a

$$Ma = \mu LV.$$

Expressing the above in model-prototype ratios,

$$\rho_r L_r^2 V_r^2 = \mu_r L_r V_r$$

and

$$\frac{\rho_r L_r V_r}{\mu_r} = 1 \qquad (3)$$

which is usually written

$$\frac{L_r V_r}{\nu_r} = 1 \qquad (3a)$$

where ν is the kinematic viscosity defined as $\frac{\mu}{\rho}$.

The Reynolds number is simply $\frac{LV}{\nu}$. Equation 3a states that for complete similarity with respect to viscous effects the Reynolds numbers should be equal in model and prototype.

The Cauchy or Mach Law

In those special cases where either surface tension forces or forces resulting from elastic compression predominate, the basis for similitude is obtained in a similar manner by equating the ratio of surface tension forces or the ratio of elastic forces to the ratio of inertial forces. In the case of surface tension forces, one obtains the Weber law:

$$\frac{\rho_r L_r V_r^2}{\sigma_r} = 1 \qquad (4)$$

where σ is a measure of surface tension.[9] In the case of elastic forces one obtains the Cauchy or, as it is sometimes known, the Mach law:

$$\frac{\rho_r V_r^2}{E_r} = 1 \qquad (5)$$

where E is the bulk modulus of the fluid. An alternative form of this expression is obtained by introducing the celerity or velocity of a pressure wave;

$$C = \sqrt{\frac{E}{\rho}},$$

and defining the Mach number as:

$$M = \frac{V}{C} = \sqrt{\frac{\rho V^2}{E}} \qquad (5a)$$

Other Similitude Aids

Two additional parameters useful in hydraulic model testing are the cavitation number and the Karman number. The cavitation number serves as an index by which the experimenter can predict the effects of cavitation in prototypes. The Karman number offers a means of establishing the same type of turbulent flow in the model as exists in the prototype and is useful in the design of river models.

Alternative Method for Developing Similitude Relationships

A somewhat different approach to the development of the dimensionless parameters such as the Froude and Reynolds numbers, while not new, has been increasing in popularity in recent years. The method appears to be more difficult in a mathematical sense then the one presented and for this reason it is not given in detail. It will be sketched, however, for those who may be interested because it is held to be a more

[8] Russell, George E., "Hydraulics," Fifth Edition, Henry Holt Company, 1945, p. 93.
[9] Rouse, Hunter, "Elementary Mechanics of Fluids," John Wiley and Sons, 1946, p. 328.

fundamental and general method. It consists in general of the following procedure which is used by Goldstein.[10]

The fundamental equations of the fluid motion are written first. Then the variables are replaced by dimensionless variables formed by relating each variable to the value of the variable at some reference point in the system under consideration. For example, linear variables may be expressed by their ratios to some reference length such as pipe diameter, channel depth, etc. Velocities may be expressed by their ratios to a known velocity such as the velocity of approach or a fictitious velocity such as the mean velocity of transport (discharge divided by area of section). The fundamental equations are restated in terms of the dimensionless variables. The transformed equations can be shown to be identical with the original equations except for the insertion of one or more coefficients. These coefficients are the Froude, Reynolds, Mach, and Weber numbers.

When the values of the coefficients are the same for two flow occurrences, the details of flow are determined by the same equations. Although in the usual case the equations cannot be solved, the conditions required to establish the applicability of the equations to the two flow occurrences also establish the conditions for similitude. The concept of similitude used here differs slightly from that previously discussed. Instead of defining similitude as equal ratios of flow variables at corresponding points in model and prototype, it is defined as the condition wherein the ratio between the variables at two locations in the model is equal to the corresponding ratio in the prototype. One concept can be derived from the other by an algebraic transformation. Partial similitude is obtained by neglecting one or more of the terms in the fundamental equations as the situation permits. For example, the neglect of all terms except those for the gravity forces results in the Froude law as the basis for the similitude.

Significance and Application of Similitude Laws

To satisfy more than one law in any given case would require that the physical properties of the testing fluid be variable over rather broad limits. For example, to satisfy the Froude and Reynolds laws simultaneously it would be necessary that:

$$\frac{V_r}{\sqrt{(gL)_r}} = \frac{L_r V_r}{\nu_r}$$

Since the gravitational constant is nearly always the same in model and prototype, that is $g_r = 1$, the kinematic viscosity ratio would have to be related to the length scale as follows:

$$\nu_r = L_r^{3/2} \qquad (6)$$

Satisfaction of this criterion for a hydraulic structure model with a length scale of 1 to 25 would require a testing fluid with a kinematic viscosity 1/125 that of water. Such a fluid is not available and if water is used as the testing fluid as is customary, the kinematic viscosity ratio is nearly unity.

Since the physical properties of practical testing fluid are such that only one similitude law can be satisfied in a given instance, the accepted procedure for applying similitude relations consists of selecting and applying the dominant law.

The conditions assumed in formulating the Froude law (Equation 2) are approximated in the case of turbulent flow with a free water surface, since the effects of gravity outweigh those of viscosity, surface tension, and elasticity. However, since the viscous forces are neglected, every effort should be made to minimize them by using large models and smooth boundaries. Usually, when the Reynolds number of the model exceeds a value of 10,000, and depth of flow is substituted for L in the Reynolds number, the viscous forces are relatively unimportant. If viscous effects cannot be ignored, it is possible to make compensating adjustments. Since most hydraulic problems involve free surface phenomena in one form or another, the Froude law of similitude is used more frequently than any of the other laws.

Steady flow in a pressure conduit, or flow around a deeply submerged body, approximates the conditions assumed in formulating the Reynolds law of similitude. Since there is no free surface connected directly with the flow pattern and the flow is steady, the forces of surface tension and elasticity are eliminated. Further, it can be proven that the gravity forces balance out and therefore do not affect the flow pattern. The Reynolds number in the model rarely can equal that of the prototype, but fortunately this is not necessary. If the Reynolds number of a model with smooth boundaries exceeds 1,000,000 for all per-

10 Goldstein, S., "Modern Developments in Fluid Mechanics," Oxford University Press, 1938, Vol. 1, p. 99 and seq.

tinent test flows, the boundary resistance forces become essentially independent of the Reynolds number. This can be shown by referring to Figure 45, where the dimensionless friction factor "f," in the Darcy-Weisback formula $h_f = f \frac{L}{D} \frac{V^2}{2g}$, is plotted with respect to the Reynolds number for flow in pipes with various relative roughness coefficients, denoted as $\frac{\epsilon}{D}$. For values of $\frac{\epsilon}{D}$ less than 0.001 the friction factor becomes nearly constant when the Reynolds number is greater than 1,000,000, and the ratio of resistance forces, model to prototype, depends only on the ratio of relative roughnesses. To obtain a satisfactorily large Reynolds number in the model, it is often necessary to increase the velocity by using exaggerated heads.

When surface tension is the predominating factor in a problem to be studied, the other three forces are usually insignificant and the Weber law applies. In engineering, surface tension is manifest in capillary waves on a water surface, capillarity in manometer tubes, capillary wetting action in filter beds and earth dams, tendency of a jet from an orifice to assume a circular cross section regardless of shape of orifice, and definite changes in flow over weirs at low heads. Surface tension seldom plays an important role but its effects are ever present and troublesome in model testing.

Except for cases of unsteady flow, especially water-hammer problems, the similitude based on the Mach number has little application in hydraulic model testing. Since most water-hammer problems yield readily to analytical methods, model studies are rarely needed. On the other hand, aerodynamic testing has led to an extensive development of the use of the Mach law to deal with problems involving the flow of gases at velocities exceeding the speed of sound, and water entry problems of ballistics have recently brought the analysis of liquid flow to the same stage.

Similitude Ratios

On deciding which of the four dimensionless parameters is relevant to the problem at hand, the similitude ratios can be developed from that parameter. For a case of open channel flow where the Froude law applies, the model-prototype relationships are obtained directly from Equation 2.

$$V_r = \sqrt{g_r L_r}$$

Since $g = \frac{\gamma}{\rho}$ the velocity ratio can also be written

$$V_r = \left(\frac{\gamma L}{\rho}\right)_r^{1/2}$$

Other kinematic relationships are as follows:

$$\text{time ratio } t_r = \frac{L_r}{V_r} = \frac{L_r}{\left(\frac{\gamma L}{\rho}\right)_r^{1/2}} = \left(\frac{L\rho}{\gamma}\right)_r^{1/2}$$

$$\text{acceleration ratio } a_r = \frac{L_r}{t_r^2} = \left(\frac{\gamma}{\rho}\right)_r$$

$$\text{discharge ratio } Q_r = \frac{L_r^3}{t_r} = L_r^{5/2} \left(\frac{\gamma}{\rho}\right)_r^{1/2}$$

The dynamic relationships are obtained in the same manner:

$$\text{mass ratio } M_r = L_r^3 \ \rho_r$$

$$\text{force ratio } F_r = M_r a_r = (L^3\rho)_r \left(\frac{\gamma}{\rho}\right)_r = (L^3\gamma)_r$$

These and additional relationships are listed for reference in Table 1. When the fluid in model and prototype is the same, both γ_r and ρ_r become unity.

Even though the model boundary surfaces are made as smooth as possible to minimize viscous effects, such as resistance, it is often necessary to make adjustments in the model design or operating technique to compensate for them. There are several methods by which such compensations are effected and they will be discussed in connection with the type of model to which each is applicable.

Table 1

Characteristic	Dimension	Scale ratios for the laws of	
		Froude	Reynolds
Geometric properties			
Length	L	L_r	L_r
Area	L^2	L_r^2	L_r^2
Volume	L^3	L_r^3	L_r^3
Kinematic properties			
Time	t	$\left[\frac{L\rho}{\gamma}\right]_r^{1/2}$	$\left[\frac{L^2\rho}{\mu}\right]_r$
Velocity	Lt^{-1}	$\left[\frac{L\gamma}{\rho}\right]_r^{1/2}$	$\left[\frac{\mu}{L\rho}\right]_r$
Acceleration	Lt^{-2}	$\left[\frac{\gamma}{\rho}\right]_r$	$\left[\frac{\mu^2}{\rho^2 L^3}\right]_r$
Discharge	L^3t^{-1}	$\left[L^{5/2}\left(\frac{\gamma}{\rho}\right)^{1/2}\right]_r$	$\left[\frac{L\mu}{\rho}\right]_r$
Dynamic properties			
Mass	M	$(L^3\rho)_r$	$(L^3\rho)_r$
Force	MLt^{-2}	$(L^3\gamma)_r$	$\left[\frac{\mu^2}{\rho}\right]_r$
Density	ML^{-3}	ρ_r	ρ_r
Specific weight	$ML^{-2}t^{-2}$	γ_r	$\left[\frac{\mu^2}{L^3\rho}\right]_r$
Pressure intensity	$ML^{-1}t^{-2}$	$(L\gamma)_r$	$\left[\frac{\mu^2}{L^2\rho}\right]_r$
Impulse and momentum	MLt^{-1}	$\left[L^{7/2}(\rho\gamma)^{1/2}\right]_r$	$(L^2\mu)_r$
Energy and work	ML^2t^{-2}	$(L^4\gamma)_r$	$\left[\frac{L\mu^2}{\rho}\right]_r$
Power	ML^2t^{-3}	$\left[\frac{L^{7/2}\gamma^{3/2}}{\rho^{1/2}}\right]_r$	$\left[\frac{\mu^3}{L\rho^2}\right]_r$

In closed conduit problems where the Reynolds law applies, another set of similitude ratios may be developed from Equation 3 as follows:

$$\text{velocity ratio } V_r = \left(\frac{\mu}{L\rho}\right)_r$$

$$\text{time ratio } t_r = \frac{L_r}{V_r} = \frac{L_r}{(\mu/L\rho)_r} = \left(\frac{L^2\rho}{\mu}\right)_r$$

$$\text{acceleration ratio } a_r = \frac{L_r}{t_r^2} = \left(\frac{\mu^2}{L^3\rho^2}\right)_r$$

$$\text{and discharge ratio } Q_r = \frac{L_r^3}{t_r} = \left(\frac{L\mu}{\rho}\right)_r$$

These and other relationships for flow in which viscous forces predominate are tabulated for ready reference in Table 1.

Except for problems dealing with lubrication and a very limited number of hydraulic structure and equipment problems for which the prototype Reynolds number is low, these ratios cannot be used. For example, in a hydraulic model having a scale ratio of 1 to 20 it would be required that the velocity ratio be 20 to 1 to satisfy the Reynolds law. In most problems the prototype velocity will exceed 5 feet per second and a model velocity of over 100 feet per second would be required. Such velocities are beyond the capacity of most hydraulic laboratories. Usual procedure is to recognize the applicability of the Reynolds law of similitude and approximate it by operating the models at the highest velocity attainable in the laboratory.

In a similar manner, the similitude relationships governed by the Weber and Mach parameters can be developed from Equations 4 and 5, respectively. Treatises on hydraulic similitude and dimensional analysis are listed in footnotes 11, 12, 13, and 14.

Distorted Models

Models of river channels, estuaries, floodways, and canal structures are often distorted geometrically when the length is great in proportion to width and depth, otherwise the channels would be of insufficient cross section to obtain representative flow conditions unless the model were extremely large. In a distorted model, the depth scale ratio is not the same as the length and width scale ratios, and in an occasional model different scale ratios are used for length, width, and depth. This necessitates revision of the foregoing similitude relationships. The principal limitation, when considering a distorted model, is that the model should be proportioned so that conversions of kinetic to potential energy, or vice versa, will not be distorted beyond a tolerable degree. When a model is distorted, the distortion is planned to accomplish a definite objective. The results obtained from such a model are definitely limited to this objective.

The advantages of distorted models are: (1) sufficient tractive force can be developed to produce bed load movement with a reasonably small model and available model sediment; (2) water surface slopes are exaggerated, and therefore easier to determine; (3) the width and length of the model can be held within economical limits for the required depth; and (4) operation is simplified by use of a smaller model.

Several disadvantages are: (1) velocities are not necessarily correctly reproduced in magnitude and direction; (2) some of the flow details are not correctly reproduced; (3) slopes of cuts and fills are often too steep to be molded in sand or erodible material; and (4) there is an unfavorable psychological effect on the observer who views distorted models.

[11] "Hydraulic Models," ASCE, Manual on Engineering Practices No. 25, 1942 and 1945.

[12] "Dimensional Analysis and the Principles of Similitude as Applied to Hydraulic Experiments with Models," Hydraulic Laboratory Practice, ASME, 1929, p. 775.

[13] "Model Experiments and the Forms of Empirical Equations," Transactions, ASME, 1915, Vol. 37, p. 263.

[14] "Hydraulic Similitude," Engineering Hydraulics, Edited by Hunter Rouse, John Wiley & Sons. p. 136.

The similitude ratios for distorted models based on the Froude law are:

Horizontal length ratio	$= L_r$
Depth ratio	$= D_r$
Horizontal area ratio	$= L_r^2$
Vertical area ratio	$= L_r D_r$
Slope ratio	$= D_r / L_r$
Velocity ratio	$= D_r^{1/2}$

In problems requiring the use of distorted models, resistance often plays a significant part. Then the ratio of model to prototype resistance must be made the same as the ratio of gravity forces. The resistance may be related to the velocity by means of the well-known Manning formula.

$$V = \frac{1.49 R^{2/3} S^{1/2}}{n},$$

where S = the resistance slope
R = hydraulic radius
n = roughness coefficient

In order that the resistance slope equal the channel slope

$$S_r = \frac{D_r}{L_r} = \frac{n_r^2 V_r^2}{R_r^{4/3}} = \frac{n_r^2 D_r}{R_r^{4/3}},$$

then

$$n_r = \frac{R_r^{2/3}}{L_r^{1/2}} \text{ or } n_m = n_p \frac{R_r^{2/3}}{L_r^{1/2}}$$

The value of the hydraulic radius ratio (R_r) does not bear any fixed relation to the length and depth scales but varies with the shape of the cross section. Then, for any given reach, the R_r and the L_r are known and n_r can be computed, and applied to the prototype n to obtain the required value of n for the model. Usually the required value of n in the model must be obtained by artificially roughening the model channels. Unless basic data on the values of n for different types of roughness are available the appropriate roughness must be obtained by trial and error. The model with roughness adjusted for a particular depth will yield dependable results for flow at or near that depth. If problems of several depths occur, model roughness should be adjusted to give an average friction that is approximately right for each depth, or the roughness may be varied with depth for a closer approximation at all depths. The fact should not be overlooked, however, that the Manning n is strictly a roughness characteristic, and that the Manning formula applies only at sufficiently high values of Reynolds number where resistance forces are proportional to the square of the velocity.

Generally speaking, distorted models are not adapted to situations where curvatures in the water surface are involved. For example, the flow through spillways or between bridge piers will not be correctly represented in a distorted river model.

The Karman Number

An important precaution in testing distorted river models is to be sure that flow in the model is of the same type as that in the prototype. Flow is invariably turbulent in rivers and, as friction forces predominate, it appears that a criterion for model design would be the Reynolds parameter or some variation of the factors which constitute this number. A practice in some American laboratories has been to adjust the scales in the model so that when the hydraulic radius is substituted for length in the Reynolds number, the value will be not less than 2,500. The Waterways Experiment Station of the Corps of Engineers has used the criterion VR = 0.02 to insure turbulent flow, where V is average velocity in the channel and R is hydraulic radius. This corresponds to a limiting value of 1,800 for the Reynolds number for a temperature of 66° F.

The Neyrpic laboratory at Grenoble, France, goes a step further and undertakes to insure that the proper type of turbulent flow will be established; that is, a flow that may be described as the "rough wall type" or "rough turbulent type." This type is required in models for all cases except for very sluggish rivers. It is claimed that the Reynolds number, in the usual application, has never been a completely satisfactory parameter for open channel flow, and is not an adequate criterion for river model turbulence. In place of the Reynolds number the following parameter is proposed and called the "roughness Reynolds number" or the "Karman number," in honor of Professor von Karman of the California Institute of Technology:[15]

[15] Craya, A., "Similitude des modeles fluviaux à fond fixe," La Houille Blanche, 1948, Vol. 3, No. 4, p. 346.

$$\overline{K} = \frac{kV_*}{\nu},$$

where

$\overline{K}$ is the "Karman number,"

k is a roughness parameter, which for the usual range of slopes is proportional to the sand grain size

V_* = shear velocity $= \sqrt{RSg}$

In proportioning models to obtain the proper type of turbulence, it is recommended that the Karman number be not less than 100. This criterion was used to advantage on a model of the Rhine River at the Neyrpic Laboratory.

LABORATORY STUDY

General Considerations

For the purposes of this monograph a laboratory study may be defined as any investigation of a hydraulic problem carried on by the laboratory staff. An investigation may be an analytic study, a laboratory experiment, a field testing project, or any combination of these. If application of extablished design procedures and available data fails to resolve a problem in a satistory manner, a laboratory study should be made.

If the prototype structure is in the design stage the laboratory investigation includes only analytical and model studies and there are definite limitations. The theory of fluid mechanics is far from complete and models possess definite scale effects which are not always predictable. So it is essential that an early decision be reached regarding the nature of the basic hydraulic problem. At this stage the problem should be thoroughly examined and discussed by design and laboratory engineers. The laboratory engineer will thus acquire a familiarity with design considerations and construction limitations which will aid in the selection of the appropriate technique and will generally assure the development of a practical solution. The discussion will also serve to familiarize the designer with the potentialities as well as the limitations of a model study of the problem.

Although it is desirable to define the problems involved in any study, it is not always possible to do so in advance of the actual construction of a model. In such cases it is necessary to proceed with the selection of a model scale, and with actual construction, on the basis of experience with similar problems. The designer's request for study may specify an over-all check of hydraulic performance of a design which may be stimulated more by uncertainty with respect to general arrangement than by recognition of specific sources of malfunctioning. Operation of the model will generally disclose, to a trained model operator, the need for design changes.

Various types of hydraulic structures and machinery will be discussed in other sections of this monograph with a view to imparting a background of performance criteria which will be helpful in appraising the hydraulic behavior of a model or prototype.

Before starting construction of a model, account must be taken of limitations imposed by funds, time, and availability of personnel, in relation to the size or type of model. The desired accuracy of final results will materially affect the type or extent of the investigation. For example, an extremely small spillway model may be used to determine the effect of upstream or downstream river flow or erosion tendencies, but establishment of discharge coefficients for gates or an overflow crest requires one many times as large. Limitations of space or pumping facilities are often controlling factors. For example, a large model of a river system can be tested satisfactorily only in the largest laboratories. However, it may be possible to construct and test a section at a time if a satisfactory method of combining the results can be devised.

It is well to remember that, no matter how carefully a model is designed or constructed, it will not contribute an automatic solution. It does provide information which must be interpreted in the light of a knowledge of basic mechanics and hydraulics, as well as experience. Although a model serves to demonstrate fundamental principles that apply equally well to the prototype, it is operated according to a similitude law which is seldom satisfied. Thus, any direct translation of results must be carried out with understanding and restraint.

Prototype Information Required

Information for use in planning a model should include an over-all plan and section of the proposed or existing structure, with sufficient detail to determine the shapes and characters of all surfaces over which the flow will pass. In some cases, topography of the site and surrounding area, results of foundation test boring, and details of any other related structures (particularly the hydraulic features) may be necessary. Knowledge of the type and current state of material composing the riverbed and the canyon walls may be useful. A reliable curve showing water stage with respect to discharge for the river downstream is important. In the case of river or estuary problems, historical data, such as discharge, water stage, tide salinity, cross sections of channels, and soundings are essential. For spillways and outlet works, a complete description of proposed operating conditions and the range of discharges to be expected should be available.

Problems relating to a model study should be discussed with the designers or other responsible persons familiar with the project. Full knowledge of local and general design conditions that may affect hydraulic performance will often save considerable work and time. The importance of becoming thoroughly familar with design requirements early in the study cannot be overemphasized.

Design and Construction of Model

Success or failure in achieving desired results with least work and cost depends largely on the design of the model. The first step is to select such a scale that similarity of the occurrence being studied will be preserved in the model. The model should be made as large as practicable, considering cost and benefits that may be derived. Increasing a model size may desirably enhance its usefulness and improve accuracy of results. At some point, however, depending on the type, the increase in cost and difficulty in operation will offset the advantage in size. Economy would dictate that the model be as small as possible and still yield valid results. But proving that the results from the small model are valid can be troublesome and time-consuming. Current practice is to follow precedent when available and to err on the side of largeness so far as this is permitted by limitations of available space and water supply.

The following scales have been used successfully in model studies in the past and may be useful as a guide. Spillways for large dams have been constructed on scales ranging from 1:30 to 1:100. For medium-size spillways, the model usually should not be smaller than that determined by a scale of 1:60. Outlet works having gates and valves are constructed to scale ratios from 1:5 to 1:30. Canal structures, such as chutes and drops, are usually constructed on scales from 1:3 to 1:20. The range of horizontal scales for river models is usually between 1:100 and 1:1000. The vertical scale for distorted river models is usually between 1:20 and 1:100.

Minimum sizes have been set for certain types of studies. Individual models of valves, gates, or conduits are specified to be equivalent to at least 4 inches in diameter. The scale of an outlet model often is determined from this minimum valve diameter. Models of canal structures should exceed 4 inches in bottom channel width, and spillway models should be scaled so that normal heads over the crest exceed 3 inches. If models are constructed to dimensions smaller than these they are difficult to build, and there is a possibility of introducing similitude defects. Model valves and gates for use in development work or for rating of prototype control devices should be equivalent to at least 6 inches in diameter. Time and expense may also be saved by choosing the scale for tunnel or conduit models to accommodate a standard pipe size or a piece of pipe already on hand. Also, advantageous use can be made of the parts of previously tested models. The model scale, which may affect the entire study, is dependent on many factors and should be carefully chosen.

A model need not be made strictly like the prototype. If the surfaces over which the water flows are reproduced in shape and the roughness of the surfaces is approximately to scale, the model will usually serve its purpose. Models of spillways are commonly constructed of sheet metal soldered to metal-ribbed framing, or they are made of smooth concrete mortar, screeded to a rigid frame of metal ribs. Spillway crests have also been constructed of paraffin, beeswax, or wood. Paraffin and beeswax can be screeded to shape easily and accurately. On the other hand, wood exposed to the water is not recommended for the more permanent portions of models, because of shrinking and swelling. Well oiled wood, however, is satisfactory for piers and and training walls of spillways, as these parts are usually subjected to many revisions.

Models representing tunnels or closed conduits are constructed of sheet metal or sheet plastic. Sheet metal is less expensive and can be used where it is not necessary to observe the flow. Where visibility of flow is desired or where warped surfaces are involved, plastic conduits are much to be preferred.

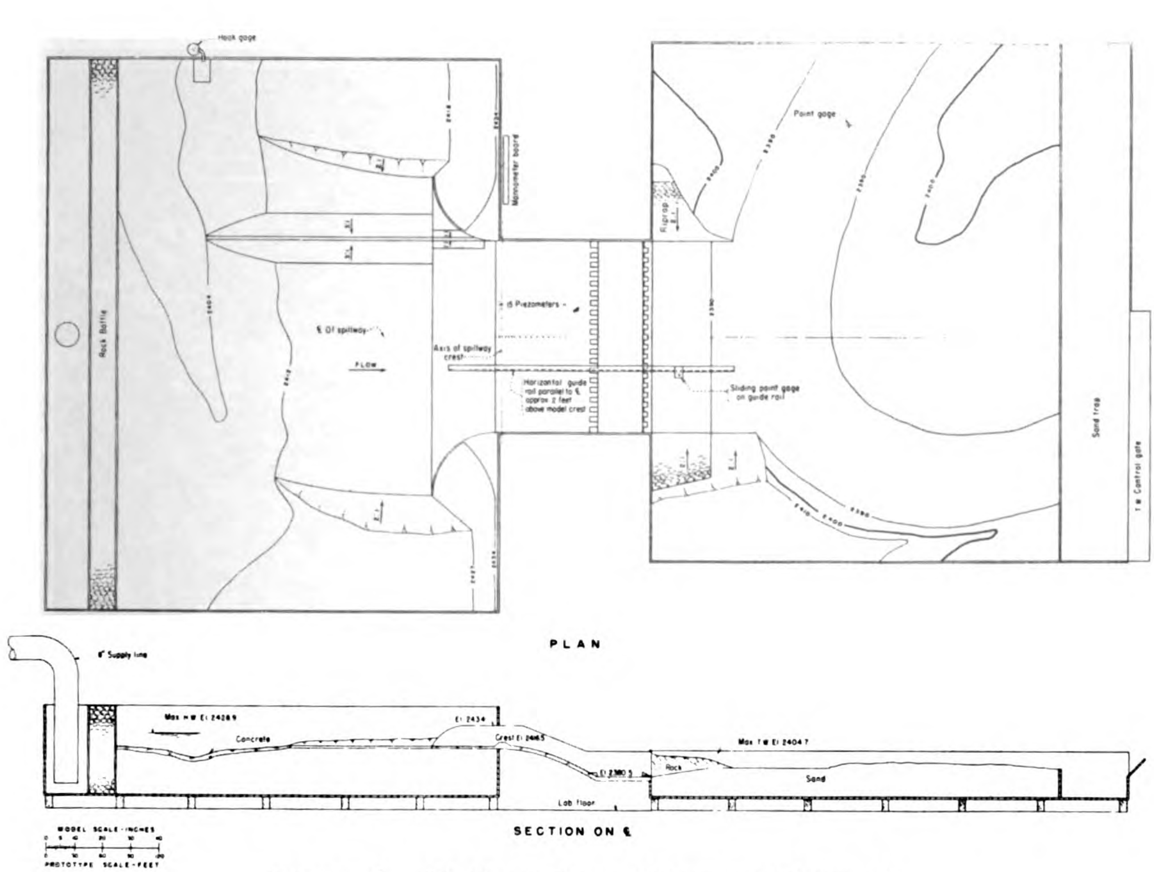


Figure 24. Dickinson Dam spillway--model layout

Valves and gates of the smaller sizes may be fabricated of plastic. However, for development work, where high heads are involved, they are constructed of bronze. Models of open canal structures are constructed of wood, or of wood covered with sheet metal lining. Where wood only is used, it is well oiled and the joints calked with white lead or oakum.

Models of the stable portions of riverbeds and canyons often consist of a cement and sand mortar, plastered over metal lath which is supported on wood framework. Sand or gravel is placed on this mortar to represent the loose or movable river material. Models of river systems may be constructed of concrete, where a fixed bed is employed, or entirely of sand or coal dust where movement of sediment is to be studied.

A head box and a tail box are usually built for each model (Figure 24). The head box serves as the lake, or reservoir, ahead of the structure and insures quiet approach conditions. It may be constructed of wood, lined with sheet metal, or may consist of a box built of brick or masonry blocks. In either case, it must be watertight, so that the discharge through the model can be measured accurately. The head box contains a baffle arrangement, consisting of gravel, a series of screens, or a set of vanes, to still the flow before it enters the model. The box contains the head gage which is used to measure the elevation of the water surface in the reservoir.

The tail box is at the downstream end of the model (Figure 24). It contains a gate, or other device, by which the tail water level can be varied. Sand or gravel is often placed in the tail box to aid in the study of scour patterns, and designs are judged to some extent by their scouring tendencies. The tail box need not be watertight, but time and trouble are often saved by making it that way. For this reason the fabrication of the tail box usually resembles that of the head box. The tail box contains a tail water gage to indicate the water stage in the river downstream from the structure.

Instruments are essential in model testing. The importance of their proper installation and use is strongly emphasized, because comparison of measurements is the deciding factor in many hydraulic designs. Provision should be made for suitable instrumentation while the model is in the de-

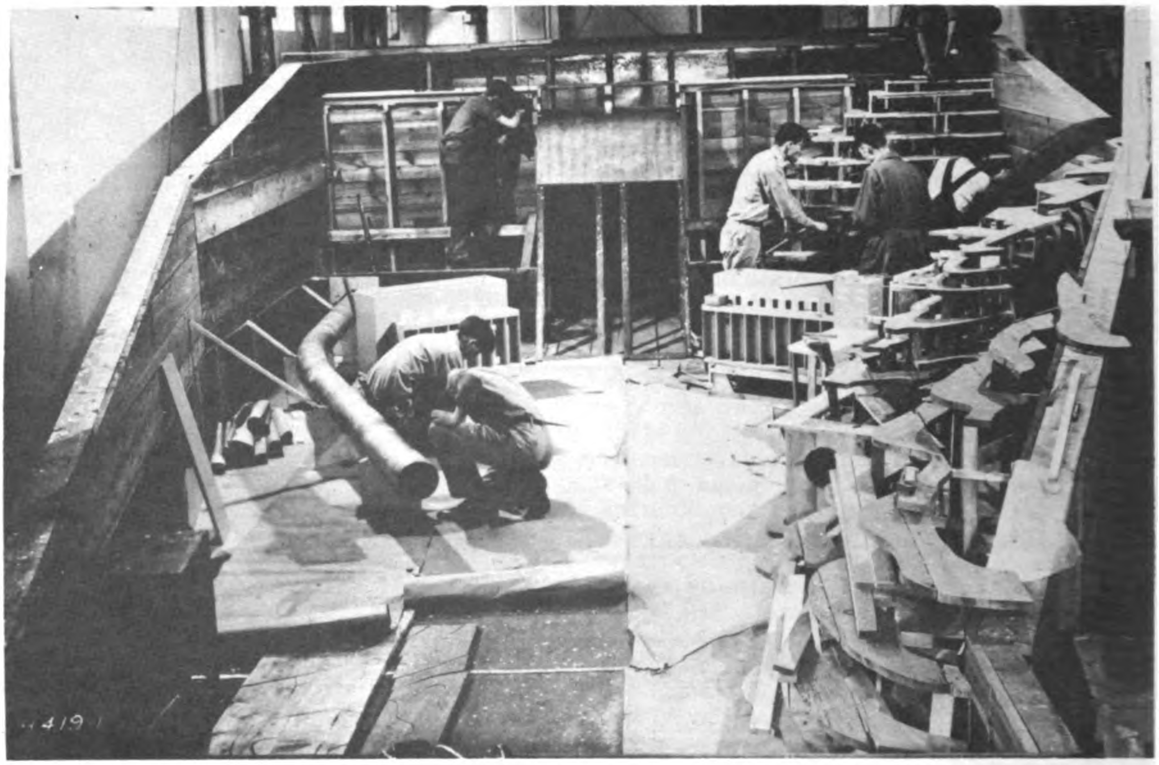


Figure 25. Model of dam spillway under construction

sign stage. Instrument connections and piezometers are relatively easy to install early in the construction, whereas their installation may be very difficult after the model is completed. Sufficient piezometers should be placed in the more critical portions of the model that measurements from them will completely define the local action. Piezometers are relatively cheap; they should be provided in generous numbers to offset any oversight in defining the critical points.

Accurate and clear model drawings prevent time-consuming errors in construction. Drawings should contain sufficient information to enable the model makers to build a structure which conforms to the designer's specifications (Figure 24). This figure shows an over-all model drawing of an earth dam spillway. Other drawings are necessary for details. Where model makers are experienced in hydraulic model construction, many drawing details may be omitted, and much of the method of construction may be left to the shop foreman. Important parts which will affect the performance of the structure, or special features that are new in concept, should be detailed completely. Working drawings should include all information necessary for installation of instruments, such as piezometers, pressure cells, velocity measuring devices, etc., and thereby obviate the necessity for extensive changes after the model is completed. Figure 25 shows an overfall spillway model under construction. Since it is necessary to obtain greater accuracy in building a model than that to which most craftsmen are accustomed, care should be taken to see that the required tolerances are met, particularly in the critical parts. Greatest accuracy should be maintained where there will be rapid changes in direction of flow and where high velocities will prevail. The stage of the testing program will affect the type and accuracy of construction. In early tests, where many schemes are tried to determine their over-all feasibility, the construction need not be as carefully finished as in the final stages of testing where the data required are to be used for operating the prototype structure.

There are many short cuts and tricks of the trade to be learned in the construction and testing of models. These can best be learned by experience. For example, the construction of the model should be made sufficiently flexible to allow considerable modification with a minimum of rebuilding. A model with a spillway stilling basin set on or close to the laboratory floor will require extensive rebuilding if the basin is to be lowered. Provision in the original plan to keep the model sufficiently high to permit the basin to be lowered without major rebuilding will make this revision comparable to a routine adjustment.

Operation of Model

The operating program should be carefully planned to evaluate the worth of the design under study. In general, the evaluation consists of proving, by qualitative and quantitative tests, that the design meets the operation requirements. For an undistorted model, the operating program can be divided into two phases; adjusting, and testing. In the case of a distorted, movable-bed model, a third phase, verification, must also be considered. Adjustments in the model consist of preliminary trials to reveal defects and inadequacies. This phase should not be hurried; it is important and economical, in the long run, to spend the time required to make certain that the model behaves as intended and that the instrumentation is satisfactory. Often, minor alterations in design are suggested by these tests, such as partial redesign or revision or the shifting of some measuring instrument. It is during the adjustment phase that the experimenter becomes acquainted with the peculiarities of the model and becomes adept in its operation.

When the model is of the movable-bed type, the adjustment phase involves verification tests in which past recorded action of the prototype must be duplicated in the model, otherwise, reliable quantitative results cannot be obtained. Because such tests are often the only basis for similitude, verification is an extremely important part of the operation of movable-bed models.

Testing should consist of systematically examining each proposed design with regard to possibilities for improvement, reduction in cost of construction, and reduction in maintenance costs for the prototype. The experimenter must exercise patience, imagination, and ingenuity and be capable of interpreting the model results correctly. He should work in close cooperation with the designing engineers, since they should be continually informed as to progress and consulted concerning future testing. Data should be analyzed concurrently with the testing, to prevent accumulation of unnecessary data. When possible, functional relationships among the different variables should be applied to the data to aid in detection of erroneous measurements.

In addition to the regular testing, the experimenter should be alert to obtain general information which will be of value eventually (for compilation), even though it is not of interest to the designers at the time. Results of experiments on many models are usually required in assembling general design information, and it is a waste of time and money to construct models and not obtain all the useful information. For example, the following general information is desired on spillway and outlet works design: (1) calibration curves for all spillway crests; (2) complete calibration curves for all types of gates; (3) water surface profiles through gate sections of spillways and other open channel structures; (4) information on the spreading of jets in flat chutes; (5) stilling pool actions for various types of sloping aprons and baffle pier arrangements; and (6) effects of negative pressures on the stability and discharge of overfall dams. Some of these data are obtained in routine testing, but constant vigilance will be necessary to assure that the remainder of the information is obtained where possible. Such information can be very valuable in connection with verification studies by actual field measurements.

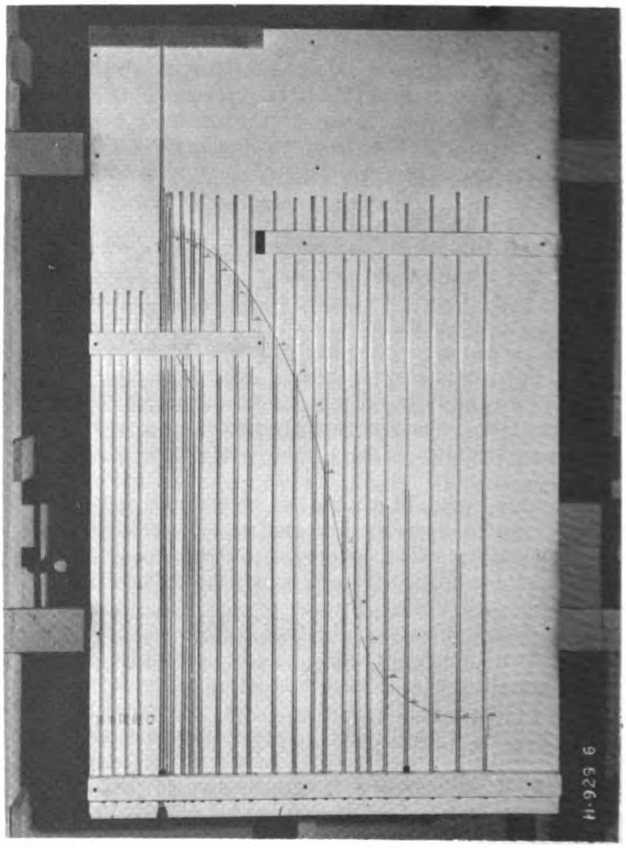
Since the end product of any model study is the report which transmits the findings and recommendations, the experimenter must maintain a complete and accurate set of notes, including a diary. Dates may be of special significance in the future. Negative as well as positive results should be recorded. A complete photographic record of all important tests is indispensable; it often eliminates the necessity of repeating tests. The importance of presenting a clear, concise, well organized, and well illustrated account of the experimentation can best be appreciated by attempting to assimilate texts of a few existing reports on subjects unfamiliar to the reader.

OPEN CHANNEL FLOW

General Considerations

The particular and detailed aspects of hydraulic laboratory practice as applied to one of the broad types of models, open channel models, will be discussed in this chapter. The design of each type of open channel structure will be discussed to the extent necessary to impart a background against which the experimenter may judge a partic-

A. Overfall spillway



B. Pressures on overfall

Figure 26. Anchor Dam spillway model, scale 1:30

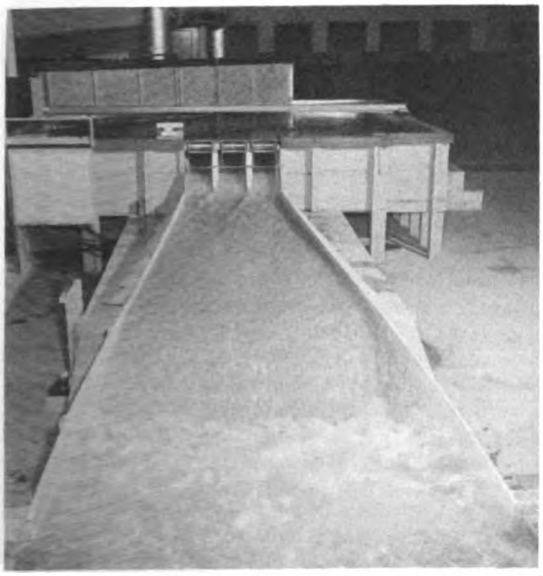


Figure 27. Model spillway of Trenton Dam, scale 1:54

ular design under study. It has been demonstrated repeatedly that a knowledge of the designer's viewpoint and a familiarity with the assumptions he must make in executing a hydraulic design are essential in working out practical solutions with hydraulic models.

Overfall Spillways

Spillways must be constructed to various profiles, but the most common is the overfall having a curved crest designed for a particular head, such as that shown in Figure 26A. Another type is the flat open chute spillway often used on earth dams, on which a small overflow crest is used at the gate section, as in Figure 27. The spillway crest may be surmounted with gates for regulation of the flow or there may be free overflow without controls.

Spillway models are built geometrically similar to their prototypes, and, because gravity forces predominate, dynamic similitude, according to the Froude law, is closely approximated. Viscous effects and surface tension are usually neglected. It has been found that model spillway gates should not be less than about 6 inches in width, if the model is to be used for crest studies or determination of discharge coefficients. If this dimension cannot be attained, when several gates are involved, a larger model of one or two gates may be necessary. Head on the crest of a spillway model should not be less than 0.1 of a foot; a greater head is desirable.

1. Crest coefficients and pressures.--Model tests are usually required to aid in the design of overflow structures and to insure adequate and safe operation of the prototypes. Of great importance is the determination of the discharge capacity, which must be made with high precision, since a small error in measurement in the model may be equivalent to a considerable quantity of water in the prototype because of large discharge scale ratio. If the crest is to be submerged during spillway operation, the effect of submergence on the discharge coefficients should be investigated.[16] (When the elevation of water surface in the channel below a spillway is at or above the elevation of the crest, the crest is said to be submerged.) Even in cases where the crest is not submerged as judged by visual inspection, if the head is approximately equal to the vertical distance from crest to apron, the effect on the discharge coefficient should be determined. With the discharge coefficients determined, using the actual crest length measured in the model for the expected range of operating conditions, the proper length of spillway crest may be calculated. The crest length as finally chosen should be tested to make sure that the reduction in capacity caused by unpredictable end contractions, unusual approach conditions, or other factors, is not excessive, and to provide accurate rating curves for operating the prototype structure.

Before making final conclusions as to the adequacy of the crest length, the profile of the spillway should be tested for the existence of subatmospheric pressures. For most spillways, it is desirable to have positive pressures on the spillway face for all but the maximum floods, and these should be only slightly subatmospheric if the structure is to operate frequently. In some cases where operation is infrequent, it is desirable to design the crest deliberately for subatmospheric pressure to increase the discharge coefficient and in this way economize on the cost of the structure. An example of a high coefficient crest is Anchor Dam, Figure 26A. The overfall shape was designed for a vacuum of 5 percent of maximum head on the crest. In any case, the subatmospheric pressure on a spillway face should not be lower than -15 feet of water on the prototype. A more conservative figure is -10 feet if the measurements are made with

16 Department of the Interior, Bureau of Reclamation, "Studies of Crests for Overfall Dams," Bureau of Reclamation, Denver Federal Center, Denver, Colorado, 1948, Boulder Canyon Project Final Reports, Part VI--Hydraulic Investigations, Bulletin 3.

a water manometer which has a damping effect on pressure fluctuations. Figure 26B shows the manometer board which was used to measure pressures on Anchor Dam. Very little is known about the action of prototype spillways if the pressures are below -10 feet of water. It is believed, however, that there is a tendency for the partial vacuum to be relieved at intervals by air breaking through the nappe surface or between the nappe and a side boundary. This relief would most certainly be periodic and would subject parts of the structure to alternating pressures, a condition considered undesirable. It is especially undesirable, and may be dangerous, if the changes in pressure are transmitted to gates, valves, or other mechanical equipment. During model operation, the appearance of the flow will often indicate difficulties not apparent in the data obtained from the discharge and pressure measurements. The water surface will indicate to some extent the correctness of the design. Poor approach conditions will result in surface disturbances which may be evident over the entire spillway face.

2. Flow characteristics.--Excessive end contractions, improper pier shapes, and very sharp radii in flow boundaries will result in rough water surfaces. For proper performance, the water surface in the approach and on the spillway face should be as smooth as is consistent with cost. The spillway for Trenton Dam on the Republican River in Nebraska (Figure 27) is a good example. The water surface is consistently smooth and the spreading action in the chute and pool is excellent. In any case, the configuration of the water surface should be reasonably uniform to avoid interference with the operation of the stilling basin or other energy dissipating device. Interpretation must include an allowance for the fact that models, especially small ones, do not fully indicate the magnitude of the surface disturbances to be expected in the prototype. The higher relative viscosity and surface tension in the model means that air entrainment is not to scale. A small surface disturbance, which in the model appears to be attached to the main sheet of water, may in the prototype become detached and result in excessive spray formation.

Profiles of the water surface for limiting conditions should be determined to aid in locating the gate trunnions and to be certain that the flow will clear gate counterweights or parts of the structure not reproduced in the hydraulic model.

3. Gate operation.--When the spillway shape and other details have been determined the gates may be calibrated for discharge through partial openings. Curves of reservoir elevation versus discharge are usually determined for each of the representative or probable gate openings, giving a family of curves, Figure 28. Discharges are more consistent if calculated from coefficients taken from smoothed coefficient versus head curves. In most studies of spillway performance, all spillway gates are assumed to be open an equal amount. In the model tests, consideration should be given to the apron performance during the interval of opening the gates to uniform opening, since all gates will not be opened simultaneously and the rate of rise of the gates must be synchronized with the rate of rise of the tail water. Too sudden opening of the gates usually results in improper and dangerous apron performance. To prevent costly mistakes in prototype operations, a gate-operating schedule should be made during the model tests. The proper order of opening the gates should be carefully determined to prevent unsymmetrical flow and resulting eddies. If any special precautions are necessary, these should be noted both for the opening and closing procedures. The sequence of opening the gates and the limiting increment should be studied on the model. The maximum allowable increment differential should be determined by the action of the energy dissipator at the base of the spillway. Too large an increment will result in either excessive riverbed erosion or excessive apron cost.

If the spillway under test has flow over the gates, as in the case of drum or leaf gates, the problem of ventilation should be investigated, since the action of the flowing water tends to evacuate air from the space beneath the nappe. If a continuous supply of air is prevented from entering, because of the nappe sealing against the spillway piers, a vent should be provided to prevent lowering of the pressure which can cause instability of flow. The pressure reduction is, in effect, an added load on the gate. Any fluctuation of pressure can result in vibration or oscillation which, under unfavorable conditions, might result in failure of the gate. In addition to the factors discussed, there are usually other special problems connected with each particular structure. Determination of the size and type of gate may be of special interest. Pressures on gate surfaces, unbalanced pressure on spillway piers or training walls, proper curvature of training walls, and special hoods or deflectors on the spillway face are all items that have been investigated.

Tunnel Spillways

There are two predominant types of tun-

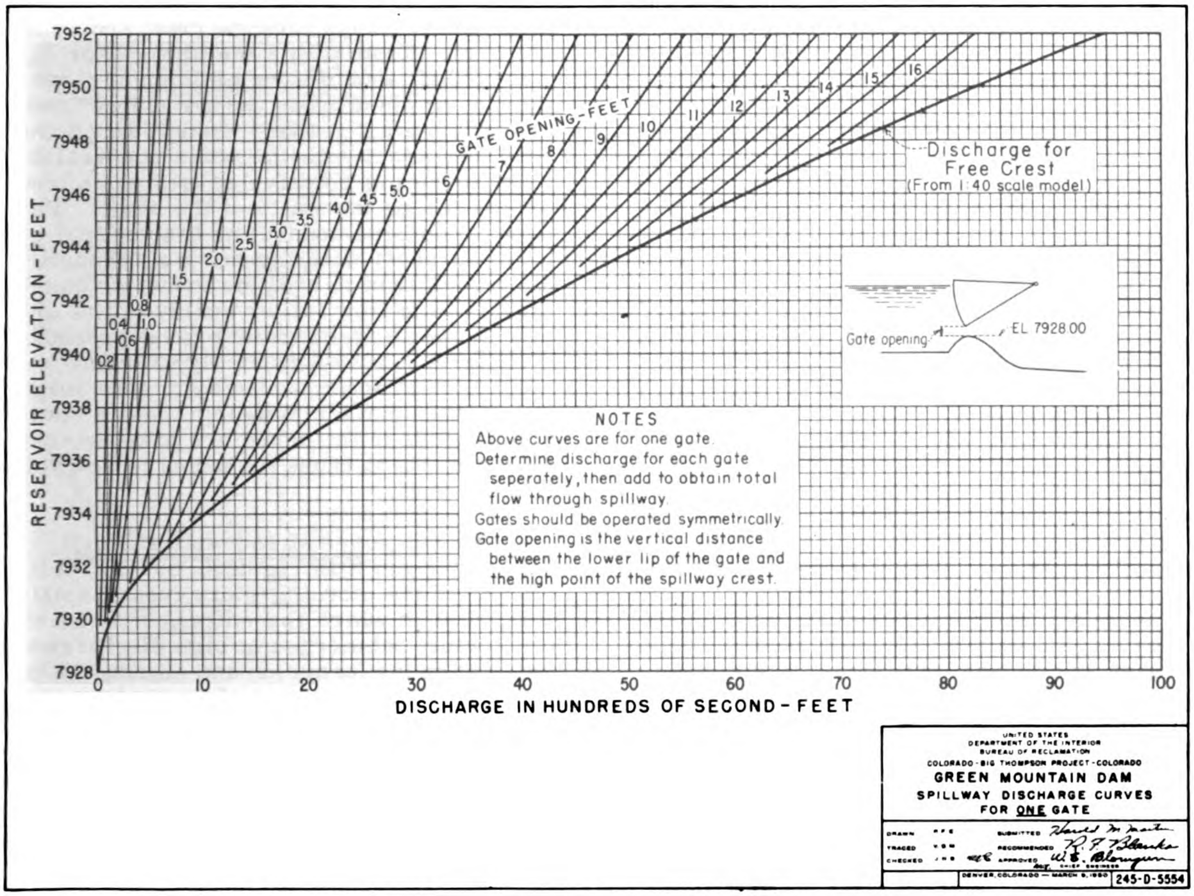
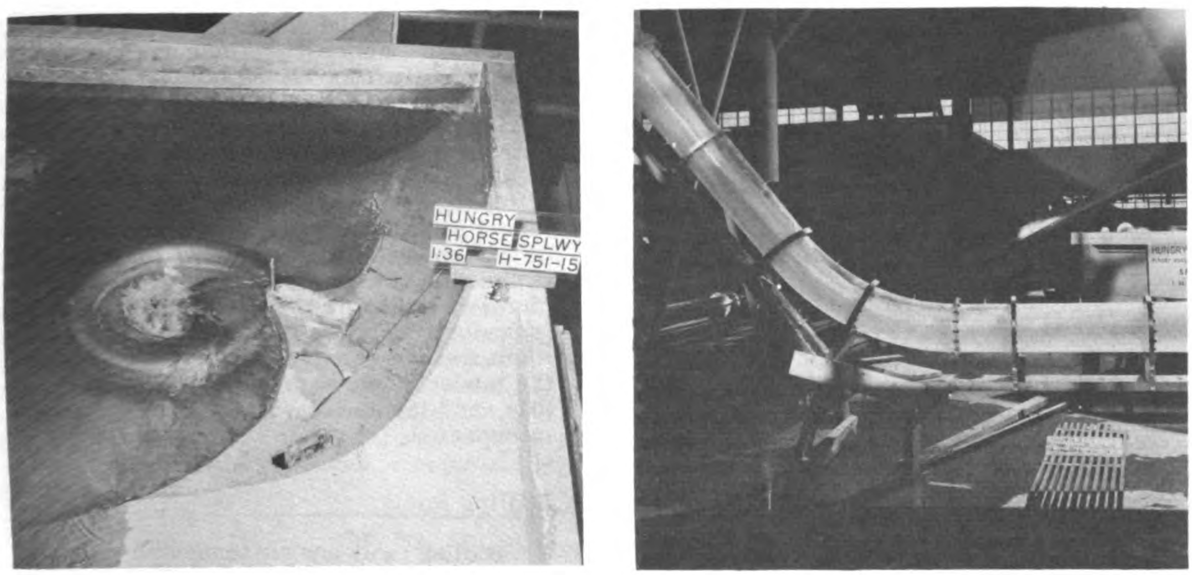


Figure 28. **Green Mountain Dam spillway discharge curves for one gate**



A. Morning glory spillway with ring gate **B. Inclined and horizontal tunnel**

Figure 29. **Model spillway for Hungry Horse Dam scale 1:36**

nel spillways, those having inclined shafts and those having vertical shafts. The inclined tunnel spillway can be observed in several forms. The upstream portion or collecting structure at the entrance to the inclined tunnel may consist of a side channel, a semicircular overflow weir, a morning-glory, or plain gate structure. These collecting structures may or may not be surmounted with gates. In all cases a transition is required between the collecting structure and the tunnel. The tunnel lay-out (Figure 29B) is quite similar in all designs. It consists of a shaft, inclined at approximately 45°, which is joined to a horizontal tunnel by a vertical curve. The horizontal tunnel usually is a portion of the former diversion tunnel. Structures having an inclined shaft usually operate with open channel flow, as this simplifies aeration, flow, and pressure problems. The vertical-shaft spillway of the morning-glory type derives its name from the entrance shape, which resembles a morning-glory. A circular weir having a conical transition to the shaft diameter is the usual entrance. The morning-glory may have a free crest or be surmounted by a ring gate or radial gates for regulation of the flow. Often piers are placed on a free crest to aid in stabilizing the flow. The shaft may be circular and of constant diameter, or it may be tapered to a smaller diameter as the depth increases. These spillway crests are designed to operate with free flow, submerged flow (vertical shaft flowing partially full), or both.

1. Pressure and flow conditions.--Pressure conditions within the spillway entrance shaft and discharge tunnel are of major concern, especially on morning-glory spillways, since severe subatmospheric pressures at any location will in most cases produce considerable vibration and noise and are likely to be the cause of cavitation action and destructive pitting of the spillway surfaces. Piezometer openings in the model should be located with care and in generous number, especially on the morning-glory spillway structures. The spillway for the Hungry Horse Dam, Figure 29, required extensive studies as to pressures on the ring gate, pier, and in the throat of the vertical portion of the shaft for all gate positions. As subatmospheric pressures were experienced with the venting arrangement of the preliminary design, the model served to determine the size and location of auxiliary air vents.

Smooth flow throughout the structure, whether it operates as a closed conduit or an open channel, is important. The approaching flow should be as near normal to the crest of the collecting structure as possible and the depth over the crest should be uniform. Flow within the vertical or inclined shaft should have a minimum of turbulence or disturbance, as turbulence tends to entrain air. A tapered shaft keeps the flow in contact with the walls and assists in maintaining positive or near-positive pressures on the walls of the shaft. The flow in the bend connecting the vertical or inclined shaft with the horizontal tunnel should not tend to seal the tunnel or cause the tunnel to flow full or nearly full at any point. Such a condition may cause pressure conditions to vary sufficiently to create unstable action accompanied by intense surging, noise, and air pumping. Sharp horizontal bends in the tunnel are to be avoided as these tend to cause the tunnel to flow full intermittently.

2. Submergence at entrance and exit.--One of the most intriguing, but difficult, studies of the morning-glory type of spillway, that in which the entrance becomes completely submerged at high discharges, pertains to the formation and elimination of vortices which invariably form and persist within the entrance of the structure. The vortex is undesirable in that it causes a reduction in capacity for a given head and induces air entrainment, turbulence, and vibration. Many devices have been placed at various locations in the structure to prevent the vortices from forming, but none have been completely successful. Piers placed on the crest, a curtain wall within the entrance, and control of approach conditions are methods commonly used.

The exit of the discharge tunnel should not operate submerged because unstable flow conditions involving fluctuating pressures might be induced. Objectionable vibration would no doubt be present and the entrained air compressed by submergence at the tunnel exit might be discharged intermittently with explosivelike action from the tunnel portal. A study of this action would be extremely difficult on a small model, so, if possible, the exit should be placed sufficiently high to prevent a hydraulic jump from forming in the tunnel for all but minimum discharges. If this cannot be avoided, the model should be made as large as feasible, and the action of the model carefully interpreted.

Stilling Pools

Stilling pools are constructed in conjunction with spillways, outlet works, or other devices that discharge water at high energy. The term stilling pool is often used in the broad sense to include all devices used to protect the main structure from damage by

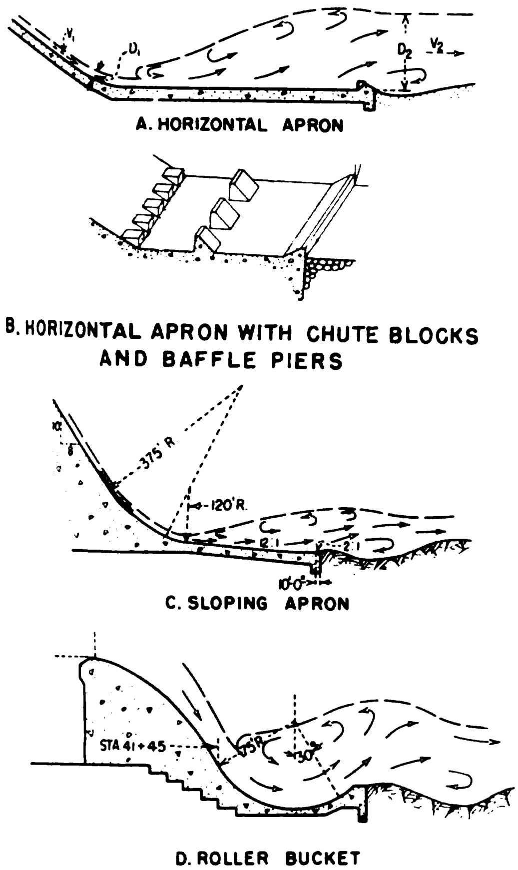


Figure 30. Stilling pools for spillways and outlet works

high-velocity water. Although all stilling pools are energy dissipators, some are principally energy distributors which expand the high-velocity jets. There are several types of stilling pools, and each type is suitable under a definite set of conditions. It is important that laboratory personnel understand the various types to enable fair judgment of the performance of any one.

1. Types of stilling pools.--The most common type of stilling pool is the true hydraulic jump basin with horizontal apron, Figure 30A. The energy is both dissipated and distributed before the water is released into the unprotected channel downstream. The stilling pool with horizontal apron is most suitable when the D_2 curve, computed from the formula

$$D_2 = -\frac{D_1}{2} + \sqrt{\frac{D_1^2}{4} + \frac{2V_1^2 D_1}{g}},$$

coincides with, or closely approximates, the tail water curve for the river for all discharges (Curve A, Figure 31). Identifica-

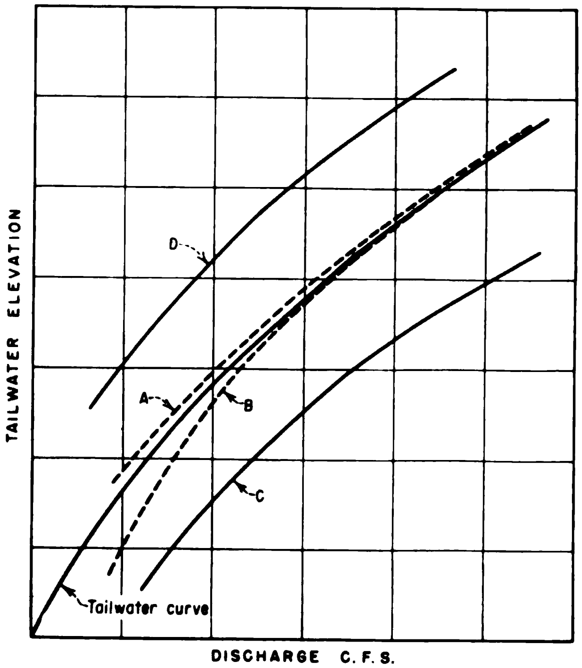


Figure 31. Relation of D_2 curves to tailwater curve

tion of the symbols may be made by referring to Figure 30A. The near coincidence of the D_2 curve with the tail water curve means that the tail water will be correct for a good hydraulic jump at all discharges. The length of apron necessary to develop the jump fully is from $4D_2$ to $6D_2$, depending on the Froude number of the incoming flow. The length can be reduced to 2 to $3D_2$ by the addition of chute blocks and baffle piers to the apron as shown in Figure 30B. The addition of the stilling basin appurtenances also provides a considerable factor of safety against jump sweepout if the tail water elevation should be lowered by future degradation of the river bed downstream from the dam. Use of this type of pool is limited, however, to installations where the incoming velocity is no greater than 50 to 60 feet per second.

Should the computed D_2 curve differ from the tail water curve of the river as in B, Figure 31, it is necessary to investigate the proposed stilling basin for the partial discharges for which a deficiency in tail water exists. It may be necessary to make the basin deeper for the less-than-maximum conditions. The performance of a particular basin may be predicted with a good degree of accuracy by determining the Froude number of the incoming flow and selecting from Figure 32 the type of jump action that will occur.

The design of hydraulic jump stilling pools requires more discussion than can be

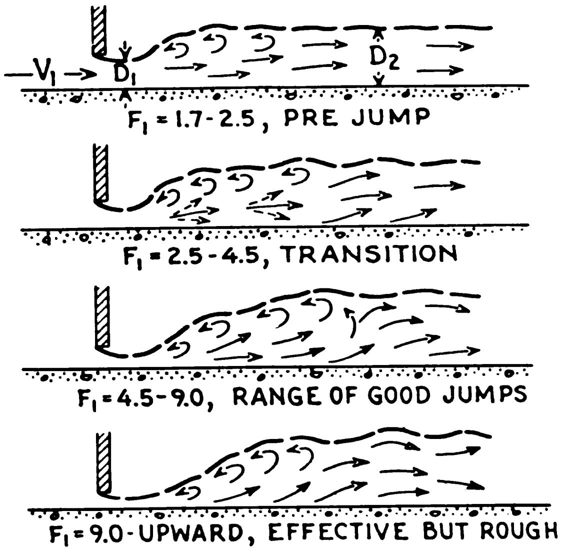
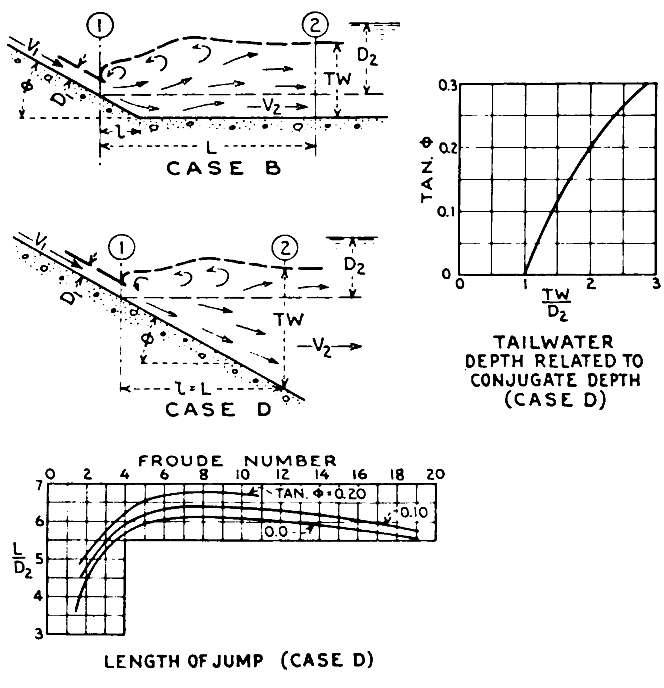


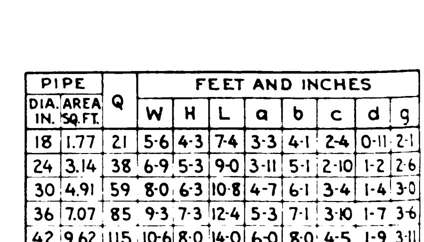
Figure 32. Jump forms.

given here. It is suggested that Engineering Monograph No. 25, Hydraulic Design of Stilling Basins and Energy Dissipators, be consulted for the solution of specific design problems. Monograph No. 25 contains a comprehensive analysis of stilling basin problems and contains information and design rules for four different types of hydraulic jump stilling basins utilizing a horizontal apron. Sample problems are also included.

Figure 30C shows a sloping apron and Figure 33A shows some of the design charts from Engineering Monograph No. 25 needed to determine the dimensions of the basin. As indicated in the charts, a sloping apron requires greater depth and length to contain the jump action than does a horizontal apron. Steeper slopes require greater depths and lengths than flatter slopes. Contrary to some published information, it has been found that a sloping apron does not necessarily provide improved jump action or stability over that occurring on a flat apron. A sloping apron will perform on a par with a horizontal apron, regardless of the apron slope, if the stilling basin proportions are correct. Therefore, the choice between a horizontal or a sloping apron should be made solely on the basis of cost. For example, an 8-to-1 apron slope on the Folsom Dam made possible a saving of approximately one million dollars over the horizontal type, as not only was excavation saved, but also it was possible to raise the toe of the dam, effecting a saving in concrete as well. The many factors affecting stilling basin design for sloping aprons are discussed in Engineering Monograph No. 25.



A. Jump characteristics on sloping apron.



PIPE		Q	FEET AND INCHES							
DIA. IN.	AREA SQ.FT.		W	H	L	a	b	c	d	g
18	1.77	21	5-6	4-3	7-4	3-3	4-1	2-4	0-11	2-1
24	3.14	38	6-9	5-3	9-0	3-11	5-1	2-10	1-2	2-6
30	4.91	59	8-0	6-3	10-8	4-7	6-1	3-4	1-4	3-0
36	7.07	85	9-3	7-3	12-4	5-3	7-1	3-10	1-7	3-6
42	9.62	115	10-6	8-0	14-0	6-0	8-0	4-5	1-9	3-11
48	12.57	151	11-9	9-0	15-8	6-9	8-11	4-11	2-0	4-5
54	15.90	191	13-0	9-9	17-4	7-4	10-0	5-5	2-2	4-11
60	19.63	236	14-3	10-9	19-0	8-0	11-0	5-11	2-5	5-4
72	28.27	339	16-6	12-3	22-0	9-3	12-9	6-11	2-9	6-2

B. Impact type stilling basin.

Figure 33. Typical stilling basins.

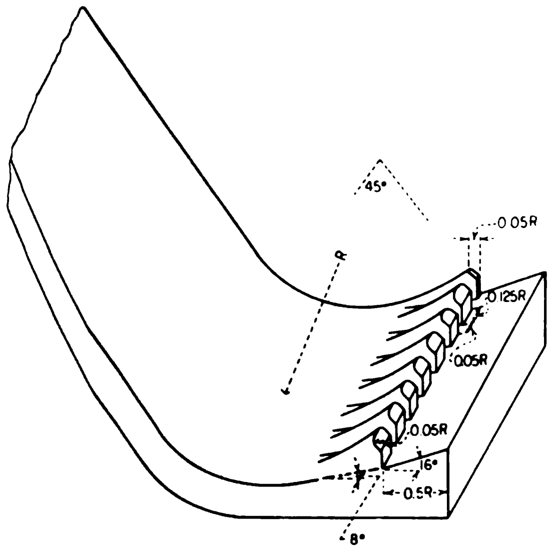


Figure 34. Slotted bucket

Should the computed D_2 curve fall definitely below the tail water curve, as in C, Figure 31, the stilling pools described above are not applicable. Where the tail water depth is excessive, one of three bucket types to be discussed may be feasible. The roller bucket, Figure 30D, is effective for tail water depths which are great compared to those of the conventional apron type of pool. This type of bucket was used on the Grand Coulee Dam spillway and performs best when deeply submerged. The cost of the roller bucket is usually favorable, but it has the disadvantage that a very violent ground roller develops downstream which can deposit large boulders in the bucket if the flow entering the bucket is unsymmetrical. The scouring action of these boulders is responsible for the surface damage to the Grand Coulee Dam bucket. Symmetrical flow, on the other hand, will prevent this abrasive material from entering the bucket. A model study is advisable when this type of stilling device is under consideration.

The slotted bucket, Figure 34, is a modification of the roller bucket which differs from the solid type because of the slots and the short apron downstream. The hydraulic action, however, is entirely different in that the rough surface conditions and the violent ground roller experienced in the solid bucket are eliminated. Water flowing through the slots produces a spreading of the jet rather than a concentration, and at the same time, counteracts a large portion of the former violent ground roller. The slotted bucket operates well over a wide range of tail water depths but requires a tail water depth somewhat greater than that required for a conventional horizontal stilling pool. A short apron downstream from the slotted bucket, sloping upward at an angle of 16° with the horizontal, serves to spread the individual jets flowing through the slots before they leave the apron, thereby reducing the violence of the ground roller. The usual dimensions of the bucket itself are given in terms of the bucket radius in Figure 34. Information for determining the radius of the bucket, and the vertical placement of the invert between the maximum and minimum tail water elevations is given in Engineering Monograph No. 25.

A third type of device, which is sometimes called a flip bucket, may be used for practically any tail water condition, Figure 35. This bucket may be near the tail water surface, or it may be far above it. In either case, the jet discharges into the atmosphere as it leaves the bucket. The flip bucket produces very rough water surfaces in the river downstream and is conducive to the formation of spray, characteristics which are not desirable where powerhouses are in close proximity to the spillway. A model study is always advisable in the design of this type of bucket.

If the tail water curve is unknown, as sometimes occurs in small drainage areas, an impact type stilling basin may be used, as shown in Figure 33B. Standard basins for discharges of from 21 to 339 cfs have been developed from model tests and prototype performance has been found to be satisfactory. Larger discharges may be handled by using multiple units side by side. A table of basin dimensions and the design rules are given in Engineering Monograph No. 25.

2. Visual appraisal of action.--In all types of stilling pools, accessories consisting of chute blocks, baffle piers, end sills, or deflectors may be used to aid in obtaining the desired results at least cost. However, the designer should be sure that the safety of the structure does not depend primarily on relatively small accessories that may be damaged or lost as a result of continued operation. The purpose of a stilling pool is to protect the main structure and the river immediately downstream. The basin itself should not be considered expendable, and effort should be made to insure it against damage during operation. This will be accomplished, in large measure, if the stilling device in operation provides a flat water surface with no significant waves and with no local scouring velocities. Uniform velocity distribution at the downstream end of the basin with a minimum of channel erosion is also essential to good operation. The relative effectiveness of different model basins may be determined by comparing meas-

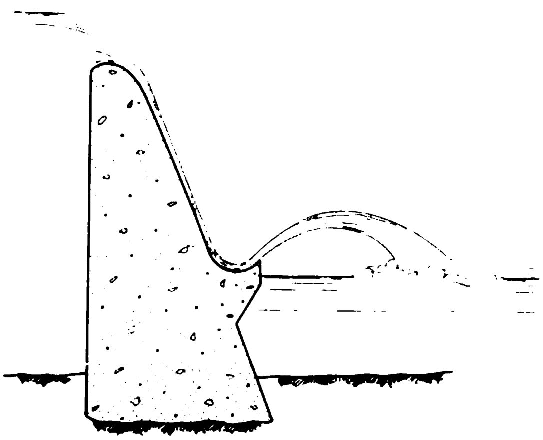


Figure 35. Flip bucket

ured velocities, wave heights, pressures, and erosion of the downstream riverbed. Observation of the general appearance of the operation, preferably recorded as a photograph, will facilitate evaluation of the effectiveness of a design. In making studies of a series of stilling pool designs, the procedure to be followed will vary from one study to another. In general, however, by following a predetermined plan of testing, much unnecessary work can be avoided and a maximum of information obtained.

Evaluation of various designs by general appearance of hydraulic action is especially advantageous in the study of preliminary or experimental designs. An experienced operator can usually predict, by watching the performance for a short time, whether a proposed design shows promise or whether the scheme should be abandoned. If it appears that with minor changes the design can be made workable, other tests may be made to corroborate the operator's judgment. However, in many cases there is no need for any but a visual appraisal of the performance of a particular design. The presence of extreme turbulence, waves, high velocities, high-velocity eddies, or pulsating flow beyond the limits of the structure, indicates need for further refinement of design or complete abandonment of the scheme. If preliminary observations indicate that further testing is required, the basin should be subjected to tests covering the complete range of discharges and headwater elevations. For each discharge and headwater condition, the permissible range of tail water elevations should be determined. Pools that do not have a fairly wide range of permissible tail water are not ordinarily practical because unexpected lowering of the tail water may result from degradation of the stream bed or reduction in power plant discharge. Unless the basin meets the requirements for discharge and the expected range of tail water, further tests need not be made until suitable revisions have been completed. If the pool operation is satisfactory over the desired ranges of discharge, headwater, and tail water, the effect of the operation on the downstream channel may be determined.

3. Erosion studies.--Erosion studies are made to judge the effect of a particular stilling pool design on the downstream channel and to compare the relative effectiveness of various stilling pool designs. They may be made to give either qualitative or semiquantitative results. For preliminary tests, the qualitative method is usually sufficient; but before recommendations are made for a final design, the quantitative tests are often advisable.

In making qualitative erosion studies, a readily erodible sand is used to represent the channel below the stilling pool. The measured depths and extent of the erosion will indicate the relative effectiveness of the pool under test when compared with the erosion patterns for other pools, if the same movable bed material is used. Erosion patterns thus produced may or may not represent the depths of erosion to be expected in the prototype, depending on the choice of the model bed material. Regardless of the choice, however, the model will indicate erosion tendencies and patterns, if it is operated for a period sufficient to produce a stable bed, and, if the prototype operates until a stable erosion pattern is formed, the model and prototype erosion will be similar.

To obtain a better conception of the depth and extend of the prototype erosion to be expected, an estimate of the ability of the prototype channel to withstand erosion can be made based on the particle size or bedding characteristics of the prototype material. Accurate prediction of the velocity at which the material will begin to erode is difficult, but engineers familiar with prototype operation will concur to a surprising degree on the critical eroding velocities. Using the agreed-upon value of eroding velocity as a guide, mixtures of sand and cement are prepared, cured, and tested in a special apparatus. Material similar to the sample which just begins to erode when the scale value of the critical eroding velocity is reached is used in the model riverbed. The testing apparatus consists of a small flume with a rectangular depression in the bottom. Water is supplied to the flume under a sluice gate which may be raised or lowered to vary the depth of water. The velocity of the water, passing over the sample which fits snugly into the depression and is flush with the flume bottom, may be varied by changing the head on the sluice gate. By starting at low velocities and gradually increasing to higher velocities, the critical value, when erosion begins, may be determined. Occasionally an undisturbed sample of the prototype bed is available and the critical erosion velocity may be determined in the same equipment.

When a model test is made using this mixture in the movable bed, the erosion pattern produced represents closely the prototype erosion if the initial velocity estimate was correct. To determine the effect of an error in estimating the critical erosion velocity, other tests similar to the one just described should be made for velocities both double and half the first estimated value. If the patterns produced by the three tests are acceptable, no further tests need be made. If, on the other hand, one or more of the tests indicate excessive erosion, changes in the stilling basin or other corrective measures may be necessary.

4. Measurement of pressure, velocity and turbulence. --Measurements of pressure are made on the various parts of a stilling pool for two distinct reasons: (1) to determine whether pressures conducive to cavitation will occur on baffle piers, sills, or other parts of the structure exposed to high-velocity flow and (2) to aid the designer in determining the loads to be expected on dividing or retaining walls or other parts of the structure subjected to unbalanced water loads.

In problems where the stilling pool size must be kept to a minimum, baffle piers in particular should be investigated to be certain that subatmospheric pressures are not so severe that cavitation will occur in the prototype. If spreader piers are used in the basin, these should also be investigated. End sills are usually not subjected to as high a velocity as parts of the basin upstream, but if any doubt exists, pressures on the sill should also be investigated. In every case where a curvature in the floor is used to train or spread the flow entering the basin, the pressure distribution should be determined to insure against excessive negative or unsteady pressures. Pressures on walls or other parts of the stilling pool are often useful to the designer in determining unbalanced loads on the structure. These unbalanced loads are greatest when the basin is operated unsymmetrically. The operator of the model is usually responsible for anticipating unusual operating conditions in the prototype and making measurements needed by the designer. It may be necessary to alter the usual type of model construction to obtain satisfactory piezometer installations. For example, where wood blocks will suffice for the ordinary baffle piers, one or more metal piers containing the piezometers may be required for accurate piezometer installation. Because of the care and time required in installing the piezometers, it may be advisable to delay taking pressure measurements in a basin until a fairly workable scheme has been developed. On the other hand, measurements should not be delayed so that time is wasted developing a design that cannot be used because of poor pressure conditions.

Other observations which should be made in the course of a series of stilling pool tests include measurement of eddy velocities or local velocity concentrations, water surface profiles along training walls or other parts of the structure, and measurement of wave and surge heights in the downstream channel. Wave and surge heights are particularly important when the discharge is passed directly from the basin into an unlined canal because they have destructive effects on the sloping banks. Readings on staff gages are usually sufficient to determine wave and surge heights but high-speed motion pictures have shown that direct readings give less height than actually occurs, particularly when the wave period is short. Where model-prototype comparisons are to be made or wave heights are of primary importance, the heights and periods should be obtained by motion pictures, oscillograph records, or other satisfactorily precise methods..

Outlet works

An outlet works usually consists of single, double, or multiple tunnel outlets controlled by valves or gates designed for regulation of flow at partial openings, and a stilling pool or other device to dissipate the energy of the water discharged. Various types of valves and gates are discussed in detail under the subject "Closed Conduit Flow."

In many studies, determination of the type of valve to be used is affected by costs or by the nature of the desired prototype performance. In cases where there is question as to the most appropriate type of regulating device, tests should be conducted to determine the relative merits of proposed devices, including a consideration of costs. The cost of the associated stilling pool structure should also be considered since different valves may require different types of pools. At the time of this writing, the

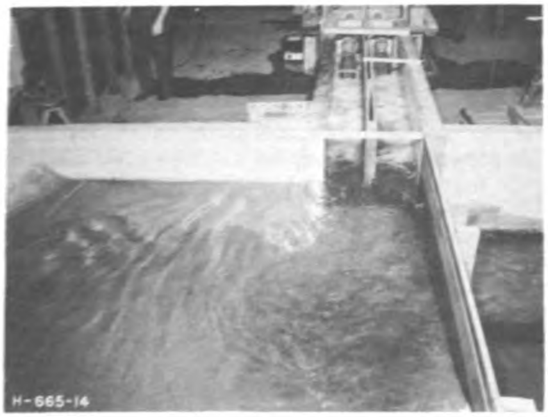
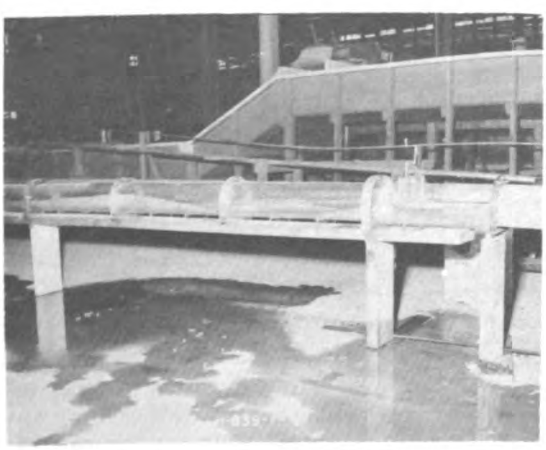


Figure 36. Model stilling pool for Boysen Dam outlet works

hollow-jet valve (Figure 51) has largely replaced other types in new installations. However, research on other valves is continuing and improved and cheaper equipment may be developed to replace the hollow-jet valve. Two hollow-jet valves are used for outlet regulation at the Anchor Dam, Figure 26A. Slide gates of various types are sometimes used on the smaller outlets, but flow regulation is difficult unless the gate is at the end of the conduit. Valves and gates, for use in outlet works, may be classified in two general types: (1) those that produce a solid or converging jet and (2) those that produce a diverging or hollow jet. These usually require different types of stilling pools because the jet from one remains solid while that from the other spreads more readily on entering the basin. Should a gate be used instead of a valve, a still different type of pool may be feasible.

1. Outlet works model design.--In making model tests of an outlet works having the flow control at the end of the conduit, the first objective is to obtain geometrical and dynamic similarity between model and prototype. The model valve should be geometrically similar to its prototype, but since dynamic similarity cannot be obtained at the downstream end of the conduit by constructing a geometrically similar conduit, because of excessive friction in the model conduit, it is necessary to simulate prototype conditions by other means. In small models, friction losses are proportionately greater in the model than they are in the prototype, due partly to the impossibility of scaling down the roughness of the prototype conduit surface. However, by making the conduit length in the model shorter than indicated for geometrical similitude, the corresponding prototype pressures and velocities can be reproduced in the model. This method is advisable if the entrance or intake structure is part of the model study, since the entrance may also be built and studied at the same time. In many cases, entrance studies are not considered to be necessary and only a short piece of approach conduit immediately upstream from the valve need be constructed. This approach conduit may then be directly connected to a water supply source. A piezometer is usually installed in the conduit, one diameter upstream from the valve, for measuring the head on the valve. Water is then supplied to this point in the quantity and velocity required, as in-



A. Two slide gates and tunnel downstream. Flow is from right to left

B. Tunnel and stilling pool

Figure 37. Outlet works model-Keyhole Dam, scale 1:20

dicated either by studies in a larger model or by analytical studies and computations in which the estimated resistance of the conduit has been considered. Thus, when the model discharge is regulated to give the computed pressure at the piezometer, the model valve discharge will properly represent the form of the jet and the energy being delivered to the model stilling pool. Figure 36 shows a model of the outlet works for the Boysen Dam in which there is only a short piece of conduit upstream from the valve. Figure 37 shows a model of the outlet works for the Keyhole Dam, which is gate-controlled. Only that portion of the conduit downstream from the gates was constructed.

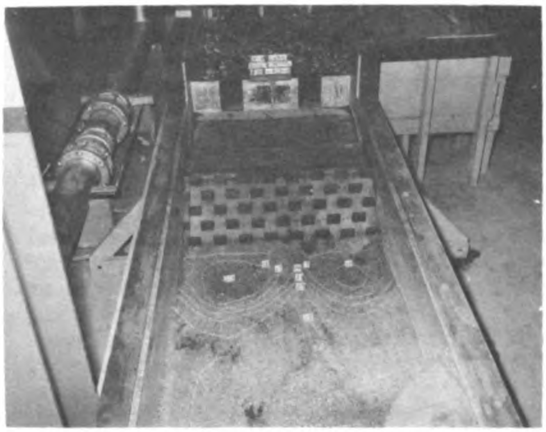
It has been found from experience that model valves should be not less than 3 inches in nominal diameter. Smaller valves require special care in operation to insure accurate results. Furthermore, the 3-inch or larger valves discharge sufficient water to make observers appreciative of the problems to be investigated.

Tests on outlet works stilling pools should be made as explained in the section dealing with stilling pools. There is, however, a definite trend toward elimination of the hydraulic jump pool from outlet works structures. Because the jet from a valve or gate is necessarily concentrated, excessive size in the basin pool is usually needed to break up and spread the flow so that a jump can be formed. Several new types of pools have been developed in the hydraulic laboratory for use on various Bureau projects. These perform as well or better than those of the conventional jump type and are smaller, more compact, and less costly. They use a modified hydraulic jump. The principles of these basins are explained in a paper presented at the Fourth Congress on Large Dams.[17]

Canal Structures

Since canals are usually uniform in cross section and slope, problems of canal design may ordinarily be solved by analytical methods. However, canal structures such as drops, overchutes, turnouts, wasteways, river crossings, or combinations of these may present problems in which the laboratory can assist in obtaining satisfactory performance. Hydraulic models of canal structures are built geometrically

A. Control and drop for trapezoidal canal



B. Typical wash overchute structure

Figure 38. Canal structure models

similar to their prototypes and, gravity being the predominating force, the Froude law governs. In unusual cases, where friction forces are suspected of being so large as to affect the model tests, consideration should be given to adjustment of the model to compensate for excessive friction. An example is a long flat chute where friction dissipates an appreciable part of the total energy contained in the flow. Many canal structures are identical in principle with spillways and stilling basins. Consequently, no attempt will be made to repeat here the criteria for the study of chutes, stilling pools, pressure measurements or other subjects already discussed. The following discussion pertains to features of the structures which are peculiar to canals.

1. Drops and overchutes.--In the main canal, or in a wasteway which branches

17 Peterka, A. J., and Tabor, H. W., "Progress in New Designs for Outlet Works Stilling Basins," Quatrieme Congres des Grandes Barrages, New Delhi, India, 1951; Commission Internationale des Grandes Barrages.

from the main canal, it may be necessary to lower the canal level abruptly by means of a chute (often called a drop). The chute conducts the water from the upper level through a stilling pool and a transition into the lower canal. A check is sometimes used at the top of the chute to maintain normal depth in the upper canal for all rates of flow. Checks may consist of weirs, gates, stop logs, specially designed notches, or other mechanical schemes for regulating the flow. A check and chute, trapezoidal in cross section, is shown in Figure 38A. In developing a chute structure the check should be carefully calibrated to insure that the required amount of water will be passed for all elevations of the upper canal water surface. A curve of discharge versus water surface elevation will provide the needed information for any particular control. Checks and weirs often produce peculiar wave formations in the chute which should be studied and corrected if necessary. The stilling pool should be proportioned to operate over the desired range with a minimum of surges and waves. Operation of rectangular chutes and stilling pools has been fully discussed. Trapezoidal chutes and stilling pools are common for canal structures. Grooves in the chute floor and baffle piers in the stilling pool may aid in providing satisfactory stilling action in a trapezoidal pool. In some drops where a natural tail water depth does not exist, a control or check may be used at the end of the stilling pool to increase the pool depth and prevent high-velocity flow from entering the lower canal, creek, or natural drainage area. Waves and surges originating in the stilling pool require special study in canal structures because they may damage the canal downstream. It is usually more economical to design the pool to prevent waves and surges than to provide bank protection such as riprap or lining. Baffle piers or sills may help to reduce wave action but the possibility of their effectiveness being reduced by collection of weeds or debris should be considered. The transition from the stilling pool to the canal should be geometrically simple and should not expand too rapidly. Rapid expansions of any kind promote unstable flow and appear to magnify surface waves and surges. Eddies produced by improperly designed transitions tend to undermine the ends of the transition walls.

Overchutes are used to conduct run-off from washes and ravines over a canal. In the more arid sections of the country, many structures of this type may be necessary for one canal. Without them, tremendous amounts of silt and debris would find their way into the canal and washouts would be caused by the flow overtopping the canal banks. Cost of removing the debris or repairing washout damage might be large. A typical overchute resembles a bridge over a canal with a slab of concrete paving downstream, appropriate training walls, and an energy dissipating device. Model studies are concerned generally with determining the capacity and the effectiveness of dissipation. A typical overchute is shown in Figure 38B. The flow is conducted over the bridge and then passes over sloped paving containing rows of baffle piers over which backfill is placed to the level of the downstream channel. The piers are so arranged that the energy in the flow passing over the piers is dissipated at a rate equal to that at which is would accumulate during free fall. Thus, regardless of how far the water tumbles over the piers, the energy content of the flow remains approximately the same. Degradation of the downstream channel merely exposes additional apron and piers and the erosion pattern is no more severe than before.

2. Turnouts and river crossings. --Turnouts serve to divert flow from a main canal into a lateral system and may present an operating problem, particularly if high-velocity flow occurs in the vicinity of the turnout. If a bend influences the normal flow pattern near the turnout, or if trashracks are placed on the turnout to prevent floating weeds or debris from entering the canal laterals, an operating problem may likewise be presented. Hydraulic models have been used to solve a variety of turnout problems which are usually concerned with obtaining the proper quantity of flow regardless of canal flow conditions. Rating curves from a model for various turnout gate openings and a range of canal flows will usually provide the necessary information. Pressures in the turnout may require consideration if high-velocity flow is encountered in the canal or turnout. Tests may also be necessary to determine whether a turnout is likely to cause trouble by intercepting floating weeds or debris in the canal. If hand-operated cleaning rakes are used on the trashrack, it may be desirable to use a rack and turnout that model comparison tests have demonstrated will intercept the least trash.

River crossings are used where a canal crosses a river or other flowing stream and may be studied to enable selection between open channel and closed conduit types. Inverted siphons, which in reality are pressure conduits, are used where a canal crosses a depression and the slope of the canal is too great for an open ditch. Unusual entrance, pressure, exit, velocity, or bend problems may make model studies desirable.

River Channel Models

The term "river channel model" is used in a broad sense, and applies to all models in which the slope of the water surface throughout the model is relatively flat. These models usually involve studies of flow patterns or movements of sediment and can be classified in two types: fixed-bed models and movable-bed models.

Typical problems studied with river models include (1) effects of closing or opening certain channels in a multiple channel river, (2) hydraulic losses at constrictions such as bridge piers or cofferdams. (3) probable erosion around cofferdams or other structures, (4) characteristics of flow in and around diversion schemes, (5) effects of currents and waves on river navigation, and (6) tidal and estuary problems.

1. Fixed-bed models.--Problems involving relatively long stretches of a canal or river, wherein actual changes in bed configuration are not critical, are usually studied with the aid of fixed-bed models. Such problems include study of backwater effects in a river due to obstructions or the changes in backwater conditions due to channel improvements, flood routing studies, determination of flow distribution in estuary channels, etc. Although the Froude law of similitude is applicable, special attention must be given to the similarity of boundary resistance, which is of major importance and cannot be ignored or adjusted when the results are interpreted. In the case of a large model the velocity may be high enough to insure turbulent flow and the minimum model roughness may give smooth enough boundaries to represent the prototype properly. Then a geometrically similar model may be used.

Unless the model is unusually large, a distortion in slope is required (1) to offset the disproportionately high resistance of the model boundaries and (2) to obtain a sufficiently high value of Reynolds number to insure turbulent flow.

Distortion in slope required to satisfy condition (1) can be computed with sufficient precision by the Manning formula.

$$V = \frac{1.49}{n} R^{2/3} S^{1/2}$$

Since the Froude law requires that $V_r = \sqrt{D_r}$ the formula may be written

$$V_r = \sqrt{D_r} = \frac{R_r^{2/3} S_r^{1/2}}{n_r}$$

Substituting D/L for S results in the expression:

$$D_r = \frac{R_r^{2/3} \sqrt{D_r}}{n_r \sqrt{L_r}}$$

This expression may be rearranged to the following:

$$D_r/L_r = \frac{n_r^2 D_r}{R_r^{4/3}} \qquad (8)$$

If roughness coefficient n is known for model and prototype, n_r is known and the distortion D_r/L_r can be computed for the hydraulic radius ratio of the mean section.

When the slope distortion required to satisfy condition (2) is greater than that required for condition (1) the model must be made rougher by artificial means to compensate for the exaggerated slope.

The required model roughness can be computed from Equation (8) if the distortion and the prototype n are known. Then the model boundaries are roughened by various means such as distributing bits of mortar, grains of sand or gravel until the required value of n is obtained. When general data regarding the equivalent roughness k of various devices such as screens or expanded metal lath are available the model roughness can be designed. If such information is not available, the process is one of cut and try. The model with roughness adjusted for a particular depth will yield dependable results for flow at or near this depth. When a problem involves several depths, the model roughness should be adjusted to give an average resistance which is approximately correct over the desired range or the roughness may be varied with depth for a closer approximation. Figure 39 shows a fixed-bed model which was used to study the distribution of stream and tidal flow with respect to salinity intrusion into the Sacramento-San Joaquin Delta in California. The distortion of this model was 48 as dictated by space limitations. It required artificial roughness in the form of hardware cloth placed vertically in the channels and folded in such a way that it formed a zigzag pattern.

Figure 39. Distorted fixed-bed model of Sacramento and San Joaquin Rivers: horizontal scale 1:4800 and vertical scale 1:100

In many problems the roughness of the prototype is not known and therefore cannot be reproduced in the model. In such cases studies are made for a range of resistance which is estimated to encompass the prototype resistance.

2. Movable-bed models.--There are many open channel problems involving scouring, deposition, and transportation of channel-bed material. Such problems are studied by means of movable-bed models. In some cases, however, limited studies of problems of this type can be made by investigating current direction and velocity in fixed-bed models. Despite the limitations of the similitude obtainable with movable-bed models, they have proved to be a valuable aid in the solving of complex problems involving the shifting of stream bed materials. Similitude in movable-bed models defies the mathematical analysis which can be applied to models involving hydraulic structures or other fixed boundary studies. Instead of arranging the various hydraulic forces involved to meet definite requirements laid down in any law of similitude, the successful prosecution of a movable-bed model study requires that the combined action of the hydraulic forces bring about similitude with respect to the all-important phenomenon of bed movement, the essence of this type of model study.

The general design approach is that of selecting scales and bed material which will result in bed movement of a nature generally similar to that in the prototype, taking into account the relative effects of the various discharges from minimum to maximum. There are two prerequisites to such a design approach: first, thorough knowledge of the characteristics of the prototype based on the collection and study of hydraulic and hydrographic data; and second, experience in the field of movable-bed hydraulic models.

When the dimensions of a water course, including the particle size of the bed material, are scaled down to model size a discontinuity is encountered. The usual model scales result in sediment particles which are so small that they no longer act like bed material, but tend to become either suspended or compacted into an unyielding bed. The necessity for using bed material of particle sizes greater than required on a dynamic basis necessitates the use of geometrical distortion in order to obtain the essential movement of the bed material. The vertical scale ratio is therefore made larger than the horizontal scale ratio. Thus, all slopes in the model are exaggerated. The distortion (vertical scale divided by horizontal scale) may vary from 2 or less to as high as 7, or even more in special cases. Generally speaking, distortion should be kept as low as possible without reducing bed movement appreciably.

In problems involving the study of sediment action relating to a structure such as the canal intake and sluiceway of a diversion dam, distortion which would include the structure, is not advisable. In such cases, the model must be large enough to insure movement of available model sediment. An example of this type of study is the Courtland Canal headworks of the Superior-Courtland Diversion Dam. 17A The model shown in Figure 40 has an undistorted scale ratio of 1 to 15. The model sediment was a sand having a mean particle diameter of 0.17 mm and a condition of general sediment movement was obtained by permitting the bed of the model reservoir to assume a distorted slope by continuously feeding in sediment at the supply end. Since the problem was qualitative in nature, it could be studied because the model sediment moved readily.

Although vertical exaggeration is usually necessary from the standpoint of obtaining sufficient bed movement, it does introduce certain undesirable effects, some of which have been mentioned previously but which warrant additional discussion at this point. The exaggeration may increase the slopes of the model banks beyond their angle of repose so that they will no longer stand. Distortion also increases the longitudinal slope of the stream, thus tending to upset the flow regimen to a point where artificial model roughness is required to restore it. The vertical exaggeration also causes distortion of the lateral distribution of velocity and kinetic energy.

The difficulties of compromising resistance and bed movement have been minimized by using various model bed materials of lower specific gravity, so that less scale distortion is required to produce proper movement. Among such materials are coal, pumice, sawdust, ground plastics, etc.

A bed load feeding apparatus is usually desirable for a movable-bed model. It is provided at the channel entrance to supply bed load to the model. The rate of feeding should be adjustable so that it can be varied as required to accomplish proper duplication of prototype action. A special feeding device developed in the laboratory was described under the subject of Laboratory Equipment. Another device which has been used is a submerged elevator which raises sediment to the channel bed where it is moved on by the water flowing over it. When it is necessary to recirculate sediment to attain equilibrium conditions quickly, a jet pump in which there are no moving parts may be used to pick up sediment from a pump at the end of a model.

3. Verification of river models.--The verification of a movable-bed model is an intricate, cut and try process of progressively adjusting the various hydraulic forces and varying the model operating technique until the model will reproduce, with acceptable accuracy, changes in bed configuration which are known to have occurred in the prototype between certain dates. In this way, the accuracy of the functioning of the model is established, and certain of the scale ratios, such as time and discharge ratios, are determined experimentally. The verification procedure may consist of the following steps:

a. Two prototype bed surveys of past dates are selected. (The time between these two dates being known as the verification period.) The model bed is molded to conform to the earlier survey.

b. The hydraulic phenomenon which occurred in the prototype during the verification period is simulated in the model to the proper time scale (which is estimated to begin with), to see that all of the regulative measures undertaken by nature during that period are reproduced in the model at the proper time.

c. The model bed is surveyed at the end of the period, and the model is considered to be satisfactorily verified only when this survey checks the prototype survey with acceptable accuracy.

During the cut and try verification process, it may be found necessary to manipulate the time scale, discharge scale, rate and manner of bed load feeding, or the slope scale of the water surface, and perhaps the gradation of the bed materials. Often, it is necessary to use scales for time and discharge which vary with stage so that each model stage will effect its proper share of movement. It is evident, therefore, that the verification phase of a movable-bed model calls for an intimate knowledge of the prototype as well as experience with this type of model study.

After a satisfactory verification of a river model, the degree of similitude attained remains largely a matter of judgment. The similitude is based on this

17A. Martin, H. M. and Carlson, E. J. "Model Studies of Sediment Control Structures on Diversion Dams," Proc. Minnesota International Hydraulics Convention, IAHR, ASCE, Sept. 1-4, 1954.

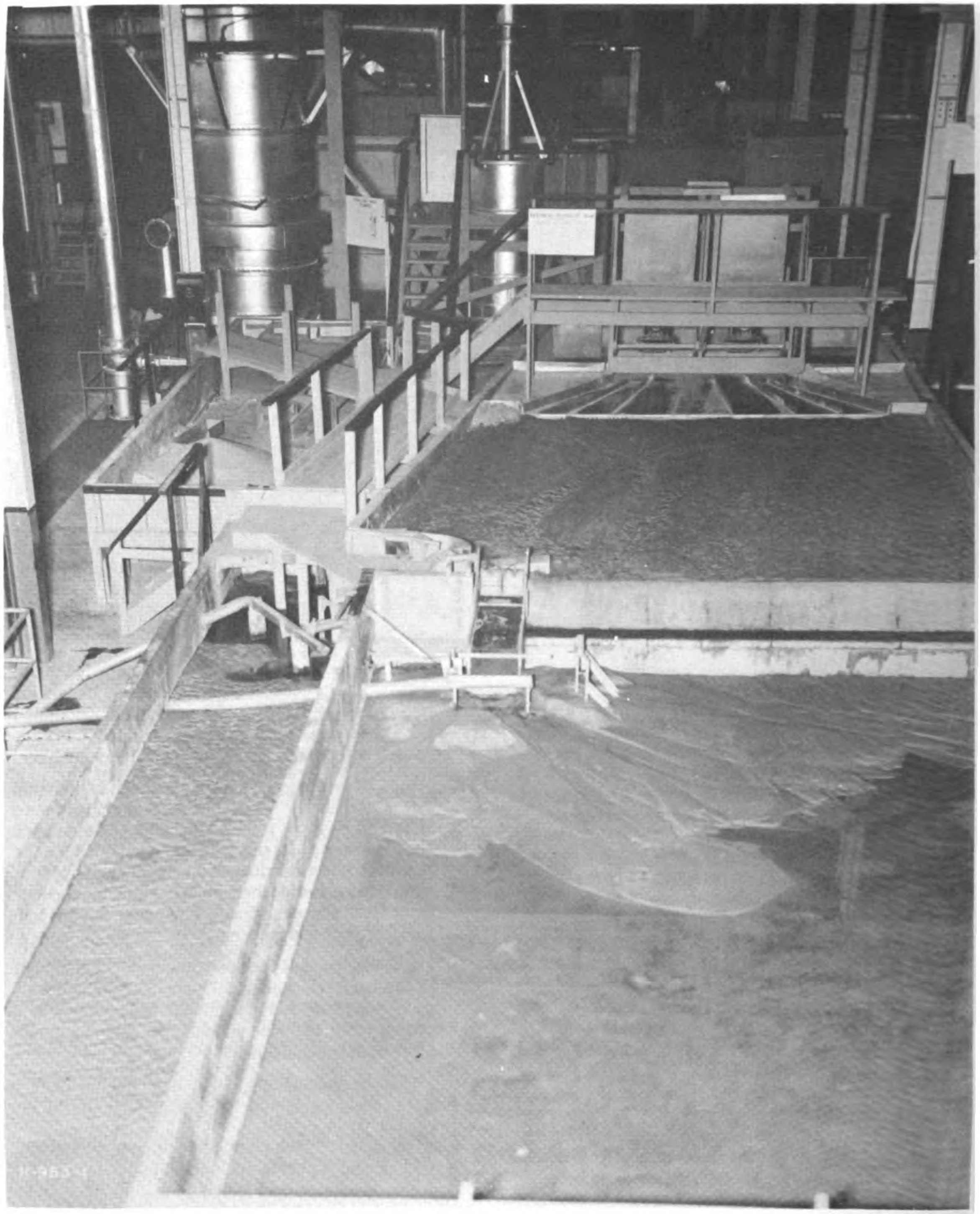


Figure 40. Superior-Courtland Diversion Dam and Headworks model; movable bed; undistorted

general reasoning: if the model accurately reproduces changes which are known to have occurred in the bed of the prototype, it can be relied on in predicting changes of a similar nature which can be expected to occur in the future. These dynamic distortions inherent in the verification of a movable-bed model place an important limitation on the type of tests which can be made and thus on the type of results which can be obtained from the study. Since the verification is achieved on the basis of an adjusted simulation of recorded prototype phenomena, the model cannot be expected to respond accurately to conditions which involve a drastic departure from those involved in its verification. In the final analysis, the validity of the results of a movable-bed river model study and the interpretation of its results are largely dependent on general judgment and reasoning, the basis of such reasoning being the verification of the model, a knowledge of the prototype, and familiarity with the general characteristics of such models.

Fish Structures

Fish structures consist of ladders, racks, and traps. Fish ladders are used to allow the fish to be self-transported from a lower to a higher level. Fish racks are used to stop fish from migrating upstream. A rack is usually used in conjunction with a trap. Fish traps are simply large baskets in which fish are trapped for scientific study.

One problem, common to all types of fish structures, involves the method of introducing the flow so as to avoid high velocities and extreme turbulence. Hydraulic models can be very helpful in observing currents and turbulence and can thus enable confirmation of the assumptions and principles on which the structures are designed.

Salmon do not feed during migration upstream, but live from the energy stored in their bodies. Thus, an easy passage must be provided at any obstruction in their usual path. Since fish, in migrating, proceed upstream and will not reverse their direction, it is essential that their progress be facilitated by placing the entrance to a fish structure, such as a fish ladder, adjacent to an obstruction in the river. Also, the salmon must be quickly attracted to the entrance by turbulence created by the flow from the ladder as it spills into the river. Some turbulence must be created in the ladder by water falling over weirs to induce the fish to proceed up the ladder. If the turbulence is excessive, the fish may jump out of the ladder. and if the velocity of flow in the ladder is too great, it may require expenditure of too much energy by the fish. Should the exit of the ladder be placed near an intake structure, the fish leaving may be caught in the swiftly moving water approaching the intake and be swept downstream. A model of a typical fishway is shown in Figure 41.

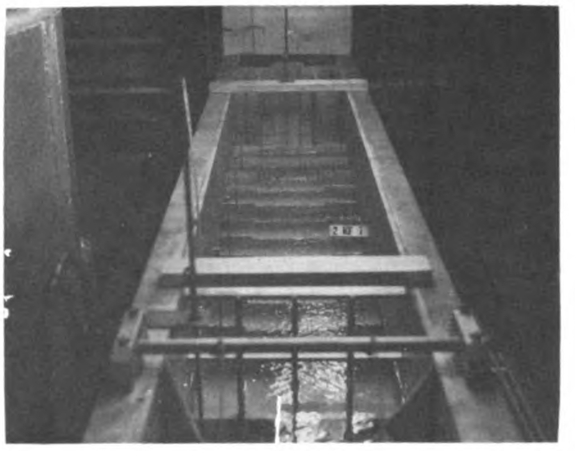


Figure 41. Fish ladder model-Keswick Dam--scale 1:20

Fish racks and traps are used to divert the fish from their usual path into a basin or other structure for the purpose of trapping. As mentioned before, excessive turbulence and velocities should be avoided. This may be an easy matter when the racks are clean, but tests should be made to determine the effects of a rack partially clogged with debris. As the successful use of these structures depends on the habits of the fish, the advice of authorities on fish, both State and Federal, should be sought before a particular design is recommended. Relatively few fish structures have been built and improvements in present designs are highly probable.

Lock Model Investigations

Filling and emptying systems for navigation locks present the most difficult problem in lock design and a hydraulic model provides the only means for obtaining a satisfactory system. The emptying system usually makes use of the same facilities used to fill the chamber, but experience has shown that emptying the chamber presents problems if the filling system operates satisfactorily. Filling the chamber, therefore, presents the major problem.

In modern navigation locks, the time necessary for filling the chamber must be as short as possible, to minimize over-all lockage time. In some high lift locks, it is often necessary to fill a chamber 110 by 600 feet to a depth of 80 feet in 10 to 12 minutes. Peak inflow may reach 5,000 cubic feet per second. Thus, flow into the chamber must be exceptionally well distributed to prevent boils and currents in the lock chamber which might swamp small craft or cause larger craft to break their mooring hawsers and damage themselves or the lock gates.

In providing smooth uniform flow conditions in the lock chamber other problems will become apparent. For example, if the ports and culverts or other flow passages are economically designed, cavitation pressures will probably occur in critical parts of the structure. Piezometers in the filling conduits should be used to be certain that pressure conditions are satisfactory throughout the filling cycle. Study of a separate larger model of one of the ports may be necessary to eliminate these pressures and obtain an efficient port. The speed and method of opening the filling valves will affect the size of the surges in the lock chamber and many trials may be required before a uniformly increasing discharge into the chamber is attained. Smooth filling of the system may necessitate admission of air. The relation between port and culvert size should be investigated since only a limited number of ports properly spaced can be used efficiently with a culvert of given size.

Filling and emptying systems and the large models of the port are governed by the Froude law, even though the chamber is filled through long culverts and viscous effects undoubtedly play a part in the hydraulics of the system. Model-prototype comparisons of several locks have shown remarkable agreement in filling time and general over-all performance where the prototype was constructed following recommendations derived from models based on the Froude law and constructed to reasonable size.

To evaluate the model filling and emptying systems, model ships or barges having scale values of the prototype displacement and size should be moored in the chamber. A measuring and recording system should be devised for the model that will indicate precisely and simultaneously the time of filling the chamber, the speed or rate of opening of the filling valves, the horizontal components of the forces acting on the model ships or barges, and other data deemed necessary for evaluation of the filling system design. Experience with lock models has shown that the filling valves should be opened by a motor-driven variable-speed device so that tests may be reproduced with precision and that all data should be recorded mechanically on a moving strip of paper on which exact time intervals are also recorded.

Other problems usually arise. It has been found inadvisable to spill water from the chamber during the emptying operation into an area where other craft may be waiting for lockage. Provisions should be made to protect these craft from the turbulence created by the emptying operation. Also, the tail water into which the emptying conduits discharge must be at substantially the same elevation as the tail water below the main lock gates. Otherwise, the main gates will have to be opened against a static head, an unnecessary and expensive procedure. Lock approach conditions in the upstream and downstream areas should be investigated by means of model barges on a separate small model in which the upstream and downstream approaches can be constructed. Lock approaches should be free of cross-currents, high-velocity flow, waves, and any other objectionable occurrences that might cause difficulty to craft entering the lock. It is difficult to accomplish a satisfactory design for locks close to a spillway.

The problems outlined are representative of the many problems which may be encountered. Publications[18] are available which discuss the many aspects of lock problems more thoroughly and these should be studied before starting any investigation.

CLOSED CONDUIT FLOW

General

Laboratory investigations of closed conduits may pertain to a wide variety of hydraulic facilities, systems, equipment, and devices, ranging from large outlets, penstocks, or aqueducts, to small fluid passages of machines or their controls. Although closed hydraulic systems may serve entirely different purposes, they consist of similar component parts and involve investigations of similar nature. This is true even though it may be desired to minimize hydraulic losses in some cases, as in the design of a power penstock, while it is important that the losses be a maximum in others, as in the design of an energy dissipator. Each problem may concern one or more of numerous hydraulic characteristics or conditions. The laboratory is concerned mainly with those in the following list.

18 "Hydraulic Model Investigations of Lock Filling and Emptying Systems," TVA, Knoxville, Tennessee, Technical Monograph No. 64.

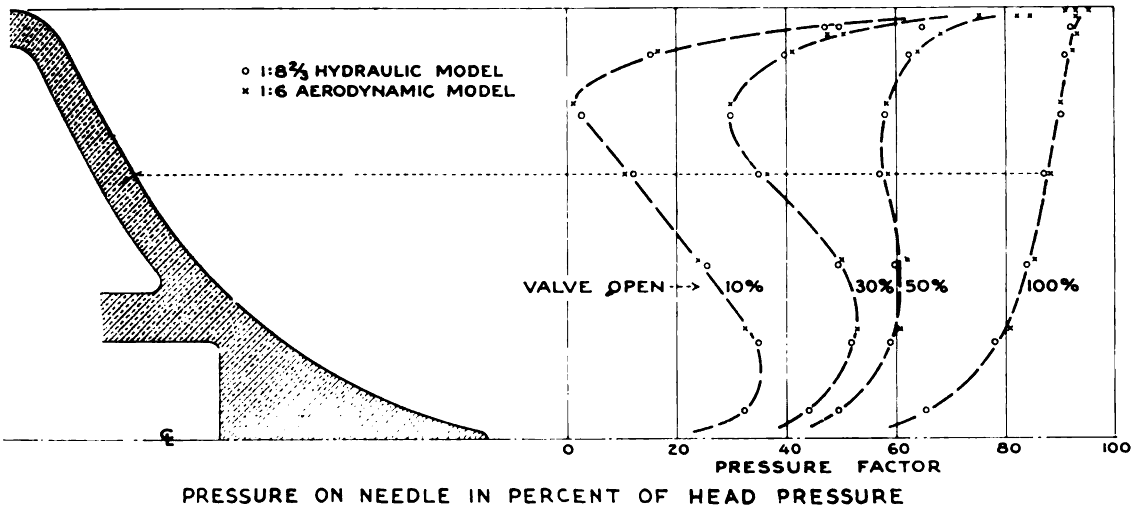


Figure 42. Comparison of pressure distributions obtained from hydraulic and aerodynamic models

Many of these subjects will be covered in more detail in following discussions that deal with particular hydraulic systems, equipment, devices, and their component parts.

(a) Conduit size (based on efficiency or capacity)

(b) Entrance shape (based on pressures and losses)

(c) Losses and flow action in pipe sections, bends, branches, fittings, and special shapes

(d) Energy gradients and their relation to flow passage alinement

(e) Effects of additions or alterations to existing systems (pressures, losses, and flow conditions)

(f) Controlling device design (with respect to capacity, efficiency, pressure distribution, losses, boundary shape, flow action, hydraulic practicability, and torque or other forces on operating mechanism)

(g) Operating characteristics of hydraulic equipment and component parts. (Operating procedure, determination of capacity data for rating curves and tables, and flow action, including water-hammer and cavitation.)

Tests for investigation of these characteristics or conditions are usually conducted on models or equipment, with water as the fluid. However, oil, air, or any other suitable fluid might be used. Of the fluids other than water, air is the most widely used in the laboratory. Testing with air offers many advantages in certain types of investigations. However, its use may necessitate special considerations, techniques, and boundary conditions which must be recognized by the laboratory engineer if he is to obtain the answer to his problems. The following section provides background information pertaining to the use of air as a fluid in solving hydraulic problems. Models are made geometrically similar to their prototypes except as otherwise indicated.

Testing with Air as a Fluid

Use of low-velocity air for studies of hydraulic equipment such as valves, gates, and other closed-conduit devices offers quick, inexpensive solutions to many hydraulic problems. A model can be constructed and tested more rapidly and with an equal degree of accuracy, particularly when a high degree of refinement is not involved. If air velocities are kept below 300 feet per second (about 1/4 sonic speed) the effects of compressibility can be ignored, and computations made by hydraulic formulae. If the results are expressed in dimensionless terms they can be applied directly to the prototype. Figure 14 shows one of the laboratory centrifugal blowers supplying air to a model of a conduit entrance. Low-cost lightweight material can be used for the test structure and changes made readily. Modeling clay, molding plaster, or wood serve admirably for boundaries in cut-and-try experiments. Proper representation of boundary conditions is one of the most important factors to be considered when making tests with low-velocity air. Results are likely to be in error and of little or no value if the boundary conditions are incorrectly represented. Data on Figure 42 are examples of results from air and water tests. Several articles have been written on low-velocity air testing.[19]

Though the use of air in place of water as a test fluid is limited primarily to closed conduit flow, this method is a most useful expedient for many model studies. A minimum of equipment is required and there is no problem of storing and disposing of the fluid as in the case of water unless smoke having objectionable properties is used to enable observation of flow action. Moreover, the usual difficulties encountered in bleeding piezometer tubes are nonexistent. Greater precision might be required with air because the liquid columns used to indicate pressure or velocity are usually not more than a few inches in height, but suitable precision instruments for air testing are available. The advantages of using air for solving hydraulic problems are indicated by the fact that one large European manufacturing company maintains a laboratory entirely devoted to aerodynamic testing of hydraulic machines.[20] In many cases it is advantageous to employ sectional models in air testing. These are usually pie sectors of valves or similar devices from which the air passes directly into the atmosphere. If the models of such devices are to be of the whole structure, many of the component parts may be turned from wood, molded from plastic, or machined from plastic castings. If the sector-type model is used it is often convenient and desirable to construct it of molding plaster, shaping it through a process of building up and scrapping off the surplus plaster by means of sliding or revolving templates, as the material sets. Sector models of less than 180° are most common.

Cavitation

One of the main problems with which the laboratory is concerned in closed conduit flow is that of developing cavitation-free designs or determining what alterations are necessary in existing designs to eliminate or minimize vibration and damage from cavitation. The present-day concept of cavitation is explained in the following paragraphs to provide background for model testing in which cavitation is of concern.

Although the engineering profession in general has accepted the present-day theory concerning the nature of cavitation only within the past 15 years, cavitation itself is not new. Engineers have viewed with apprehension the pitting or erosion of surfaces resulting from cavitation in hydraulic structures for many years. Cavitation-erosion in three different structures is shown on Figure 43. Many attempts were made to prevent, eliminate, or resist it, but little success was attained until general application was made of the present theory of cavitation, the source of which action is attributed to subatmospheric pressures which approach vapor pressure.

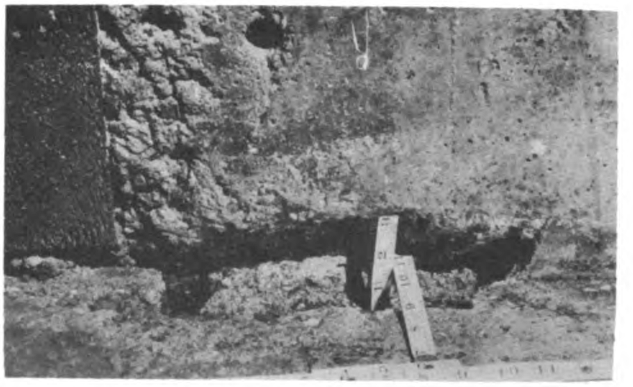
The term "cavitation," as used in this chapter, is defined as follows: cavitation in a hydraulic structure occurs when the pressure at some point is reduced to the vapor tension of the flowing fluid (about 0.5 psi absolute for water). Cavitation is the action which takes place in this low-pressure region and consists of the formation, transportation, and collapse of vapor cavities. The cavities which form in the regions of vapor pressure are carried downstream by the flowing fluid to collapse or implode as they reach a zone of higher pressure. The tremendous forces that accompany implosions which occur on solid boundary surfaces cause disintegration of the boundary material. This destruction is termed "cavitation-erosion." Basically, cavitation results from the relative motion of a fluid and solid, either or both of which may be in motion at the same time.

Cavitation in structures may be induced in many ways. The following causes are encountered most frequently: (1) curving a boundary surface too rapidly away from the normal path of high-velocity fluid streams; (2) permitting irregularities or discontinuities in boundary surfaces which are subject to high-velocity flow; (3) using extreme variations in elevation in a conduit system which produce siphonic action if not controlled properly; (4) moving unstreamlined objects through liquids at high speeds or passing high-velocity flow over unstreamlined objects; and (5) expanding the flow area too rapidly in the direction of motion.

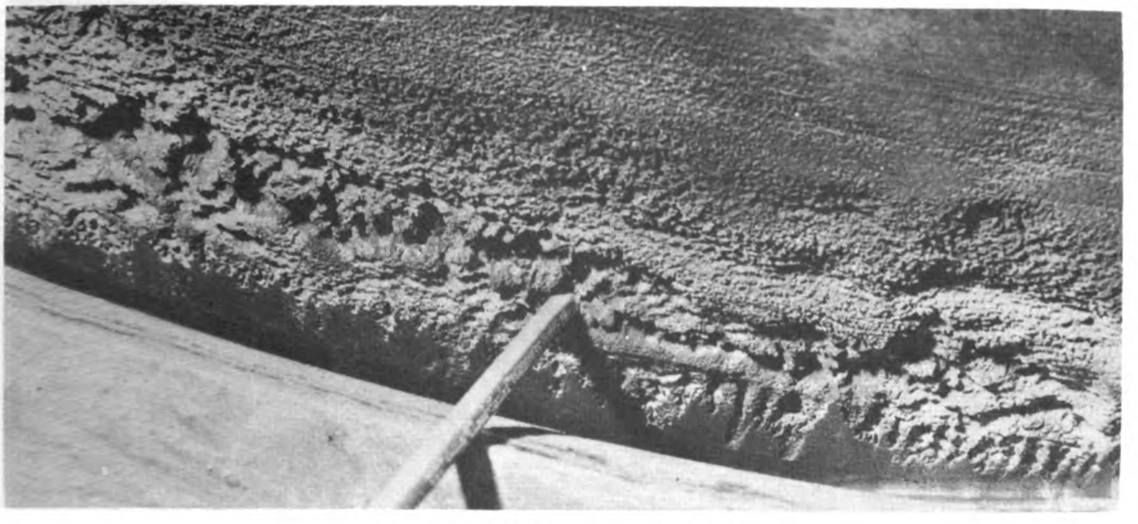
Regardless of its origin, cavitation is likely to cause dangerously unstable operating conditions or structural weakening, because of destructive disintegration of boundary surfaces. More or less vibration

19 Ball, James W., "Model Tests Using Low-velocity Air," Proceedings, ASCE, Vol. 77, Separate No. 76, June 1951.
20 Keller, C., "Aerodynamische Versuchsanlagen fur Hydraulische Maschinen," Schweizerische Bauzeitung, Vol. 110, No. 17, October 1937.

A. Pitting which has penetrated through a 5/8-inch penstock plate.



B. Pitting of concrete and steel downstream from a large gate



C. Pitting of semi-steel on the needle of a needle valve

Figure 43. Illustrations of Cavitation

is always present, as are popping and crackling noises known to hydraulic engineers as "crepitation."

When cavitation is present in a structure, the destructive action may become so severe as to require corrective treatment or imposition of certain restrictions in operation. One or more of the following might need to be applied: (1) imposition of strict operating limitations; (2) supplying air to the low-pressure region in order to increase the pressure to a value above the vapor pressure of the fluid; (3) alteration of all or a portion of the flow passage; (4) streamlining parts directly in contact with high-velocity flow; (5) maintaining the damaged portion by periodically placing more resistant material in the eroded areas; or (6) allowing the pitting to continue until it ceases naturally.

1. Preventing cavitation in new structures.--With today's engineering knowledge and experience, it is possible to develop cavitation-free designs. Because much uncertainty still exists regarding basic relationships among the hydraulic properties involved, it is seldom possible to accomplish this through calculations alone. Where there is question as to the validity of the assumptions used in the calculations, they should be verified by hydraulic investigations of scale models or special apparatus constructed for that purpose. Any design may be tested for cavitation in a hydraulic laboratory by making suitable measurements on a model. The reliability of the information obtained in this manner as criteria for use in predicting what will take place in the full-size structure will depend on the suitability of the model, the accuracy and ap-

propriateness of the data taken from it, and the rationality of the analysis and interpretation of the data.

2. Testing for cavitation.--From the very nature of the cavitation phenomenon, it is evident that pertinent tests will involve determination, by extensive measurements, of pressure intensities and their distribution on flow boundary surfaces.

There are two different techniques for testing geometrically similar models: (1) operation at atmospheric pressure, and (2) operation at scaled atmosphere, or reduced pressure. When the problem involves prediction or elimination of cavitation erosion, the first technique is suitable. Detail measurements of pressure distribution in the model may reveal subatmospheric pressures which, scaled to the prototype terms, may approach vapor tension of the liquid. This may be regarded as sufficient evidence of cavitation in the prototype. For example, tests of a model valve indicate subatmospheric pressures of 5 feet of water at a model head of 50 feet. If the prototype head is 300 feet, subatmospheric pressures of 30 feet may be predicted, and cavitation in the prototype is possible. In practice, the model shape would be revised to eliminate the subatmospheric pressures, or, in special cases, vacuums not to exceed one-half atmosphere in the prototype are tolerated. Negative pressures greater than one-half atmosphere are considered to be in the realm of potential cavitation, since the irregularities of the boundary may create local reductions in pressure and thus produce local cavitation. In this type of testing, the location of a piezometer is of major importance. Piezometers must be located so that pressure distribution in all critical areas selected on the basis of judgment or analytical studies will be delineated. In general, the piezometer openings should be located in expanding sections, curved surfaces, and downstream from discontinuities. Cavitation is not actually produced in the model in this method of testing with either water or air as the test fluid. The extent of cavitation and cavitation-erosion cannot be predicted from pressure measurements, but the fact that it will occur can be ascertained from the model data. The procedure to be followed in analyzing pressure data depends on the scale ratios of heads used in testing. Either the recorded model pressures for a given head may be multiplied by the head scale ratio to obtain the prototype values, or pressure factors (ratio of pressure at any point to the total head on the system) may be established through the model tests. When the scaled model values (model pressures converted to prototype pressures) indicate pressure intensities at or below absolute zero, it is assumed that vapor pressure will occur on the prototype and that cavitation will be present. Pressure distribution on a surface of the full-size structure can be predicted with a high degree of accuracy if the scaled values indicate pressures above the vapor tension of the fluid. On the other hand, if pressures below the vapor tension are indicated, the results cannot be scaled up. When such conditions are present, it is expedient to use a method known as vacuum-tank testing.

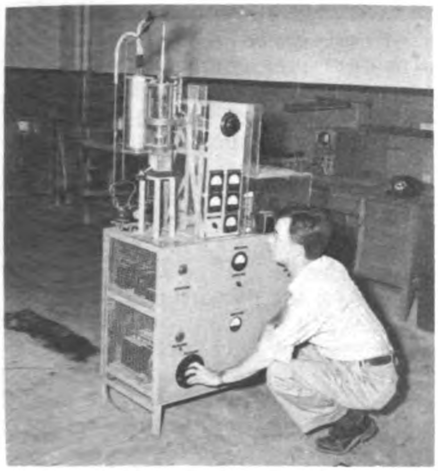
The vibration and energy loss effects of cavitation can be studied best in reduced pressure models. These tests are performed in water tunnels or any closed system wherein subatmospheric pressures of various magnitudes can be maintained at the test section. Such a test facility makes it possible to duplicate cavitation conditions in the model by arranging equal values of the cavitation numbers of the model and prototype. The cavitation number may take many forms, one of which is:

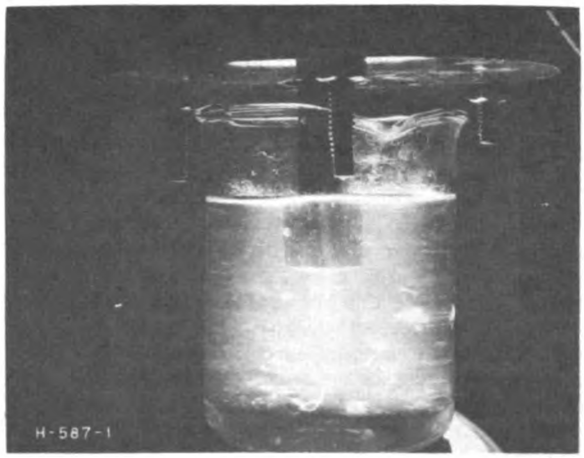
$$K = \frac{P_x - P_v}{\frac{V^2}{2g}}$$

where P_x is the pressure at a particular point in the flow system, P_v is the vapor tension pressure for the flowing fluid, and V is the velocity at some reference point.

For the same values of K in model and prototype, the patterns of cavitation and, hence, the flow efficiencies will be the same. Since the frequency of vibration and the rate of pitting will vary in proportion to the velocity and since the physical properties of the boundary materials will also be involved, such factors must be given consideration in predicting the structural behavior of the prototype from the model test.

Vacuum-tank testing for cavitation is much more complicated than open testing, but it simplifies the determination as to whether cavitation will occur and what the pressure distribution on a surface will be in the presence of cavitation. Using a model scale of 1:14 and an atmospheric pressure of 28 feet of water, the pressure in the tank or model atmospheric pressure would have to be about 2 feet of water absolute. Leakage of air into the system is an inherent difficulty in this type of testing. Pressure measurements may become more difficult due to vaporization and the special equipment required. Special pumps with liquid seals would facilitate the work.





A. The magnetostriction oscillator

B. Producing cavitation with the oscillator

C. Eroded Sample

Figure 44. Testing metals resistance to cavitation with a magnetostriction oscillator

3. Resistances of materials to cavitation.--It is possible by using special materials on the model to show which areas will be damaged by cavitation action and the extents of the areas. Erosion by cavitation has been used extensively to determine the relative abrasion resistances of engineering materials. For metals, the magnetostriction oscillator is an effective device. A photograph of the apparatus and a test specimen are shown on Figure 44. Other engineering materials have been tested in venturi-shaped passages and modifications thereof. A cavitation cone, constructed within a transparent pipe, has been used to test paint coatings. Another device having

a shape similar to that of a turbine draft tube has been used, especially to test concrete specimens. Submerged rotating discs with small obstructions in or on them have been used successfully to induce cavitation.

Flow in Pipes

Design of conduits for conveying water usually involves problems of determining hydraulic losses and measurement of the quantities of flow or the pressure intensities at particular points. Hydraulic losses in a conduit system are usually determined by piezometric measurements or application of accepted formulae and known coefficients, so their determination does not as a rule involve extensive laboratory studies. Friction factors for a wide range of Reynolds number often used for this purpose are contained in Figures 45 and 46. The information in Figure 45 is for concrete pipe and that in Figure 46 for continuous interior steel pipe. The Bureau of Reclamation has published a more complete treatise on pipe friction.[21] For special cases, such as unusual fittings, changes in section, or an energy-dissipating device, laboratory tests may be required. If a complicated system is involved, the laboratory may construct a schematic model of the system to simplify the problem.

Siphon

A siphon may be defined as a closed conduit, a part of which rises above the hydraulic grade line. The negative pressure which prevails in a siphon gives rise to hydraulic problems that are complicated because there are two factors to deal with--air and water. Siphons in automatic spillways and canal wasteways usually consist of an entrance, throat, barrel or siphon leg, and an outlet tube as indicated in Figure 47. Siphon elbows, used recently in lieu of control valves at the ends of pump discharge lines, are really special cases and are discussed separately in the paragraphs under Siphon Elbows in Pump Discharge Lines. High efficiency, adequate capacity, stability of operation, and ease and time of priming are requisites of a good siphon, Except for special cases, satisfactory siphon structures may be obtained by following the general design rules listed in Figure 47. These were established by a comprehensive, generalized model study carried out in the laboratory some years ago. The details of the study and the data resulting therefrom have been reported elsewhere[22] so no attempt will be made to repeat the information. The study included evaluations of the influence of entrance design, elevation of the lip, size of the air vent, proportions of the throat, shapes of the siphon leg and outlet tube, and effectiveness of various devices designed to decrease the priming period. The discharge capacities and efficiencies of various siphon designs as well as the necessary conditions for avoiding cavitation were also evaluated.

The Froude Law of similitude is applicable to siphons because the open channel conditions at the entrance and exit have large influence on the operation. Also, the conditions during the priming cycle are clearly governed by weight forces which require observance of the Froude similitude relationships. Boundary resistance during normal full flow operation does not conform to the Froude law but the effects are minimized by making the model as smooth as possible. When the model results are extrapolated to the prototype the disproportionately high resistance is adjusted through consideration of the Reynolds number and relative roughness. Occurrence of two-phase flow, air and water, during the priming cycle emphasizes the effects of surface tension which can be minimized only by avoiding small models. In cases where the time of priming is critical, the influence of surface tension should be evaluated by using at least two models with different scale ratios.

Siphon Elbows in Pump Discharge Lines

Siphon elbows at the ends of pump discharge lines are often used in place of valves or gates for the purpose of preventing backflow from the receiving reservoir during pump outage. During normal operation the siphon elbow flows full and a negative pressure prevails in the siphon bend or throat section. When the pumps are stopped for any reason, an air relief valve, or siphon breaker, admits air to the throat, raises the pressure, and disrupts backflow through the siphon. Design of these siphons has not been standardized and further model studies may be required. The discussion of siphon action and similitude relationships in the preceding section would apply in such studies. There are features involved in siphon elbows which require special mention.

21 Bradley, J. N., and Thompson, L. R., "Friction Factors for Large Conduits Flowing Full," Bureau of Reclamation Engineering Monograph No. 7.

22 Owen, T. G., "Developments in Design of Low-head Siphons and Diverging Chutes Resulting from Model Tests of Wasteway No. 2," Hydraulic Laboratory Report No. Hyd-108, March 1942.

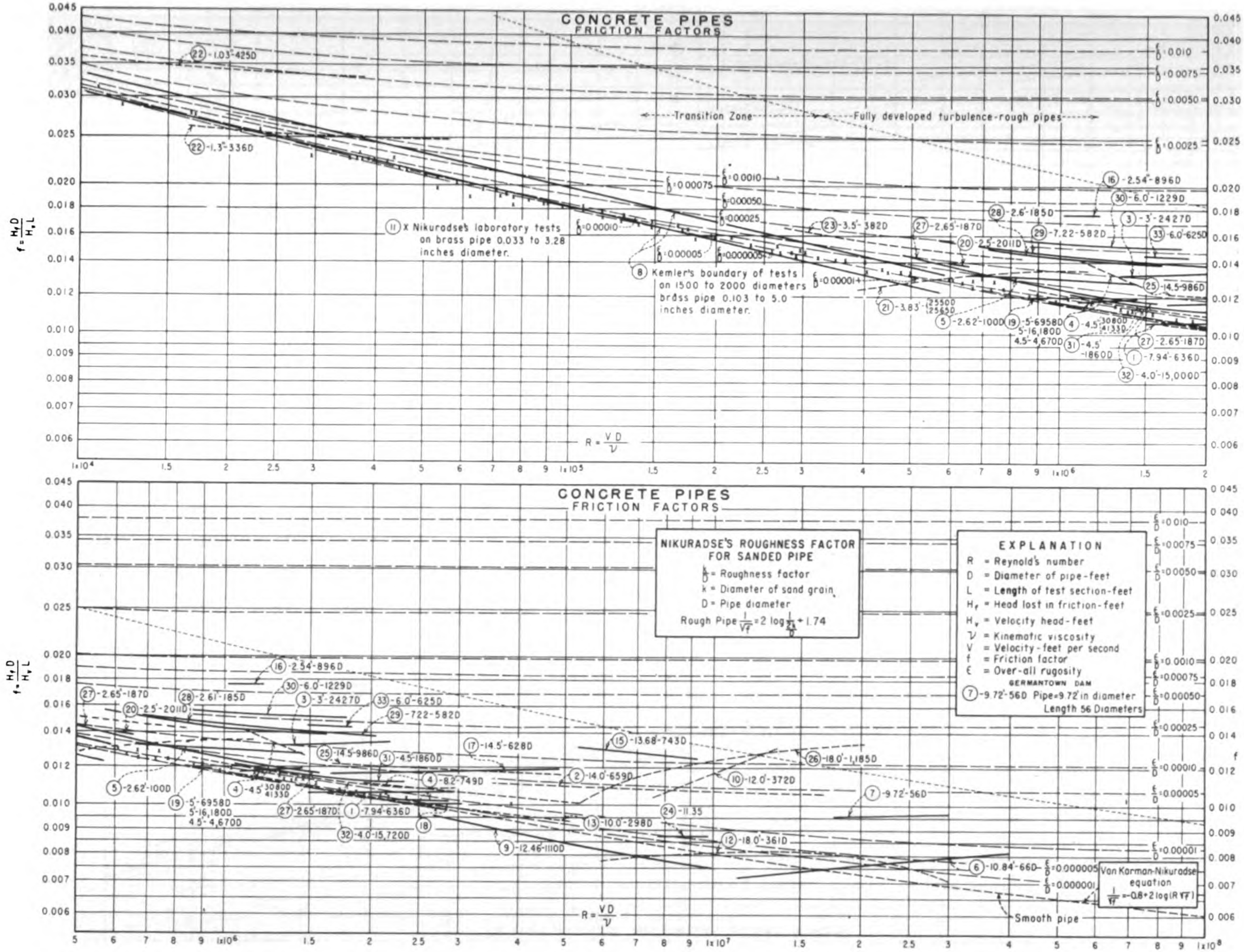


Figure 45. Friction factors for concrete pipe

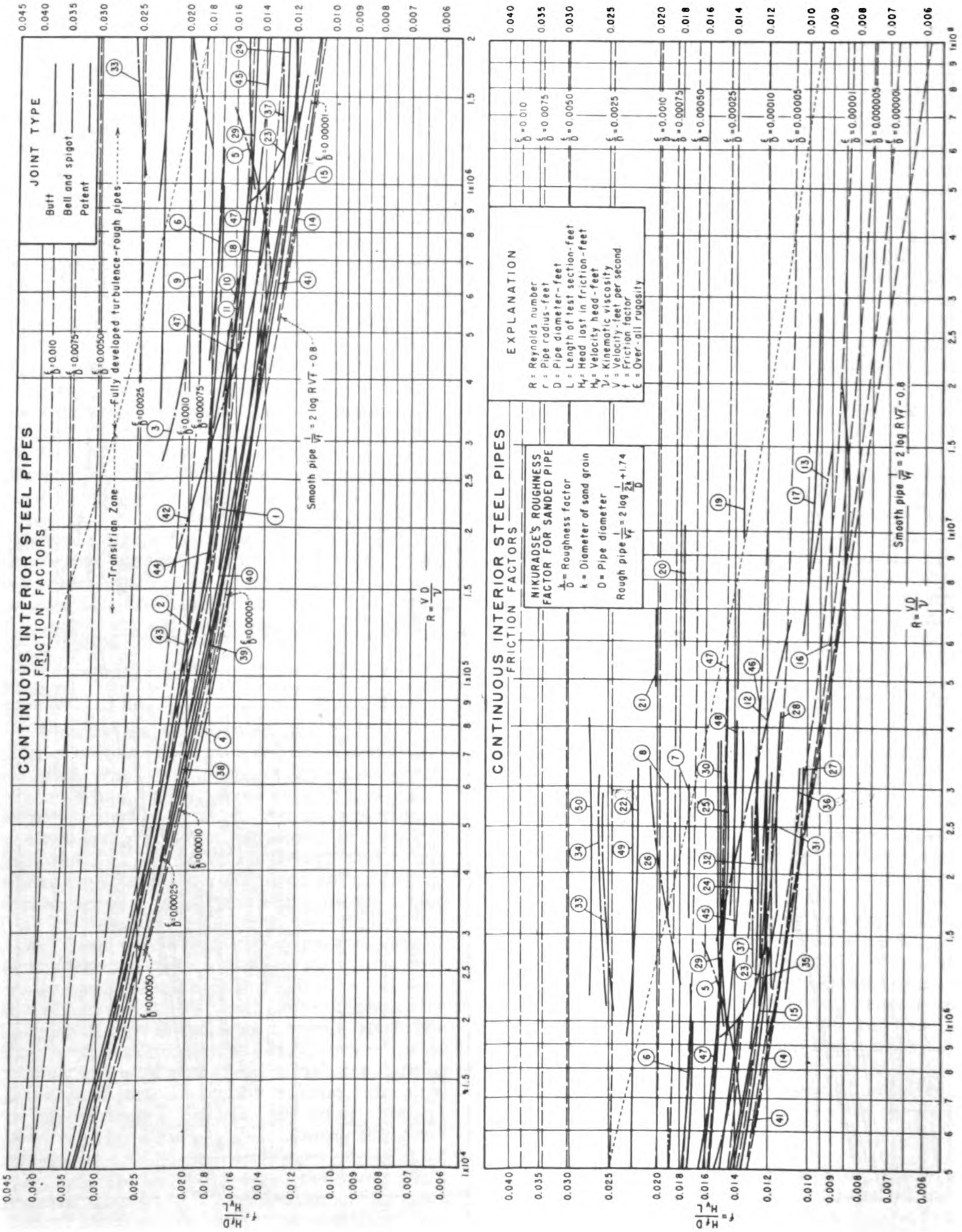
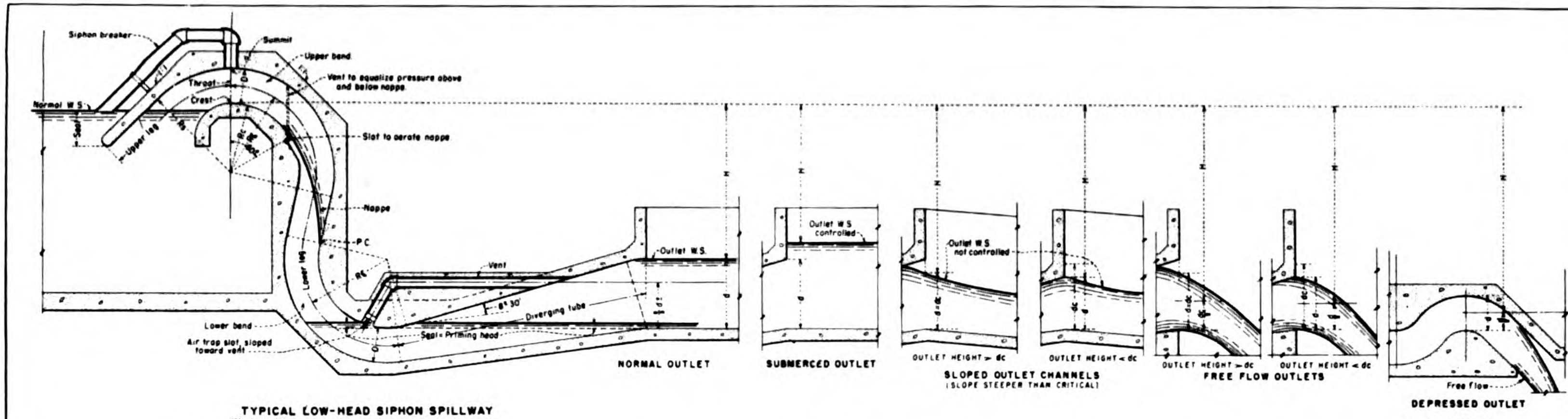


Figure 46. Friction factors for continuous interior steel pipe



TYPICAL LOW-HEAD SIPHON SPILLWAY

(Maximum head = atmospheric pressure equivalent at site)

DESIGN DATA

$q = CD\sqrt{2gH} \leq R_c\sqrt{0.7h(2g)} \times \log_e \frac{R_s}{R_c}$ = (Vortex flow—Max q)

q = Cubic ft per sec per ft of crest width
Rc = Radius of crest at throat in ft
Rs = Radius of summit at throat in ft
D = Throat size in ft
d = Depth of water at outlet in ft

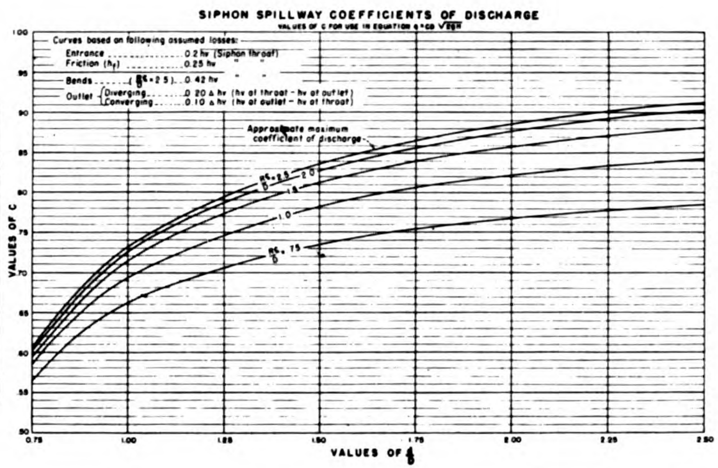
h = Atmospheric pressure at site in ft of water
H = Available head = (See outlet details)
C = Discharge coefficient based on $\frac{d}{D}$ and $\frac{R_c}{D}$
g = 32.16

SECTION	DESIGN	REASONS	REMARKS
Upper leg	A well proportioned transition providing a gradually contracting area. Inlet area should be 2 or 3 times the area of the throat.	Provides accelerating velocity. Reduces entrance losses.	Entrance loss for form shown suggested as 0.2 Δ hv
Throat	Size depends upon required capacity. Recommended ratio of Rc to D is 2.5. For forming with timber D is 2'-0" min.; metal forms left in place may be used for less than 2'-0". Recommended width of throat = 2 D.	Keeps bend losses low.	Wide crests and shallow depths reduce priming time.
Upper bend	Recommended ratio of Rc to D is 2.5. Aerating slot is set at 60° from crest. Pipe vent, set in sidewalls, connects slot to summit area.	Keeps bend losses low. Slot prevents nappe from clinging to crest. Vent insures nappe trajectory.	Area in upper and lower bend is same as area at throat.
Lower bend	P.C. is set to contact nappe. P.C. is a point on trajectory of nappe as determined from desired priming head at crest. Recommended ratio of Rc to D is 2.5. Top of siphon at vertical section is set at elevation of outlet bottom. Air trap elevation is elevation of outlet bottom plus desired priming head. Vent area = throat area ÷ 48 (approx.)	Provides seal and exhausts air from upper bend. Provides seal of lower bend and insures creation of partial vacuum. Vent exhausts entrained air with negligible loss of discharge.	Vent decreases priming time, increases priming head slightly.
Diverging tube	Outlet end set at desired elevation. Length of tube is set by 8°30' angle of divergence. Vent outlet set at ½ depth (±) of outlet at max. capacity.	Provides transition. Increases discharge.	Discharge loss for form shown suggested as 0.2 Δ hv
Siphon breaker	A pipe vent set at summit with inlet end set at or slightly below normal water surface. Area = throat area ÷ 24 (Min.)	To break siphonic action slowly; prevents severe vibration.	If practicable, adjustable inlet on siphon breaker is desirable.
Seal at intake	Lip of upper leg is submerged an arbitrary distance, but should be set deep enough to prevent excessive draw down.	Prevents "gulping" of air, and provides more normal action of siphon breaker.	

Note: Design recommendations given above are presumed to give minimum losses according to experiments. Deviation therefrom should include consideration of additional losses.

SIPHON SPILLWAY COEFFICIENTS OF DISCHARGE

VALUES OF C FOR USE IN EQUATION $q = CD\sqrt{2gH}$



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"Design of A High-Head Siphon Spillway" by Elmer Rock, A.S.C.E. Transactions, Vol 105, Page 1050
"A Study of Low-Head Siphons" by T.G. Owen - 1938
U.S.B.R. Data on Model Tests, Wasteway No 2 (Yakima Project) - 1942
A.S.C.E. Transactions - 1934, Page 986
A.S.C.E. Transactions, 1939, Page 1789
"Siphon Spillways" by A.H. Naylor - 1935

RECOMMENDED DESIGN PROCEDURE

1. Determine crest Elevation (Canal W.S. EL + Freeboard desired)
2. Determine head available (crest EL − W.S. EL at outlet)
3. Select preliminary values for D and Rc and solve for max q using Vortex flow formula: $q = R_c\sqrt{0.7h(2g)}\log_e\frac{R_s}{R_c}$ (Recommend D = 2'-0" Min. and $\frac{R_c}{D} = 2.5$)
4. Select values of d and H and obtain value of C from graph. Solve equation $q = CD\sqrt{2gH}$ (q must not be greater than value obtained in (3)).
5. Solve for required crest width = $\frac{Q}{q}$. Select crest width and number of siphons to be used for total Q and determine q for this crest width.
6. Repeat (3) and (4) until $q = CD\sqrt{2gH} \cong R_c\sqrt{0.7h(2g)}\log_e\frac{R_s}{R_c}$
7. Select priming head by determining the canal W.S. EL at which operation is desired. If priming head is less than $\frac{D}{2}$ locate P.C. of lower bend on trajectory of nappe.
8. Determine size and inlet EL of siphon breaker (Area of siphon breaker = Area of siphon throat ÷ 24).
9. Select and design priming aids to be used (Slot to aerate nappe, Vent to equalize pressure above and below nappe, and vent at lower bend).

103-D-46

Figure 47. Siphon nomenclature, Wasteway No. 2 model studies

When the pump discharge line empties into a canal, the transition from conduit to canal is accomplished partially, at least, in the siphon elbow. The diverging transition portion of the elbow influences both the priming action and the efficiency; its performance is checked carefully in a model study. Ability of the siphon to prime itself and to evacuate any air which might accumulate at the crown during operation should be investigated. Design and operation of a model would be based for the most part on the Froude similitude. However, to settle the question of whether or not the siphon action will be interrupted by an accumulation of air separating from the water as it passes through the low-pressure region, it might be necessary to arrange the model so that prototype pressures and velocities prevail. Such a condition may be arranged for a small portion of the elbow near the crown by using distorted siphon head and discharge in the model. A study of this type was made for the siphon elbows of the Grand Coulee Pumping Plant discharge lines.[23]

The Inverted Siphon

The term "inverted siphon" is a misnomer but it is applied to the case where a conduit dips to pass under a river or follows the ground surface across a valley. The inverted siphon does not require priming, as it is filled by gravity, and the flow of the water naturally replaces the air as the system is placed in operation. Inverted siphons may be parts of closed systems or parts of what are otherwise open canal systems. The flow capacity of a siphon running full, as affected by friction and exit and entrance conditions, is given by established hydraulic relationships for flow in a closed conduit. Laboratory studies involving models are seldom required.

Nevertheless, many problems may be experienced with siphons which are parts of canal systems. Where sections at entrance and exit run partially full, air entrainment in the water can be very troublesome. The entrance should be designed to give a minimum of turbulence. Flow should not plunge into the barrel in such a manner that it will entrain excessive air. If such action cannot be avoided, the velocity should be sufficiently high to carry the air through the barrel without segregation, or should be sufficiently low to allow the air to separate and discharge from the siphon at the upstream end. Difficulties can be avoided by designing so that there will not be pockets (high points) in the alinement where air might collect, as this action tends to reduce the siphon capacity and results in objectionable turbulence if quantities of air are released intermittently from the high point. If the entrance runs full and other parts of the siphon immediately downstream are operating with a free-water surface, sufficient venting should be provided to prevent reduced pressure at the water surface which might result in the siphon entrance channel operating alternately as a closed conduit and an open channel. Model tests on inverted siphons (Froude similitude) might include observation of the flow within the entrance, because the flow conditions affect the entrainment of air, the capacity of the system, and hydraulic losses of a local nature in various portions of the structure. If the lowest portion of the siphon barrel is considerably lower than the entrance and exit, ample drainage facilities for the high head conditions are usually necessary. Such drain facilities are known as blowoff structures. Hydraulic tests of drains for inverted siphons are discussed in a laboratory report.[24]

Power Penstocks

In power penstocks it is important that as much of the total head as possible be conserved for power generation. Losses within the system must therefore be held to an economical minimum. This might be accomplished by using a minimum length of penstock, or a large penstock, or by streamlining parts of the penstock. The length of the penstock is usually governed by the relative locations of the dam and powerhouse; and is fixed before the problem reaches the laboratory. In most cases, the same is true of the penstock size, for the most economical size can be computed by the designer. The laboratory study therefore involves the streamlining of parts of the penstock to obtain an optimum design and may consist of ascertaining the relative merits of two or more proposed treatments.

23 Tessitor, F., and Hebert, D. J., "Hydraulic Model Studies of a Siphon Elbow Proposed for the Grand Coulee Pumping Plant Discharge Lines," March 1945. Kotz, S. E., and Simmons, Jr., W. P., "Hydraulic Model Studies of the Siphon and Feeder Canal Transition for Grand Coulee Pumping Plant," December 1946, Hydraulic Laboratory Reports No. Hyd-163 and Hyd-224, respectively.

24 Colgate, D., "Hydraulic Model Studies of the Stilling Well for the Blow-off Structure--Soap Lake Siphon," Hydraulic Laboratory Report No. Hyd-277, April 1950.

The main factors to be considered are entrance shape, alinement of the conduit, type and abruptness of bends, influence of branches, rate of change of area for expanding or converging sections, and the influence of obstructions such as the trashrack structure over the entrance, the coaster gate supports at the entrance, or a butterfly valve within the line.

1. Entrance shape. --Velocities in penstocks are usually less than 15 feet per second, thus the shape of the entrance is not as important as in outlet conduits where higher velocities exist. It is desirable, however, to flare the entrance sufficiently to decrease the loss to an economical minimum. However, the size of the entrance opening should be as small as practicable to minimize the cost of the closure or bulkhead gate. The entrance cross section may be circular, or may be rectangular or square with a transition to a circular conduit. Pressure taps or piezometers are installed in a model at points within the entrance and downstream from it to permit measurement of pressure gradient and losses resulting from various shapes or degrees of streamlining. Abrupt changes in pressure gradient are indicative of irregularities of the flow pattern and usually are accompanied by excessive head loss. A pressure gradient which changes gradually is the most desirable. Optimum design is a compromise between good hydraulic efficiency and practical considerations in design and construction.

2. Trashrack loss. --The purpose of the trashrack structure is to prevent waterlogged pieces of wood and other foreign material from entering the system and damaging the turbine runner, also to prevent clogging of valves. Laboratory studies usually involve determination of losses for special trashrack structures of two or more proposed designs.

Another common problem in the design of trashracks is the influence of the structure itself on pressure distribution within the penstock entrance. Of particular concern in this respect is the nearness of the bottom support of the trashrack to the penstock opening. This factor becomes increasingly important as the velocities through the conduit increase; thus it is of more concern in outlet conduits than in power penstocks. The same consideration applies to obstructions near the entrance such as gate grooves, supports, etc.

3. Emergency control. --If an emergency shut-off device, such as a butterfly valve, is to be placed in the penstock, it is important to know the head loss for normal operation. Again, piezometers and their locations play important roles. It is desirable to use geometric models and test them under heads of such height that the Reynolds number approaches that for the full-size structure.

4. Dual use of penstock. --Many large structures must serve a dual purpose. A penstock may be a temporary diversion tunnel during construction, operating as an open channel. The problem will not be complicated unless there are expanding or contracting sections involved, or the exit end is somewhat lower than the entrance. In such cases, design of these sections should be governed by the more critical condition. A gradual transition for the closed conduit flow may be a discontinuity for the open channel condition. The pressure gradient through the system should be investigated if the downstream end of the conduit is substantially lower than the entrance. In many cases, the gradient must be raised by placing a temporary cone or restriction at the downstream end of the conduit.

5. Water-hammer. --Vibration and water-hammer are often factors of such importance that study of transient phenomena is warranted. Closing or opening a gate or valve too rapidly might induce dangerous pressure surges. Studies by analytical or graphical methods are usually adequate; only in special cases are model studies required. Model studies of such problems involve application of the Mach similitude and use of special electronic equipment. Water-hammer problems are so diversified in type that it is impractical to attempt their discussion in this text.

Outlet Conduits

The function of an outlet conduit is to release water from the reservoir behind a dam. The conduit may pass through the dam or around it through an abutment to the river downstream. The efficiency of the conduit in this case is of secondary importance, since it is not necessary to conserve head. Of primary importance is safe handling of the required velocities and discharge. Streamlining is important because, with the high velocities involved, there is danger that cavitation will be induced by irregularities or discontinuities in the flow boundary. An outlet consists of several parts: a trashrack structure, a streamlined entrance, a main conduit, a gate, valve, or other control device (which can be either at the entrance, in the line, or at the end of the conduit), and an exit from which the water discharges into a stilling pool. Testing procedures related to the stilling pool were discussed under Open Channel Flow, Stilling Pools.

1. Streamlining. --There is a decrease in the pressure in a conduit as the velocity through it is increased by opening the control gate or valve. The pressure is reduced further if the exit of the conduit is lower than the entrance. Pressure may be decreased locally if there is an abrupt change in boundary alinement. It is possible for the pressures at certain points to reach the vapor tension of the flowing fluid, thereby causing cavitation and possibly the severe pitting known as cavitation erosion. The reasons for streamlining outlet conduits and penstocks differ widely. For the conduits, the important consideration is avoidance of cavitation; for the penstocks, it is imperative that head loss be minimized.

2. Trashrack. --Loss through the trashrack is of importance; not from the standpoint of conservation of head, but because it affects pressure distribution in the entrance. The relative positions of the bottom of the trashrack and the bottom edge of the outlet are of particular significance.

3. Influence of gate grooves. --Gate grooves or obstructions on the face of the dam near the entrance or at some point within the conduit may change the flow pattern sufficiently to induce troublesome subatmospheric pressures and such possibilities should be investigated thoroughly.

4. Entrance shape. --The shape of the entrance may have a critical effect on pressure distribution, and should be so proportioned that near-atmospheric pressures prevail for any head. Standard entrance shapes have been developed, by general model studies, for circular outlets which are normal to the face of the dam. One such shape (a type of "bellmouth") is the surface generated by revolving a quadrant of an ellipse about the axis of the circular outlet. The semimajor axis of the ellipse is 0.5 of the outlet diameter, and the semiminor axis is 0.15 of the outlet diameter. The position of the elliptical quadrant is such that the ellipse, at the end of its semimajor axis, is tangent to the face of the dam, and, at the end of its semiminor axis, is tangent to the periphery of the circular outlet. If the shape is not one of the standards, the pressure distribution in the entrance should be thoroughly investigated at the highest velocity available in the laboratory. Although some work has been done to establish standard shapes for square and rectangular entrances[25] the pressure distribution in these shapes should be studied.

5. Relative elevations of entrance and exit. --The exit end of the conduit should not be placed too low with respect to the entrance, particularly if the area of the flow passage is constant throughout its length. If the conduit is constricted near the downstream end, a lower exit may be used. A bellmouth entrance designed for zero pressure will be subjected to lower pressures if this difference in elevation is increased. If the drop is sufficient to cause vapor tension at the entrance, there will be cavitation and thus severe pitting (cavitation erosion).

6. Alinement of conduit. --The pressure conditions associated with abrupt changes in boundary alinement or short-radius curvatures should be investigated to insure that there are no negative pressures which might lead to cavitation erosion in the prototype.

7. Pipe roughness. --Though it is seldom of such importance as to require special studies, the roughness of the model conduit should usually be as smooth as possible. More often than not, it will be impossible to make the model roughness to scale. It is sometimes necessary to shorten the model conduit to eliminate excessive friction losses and represent true pressure conditions in the model system. Losses in excess of those expected for the prototype might in some cases be overcome by increasing the head to offset them. Computations[21] are usually satisfactory for this purpose.

8. Conduit control. --Flow through a conduit may be controlled by any of several devices which might be located at the entrance, in the line, or at the downstream end of the system. The device may be adjustable or nonadjustable depending on the requirements downstream. Adequate aeration immediately downstream from the control is essential to prevention of cavitation and vibration, particularly if the control is to be used for regulation. Aeration is less important if operation is in the open or closed position only; however, it is desirable to provide sufficient air to minimize vibration during the opening and closing cycles. Care must be taken to investigate thoroughly the pressure distribution and intensity within the conduit system which results directly from the control device. Its influence on the capacity is of importance also. A dis-

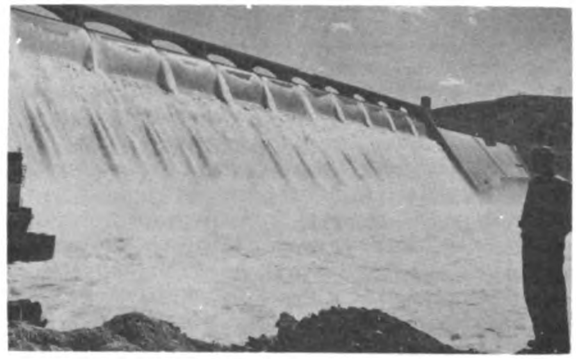
[25] Thomas, H. A., "Design of Bellmouths for Entrances to Conduits of Circular, Square, and Rectangular Cross-sections," Carnegie Institute of Technology, July 1946.

cussion on controlling devices, including types of gates and valves, is contained in subsequent parts of this discussion of closed conduit flow.

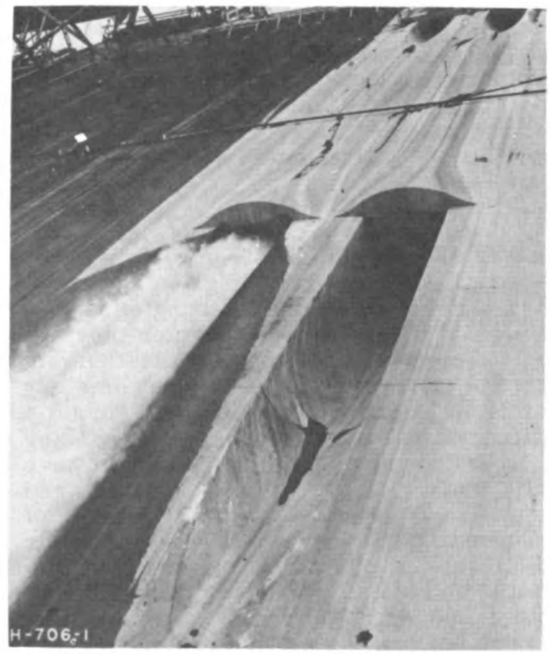
9. Conduit exit shape.--The design of the exit end of an outlet conduit is governed by the disposition of the water released from the system. If spray is a consideration and the conduit terminates at the downstream face of the dam high above the tail-water surface, a special bend may be installed near the downstream face, together with a trough on the face. The bend to direct the water down the face of a dam results in an exit that is lower than the main portion of the conduit. A cone is usually placed at the end of the outlet to offset the pressure reduction caused by the difference in elevation. Pressure conditions in the immediate vicinity of the intersection of the trough with the face of the dam, particularly downstream from it, are of concern. The pressures should not be so largely negative as to induce cavitation or cause instability in the flowing jet. The cross-sectional shape of the trough is designed to avoid large fins of water which tend to form at the walls. An effective shape for a circular exit is one with parallel sides spaced one exit diameter apart and having a semicircular invert with a radius equal to that of the exit opening. The exits for the river outlets at Shasta and Grand Coulee Dams, Figure 48B, are examples. This design is particularly appropriate for outlets which do not serve as regulators or for those which are used only in opened or closed positions. If the conduit is of a horseshoe cross section with a flat bottom, a trough having vertical sidewalls is more desirable. This shape is particularly suitable in cases where regulation of the flow quantity is required. Aeration of the vertical curve near the exit, for operation at partial capacity, is facilitated by this shape.

10. Effect of spillway flow on conduit exit.--Where the outlets terminate in the face of a dam having an overflow spillway, water from the spillway must flow over the exits. A study of pressure conditions and flow action at an exit is essential where the spillway and outlets are to operate either simultaneously or singly. The action of the spillway flow at the exit, when water is not being released through the outlet, is of particular concern. Spillway flow should not be allowed to impinge within the trough of the outlet, for such action produces considerable spray and might result in critical local pressure conditions (Figure 48A).

11. Deflectors over outlet trough.--It is important that there be proper aeration of the outlet conduit (usually by large vents placed immediately downstream from the control), so that the subatmospheric pressure in the conduit will not materially lower the trajectory of the spillway stream as it passes from the upper edge of the trough to the lower edge. However, this alone will not eliminate impingement of the spillway stream on the bottom surface of the trough. A flow deflector immediately upstream from the trough raises the trajectory sufficiently to cause the flow to clear the opening and prevent impingement, thereby reducing undesirable spray and splash to a minimum. Shape of the deflector may be worked out in the model, as in the designs for Grand Coulee and Shasta Dams.



A. Spillway flow at Grand Coulee Dam passing over outlet deflectors



B. Deflectors at upper ends of Shasta Dam outlet exit troughs

Figure 48. Outlet exits in face of overflow spillway

Controls for Closed Conduit Flow

A control is defined as any device used to restrict or stop the flow through a conduit by closing the passageway. For the purpose of this discussion, controls will be classified in four general groups, in terms of the movements of their control elements, as follows: (1) those having a plunger or needle for control, such as needle and tube valves, in which a needle or tube moves in or against the direction of flow to contact a seat and stop the flow; (2) those in which the flow is cut off by a sliding leaf moving across a seat that is perpendicular to the axis of the pipe; (3) those in which a plug, sphere, or rotating leaf controls the flow, such as plug valves, sphere valves, and butterfly valves. All contain rotating elements which permit free passage in one position, but decrease the flow at others; and (4) those which act to check backflow in a line, such as flap or check valves.

1. Requirements of controls.--Regardless of the type, the basic problems in the design of control valves or gates are similar. They must be built strong enough to withstand the pressures to which they are to be subjected. The operating mechanism must be powerful enough to overcome any frictional drag or unbalanced hydraulic forces which may occur during the opening and closing movements. They must have sufficient capacity to release the required amount of water and at the same time be of economical construction. They must be able to operate through the required range of heads without damage by cavitation, either to the devices themselves or to the downstream conduits and, if special circumstances require, the devices must be able to operate at various openings free of cavitation and with a minimum of vibration. Also, adequate seals must be provided to assure a minimum of leakage.

2. Laboratory tests.--In the development of large valves or gates for high-head installations, where there is no precedent upon which to base design, it is desirable to study many of the hydraulic characteristics by conducting tests on scale models. The studies include evaluation of the unbalanced forces which occur on moving parts during the operating cycle; measurement of discharge capacities; recording of pressure intensities, paying particular attention to severe subatmospheric conditions which might induce cavitation; reshaping of questionable flow passageways to prevent cavitation; provision of adequate air relief at points where critical low pressures are certain to exist; measurement of torque of rotating elements; and study of the adequacy and mechanics of seals. Vibration of a system when a valve or gate in the line is operated at partial openings is of concern. In some cases an enormous amount of energy must be dissipated in such action. Very little has been done to isolate the variables and their contributions to vibration.

3. Force and torque measurements.--Forces in hydraulic model testing are measured directly and indirectly. The direct method is that of weighing or using some type of force-recording facility which has been calibrated to some standard. There are so many varieties of apparatus that only the principles upon which some operate will be mentioned here. Some use the displacement of a spring to indicate the force involved, some the balancing of force action on levers. Some depend on the elongation of a metal bar, rod, or other shape, and some are based on the relationship of torsional displacement of a metal rod or other structural shape. The indirect method of measuring force is by integration of the pressure distribution curves for the surface or portion of the structure under study. Both methods are very useful in evaluating the forces acting on the moving portions of outlet control devices and the torque to be overcome in the operation of these devices.

Force or torque on an operating mechanism or rotating part is usually measured by weighing. The arrangement may contain a lever or system of levers containing a commercial type of scale or improvised or specially designed apparatus. The nature of the arrangement should be such that the frictional resistance in the model will be evaluated, or eliminated from the final results.

4. Leakage.--Sealing of gates or valves is important, particularly under high heads where flows through leakage passages are at high velocity, for such action induces what is known as "wire drawing" and damage to the contact surfaces is a certainty. Sealing may be by direct contact of the moving leaf or needle with the stationary body or housing or by a unit which flexes or extends from a recess to form the seal after closure. If the model used for a general study of the problem is too small to include details of the sealing mechanism, it may be necessary to study a separate model of the seal. This model may include the entire seal or a small portion of the seal, depending on the nature of the problem. Seals for gates and valves are discussed in detail in another section of this monograph.

5. Discharge measurements.--Regulating and controlling devices for closed conduit

flow are compared in terms of their abilities to discharge fluids under a given head. The head-versus-discharge curve is useful in this respect. However, in many cases other parameters are used, the choice depending on the nature of the problem and method preferred. The relation between coefficient of discharge and head is most popular. Such a parameter should be used with caution since its magnitude can be made to vary with the dimensions used in its computation and thus it may not be a true basis for comparison. For example, the coefficient of discharge might be obtained by using an area based on the exit diameter of a valve, or on the inlet diameter, the conduit diameter, or any dimension of the flow passage. Thus the coefficient might be made to vary widely for controls having the same capacity. The relationship $Q = CA\sqrt{2gH}$ is used. Here "C" is the coefficient of discharge, which varies with the area "A" and the head "H." Customary practice is to use the area based on the inlet diameter of the control and the total head (pressure head plus velocity head) at a point one diameter upstream from the inlet, and to measure the discharge quantity. Friction has only a minor influence in conduit controls and is neglected in the model tests. Capacity curves or tables, from which the amount of water discharged from the full-size structure can be ascertained with a high degree of accuracy, are prepared from the model data. These curves or tables may be used in prototype operation.

Classification of Controls

Though all outlet controls serve a similar purpose, their mechanical and hydraulic characteristics may differ widely in many respects. For this reason, descriptions of the various types of controls and their outstanding characteristics and peculiarities will be presented.

1. Needle valves in outlets.--Several types of needle valves have been used for high-head outlets.[26] The early types had curved surfaces at the nozzle exits and diverging passages between the needles and nozzles at certain degrees of opening. Divergence in the water passage, which conformed to the constant area flow-passage criterion, helped to maintain a high rate of discharge at full valve opening but induced severe subatmospheric pressures when the valve was operated under high heads. Cavitation was the result. Because of cavitation in these designs, particularly at partial openings, it has become standard practice to eliminate curved surfaces in the nozzles and divergence in the flow passages between the needles and nozzles. The jet from the needle valve is solid in appearance and changes in diameter with the valve opening. It is very stable unless considerable cavitation is present near the exit. In this case, the alternate increase and decrease of pressures near the cavitation zone may induce an unstable condition, and cause fluttering of the jet and intermittent spurts of spray from the jet surface immediately downstream from the valve. Elimination of cavitation, with minimum alteration in raising the pressures to acceptable values, restores flow stability. The hydraulic characteristics of a cavitation-free needle valve are shown on Figure 49.

2. Tube valve.--The tube valve has evolved from a simplification of the needle valve. It is essentially a needle valve with the downstream needle tip removed, thus making a tube of the closing element and decreasing the weight slightly as compared to that of a needle valve. The hydraulic characteristics are similar to those for the needle valve, but the jet, though not so variable in size for various openings, becomes unstable at small openings, and tends to flutter and disintegrate into a cloud of spray.

Special requirements sometimes necessitate development of special tube valve designs. An in-the-line installation of a tube valve as a regulating device is an example. High capacity for a given diameter is a requisite in this case, thus the flow passage must interfere as little as possible with the moving fluid. A larger diameter or longer valve is required to attain increased capacity. Electric analogy studies are helpful in determining the initial degree of streamlining of the flow passage, but care must be exercised in analyzing the data and attempting to carry the study beyond the limitation of the apparatus.

When any valve is placed in a line as a regulating device, there must be provision for aerating the jet as it leaves the valve and discharges into the conduit downstream. Otherwise, severe subatmospheric pressures will be present and may possibly cause cavitation and damage to the conduit walls. The size of the vent duct should be sufficient to transport the air at a velocity low enough to prevent whistling. Abrupt bends and sharp entrance shapes should be avoided; they may induce local regions of high velocity and produce objectionable noise. Field and laboratory investigations

[26] Hebert, D. J., and Ball, J. W., "The Development of High Head Outlet Valves," Appendix 14, Report on Second Meeting, International Association for Hydraulic Structures Research, Stockholm, June 6-7, 1948.

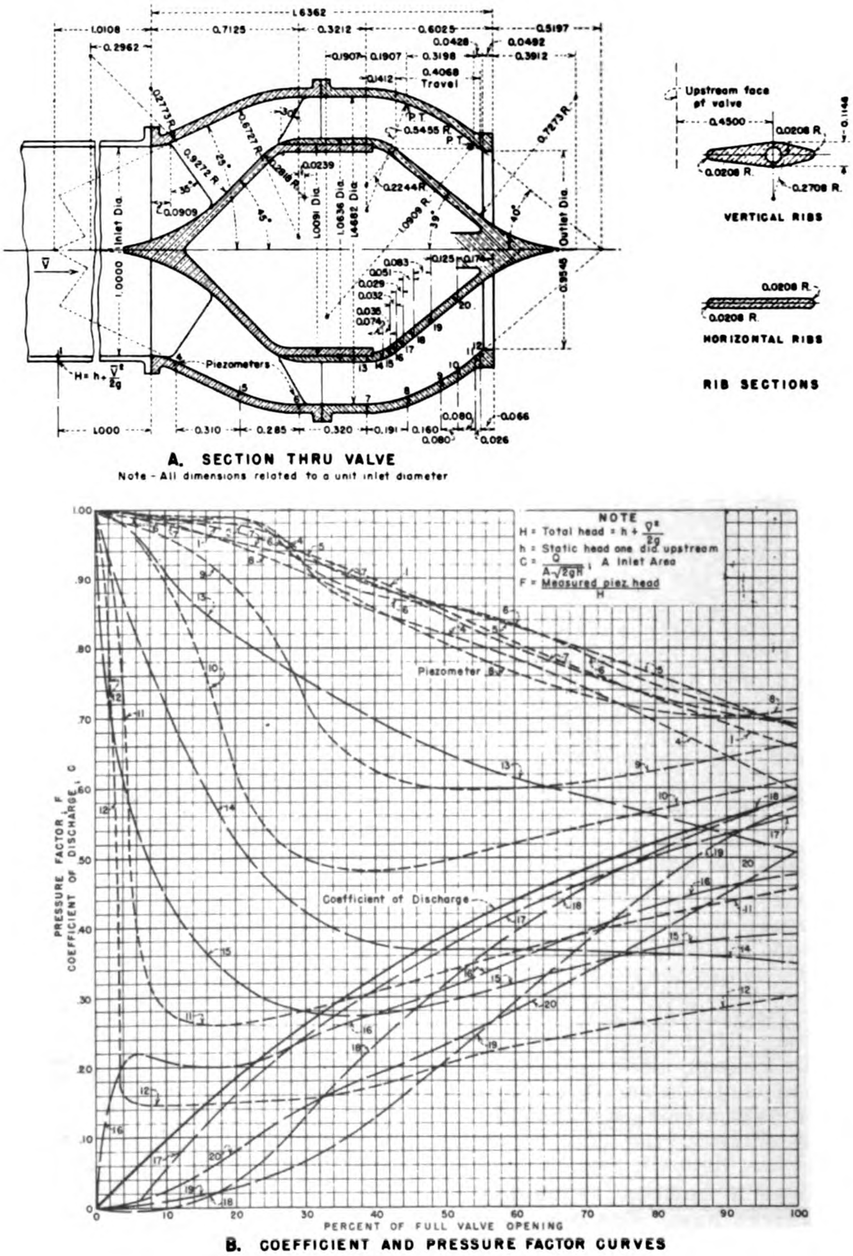


Figure 49. Friant Dam needle valve, 6-inch model

indicate that the air demand for the full-size structure can be determined through use of model data. The hydraulic characteristics of a cavitation-free tube valve are shown on Figure 50.

3. Hollow-jet valve.--The hollow-jet valve is a further simplification of the needle valve. It was developed to attain a greater reduction in weight and thus decrease the cost of a regulating valve for control at the end of a line. The hollow-jet valve resembles the downstream end of the needle valve except that the direction of flow is reversed. The discharging jet is a hollow cylinder and the control element moves against the flow to close. As in any valve, the shape of the flow passage is of major importance since it influences the pressure gradient which might in certain cases be lowered sufficiently to induce cavitation. For the same size, this valve will handle as much as 35 percent more water than a needle valve. Another advantage is

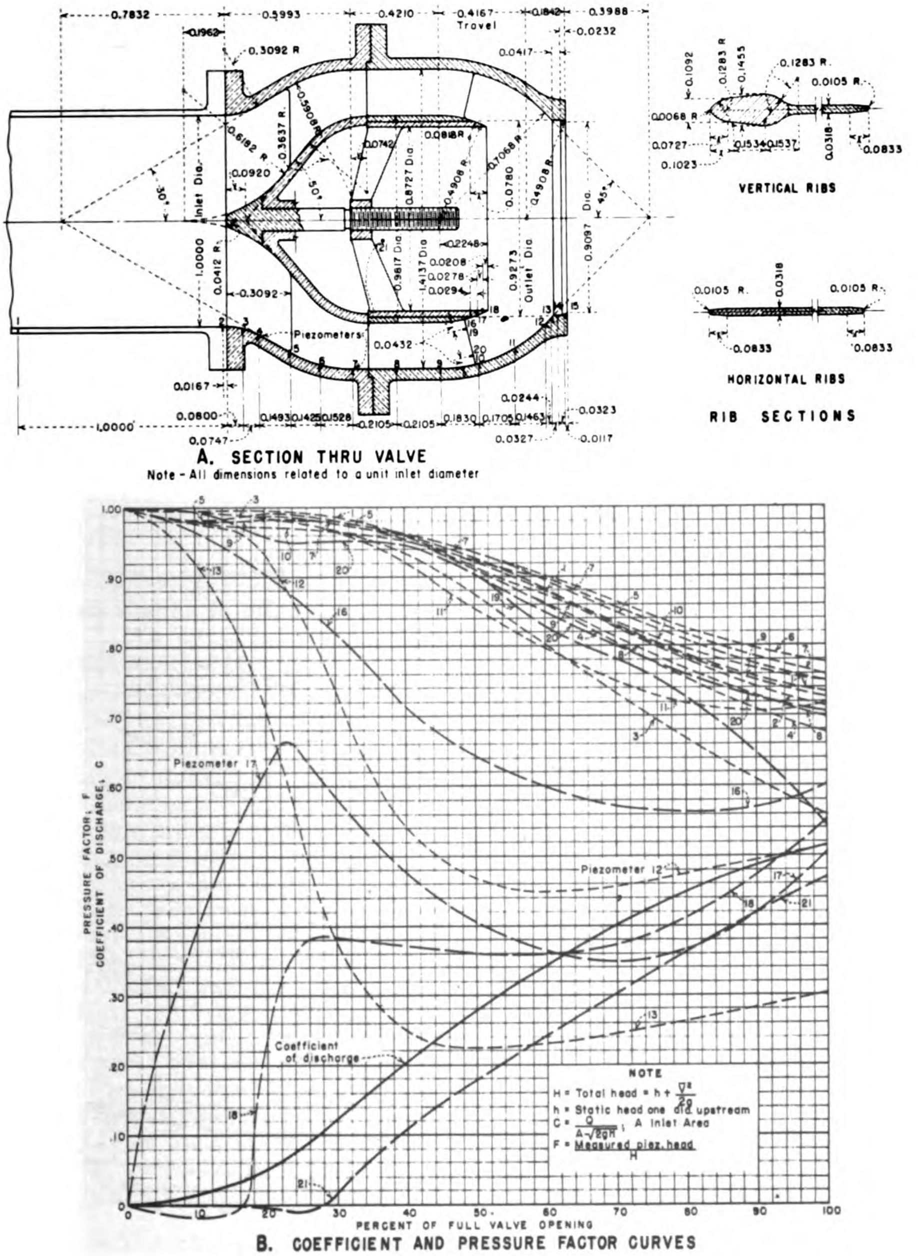


Figure 50. Friant Dam tube valve, 6-inch model

that the body is not subjected to full reservoir head at any time, thus the body is of much lighter construction than that of the tube valve or the needle valve. The stream from the hollow-jet valve, unlike that from the tube or needle valve, has a constant outside diameter, so the flow is distributed over a wider area and dissipation of energy is facilitated, particularly at small valve openings. Pressure conditions, capacity, and ease of operation are the main factors considered in model tests. The hydraulic characteristics of a hollow-jet valve are shown on Figure 51.

4. Butterfly valves. --The butterfly valve derives its name from the shape of its shutoff element. It is essentially a flat circular leaf that rotates about a diametrical axis in a short piece of pipe to attain regulation. It is used extensively for emergency closure of power penstocks, but has served in a few instances as a regulating device at the end of an outlet conduit. When used for emergency

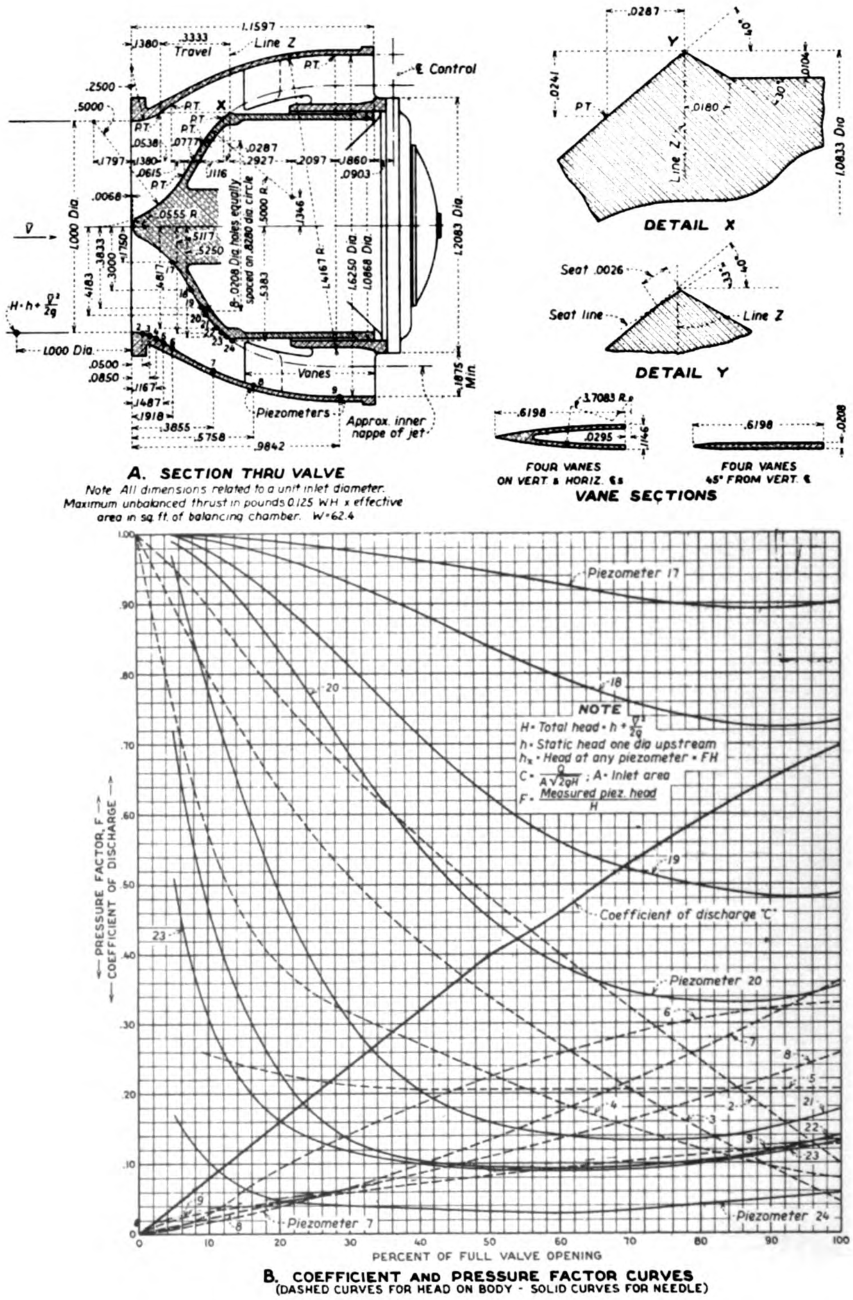


Figure 51. Hollow-jet valve, 6-inch model, for Anderson Ranch Dam

closure, the valve remains either in the wide open or closed position and never at partial openings. At partial openings, the obstruction to flow and the nature of the water passage is such that severe subatmospheric pressures, and thus cavitation, are likely to occur if high velocities are present. The shape of the leaf is very important for either type of use. When a butterfly valve is used in a penstock the hydraulic losses should be a minimum for the fully open position. When the valve is located at the end of the line as a regulator, the design should be such that severe subatmospheric pressures will be limited to a small operating range or eliminated. In many commercial designs, the leaf is so shaped that cavitation occurs under high heads when the valve is at or near the wide-open position. Proper shaping of the leaf

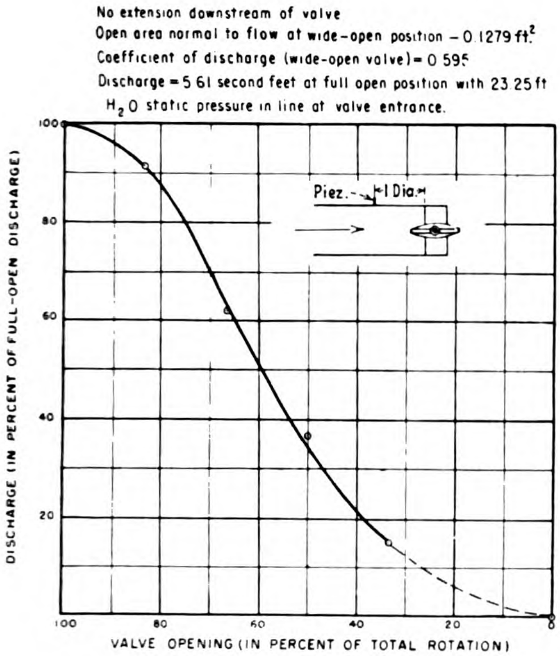
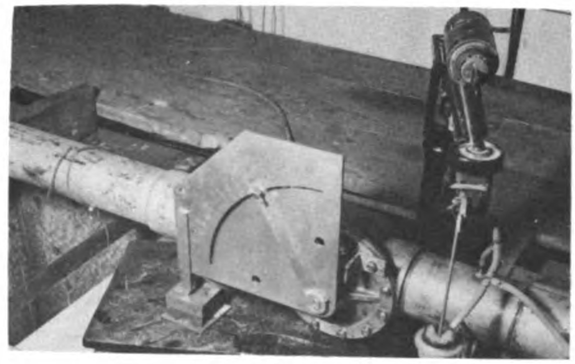


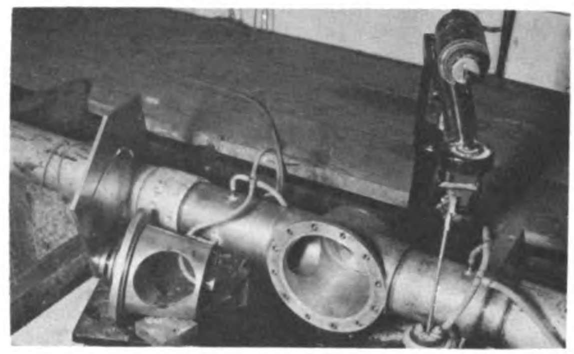
Figure 52. Free-discharge characteristics of a 6-inch butterfly valve

should eliminate this condition. The coefficient of discharge for a butterfly valve, based on the pipe diameter and the total head one diameter upstream from the valve is approximately 0.60 for the free-discharge condition. For the in-the-line arrangement the coefficient is variable since it is influenced by the pressure characteristics of the conduit immediately downstream from the valve. Though simplicity in construction is indicated for all parts of this valve, some difficulty has been experienced in attaining adequate sealing. This trouble arises mainly from the fact that the water acting on one-half of the leaf surface tends to deflect the leaf away from the sealing surface. The free-discharge characteristics of a 6-inch butterfly valve are shown on Figure 52.

5. Plug valves.--The plug valve also derives its name from a distinctive characteristic. The valve is composed of a cone-shaped plug which fits snugly into a housing connected to two or more pipes. If the plug is to act as a shut-off in a single line, a single hole perpendicular to the axis of the plug passes through it to make a continuous passage when the system is operating. A model of a typical plug valve is shown on Figure 53. These valves may also be of three- or four-way construction. The plug-type valve is used extensively in small lines or where three- or four-way installations are necessary. Generally, they are not used for regulating because the water passage for partial opening is conducive to formation of zones of subatmospheric pressure



A. Test assembly showing torque measuring arm



B. View showing plug valve and body

Figure 53. Typical 4-inch plug valve

and cavitation. However, they have been used satisfactorily in combination with orifices to reduce and regulate pressures. The torque tending to rotate the plug during the closing or opening cycle varies widely, reaching a relatively high maximum when the plug approaches the wide-open position.

6. Sphere valves.--The sphere valve derives its name from the spherical shape of the control element. Its construction is similar to that of the plug valve and it functions in the same manner. The hydraulic characteristics of this valve are similar to those of the plug valve and any studies pertaining to it will be of the same nature.

7. Check valves and flap gates.--The term check valve originates from the function performed by the device. Generally, this valve checks the flow in one direction and allows flow in the opposite direction. There are many types of check valves, but the most common has a hinged disc or flap for the checking element which is closed automatically by any tendency for the flow to reverse direction. Another common type is that in which the moving element is con-

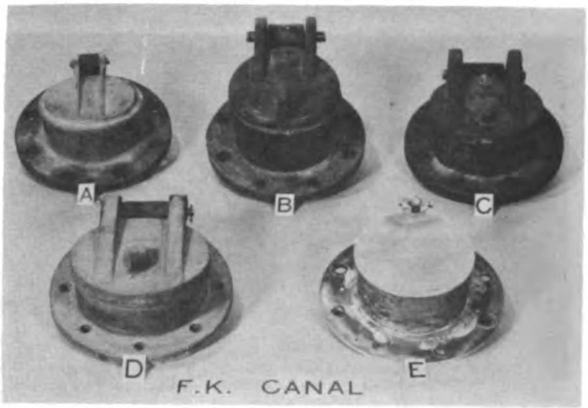


Figure 54. Flap gates

tained in guides and seats on a sealing surface when reversal of flow tends to occur. The moving element may be spherical, cylindrical, or of other shape. Check valves of this type usually are small. Design of the water passage around the check element is very important. Changes in the flow passage shape, resulting from the movement of the element, may induce pressure changes which can cause rapid oscillating motion of the element, and thus produce pulsating flow and vibration. Cavitation is likely to occur if the passage is not designed to eliminate severe subatmospheric pressures. The term check valve is applied to both types when they are used in a pipe line; the term flap gate refers to the hinged flap or disc type when it is at the end of the line.

Several flap gates used in irrigation work are shown in Figure 54. This type of device is used at the exits of pump discharge lines to prevent backflow when there is an interruption of power to the pump units. Also, they are placed in canals or drains for permitting excess water to escape from under the canal lining and thus prevent rupture of the lining. The outstanding disadvantage of the flap gate at the end of a pump discharge line is that it slams shut with intense shock by hydraulic action when an interruption of power stops the discharge. In some instances, this shock has been so severe that the safety of the structure has been endangered, and in others, the gate itself has been damaged. This has been particularly true where the pump line is designed to retain its prime for shut-down periods and provision cannot be made for aerating the pipe near the exit of the discharge line when the pump is stopped. The intensity and nature of the slam are, therefore, important considerations when the flap valve is being investigated by scale models. The hydraulic action at the gate, due to the backflow of water, is the major factor contributing to the shock. The weight of the gate alone contributes very little to the shock intensity. Three distinctly different methods for reducing the shock have given various degrees of success. A dashpot, very effective on small sizes, may be used to absorb the shock. Forced closure of the gate before reversal of flow in the system may be used quite effectively. Injection of compressed air in sufficient quantity at the proper instant and under adequate pressure is considered effective. In addition to decreasing the closing speed of the gate, the injected air provides a cushioning action as the water in the line tends to flow backward through the pump. Injection should take place just ahead of the gate closure. The relative times at which power interruption, reversal of flow, and closure of the gate occur is important. Other factors which must be considered include: pressure intensities at various points in the system, particularly on the pump side of the gate; intensity of the shock, which is extremely difficult to measure and in most cases must be assumed to be proportional to a measured movement of the structure or some part thereof; the position of the gate at any instant during the closing cycle; and the angular velocity or speed of the gate at any point in the closing cycle. Special electronic instrumentation, with an oscillograph for recording the events, is a requisite. Records should be taken simultaneously and continuously during the operating cycle. Care must be exercised to assure that some peculiarity of the model does not influence the record so as to result in erroneous analysis of the problem.

8. <u>Slide gates</u>.--The slide gate is used occasionally to control the flow through high-head conduit systems. Guides, wheels or rolls, and tracks are provided in many cases to reduce sliding friction, particularly when both the conduit and total force are large. Most slide or gate valves should not be operated at partial openings. The leaf should remain in the closed or fully open position except for the opening or closing cycle, unless the gate is designed especially for regulation. Use of the slide gate for regulation necessitates aeration of the conduit immediately downstream and special treatment of parts of the housing or gatewell, unless the entire system is under sufficient pressure to prevent development of vapor pressure at any point.

Because of the numerous restrictions which must apply when the slide gate is used for control of high-head conduits, many types of gate valves have been developed. Some of them will be described here.

a. <u>Ring-follower gate</u>.--The most efficient outlet conduit is one having continuous flow boundaries and no obstructions. A control known as a ring-follower gate has been developed for use in circular conduits where regulation of discharge is not required and where high efficiency and smooth flow without subatmospheric pressures are requisites. The leaf of this gate moves perpendicular to the axis of the conduit. One end is solid to shut off the flow and the other end has a hole through it of the same diameter as the conduit so that a continuous passage is formed for the wide-open condition. The name of this gate originates from the fact that the ring, or the end of the leaf with the opening in it, follows the shut-off portion of the leaf to form the continuous water passage for the fully open condition. The Paradox gate and the ring-seal gates are variations of this type in which the sealing devices differ materially. The ring-follower gate should not be used for regulation, particularly for high heads. The sharp corners of the opening through the leaf and the housing are conducive to cavitation, and it is difficult to aerate the gate satisfactorily. Also, there is possibility of destructive vibration if this type of gate is operated at partial openings for long periods. The opening in the gate leaf should coincide with the conduit to form a continuous flow passage when the gate is in the wide-open position. The hydraulic downpull on the leaf of a ring-follower gate is important since the design of the operating mechanism is dependent on the magnitude of this force. Pressure intensities within the gate housing, the head loss due to the gate, the type and adequacy of the seal, and the aeration downstream, for normal operation, are factors to be considered in hydraulic model investigations.

b. Regulating leaf gates. --The ordinary leaf gate, operating in grooves or slots, is not readily adaptable for regulation of flow where high heads are involved. In all high head gates, special consideration must be given to the shape of the gate groove and the pressure and flow conditions within it. Flow through the gate at partial opening is complicated in that the degree of contraction at the bottom edge of the gate is much greater than at the walls below it. The tendency is for the flow to be deflected downward at a steep angle, making it impinge in the gate groove to produce extremely undesirable conditions in the groove and housing. Such objectionable conditions may be greatly reduced by expanding and reducing the conduit immediately upstream from the gate. The diameter at the gate should be approximately that of the conduit. This treatment tends to equalize the contraction on all sides of the jet as it emerges from the downstream side of the gate. Sealing at the upstream side of the leaf instead of the conventional downstream sealing helps to simplify the problem. Upstream seals eliminate the hydraulic downpull and allow the gate leaf to be shaped so as to cause the flow to spring free of the gate grooves. This facilitates aeration and reduces the vacuum. The weight of this type of gate is much less than that of other regulating devices and the capacity, in most cases, is greater. Structural and hydraulic characteristics for a regulating leaf gate are shown on Figure 55. If the seal is to be on the downstream side and the pressure relieved in the upper part of the housing or bonnet of the gate valve, it is possible to get an uplift instead of a downpull depending on the shape of the bottom of the leaf.

9. Radial gates. --Radial gates are generally used in conjunction with open channel flow; but may be used in closed conduit flow also. The problems are similar to those experienced in open channel flow; however, more consideration must be given to the design of the seals, the forces acting on the gate, and the pressure distribution on the gate and in the conduit downstream. Small irregularities in water passage surfaces, unimportant in open channel flow under low head, may induce cavitation in the closed conduit.

Bulkhead Gates

A bulkhead gate, as the name implies, is one which isolates or separates a portion of a system or structure from other parts. One of the most common of this type is that used to unwater conduits which are embedded in dams and receive their supply from the reservoir--the entrance being at the upstream face of the dam. This particular type is usually lowered by hoist down the face of the dam to cover and seal the conduit entrance before unwatering. For the purpose of this monograph, the bulkhead gate will be defined as one which is lowered into place under balanced pressure, a condition present when there is no flow through the conduit. The hydraulic problems concerning bulkhead gates under balanced conditions are few. It is only when water in motion influences the placing or operation of the bulkhead that hydraulic model studies are made. These studies usually have to do with positioning of the gate and measurement of line pull or hoist capacity required. In cases where transportation by floating in water is required to place the gate or special bulkhead in position, the number of hydraulic problems increases and may include flotation and stability problems as well as studies concerning the number,

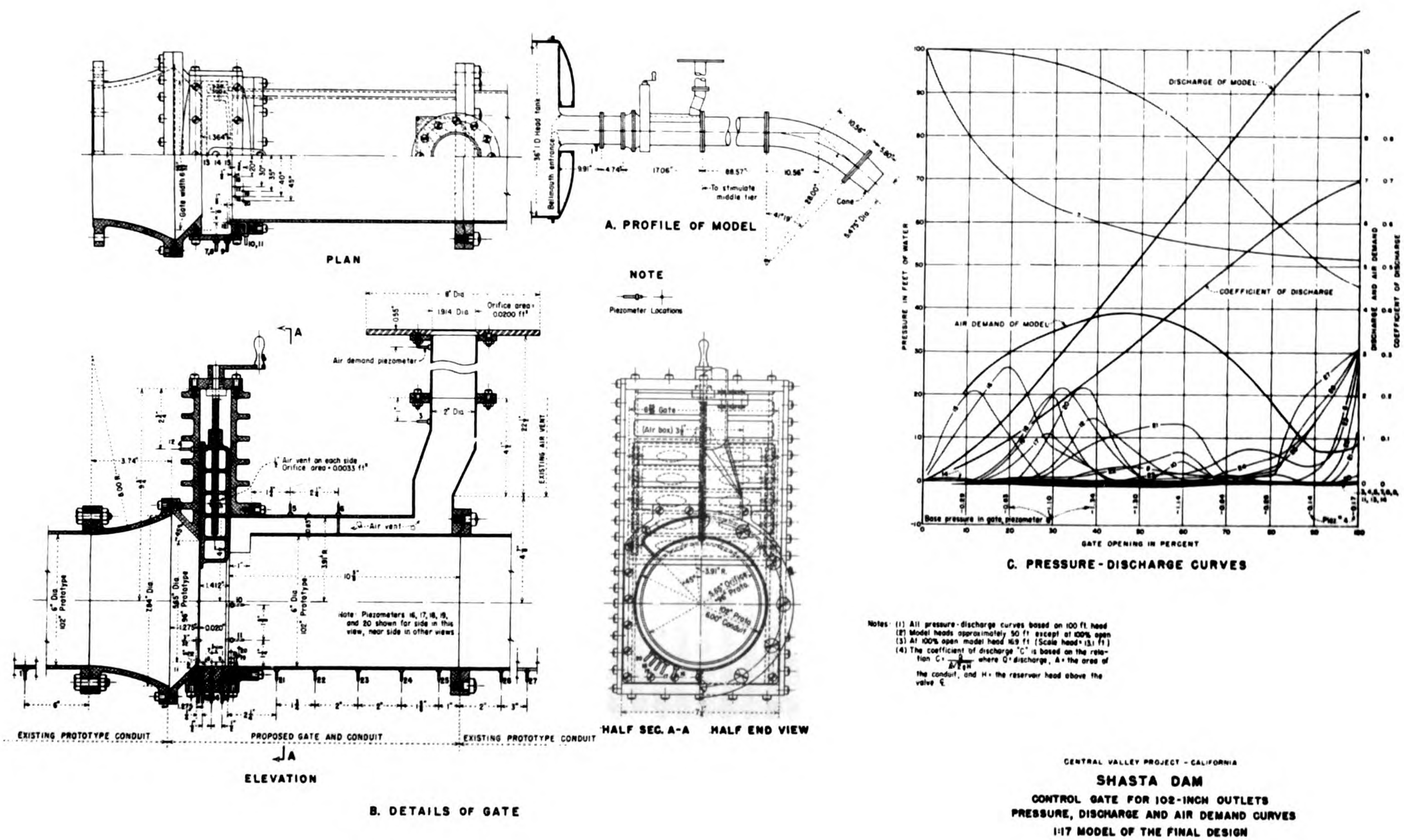


Figure 55. Structural and hydraulic characteristics for a rectangular leaf gate

size, and positions of the lines to maneuver it into position. Studies of this type will be discussed in the paragraphs under Floating Equipment.

Coaster Gates

Emergency closure, when water is flowing through power penstocks or outlet conduits, is a possibility during normal operation of a project. A leaf gate, known as a coaster gate, is usually lowered over the entrance opening. As the gate moves downward across the opening, it is subjected to a differential head which pushes it against the face of the dam with such force that the tremendous frictional resistance precludes use of a simple sliding bulkhead. Wheels or rollers are mounted on the gate to allow it to coast on the face of the dam. Tracks and guides are provided to facilitate the positioning of the gate. Various types of hoists are used to operate these emergency coaster gates which serve also as bulkhead gates. The major problems pertaining to coaster gates are vibration due to hydraulic conditions and the downpull forces on the gate during closure when water is flowing through the system.[27]

1. Hydraulic downpull force. --When a coaster gate is lowered across a conduit passage to cut off the flow, the water surrounding the gate moves toward the conduit entrance with a velocity that varies with the position of the gate and the nearness of the flow to the opening. Since a reduction in static pressure accompanies an increase in velocity, the static pressures on the surfaces of the gate will vary. The velocities underneath the gate are high, so the static pressures on the bottom surface will be low with respect to those on the top surface or wherever the velocities are less. An unbalanced downward force, known as hydraulic downpull, will exist because of these pressure differentials. The downpull force may be excessive, its magnitude varying from a small amount to many times the dead weight of the gate, depending on the depth of the water over the conduit opening, the thickness of the gate, the shape of the bottom surface of the gate, and the ratio of the width of the opening to the width of the gate. The magnitude of this downpull force and the factors which influence it are of major concern in hydraulic model studies because they affect the design and capacity of the operating hoist.

27 Warnock, J. E., and Pound, H. J. "Coaster Gate and Handling Equipment for River Outlet Conduits in Shasta Dam," Transactions, ASME, Vol. 68, No. 3, p. 199.

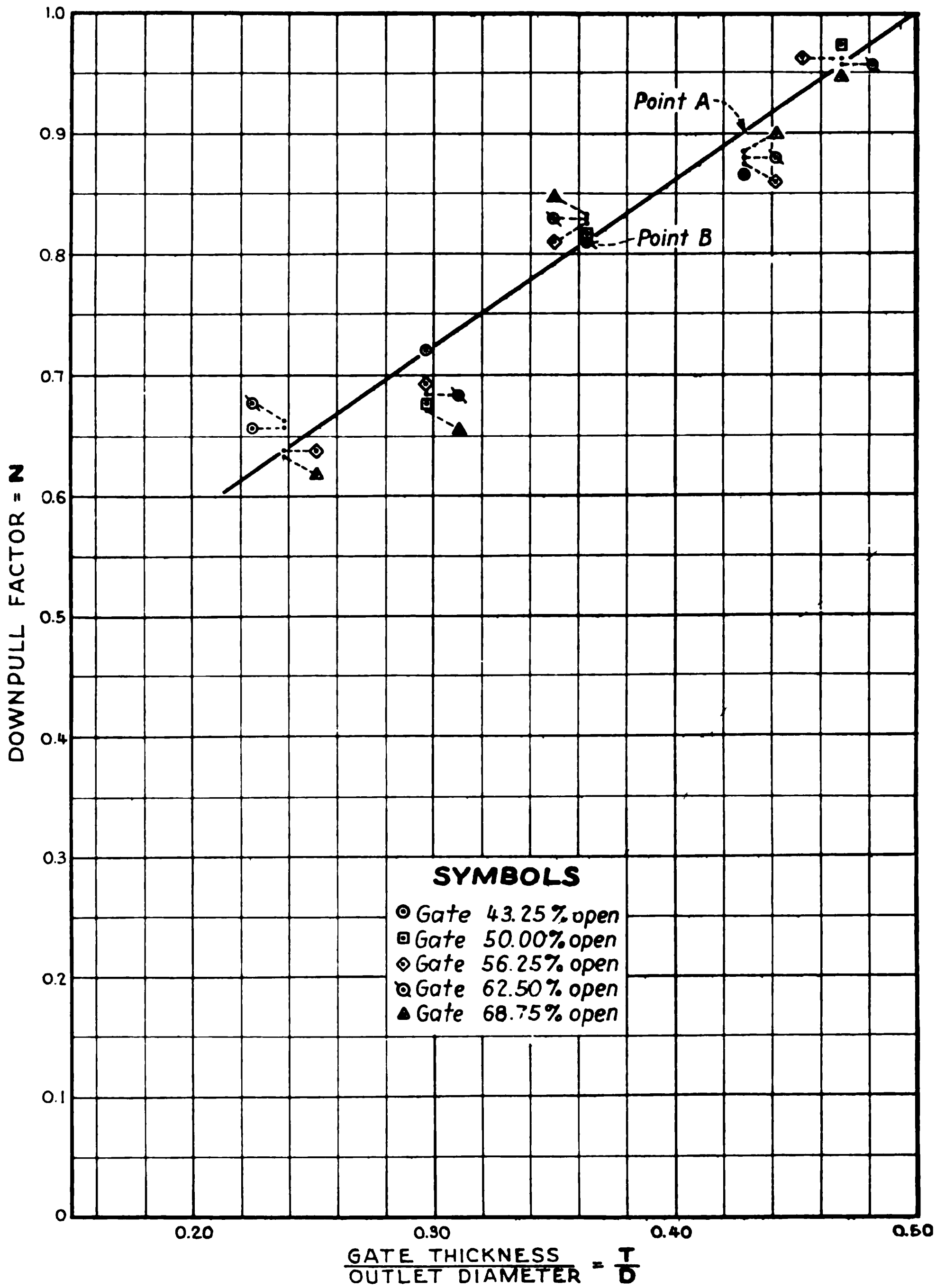
a. Effect of entrance elevation. --The depth of the conduit below the reservoir water surface is usually established by the physical features of the dam and its appurtenances, therefore the maximum head under which the gate must operate is not a variable in most hydraulic investigations. The maximum downpull will occur at maximum head.

b. Effect of gate width and thickness. --For a particular opening and shape of the bottom surface, the minimum downpull will result when the thickness of the gate is a minimum and width of the gate is made as nearly equal to the width of the opening as practicable. This ratio of gate width to opening width is usually fixed by structural requirements and as a rule is not determined by hydraulic investigation. The effect of gate thickness on the downpull force, for a particular design, is shown in Figure 56.

c. Effect of bottom surface shape. --The velocity distribution and pressures on the bottom surface of the gate are influenced materially by the contour of this surface. The lower the velocity, the larger the static pressures, and the smaller the downpull force. It follows that the shape of the bottom surface of the gate which produces the lowest velocities on the surface is the most effective. It is important that the lip of the gate be placed as close to the face of the dam as possible, as this increases the area for the pressure acting in an upward direction. The velocity through the conduit at the time of closure has a material influence on the magnitude of the downpull force. The efficiency of the conduit control downstream is therefore a factor to be considered and evaluated in hydraulic model tests. Pressure curves for different gate bottom shapes are shown in Figure 57.

d. Effect of seal. --The device for sealing the gate after it is in place over the conduit entrance may seem insignificant from the standpoint of its influence on the downpull force; however, the location and nature of the seal may have considerable effects on the magnitude of this force. Model study should include investigation of these effects.

e. Effect of trashrack. --Pressure distribution in a conduit entrance varies with the nearness of the base of a trashrack structure to the opening, thus the same factor influences the flow during closing of a coaster gate. The effect of elevation of the trashrack on downpull is shown in Figure 58.



NOTES

Tests were made by varying thickness of final design gate on Shasta dam Outlet model, figure 2.
The change in downpull force, **F**, by a change in gate thickness from T_1 to T_2 may be expressed as

$$\frac{F_1}{F_2} = \frac{N_{\frac{T_1}{D}}}{N_{\frac{T_2}{D}}}$$

Figure 56. **Effect of gate thickness on downpull**

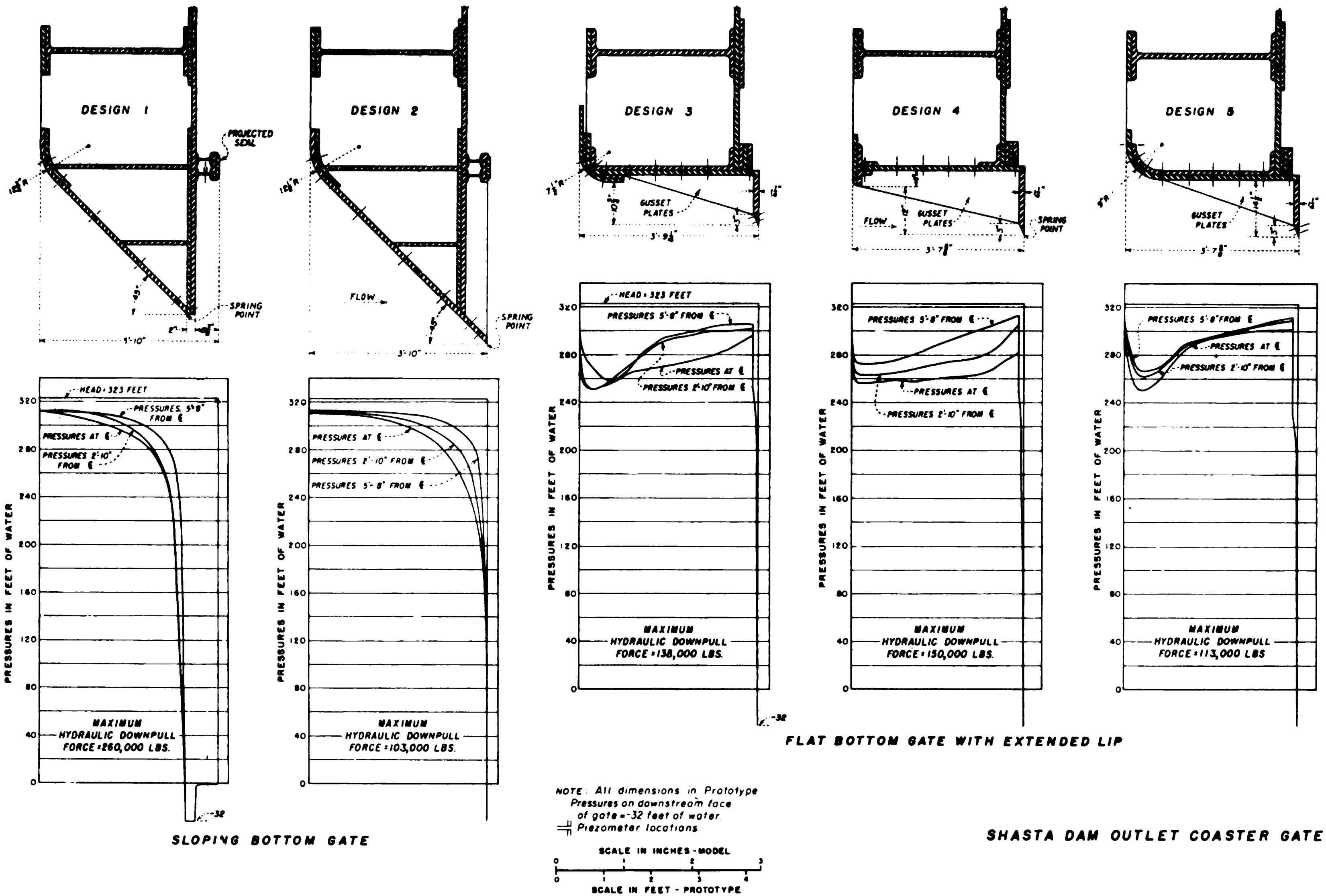
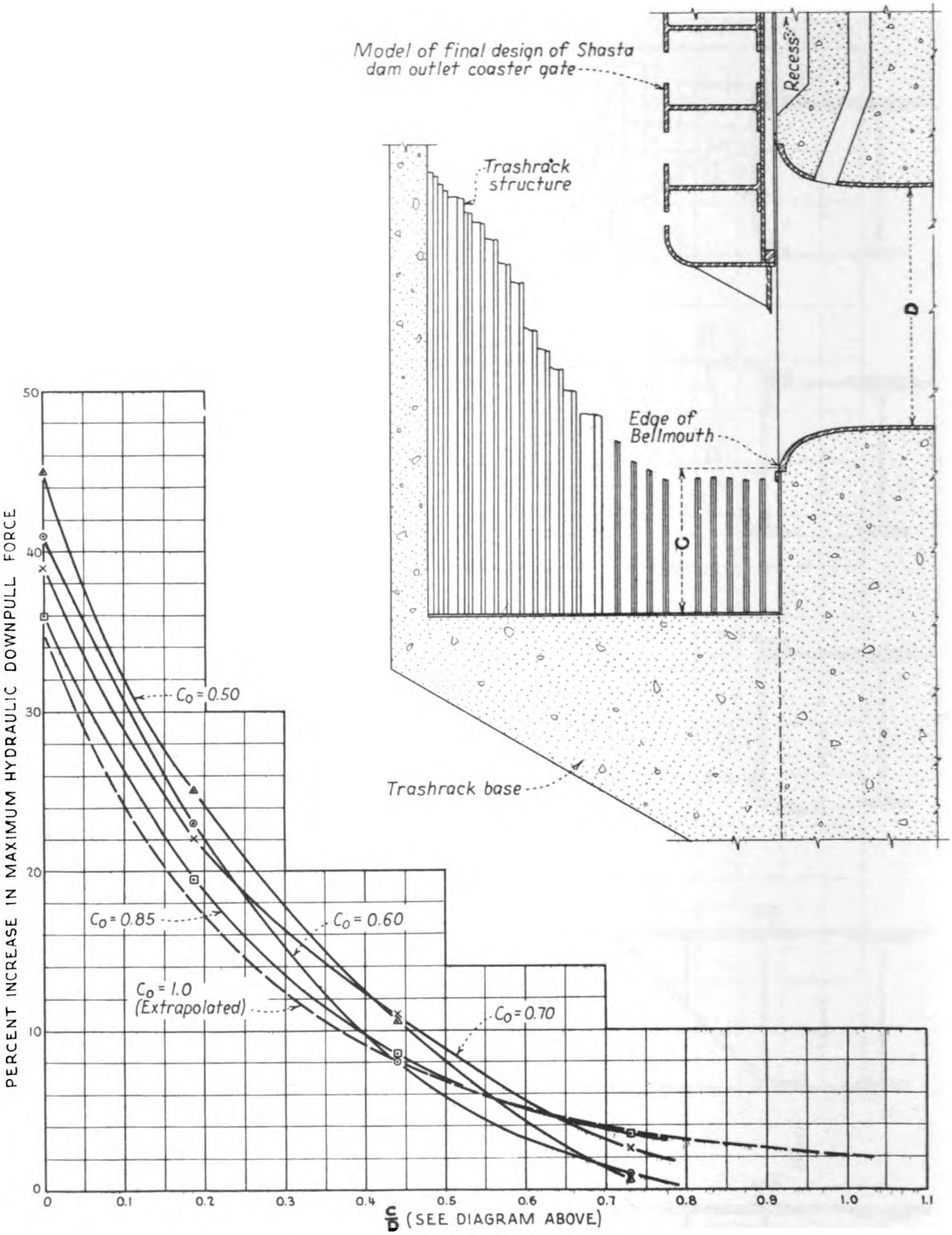
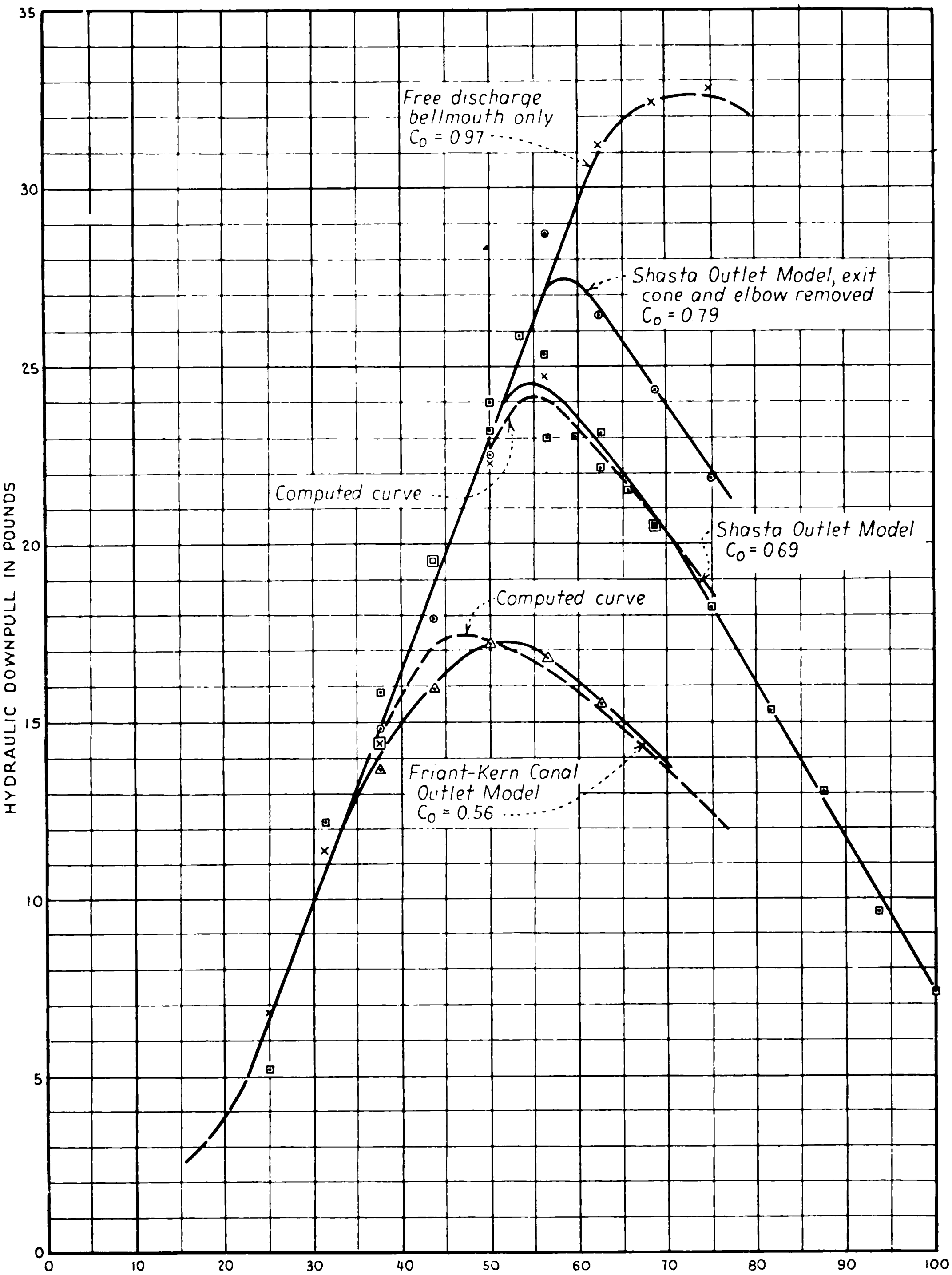


Figure 57. Pressure curves of various gate bottom shapes



NOTES
C_o = Overall discharge coefficient of outlet.
With recess in face of dam, use curve, C_o = 1.00

Figure 58. Effect of trashrack base on hydraulic downpull



Restriction of flow through Outlet is expressed as a coefficient C_o which includes all losses, obtained from the expression $Q = C_o A\sqrt{2gh}$ where Q = discharge, H = total head, A = area of conduit. All data for constant model head of 20 feet.

Figure 59. Effect on downpull of restricting outlet flow

f. Effect of conduit pressure and recess in face of dam.--At partial openings, the pressure in the conduit immediately downstream from the gate decreases as the opening decreases, and becomes subatmospheric at some gate opening, the magnitude of which depends on the nature of the restriction in the conduit downstream. The effect of the downstream control on the downpull is shown in Figure 59. The conduit pressure may affect the downpull in two ways. First, it acts to increase or decrease the effective head on the opening, causing changes in velocity and a larger or smaller pressure differential or downpull force. Second, variations in conduit pressure, if exerted against the lower surface of the upper portion of an extended seal. will vary the differential head on the seal and cause variations in downpull. Aeration by venting downstream from the gate acts to reduce the downpull on the seal if it increases the static pressure on the low pressure side of the seal. The same effect, but in a greater degree, can be gained by placing a recess in the face of the dam, as illustrated in Figure 60. This recess acts to equalize the pressures on the bottom and top sides of the upper seal section and thus tends to eliminate that part of the downpull force contributed by the seal. The shape of the recess is of particular interest.

g. Downpull measurements.--The hydraulic downpull on a model may be determined by direct force measurements or by integration of pressure distribution curves. Direct force measurements are usually made by providing some weighing instrument in the operating mechanism. If this method is to be used, records should be obtained with the gate moving upward as well as downward. This will enable isolation of frictional resistance. Weights of the gate, in and out of water, should be known, to enable determination of the true hydraulic downpull force. Piezometers are often installed in the gate surface to establish pressure distribution curves from which the hydraulic downpull force can be determined by integration. Many piezometers are required in the bottom surface, a few in the top surface, and others in critical areas. Records should be obtained for several positions of the gate, to avoid overlooking any critical condition.

Seals

The primary purpose of seals in hydraulic equipment is to prevent leakage past control devices such as gates, valves, or bulkheads. There are many types of seals. They vary widely in shape and depend on various principles for operation. Problems concerning seals differ with differences in control, design and operation, and with differences in the head under which the seal is to be used. Seals for coaster and fixed-wheel structural steel gates under high heads (100 feet or more) are of particular concern. The music note, or "J," seal has been used extensively but has given unsatisfactory field service, particularly where gate closure has been made with unbalanced head across the gate (emergency closure). An improved high-head seal known as the double-stem type has been developed through laboratory tests.[28]

Another important problem is encountered in sealing the ends of radial gates subjected to relatively low heads. Different types of seals have been used with various degrees of success, and tests have been made to develop a satisfactory design.[29] A limited test program has shown a molded rubber angle seal to be the best. A satisfactory seal without excessive friction drag is a major consideration for low-head seals because the size of hoist equipment will depend largely on the magnitude of the drag. Some success in reducing the friction drag has been realized by using a smaller area of rubber-to-metal contact at the sealing surface.

Because of difficulties in attaining elastic similitude for model seals and in representing severe subatmospheric conditions which might be present with high velocities, laboratory tests are usually made on sections of the full-size seal under prototype heads. These seal sections and pertinent parts of the gate or control are encased in a housing having transparent windows and a water supply and other facilities to subject the seal section to conditions that, so far as practicable, are the same as those in the field. The seal behavior under various operating conditions and head, required actuating pressures, seal leakage, wet and dry frictional drag of the seal on the seat, extrusion action of the rubber in clearance spaces, and durability of the seal under reservoir head for long periods, are the main characteristics to be considered in a laboratory investigation.

28 Case, W. C., "Hydraulic Laboratory Tests of Seals for High-head Coaster and Fixed-Wheel Structural Steel Gates," Hydraulic Laboratory Report No. Hyd-311, July 1951.
29 Case, W. C., "Hydraulic Laboratory Tests of Seals for Radial Gates," Hydraulic Laboratory Report No. Hyd-323, January 1952.

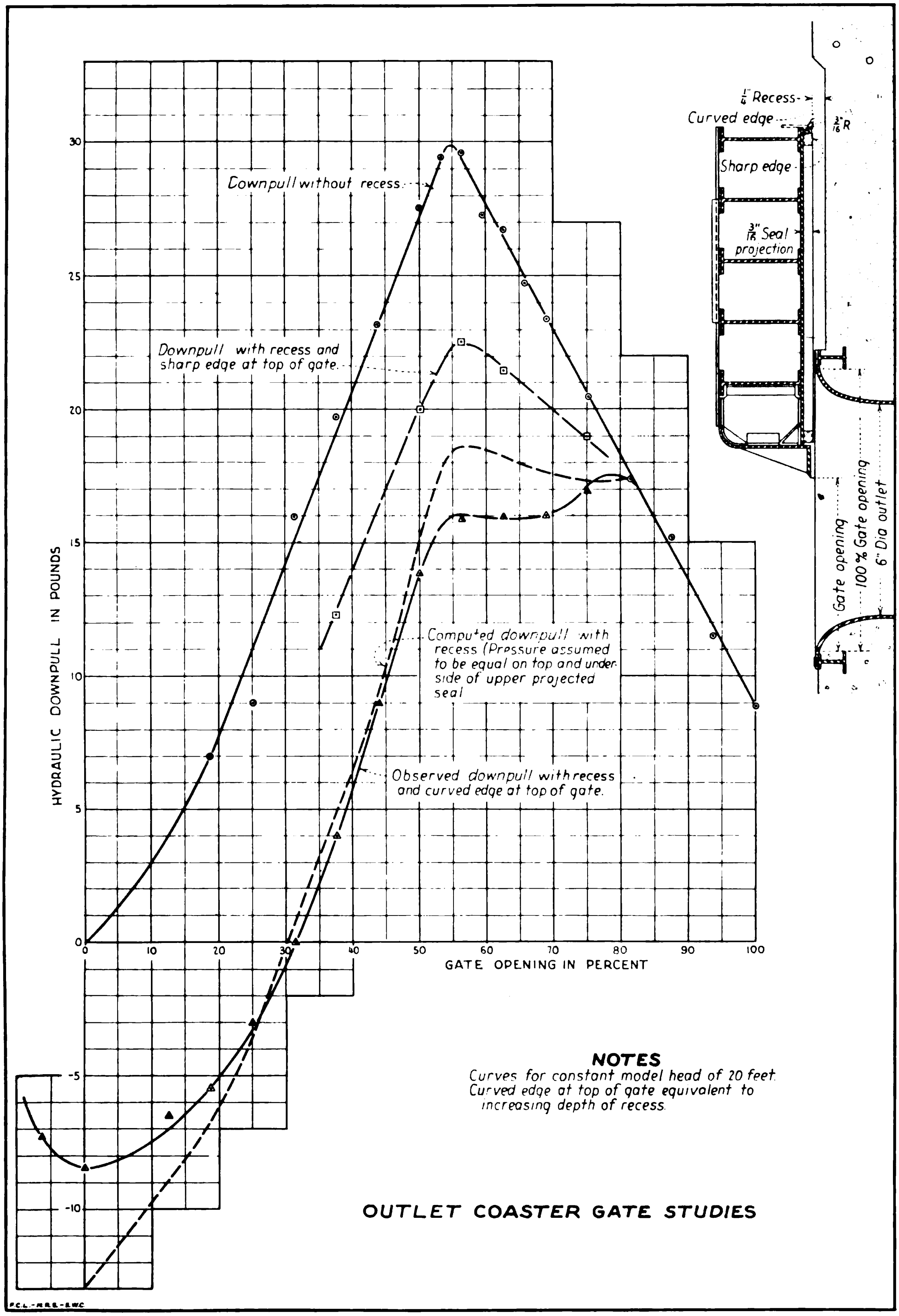


Figure 60. Effect of recess on downpull

Flow Meters

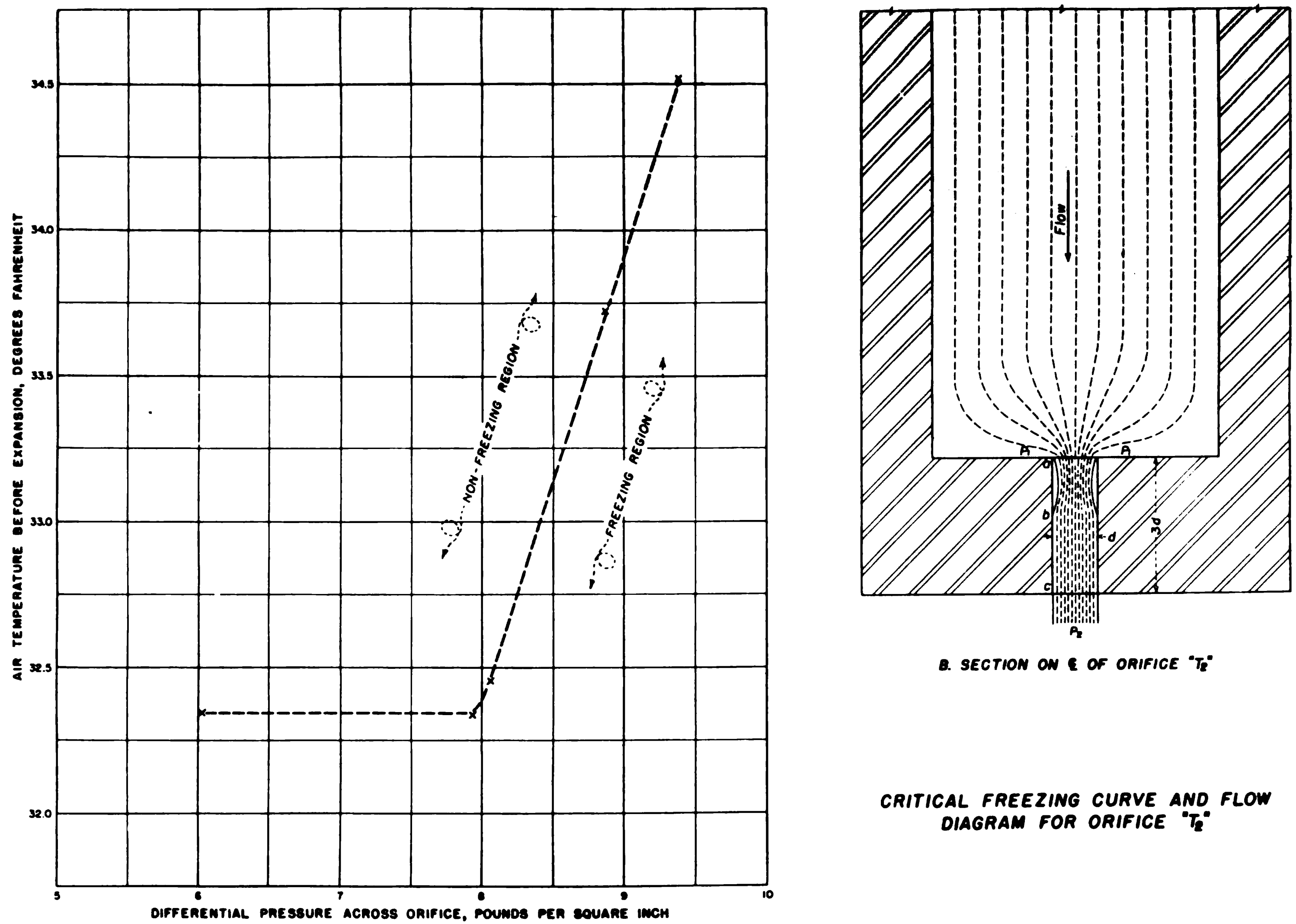
Flow measurement devices or meters for closed conduits may be classified as: (1) those which depend on the dynamic force of the fluid for their operation, and (2) those which measure volumetrically or by weight. Devices which are based on some quality of the dynamic force of the fluid for their operation include venturi meters, orifice meters, Pitot tubes, flow nozzles, propeller meters, and their variations. Flow indicating and totalizing meters operating on these principles are used in irrigation practice. Volumetric meters are used principally for measurement of small quantities and are found in scientific instruments, gasoline pumps, domestic water meters, gas meters, and other similar devices. Laboratory studies on meters include calibration under various operating conditions, and determination of losses, coefficients, limitations, and applicability. The meter is placed in a setting where its discharge can be directed into a volumetric calibration tank or can be passed through a volumetrically calibrated measuring device. The test procedures and techniques are similar to those used in studies of other types of controls and vary with the nature of the meter under study.

Orifices range in variety from the sharp-edge type in a thin plate to a tapered or expanding opening through a thick plate. The latter would be termed a nozzle or a special type of orifice. The device has many applications, but is usually placed in the line or at the end of the line. In either case, a constriction is formed which results in a differential pressure that varies with the velocity of the flow. Because of this characteristic, the orifice is used extensively for determining quantities of flowing fluid.

There are numerous other engineering applications, but the laboratory is concerned mainly with flow measurement. In most cases flow is measured by computations in which the test data are used in formulas or hydraulic laws already established. Occasionally, the laboratory is called on to study the characteristics of a special orifice or nozzle design or of a standard design placed in a peculiar setting. Tests in such cases usually involve extensive discharge and pressure-differential measurements. Discharge quantities are commonly obtained from some standard or volumetrically calibrated laboratory device; the differential pressures are obtained by piezometer connections and a suitable recording gage. The locations of the pressure taps are important. They should be placed to give consistent results. In many cases, standard locations have been adopted by experimenters. These locations have been determined through years of practice and should be used wherever possible. The locations of the pressure taps should be recorded with the test data, otherwise the information loses much of its usefulness. If losses due to an orifice or nozzle are important, as in the case of the opening between a pipe line and a surge tank, sufficient piezometer connections should be provided to assure a complete and satisfactory analysis of the action involved. The coefficient of discharge, pressure differential, and head-loss coefficients, expressed in terms of the velocity head, are useful parameters. The approach and downstream conditions are of great significance in studies of this type and care should be taken to reproduce those conditions correctly in the model. Models of orifices and nozzles should be as large as practicable, to assure flow conditions similar to those for the prototype. In considering pressures and discharge capacity, the model is usually tested at scale heads. However, operation at prototype heads is desirable in many cases, especially if subatmospheric pressures occur in the nozzle passage. A model study of the shape of the jet and its trajectory should be made at scale heads, except that it is desirable that studies involving jets containing mixtures of air be made at prototype heads.

Nozzles for Ice Prevention

In cold climates, it is desirable to prevent ice from forming in the reservoir adjacent to trashracks, gates, or other appurtenances on the face of a dam where damaging stresses might result from expansion and contraction of the ice sheet. A system of air jets, submerged at various points on the face of a dam, has been used successfully to induce circulation of the warmer water from the bottom of the reservoir to the surface. Many hydraulic problems are involved in the design of such a system. The nozzles for the air jets must give a maximum circulation for a minimum quantity of the circulating fluid (air in this case). The shape of the nozzle passage must not be such as would cause excessive expansion, locally or otherwise, for such action results in reduction in both temperature and pressure and often causes freezing of the water in the immediate vicinity and plugging of the air nozzle. A typical curve showing critical temperature and pressure zones is depicted in Figure 61. Hydraulic studies involve measurement of air quantities, pressure differentials, temperatures, effectiveness in inducing circulation, and probability of freezing under operating conditions. An insulated pressure



A. CRITICAL FREEZING CURVE FOR ORIFICE "T2"

B. SECTION ON ℄ OF ORIFICE "T2"

CRITICAL FREEZING CURVE AND FLOW DIAGRAM FOR ORIFICE "T2"

Figure 61. A typical curve showing critical temperature and pressure zones

tank, with a satisfactory means of controlling the temperature of the water, is required. Resistance-bulb thermometers are useful in this type of study.

Mechanical Pumps in General

Mechanical pumps may be classified in many ways, but for the purpose of this monograph classification as to principle of operation will be sufficient. When so classified, these pumps fall in three groups: (1) centrifugal pumps, which utilize rotation and centrifugal force in their operation; (2) positive displacement pumps, which intermittently displace the fluid by action of rotating or sliding elements in conjunction with check valves, and (3) propeller pumps, which impart axial movement to the fluid in the same way that an airplane propeller moves air.

Many problems arise in designing and adapting a pump for a given purpose, and many of these problems may best be studied in the laboratory, especially where the installation is of unprecedented size, or high efficiency of the pump is required, or an unusual operating condition exists. However, much work in pump development has been done by manufacturers, and for the usual installations, a pump of a given size and efficiency may be selected by reference to established data. Laboratory studies of centrifugal pumps are generally limited to writing of specifications, making acceptance tests of given designs, or investigating special operational problems. Acceptance tests are made to determine whether a pump conforms to specifications covering discharge capacity, efficiency, and general performance. Such tests may be made in advance of installation on a scale model constructed and tested by the manufacturer or they may be made later on the service pump. Model tests are made for large installations, and acceptance of the design is based on the model performance. Special problems of operation may arise which require a model study. For example, the power required to start a pump may be in excess of the capacity of the power line, in which event it is necessary to bring the pump up to speed in air before permitting the water to flow. Studies of the changes in voltage, current, pressure, shock waves, and cavitation, as pumps are started and stopped under various conditions, have disclosed interesting facts.[30]

Jet Pumps

The jet pump, as its name implies, is a device which depends on the force of a relatively small high-velocity jet for its pumping action. The jet discharges into a cylindrical chamber of water and imparts motion to the surrounding fluid by an exchange of momentum. The elevation to which the combined flow is lifted depends on the efficiency of the momentum exchange. The pump is useful, particularly for such purposes as providing cooling water for power generatorsand water for operating fish traps and ladders. If the total supply of water for such needs were taken directly from the reservoir above a power dam, a considerable amount of valuable water could not be utilized for power generation. With the jet pump, a relatively small quantity under a high head from the reservoir is used to pump water from the tailrace after it is released from operating power units. Jet pumps in large sizes are not standard manufactured items because there is insufficient demand and because of the special requirements of each installation. Large installations are studied in the laboratory.

The many variables involved, uncertainty concerning theoretical treatment, and the nature of jet-pump action introduce complexities in the design of these pumps. Such factors as the shape, velocity, and direction of the driving jet: ratio of diameters of the driving jet and mixing tube; size, length, and shape of mixing tube; size, length, and divergence angle of diffuser tube; size and arrangement of discharge conduit, ease of pumping; quantity of water required; pressure conditions within the pump; possibility of cavitation; and efficiency must be considered in any model study pertaining to the design of jet pumps. Should some of these factors be inconsistent with others, the efficiency will be affected adversely. Jet-pump action often causes cavitation at the boundary between the fast-moving jet and the slower-moving water set in motion by the jet. Also, cavitation may be caused by expansion and deceleration of the water in the mixing tube, The conditions may create difficultyin transforming model results to prototype conditions, and necessitate special test procedures. The principal requirement is that the model be operated at prototype driving, suction, and pumping heads. This method of testing is the most reliable for determining the pressure intensities at points within the model where testing in a vacuum tank is not possible. However, the method has disadvantages in that it is difficult to ascertain

30 "Studies to Determine Suitable Methods for Starting and Stopping the Pumps in the Granby Pumping Plant--Colorado-Big Thompson Project, Colorado," Hydraulic Laboratory Reports No. Hyd-113 and -150, Bureau of Reclamation, June 1942 and September 1944, respectively.

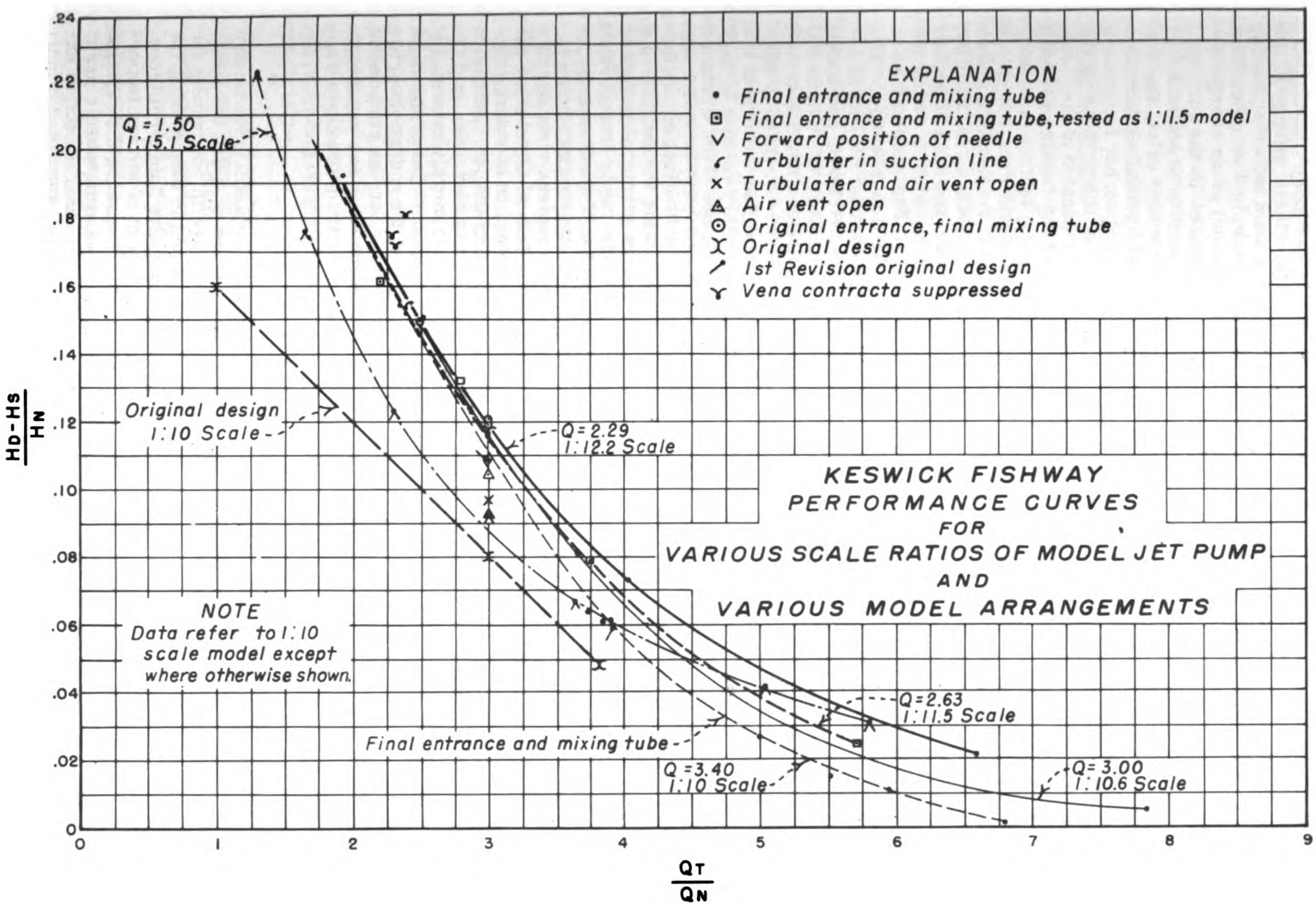


Figure 62. Jet pump model performance curves

the optimum length for the mixing tube and the conditions inducing cavitation in the tube. Admission of a small amount of air at the outer edge of the driving jet as it leaves the pump nozzle, where it will reach the boundary between the driving and driven jets, has proved beneficial in minimizing noise and vibration from the cavitation. A jet pump is considered to be well designed when the driving quantity is a minimum for a given driven quantity. The ratio of these quantities will depend on the ratio of the pumping head to the driving head. Performance curves for various model arrangements are shown in Figure 62.

Hydraulic Turbines

A hydraulic turbine may be defined as a prime mover actuated by moving water to generate electric power. The water is conveyed to the turbine by a conduit or penstock and discharges from it into the tailrace. There are many types of turbines and many variations, each of which has its own sphere of usefulness. The type to be used requires special consideration, in the light of the specific service and operating conditions. Modern turbines may be classified as impulse, reaction, or axial flow machines.

Much of the research involved in developing turbine designs, particularly that concerning the turbine scrollcase, wicket gates, and runners, has been conducted by turbine manufacturers. Turbines are purchased by specification, and model studies are needed only in cases of unusual size, peculiar operation requirements, or high efficiency requirements. Under these conditions the manufacturer constructs and tests a model and acceptance of the designs is based on the model results. The laboratory might take part in the tests by making an inspection of the manufacturer's model construction and operating procedures and checking the analysis of data, or by actively participating in the model testing and analysis of data. It might be necessary to study the effect of pressure or shock waves resulting from sudden shut-down or change in load, or to test the draft tube to obtain the greatest efficiency for the space available. Such special tests involve extensive pressure, discharge, and velocity measurements, the instrumentation and procedure for which vary with the problem.

31 "Hydraulic Studies of a Water Jet Pump for the Keswick Dam Fishtrap--Central Valley Project, California," Hydraulic Laboratory Report No. Hyd-154, Bureau of Reclamation, Sept. 1944.

Energy Absorbers

It is sometimes desirable to dissipate, absorb, or convert into nondestructive form a large portion of the kinetic energy of the flow from a conduit system before releasing it into a natural stream bed or other flow channel. A device known as an energy absorber or dissipator is used for the purpose. Where the space for such an installation is limited and flow restrictions exist, the design presents complex hydraulic problems. As a rule, the energy absorber for closed conduit flow makes use of one or more of the following actions: (1) expansion of the flowing jet, (2) impact of the flowing jet on a solid or liquid surface, (3) surface or boundary friction, and (4) disintegration of the flowing jet. Expansion of the jet while maintaining nearly uniform velocity is the most effective of these actions and most absorber designs include a flow chamber in which the expansion takes place. The chamber usually forms part of the flow passage. The shape and size of the chamber depend on local conditions, flow restrictions, and space limitations. The chamber is effective if fairly uniform velocity can be maintained in the flow passage. However, there is danger of cavitation in energy absorbers of the jet expansion type when they operate at high heads if the water passage is not designed properly. A device that depends on the impact of the jet upon a solid surface or another jet would be of little value without use of a chamber having an area greater than that of the conduit, where the jet can be transformed into a slow-moving mass of small eddies. A combination of baffles and an expanding section is effective but requires considerable space. The same is true of any device the operation of which depends on boundary friction or disintegration of the jet. Diffusion of the jet is of prime importance, and the actions listed above are merely means for obtaining the best diffusion. One of the most common installations of the energy absorber is in power penstocks for the purpose of minimizing water-hammer or shock waves in the system when the wicket gates close rapidly.

Hydraulic problems governing the design of an energy absorber are of such a nature as to require extreme care in conducting the model investigations and in interpreting the resulting data. Comparative fluid turbulence, friction, and entrainment of air are important considerations. Pressure distribution and the existence of subatmospheric pressures may influence the absorber action extensively. If pressures on the model indicate cavitation in the full-size structure, the model results may not be representative.

Provision of vents for aeration of critical low-pressure areas is effective in minimizing cavitation. Model tests are very useful in establishing the size and location of these vents. The very nature of an energy absorber necessitates extensive pressure measurements during hydraulic model tests. Measurements of discharge quantities and magnitude and distribution of velocities at particular points are required. Also, tests should be made to assure satisfactory performance at operating heads.

Floating Equipment

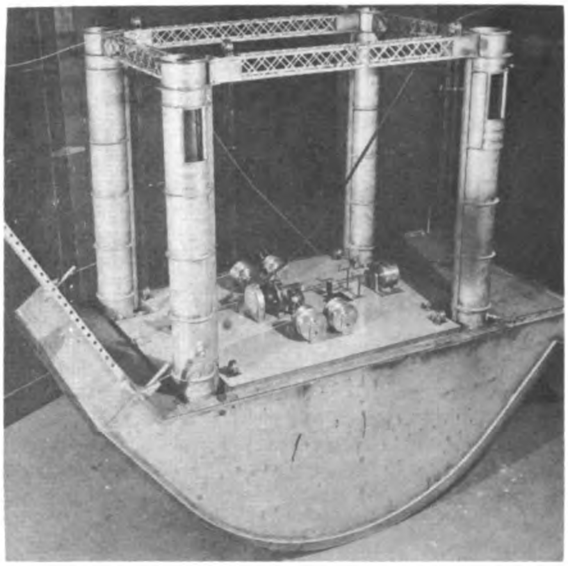
Floating equipment is often used in the construction, operation, and maintenance of dams and their appurtenant features, but the hydraulic laboratory is seldom involved in its testing. Occasionally there is need for special equipment, such as caissons and bulkheads for unwatering and for use in repair of underwater facilities or portions of a structure. The bucket and face caissons for Grand Coulee Dam[32] and the bulkhead for the spillway gates at Parker Dam are examples of such special equipment. The properties, characteristics, and design criteria applying to these structures are the same as those pertaining to the design of boats, ships, drydocks, and other floating structures. Hydraulic model tests may involve determination of size and direction of forces to be exerted on cables used for maneuvering the equipment; optimum position for maneuvering facilities; evaluation of the coefficient of drag for various positions of the equipment for different velocities of flow; maneuvering procedure and limitations; stability or ability of equipment to remain upright or within a given angle of tilt or list during operation; the amount and location of ballast and its effect on stability; determination of metacentric height and its effect on motion of the equipment in choppy waters; magnitude and effect of velocity and turbulence on equipment when it is in final position on the structure; procedure in flooding and unwatering working compartments; and hydraulic characteristics of auxiliary equipment such as drydocks, special operating barges, or special floating compartments.

The size of a model for flotation studies is usually governed by the size and properties of the material to be used in the construction of the model or the size of facilities available for model operation. The model size should be sufficient to minimize the effect of surface tension. Both metal and plastics have been used successfully for construction of models of floating bodies. An all-metal model of a floating caisson is shown in Figure 63. Plastics used should be types that absorb very little water.

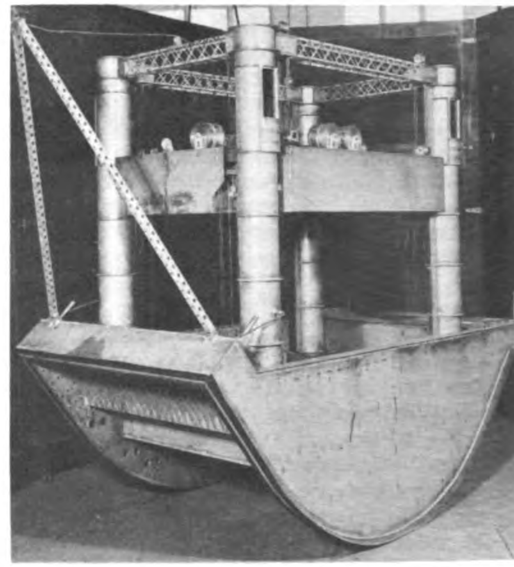
It is often necessary to tow or maneuver floating equipment in flowing water. Design of facilities for such operation is influenced by forces that will act on the floating vessel as the result of relative motions of the vessel and the water. If the vessel is not of a shape for which the drag coefficient is known, it may be desirable to conduct tests to determine the coefficient for different velocities and directions of impact (Figure 64). With this information and knowledge of the velocity and its direction, it is possible to compute forces in lines used for maneuvering the equipment from point to point in the moving water. However, such computations may be complicated by lack of information concerning the directions and magnitudes of the current velocities. If such information is not available, flow conditions will have to be reproduced in a model of the same scale as that of the model of the floating vessel. If the flow pattern is complicated by several sources of flow, such as might be the case immediately downstream from a dam, such a model is essential in establishing the number of maneuvering lines required, the orientation of these lines with respect to the path of the vessel, the forces acting on each line (for the purpose of establishing the cable size), and the capacities of operating hoists, puller machines, or other towing facilities.

A floating vessel is said to have stability when it tends to return to the original floating position after being displaced from it by an outside force. It is said to be unstable when it continues to move in the direction of displacement when such a force is applied. The term "neutral stability" is applied when the vessel tends to remain in the displaced position when all external forces are removed. A vessel should have sufficient stability in all directions to withstand any outside force or concentrated weight which might be applied. If it is to be moored or anchored to some stationary object, in water having a choppy or turbulent surface, the stability should not be so great as to induce rapid rocking motions. Such motions increase the difficulty of making satisfactory hitches or anchorages. Metacentric height is defined as the distance from the center of gravity to the intersection of the vertical axis of the vessel and a vertical line through the center of buoyancy when the vessel is displaced at a very small angle (Figure 65). To have stability the metacenter must be

[32] Ball, J. W., "Model Studies Pertaining to the Repair, Maintenance, and Protection of the Spillway Bucket and the Protection of the Tailrace Slopes and Downstream River Banks at Grand Coulee Dam," Hydraulic Laboratory Report No. Hyd-174, June 1945.



A. Side view of caisson-powered operating barge in lower position



B. Oblique view of caisson showing working space. Operating barge in raised position

Figure 63. Completed model scale 1:20, of the floating caisson and its operating barge used in the repair and maintenance of the spillway bucket of Grand Coulee Dam

above the center of gravity. When the metacenter and center of gravity are one and the same point, the vessel will have neutral stability about the horizontal axis. When the metacenter is below the center of gravity, any displacement will tend to overturn the vessel and the vessel is considered unstable. Ballast or weighty material used to insure stability should be selected carefully and placed properly because it affects the center of gravity and might shift it sufficiently to induce instability. This is particularly true if the ballast is liquid having an unconfined surface. An undesirable list (tilt) or capsizing might result from improper use of a liquid ballast. The metacentric height of a vessel may be determined experimentally by the inclination method,[32] which makes use of equal weights placed on the deck of the vessel so as to give different degrees of tilt.

FIELD STUDIES

General Considerations

Hydraulic field studies are usually considered to be extensions of laboratory investigations to the prototype level. The field and laboratory studies are basically analogous, the difference being merely the fact that the prototype structures are used in the field studies and scaled models are used in the laboratory studies. Familiarity with design considerations is necessary in each case.

Thorough understanding of the laws of hydraulic similitude, of methods and procedures employed in the laboratory, and of analysis of the results of the investigations, is essential in field studies.

Where hydraulic problems are to be considered in a broad sense, a program of laboratory and prototype studies should be carefully integrated. In some instances the laboratory cannot provide the solution without field assistance; for example, where a laboratory simulation of tide is required, the actual tidal phenomena must be observed and be correctly applied in constructing and operating the model. Sometimes a laboratory study would be of no value; for instance, in determination of insufflation (entrainment) of air in a high-velocity jet of water.

It should not be assumed that field studies always require installation and operation of expensive equipment by a large crew of testing engineers. The studies often can be performed by one engineer familiar with hydraulic problems, who observes a particular action during regular prototype operation. In other instances, however, use of elaborate equipment may be necessary.

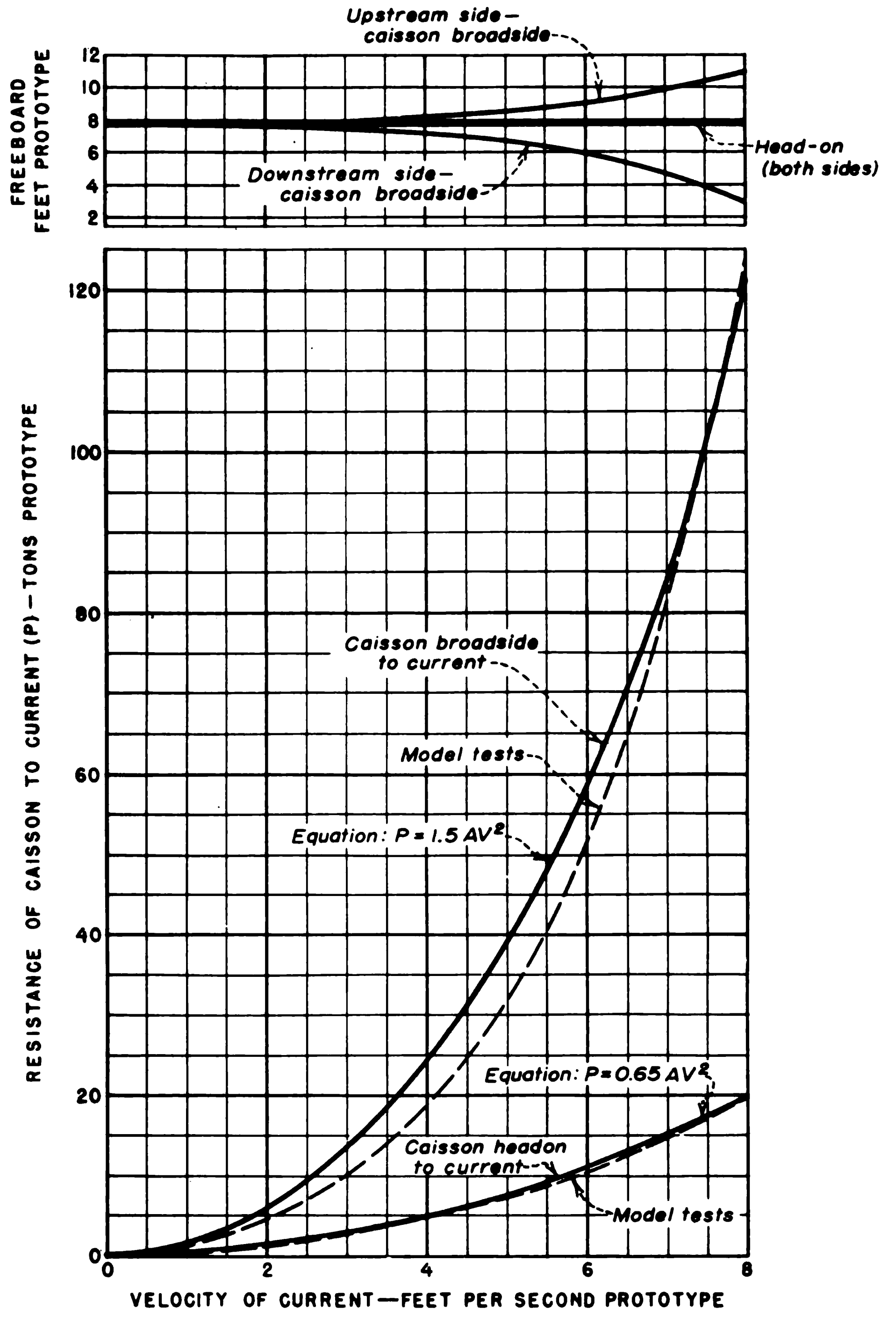
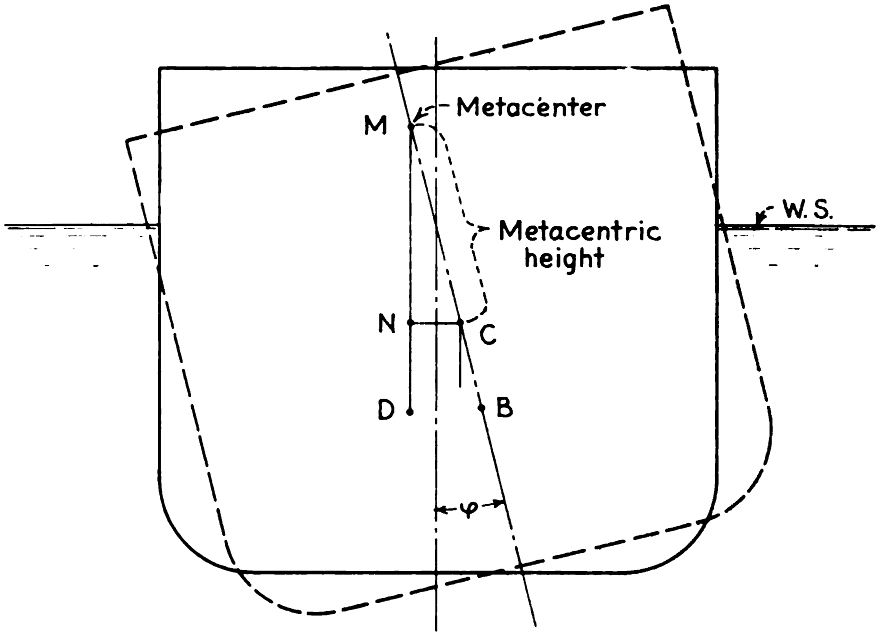
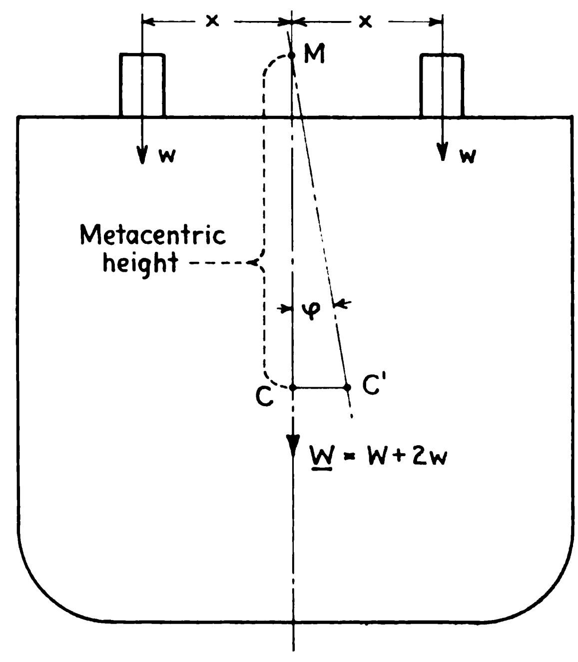


Figure 64. Resistance of caisson to currents of different velocity



A. METACENTER OF FLOATING VESSEL, ROLLING DISPLACEMENT



B. ARRANGEMENT FOR EXPERIMENTAL DETERMINATION OF METACENTRIC HEIGHT

Figure 65. The metacentric height of a vessel

Although many accurate predictions can be made from hydraulic model studies, there are cases in which lack of information concerning specific scale effects creates some uncertainty in the mind of the designer. Where this situation exists, provision should be made for verification of the model test results by prototype investigations and, if necessary, corrective measures should be applied to the completed structure. Such correlation between model and prototype performances is of great value in subsequent structure design.

Some hydraulic problems may be wholly or partially solved, at little if any more cost than that for model investigations, by studies of field structures in operation. Such field studies have the advantage of yielding quick results directly from undistorted prototypes. The rapidly increasing number of newly built hydraulic structures throughout the United States affords a diversified field of opportunity for useful observations and investigations.

Measurements

Difficulty in obtaining accurate prototype data is probably the main reason for delay in progress in this field of hydraulic measurements. Field measurements must be well planned and conducted with the same care and, if possible, the same degree of accuracy as those made in the laboratory. Laboratory equipment will ordinarily not be applicable for field observations, because the forces involved are greater than those encountered in the laboratory. Some test equipment has been developed commercially, but, in general, the testing apparatus must be designed and constructed for specific field tests. This does not mean that each particular field test must have specially designed test equipment, but, rather, that most field testing equipment should be designed for field application and for repeated use.

Recording instruments are used more in field studies than in laboratory tests, because observation stations are more widely separated, longer time increments may be involved, and experienced observers may not be readily available.

1. Measurement of discharge. --The scope of a series of field measurements does not ordinarily permit inclusion of a costly measuring device for the sole purpose of the test program. Search should be made for a measuring device that is available at or near the structure. The discharge passing the structure may be measured at an existing gaging station by use of current meters or of a Pitot tube, weir, orifice, Parshall flume, or commercial meter. If such devices are not adaptable, consideration should be given to the salt velocity, color velocity, or Gibson method. The method selected will depend on the conditions to be met, such as size and type of channel, quantity of water, velocity, and accuracy desired. For very large flows at extremely high velocities, the salt velocity method will probably be most economical and practical. Techniques employed in the application of these methods are covered in other publications and will not be repeated here.

2. Measurement of pressure. --Statements in regard to pressure measurements in the previous section on Instrumentation apply generally to both model and prototype studies. Because the velocities encountered in the prototype study are generally higher than in the laboratory, extreme care must be exercised in the installation of piezometer openings. A small burr in the piezometer orifice will seriously affect the readings, and excessive surface roughness, either upstream or downstream from the piezometer tap, will influence the measurements.

Air entrapment in the leads between the piezometer orifices and the measuring devices should be prevented by avoiding placement of piezometers at tops of penstocks or at other places where air is likely to collect. Entrance of air into the pressure measuring system is much more troublesome in the field than in the laboratory, for the flow normally contains more air and the lead tubes are considerably longer.

Accurate results cannot be obtained if the air is not "bled" from the leads. Valves through which this air may be released must be integral parts of the system. A high-pressure water supply, if available, is extremely helpful in purging the system of air bubbles. Examples of piezometer installation in the field are shown in Figures 66, 67, 68, and 69.

For low pressures, it is common practice to use water or mercury manometers. A special type of high-head mercury pot gage has been developed for use in field studies. Details of this gage are given on Figure 70, and the instructions for its use in Table 2.

Pressures that exceed the capacity of the mercury gage are measured by a fluid-pressure scale. This instrument has a capacity of 300 psi and is graduated in tenths of pounds. It is readily portable and easy to install and manipulate. This instrument is shown photographically in Figure 20B.

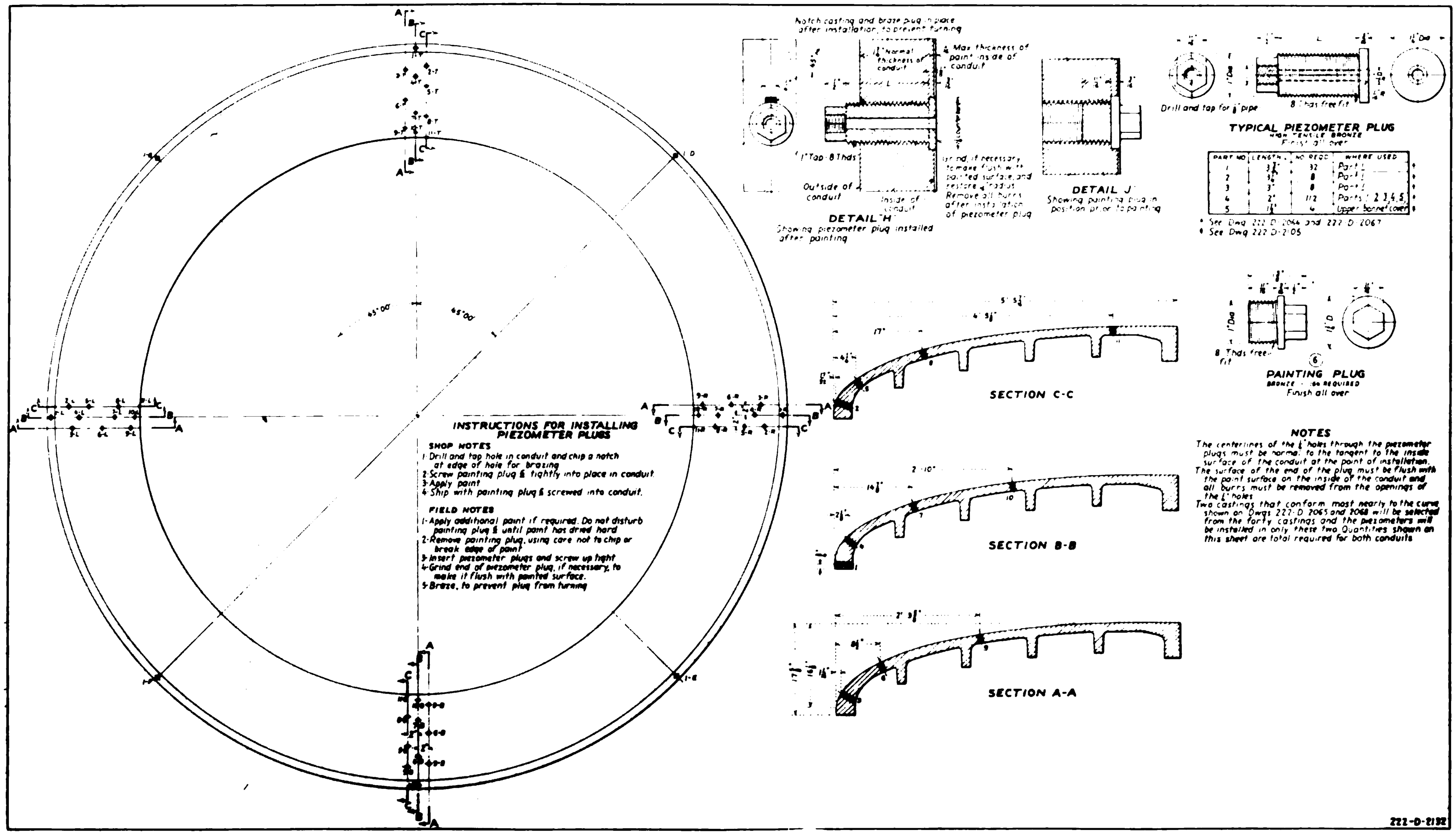


Figure 66. Installation of piezometer plugs

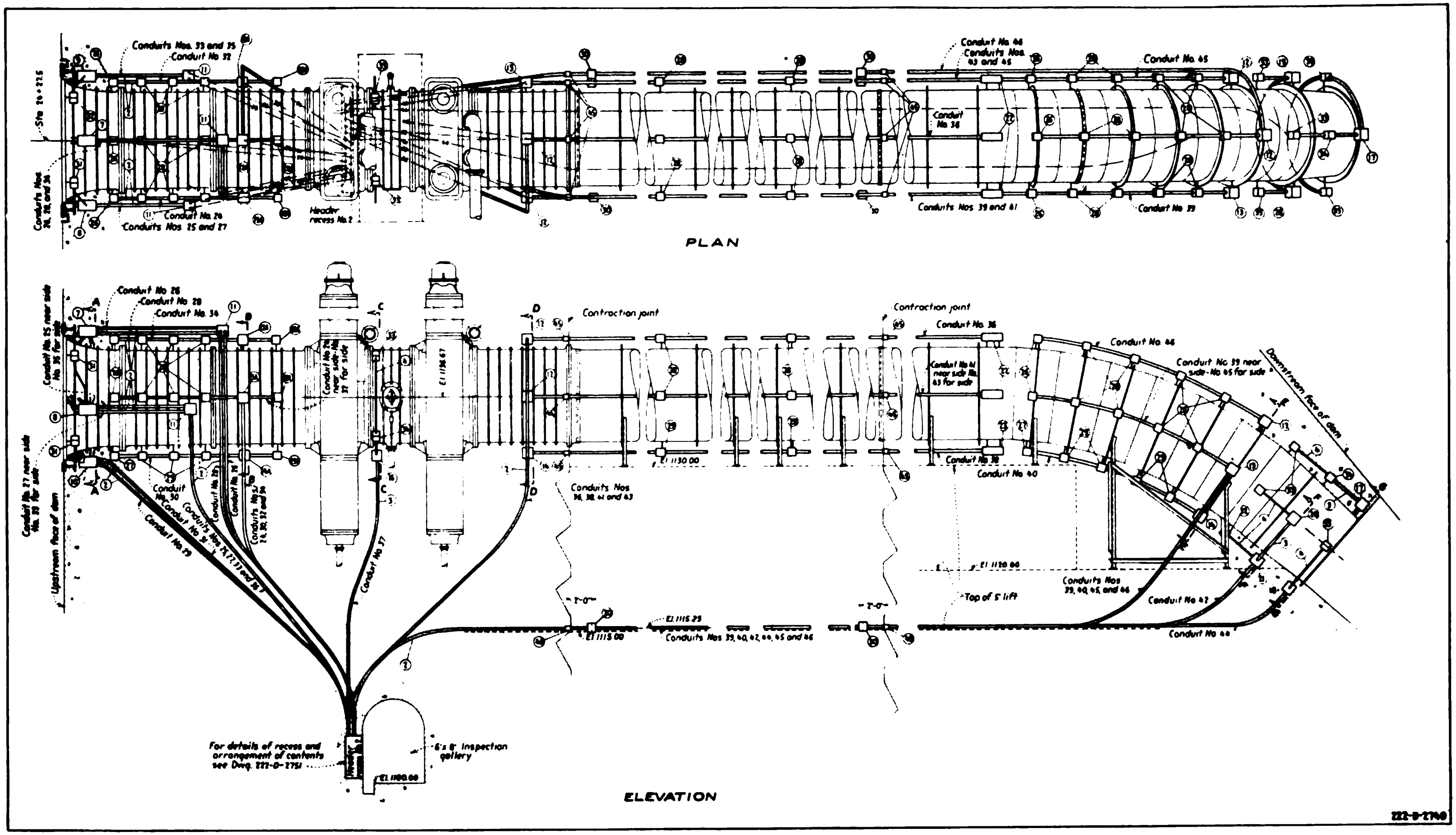


Figure 67. General installation of piezometer piping

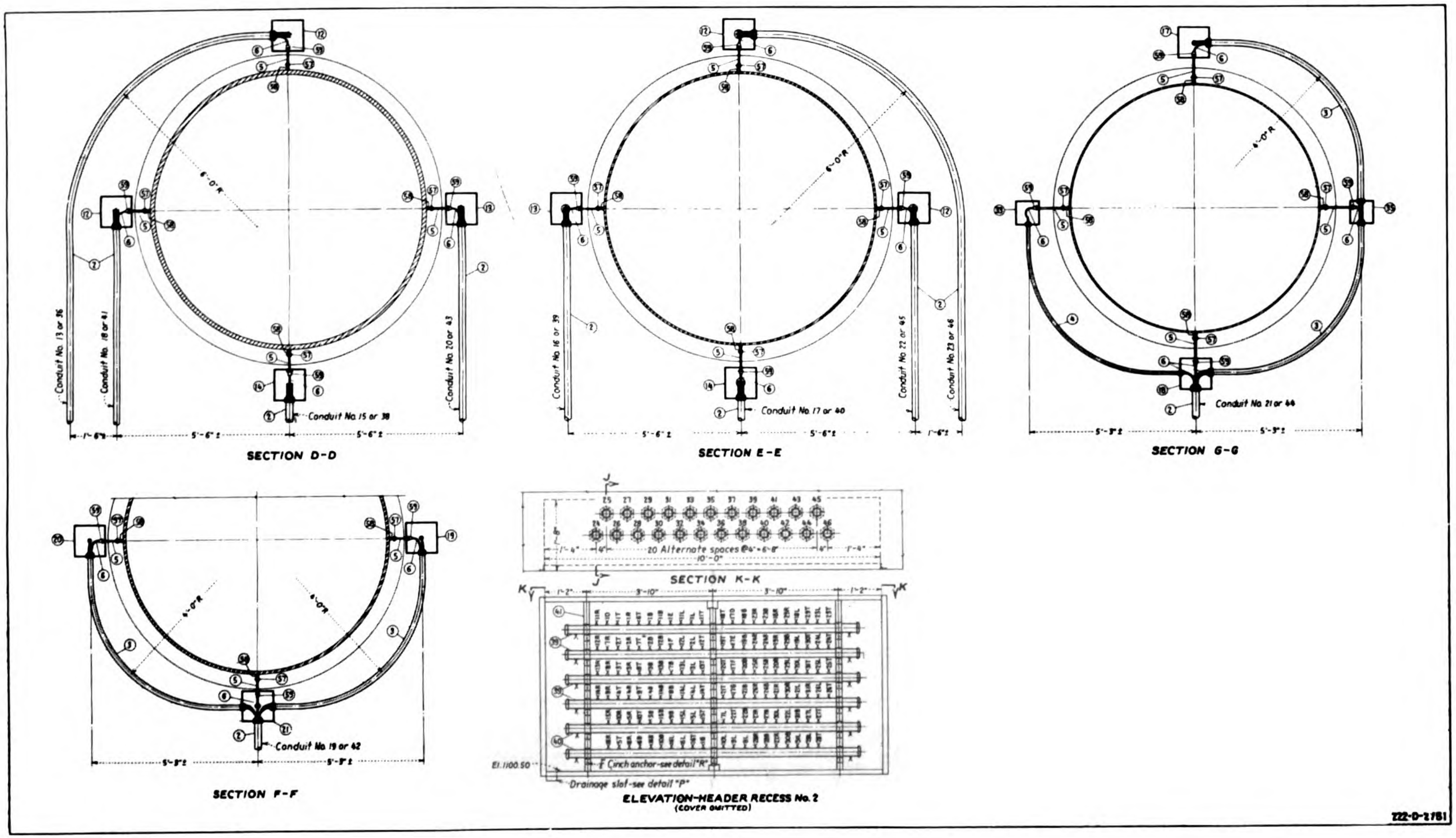


Figure 68. Installation details of piezometer piping

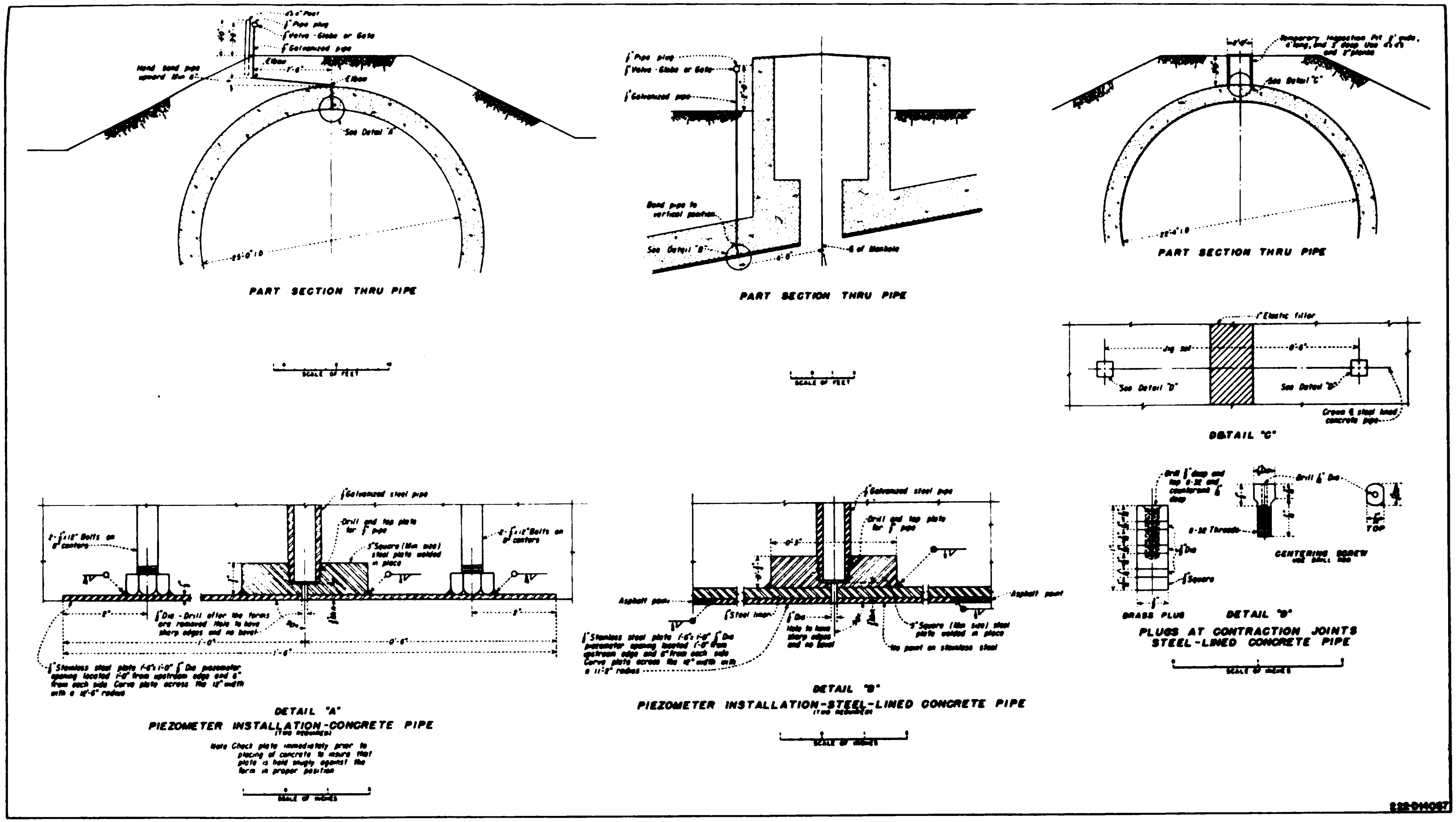


Figure 69. Piezometer installation, Soap Lake Siphon

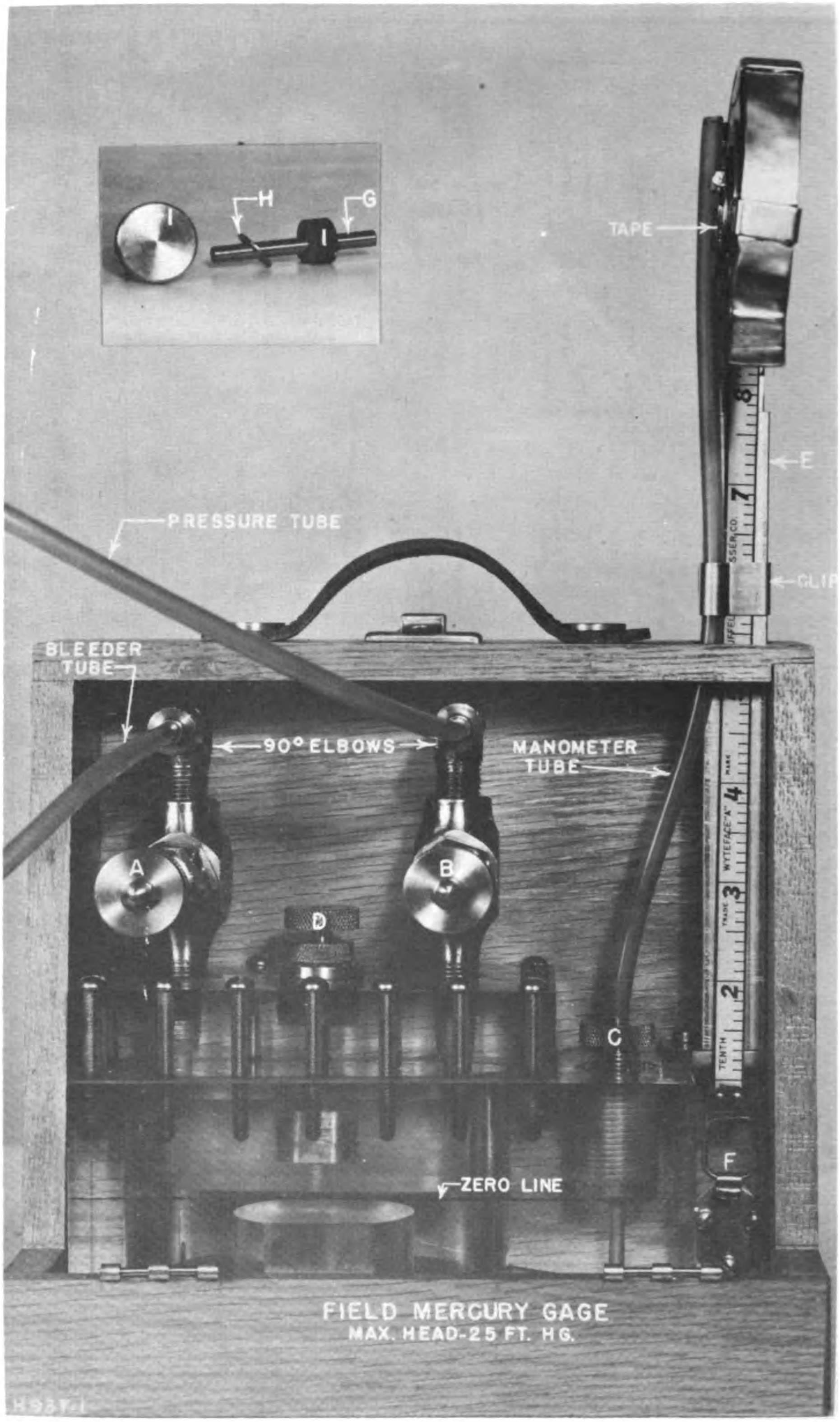


Figure 70. High-head mercury pot gage

Table 2

DIRECTIONS FOR FILLING AND PLACING FIELD MERCURY GAGE IN SERVICE

1. Set the gage box on a surface as nearly horizontal as is obtainable. Remove lid by dropping it and lifting it from the hinges. Open needle valves "A" and "B" Remove plug "C" (or C-1) and insert funnel into opening.

2. Move displacement plunger into its uppermost position by turning knurled wheel "D" clockwise. Pour 5 pounds of mercury through the funnel into the mercury pot. (The gage is frequently furnished containing the proper amount of mercury.)

3. Close bleeder valve "A" and pressure valve "B". Connect one end of the bleeder tube to the 90° elbow in bleeder valve "A", leaving the other end free. Connect one end of the pressure tube to the 90° elbow in the pressure valve "B". Connect the other end to the source of pressure.

4. Assemble the manometer staff extensions "E" and insert the tape and tape holders in the upper end. (Eight extensions totaling 25 feet are provided; however for convenience, assemble only the number required to measure the head encountered.) Insert the staff down through the opening in the top of the box, and over the nipple provided on the gage. Unreel the steel tape, insert the free end through the top opening of the box, and place the ring over hook "F"

5. Slip the stainless steel insert "G" into one end of the manometer tube, and thread the end of the tube down through the top of the box. Slide plug "C" over the end of the tube, allowing one-half inch of the tube to protrude. Slip the metal washer "H" over the protruding end of the tube, followed by the rubber gasket "I". Insert this assembly into the "G" opening, and screw plug "C" in snugly with the fingers.

6. Extend the free end of the manometer tube up the face of the manometer staff extensions, and fasten both the tube and the tape to the staff with the clips provided.

7. Open pressure valve "B". Open bleeder valve "A" to expel air from the gage and the pressure tube. Let the water run until the flow from the bleeder tube is even and undisturbed. Close valve "A", and the gage is ready for service.

8. With valve "B" open, bring the top surface of the mercury in the pot to the zero line by turning the knurled wheel "D". <u>This must be done before each pressure reading.</u>

9. Read the height of the mercury column in the manometer tube. If the surface of the mercury column rises and falls too rapidly to obtain satisfactory readings, slowly close valve "B" until the movement is damped, being certain that the surface pulsates sufficiently to insure that the pressure line is "live".

10. Assuming a temperature of 15° C (59° F), the mercury reading is converted to feet of water by multiplying by the constant 13.547 and to pounds per square inch gage pressure by the constant 5.867.

11. To take the gage out of service, close valve "B" and open valve "A", allowing the mercury in the manometer tube to run into the mercury pot. Close valve "A" and detach the bleeder tube and the pressure tube. Remove plug "C", thus disconnecting the manometer tube. Insert shipping plug "C-1" (identical to plug "C" except that it has no hole drilled through the center) to prevent spilling of the mercury.

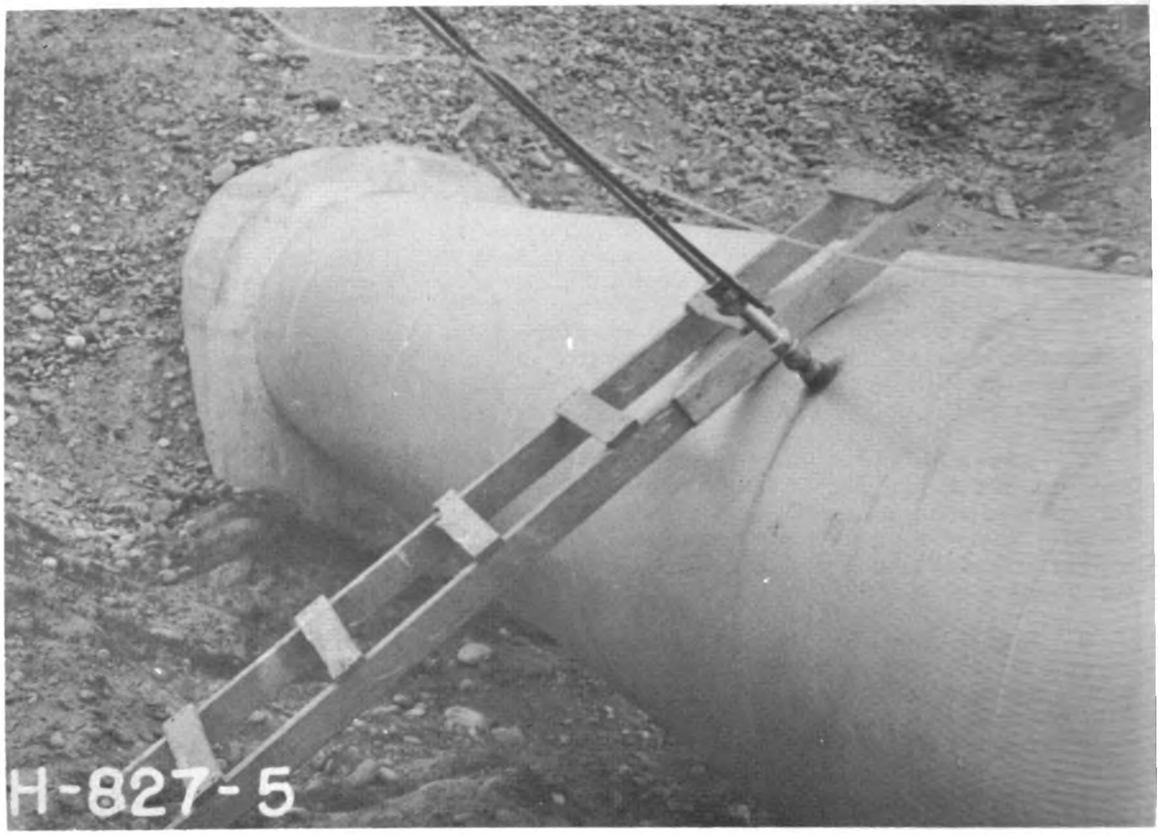


Figure 71. Measurement of velocity of water in a pipeline with a Cole Pitometer

Another type of pressure measuring device used in field studies is the recording Bourdon gage. The instrument has the advantage of recording, but it must be checked often to insure accurate results. Special pressure measuring instruments are sometimes necessary, particularly for use in isolated places or where the pressure changes rapidly. An electrical gage is generally used in these circumstances. General characteristics of design and fabrication of such equipment have been covered under the section "Special Instrumentation." A pressure cell in circuit with an oscillograph is commonly used in field studies where rapid fluctuations occur. This equipment is shown in Figure 21.

3. Measurement of water surface.--Where questions of freeboard, roughness coefficient, and head in free-flow structures are involved, accurate measurements of elevation of water surface must be made. Low velocities are found in the relatively narrow channels used in the laboratory where water surfaces are smooth and well defined, but at prototype structures the velocities may be high, the channels wide, and the water surface poorly defined or merely a mass of spray and aerated water. In obtaining the elevation of water surface at prototype structures, it is usually necessary to employ a stilling well. Ordinary staff gages mounted on the sides of structures are often used, and use of water level recorders is extensive. One type of gage that has been largely used in measuring elevations of water surfaces is the electric tape gage. This consists of a graduated cable or steel tape weighted at the lower end. The cable or tape is a part of an electrical circuit and is grounded to the structure. A light or meter indicates when the energized tape or cable touches the water surface. The instrument is inexpensive and portable.

4. Measurement of velocity.--Standard instruments, such as current meters and various types of Pitot tubes, are very generally used for velocity measurements. Salt or color also may be used, particularly where large flows are involved and velocities are such that observations can be made without a large amount of equipment. The instrument shown in Figure 71, known as a Cole pitometer, is a type of Pitot tube having upstream and downstream legs. It is reinforced to withstand velocities up to 30 feet per second. The pitometer is well designed and easily manipulated, particularly

in closed conduits.

Air velocity measurements are ordinarily performed by means of orifices, Pitot tubes, or commercial anemometers.

5. Other measurements.--Measurement of time is often necessary in field studies. The basic equipment is probably the stopwatch. Contrary to common opinion, the stopwatch is not often accurate over a long range of time. Where precise measurements are required, a recording oscillograph is used. This apparatus employs a tuning fork for control of the timing and is extremely accurate. To maintain the oscillograph in proper condition, the tuning fork is checked periodically against highly accurate time signals broadcast by the National Bureau of Standards radio station WWV at Washington, D. C. Broadcast frequencies, in megacycles, are 2.5, 5, 10, 15, 20, 25, 30, and 35. Station announcements are given in voice and by radio impulses at 5-minute intervals. Another timing device that is used and is quite accurate but less complicated than the oscillograph is a small direct-current motor having a jeweled escapement. The accuracy of this motor is sufficient for nearly all field time measurements.

Pendulums are also employed occasionally for time measurement. Another common method is to use 60-cycle current to drive ordinary clocks.

Although measurements of length and elevation are very important, no special discussion is considered necessary. Such measurements should actually be performed, not taken from drawings.

Another common measurement necessary in field studies and usually warranting special consideration is the determination of gate positions. Gate position indicators are generally not as accurate as is desirable, but satisfactory measurements may be made by use of some simple device, such as a leveling rod, if space permits; or an engineer's chain tape may be used in conjunction with a pointer that will indicate the true gate position.

Measurements of air and water temperatures, relative humidity of the air, barometric pressure, rainfall, and many other conditions that are important in field testing are made by conventional methods. At times, unforeseeable conditions require special attention. In any event, recording instruments should be used if practicable.

In the study of vibration or rapidly fluctuating water pressures, electronic equipment, described in a previous section, is employed. It should be realized that the forces involved in field tests are much greater than those encountered in the laboratory.

Photographic Equipment

Photographic equipment, both moving and still, is essential in field studies. Particularly advantageous is the ultra-high-speed camera which operates at 4,500 frames per second. The equipment should include such accessories as wide angle and telephoto lenses, filters, flash guns, and tripods. Photographs of hydraulic actions, conditions of structures, and arrangements and readings of instruments are invaluable as records.

Types of Investigations

Hydraulic investigations in the field are generally of the following types:

Investigation of field operating difficulties

Extension of studies to prototype level

Model-prototype conformance studies

Performance tests of hydraulic structures and machinery.

There are problems for which complete solutions cannot be obtained in the laboratory. Among these are: (1) negative pressures in conduits, gates, and spillways; (2) coefficients of roughness; (3) vibration; (4) turbulence; (5) high velocity flow and air entrainment; (6) scour in riverbeds. It is not considered expedient or necessary to include discussions of all of these in this writing. The four main types of studies will be discussed only to the extent necessary to guide technicians in formulating plans and arriving at conclusions. Illustrative examples are included.

1. Investigation of field operating difficulties.--These difficulties may arise because of some lack of information during the design period or because of unforeseen requirement. Observations on the outlet works at Alcova Dam exemplify study made for the purpose of investigating field operating difficulties.[33] In this instance, the two 84-inch needle valves operating under a maximum head of 162.5 feet discharge directly into a 20-foot diameter tunnel. Air was supplied to the tunnel through a 6- by 8-foot rectangular shaft extending from the top of the dam to the tunnel just downstream from the valves. Difficulty was occasioned by low pressure in the valve operating chamber during normal operation, the result being damage to equipment in the service elevator shaft.

33. Thomas, C. W. "Report on Inspection Trip to Study Operation of Outlet Works--Alcova Dam, Kendrick Project, Wyoming," Hydraulic Laboratory Report No. Hyd-42, November 1938.

The air velocities in the shaft for various valve openings were determined with a Pitot tube. The results of these tests are shown in Figure 72. Measurements at the structure also included pressure in the discharge tunnel, air temperature, barometric pressure, and quantity of water discharged. Observations were made of mechanical behavior of the valves, evidence of cavitation, hydraulic conditions in the stilling pool at the end of the discharge tunnel, and of other associated conditions. The results of these tests permitted a partial solution of the problem of air demand by improving the entrance to the air shaft and thereby minimizing the loss. Basic data were acquired from which to establish the relationships between air demands of valves discharging into tunnels.

This was a study in which very little special equipment was needed.

2. Extension of hydraulic studies to prototype level.--In this second type of field investigation the model is used to the utmost, but the final result can only be obtained through prototype study. An outstanding example is the investigation of the diffusion of ocean salinity in the estuaries of the Sacramento-San Joaquin River System in California. In this area the configuration of the channels and the tidal action cause saline and fresh waters to mix throughout the widths and depths of the flows. The conditions are in contrast with the normal action in estuarine channels where the more dense salt water enters the landward areas as wedges along the bottoms of the water courses, and the fresh water flows out to sea over the tops of these wedges.

This California invasion of saline waters into the delta has occurred annually during periods of low outflow as far back as records exist. Before agricultural and industrial development altered the condition of natural stream flow by depletions, the degree of salinity reached was considerably less. After these developments, however, the extent and degree of invasion increased until large losses were incurred by industrial, commercial, agricultural, and municipal interests.

Development of the agricultural areas in the San Joaquin Valley included two proposed schemes for transport of fresh water across the delta area. One involved construction of a costly artificial channel. The other contemplated use of natural channels with some artificial cross connection and channel improvement. Hydraulic problems involved in determination of whether harmful mixing would occur are very complex examples of unsteady flow, and since theoretical solutions for even the simplest cases of this type are questionable, a hydraulic model was constructed. The model could not be adjusted perfectly to meet all phys cal requirements, so many of the concl sions were subject to confirmation by pr totype tests.

The objectives of the field studies wer (1) to provide information for an order step-by-step procedure for construction a over-all development of the irrigation pro ect; (2) to supply information and practic methods of controlling the intrusion of oce salinity by requested releases of stored wa ters; and (3) to provide information for add tional development in the areas. The fie tests involved numerous discharge measure ments at various points in the delta. Thes were made with current meters at time when river stages were reflecting season changes in flow. The problem was aggra vated by tidal action. An essential part the study was the measurement of salinit at various stations to determine salinit gradients. The salinity meter, which is a electronic apparatus devised for this pur pose, has been described previously an is shown in Figure 22. Eleven salinity me ter stations were established. The record from these stations, properly averaged an interpreted, and used in conjunction with th distances between the stations, provide val ues for the salinity gradients. Distributio of salinity in any particular channel was me tered by a portable salinity meter mounte on a boat. The electronic analogue computer, previously described, and shown o Figure 23, has been of great assistance i interpreting the field data and in predicting the results of proposed channel changes.

The complexity of this salinity intrusion problem prevents a thorough discussion here. However, a sufficient description has been given to illustrate the fact that the model study had to be supplemented by prototype study before definite conclusions could be reached.

3. Model-prototype conformance studies.--Although in many instances the laws governing the relationship between the hydraulic model and its prototype have been thoroughly determined, there are occasions where these laws are unknown. In such circumstances, field tests of the prototype structure are required for verification of the predictions made from hydraulic model tests during the design studies. Such a conformance study was made on the coaster gate for the river outlets through Shasta Dam. 34

The coaster gate is designed to close any

34. Lancaster, D. M. "Model-Prototype Comparison of the Hydraulic Forces Acting on the Outlet Coaster Gate--Shasta Dam--Central Valley Project," Hydraulic Laboratory Report No. Hyd-233, Oct. 1948.

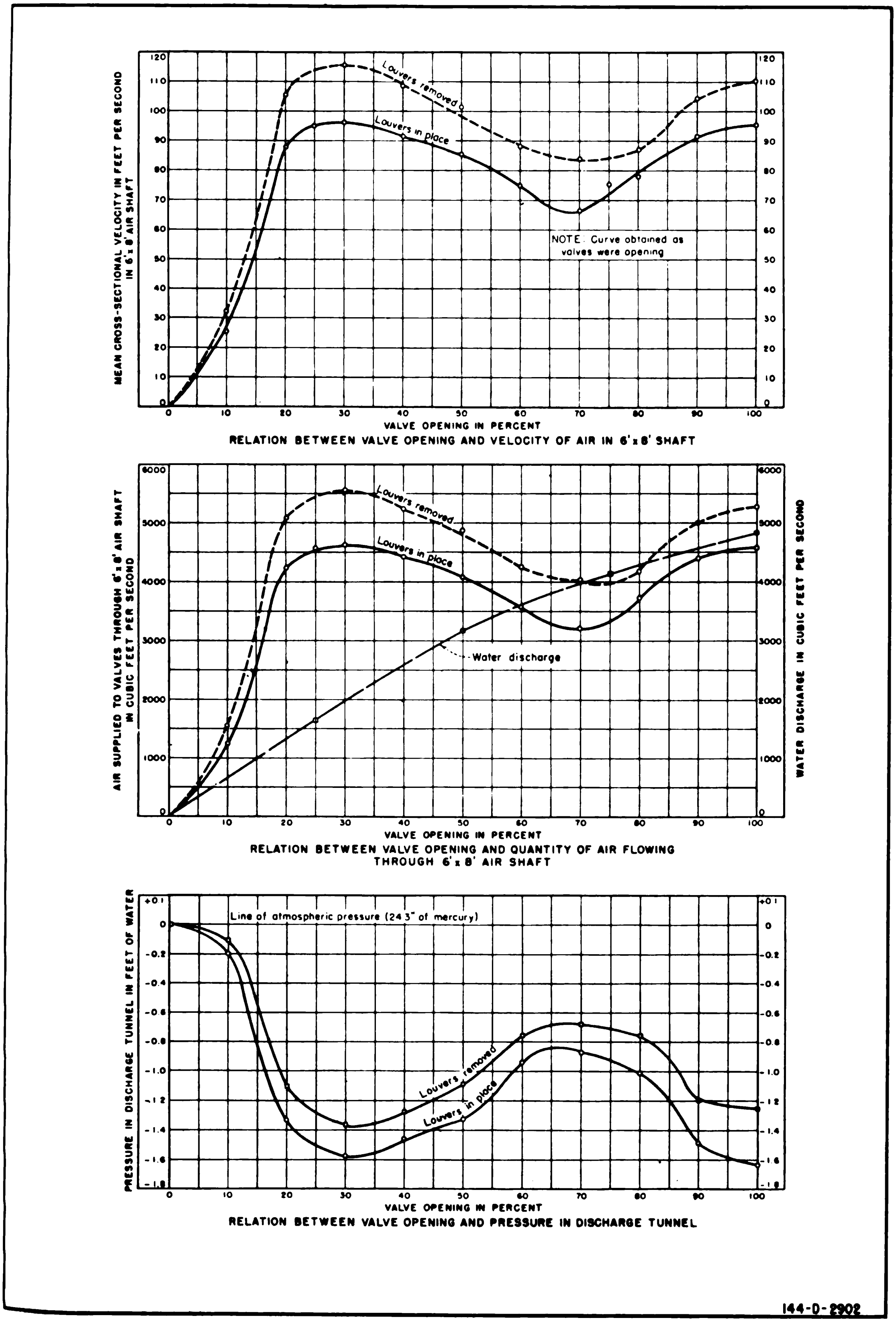


Figure 72. Field performance tests of needle valves, Alcova outlet works

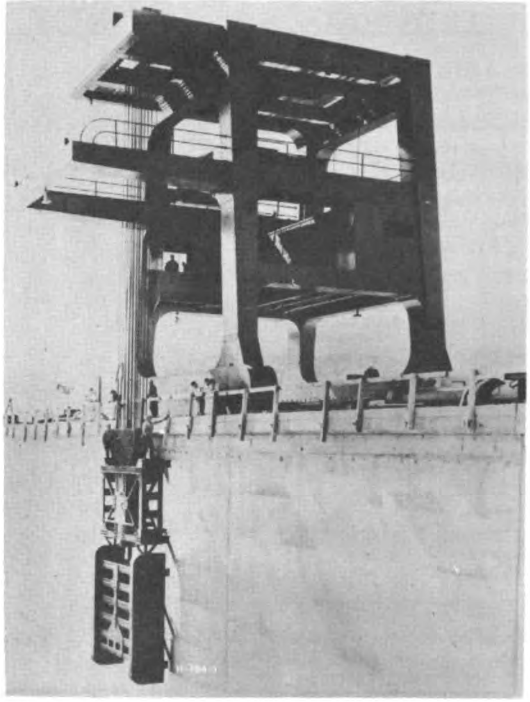


Figure 73. Coaster gate and handling equipment for river outlets, Shasta Dam

one of the 18 river outlets through the spillway section of Shasta Dam in the event of a failure of the control valve in the outlet or for valve maintenance. The gate is a rectangular steel structure having a skin plate riveted to the downstream side of horizontal beams which are supported by vertical girders and mounted on roller trains (Figure 73). In operation, the gate is lowered, by its own weight, in guides on the face of the dam. The outlets are arranged so that 14 sets of guides serve all 18 conduits.

Normally, the coaster gate is operated under balanced hydrostatic pressure with no flow through the outlet. However, design requirements were dictated by the conditions existing during emergency closure under maximum head. Under this condition, the gate would be subjected to large unbalanced pressures. The increased velocity under the gate would cause reduction in pressure and create a downpull force. The magnitude of this force could not be neglected in the design of the handling equipment, for this equipment was limited by the permissible load on the bridge across the spillway. The force could only be approximated analytically, since pressures on the bottom of gate are a function of the flow velocity, the shape of the bottom of the gate, and the gate opening. The magnitude of the downpull force was, therefore, dependent on the velocity distribution and the flow pattern underneath the gate, the pressure reduction at any point on the lower portion of the gate being equivalent to the velocity head at that point. So a hydraulic model was used to evaluate the hydraulic forces. These studies also enabled development of a new shape for the bottom of the gate, and this, together with a properly shaped recess in the face of the dam above the inlet to the conduit, reduced the downpull force from 260,000 to a safe value of 70,000 pounds. To minimize vibration caused by low pressures in the outlet entrance at partial gate openings, provisions were made in the prototype structure to admit air to the area immediately downstream from the coaster gate. The size of the air supply line was established at 10-inch diameter as a result of the hydraulic model tests.

The model studies were verified by field tests on the coaster gate in which an emergency condition was simulated by operating the gate over a lower river outlet through Block 45 with the control valve wide open.

a. Test equipment.--To evaluate the downpull, an SR-4 bonded resistance wire strain gage, Type A-5, was mounted on the hoisting gate stem. The change in resistance of the strain gage was measured by a portable strain indicator. The indicator consists essentially of a Wheatstone bridge, a galvanometer, and variable resistors for maintaining the balance of the bridge. A second strain gage of the same type was

Figure 74. Strain gages for measuring forces on coaster gate, Shasta Dam

secured to a dummy bar placed adjacent to the hoisting stem (Figure 74). By using this dummy gage all effects of temperature

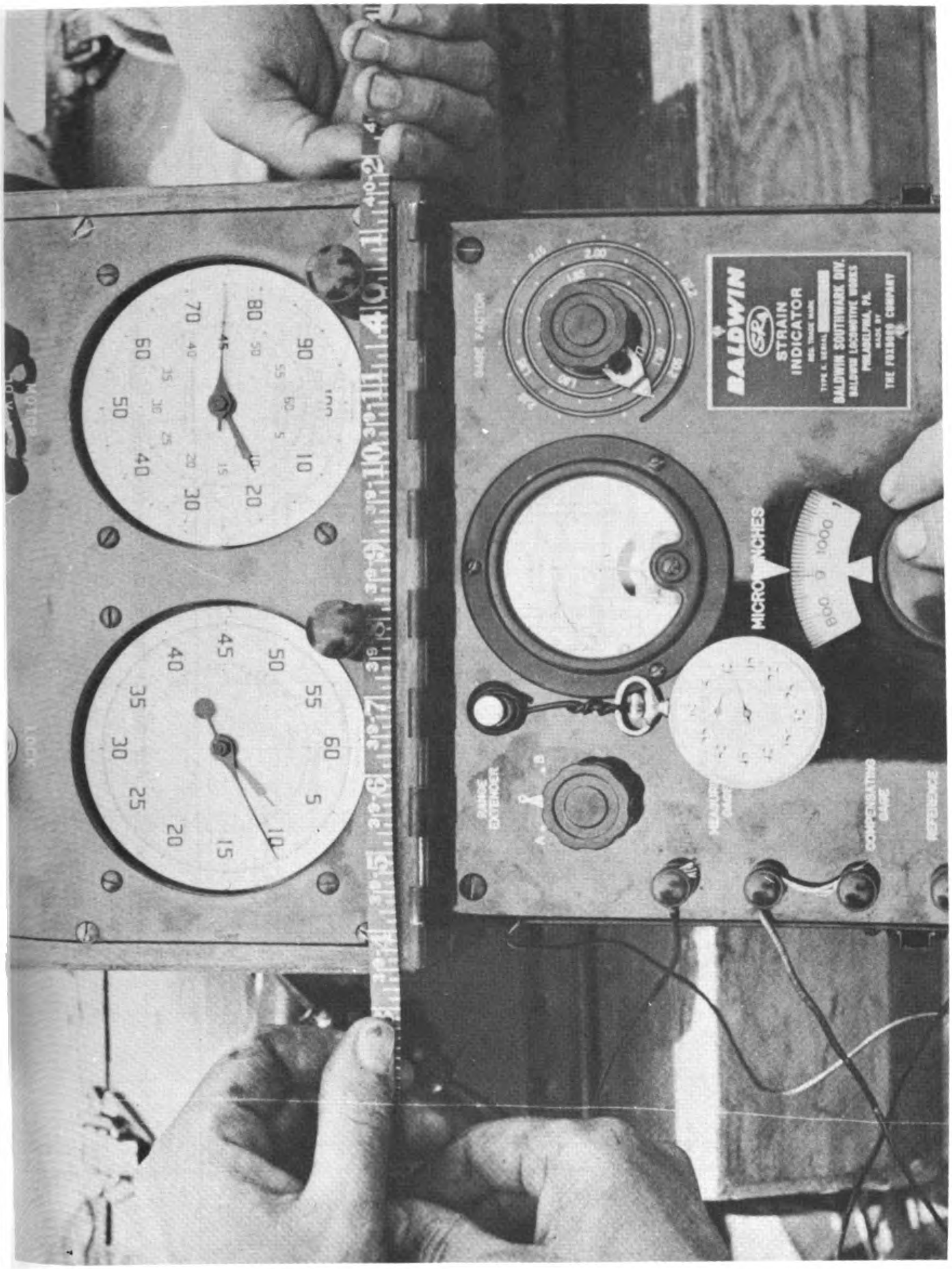


Figure 75. Method of recording data for outlet coaster gate tests, Shasta Dam

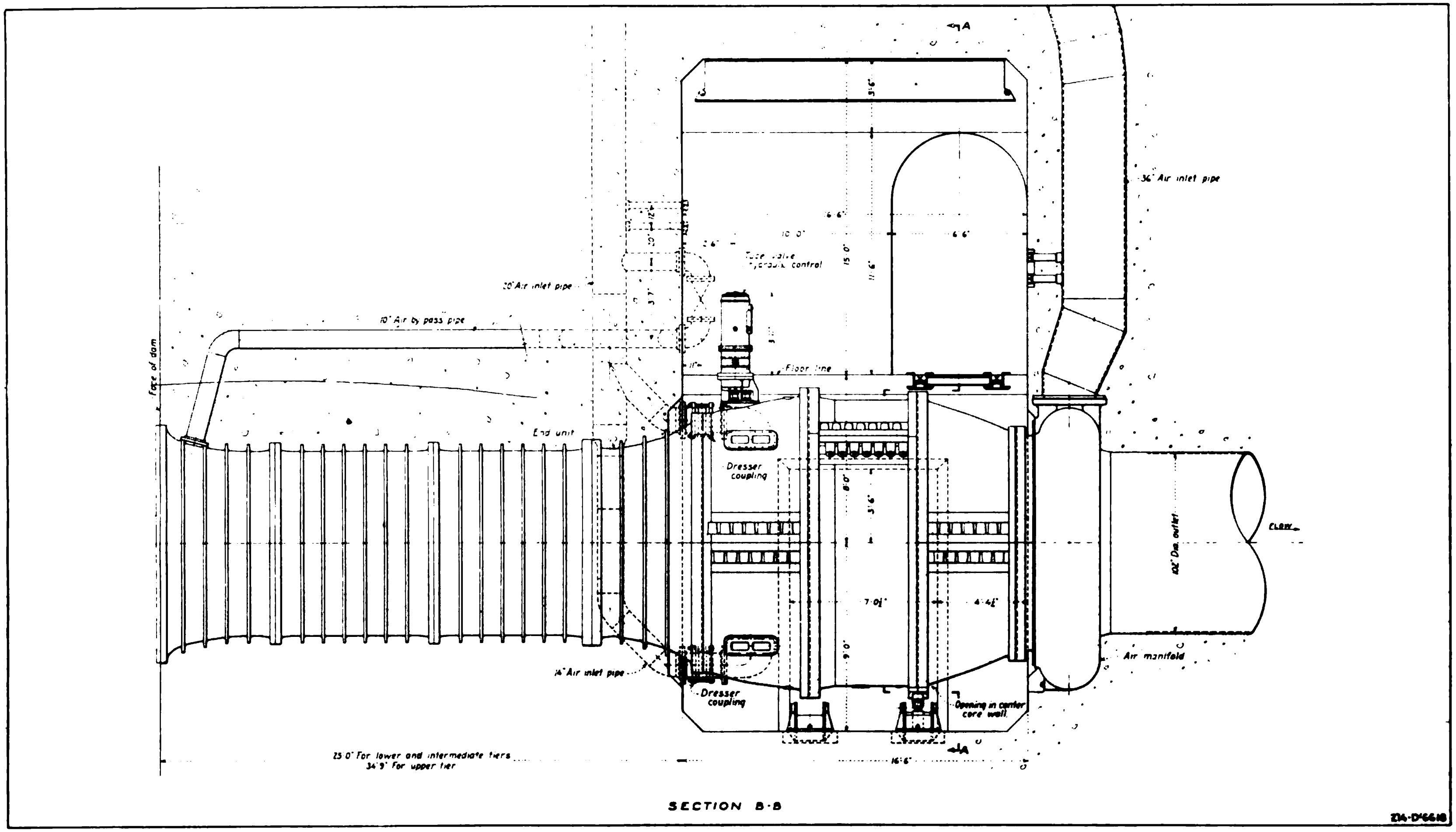


Figure 76. Air supply piping for coaster gate, Shasta Dam

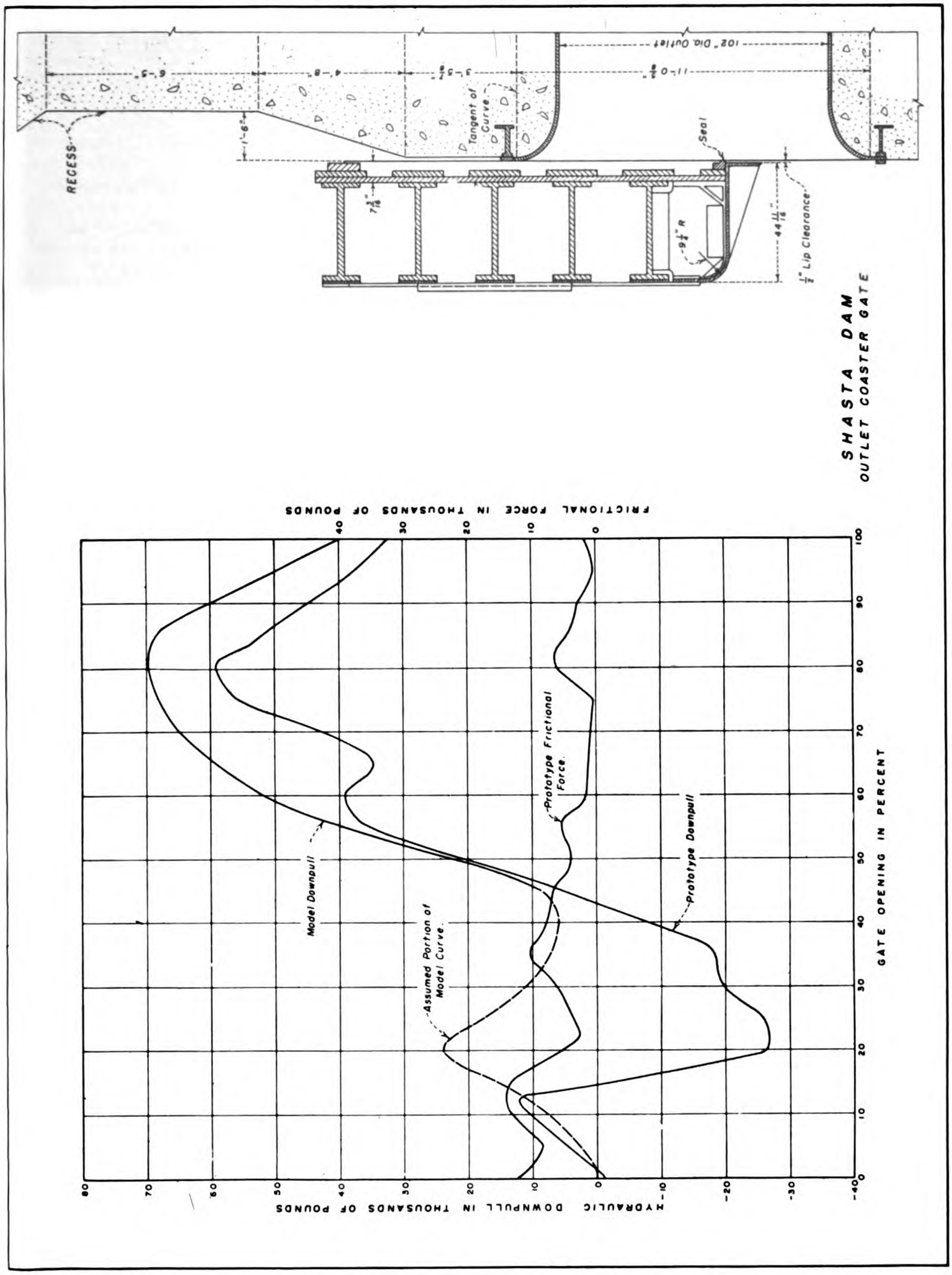


Figure 77. Model-prototype comparison of hydraulic downpull, and prototype frictional force

were automatically balanced out and the two resistance arms of the bridge were made more nearly identical.

The position of the gate was indicated by an engineer's chain tape secured on a cable, One end of the tape was anchored to the coaster gate and the tape was kept taut by a counterweight at the free end. This cable was rigged in such a way that the tape passed over the bridge across the spillway. The strain indicator and tape were grouped together, permitting the recording of the data with a 35-mm motion picture camera operating at the approximate rate of three frames per second. A stopwatch was included in the photographic field to verify the clock in the metering instrument. Figure 75 is an enlargement of one of the frames in the motion picture film. Simultaneously with the determination of the strain, the maximum quantity of air admitted immediately downstream from the coaster gate was ascertained by use of a Cole pitometer. The orifice tips of the instrument were placed in the center of a 20-inch diameter pipe which, by means of a manually controlled valve, was connected to the 10-inch pipe leading to the crown of the conduit immediately downstream from the entrance (Figure 76). A recording was also made of the minimum pressure in the outlet immediately downstream from the coaster gate.

b. Test procedure. --The tests were performed by starting with the coaster gate in a closed position (normal operating valve wide open). The observations of strain, air velocity, and pressure were made while raising the gate to a position several feet above the entrance to the outlet where unbalanced hydraulic forces no longer acted on the gate. Strain measurements were again recorded while lowering the gate to the sealing position. The head on the centerline of the conduit during the manipulation of the gate was 265 feet. The maximum design head is 323 feet.

c. Test results. --The maximum hydraulic downpull acting on the coaster gate was in reasonable agreement with the value determined with the hydraulic model. The recess in the face of the dam (Figure 77) immediately above the entrance to the outlet was effective in equalizing the forces on the upper horizontal seal assembly. However, contradictory to predictions, this balancing action also occurred at the smaller gate openings where an uplift force existed. This force was not large enough to prevent satisfactory movement of the emergency gate, but it does exemplify the condition that could be attained wherein the gate would fail to function properly. A comparison of the results obtained from the prototype with those predicted from the hydraulic model is shown on Figure 77.

4. Performance tests on hydraulic structures and machinery. --Field performance tests are necessary in many instances to determine the operating characteristics of hydraulic structures and machines. The reasons for conducting such tests may be (1) to ascertain if design criteria have been met; (2) to provide a basis for settlement between the contractor and the purchaser; (3) to acquire data for future designs; or (4) to determine if additions or changes are necessary in the structure or machine.

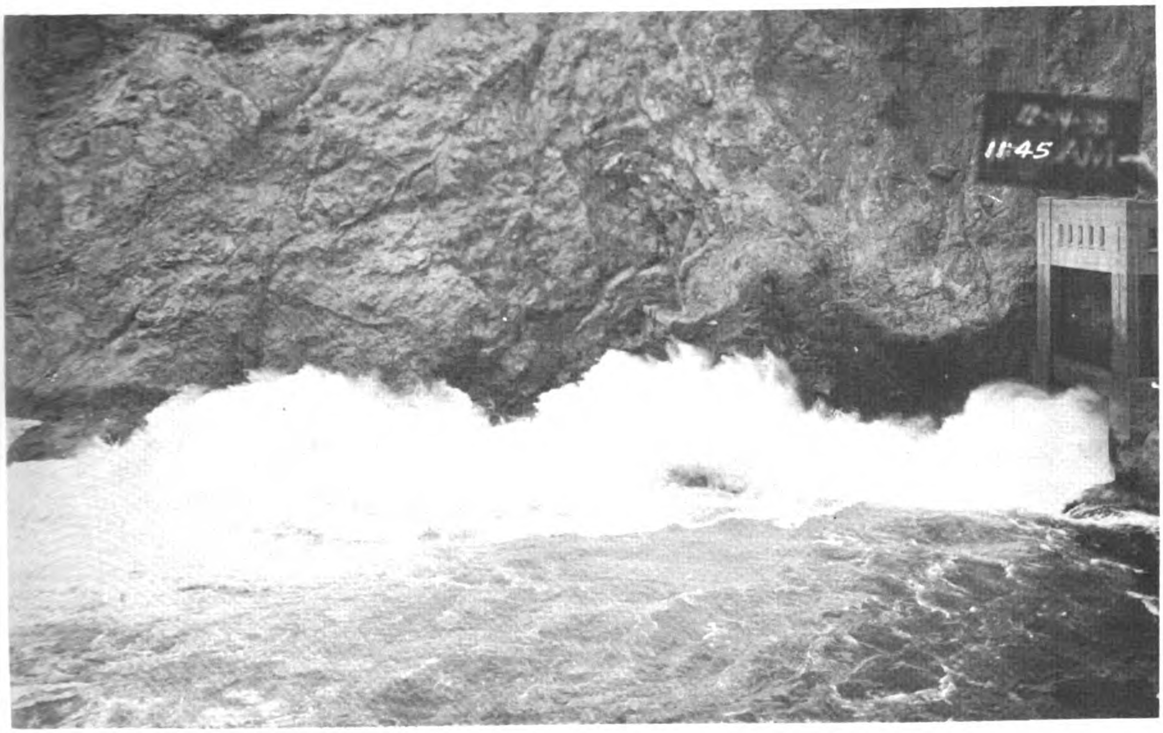
Field study of the outlet works for Hoover Dam is an example of a field performance test of a hydraulic structure. The outlet works consist of twelve 72-inch needle valves discharging into diversion tunnels. There are also twelve 84-inch needle valves in the canyon walls at a higher elevation. The outlets which discharge into the tunnels are arranged in batteries of six on each side of the river. The combined discharge of the six valves in each tunnel may be as much as 21,400 second feet, under a maximum head of 454 feet.

Extensive model tests were made of these installations to determine the arrangements of valves and numerous other features.[35] There was question concerning the necessity for supplying air downstream from the valves to replace that ejected by the water from the valves. Results of the model studies were not definite as to whether shafts would be necessary for supplying air. So field studies were initiated to settle the question.

The operating conditions in the Arizona Tunnel No. 3 were studied on a single day with flows increasing progressively from 10,000 second feet to full capacity of slightly over 20,000 second feet. The following day a similar study was made on the Nevada Tunnel No. 2 with flows increasing progressively from no flow to full capacity. During the program, pressure conditions were measured in the tunnels below the needle valves by means of mercury and water manometers, and still and motion pictures were taken of flow conditions in the river below the portals. There were also oscillograph measurements of the pressure and strain conditions at critical points along the penstocks. The tests required a complete record of reservoir, tailrace, and river elevations. As a result of the tests, it was not considered necessary to supply air to the needle valves in the tunnels downstream.

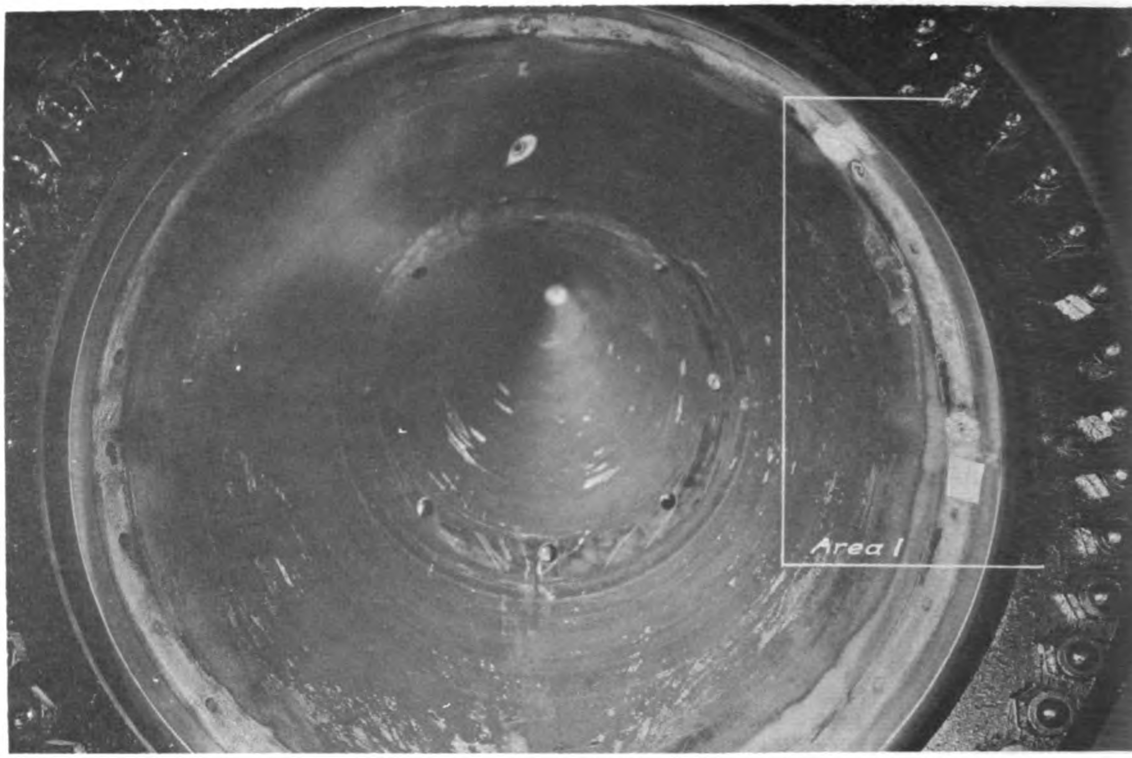
35. Blomgren, W. E. and others, "Tests to determine operating characteristics of tunnel-plug outlet works at Hoover Dam-Boulder Canyon Project," Hydraulic Laboratory Report No. Hyd-49, Feb. 1939.

A. Conditions below Arizona tunnel No. 3 at Hoover Dam with discharge of 20,500 cfs

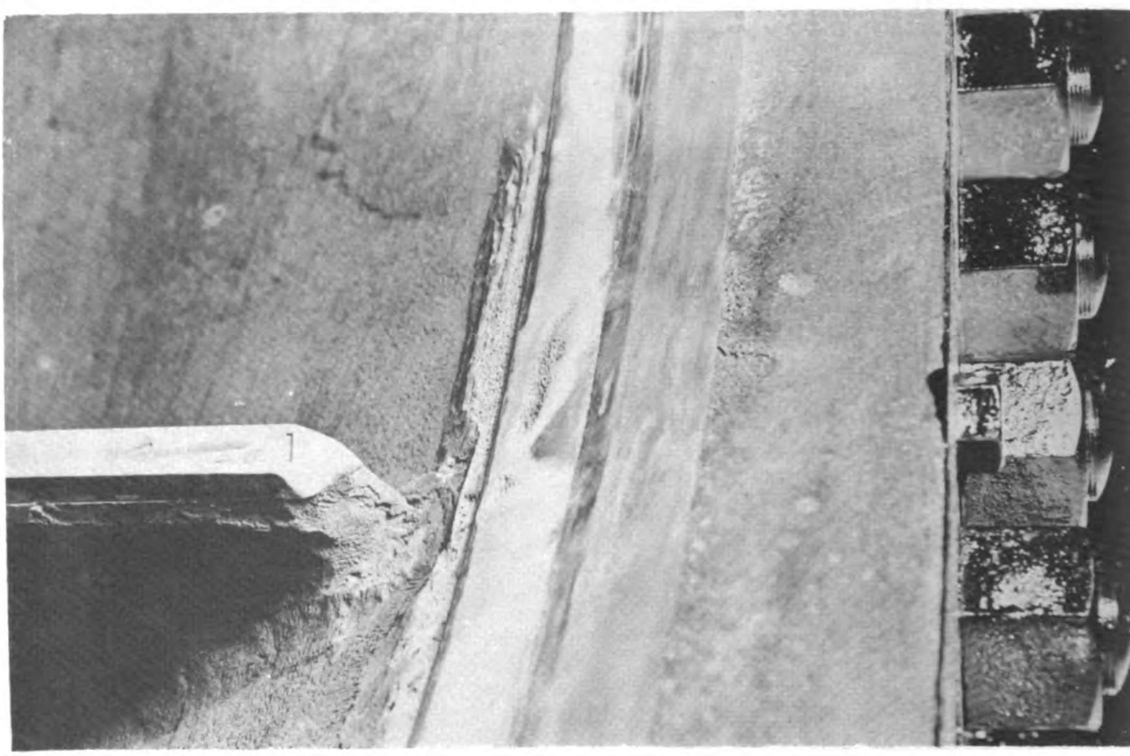


B. Conditions below Nevada tunnel No. 2 at Hoover Dam with discharge of 20,500 cfs

Figure 78. Outlet works flow conditions



A. Entire needle



B. Cavitation at downsteam end of needle guide

Figure 79. Cavitation on needle valve A-10 in Arizona tunnel plug outlet works at Hoover Dam

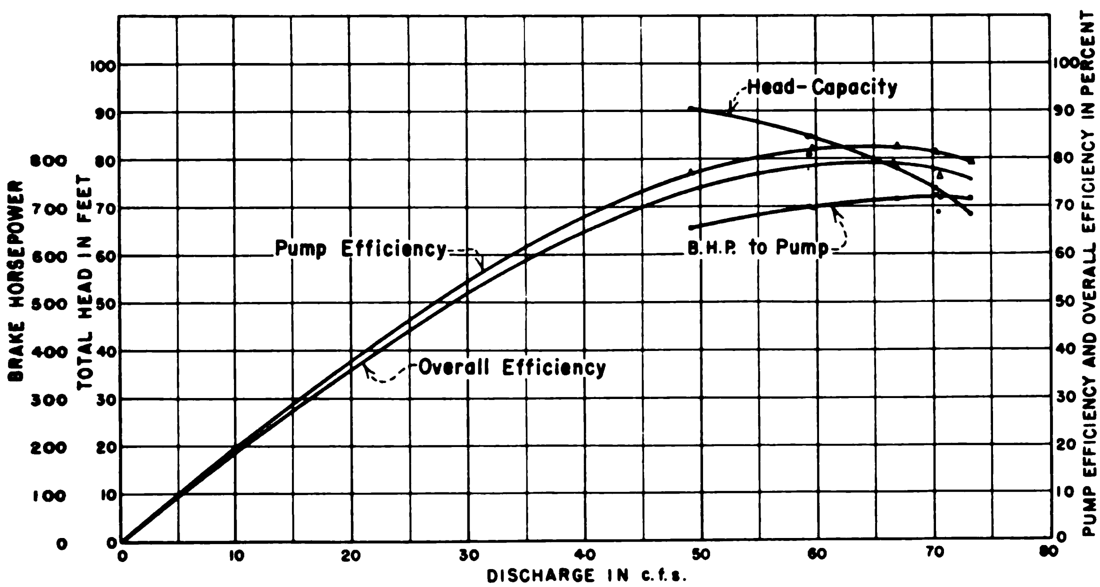


Figure 80. Pump performance curves

Inspection of the structures, after the tests, disclosed that severe cavitation was present in the needle valves. It was also obvious that the remnants of the lower cofferdam in the river channel downstream from the outlet works were causing some adverse flow conditions, particularly when the needle valves were discharging. As a result of the tests, it was recommended that certain river improvements be effected as soon as practicable. Other deficiencics in the structure were noted, but these were minor, and corrective measures were suggested immediately following completion of the tests. Pictures of flow conditions during the tests and cavitation conditions in the needle valves are shown on Figures 78 and 79.

Field performance tests on hydraulic machines include tests on turbines and pumps. The procedures and techniques for testing hydraulic machinery are very well covered in the Power Test Codes, 1949, Hydraulic Prime Movers, PTC 18-1949, published by the American Society of Mechanical Engineers, and in "Standards of Hydraulic Institute," published by the Hydraulic Institute, New York City. In these tests, it is important that the minimum requirements be met if possible. If not possible, an understanding among all interested individuals must be obtained.

A pump test was conducted at Pumping Plant D of the Klamath Project. To establish as much of the characteristic performance curves as possible, the test was made at various heads. The additional head, above that existing at the time of the test, was obtained by partially closing a temporary gate that controlled the elevation of the water surface in the tower in which the pump discharge lines terminated.

Data required for determination of pump performance are as follows:

a. Measurement of power input to the motors
b. Measurement of the total pumping head in feet
c. Determination of the rate of discharge of the pumps in feet per second

There should be additional observations concerning temperatures of the water pumped, motor room, air discharged from the motor winding, motor core iron, the motor thrust bearing. Vibration should also be observed. Line voltage and amperage, and exciter amperage during test should also be recorded.

Power input to the motors was measured by two calibrated watt-hour meters. The suction head was measured with a mercury manometer, and the discharge pumping head by a special 25-foot mercury gage. The discharge from the pumps was measured over a weir constructed in accordance with the standards of the Hydraulic Institute. Results of the tests of one unit are shown on Figure 80.

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