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UNITED STATES
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HYDRAULIC LABORATORY REPORT NO. 193

- - -

HYDRAULIC LABORATORY MANUAL

by

J. N. Bradley

- - -

Denver, Colorado
February 1, 1946

UNITED STATES
DEPARTMENT OF THE INTERIOR
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UNITED STATES
DEPARTMENT OF THE INTERIOR
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Branch of Design and Construction
Engineering and Geological Control
and Research Division
Denver, Colorado
February 1, 1946

Laboratory Report No. 193
Hydraulic Laboratory
Compiled by: J. N. Bradley

Reviewed by: D. J. Hebert

Subject: Hydraulic Laboratory Manual.

ORIGIN AND DEVELOPMENT OF HYDRAULIC LABORATORIES

1. Origin. Hydraulic model testing had its inception in the Bureau of Reclamation when, in August 1930, a staff of 12, including engineers, carpenters, and laborers, was organized in the hydraulic laboratory of the Colorado Agricultural Experimental Station at Fort Collins, Colorado. The magnitude of the appurtenant structures for Boulder Dam, at that time unprecedented, was the motivation of the laboratory. The expansion of hydraulic plant and personnel in the ensuing years is a manifestation of the success with which the studies of many hydraulic structures have supplied vitally needed information to the design department.

Of the first models in the Fort Collins laboratory, one was of the shaft spillways originally proposed for Boulder Dam. The studies proved conclusively that this type of design was unsuitable for the anticipated operating conditions. During the following two years a type of side-channel spillway was developed through modification of various models until a satisfactory design was obtained.

2. Development. Since 1930, hydraulic laboratories have been established and operated in various places as particular needs arise. During the clement seasons from 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 was available. It was in that laboratory that the final design of the side-channel spillways for Boulder Dam was studied to a scale of 1:20; a complete model of the Imperial Dam and its appurtenant structures to a scale of 1:40 was studied for two seasons to develop the final arrangement of the structures;

and the final design of the Grand Coulee spillway bucket was studied on a scale of 1:15. A hydraulic laboratory was maintained continuously from 1934 to 1937 in the basement of the Old Customhouse in Denver, Colorado, in which many of the smaller structures, designed by the Bureau during that period, were studied to evolve satisfactory and more efficient designs. In 1937, when the addition to the new Customhouse in Denver was completed, the equipment in the Old Customhouse was moved into the present laboratory. The Fort Collins laboratory building was expanded to about fourtimes its original size in 1935 to meet the ever increasing load of assignments. However, with the new laboratory established in the Customhouse, the personnel from Fort Collins were gradually absorbed until the fall of 1938 when the Bureau completely discontinued its operation of the Fort Collins laboratory. In 1939 laboratory facilities were installed in the Arizona canyon wall outlet house at Boulder Dam to utilize a head of 350 feet and a discharge of 200 second-feet in the study of the final design of the tube valves developed to control the outlets in Shasta Dam. That laboratory was operated for several months in 1940, in 1941 and again in 1945, its program being geared to studies made in the Denver laboratory on a smaller model.

PRESENT LABORATORY EQUIPMENT AND FACILITIES

3. Laboratory arrangement. Of the hydraulic laboratories mentioned above, the Customhouse laboratory in Denver, is the only one in operation at the present time. It is equipped to accommodate any type of hydraulic experiment that can be housed in the available space. For this reason, and also because of the fact that it has been given little publicity in the past, a fairly complete description of the laboratory, equipment, and accompanying instrumentation will be described.

A plan of the Customhouse laboratory in Denver, Colorado, is shown on figure 1. The laboratory has the disadvantages of being located in the basement of an office building in which the head-room is limited and the space is interrupted by columns. This was unavoidable even though the

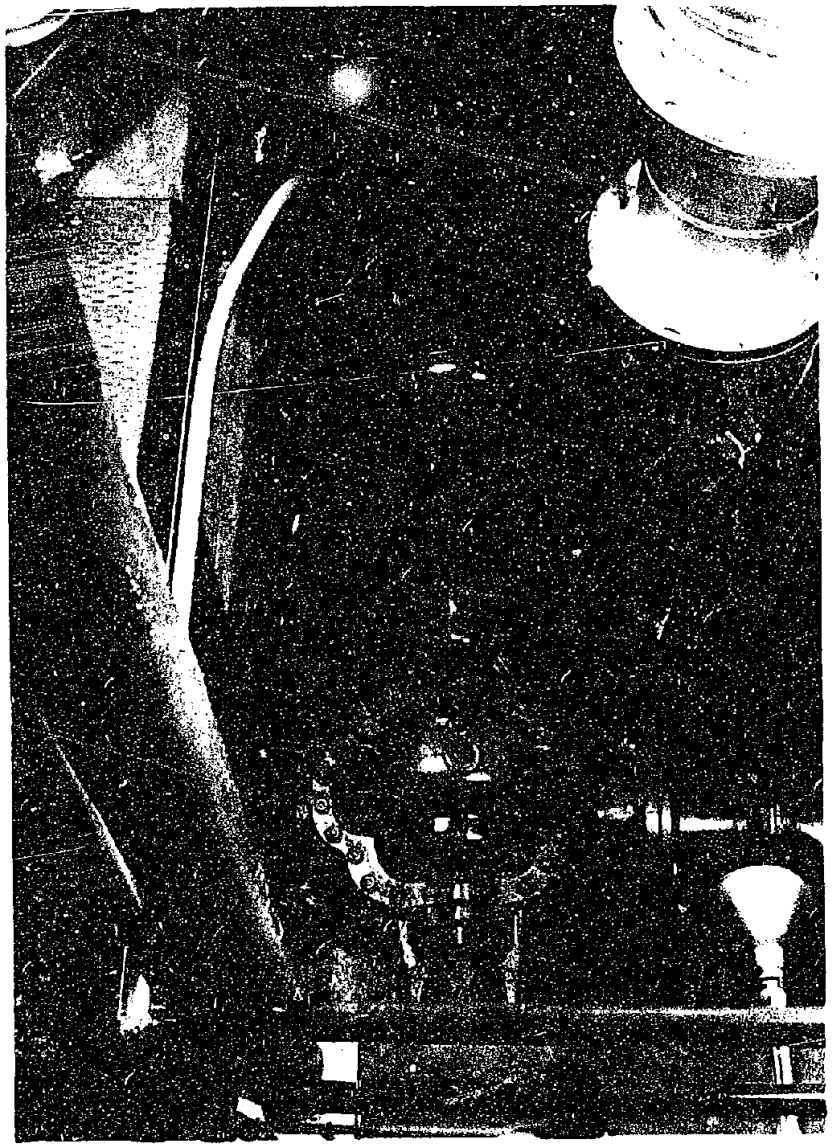
8" 30 HP PUMP
6" 3 WAY MOTOR OPERATED COCK
3" ORIFICE METER
VENTURI GAGE BOARD
4" VENTURI METER
6" VENTURI METER
8" VENTURI METER
ORIFICE METER GAGE BOARD
6" AND 8" THROTTLING VALVES
6" AND 8" 3 WAY MOTOR OPERATED COCKS
6" AND 8" THROTTLING VALVES
VENTURI GAGE BOARD
4" VENTURI METER
8" VENTURI METER
4" HP VERTICAL PUMP
4" 10 H.P. PUMP
DRAIN
PIPETTE TANK
DEPTH 5.0 FEET
DEPTH 3.0 FEET
DEPTH 3.0 FEET
DEPTH 3.0 FEET
OVERHEAD FEEDER LINES
VOLUMETRIC CALIBRATION TANK
DEPTH 7.0 FEET
DRAIN
DEPTH 5.0 FEET
DRAIN
CONTROLLER
VENTURI GAGE BOARD
12" 90 H.P. PUMP
DRAIN
DEPTH 4.25 FEET
DEPTH 4.25 FEET
DEPTH 4.25 FEET
ELEVATOR
DRAIN
CONSTANT LEVEL TANK
OVERHEAD FEEDER LINES
8" AND 12" HYDRAULICALLY OPERATED GATE VALVES
8" VENTURI METER
12" VENTURI METER
DEPTH 5.0 FEET
DRAIN
120'-11"
51'-6"

laboratory was incorporated in the original plans of the building, however, location of the laboratory in the same building as the design office has the advantage that designers can visit frequently and keep in direct contact with the experimental work. Experience has demonstrated that a greater volume of work is submitted to the laboratory under these conditions with greater satisfaction to all concerned. A second advantage is that air and water temperatures in this building are practically constant throughout the year, thus two variables are automatically eliminated during testing.

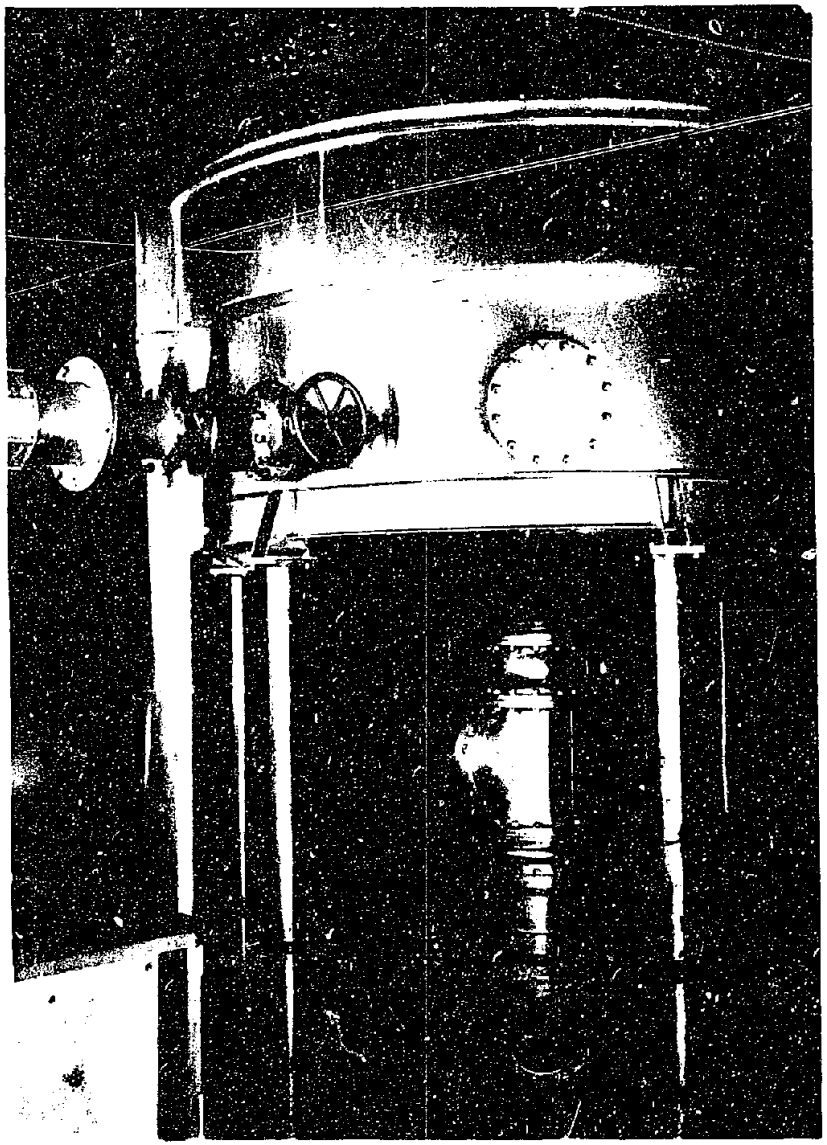
The laboratory storage consists of channels under the floor as shown in figure 1. These vary in depth and width and are interconnected. Water from the city mains is used for initial filling of the channels. Once filled, water is pumped from these channels, circulated through models, returned to storage, and recirculated. The majority of the channels shown on figure 1 are covered with steel floor plates.

4. Pumps and measuring devices. The main equipment consists of a 12-inch centrifugal pump, with a capacity of 10 second-feet, powered by a 90-horsepower, variable-speed, slip-ring motor, as shown in figure 2A. A constant-level tank, figure 2B, is used in conjunction with the pump as a surge suppressor or by-pass. The main line branches into an 8-inch and a 12-inch pipe in which are located venturi meters for measuring the discharge, figure 3A. Immediately downstream from the meters are hydraulically operated gate valves which are used for throttling the flow. The piping layout is shown in the lower left-hand corner of figure 1. Beyond the valves, overhead feeder lines conduct the water to the various models. All bleeders, gages, and hydraulic and electric controls have been concentrated on one central board for this particular portion of the laboratory system, figure 3B.

The equipment on the opposite side of the laboratory consists of an 8-inch pump with a capacity of 5 second-feet and a 6-inch pump with



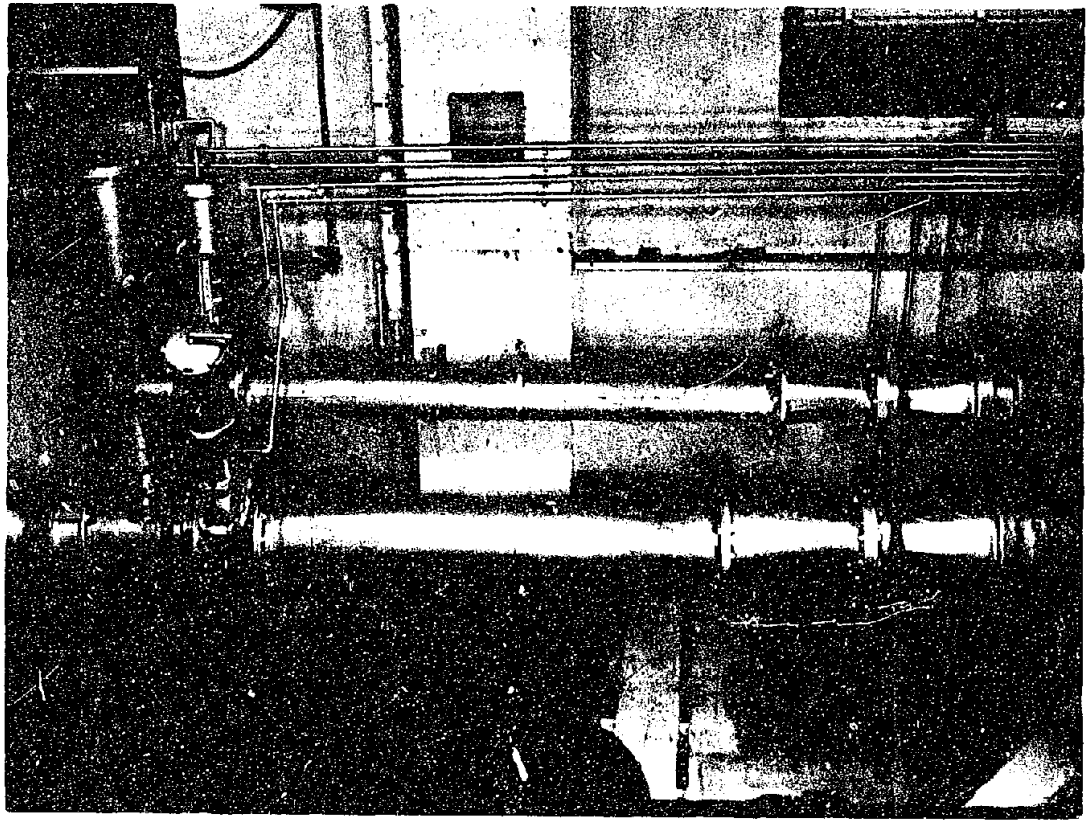
A - Twelve-Inch Laboratory Pump



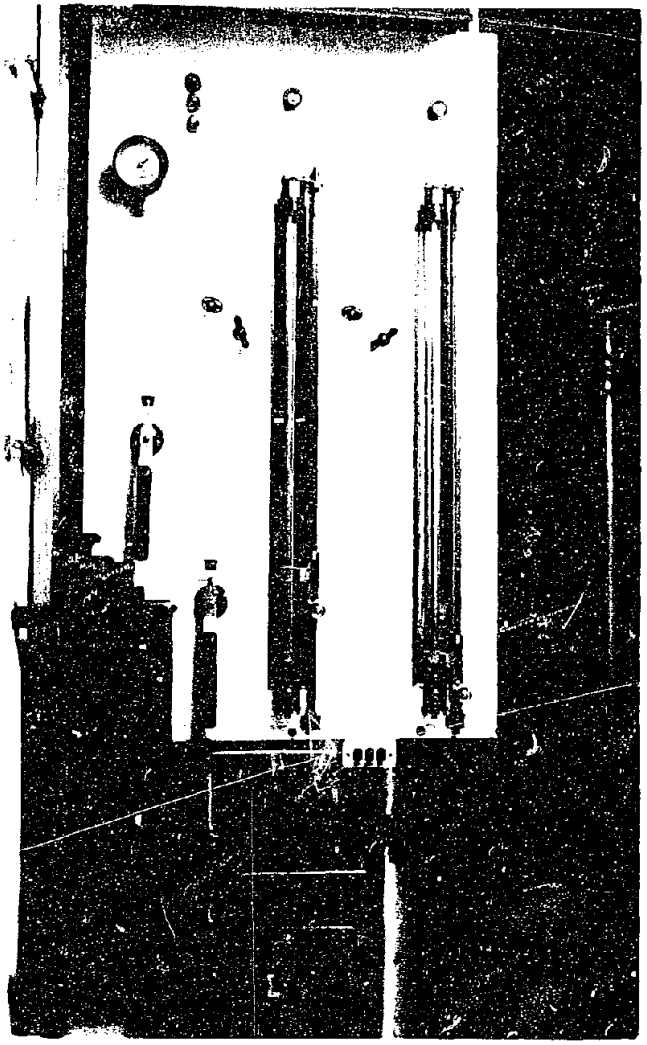
B - Laboratory Constant Level Tank

MAIN LABORATORY PUMPING SYSTEM

FIGURE 2



A - Eight- and Twelve-Inch Venturi meters with accompanying hydraulically operated throttling valves.



B - Gage Board and Controls for Eight and Twelve-Inch Venturi Meters.

MAIN LABORATORY PUMPING SYSTEM

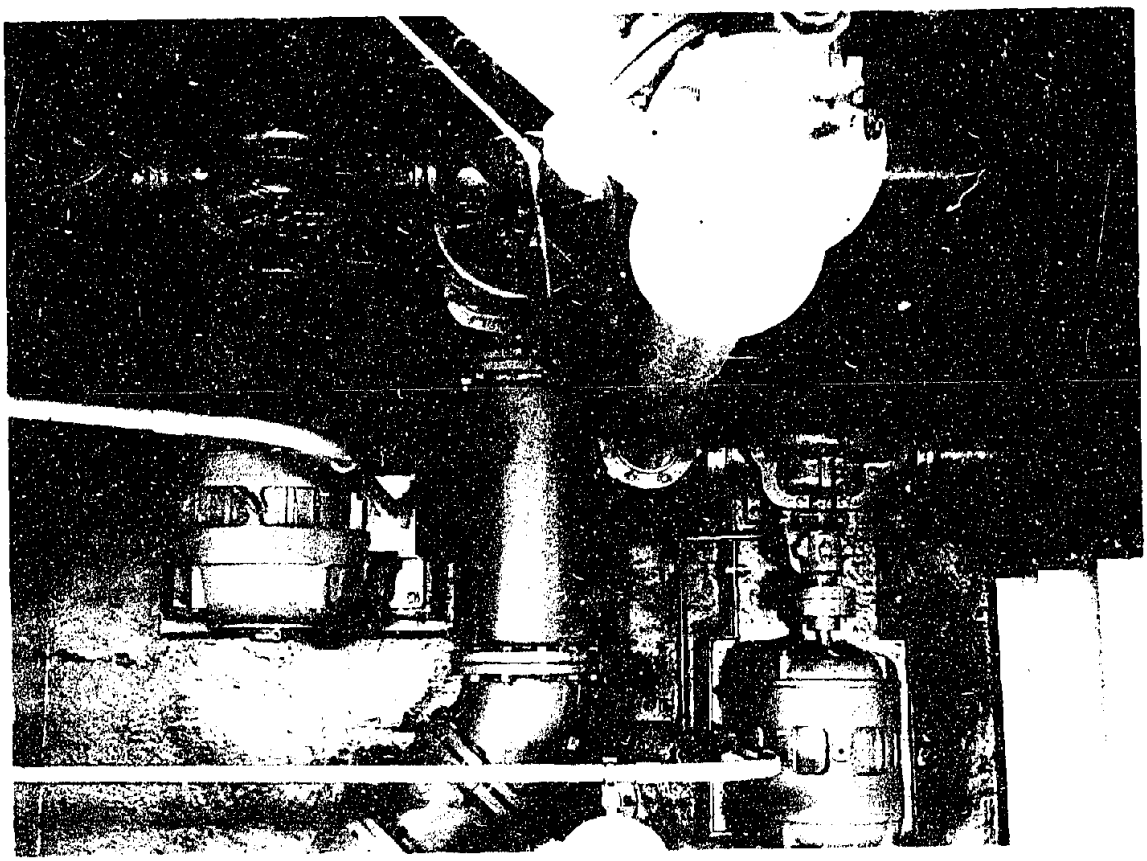
FIGURE 3

a capacity of 2.5 second-feet, both driven at constant speeds, figure 4A. These pumps discharge through a bank of three Venturi meters installed in 4-, 6-, and 8-inch lines, figures 1 and 4B. A 3-inch line also parallels this bank in which a variable orifice meter is employed to measure small discharges down to a few gallons of water a minute. The system is so arranged that by means of three 3-way motor-operated plug-valves, any one of the four meters can be connected directly to either pump, or the two pumps can be run simultaneously using different meters for measurement of the discharges. This system is flexible and all controls are centrally located on or adjacent to the gage board, figure 4B. The small lights at the top of the board indicate the positions of the three 3-way valves. A series connection from the discharge side of the 12-inch pump to the suction side of the 8-inch pump adds to the flexibility of the system, figures 1 and 4A. This connection makes it possible to develop a head of approximately 180 feet.

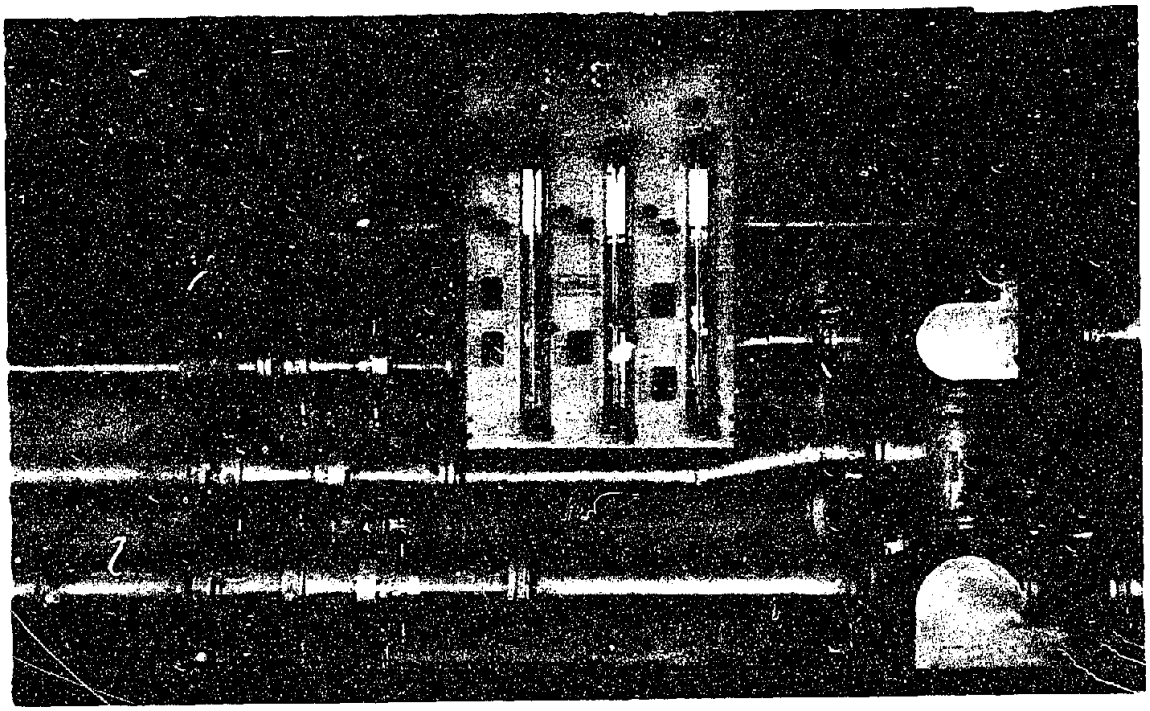
A third and smaller system located in the far end of the laboratory, the upper right-hand corner of figure 1, consists of an 8-inch vertical pump of 3 second-foot capacity which discharges through either a 1.5-inch flow nozzle or a 6.5-inch modified Venturi meter. All controls are located in close proximity to a gage board similar to the other two.

Each meter in the laboratory is connected directly to a separate mercury manometer gage. By so doing, leakage from valves, cocks, and by-passes is eliminated, which adds to the reliability of the system. The manometers are of the pot type with ½ inch inside diameter, heavy-duty, gage glasses. This size of glass tube is large enough to make the manometers self-damping. All metal parts are of stainless steel to resist rust and amalgamation by mercury. The gages are a special laboratory design as no satisfactory commercial manometer could be purchased.

It can also be noted from figure 1 that each of the three supply systems are interconnected in such a way that any one of the four pumps can supply water to any point in the laboratory. These interconnected



A - Eight- and Six-Inch Laboratory Pumps



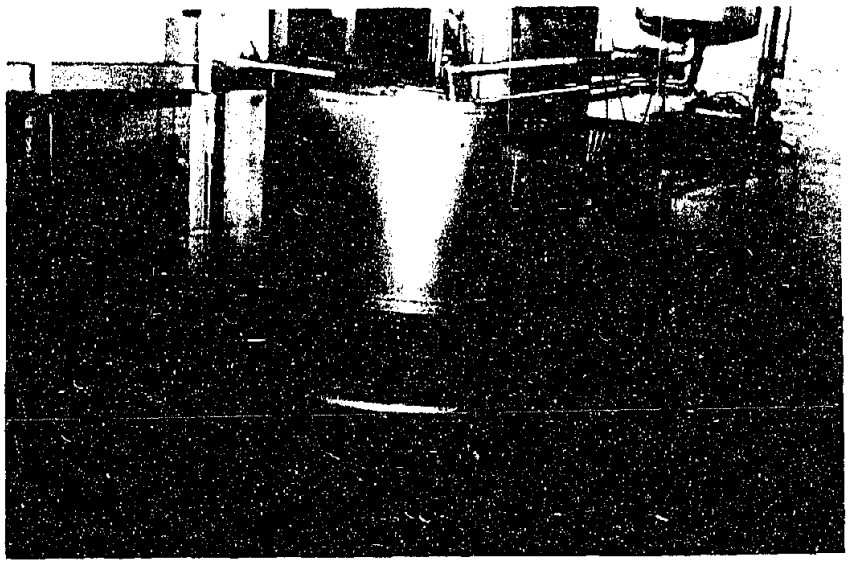
B - Motor and Control - For Eight- and Six-Inch Laboratory Pumps

SECONDARY LABORATORY PUMPING SYSTEM

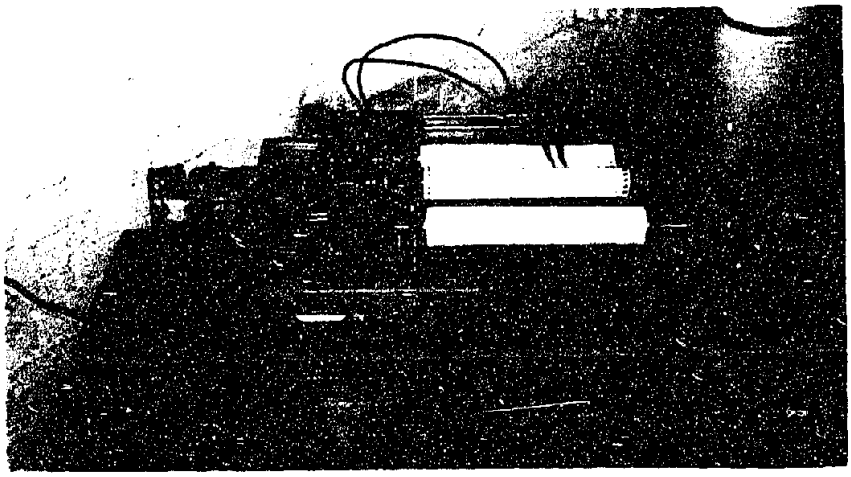
lines also lead to the laboratory calibration tank, figures 1 and 5A, by which all meters in the laboratory are calibrated and checked at definite intervals. A schematic diagram of the piping system and pumping equipment is shown on figure 6 together with the characteristic curves for the pumps, and calibration curves for the Venturi meters. The discharge through any meter is obtained by reading the corresponding manometer gage and referring to the curves on figure 6. A similar set of calibration curves for the orifice meter is shown on figure 7. The orifice meter manometer is shown at the extreme left in figure 4B.

5. Calibration equipment. The volumetric calibration tank, figure 5A, has a capacity of approximately 400 cubic feet. The size was limited by the volume of the laboratory reservoir under the floor. The limited size of the tank, however, is compensated for by reliable and extremely accurate timing methods. The volumetric tank, a section of which is shown on figure 8, resembles a chemist's pipette with a large body and small neck. By means of the swing spout, shown on the same figure, the inflowing water can be diverted into the volumetric tank or by-passed back to the reservoir. To make a calibration run, which would represent a point on one of the curves of figures 6 or 7, a steady flow is established through a Venturi meter with the swing spout in the by-pass position. By means of a pneumatic jack the swing spout is shifted, the tank filled to some point in the neck, and the swing spout 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 are taken from the mercury manometer gage accompanying the Venturi meter being rated. The volume of water in the calibration tank is determined by entering figure 8 with the water-surface reading obtained from a hook gage located in the neck of the tank and also the temperature of the water. A theromometer is permanently located in this tank. The timing can be accomplished in two ways. One source of time signals is a Seth Thomas pendulum clock equipped with a photoelectric cell, figure 9A. The signals are transmitted electrically to a chronometer located near the calibration

FIGURE 5



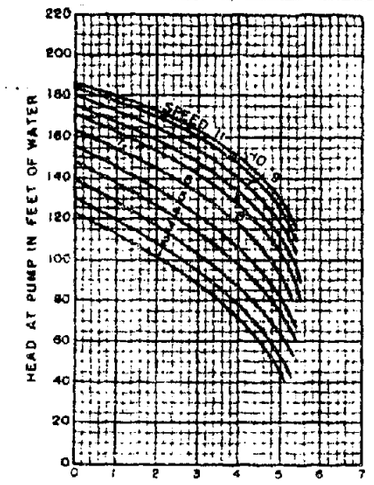
A - Volumetric Calibration Tank

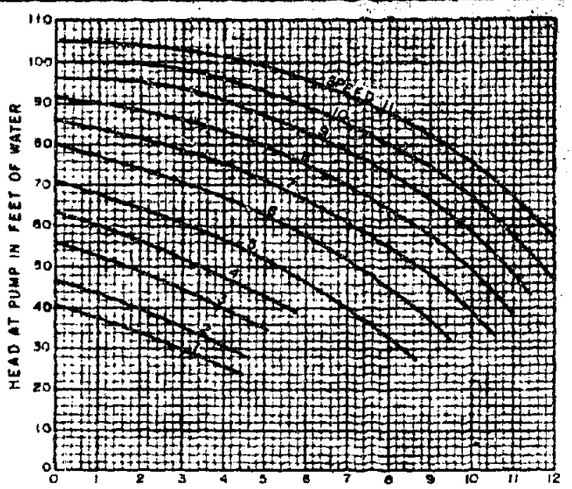


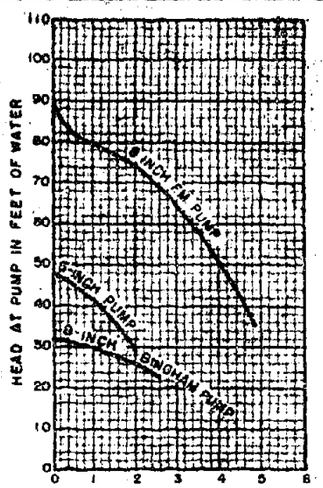
B - Chronograph for Recording Calibration Operation

C - Pipette Calibration Tank

EQUIPMENT FOR CALIBRATION OF LABORATORY METERS







HEAD-DISCHARGE CURVES FOR LABORATORY PUMPS

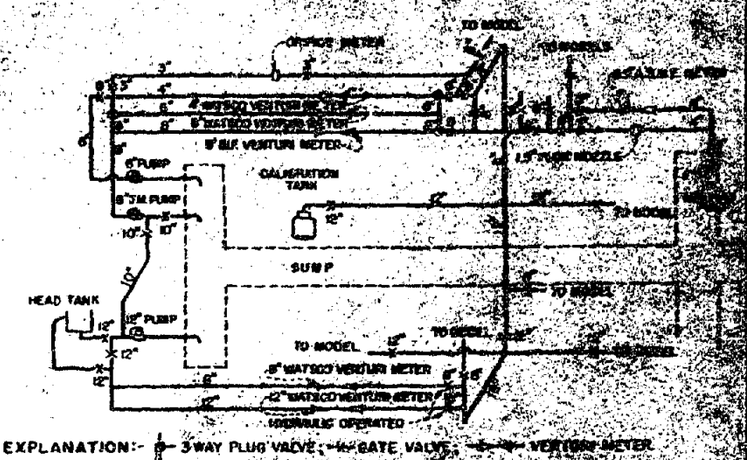
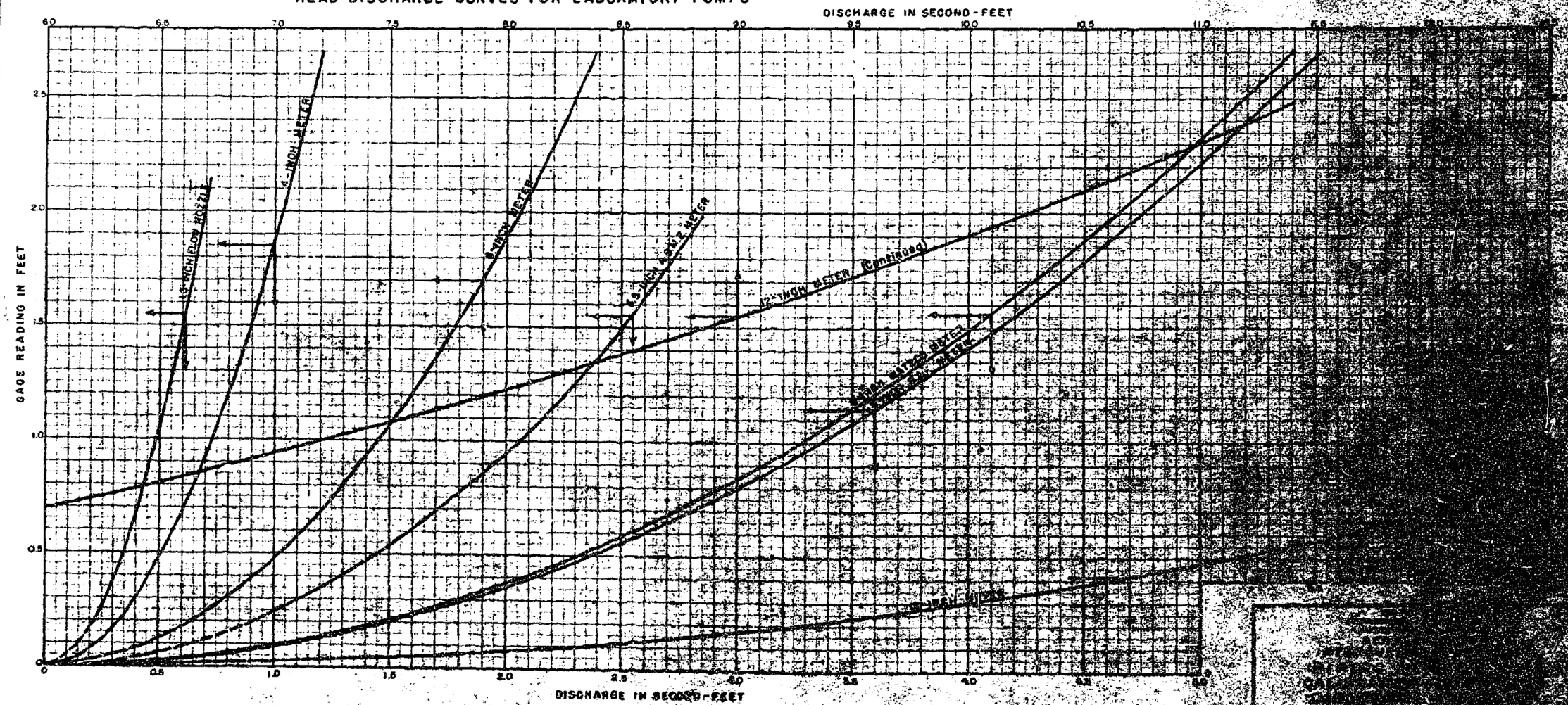
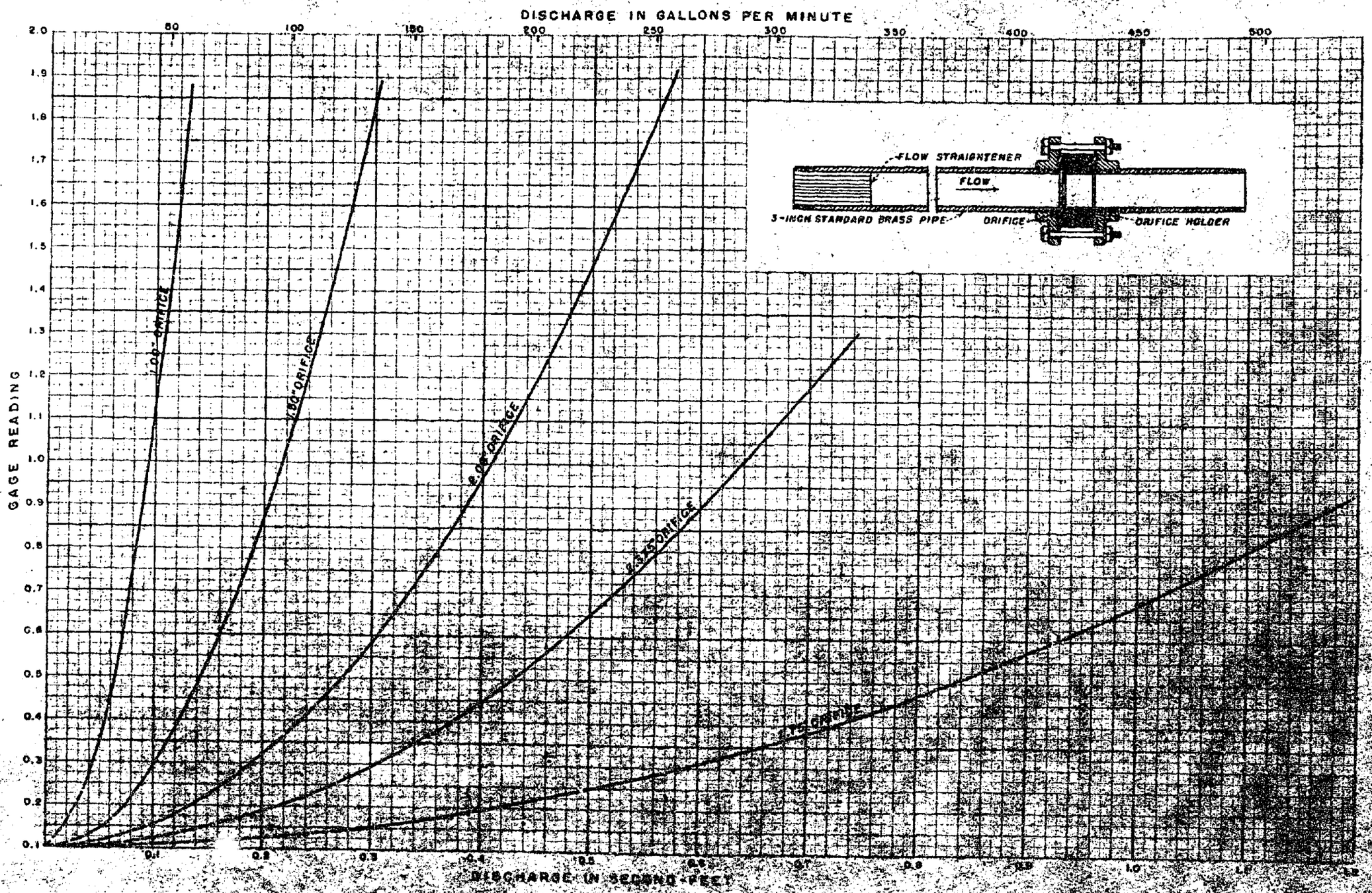


DIAGRAM OF PIPING SYSTEM FOR HYDRAULIC LABORATORY



CALIBRATION CURVES FOR LABORATORY VENTURI METERS

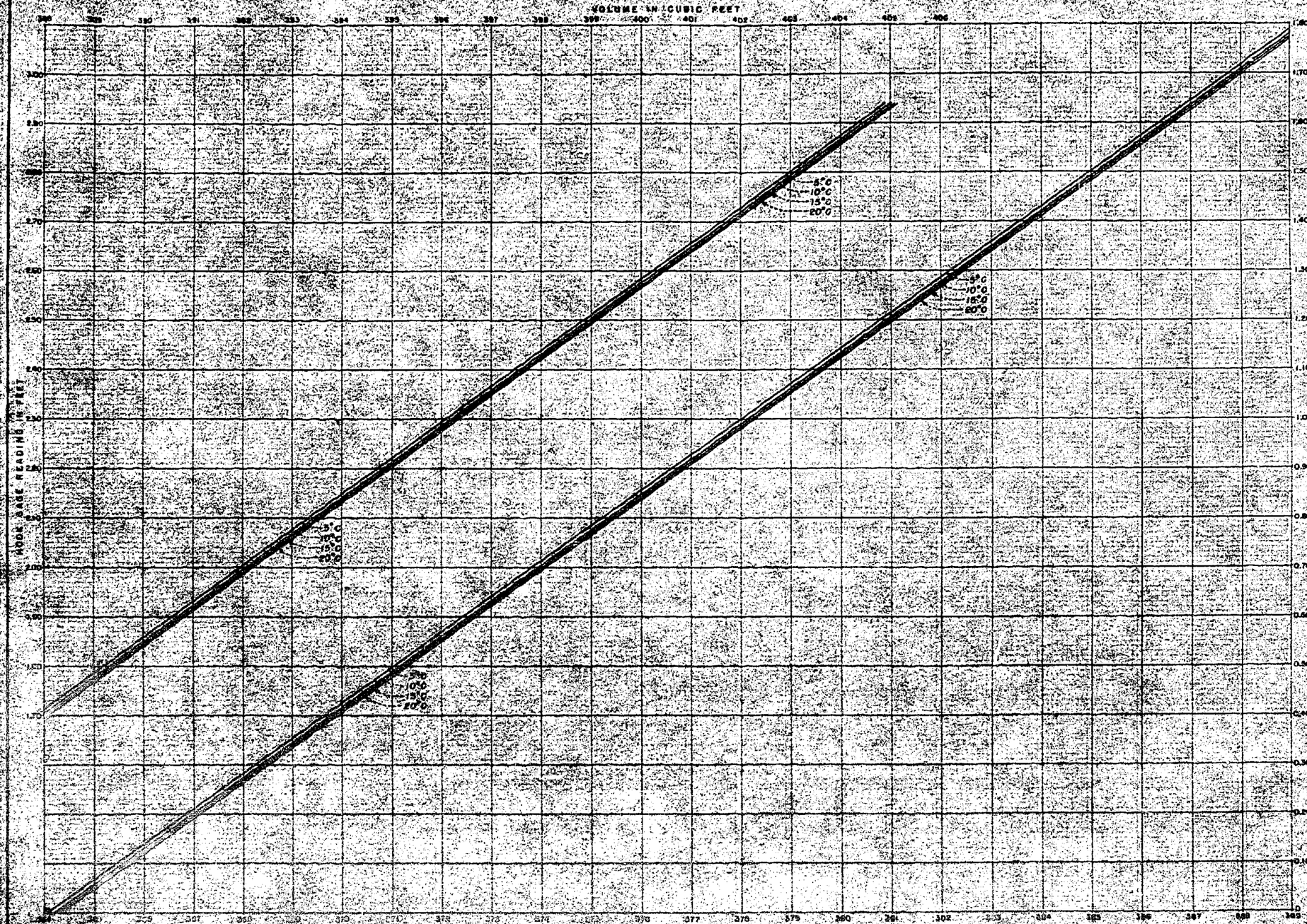
FIGURE 7



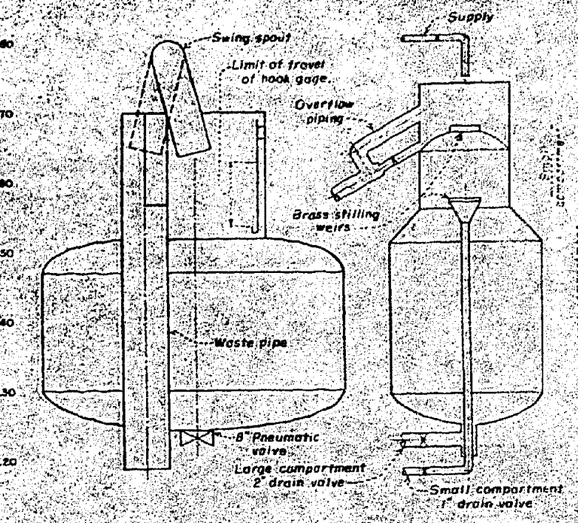
CALIBRATION CURVES FOR LABORATORY ORIFICE METERS

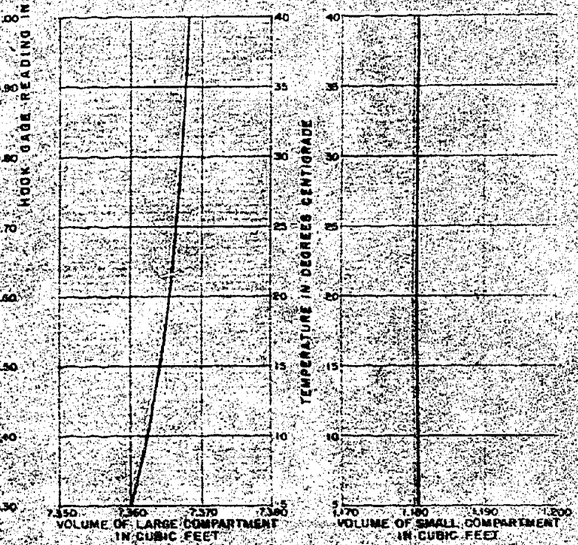
DENVER HYDRAULIC LABORATORY
CALIBRATION CURVES FOR
LABORATORY ORIFICE METERS

FIGURE 8



CALIBRATION OF VOLUMETRIC TANK





CALIBRATION OF PIPETTE TANK

Calibrations completed February 1, 1941.

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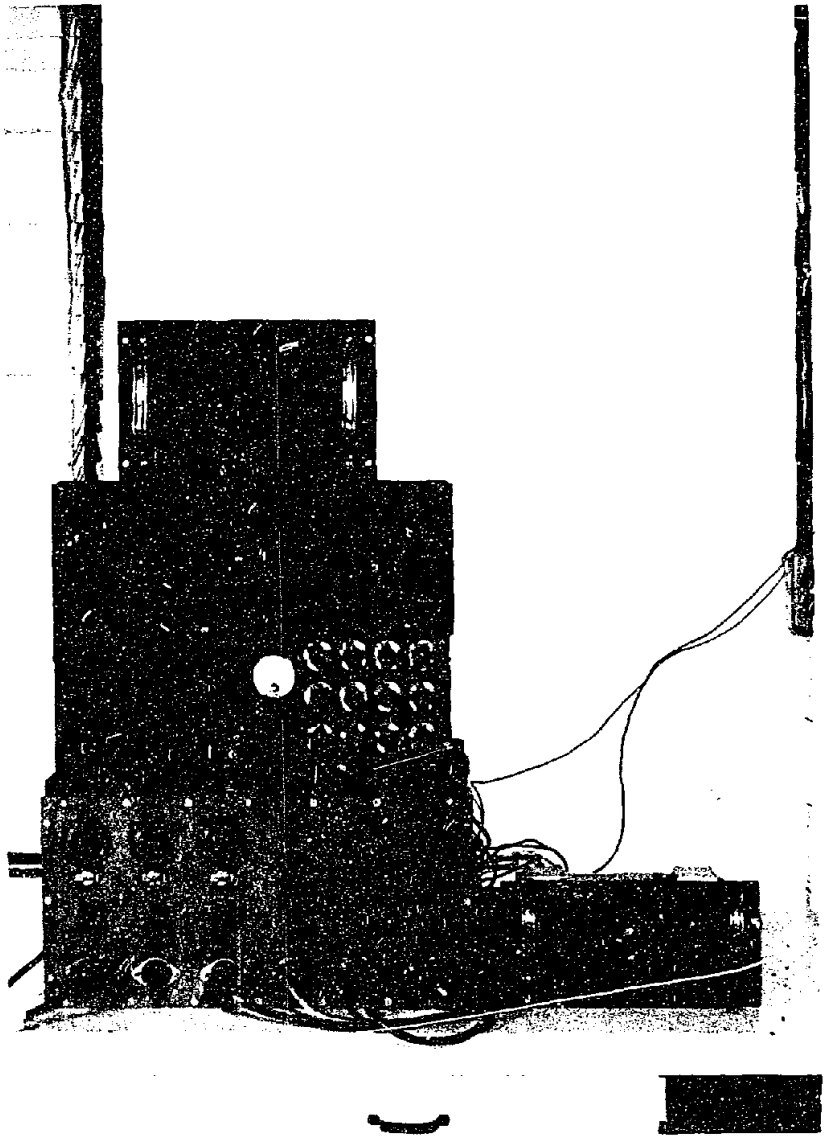
CALIBRATION OF PIPETTE AND VOLUMETRIC TANKS

X-D-2423

A - Pendulum Clock with Foot -Electric Relay



B - Twelve Channel Oscillograph Tuning Fork Timer, Six Channel D C Bridge Circuits and one amplifier

FIGURE 9

LABORATORY TIMING SOURCES

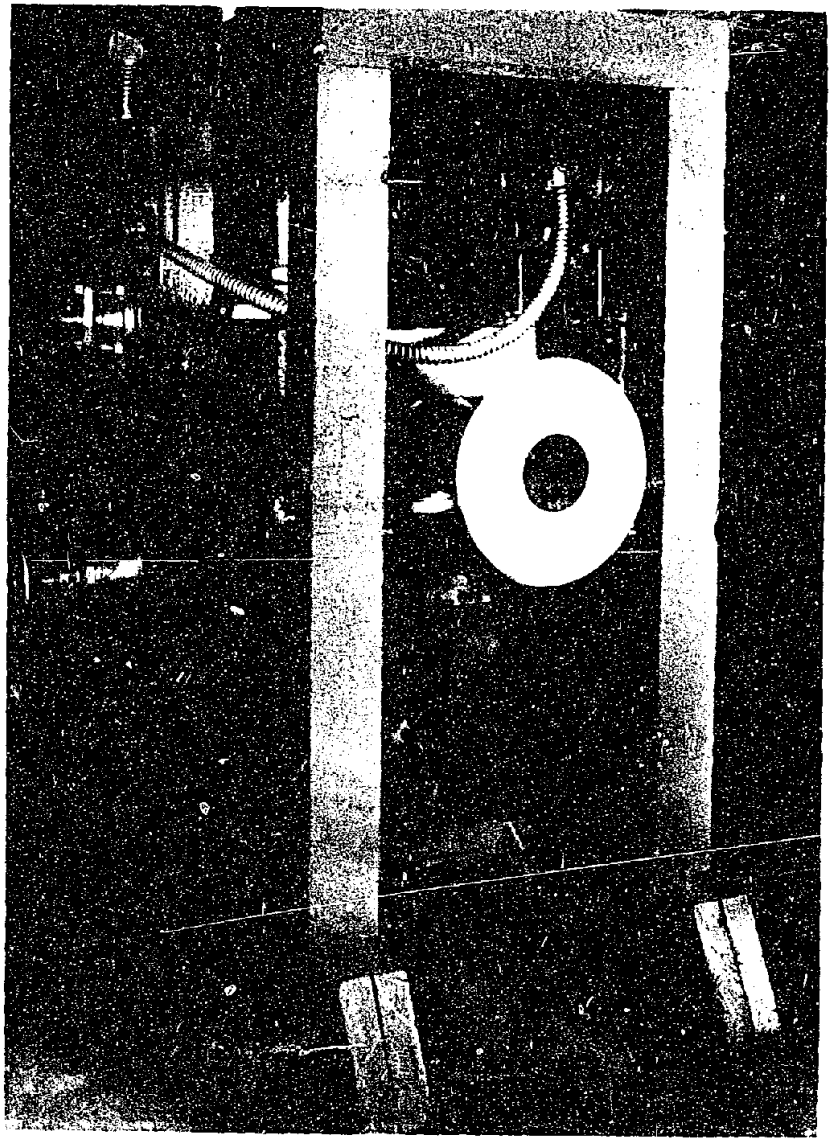
tank, figure 5B, which actuates a magnetic pen that registers a line, for each impulse or second, on a roll of paper. A synchronous motor and a set of gears moves the paper at various speeds, depending on the accuracy desired. A second magnetic pen energized from a mercury switch on the swing spout records a line on the paper at the beginning and the end of each run. The chronometer makes it possible to measure time to about one-fiftieth of a second. Incorporated in the chronometer is a magnetic counter which records full seconds of time. This can be cut in or out of the circuit at any instant, thus making it necessary to use the pens only at the beginning and end of a record to indicate fractions of seconds. If more accurate timing is desired, the laboratory oscillograph, figure 9B, can be used in conjunction with the seconds counter to record fractions of a second at the beginning and end of each calibration record. An electric tuning fork incorporated in the oscillograph serves as a timing source and records each one-one hundredth of a second. By this means time can be read to one-one thousandth of a second if desired. The swing spout on the volumetric tank, which is actuated by a pneumatic cylinder, can be adjusted by means of a valve to move at any desired rate of speed, thus adding to the accuracy of the timing scheme.

A small pipette tank, shown in figure 5C, was used to calibrate the larger volumetric tank. The pipette tank consists of two compartments, as shown on figure 8, 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. Calibration of the volumetric tank was accomplished by repeatedly filling the pipette tank with water and emptying it into the larger one, at the same time keeping an accurate record of water temperatures. In this way the calibration shown on figure 8 was obtained. Prior to calibration of the large tank, the small pipette was removed from the position shown and calibrated by weight, using a sensitive 500-pound scale.

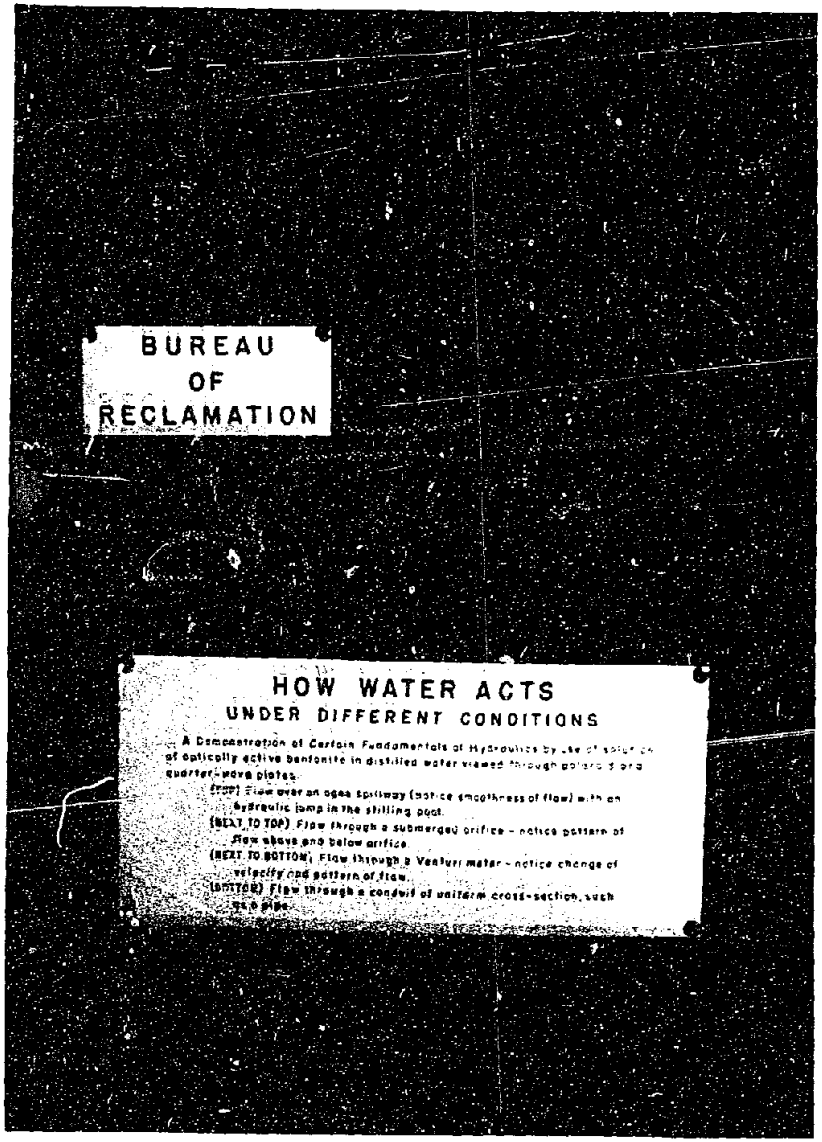
6. Other laboratory equipment. In addition to the above basic hydraulic equipment, the laboratory possesses a blower by which various closed conduit work, such as valves, gates and pipe-line problems, may be solved quickly using air as fluid rather than water. Models can be constructed of plaster of Paris or wood and the testing completed before similar hydraulic models, which must be constructed of more durable materials, are out of the foundry. The scope of the air model is limited to closed conduit work but, where applicable, the results have closely checked those obtained on corresponding hydraulic models. Metering equipment for the air blower consists of thin-plate intake and discharge orifices, which have proven sufficiently accurate and versatile for any air-flow measurement yet encountered in connection with the blower testing. The blower is of the positive displacement rotary type, driven by a 5-horsepower motor, which will deliver air at the rate of 6 cubic feet per second at approximately 2p.s.i. pressure. A photograph of the blower and a measuring orifice is shown on figure 10A. The total air flow is measured by the intake orifice shown in the foreground. The blower is driven at a constant speed of 600 r.p.m., and the rate of flow through the model is regulated by the by-pass which is a vertical pipe in the background. A discharge orifice located at the end of this pipe is used to measure the rate of flow of air wasted. The discharge passing through the model is therefore the difference between the total flow and that wasted through the by-pass. As soon as conditions permit, it is the intention to purchase a larger capacity variable pitch centrifugal blower for model testing which may be throttled to obtain the desired capacity.

Another interesting piece of laboratroy equipment which can be used for demonstration purposes or for testing of tentative designs of hydraulic structures is the fluidpolariscope*(26). This piece of apparatus 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 figure 10B in the following order from front to back: polaroid lens quarter

*See reference 26 at back of manual.



A - Rotary Positive Displacement Blower showing Measuring Orifice and By-Pass



B - Fluidpolariscope Showing a Model in Operation

MISCELLANEOUS LABORATORY EQUIPMENT

FIGURE 10

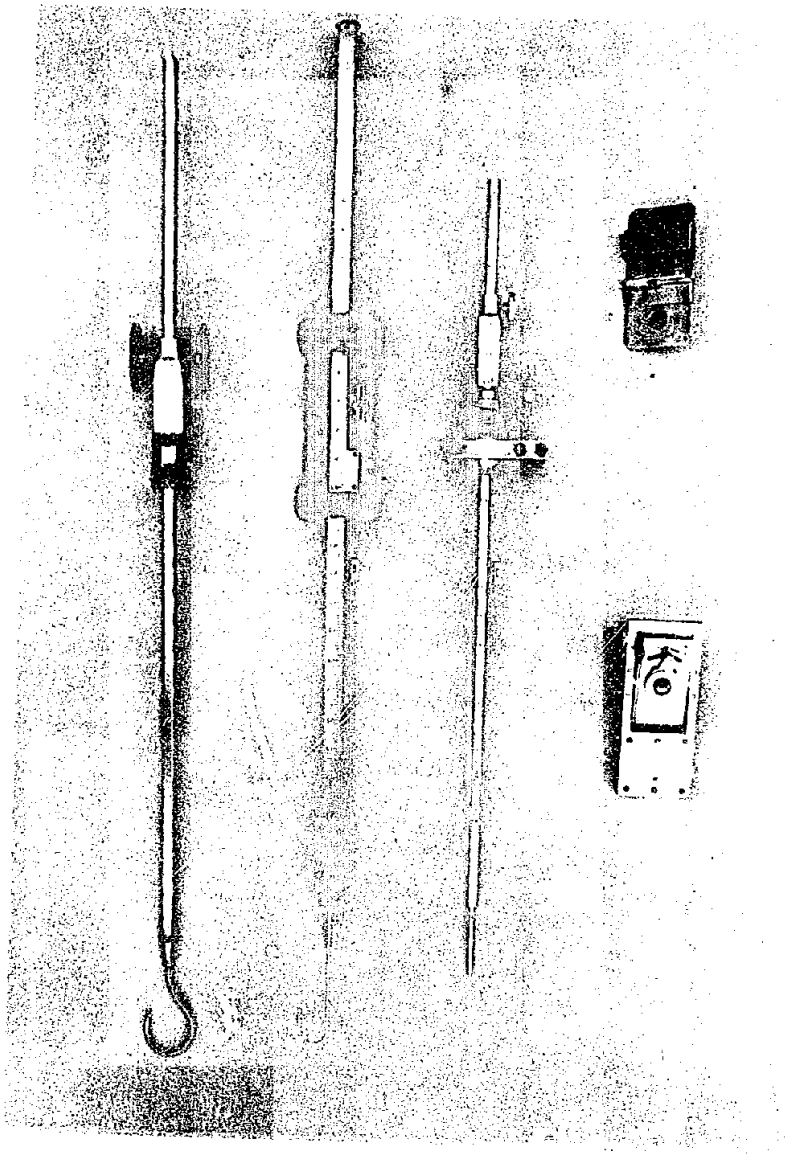
wave plate, model to be tested, quarter wave plate, polaroid lens, opal glass diffuser plate, and source of light. The models to be tested are necessarily two dimensional, and are encased in transparent closed vessel usually about one-half inch in width. The vessel, which contains an inlet and outlet, 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, 0.01 percent of sodium pyrophosphate and a small trace of aerosol. The bentonite particles consist of platelets sufficiently small that there is no tendency to settle out of the suspension or rise toward the surface. They move exactly as the fluid apparently having no appreciable inertia in themselves. In the analysis of fluid flow the bentonite so modifies the light passing through the stream that colored bands or "fringes" are visible, connecting points where a particular rate of change in the speed of the fluid 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 matter of hours, saving much time and money should the design in question prove undesirable. Should the design show merit but require further investigation, an air or hydraulic model can then be constructed and tested.

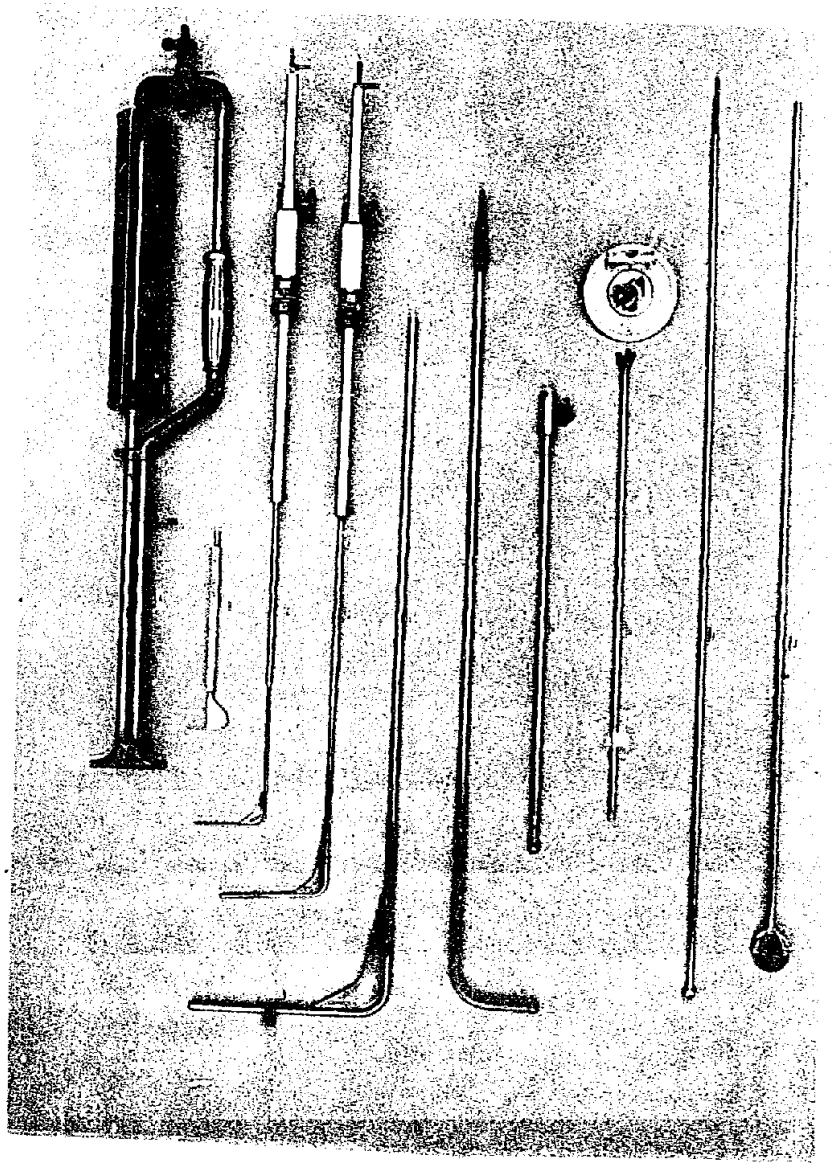
LABORATORY INSTRUMENTS

7. Instruments for water-surface and velocity measurements. In this hydraulic laboratory where assigned problems are diversified, it is continually necessary to design and construct instruments to accomodate the work at hand. A brief description of the more common instruments will be given here.

The most common laboratory instruments for obtaining water surface elevations are the hook and point gages. Three different designs shown on figure 11A are all operated through rack-and-pinion arrangements



A - Point Gages, Hook Gages and Mounting Devices



B - Instruments for Measuring Velocity

LABORATORY INSTRUMENTS FOR MEASURING WATER-SURFACE ELEVATIONS AND VELOCITIES

FIGURE 11

and are available commercially in various lengths and weights. As can be observed from figure 11A, there are also different methods of mounting these gages.

Instruments for measuring velocities are as numerous as the conditions under which measurements are required. Instruments suited to measure moderate or high velocities are not accurate at low velocities, and those designed for low velocities are too fragile to withstand the higher velocities. Furthermore, some instruments are more suitable for open than closed conduit work. The most common velocity measuring devices are the pitot tubes, some of which are shown in figure 11B. The tube denoted as B consists of a single kinetic leg and is used principally for measuring air velocities.

The pitot tubes, C, D, and E, are of the Prandtl type consisting of both kinetic and static legs, designed for a coefficient of unity. The tubes vary in size according to the requirements of the work to be performed but all are exactly similar in dimensions. This type approaches an all out purpose pitot tube as it works well over a wide range of velocities. The shape however, limits its use to principally open channel work.

Tube F is shown as a ball-nose pitot tube in which 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 degrees with the horizontal shaft, leads to the static connection. This tube has the advantage of possessing a coefficient of 1.5 over the range of velocities for which it was designed.

Tube G, known as a Staukugel or pitot sphere, also employs a sphere but is supported from a vertical shaft. The chief characteristic of this tube is the sphere contains five ports arranged in the form of a cross. The center or kinetic port is directed into the approaching current by rotating the tube about its vertical axis until the pressure at the two outside horizontal ports is balanced. This means that the kinetic

port is directed such that it coincides with a vertical plane parallel to the flow and passing through the center of the shaft of the tube. There is no mechanical adjustment by which the pressure can be balanced on the two outside vertical holes. Pressures are observed at the five ports and, by means of calibration curves, the velocity vector is computed in magnitude and direction. The special property of this device is its direction finding feature.

Three methods of mounting pitot tubes and point gages for open channel work are illustrated on figure 11A. 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 11A, 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.

Tube H 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 accompanying this pitot tube make it adaptable for large as well as small conduits. Usually packing glands are provided 90 degrees apart in the conduit, thus it is possible to make two velocity traverses, one normal to the other. Tube H has three ports, in line, normal to the center line 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 coefficient approximates 1.33. The coefficients vary with the velocity for spherical and cylindrical pitot tubes as the pressure distribution on these shapes changes with Reynold's number. For this reason spherical and cylindrical tubes must be used only within the calibrated range.

Tube I is a pressure probe designed to measure static pressures in the interior of a fluid. It may also be used to determine the direction of flow.

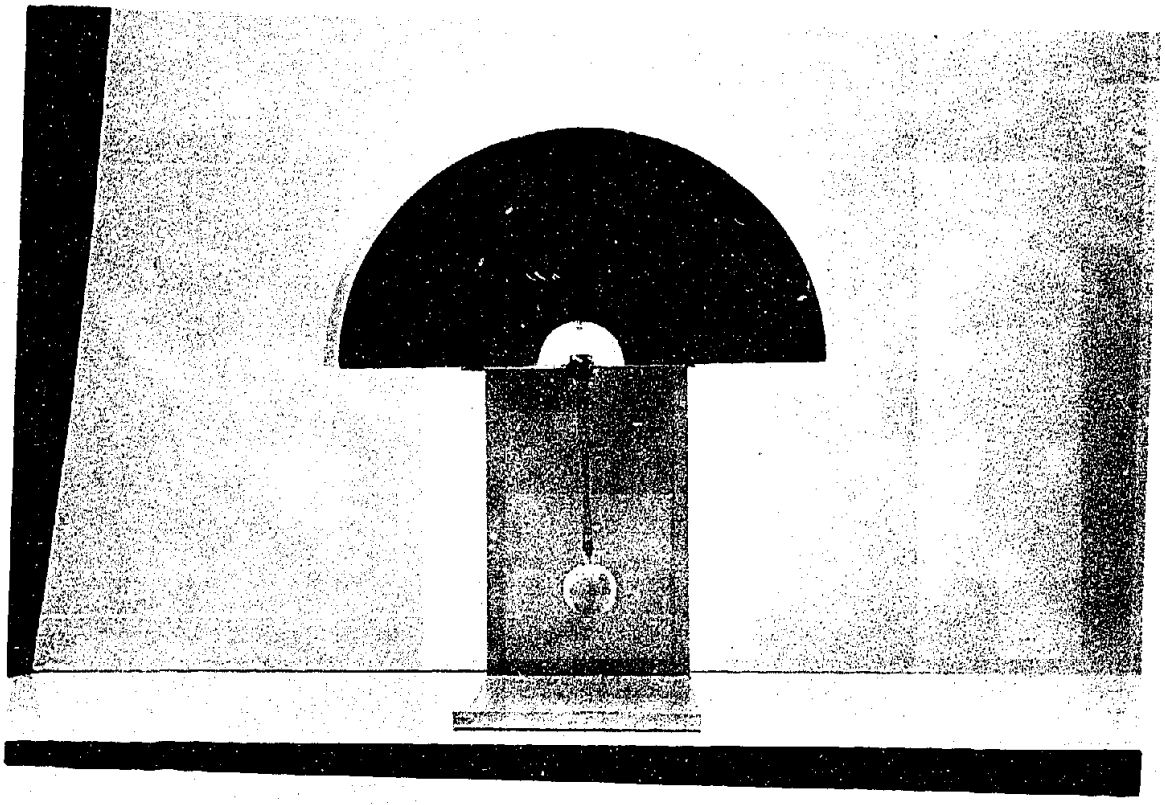
The probe consists of two chambers, separated by a diaphragm; each chamber is exposed to surrounding conditions by a small hole in the outside wall, and each chamber is connected with the recording instrument by small tubes encased in the main shaft. The probe is rotated until the pressure is balanced on the two sides of the diaphragm. The probe is then lengthwise in the direction of flow and the measured pressure is very nearly equal to the true static head because the distance from the leading edge equals the corresponding distance as determined for pitot tubes.

The instrument shown as J on figures 11B is a miniature electric current meter which is used in open channel work for measuring low 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 head set, chronograph or oscillograph.

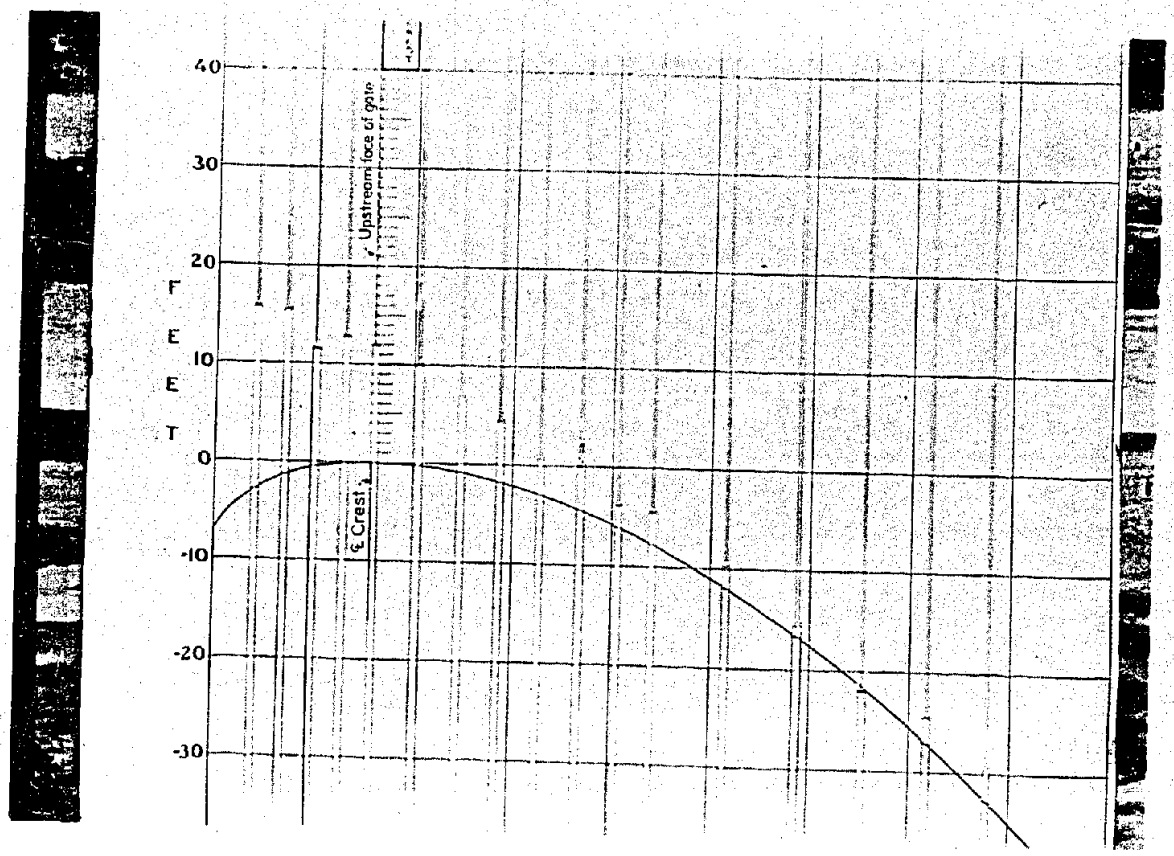
The instrument shown as A on figure 11B is known as a Bentzel Tube and is designed for measuring velocities ranging from 0.4 to 3.5 feet per second. With the nose of the tube submerged in the flow, a vacuum pump is used to evacuate all air from the two legs. This establishes flow through the gage, up the leading leg and down the trailing leg. Incorporated in the leading leg, is a conical glass tube containing a rubber float, with 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 velocity in feet per second directly.

Another instrument which is well adapted to the measurement of low velocities, is shown in figure 12A. It was developed by the laboratory for measuring continually changing velocities in tidal estuary models. As shown in the figure, the instrument is adjusted to read instantaneous velocities directly from plus 0.4 to minus 0.4 foot per second. It consists of a plexiglass sphere supported on the end of an arm of the same material. The arm is suspended from the frame in pendulum-like fashion. The instrument can be adjusted for other velocity ranges by shifting the arm, which is adjustable, or substituting spheres of other diameters. For general laboratory use, several sizes of this instrument

FIGURE 12



A - Pendulum Velocity Meter



B - Pressure Manometers

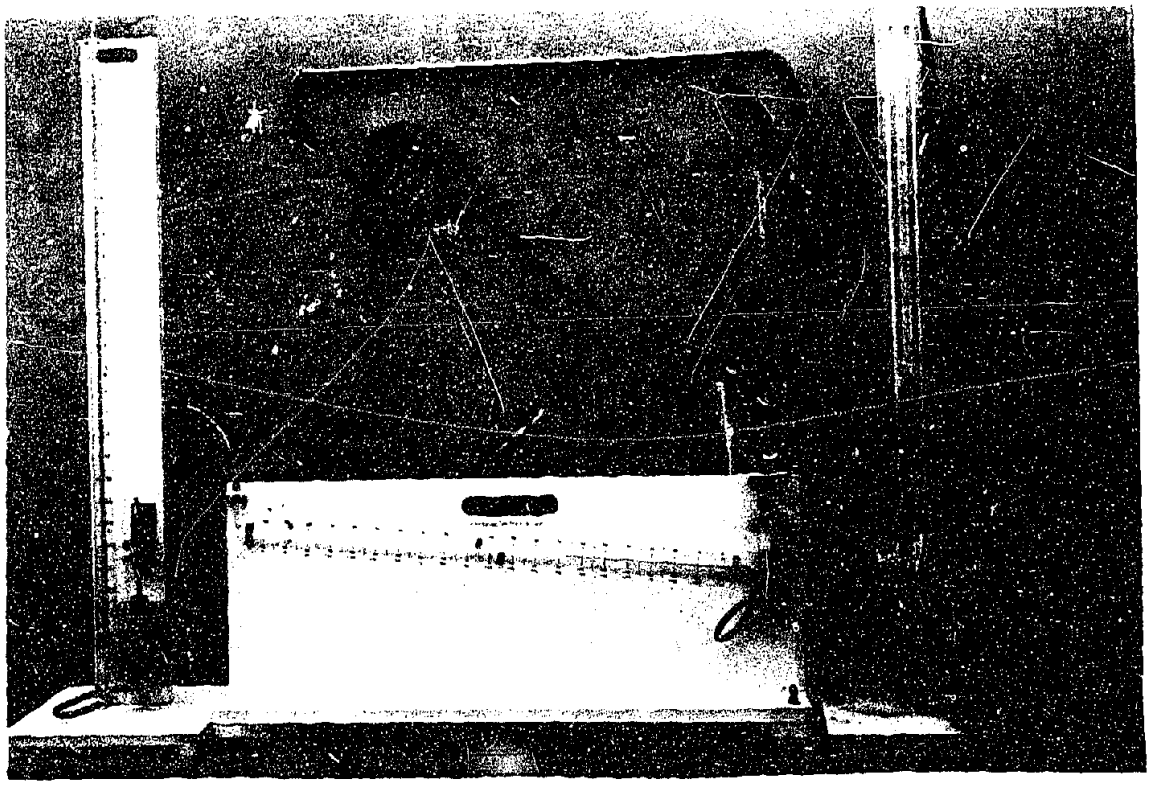
LABORATORY INSTRUMENTS FOR MEASURING VELOCITIES AND PRESSURES

are desirable.

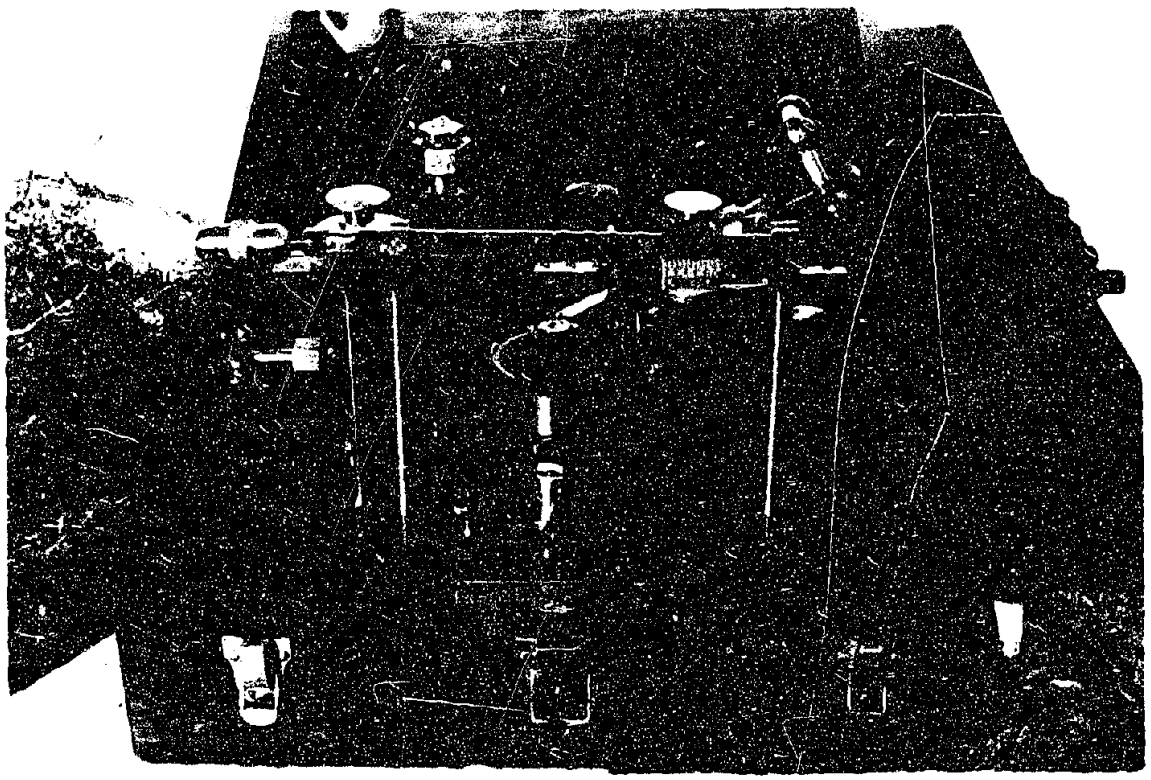
8. Instruments for measuring pressure. The simplest device for measuring positive water pressures is the single leg manometers shown on figure 12B. Each glass tube is connected by a flexible piece of tubing to a piezometer located at the point on a model where a pressure measurement is desired. The pressure is then read directly in feet of water above the point. This type of manometer is not suitable for measuring negative or subatmospheric pressures. With tubes as small as those shown on figure 12B, 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 is another simple device which can be used to measure negative as well as positive pressures. The magnitude of the pressures to be measured determines the gage liquid. Water and mercury are the most common liquids.

The gage on the left, figure 13A, 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 medium, the use of the manometer is limited to the 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 employed.

The center gage in figure 13A is a pot type sloping manometer which is used in air model testing. The sloping tube magnifies the movement ten times and the movable pot makes possible a greater over all 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 this water gage solution to counteract surface tension.



A - Pot Type Pressure Manometers and Pitot Tube Gage



B - Fluid Pressure Scale for Weighing
Pressure up to 300 p.s.i.

LABORATORY INSTRUMENTS FOR MEASURING FLUID PRESSURE

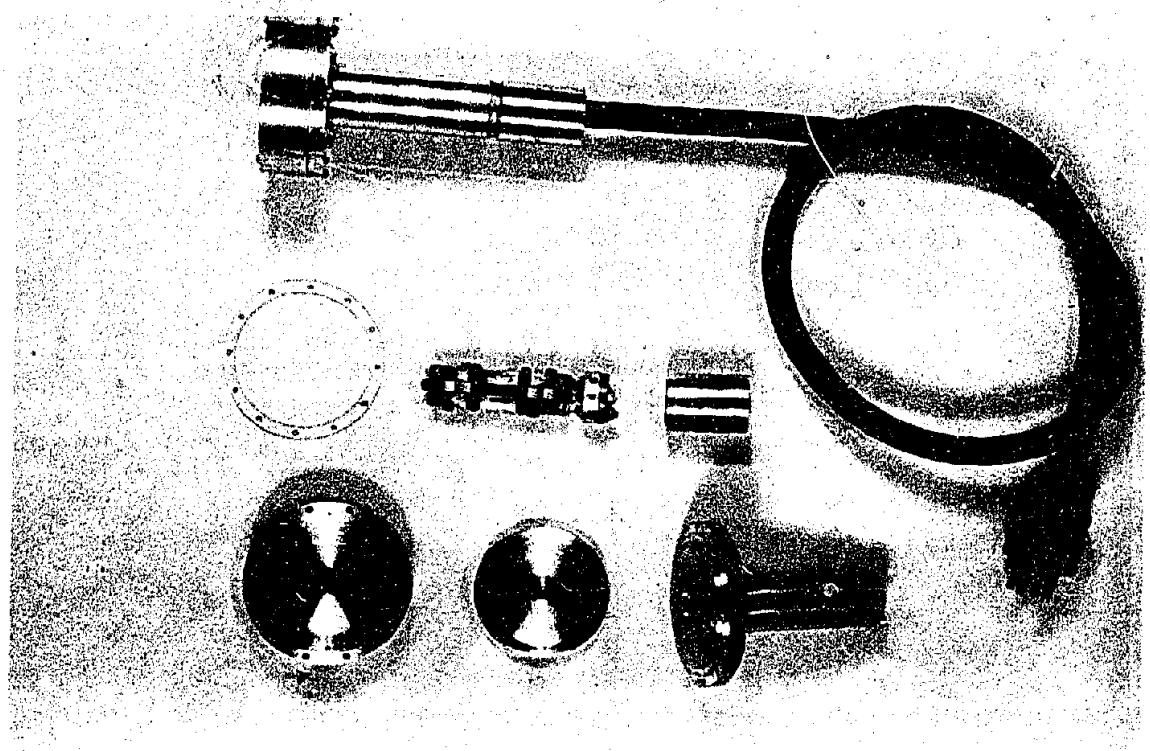
The gage on the right in figure 13A is a pitot-tube manometer. The pitot tube is connected as shown and the nose is immersed in water. A vacuum pump is then used to lower the pressure in the manometer, thus causing the water surface to rise in both legs of the gage. A needle valve in a small reservoir 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 the surface tension inherent in such small tubing. 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 the measurement of positive water pressures up to 100 feet of water, mercury U-tubes or pot gages, are usually employed. For higher pressures a Crosley fluid-pressure scale is available, figure 13B. This is an adaptation from a dead-weight testing machine in which the force of fluid pressure against a piston is actually weighed. The pressure to be measured is applied to a cylinder containing a piston. Movement of the piston in turn actuates the system of levers. Actual weighing is accomplished by loading the main lever arm to the point at which no movement of the piston occurs. The weights on the lever arm then indicate the true pressure of the fluid. The purpose of the motor on the machine is to slowly revolve the piston to eliminate static friction with the walls of the cylinder. Pressures as high as 300 p.s.i. can be measured by this instrument.

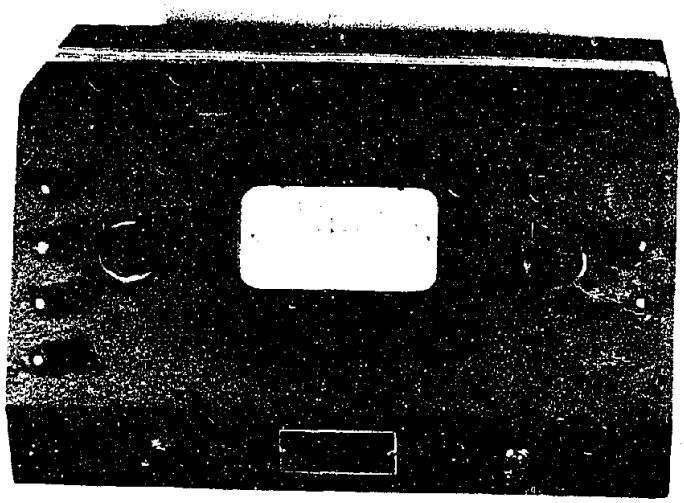
For pressures greater than the capacity of the fluid-pressure scale, Bourdon-type indicator gages are used. The difficulties involved in maintaining the accuracy of this type of gage, when it is moved often, makes it undesirable for hydraulic model work.

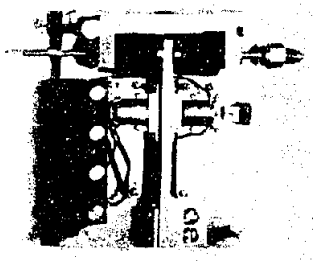
In cases where continuous or instantaneous records of pressures or pressure fluctuations are desired, electric pressure cells such as those shown on figures 14A and 17A used in connection with the laboratory

FIGURE 14



A - Carlson Strain Meter Type Pressure Cell





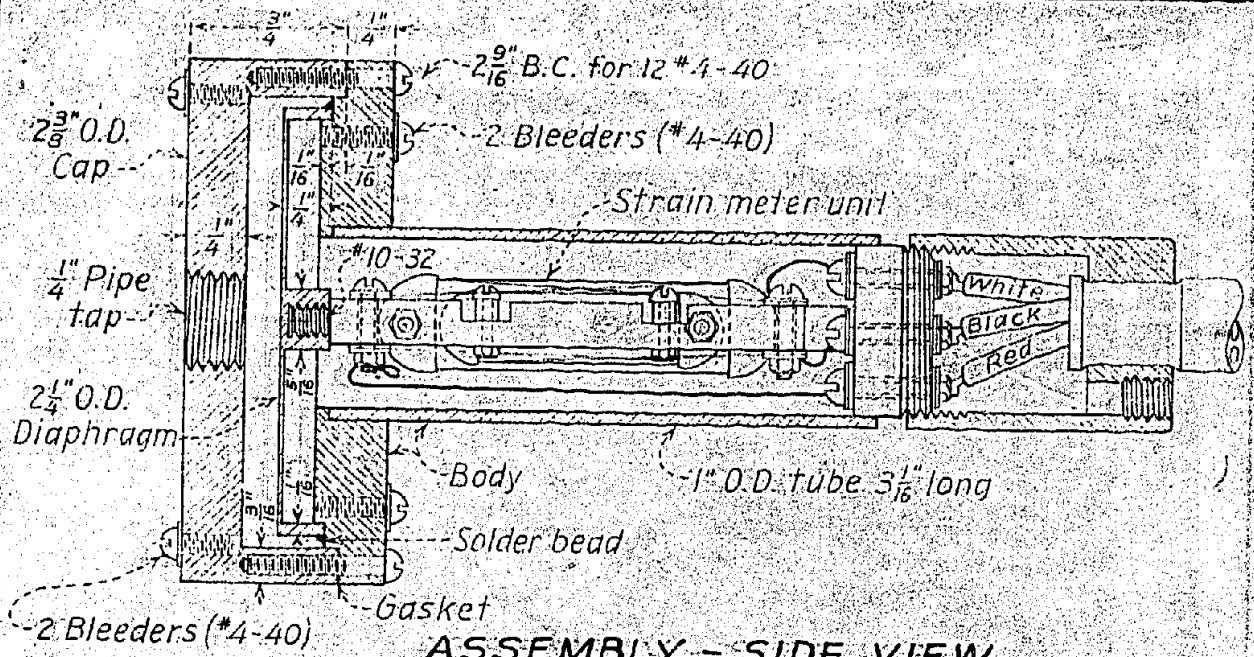
B - A C Bridge and Pressure Head

LABORATORY INSTRUMENTS FOR MEASURING PRESSURE

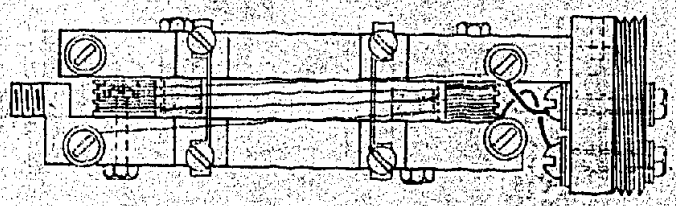
oscillograph figure 9B. A detailed drawing of the Carlson type pressure cell of figure 14A is included as figure 15. This cell consists of two steel arms, one stationary and one movable. The movable arm is rigidly connected to a diaphragm which reflexes as pressure changes are applied to the opposite side. Two coils of wire, one opposing the other, are strung under tension over porcelain insulators mounted between the steel arms. Movement of the diaphragm in either direction increases the tension on one coil and reduces it on the other, thus the instrument is adaptable for the measurement of negative as well as positive pressures. The wire, which is a special alloy developed for high-strain sensitivity, is quite sensitive to temperature changes; however, the dual coil serves to equalize the variation in the resistance between the two coils caused by temperature variations. As an additional precaution, the coils, are encased in a brass shell filled with oil. The cells are all constructed exactly the same except for the thickness of the diaphragm which varies with the pressure for which they are designed. These cells have an important advantage in that their output is large enough to be recorded without the use of amplifiers in most cases of operation.

The oscillograph and accompanying equipment are shown on figure 9B. The oscillograph is a 12-channel Heiland Research Corporation instrument with seven galvanometers having a very high sensivity, a resistance of 35 ohms and a natural frequency of 40 cycles per second. Three other galvanometers less sensative have a rather low resistance of 8 ohms and a natural frequency of 500 cycles per second. The timing mechanism which consists of an electrically driven tuning fork and generator, which drives the timing motor in the oscillograph proper, is shown above the oscillograph, on figure 9B. The timing motor drives a plotted cylinder containing a lamp which projects lines across the full width of the recording paper for each one one-hundredth of a second,

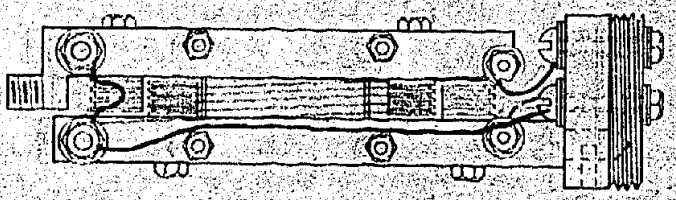
FIGURE 15



ASSEMBLY – SIDE VIEW



STRAIN METER UNIT – TOP VIEW
SHOWING WIRING OF EXPANSION COIL

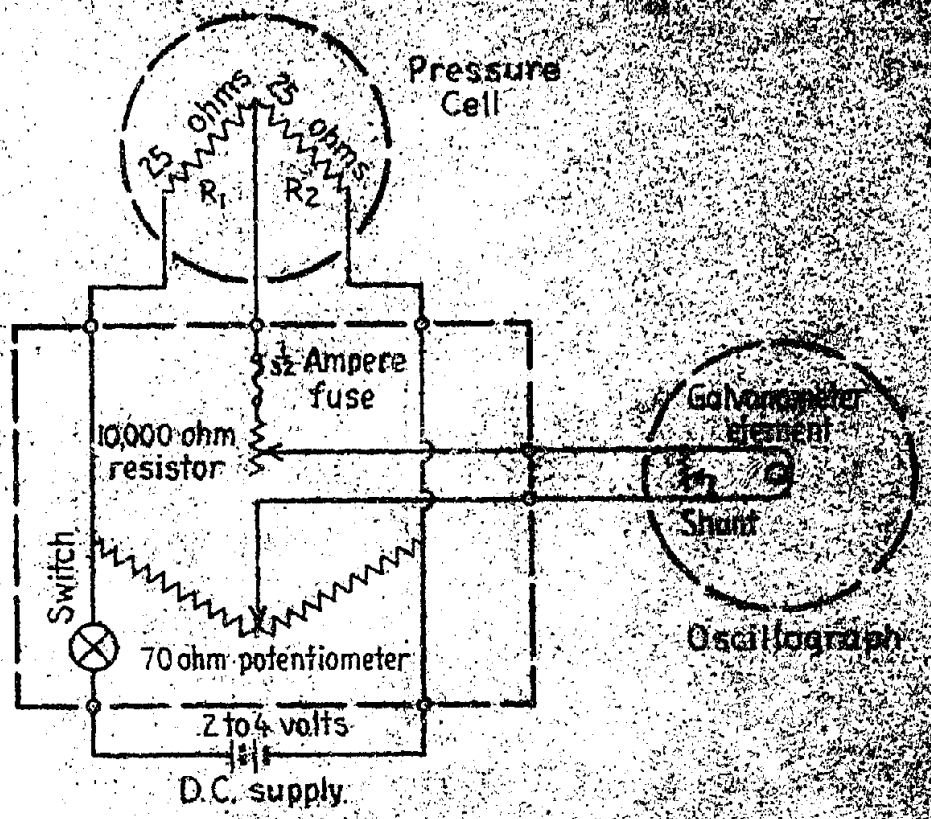


STRAIN METER UNIT – BOTTOM VIEW
SHOWING WIRING OF CONTRACTION COIL

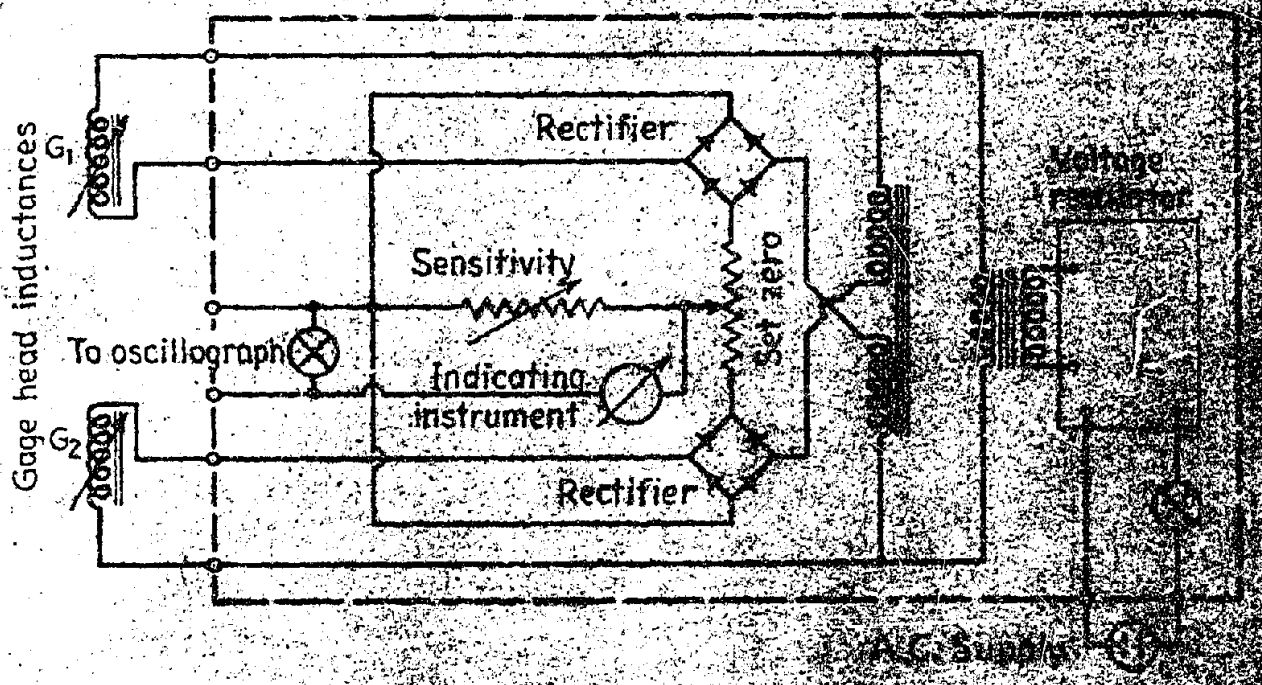
NOTES

Sealing chamber filled with G.E. No. 227 insulating compound. Strain meter unit filled with castor oil leaving 2 c.c. air space. Pressure chamber filled with water before embedment. Strain meter unit same as in Carlson stress meter.

LABORATORY ELECTRIC HYDROSTATIC PRESSURE GAGE



A. D. C. BRIDGE CIRCUIT



B. A.C. BRIDGE CIRCUIT

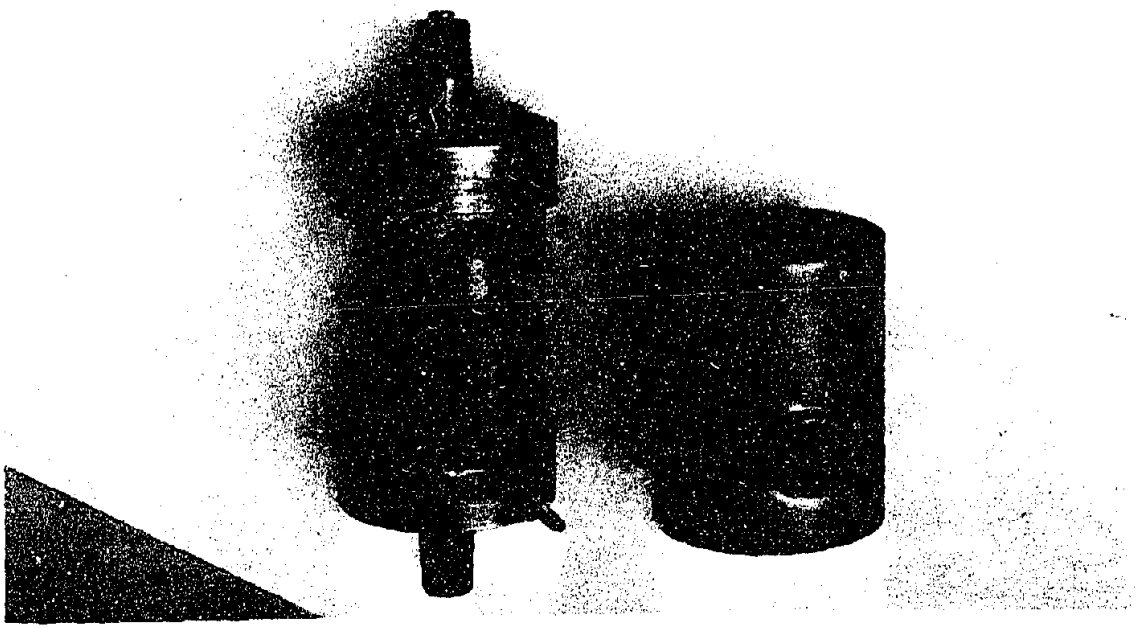
LABORATORY PRESSURE INSTRUMENTS

designed for positive and negative pressures or differential heads up to approximately 20 p.s.i. By substituting more flexible armatures this range can be reduced and the sensitivity increased proportionately. The instrument constitutes a single bridge circuit, without supplemental amplification and operates at a frequency of 60 cycles per second. The frequency response is limited to approximately one-third of the carrier frequency or about 20 c.p.s.

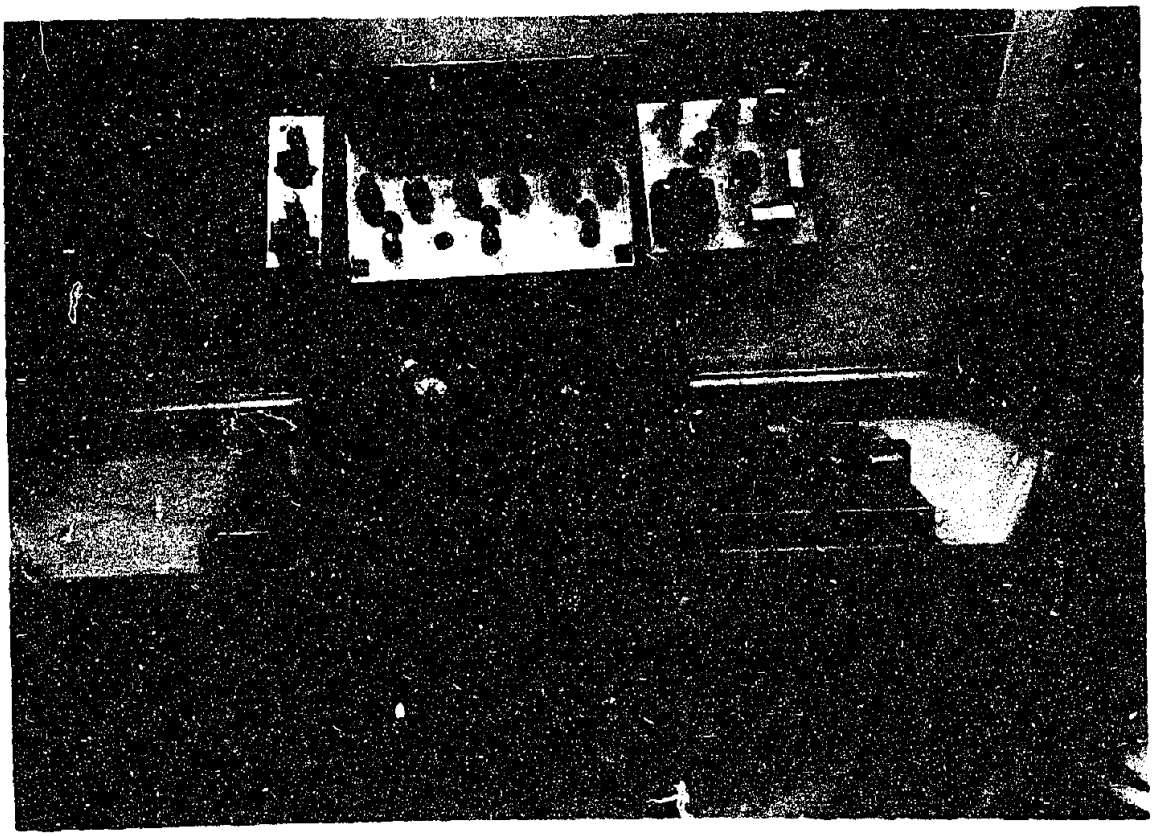
In addition to the above pressure-measuring instruments, a six-channel alternating current bridge with accompanying amplifiers was constructed in the electronics shop of the hydraulic laboratory. This operates on a frequency of 1,000 to 2,000 cycles per second thus eliminating 60-cycle interference which was prevalent in the Hathaway instrument. Reactance-diaphragm type pressure cells were also constructed for use with the above instrument. These will operate on the same principle as the Hathaway pressure head shown on figure 14B except that diaphragms replace the bellows and the new unit is more compact. The pressure cell is shown in different ways on figures 17A, 18, and 19, and the accompanying power supply, oscillator, bridge circuits, and amplifiers are shown on figures 17B and 19. This equipment is used in conjunction with the oscillograph shown on figure 9B, or with a recently acquired Hathaway oscillograph.

FUNCTIONS OF THE HYDRAULIC LABORATORY

9. Purpose. Galileo once said, "I have met with fewer difficulties in discoveries relating to the movements of heavenly bodies, notwithstanding their astonishing distances away, than investigating the motion of flowing water though taking place before our very eyes." Much has been learned in the field of hydraulics from many able men in the 300-year interval since Galileo, yet hydraulics may be still classed as a body of empirical data. It is based almost entirely on experiment. The laws governing many of its phenomena are still only partially understood

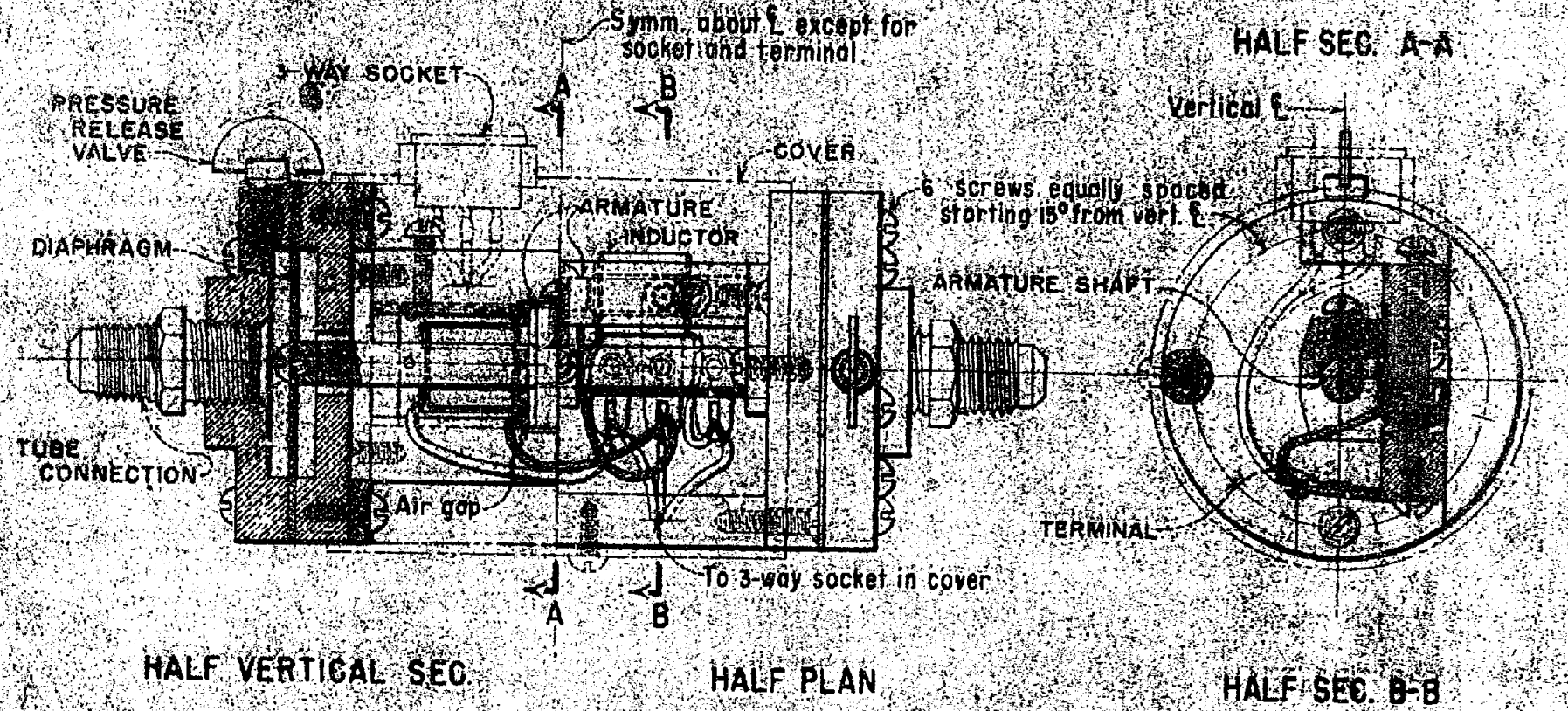


A - Reactance, Diaphragm Type Pressure Cell



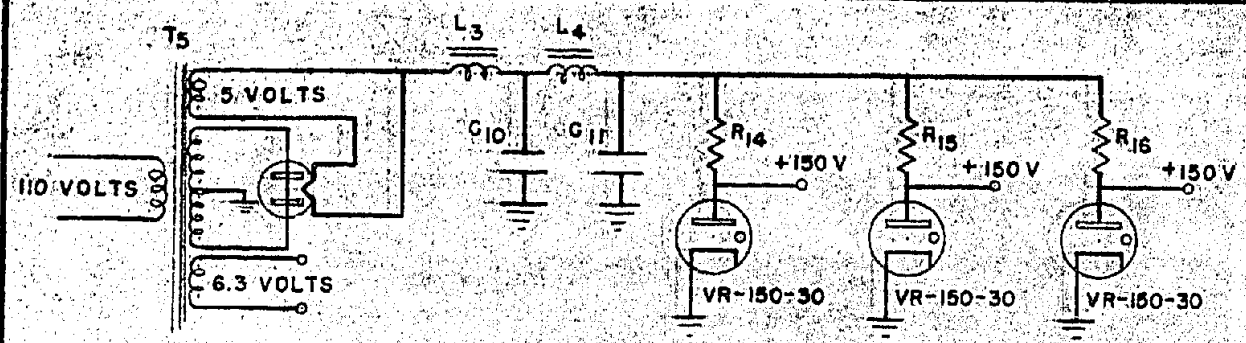
B - Power Supply, Oscillator, A C Bridge, Rectifiers and Amplifiers for above Pressure Cell

LABORATORY INSTRUMENTS FOR MEASURING PRESSURE

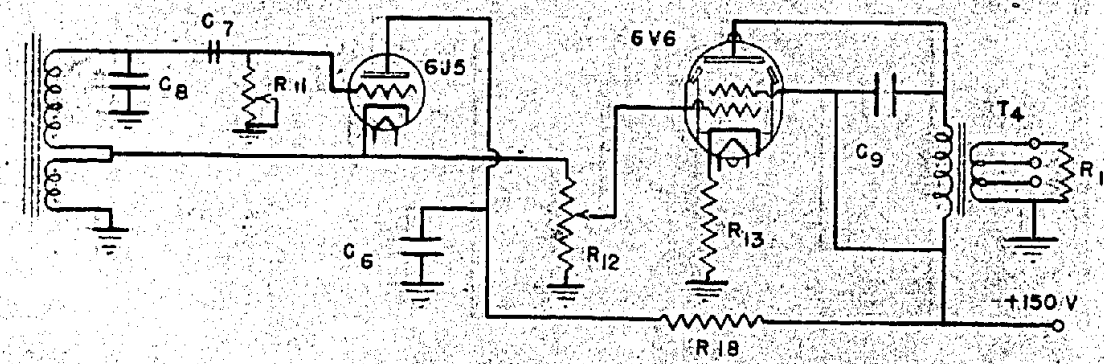


DIAPHRAGM PRESSURE CELL FOR ELECTRIC GAGE

FIGURE 19

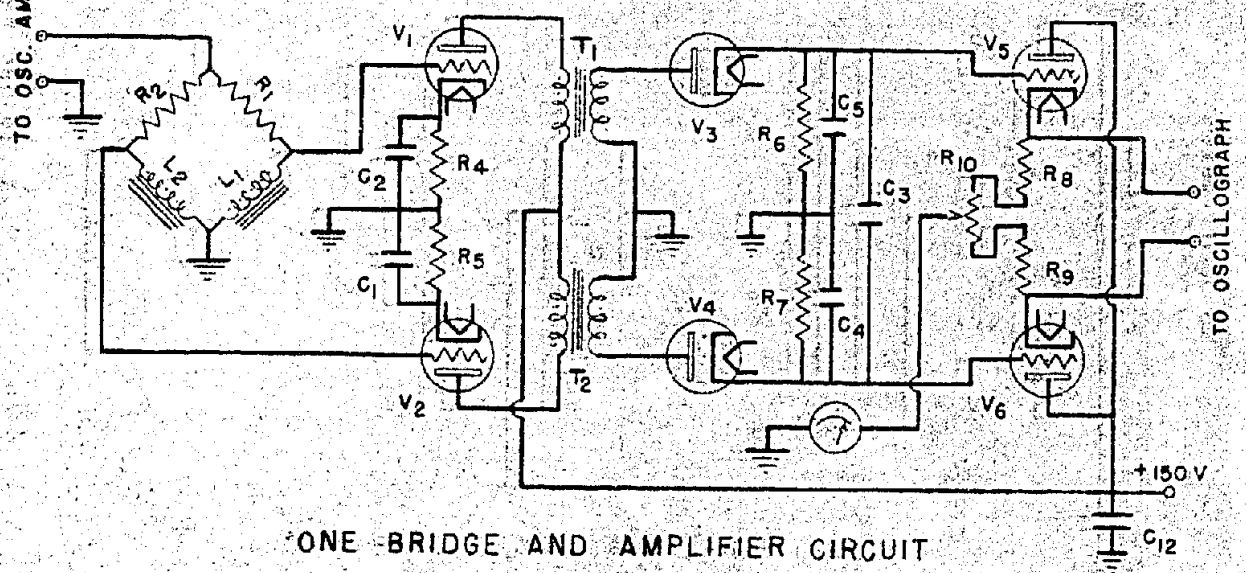


D.C. POWER SUPPLY

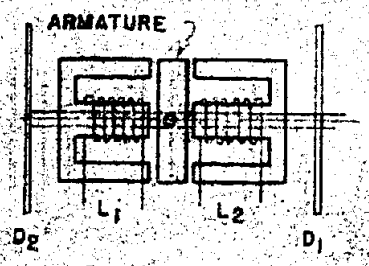


BRIDGE-POWER SUPPLY

1000-2000 C.P.S.



ONE BRIDGE AND AMPLIFIER CIRCUIT



DIAPHRAGM PRESSURE CELL

ELECTRIC PRESSURE GAGE
VARIABLE INDUCTION TYPE

SCHEMATIC DIAGRAMS OF POWER SUPPLY, BRIDGE CIRCUIT AND AMPLIFIER

and much experimental as well as analytical work lies ahead before large structures and hydraulic equipment can be safely and economically designed without the aid of supplemental experiments and model studies. Since the advent of the airplane, however, the field now known as "fluid mechanics," which includes gases and all liquids, is advancing rapidly. There remains nevertheless much experimental work to be accomplished in the basic or fundamental phase of hydraulics, or more properly fluid mechanics, before unity of understanding will be possible, not to mention the difficulties that arise in the design of a structure or machine in which the basic features of hydraulic flow cannot be segregated into component parts but exist in combinations. The analytical approach to such problems is very often unsatisfactory because of the lack of suitable mathematical techniques. The work in this laboratory consists principally in determining flow characteristics for specific hydraulic structures by means of models. The experimental work is timed to coincide with the design of the field structures. Although the extent of basic experimentation in the laboratory has been limited, some of this type of work has been accomplished and the information is available for design purposes.

The unprecedented magnitude of Boulder, Grand Coulee, Shasta, and other recent dams, together with the corresponding increase in the size of their accompanying hydraulic features made it imperative that experimental studies be carried out along with their general design. The hydraulic studies, of course, were merely one phase of the experimental work. In the interest of economical and safe design, unfamiliar factors could not be left entirely to chance or individual judgment with structures of this magnitude. It was later discovered that the savings in cost and improvement in designs of smaller hydraulic structures, made possible by model studies, were alone sufficient to justify the operation of a large hydraulic laboratory on a full-time basis. As a result of the

steady flow of information emanating from the Bureau of Reclamation hydraulic laboratories in the past 14 years, the present laboratory is now a well established part of the design organization.

10. Types of structures and equipment investigated. Since 1930, approximately 250 models of various hydraulic structures and equipment have been constructed and tested in the hydraulic laboratories of the Bureau of Reclamation, not to mention numerous experiments and other studies of a general nature which would be difficult to list. A record of the more important of these model studies is exhibited in table 1. The table includes hydraulic structures such as dam spillways, outlet works, sluiceways, canal headworks, detailed studies of large gates and valves, pumps and turbines, river models, canal structures, fish traps and related structures, siphons, entrances to closed conduits, and other miscellaneous problems including field tests on some prototype structures. The headings from columns 1 through 10 are self explanatory and refer to the prototype structures. Column 11 indicates the scale of the model used and in some cases, such as the Grand Coulee Dam, seven different models were employed in studying the structures relating to the spillway. The spillways have been catalogued as to type such as morning glory, side channel, overfall, super-elevated, etc. In some cases a spillway can fall under more than one classification. When this is true the other classifications will be found listed in column 12. By means of a code, the explanation of which follows table 1, the actual extent of the model study in each case is denoted by the letters and numeral subscripts of column 13. Letters alone indicate the portion of the model structure investigated. Where a letter is followed by a numerical subscript, it infers that a detailed study was made involving measurements such as water surface profiles, pressures, velocities, scour patterns, etc., depending on the number used. Where letters alone are shown, these indicate that visual observation was sufficient for dictating changes or of approving the final design. If similar measurements or photographs have

Table 1

MAJOR HYDRAULIC STRUCTURES STUDIED BY THE AID OF MODELS

Dam	Project	Type of dam	Type of prototype construction	Spillway gates	Type of energy dissipation	Width of stilling device (ft.)	Maximum spillway discharge (c.f.s.)	Effective length of crest (ft.)	Approximate drop - headwater to tailwater - at max. discharge (ft.)	Scale of model	Other classifications or remarks	Extent of investigation: Model	Extent of investigation: Prototype	Available literature and reports
					MORNING-GLORY SPILLWAYS									
Boulder	Boulder Canyon, Ariz.-Nev.	Conc. gravity arch	New	None	Original stream bed	-	200,000	722	550	1:60	2 spillways	ABC		[1]
Gibson	Run River, Mont.	Massive conc. arch	Alteration	6-34'x12' radial gates	"	-	60,000	198	186	1:63.6		$AB_{2.4}C$		Hyd.158,110[18]
Owyhee	Owyhee, Oreg.	Conc. gravity arch	New	60' diam. ring gate	"	-	80,000	188	310	1:45		$A_{1.8}B_{1.8.4}C$	$B_{10}D_{10}$	Hyd.159,120,37[2]
Seminoe	Kendrick, Wyo.	Conc. arch	"	"	"	-	80,000	237	190	1:60		AB		
					SIDE-CHANNEL SPILLWAYS									
Big Sandy No. 2	Eden, Wyo.	Earth fill	New	None	Trapez. chute & stilling basin	50	7,600	170	60	1:30	Trapez. open chute	$AD_{1.4}C_1D_{1.2}$		Hyd.121
Boulder	Boulder Canyon, Ariz.-Nev.	Conc. gravity arch	New and repairs	8-100'x18' drum gates	Original stream bed	-	400,000	800	520	1:100,1:60,1:20	2 spillways	$A_1B_{1.8.4}C_{1.2}D_{1.5}$	$A_{10}B_{10}C_{10}D_{5.10}$	Hyd.118,137,142,[illegible]
Box Butte	Mirage Flats, Nebr.	Earth fill	New	None	Rectangular stilling basin	40	3,000	150	50	1:24	Double side channel	$AB_4C_1B_{1.6}$		Hyd. 44,[4],[illegible] 124
Fruit Growers	Fruit Growers, Colo.	"	"	"	"	20	1,000	20	50	1:20	Semicircular overflow section	AB_4CD		Hyd.96
Grassy Lake	Upper Snake, Ida.	"	"	"	Trapez. chute & stilling basin	20	1,200	118	110	1:20	Trapez. open chute	$AB_4C_1D_1$		
Island Park	Upper Snake, Ida.	"	"	"	Original stream bed	-	5,000	257	68	1:50	Double side channel	$AB_{1.4}C_1D$		T.M.478
Moon Lake	Moon Lake, Utah	"	"	"	Spreading trajectory	-	10,000	137	90	1:40	Closed chute	$AB_{1.2.4}C_{1.2}D_5$		T.M.437
Taylor Park	Uncompahgre, Colo.	"	"	"	Trapez. chute & stilling basin	-	10,000	180	148	1:50	Trapez. open chute	AB_4C_1D		T.M.476
					OVERFALL SPILLWAYS									
Altus	Altus, Okla.	Conc. gravity arch - masonry face	"	9-20'x15' radial gates	Rectangular stilling basin	-	40,000	180	78	1:40,1:24		$AB_2C_2D_{2.5}$		Hyd.115
Davis	Davis, Ariz.-Nev.	Earth fill	"	clear free crest section 3-50'x50' slide gates	Submerged bucket	200	240,000	110 150	110	1:100,1:46		$A_1B_4CD_{2.5}$		Hyd.123, 149
Dos Bocas	Puerto Rico Reconstruction Admin.	Concrete gravity	"	None	Rectangular stilling basin	350	200,000	350	120	1:50		$A_1B_{2.4}C_2E_{1.2.5}$		Hyd.32,35,[6]
Friant	Central Valley, Calif.	"	"	3-100'x18' drum gates	"	236	86,000	200	250	1:50,1:25,1:24		$AB_{2.4}CD_{1.2.5}$		Hyd.130
Grand Coulee	Columbia Basin, Wash.	"	"	11-135'x28' drum gates	Submerged bucket	1,650	1,000,000	1,465	280	1:164,1:120, 1:60,1:20 1:40,1:30,1:15		$A_1B_{1.2.4}C_{1.2}D_{1.2.3.5}$	$A_{10}B_{10}C_{10}D_{5.10}$	[5],[4],[5],Hyd.128,[illegible] Hyd.12,33,130, 174
Hamilton	Colorado River, Texas	"	"	1-300'x28' drum gate	Rectangular stilling basin	-	200,000	300	50	1:30,1:60		$A_{1.2}B_{1.2.4}C_{1.2}D_2$		Hyd. 8
Hemdgate Rock	Colorado River Indian Irr.Serv.,Ariz.	Earth fill	"	10-34'x21.5' radial gates	Submerged bucket	298	200,000	260	50	1:60		$AB_{1.2.4}C_{1.2}D_{1.5}$		Hyd. 48
Kesick	Central Valley, Calif.	Concrete gravity	"	4-50'x50' slide gates	Rectangular stilling basin	240	250,000	200	50	1:60,1:45		$AB_{2.4}C_2D_{2.5}$		Hyd.128,78
Lahontan	Newlands, Nev.	"	Alterations	10-50'x6.75' flash gates	Steps and circular pool	-	45,000	475	90	1:60,1:5,1:4	2 spillways	$AB_{1.2.4}C_1D$		Hyd. 72, 39
Madden	Madden Dam, Panama Canal Zone	"	New	4-100'x18' drum gates	Rectangular stilling basin	445	280,000	400	160	1:72		$AB_{1.2.4}C_{1.2}D_{1.2.3.5}$	$A_{10}B_{4.10}C_{10}D_{10}$	[6],Hyd. 1, 39
Marshall Ford	Colorado River, Texas	"	"	None	"	720	200,000	700	190	1:68,1:40.5		$AB_{2.4}C_1D_{1.2.5}$		Hyd.162
Norris	Tennessee Valley Authority	"	"	5-100'x16' drum gates	"	326	200,000	800	170	1:72		$AB_{2.4}C_{1.2}D_1$		T.M.438,[18],21
Roosevelt	Salt River, Ariz.	Masonry arch gravity	Alterations	19-radial gates	Original stream bed	-	160,000	248	250	1:100	2 spillways	$AB_{1.2.4}C_1D$		Hyd.110
Shasta	Central Valley, Calif.	Concrete gravity	New	8-110'x28' drum gates	Rectangular stilling basin	375	250,000	550	450	1:65,1:40		$AB_{1.2.4}C_{1.2}D_{1.2.3.5}$		[6],Hyd.63
Wheeler	Tennessee Valley Authority	"	"	80-40'x16' radial gates	"	2,700	650,000	2,400	50	1:36		$AB_4CD_{1.2.5}$		T.M.407
					SEMI-OVERFALL SPILLWAYS									
Bartlett	Salt River, Ariz.	Conc. arch & buttress	"	8-50'x50' slide gates	Original stream bed	-	175,000	150	170	1:100	Open chute	$A_1B_{1.2.4}C_{1.2}D_5$	$A_{10}B_{10}C_{10}$	Hyd.76,[18]
Granby	Colorado-Big Thompson, Colo.	Earth fill	"	2-21'x20' radial gates	"	-	12,000	42	225	1:24		AB_4CD_5		Hyd.109
Horse Mesa	Salt River, Ariz.	Conc. arch	Alterations	8-radial gates	"	-	100,000	225	235	1:90	Also tun.spillway	$AB_{1.2.4}C_1$		Hyd.110
Moon Lake	Moon Lake, Utah	Earth fill	New	2-radial gates	Rectangular stilling basin	75	10,000	45	100	1:60,1:12	Open chute	$A_1B_{1.2.4}C_{1.2}D_{1.2.5}$		Hyd.128
Mormon Flats	Salt River, Ariz.	Conc. arch	Alterations	2-50'x50' slide gates	Original stream bed	-	150,000	100	110	1:90	"	$AB_{1.4}C_1D$		Hyd.73,Hyd.110
Ross	City of Seattle, Wash.	"	New	12-20'x18' radial gates	"	-	100,000	240	630	1:60,1:25	Overfall 2 spillways	$AD_{2.4}C_2D_4$		Hyd.126
Stewart Mountain	Salt River, Ariz.	"	Alterations	8-radial gates	"	-	100,000	262	100	1:100,1:50	Open chute	$AB_{1.2.4}C_{1.2}D$		
Vallecito	Pine River, Colo.	Earth fill	New	3-37'x19' radial gates	Rectangular stilling basin	125	50,000	111	128	1:60	"	$A_{1.5}B_{1.2.2.4}C_{1.2}D_{1.5}$		Hyd.74
					CLOSED CHUTE SPILLWAYS									
Bridge Canyon	Bridge Canyon, Ariz.	Conc. arch gravity	"	-	Original stream bed	-	400,000	-	810	1:60		$ABCD_5$		Hyd. 170
Horse Mesa	Salt River, Ariz.	Conc. arch	Alterations	One slide gate	"	-	50,000	60	235	1:90		$A_1B_{1.2.4}C_{1.2}D$		Hyd.110
Seminoe	Kendrick, Wyo.	"	New	3-14'x20' slide gates	"	-	50,000	42	190	1:60		$AB_{1.2.4}C_{1.2}D$		T.M.406,[18]
					FLAT OPEN CHUTE SPILLWAYS (RECTANGULAR)									
Alamogordo	Carlsbad, N. Mex.	Earth fill	"	3-45'x11' radial gates	Rectangular stilling basin	110	46,000	135	125	1:64		AB_4CD		[18]
Alcova	Kendrick, Wyo.	"	"	3-25'x40' slide gate.	"	200	65,000	75	145	1:72		$A_1B_{1.4}CD_{1.2.5}$		T.M.413,Hyd.87
Anderson Ranch	Boise, Ida.	"	"	2-25'x22' radial gates	"	100	20,000	80	220	1:48		$AB_{1.4}C_1D_{1.5}$		
Boca	Truckee River Storage, Calif.	"	"	2-19'x16' radial gates	"	72	8,000	38	100	1:48		$AB_4CD_{1.2.3.5}$		
Bull Lake	Riverton, Wyo.	"	"	3-23'x11' radial gates	"	100	10,000	87	60	1:50		$AB_{1.2.4}C_{1.2}D_{1.5}$		T.M.488,Hyd.110
Caballo	Rio Grande, N. Mex.	"	"	2-40'x21' radial gates	"	103	35,000	100	65	1:60		$A_1B_{1.2.4}C_{1.2}D_{1.2.5}$		Hyd.72
Cherry Creek	U. S. Engineer Office, Colo.	"	"	None	"	116	45,000	190	70	1:60	2 trapez.notches in overflow sec.	$AB_{1.4}C_1D_{1.5}$		Hyd.144
Clo Elum	Yakima, Wash.	"	"	4-37'x17' radial gates	"	200	40,000	125	110	1:50		$AB_1C_{1.2}D_{2.5}$		T.M.290,Hyd.37,130
Deer Creek	Provo River, Utah	"	"	2 radial gates	"	75	12,000	42	130	1:48		$AB_4C_1D_{1.5}$		
Elephant Butte	Rio Grande, N. Mex.	Concrete gravity	Alterations	None	Original stream bed	-	35,000	275	200	1:40		AB_4CD		Hyd. 173
Fresno	Milk River, Mont.	Earth fill	New	"	Rectangular stilling basin	150	50,000	210	60	1:50		$A_1B_{1.2.4}C_{1.2}D_{1.2.4}$		Hyd. 177
Green Mountain	Colorado-Big Thompson, Colo.	Earth fill	"	3-35'x22' radial gates	Original stream bed	-	25,000	75	240	1:40		$A_{1.5}B_{1.4}CD_5$		[7],Hyd.76,96
Rye Patch	Humboldt, Nev.	"	"	6-20'x17' radial gates	Rectangular stilling basin	110	20,000	100	40	1:50		$AB_4CD_{1.2.5}$		T.M.431
Scofield	Scofield, Utah	"	"	None	"	40	6,200	60	46	1:30		$AB_{1.4}C_1D_{1.5}$		Hyd.150
Shadow Mountain	Colorado-Big Thompson, Colo.	"	"	2-16'x18.5' radial gates	"	70	10,000	34	35	1:30		$AB_{1.4}C_1D_1$		Hyd.162
Unity	Burnt River, Oreg.	"	New	2 radial gates	"	34.5	10,000	68	50	1:36		$ABCD_5$		Hyd.130
					FLAT OPEN CHUTE SPILLWAYS (TRAPEZOIDAL)									
Agency Valley	Vale, Oreg.	"	"	3-18'x17' radial gates	Trapezoidal stilling basin	-	12,000	54	72	1:50		$AB_{1.4}CD_{1.2.3.5}$		T.M.505
Hyrum	Hyrum, Utah	"	"	3-14'x13' radial gates	"	-	8,200	42	95	1:48		$ABC_1D_{1.5}$		T.M.520
Pineview	Ogden River, Utah	"	"	3-17'x16' radial gates	"	-	10,000	51	85	1:30		$AB_{1.4}C_1D_{1.5}$		T.M.525
					OVERFALL DIVERSION DAMS									
Imperial	A-A Canal, Calif.	Concrete shell	"	None	Sloping & horizontal apron	1,188	155,000	1,188	15	1:24,1:40		$AB_{1.2.4}C_{1.2}D_{1.2.5}$		T.M.512,
Lovelock	Humboldt, Nev.	Concrete slab	"	"	Horizontal apron & sills	260	8,000	230	6	1:24		ABD		Hyd.126
Durdock	Provo River, Utah	Concrete	"	"	Submerged bucket	-	8,000	120	12	1:60		ABD_5		Hyd.128
Roosevelt Power	Salt River, Ariz.	"	Replacement	"	Horizontal apron & sills	-	150,000	600	8	1:46,1:16		$AB_{1.2.4}B_{1.2.5}$	$A_{10}B_{10}D_{5.10}$	[8],Hyd.[illegible]
Rose	Yakima, Wash.	"	New	3-110'x16' roller gates	Short horizontal apron	220	50,000	320	14	1:48		AB_4D_5		T.M.484,[illegible],Hyd.25

TABLE 3 (continued)

Dam	Project	Type of dam	Type of prototype construction	Conduit upstream from control gates	Control gates or valves	Conduit downstream from control device	Type of energy dissipation	Width of stilling basin (ft.)	Capacity at maximum head (c.f.s.)	Maximum head on gate or valve (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature
WATER-CONTROLLED OUTLETS														
Bull Lake	Riverton, Wyo.	Earth fill	New	1-8' horseshoe conduit	1-8'x8' R.F. slide gates	1-8' circular conduit	Rectangular stilling basin with hump, spreader teeth and sill	38	4,000	68	1:30	$P_{8}H_{1.8}$	S_{10}	T.M.218,Rpt.27,p.[illegible]
Cabalio	Rio Grande, N. Mex.	"	"	1-18.5' circular conduit	1-6'x7.5' R.F. slide gates	1-18.5' horseshoe conduit	"	40	8,250	70	1:30	$P_{8}H_{1.8}$		Rpt.37,p.[illegible]
Davis	Davis, Ariz.-Nev.	"	"	Open trapez. channel	1-22'x18' radial gates	Dam	Submerged bucket	44	60,000	80	1:300,1:66,1:48	$S_{8}P_{8}H_{1.8}$		Rpt.118,p.[illegible]
Grand Coulee	Columbia Basin, Wash.	Conc.gravity	"	20-8.5' circular conduits	20-8.5' diam.R.F. slide gates	20-8.5' circular conduits	Vulcanic spillway stilling basin	-	88,000	120	1:200,1:40, 1:40, 1:17	$P_{8}P_{8}P_{8}H_{1.8}$	$P_{8}P_{8}P_{8}S_{10}$	Rpt.[illegible]
				"	"	"	"	-	132,000	280				
				"	"	"	"	-	-	380				
Grassy Lake	Upper Snake, Idaho	Earth fill	"	1-8.25' circular conduit	1-30" needle valve	None	Trapez.stilling basin	-	300	98	1:12	$S_{8}H_{1.8}$		
Island Park	Upper Snake, Idaho	"	"	1-12' circular conduit	2-5'x5' R.F. slide gates	1-12' circular spillway tunnel	Vulcanic spillway stilling pool	-	3,000	70	1:30	S_{8}		T.M.478
Madison	Madison Run,Panama Canal Zone	Conc.gravity	"	2-6.67'x10' rectangular conduits	2-6.67'x10' R.F. slide gates	2-6.67'x10' rectangular conduits	"	-	61,000	154	1:72	S_{8}	$S_{11}S_{10}$	Rpt.[illegible]
Marshall Ford	Colorado River, Tex.	"	"	22-8.5' diam. conduits	22-8.5' diam.R.F. slide gates	22-8.5' circular conduits	"	-	114,500	180	1:60,1:25.5	$P_{8}S_{8}S_{1.8}$	S_{10}	Rpt.[illegible]
Norris	Tennessee Valley Authority	"	"	2-6.67'x10' rectangular conduits	2-6.67'x10' R.F. slide gates	2-6.67'x10' rectangular conduits	"	-	57,600	160	1:72	S_{8}	S_{10}	T.M.[illegible]
Scofield	Scofield, Utah	Earth fill	"	1-6.0' horseshoe tunnel	1-6.2'x4' C.I. slide gates	1-6.0' horseshoe tunnel	Rectangular stilling basin with hump and sill	18	600	50	1:30	S_{1}		Rpt.[illegible]
Shadow Mountain	Colorado-Big Thompson, Colo.	"	"	None	1-30" diam. slide gate	1-6.5' circular conduit	Vulcanic spillway stilling pool	-	350	84	1:30	H_{1}		Rpt.[illegible]
Unity	Burnt River, Oreg.	"	"	1-7.5' circular conduit	2-6.75'x2.75' slide gates	1-7.5' circular conduit	"	-	600	46	1:24	H_{1}		
Vallecito	Pine River, Colo.	"	"	2-8.0' horseshoe conduits	2-6'x9' R.F. slide gates	2-8.0' horseshoe tunnels	Rectangular stilling basin	30	3,500	80	1:60	$S_{8}S_{1}$	S_{10}	
VALVE-CONTROLLED OUTLETS														
Alamogordo	Carlsbad, N. Mex.	"	Alterations to valves	2-6.5' circular conduits	2-24" needle valves	None	Vulcanic spillway stilling pool	-	1,600	116	1:6.5	S_{2}	S_{10}	Rpt.127
Alcova	Kendrick, Wyo.	"	New	1-20' circular conduit	2-24" needle valves	1-20' circular conduit	Trapez.stilling basin	-	8,000	260	1:40	S_{1}	$S_{11}S_{10}$	Rpt.42,87,127
Anderson Ranch	Boise, Idaho	"	"	1-20' circular conduit	6-72" hollow-jet valves	None	Vulcanic spillway stilling pool	-	10,000	224	1:48,1:12,1:6	S_{2},S_{16}		Rpt.100,120,127
Arrowrock	Boise, Idaho	Curved conc.gravity	Alterations to conduits	None	2-48" Ensign valves	2-72" circular conduits	River channel	-	21,000	190	1:24	$S_{1.5}$	S_{8}	Rpt.77,1
Barclett	Salt River, Ariz.	Conc.arch & buttress	New	2-6.0' circular conduits	2-36" needle valves	None	Original stream bed	-	5,600	248 160	1:6	$S_{2.4}$		Rpt.130
Boulder	Boulder Canyon, Ariz.-Nev.	Conc.arch gravity	New and alterations	8-30' circular conduits	12-72" needle valves	8-30' circular tunnels	"	-	80,000	800	1:100,1:60,1:30	$S_{1.5}P_{8}S_{2.5}H_{1.5}$	$S_{10}S_{2.5}S_{12}S_{10}S_{10}$	T.M.410,Rpt.40,47,55,[illegible]
Deer Creek	Provo River, Utah	Earth fill	New	2-6.0' circular conduits	2-48" needle valves	None	Trapez.stilling basin	-	1,500	128	1:17.1	$S_{8}H_{1.8}$		Rpt.143,[illegible],p.[illegible], T.M.[illegible]
Friant	Central Valley, Calif.	Conc.gravity	"	4-110" circular conduits	4-96" hollow-jet valves	"	Rectangular stilling basin	150	16,000	246	1:60,1:24.4, 1:32,1:24	$HP_{8}S_{1.8}H_{1.8.5}$		Rpt.49,133,[illegible]
Ross	City of Seattle, Wash.	Conc.arch	"	2-72" circular conduits	2-72" Howell-Bunger valves	"	Original stream bed	-	6,000	400	1:12	$S_{2.4}$		Rpt.[illegible]
"	"	"		"	2-72" butterfly valves	"	"	-	4,200	270	1:12	$S_{2.4}P_{8}S_{2.4}$		Rpt.157
Seminoe	Kendrick, Wyo.	"	"	2-72" circular conduits	2-60" needle valves	"	Rectangular stilling basin	30	2,600	210	1:32.5	S_{1}		
Shasta	Central Valley, Calif.	Conc.gravity	"	6-8.5' circular conduits	10-102" tube valves	10-8.5' circular conduits	Vulcanic spillway stilling basin	-	22,600	128	1:60,1:40, 1:32,1:16.1	$S_{8}P_{8}S_{1.8}H_{1.8}$		Rpt.[illegible]
				6-8.5' circular conduits					29,200	228				
				6-8.5' circular conduits					16,200	324				
Shoshone	Shoshone, Wyo.	Conc.arch	Alterations to valves	1-10'x10' rectangular with arched roof	2-48" Ensign valves	2-4.33'x10' rectangular conduits	Original stream bed	-	2,200	220	1:6.67,1:6	$S_{1.5}S_{8}$	$S_{10}S_{12}S_{11}$	Rpt.[illegible]
Wickiup	Deschutes, Oreg.	Earth fill	New	2-2.0' circular conduits	2-30" tube valves	None	Rectangular stilling basin	31	4,000	75	1:20	$S_{1}H_{1.5}$		Rpt.26

DETAILED STUDIES OF GATES

Dam	Project	Feature	Type of prototype construction	Type of gate	Size of gate (ft.)	Capacity of one gate at maximum head (c.f.s.)	Head on gate at maximum discharge (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Reason
Boulder	Boulder Canyon, Ariz.-Nev.	Diverting Ariz.spillway tunnel	Rep.of spillway tun.	Wooden bulkhead gate	33'x51'	-	-	1:12		Rpt.155	Model for designing gate.
Davis	Davis, Ariz.-Calif.	Spillway gate slots	New	Stoney slide gate	30'x30'	60,000	30	1:20,1:12	S_{2}		Studies to eliminate cavitation.
Davis	Davis, Ariz.-Calif.	Davis Dam power penstocks	"	Rectangular counter gate	17.5'x26.7'	8,000 per gate	110	1:30	S_{7}	Rpt.150	Computation of drawdown forces.
Friant	Central Valley, Calif.	Friant Dam River outlets	"	"	11.92'x11.92'	4,000 per gate	198	1:12.83	$S_{2.7}$	Rpt.150	Determination of drawdown forces.
Friant	Central Valley, Calif.	Friant Madera canal headworks	"	"	8.92'x9.92'	750 per gate	122	1:12.17	$S_{2.7}$	Rpt.150	Determination of drawdown forces.
Friant	Central Valley, Calif.	Friant Kern canal headworks	"	"	11.92'x11.92'	1,000 per gate	134	1:12.83	$S_{2.7}$	Rpt.150	Determination of drawdown forces.
Grand Coulee	Columbia Basin, Wash.	Grand Coulee Dam power penstocks	"	"	18'x28.5'	12,000 per gate	230	1:30	S_{7}	Rpt.120	Computation of drawdown forces.
Grand Coulee	Columbia Basin, Wash.	Gates for outlet works	"	Ring follower gate	8.5' diam.	-	60	Conceptually	S_{7}	Rpt.25	
Grand Coulee	Columbia Basin, Wash.	"	"	Cylinder follower gate	"	-	60	"	S_{7}	Rpt.25	
Grand Coulee	Columbia Basin, Wash.	Aeration of spillway gates	"	Drum gate	100'x20'	Low discharge	small heads	1:60	S	Rpt.79,88	Aeration study.
Guernsey	North Platte, Wyo.	Spillway gate slots	Successfully in use	Stoney slide gate	30'x30'	52,600	30	1:12	S_{2}		Studies to eliminate cavitation.
Imperial	All-American Canal, Calif.	All-American canal headworks	New	Roller gate	75'x22'	-	2	1:20	$S_{2.4}$	T.M.[illegible],Rpt.75	Determination of operating characteristics.
Kennick	Central Valley, Calif.	Spillway gate slots	"	Stoney slide gate	30'x30'	52,200	30	1:12	S_{2}		Studies to eliminate cavitation.
Lahontan	Nevada, Nev.	Spillway gate	Alterations	Automatic flash gate	30'x4.75'	8,000 per gate approx.	7' on gate sill	1:8,1:4	$S_{2.7}$	Rpt.71	Determination of forces on gate.
Shasta	Central Valley, Calif.	Shasta Dam outlets	New	Rectangular counter gate	11'x11'	5,800 per gate	222	1:17	$S_{2.7}$	Rpt.130,[illegible]	Determination of drawdown forces.
Shasta	Central Valley, Calif.	Shasta Dam power penstocks	"	"	15'x19'	22,000 per gate	230	1:30	$S_{2.7}$	Rpt.130	Determination of drawdown forces.
Shasta	Central Valley, Calif.	Diversion tunnel counter gate	"	Slide gate	23.67'x26'	24,000 at 22% gate opening	190	1:30	$S_{2.4.75}$	Rpt.107	Determination of drawdown forces.
-	Gila, Ariz.	Pumping plant No. 1	Modification	Flap gate	8.0' diam.	800 per gate approx.	-	1:9.6	S_{8}		Reduction of shock on closing.
-	All-American Canal, Calif.	New river siphons	New	Automatic radial gate	7.5'x2.5'	700	12.6' on gate sill	1:6	S	Rpt.27	Determination of operating characteristics.
Anderson Ranch	Boise, Idaho	Flange gaskets for outlet works gates		Ring follower gates	8.0' diam.			Conceptually		Rpt.127	Investigation of rubber flange gaskets.
Boulder	Boulder Canyon, Ariz.-Nev.	Flange gaskets						"		Rpt.25	Investigation of rubber flange for flange gaskets.
Grand Coulee	Columbia Basin, Wash.	Gate seal designs		Various types of slide gates				"		Rpt.50,128	Investigation of rubber gate seals.
Grand Coulee	Columbia Basin, Wash.	Material for outlet gate seals		Ring follower gate	8.5' diam.			"		Rpt.46	Sliding friction tests on various bearing metals.
Shasta	Central Valley, Calif.	Sphere valve seal designs		Sphere valves				"		Rpt.129	Investigation of sealing method.
Utah Lake	Utah Lake, Utah	Spillway gate	New	Automatic radial gate	24'x	2,000 per gate		1:12	6	Rpt.144,[illegible]	Reduction of heating.
Norris	Tennessee Valley Authority	Norris Dam power penstocks	"	Rectangular tractor gate	15.5'x24.5'	-	178	1:28	S_{7}	Rpt.25,[illegible]	Determination of drawdown forces.
Parker	Parker, Ariz.-Calif.	Spillway gate	"	Stoney slide gate	30'x30'		30' on gate sill	1:30,1:12	$S_{2.4.5}$	Rpt.88	Vibration studies.
Parker	Parker, Ariz.-Calif.	Spillway gate slots	Alteration		30'x30'	52,000	30	1:12	S_{2}	Rpt.127	Studies to eliminate cavitation.
Ross	City of Seattle, Wash.	Outlet works at elev. 1240	New	Broome slide gate	6'x8'	2,200 per gate	300	1:12	$S_{2.7}$	Rpt.157	Determination of drawdown forces.

TABLE 1 (Continued)

DETAILED STUDIES OF VALVES

Dam	Project	Feature	Type of prototype construction	Type of valve	Size of valve in inches: Upstream diameter	Size of valve in inches: Downstream diameter	Condition of operation	Capacity of one valve at max.head (c.f.s.)	Max.head on valve (ft.)	Coefficient of discharge at 100% open: Based on upstream area	Coefficient of discharge at 100% open: Based on downstream area	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Remarks
Alamogordo	Carlsbad, N. Mex.	Outlet works	Repair and alteration	Needle valve	48	54	Free discharge	765	116	0.60	0.64	1:8.5	$Q_{2.4}$		Hyd.147	
Anderson Ranch	Boise, Idaho	"	New	Hollow-jet valve	72	"	"	2,000	325	0.70	-	1:12	$Q_{1.6}$		Hyd.144, 149	Development of valve.
Bartlett	Salt River, Ariz.	"	"	Needle valve	79	86	"	1,850	125	0.54	0.77	1:8	$Q_{2.6}$		Hyd.143	
Boulder	Boulder Canyon, Ariz.-Nev.	Penstock by-pass valve	"	Plug valve	6	4	Valve in line	4.0	90	1.00	1.00	Gen.study	Q_1		Hyd.67	
Friant	Central Valley, Calif.	Outlet works	"	Needle valve	110	108	Free discharge	4,000	245	0.52	0.835	1:12.5,1:16.25	$Q_{2.6}$		Hyd.133,90,127,134	
Friant	Central Valley, Calif.	"	"	Tube valve	110	102	"	4,000	245	0.52	0.605	1:12.5	$Q_{2.6}$		Hyd.133	
Imperial	All-American Canal, Ariz.-Calif.	Gila sluicing basin	"	Ball check valve	8	8	Valve in line	2.5	30	-	-	1:1	$Q_{2.6}$			Development of valve.
Ross	City of Seattle, Wash.	Outlet works	"	Hunger valve	72	"	Free discharge	5,800	415	0.64	-	1:12	$Q_{2.6.7}$		Hyd.136,143	
Ross	City of Seattle, Wash.	"	"	Butterfly valve	72	"	"	2,100	290	0.865	-	1:12	$Q_{2.3}P_7Q_{2.4}$		Hyd.137	
Shasta	Central Valley, Calif.	"	"	Tube valve	102	102	Valve in line	3,600	320	0.72	0.72	1:17,1:8.1	$Q_{2.6}$			Development of tube valve operating in airline.
Shoshone	Shoshone, Wyo.	"	Repairs and alteration	56" Masign valve	53.25	52	Discharges into 4.23'x10' rect.tun.	1,100	320	0.61	0.66	1:6.67,1:8	$Q_{2.4}E_2$	E_{11}	Hyd.144,145, 124,127,134	
-	Boulder Canyon, Ariz.-Nev.	Needle valve design		Needle valves	6"		Free discharge	-	500' on model	-	-	Gen.study	$Q_{2.6}.P_1$		Hyd.88, 64	Involves many variations in needle valve design.

SLUICEWAYS

Dam	Project	Feature	Type of prototype construction	Control gates or valves	Type of energy dissipation	Width of stilling pool (ft.)	Capacity at max.head (c.f.s.)	Effective length of crest (ft.)	Approx. drop: headwater to tailwater at max. discharge (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature
Bartlett	Salt River, Ariz.	Sluiceways	New	3-6'x7.5' slide gates	Original stream bed	-	5,800	-	25	1:15,1:10	$P_7Q_{2.6}$		Hyd.143
Imperial	All-American Canal, Calif.	Imperial Dam sluiceway	"	12-radial gates	Rectangular stilling pool	256.5	46,000	192	30	1:40	$AB_{1.2}.AP_{1.2.5}$	$AQ_{10}B_{10}$	T.M.515,
Murdock	Provo River, Utah	Murdock Dam sluiceway	"	1-radial gate	Submerged bucket	16	1,000	16	12	1:40	ABD		Hyd.126

CANAL HEADWORKS

Dam	Project	Feature	Type of prototype construction	Control gates or valves	Type of energy dissipation	Width of stilling pool (ft.)	Capacity at max.head (c.f.s.)	Width at entrance (ft.)	Net width at gate section (ft.)	Approx. drop: headwater to tailwater at max.discharge (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Remarks
Altus	Altus, Okla.	Altus irrigation outlets	New	8-8'x5' H.P.slide gates	Rectangular stilling basin	58	1,000	-	-	44	1:20	$Q_{1.2}$		Hyd.89,92,[illegible]	
Conchas	U. S. Army Engineers, N. Mex.	Conchas canal headworks	"	2-7.5'x5' H.P.slide gates	"	20	700	-	-	46	1:15	$Q_{1.2}$		Hyd.110,166	
Friant	Central Valley, Calif.	Friant-Kern canal outlets	"	4-96" needle valves	"	50	4,000	-	-	115	1:24.5,1:26.4,1:52	$Q_{1.2}$	-	Hyd.110,166	
Friant	Central Valley, Calif.	Friant Madera canal outlets	"	2-72" needle valves	"	28	1,800	-	-	130	1:22.44	$Q_{1.2}$	-	Hyd.68	
Needgate Rock	Colorado River Indian Irr., Ariz.	Canal intake	"	2-22'x15' radial gates	Depressed canal floor	47	8,000	48	44	5	1:60	AQ_2D		T.M.622, [illegible]	
Imperial	All-American canal intake	All-American canal headworks	"	4-75'x23' roller gates	"	400	20,000	200	200	2	1:40,1:20	$AQ_{2.4}D_2$	$AQ_{10}B_{10}$		
Imperial	Gila, Ariz.	Gila canal headworks	"	3-35.67'x16' radial gates	Trapezoidal settling basin	-	2,000	207	107	1	1:40	ABD_2			
Murdock	Provo, Utah	Murdock canal intake	"	1-16'x14' radial gate	Depressed canal floor	16	550	42	16	3	1:40	ABD		Hyd.106	
Owyhee	Owyhee, Oreg.	Tunnel No. 1 headworks	"	4-4.75'x12' H.P.slide gates	None	-	1,600	-	-	68	1:24	$W_2Q_{2.5}$		T.M.552,[illegible]	Calibration of control flume.
Ross	Yakima, Wash.	Ross canal intake	"	1-23'x15' radial gate	"	-	2,200	120	26	3	1:48	AQ_2E		T.M.496,Hyd.86	
Shoshone	Shoshone, Wyo.	Heart Mountain canal intake	"	1-10' circular ring gate	Chamber employing staggered baffles	-	1,200	-	-	100	1:12	$PP_2.5Q_2$		Hyd.98	
At Grand Lake	Colorado-Big Thompson, Colo.	Adams tunnel intake	"	2-8'x10' radial gates	None	-	1,500	220	19.5	2	1:10,1:24	$AQ_1L_2P_1$		Hyd.151	
Flathead Reservoir	Sun River, Montana	Flathead headworks	Alterations	8-3.75'x8' slide gates	Weir and baffles in trapezoidal stilling basin	-	1,000	-	-	18	1:12	Q_2E_1		Hyd.61	

RIVER MODELS

Feature	Project	Scale of model	Available literature	Purpose of study
All-American canal headworks at Imperial Dam	All-American Canal, Calif.	1:40	[illegible]	Study of silt deposition and the effect of sluicing at the headworks of the All-American Canal.
River downstream from Boulder Dam	Boulder Canyon, Ariz.-Nev.	1:160	[illegible]	Study of flow conditions and erosion in river downstream from Boulder Dam produced by discharge from spillways, outlet works, and penstocks.
" " " " "	" " " "	1:60	Hyd.163	" " " " "

TABLE 1 (Continued)
CANAL STRUCTURES

Feature	Proj.	Location	Type of prototype construction	Control gates	Type of energy dissipation	Width of stilling pool (ft.)	Capacity at max. head (c.f.)	Width of chute or length of crest (ft.)	Approx. drop - headwater to tailwater - at max. discharge (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Remarks
CANAL CHUTES AND DROPS														
Drop No. 1	All-American Canal, Calif.	Sta. 1908+00	New	6-16.25'x5' radial gates 2-15.2' x5' radial gates	Rect. stilling basin	118	8,700	91.8' at gate section	16	1:40	$ABCD_5$[22]			Take-out to Coachella Canal.
Drop No. 2	" " " "	Sta. 2101+00	"	4-16'x16' radial gates	Two rect. stilling basins	2 at 32	6,500	60' at gate section	20	1:36	$ABCD_{1.5}$[22]			Power drop
Drop No. 3	" " " "	Sta. 2472+00	"	" " "	" " " "	2 at 32	6,200	" " "	24	1:36	$ABCD_{1.5}$[22]			" "
Drop No. 4	" " " "	Sta. 2759+00	"	" " "	" " " "	2 at 31	7,000	68' at gate section	40	1:28	$AB_1C_{1.2}D_{1.2}$[22]			" "
Drop No. 5	" " " "	Sta. 3326+00	"	4-10'x15' radial gates	" " " "	2 at 22	4,200	40' at gate section	28	1:36	$ABCD_{1.5}$[22]			" "
Siphon drop turnout	" " " "	Sta. 978+00	"	4-7'x5' slide gates	Rect. stilling basin	33.5	2,000	30' at gate section	19	1:27	AB_2CD			
Coachella power drop	" " " "	Sta. 290+00	"	2-13.5 radial gates	" " " "	26.5	2,200	27' at gate section	40	1:24	$ABCD_1$		Hyd.102,	Power drop.
2.5-foot drop structures	Payette Division, Boise, Ida.	Tunnel No. 2 waterway	"	None	" " " "	-	1,000	-	2.5	1:12	BCD		Hyd.121	
Rocha Arroyo chute drop	Rio Grande, N. Mex.	Cabello reservoir	"	"	Free bucket	-	16,000	Trapez. chute	65	1:50	$A_1B_1C_1D_1.5$		Hyd.37	
Hill Coulee chute	Sun River, Mont.	Sta. 255+00	Alteration	"	Trapez. stilling basin	-	140	5' rect. chute	110	1:10	$C_1D_1.5.5$		Hyd.48	
Spring Valley canal chute	" " "	Sta. 661+00	Replacement	"	Flaring rect. stilling basin	-	1,500	Flaring rect. chute	100	1:16	CD	$B_1C_{10}D_{10}$	Hyd. 41	
" " " "	" " "	Sta. 677+00	Alteration	"	Trapez. stilling basin	-	1,500	12' rect. to trapez.	160	1:16	CD_1	$B_1C_{10}D_{10}$	Hyd. 41	
" " " "	" " "	Sta. 735+00	"	"	" " " "	-	1,500	Trapez. chute	10	1:16	BCD	$B_1C_{10}D_{10}$	Hyd. 41	
South Canal chute	Uncompahgre, Colo.	Sta. 26+00	Replacement	"	Flaring rect. stilling basin	-	960	Flaring rect. chute		1:20	C_1D_1	$C_{10}D_{1.10}$	T.M.48	
Sunnyside main canal	Yakima, Wash.	Check drop No. 6	Alteration	Drop logs	Trapez. stilling basin	-	1,500	27.1' at gate section	1.5	1:16	$A_1B_1D_1.5$	$A_{10}B_{10}C_{10}D_{10}$	Hyd.40,57,64,101,[20],[12]	
CANAL WASTEWAYS														
Alamo River wasteway	All-American Canal, Calif.	Sta. 3254+00	New	2-7.5'x5' slide gates	Flaring rect. stilling basin	-	1,500	Flaring rect. chute	24	1:24	$AB_4C_1D_{1.5}$			Also river crossing.
								18' at gate section						
New River wasteway	" " " "	Sta. 4903+00	"	4-7.5'x5.5' radial gates	Rect. stilling basin	50	5,500	Flaring rect. chute	47	1:36,1:16	AB_4CD_5		Hyd.27	" " "
								30' at gate section						
Pilot Knob wasteway	" " " "	Sta. 1100+00	"	4-14'x10' radial gates 2-7.5'x5' radial gates	" " "	140	12,000	62' rect. chute	55	1:36,1:12	AB_4CD_5		Hyd.38	"
Calexico Creek side channel wasteway	Central Valley, Calif.	Sta. 1727 Contra Costa canal	"	None	" " "	3	20	25' free crest	16	1:6	$AB_{1.4}C_1D$			
Friant Kern canal wasteway	" " "	Wasteway entrance	"	2-12' radial gates	" " "	-	3,500	26' at gate section	-	1:16	AB_4		Hyd.120	Wasteways to prevent over-topping of canal banks.
Fortuna wasteway	Gila, Ariz.	Sta. 946 Gravity main canal	"	2-10'x8' automatic radial gates	Flaring rect. stilling basin and perforated weir	-	5,000	20' at gate section	18	1:30	$ABCD_5$			
Wash overchute & wasteway	" "	Sta. 170 Gravity main canal	"	2-7.5'x5' radial gates	Rect. stilling basin	-	1,500	-	7	1:24	AB_4CD			Applies to wasteway only.
Scott W. canal wasteway	Shoshone, Shoshone	Sta. 1068+00	"	2-6'x6' slide gates	" " "	20	340	10.5' rect. chute	75	1:15	B_4			
Wasteway No. 2	Yakima, Wash.	Yakima Ridge canal	"	2-10.25'x12' radial gates 4 siphons	" " "	40	2,100	22' rect. chute	140	1:24,1:8	ABCD		Hyd.109,[19]	2,100 second-feet capacity exclusive of siphons.
CANAL WASH OVERCHUTES														
Wash overchutes	All-American Canal, Calif.	Coachella canal	"	None	Irregular	-	Various	Various	Various	1:24,1:36,1:19	$ABCD_5$		Hyd.111,	Typical of some 52 wash overchutes.
Wash overchute & wasteway	Gila, Ariz.	Sta. 170 Gravity main canal	"	"	Rect. stilling basin	-	2,500	60	15	1:24,1:16	$A_1B_1C_1D_1.5$		Hyd.123	Applies to overchute only.

FISH STRUCTURES

Dam	Project	Feature	Discharge through structure (second-feet)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Remarks
Keswick	Central Valley, Calif.	Fish ladder & trap	500	1:20,1:20,1:10	C_1B		Hyd.135	
Rock Island	U. S. Army Engineers	" " "	-	1:8	C			
Ross Diversion Dam	Yakima, Wash.	Two fish ladders	25 through left ladder	1:48	CB		Hyd.55,T.M.494	
-	Central Valley, Calif.	Nelle Ferry fish rack & trap	1,000-15,000	1:8	$C_{1.4}B$		Hyd. 88	

SIPHONS

Feature	Project	Location	Capacity per siphon at max. head (c.f.)	Max. head - headwater to tailwater (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Remarks
Low head siphon	Colorado-Big Thompson, Colo.	Estes Park sewage disposal system	1.7	1.4	1:1	BC_1F		Hyd.80	True siphon.
Grand Coulee Pumping plant	Columbia Basin, Wash.	Siphon elbow at outlet	1,600	-	1:24	$C_{1.5}F$		Hyd.143	Located on end of pump line.
Mahkeur River siphon	Owyhee, Oreg.-Ida.	North canal	425	-	1:8	0			Inverted siphon.
Wasteway No. 2	Yakima, Wash.	Yakima Ridge canal	360	11	1:8	BC_1F		Hyd.109,62,[19]	True siphon.

ENTRANCES TO CLOSED CONDUITS

Dam	Project	Feature	Type of entrance	Max. head on model (ft.)	Scale of model	Extent of investigation: Model	Extent of investigation: Prototype	Available literature	Remarks
Boulder	Boulder Canyon, Ariz.-Nev.	Intake tower	Determination of loss through intake tower	240	1:24	$B_{1.5}$		T.M.545,[9],[18]	
Grand Coulee	Columbia Basin, Wash.	Entrance to river outlets	Circular bell mouth to 8.5' diam. pipe	93	1:24.45,1:24	B_2	$B_{2.10}$	Hyd.66,81,101	
Parker	Parker, Ariz.-Calif.	Intake to power penstocks	Test to check pressures at entrance to penstocks	46	1:20	B_2			
Rose	City of Seattle, Wash.	Entrance to outlet works at elev. 1340	Rectangular entrance to 8.0' diam. pipe	290	1:12	$B_{1.5}$		Hyd.187	

HYDRAULIC-JUMP MODEL STUDIES

Dam	Project	Feature	Purpose of tests	Scale of model	Available literature
Boulder	Boulder Canyon, Calif.	Design of overfall sections	Experimental study to aid in the design of overfall dam shapes.	Gen. study	Hyd.122,116,61,[4]
Boulder	Boulder Canyon, Ariz.-Nev.	Turbine energy absorbers	Design of device to absorb energy from turbine relief valves.	1:20	T.M.573,Hyd.21.1
Elephant Butte	Rio Grande, N. Mex.	Penstocks tailrace	Development of a tailrace in which the discharge of outlet valves would not affect the efficiency of the turbines.	1:20	Hyd. 21
Grand Coulee	Columbia Basin, Wash.	Spillway drum gates	Method of preventing ice from forming on the spillway drum gates.	Gen. study	Hyd.28,32,[2]
" "	" " "	Caisson for spillway bucket repairs	Model to aid in the design of a caisson for inspection and repair of spillway buckets.	1:40,1:20	Hyd. 134
" "	" " "	Spillway bucket	Studies concerning maintenance and repair of spillway buckets.	1:40,1:20	Hyd.122
Green Mountain Dam	Colorado-Big Thompson, Colo.	Power penstocks	Comparison of losses in different types of penstock elbows.	1:29,67	Hyd. 96
Imperial	All-American Canal, Calif.	Smashing basin scrapers	To determine characteristics of silt material and power required to move scrapers	Gen. study	T.M.580
"	" " " "	Influent sluice to desilting basins	Investigation to find an efficient method of conducting influent into desilting basins.	1:1 horizontal	
"	" " " "	Percolation under dam	Study to determine extent of percolation under Imperial Dam.	1:20	
"	" " " "	Ejectors on D. S. face of dam	Tests to improve efficiency of ejectors conducting seepage water from within dam to river downstream.	1:6,1:1.4	
Parker	Parker, Ariz.-Calif.	Fording to power penstocks	Investigation of flow entering power penstocks.	1:40,1:20	
Shasta	Central Valley, Calif.	Power penstocks	Determination of losses in power penstocks.	1:25	
"	" " "	Antioch steam plant	Tests to investigate extent of recirculation of condenser cooling water from San Joaquin River.	1:100	Hyd.124

REPORTS OF FIELD TESTS (NOT BY LABORATORY PERSONNEL)

Dam	Project	Feature	Nature of tests	Extent of tests	Available literature	
Alcova	Kendrick, Wyo.	Outlet works	Inspection to study operation of outlet works.	$C_{10}B_{2.10}$	Hyd. 42	
Black Canyon	Boise, Ida.	Spillway drum gates	Investigation concerning vibration of sheet of water flowing over drum gates.	$B_{3.10}$	Hyd. 86	
Boulder	Boulder Canyon, Ariz.-Nev.	Tunnel plug outlet works	Determination of operating characteristics of tunnel plug outlets.	CB_{10}	Hyd. 49	
"	" " " "	Power and outlet works penstocks	Measurement of losses in upper Arizona penstock.	$B_{2.5,10}$	T.M.510	
Grand Coulee	Columbia Basin, Wash.	River outlet works	Investigation of pressures in outlet conduit (intermediate tier).	$B_{2.10}B_{2.5}B_{2.10}$	Hyd.61,101,[12]	
Parker	Parker, Ariz.-Calif.	Spillway	Calibration of spillway for partial gate openings	B_4		
Shoshone	Shoshone, Wyo.	Outlet works	Inspection to investigate damage and recommend repairs.	CB_{10}	Hyd.117	
"	Buffalo Rapids, Mont.	Glendive pumping plant	Performance of acceptance tests on pumps and related equipment.	B_4	Hyd.70,91	
"	Lower Yellowstone, Mont.	Thomas Point pumping plant	Performance tests, mechanical inspection and recommendations for repairs.	$B_{4.10}$	Hyd.135	
"	Klamath, Oreg.	Pumping plant D, Tule Lake, Calif.	Performance of acceptance tests on pumps and related equipment	$B_{4.10}$	Hyd.139	
"	Yakima, Wash.	Wasteway on Kittitas main canal, sta.1346+80	Study of flow of water in open channels with high gradient.	$C_{1.5,3.10}$	Hyd.55,60,65	

Explanation of Column Titles

Extent of Model Investigation on Table 1

Open-type structures	A. Entrance to spillway, sluiceway or canal headworks
	B. Overflow section
	C. Flow on face of spillway, in side channel or chute
	D. Spillway stilling basin
Closed type structures	E. Entrance of outlet works, sluiceways, or canal headworks
	F. Flow upstream from control gates or valves
Closed type structures	G. Study of gates or valves proper
	H. Flow downstream from control gates or valves
	I. Outlet stilling basin
Pumps and turbines	J. Entrance to pump intake or turbine penstock
	K. Intake line to pump or penstock to turbine
	L. Study of pump or turbine proper
	M. Pump discharge line or turbine draft tube
Siphons	N. Entrance to siphon
	O. Flow through siphon
	P. Siphon exit
Fish structures	Q. Flow through fish structure
	R. Flow at exit to fish structure

Explanation of Column Titled

Extent of Model Investigation on Table 1 (continued)

Letter alone indicates visual investigation of model.

Letter with numerical subscript indicates detailed study and type.

Subscript

1. Designates that water surface measurements are available.
2. Designates that pressure measurements are available.
3. Velocity measurements available.
4. Coefficient of discharge or rating curves prepared.
5. Scour or deposition patterns recorded.
6. Efficiency determinations.
7. Torque measurements, moments, thrust or down pull determinations.
8. Shock or vibration investigations.
9. Measurement of air entrainment or air supply
10. General photographs
11. Record, either photographic or actual measurements, of pitting produced by cavitation or impingement.

later been obtained of the prototype structure in operation, the extent of this information is indicated by the code in column 14. Reference to all literature, including hydraulic laboratory reports, technical memoranda and other published accounts relating to the hydraulic model testing of these structures, are included in the order of their importance in column 15. In general, the hydraulic laboratory reports contain the more complete records of the studies. HYD refers to hydraulic laboratory reports while T. M. indicates technical memoranda both in the files of the Bureau of Reclamation. The asterisks refer to published literature which is available from sources outside of the Bureau. The latter references are listed in the bibliography at the back of the Manual.

11. Spillways in general. More spillways have been investigated in the hydraulic laboratory than any other type of structure as is evident from table 1. A spillway on a dam is the safety valve provided to pass the maximum flow of the stream over or through the dam during flood season. For this reason, spillway design has not been left to chance or entirely to past design practice but is checked by the aid of hydraulic models. In some cases it has been actually necessary to develop a spillway design in the model. This was true in the case of the design of steep superelevated channels for the Ross Dam spillway as no precedent existed for this type of design. Spillways fall into a number of types depending on the kind of dam involved, the quantity of water to be handled, the nature of the topography at the dam site and the foundation material. Although the list in table 1 contains 57 spillways, no two are exactly alike. An attempt will be made to discuss these types and point out the extent of general design information obtained from the models.

12. Morning-glory spillways. Of the four morning-glory spillways listed on table 1, the one at Gibson Dam was in existence at the time the model studies were made. Tests on the Owyhee spillway were made

coincident with the design of the dam. The two others were abandoned before the model tests were completed. In the first case, gates and piers were added to an existing free crest, figure 20B, to increase the storage capacity of the Gibson Reservoir. As the photograph indicates, symmetry of flow to a spillway of this type is important. By means of the model, the piers and gates were arranged to produce the best flow possible under the limiting conditions. From column 13, table 1, the model study included design of the piers from visual observation of the flow, redesign of the overflow shape by means of pressure measurements on the overflow face, head-discharge calibration for the altered design, and observation of the flow in the vertical shaft and elbow below the overflow section.

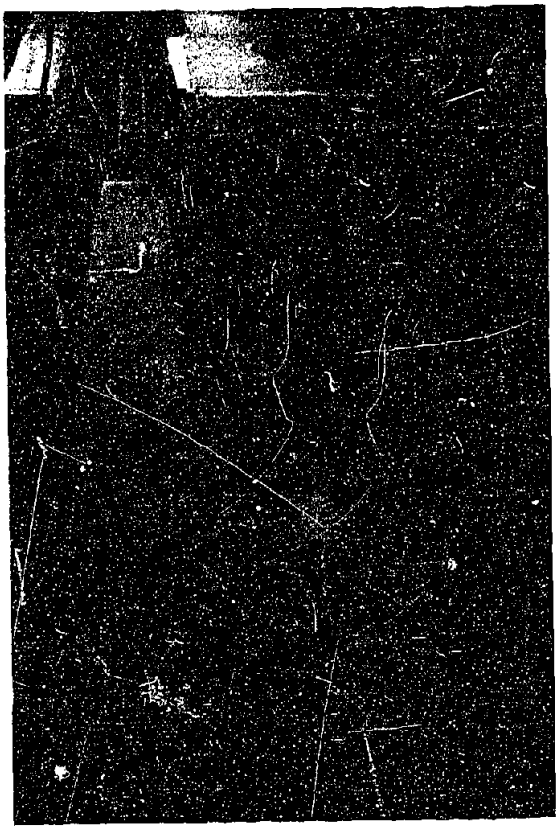
The Owyhee spillway was regulated by a ring gate, figure 20A. The model tests were similar to those above but also included water-surface measurements of the approach area and the overflow section. The flow lines in the photograph on figure 20A were produced by sprinkling small pieces of paper on the water and taking a time exposure. The model tests in these two cases provided assurance that the flow approaching the spillway was uniform, that the shape of the overflow was such that negative pressures would be small or nonexistant on the face, that the spillways were of sufficient capacity, and that flow in the tunnels and bends was satisfactory.

13. Side-channel spillways. The side-channel spillways investigated were of two general types, the first consisting of an overflow section on one side, figure 20C, and the second employing overflow sections opposing one another as in figure 20D. From the side channel, water is conducted to the river by either a closed conduit or an open chute. At the termination of the tunnel or chute, a stilling basin or similar device is often required before the water can be allowed to enter the river. In the case shown in figure 20C, an open trapezoidal chute and

A - Owyhee Ring Gate Spillway

B - Gibson Spillway controlled by Radial Gates

MORNING GLORY SPILLWAYS



C - Big Sandy No. 2 Single Side Channel Spillway

D - Box Butte double side channel Spillway

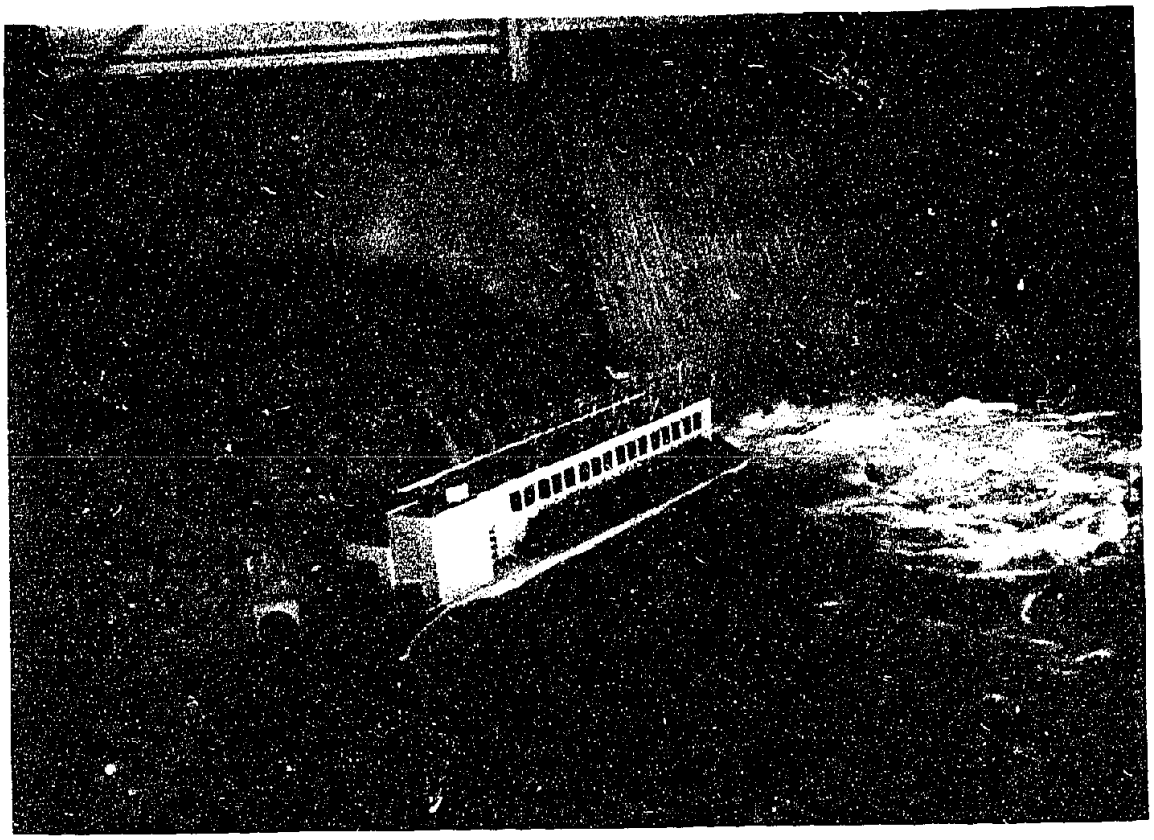
SIDE CHANNEL SPILLWAYS

trapezoidal stilling basin were employed. In the second case, figure 20D, a rectangular chute and stilling pool were used for conducting the flow to the river. The spillway discharge can be controlled by a slide gate at the entrance to the chute.

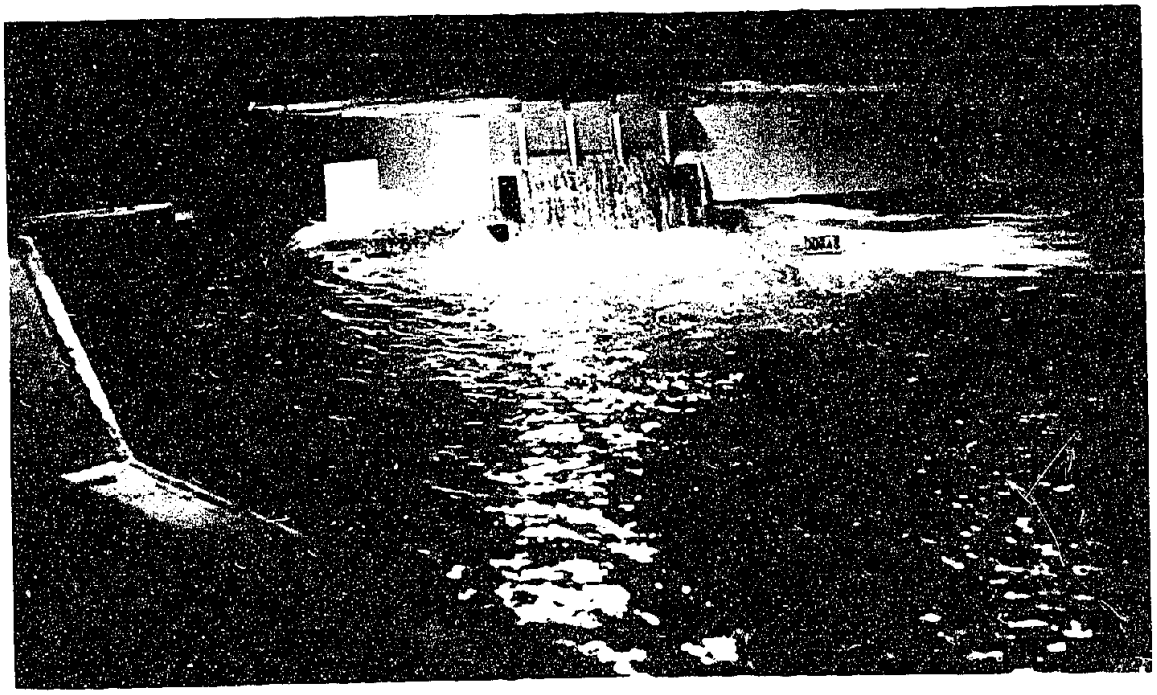
From column 13, table 1, the model work on these spillways involved a study of the approach conditions to the spillways, investigation of the flow over the ogee or overfall sections, in the side channels, in the chutes and in the stilling basins. A head-discharge rating curve was also obtained for each model. The best known treaties on the proportioning of side-channel spillways was written by Mr. Julian Hinds(27). The original side-channel spillways for Boulder Dam were designed according to this theory and it was found that satisfactory agreement existed between the model results and the analytical determinations. A comparison of the two is shown on pages 29 to 32 inclusive of bulletin 1, part VI of the Boulder Canyon Final Reports (1).

14. Overfall spillways. Fifteen overfall spillways designed by the aid of hydraulic models are listed in table 1. These vary in height, shape, type of gates, and method of energy dissipation. Model photographs of two typical overfall spillways are shown on figure 21 and line sections of these spillways are shown on figure 22C and D. The first is of Shasta Dam, the highest dam of this type constructed to date, having drum gates on the crest, and a stilling basin utilizing a sloping apron (figure 22C). The second is of Davis Dam, on which large slide gates will control the flow over the spillway crest and the stilling device will consist of a submerged roller bucket (figure 22D). During the course of model testing in the past 14 years, much general information has been accumulated and compiled to aid the designer. Where applicable, references to this material will be mentioned throughout this report.

The first consideration in designing a spillway, whether of the overfall, side-channel or morning-glory type, is the shape of the over-



A - Shasta Dam Spillway

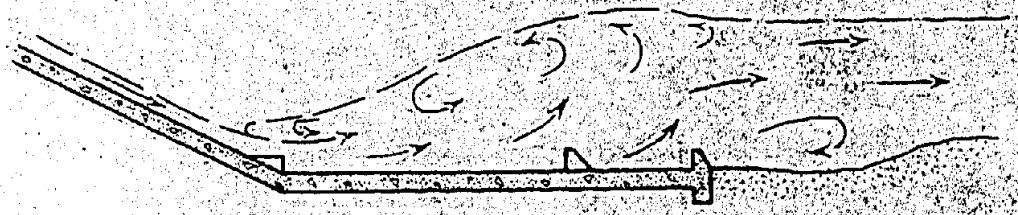


B - Lemon Dam Spillway

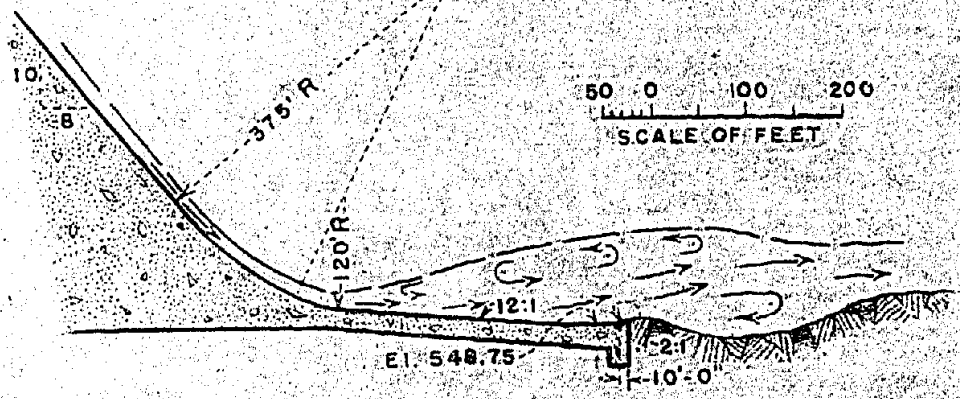
OVERFALL SPILLWAYS

FIGURE 22

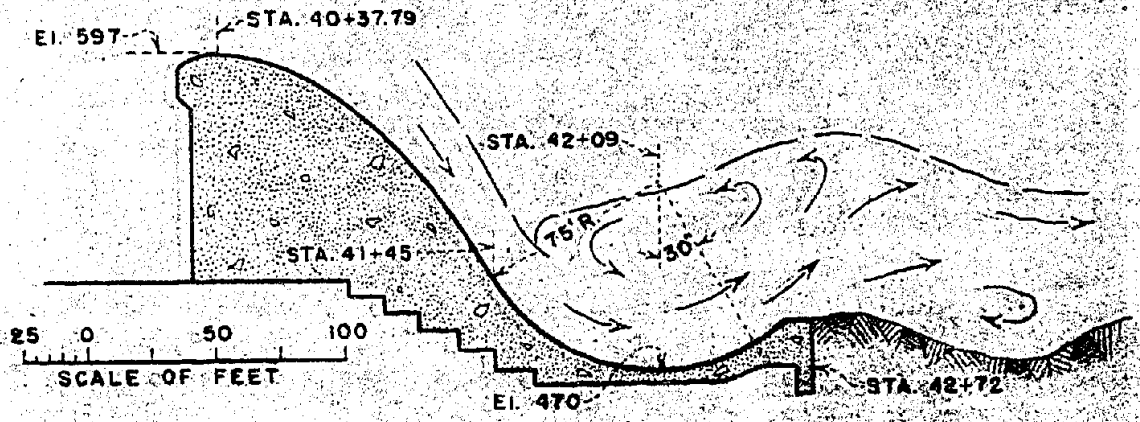
A. HORIZONTAL APRON



B. HORIZONTAL APRON WITH SPREADER TEETH AND BAFFLE PIERS



C. SLOPING APRON FOR SHASTA DAM



D. ROLLER BUCKET FOR DAVIS DAM

STILLING BASIN DESIGN
FOR SPILLWAYS

flow crest section. When possible it is usually desired to shape the overflow section to fit the lower nappe of the overfalling stream for the maximum discharge condition. This will result in the most economical as well as the most efficient shape of section on which subatmospheric pressure will be small or nonexistant. The criterion of proportioning overfall sections for little or no subatmospheric pressure in therefore a desirable basis for presenting this type of information to the designer. This has been done (6) from information gained from an extensive series of experiments which were performed in the hydraulic laboratory. The report covers the design of overfall sections quite thoroughly for any practical condition of design. Examples are also included to illustrate the method proposed for proportioning overfall sections.

It is not always possible to design a spillway section according to this criterion as, first, enlargement of the crest section is sometimes necessary to accommodate gates, secondly, a large spillway may not be structurally stable when designed according to this criterion, and thirdly, it may be desirable occasionally to construct a spillway with vacuum existing between the dam face and the sheet of water flowing over it. The practice of designing to utilize the vacuum head is to be discouraged except in special cases, and then, by persons thoroughly familiar with the problem. Although vacuum increases the discharge for a given head its value must be known within narrow limits for safe design.

15. Spillway stilling basins. The design of stilling basins for large dams is of greatest importance as a poorly designed basin can threaten the existence of an entire dam by undercutting of the foundation material. Stilling basins for the dams shown on table 1 have been conservatively determined. The criterion in design is to produce a hydraulic jump to dissipate and convert high velocity flow into a less destructive form. The purpose of the stilling basin, in which the jump is contained, is to obtain, insofar as feasible, an agreement between the tailwater for a given discharge and the height of water necessary to form a good hydraulic jump.

The relationship of the individual factors for a hydraulic jump (29) (30) (31) is given by the momentum formula:

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

the terms are explained in figure 22A. In the case of the horizontal stilling pool, the floor, which is commonly known as the apron, is at an elevation such that the tailwater depth above it will equal D_2 in the above expression for the maximum discharge condition. Horizontal stilling-pool aprons are common on smaller structures but are the exception on high dams as they are necessarily lengthy and expensive. The length of apron necessary to develop the jump is usually considered $6D_2$ for the maximum condition but this can be shortened by incorporating on it spreader teeth, baffle piers and sills as shown in figure 22B.

Over a period of years, general design data has been compiled from the laboratory on stilling basins with baffle piers, page 13 reference (32), but these are not recommended for large structures. A sill, properly placed at the end of the apron (figure 22A), produces a ground roller which tends to continually move loose material upstream against the end of the apron establishing protection from undermining. The sill on the end of the apron also controls the location of the scour by limiting it to a definite distance downstream. The position and extent of the scouring, in loose material, varies with a change in discharge or tail-water depth. The difficulty often encountered in the design of horizontal-apron stilling basins is that the position of the jump is very sensitive to a change in the value of D_2, and future retrogression of the river downstream from the dam may cause the jump to move completely off the apron.

By placing the stilling basin floor on a slope, it is possible to locate it so as to produce an approximate coincidence between the tail-water curve and the jump-height curve for all conditions of discharge.

As a result, the beginning of the jump will remain at approximately the same point on the apron for all discharges, the actual length of the jump increasing with the discharge. In the case of the horizontal floor, cited above, the front of the jump moves downstream as the discharge increases with the result that the effective portion of the apron is shortest at the greatest discharge. This is exactly the reverse of what is desired and the sloping apron tends to correct this condition. A method outlining the procedure for the design of a sloping apron can be found on page 145 of reference (5) of the bibliography. A sloping-apron design is employed in the Shasta Dam stilling basin, figure 21A and 22C.

A third type of spillway stilling basin, the roller bucket, is proposed for Davis Dam, figure 21B and 22D. This consists of a semi-circular bucket at the toe of the spillway. To perform best, the bucket requires a greater tailwater depth than the customary apron-type pool. Buckets have been used on low diversion dams with moderate tailwater depths but are more applicable to a case like Grand Coulee Dam in which a roller bucket operates under excessively high tailwater. In cases where the tailwater depth is too great to obtain a good hydraulic jump on a flat or sloping apron, the roller bucket becomes the cheapest and most satisfactory solution. General information now available on the design of roller buckets, as listed in table 1, is meager, and a program of tests to obtain data of this nature is in progress in the laboratory. An explanation of the manner in which the roller bucket operates on Grand Coulee Dam is given in references (3) and (4).

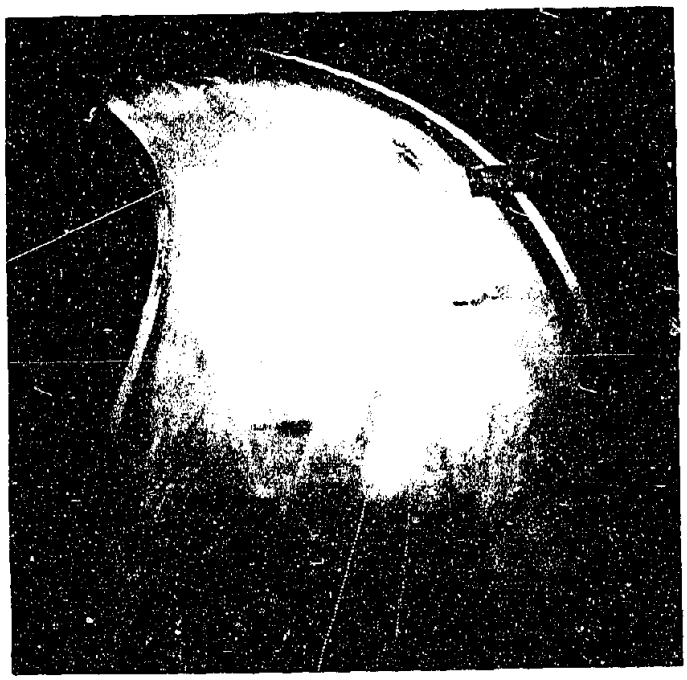
16. Superelevated spillways. Practically all of the spillways listed under the heading of superelevated spillways on table 1 will also fit into other classifications. Superelevation on spillways is not generally well understood and for this reason the spillways which were tested with superelevation features, have been listed as a group. In the design of superelevated channels for spillways, there are two salient points.

First, that it is possible to superelevate properly for only one discharge, which, in most cases is the maximum expected discharge. For smaller discharges the flow will tend to concentrate toward the inside curve of the channel. It is therefore necessary to investigate the height of the confining walls on the inside curve to be sure that these will not be over-topped at the lower flows. Second, it is less difficult to superelevate for the moderate velocities than for supercritical flow. It is therefore good practice to arrange the spillway alinement so that superelevation can begin where velocities are low, if possible. This has been done on the spillways for the Mormon Flats and Vallecito dams, photographs of which are shown on figures 23A and 23B, respectively. In both instances the superelavation is completed in flat portions of the spillways. In the Mormon Flat installation the fall is not excessive until after the water leaves the paved channel. In the Vallecito spillway the channel narrows, the gradient increases, and the alinement straightens downstream from the superelevated portion.

It is not always possible to superelevate for low velocities. This is exemplified in the Ross Dam spillways, in which it was necessary to superelevate for rapidly accelerating flow with high initial velocity. A photograph of a model of the Ross Dam spillways is shown on figure 23C. Dependable, laboratory-checked, design data have been established for superelevation on flat gradients, available in HYD 74, and references (8) (33) (34). On the other hand, dependable general design data for superelevation on steep gradients is not available, and the only safe and satisfactory method of design is reliance on the results of a trial and error test program performed on a hydraulic model. A description of the method used in determining the alinement, superelevation, wall positions and heights for the Ross Dam spillways are described in HYD 136.

17. <u>Closed-chute spillways</u>. True closed-chute spillways are not common. However, a number of the spillways listed under different

A - Mormon Flats Spillway



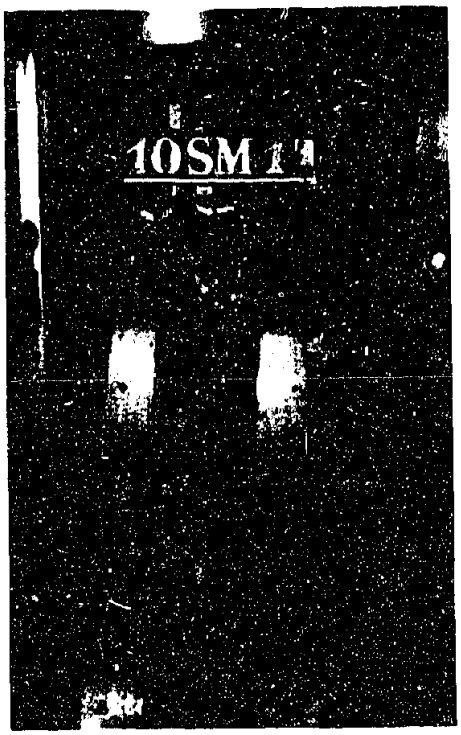
B - Superelevated portion of Vallecito Spillway

C - Ross Dam Spillways

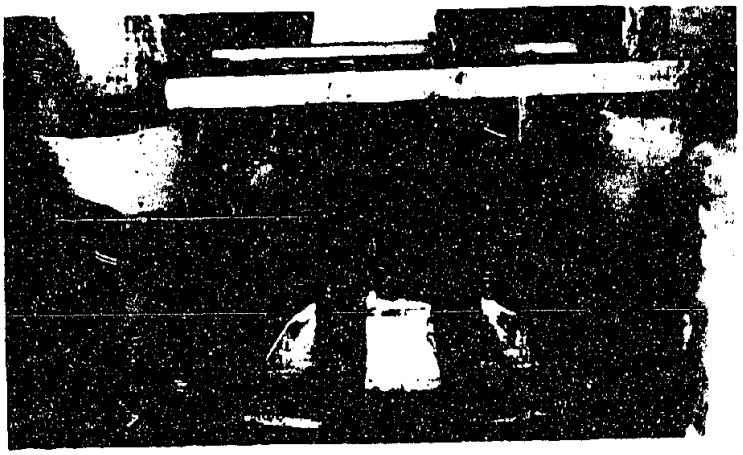
SUPERELEVATED SPILLWAYS

classifications on table 1 can also be loosely classified as closed-chute spillways. Photographs of a model of a closed-chute spillway are shown on figures 24A and 24B. No collecting structure is employed, such as side-channel spillway; instead the gate structure is located at the entrance to the chute. This necessitates placing piers and gates in the transition to the curved chute which results in complicated curvalinear flow. In the design of large structures of this type a model study is advisable to be certain that the spillway in question will actually operate as anticipated. The only reference to model studies on closed-chute spillways are those shown in table 1.

18. Flat open-chute spillways (rectangular in cross section). Chute spillways are common on earth dams and are usually designed to follow as nearly as possible the profile of the dam. They are customarily flat for some distance downstream from the gate section, then the gradient steepens as the stilling basin is approached. The entire spillway usually consists of vertical counterforted concrete walls and paving. Photographs of three earth dam spillway models are shown on figure 25. The Fresno Dam spillway, figure 25A, is typical of this type. The chute of the Alamagordo Dam spillway, figure 25B, converges, then diverges as it approaches the stilling basin. This is a means of reducing the cost of the structure, yet the flow must follow these walls or the action in the stilling basin will not be satisfactory. Deer Creek Dam spillway, figure 25C, differs from the former two in that a narrow chute is utilized with diverging section immediately upstream from the stilling basin. In every case the stilling pools consist of horizontal aprons with spreader teeth and dentated sills. The models were employed to observe approach conditions to the spillways, flow over the crest or through the gate sections, the spreading of the jets in the chutes, the action in the stilling basins, and the nature of the flow on entering the rivers downstream. Features that perform unsatisfactorily were revised until the desired results were obtained.

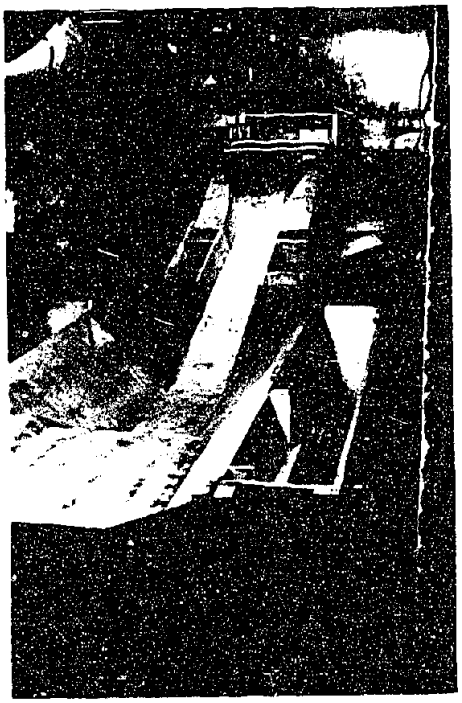


A - Chute of Seminoe Dam Spillway
(Cover Removed)



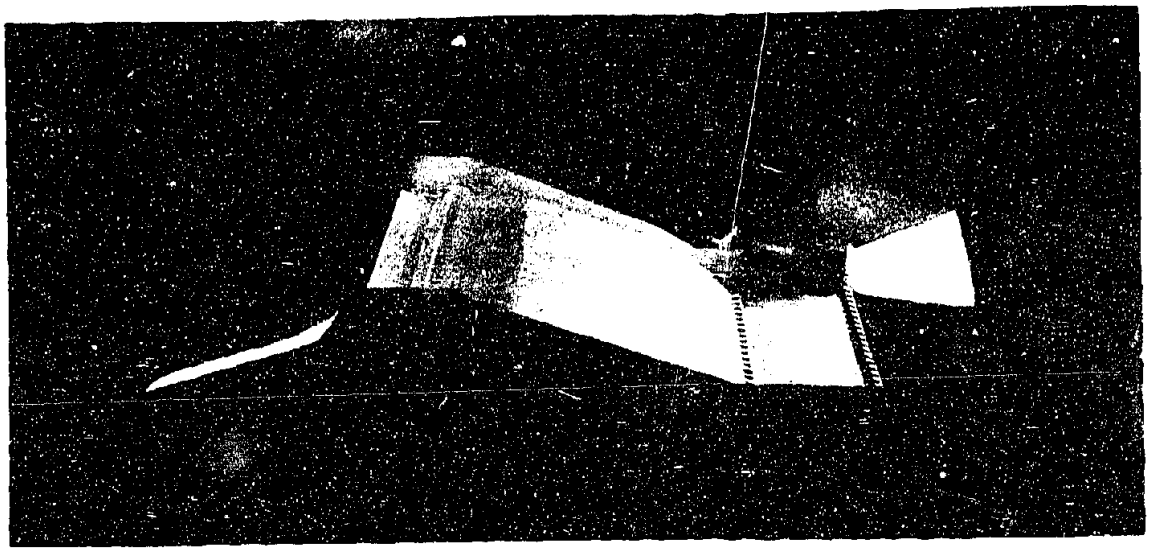
B - Entrance to Seminoe Dam Spillway

CLOSED CHUTE SPILLWAYS



C - Spillway Trapezoidal in cross section

FLAT OPEN CHUTE SPILLWAYS (TRAPEZOIDAL IN CROSS SECTION)



A - Fresno Dam Spillway

B - Alamagordo Dam Spillway

C - Deer Creek Dam Spillway

FLAT OPEN CHUTE SPILLWAYS (RECTANGULAR IN CROSS SECTION)

When a spillway is flat through the gate section as in figures 25B and 25C, a low discharge coefficient results. When feasible, the coefficient can be increased and the crest width decreased a corresponding amount by the addition of a small ogee section such as shown on the Fresno Dam spillway, figure 25A. The ogee produces a steeper draw-down curve immediately downstream from the crest allowing a larger discharge to pass through this section for a given head. The coefficients of discharge for maximum flow conditions on the Alamagordo and Deer Creek dam spillways do not exceed 3.20. On other spillways where the slope of the chute is extremely flat immediately below the gate section, the coefficient of discharge is often less than 3.0. The coefficient for maximum flow conditions on the Fresno Dam spillway, on which the ogee section was utilized, approaches 4.0. The method and source of information required for the proper design of this type of ogee section on earth dam spillways can be obtained from references (6) and (35). This particular problem is treated as example 1 in the latter reference.

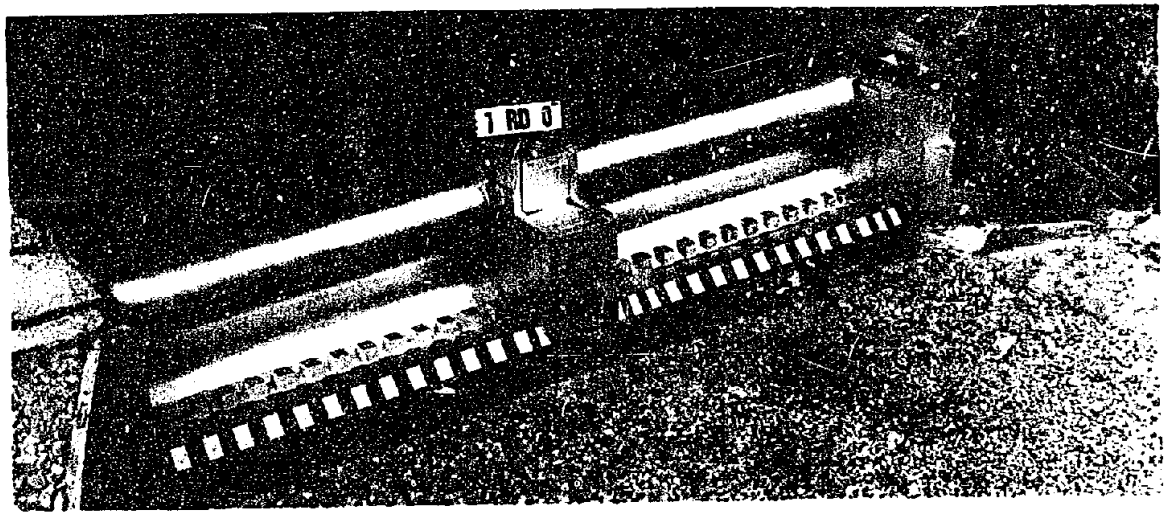
The chief concern in the design of the spillway chute proper is that the flow will spread to a nearly uniform depth across any section and remain in constant contact with diverging walls. Another consideration is that the walls will not be overtopped for some unsymmetrical flow produced by unequal gate openings. It is usually recommended that gates be operated in symmetrical combinations wherever possible, and this applies to all types of spillways.

Symmetry of flow entering the stilling basin is essential for satisfactory energy dissipation. Rectangular stilling basins with horizontal aprons are common for earth-dam spillway designs. Many of these employ spreader teeth, baffle piers, and sills in various combinations as it is possible in this way to shorten the length of horizontal apron as much as one-third. General design data for this type of stilling basin can be found from reference (32). This type of design is not recommended

for the larger installations such as Shasta and Coulee dams where large quantities of water are handled annually as there is positive evidence that the baffles and teeth of conventional design are subject to deterioration and eventual destruction.

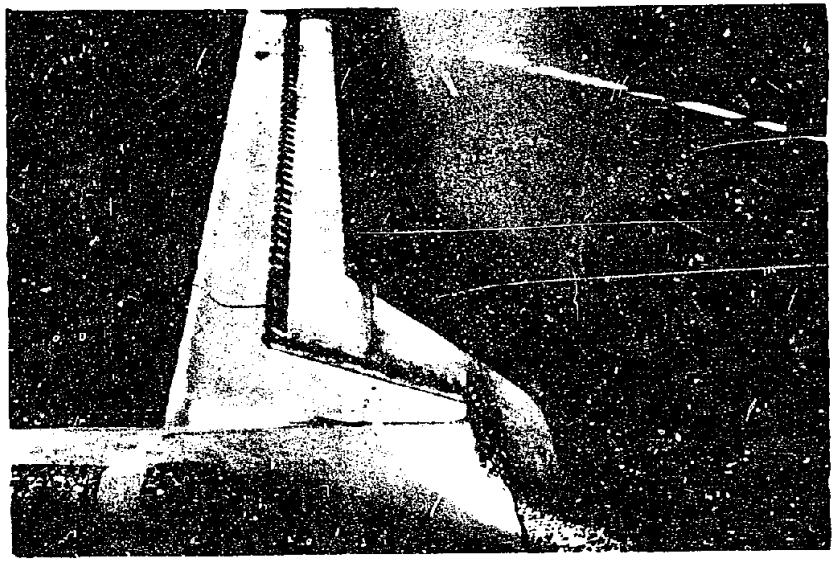
19. Flat open-chute spillways (trapezoidal in cross section). Spillways with trapezoidal chutes and stilling basins are the exception. This type of design is discouraged as seldom is the operation entirely satisfactory. The spillway shown on figure 24C, is trapezoidal in cross section from the gate section through the stilling basin. The original design was unsatisfactory. Model experiments showed that the provision of a small stilling basin immediately downstream from the gate section improved the flow in the chute. After a considerable amount of experimentation a satisfactory design, trapezoidal in section, was developed. The hydraulic jump in a trapezoidal section is not nearly as effective as a comparable one in a rectangular section as the jump in the former is not operative over the entire width of the pool. Due to a difference in elevation between the tailwater and water surface at the beginning of the jump, a continuous flow in an upstream direction persists along the side walls of a trapezoidal pool causing two large eddies equal and opposite in direction to form which disrupt the stilling-pool action. The flatter the side slopes, the more severe will be the eddies produced. It is true that a few recent designs such as Anderson Ranch Dam spillway and Big Sandy Dam No. 2 spillway, figure 20C, have been designed with trapezoidal sections and have performed satisfactorily in the model. The side slopes, however, were rather steep and the effects produced by the eddies did not prove detrimental. Where large trapezoidal chutes or stilling pools are involved it is advisable to investigate the design by means of a hydraulic model.

20. Diversion dams. In general, diversion dams established on rigid foundations, such as the Roza diversion dam shown on figure 26A, are not difficult problems. When the dam is long however, and constructed



A - Roza Diversion Dam and Fish Ladder

B - Model of Original Roosevelt Power Diversion Dam Showing Erosion Upstream from Dam



C - Model of Roosevelt Power Diversion Dam Illustrating Redesign

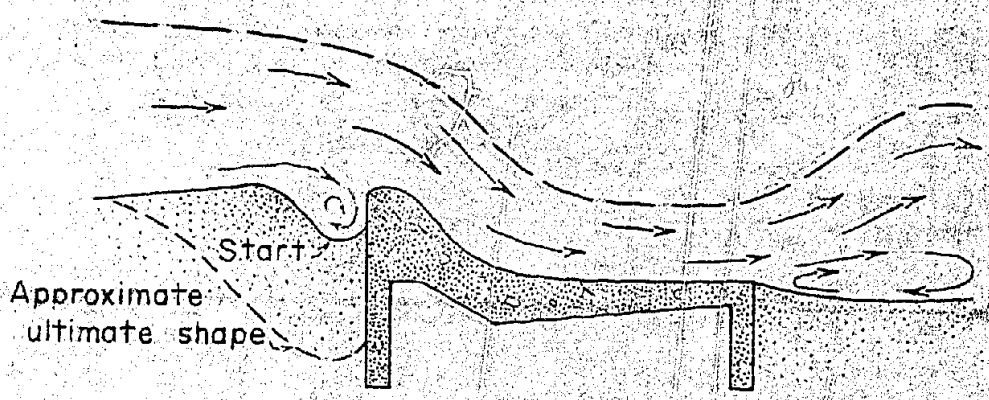
DIVERSION DAMS

FIGURE 26

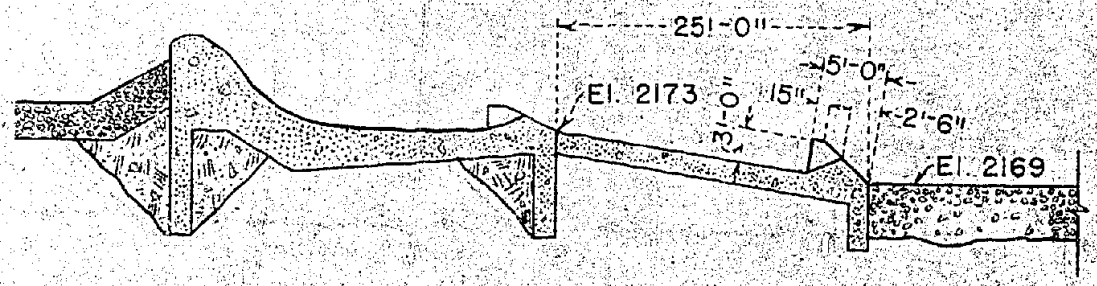
directly on loose river-bed material, and, as the Roosevelt Power Diversion Dam shown on figures 26B and 26C, a satisfactory solution may become difficult even with the aid of model studies. A section of the latter diversion dam failed in 1916, due to piping under the cut-off walls. The Salt river on which the dam is located carries a heavy bed load during large flows, and due to the shallow channel, the velocity of approach to the dam is high. Eddies forming immediately upstream from the dam crest formed holes as shown in figure 26B, by lifting bed material from the heel of the dam and carrying it downstream. At flood flows the large holes continually shifted back and forth along the upstream face of the dam, at which time a considerable amount of piping must have occurred during the larger flows. Eventually a weakened section of the dam was moved downstream where it came to rest tilted in an upstream direction.

After demonstrating the cause of the failure, the model was utilized to determine a remedy. In the redesign, an extension was placed on the downstream apron with an additional cut-off wall, and the river bed was heavily riprapped both upstream and downstream from the dam. Figure 27 shows sections of the original and altered designs of the Roosevelt power diversion dam. Within a few days after completion of the reconstructed dam in 1937, a flood equal to or greater than the one which had caused the failure occurred. Subsequent examination of the river bed adjacent to the dam showed no detrimental effects.

General design data on flow characteristics, discharge and pressures relative to submerged dams are available in HYD 182. This data was compiled from a series of experimental laboratory tests performed specifically to untangle prevailing misconceptions relative to submerged flow. In addition to being characteristically unstable, submerged flow is also tricky in its behavior. The discharge coefficient varies not only with change in submergence but also with other factors



A. ORIGINAL DESIGN



B. ALTERED DESIGN

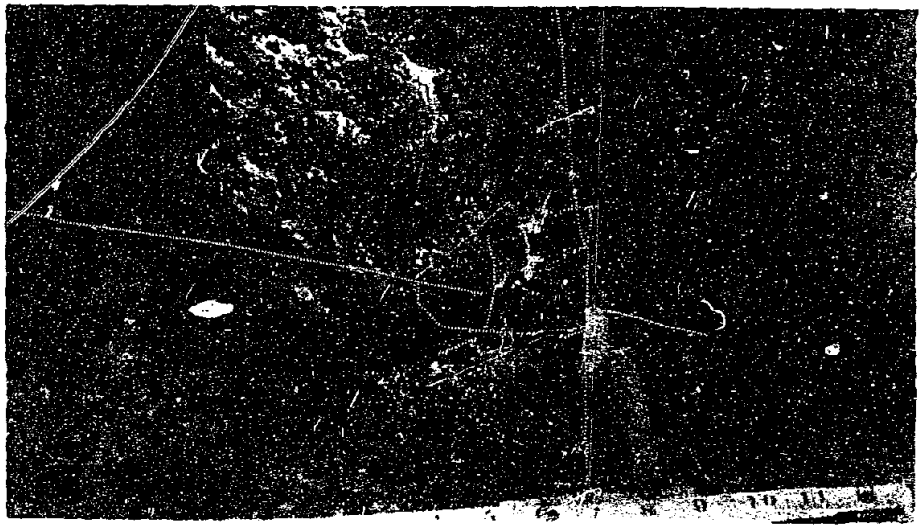
ROOSEVELT POWER DIVERSION DAM

such as the approach depth to the dam and the position of the stilling-basin apron with respect to the crest of the dam.

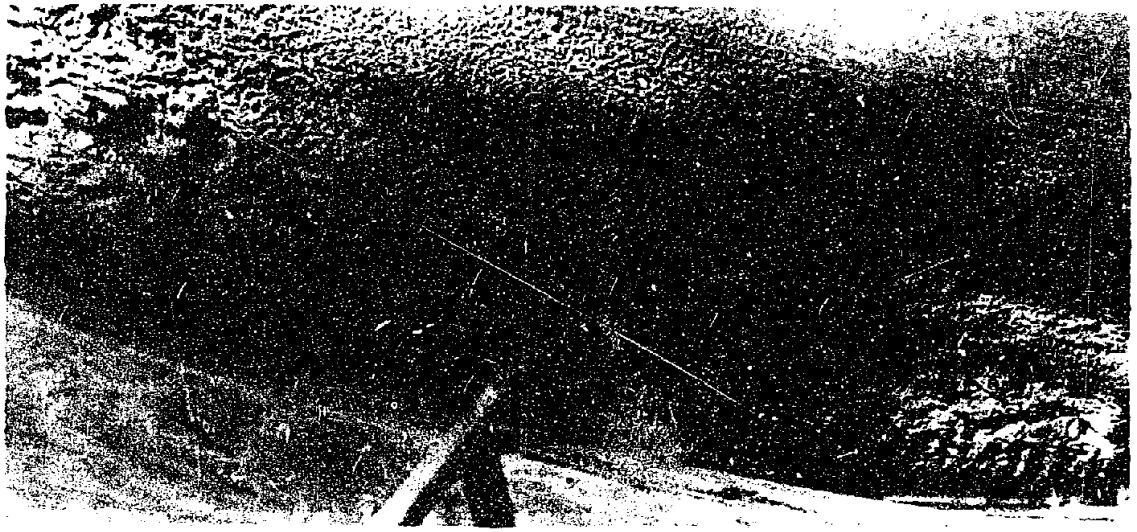
21. Dam outlet works in general. An important factor in dam design is the problem of providing for release of water for irrigation and other purposes downstream when the reservoir level is not high enough to operate the spillways, and when flow through the turbines, if provided, is not adequate. During the past few years, dam outlet works have provided the hydraulic laboratory with some of its most perplexing albeit interesting problems. As the velocities are high in the outlet works they are subject to destruction by cavitation erosion. Cavitation is a phenomenon that accompanies low pressures. Low pressures can be produced in a closed conduit, in which water is flowing at high velocities, by an obstruction, a gate slot, a gate partially opened, an improper shape of conduit entrance, a conduit improperly designed with respect to slope or shape, an expanding section, or a valve in which the areas are not properly proportioned. With water flowing at high velocities such as occurs in outlet works, usually from 50 to 125 feet per second, any of the above causes may result in a flow pattern that can produce negative pressures on these surfaces. When the negative pressures approach vapor tension, which corresponds to a negative pressure of approximately 33.2 feet of water at sea level for a temperature of 60° Fahrenheit, cavitation is imminent. It is evidenced by a sharp crackling noise known as crepitation with accompanying vibration which, if allowed to persist for any length of time, will result in pitting of the surface of the conduit and eventual destruction. Illustrations showing the results of cavitation erosion are shown for steel, concrete, and bronze on figure 28.

In the first case, figure 28A, the pitting has penetrated through a 5/8 inch steel penstock plate which was in service during ninety days of actual operation. This is only representative of what occurred in a long section of 72-inch diameter pipe which required replacement. Pitting of concrete, shown on figure 28B, usually occurs at a more rapid

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



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



C - Pitting of Cast Bronze on the needle of a needle valve

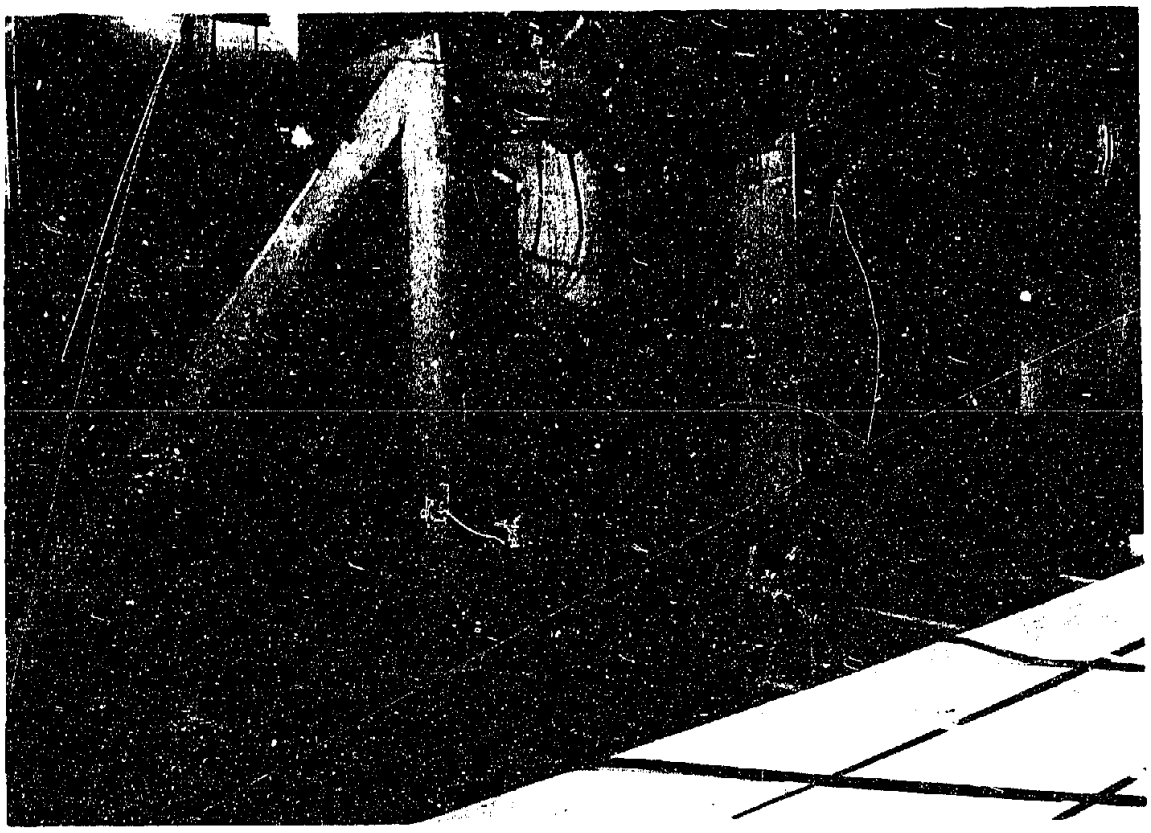
ILLUSTRATIONS OF CAVITATION PRODUCED BY LOW SUBATMOSPHERIC PRESSURES

FIGURE 28

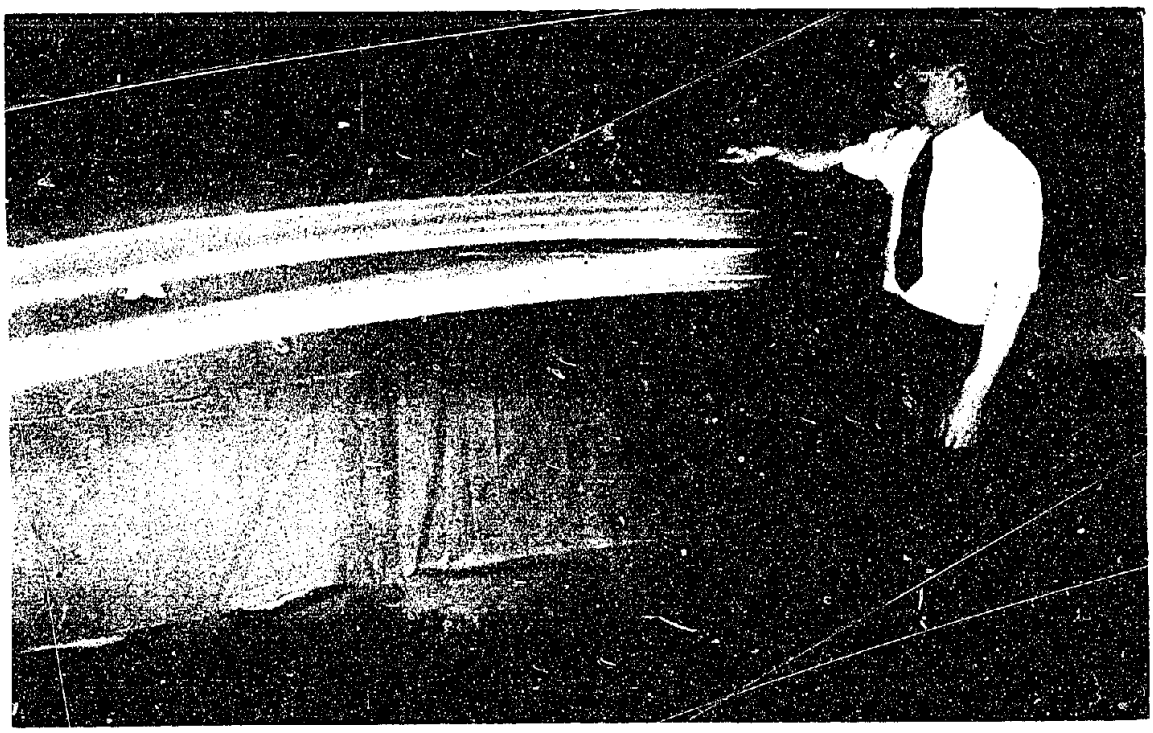
rate than for metals under similar conditions. In this case low pressures were created by flow conditions due to the presence of large gate slots. The rougher the surface becomes, the more rapid the rate of pitting. Attention is also called to pitting of the structural-steel member in figure 28B. Pitting of the bronze needle valve shown in figure 28C is not far advanced but continued use of the valve will eventually destroy the seating surfaces. This pitting was produced by an improperly designed valve. The only method that is effective in stopping this action is an alteration of the shape of the valve. It is desirable to remedy the causes of the cavitation, otherwise repairs become a continuous and costly routine. Repairs are expensive not only from the standpoint of maintenance costs but in many cases they interfere with proper operation of the power plant and dam equipment. The saving which can be credited to improved designs of gates, valves, and outlet works on the Bureau's projects in general, is sufficient alone to finance the operation of the hydraulic laboratory for some time to come. Additional information on cavitation experiences of the Bureau of Reclamation and other agencies can be found from reference (36) of the bibliography.

Some 68 separate models of gates, valves, outlet conduits, and outlet stilling basins which have been investigated in the hydraulic laboratory are listed in table 1, with an index indicating the extent of the model studies and the literature available.

22. Gate-controlled outlets. The outlet works at Grand Coulee Dam consist of three tiers of 20 circular conduits 8.5 feet in diameter, each controlled by a high-pressure slide gate of the ring-follower type. A complete model of one intermediate outlet on a 1:17 scale is shown on figure 29A and a diagramatic section of the dam through the outlets is shown in figure 30A. It is intended that the gates be operated either entirely open or completely closed. The gate slots on this type of gate are filled by the follower ring when the gate is fully open, thus a constant cross section will exist throughout the gate and conduit eliminating

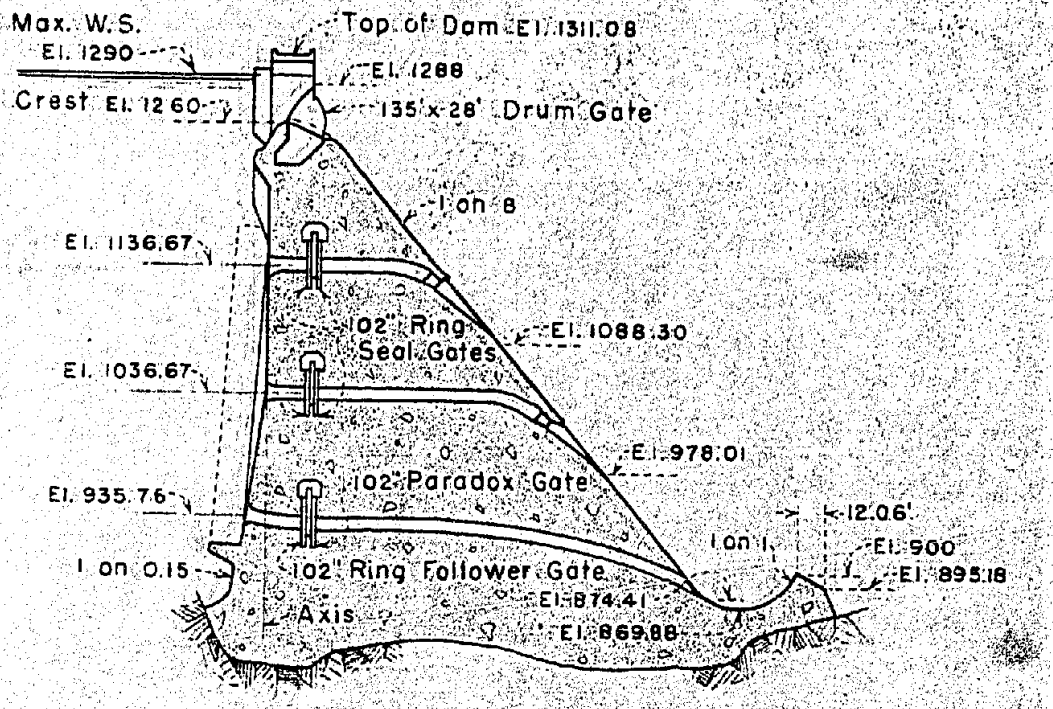


A - Six-inch model of one outlet, including ring follower gates for Grand Coulee Dam

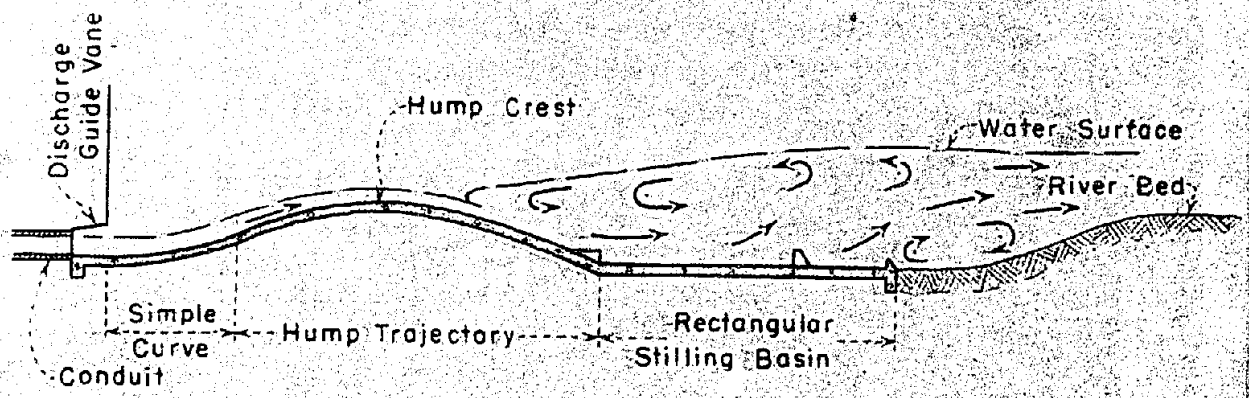


B - Six-inch hollow jet valve operating at approximately 50 percent of full opening.

MODELS OF CONTROL GATES AND VALVES FOR DAM OUTLET WORKS



A. CROSS SECTION OF ONE INTERMEDIATE OUTLET CONDUIT AT GRAND COULEE DAM



B. CROSS SECTION OF STILLING BASIN WITH HUMP - BULL LAKE DAM

STILLING BASINS FOR OUTLET WORKS

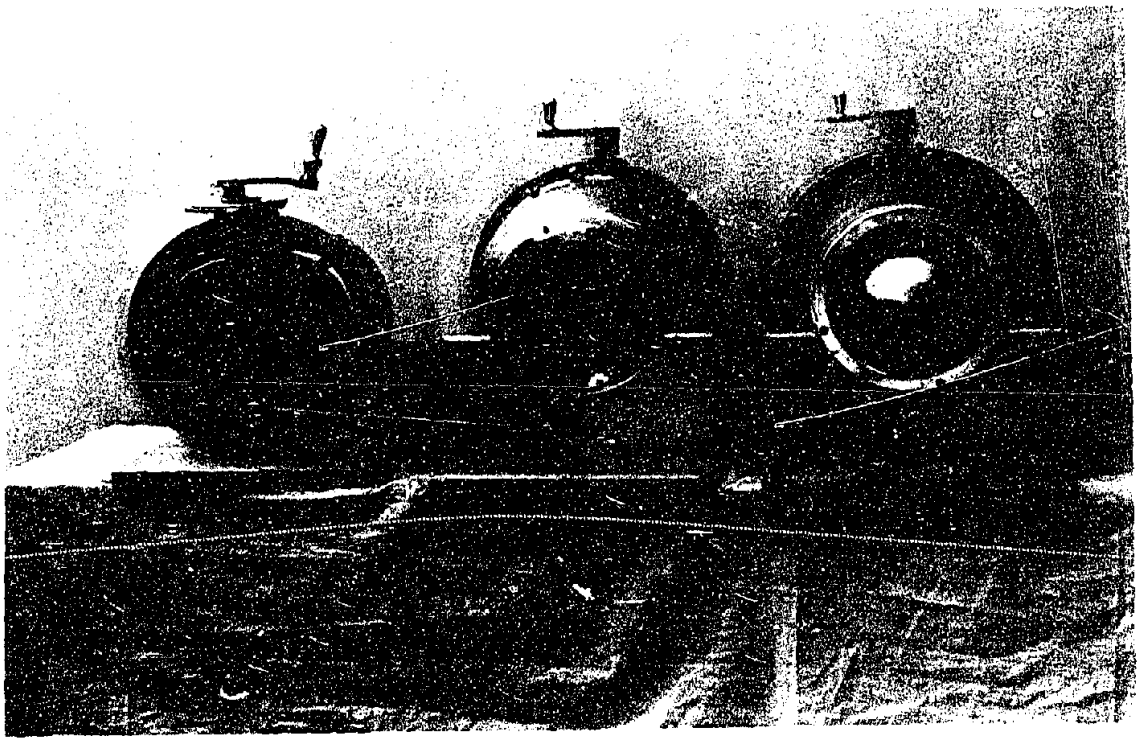
possibilities of cavitation from this source.

The entrances to the conduits are of circular bellmouth design which was developed experimentally in the laboratory. This information is applicable to all unobstructed circular bellmouth entrances, and is available in Hydraulic Laboratory Report HYD-66.

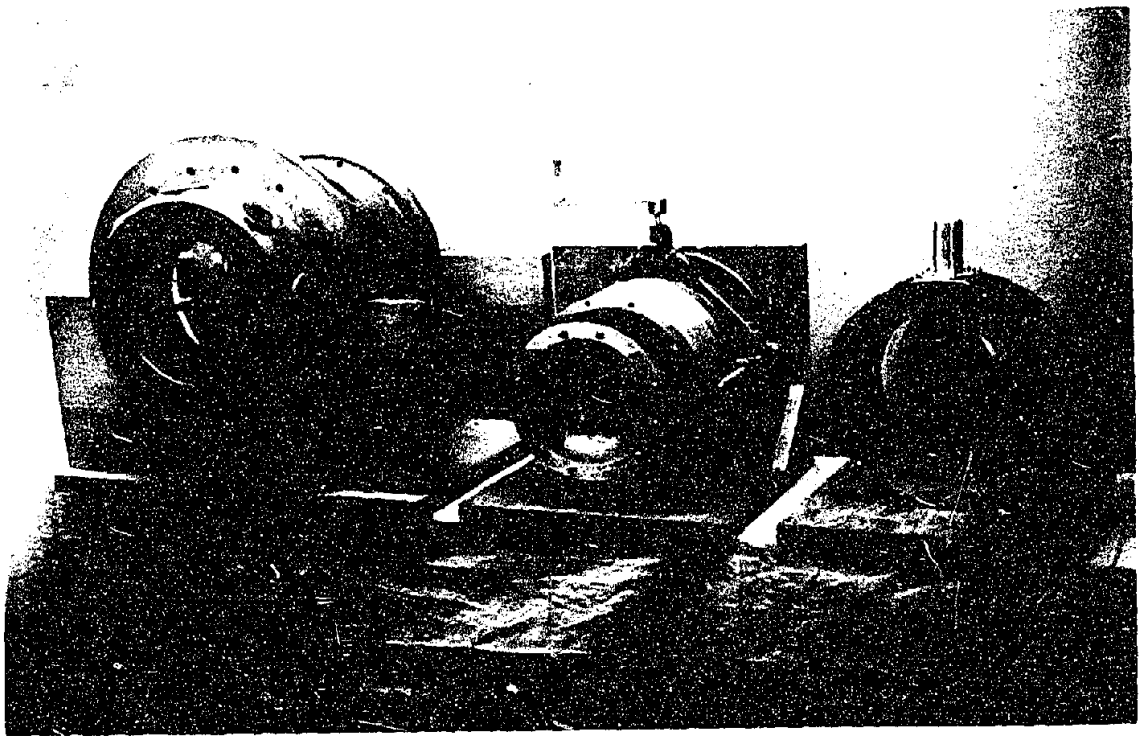
Flow from the outlets is deflected down the face of the dam by elbows at the downstream end of the conduits and the energy dissipation is accomplished in the spillway roller bucket. A more complete description of the model studies on the Grand Coulee outlets can be found from references (4) and (5).

23. Valve-controlled outlets. An outlet design which is common on both masonry and earth dams is to place a needle or similar type of valve on the end of an outlet conduit and allow it to discharge freely into a stilling basin or the river downstream. A photograph of a 6-inch model of a hollow-jet valve discharging from the end of a line is shown on figure 29B. Photographs of valve models which have been tested in the laboratory are shown on figure 31. From left to right in photograph A, as viewed from the discharge end, they are the hollow-jet, tube, and needle valves. The hollow-jet valve is the most recent design, has a higher coefficient of discharge than the other two (table 1) and is the least expensive of the three to construct. All three valves will operate without negative pressures when placed on the end of a line where free discharge can prevail. Valve cross sections, coefficient of discharge curves, and pressure curves are shown for the hollow-jet, tube, and needle valves on figures 32, 33, and 34, respectively. The information thereon was obtained from the 6-inch models shown on figure 31A.

The valves on figure 31B, from left to right, are the Ensign, Howell-Bunger, and butterfly valves. The Ensign valve is found on older low-head installations and is no longer utilized on new work. It is installed on the upstream face of the dam and discharges into a circular conduit leading through the dam to the downstream face. In practically

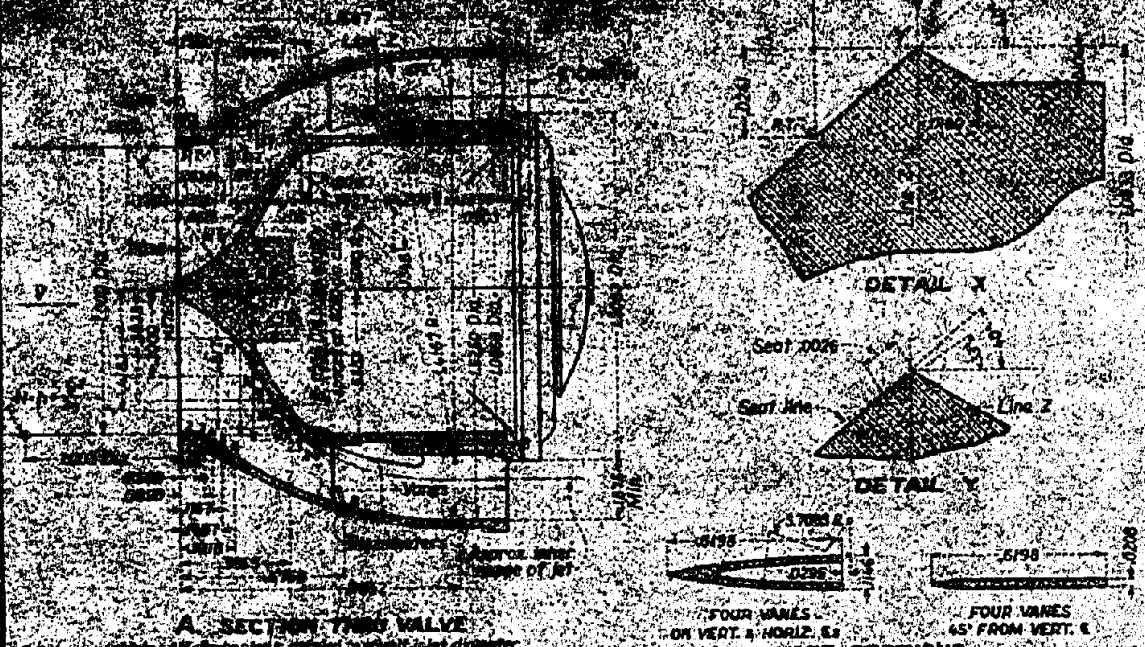


A - Six-inch models of hollow jet, tube, and needle valve



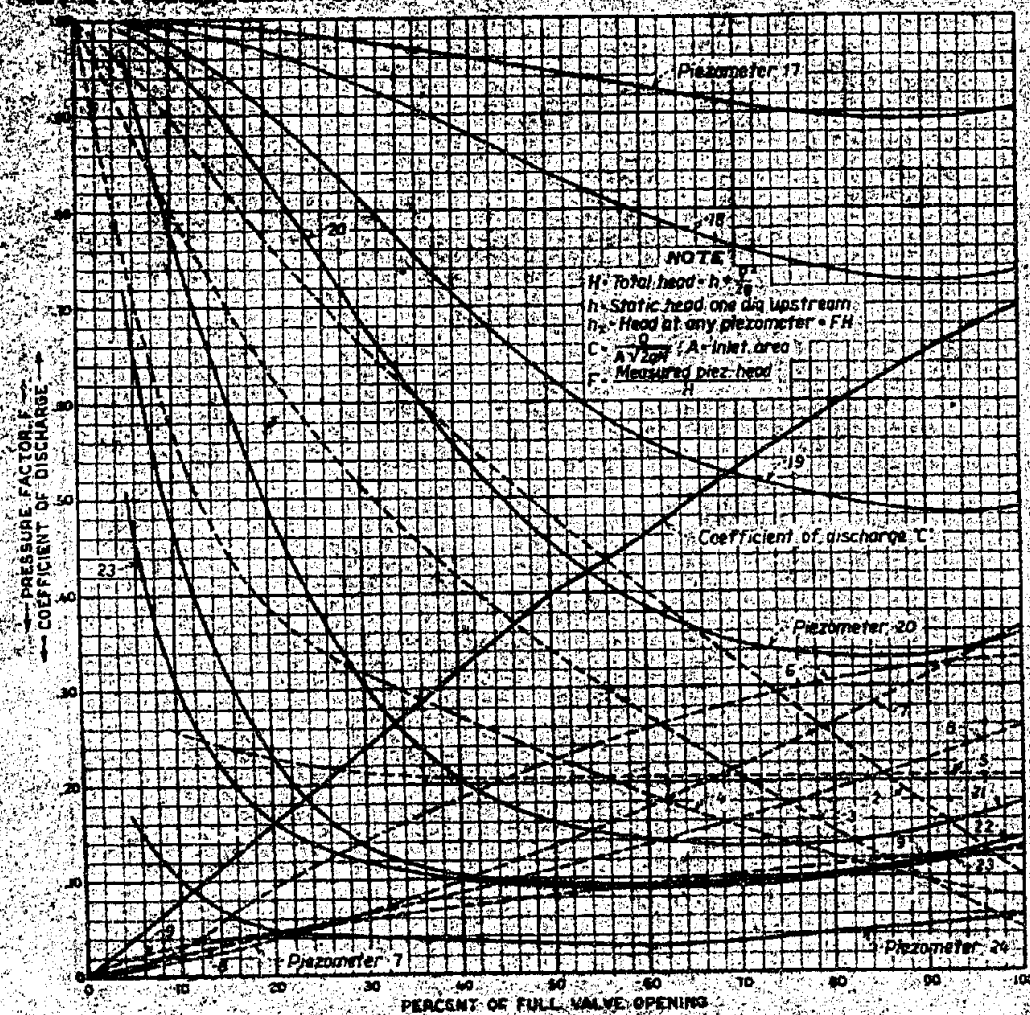
B - Six-inch models of Ensign, Howell-Bunger, and Butterfly Valves

MODELS OF CONTROL VALVES FOR DAM OUTLET



A. SECTION THRU VALVE

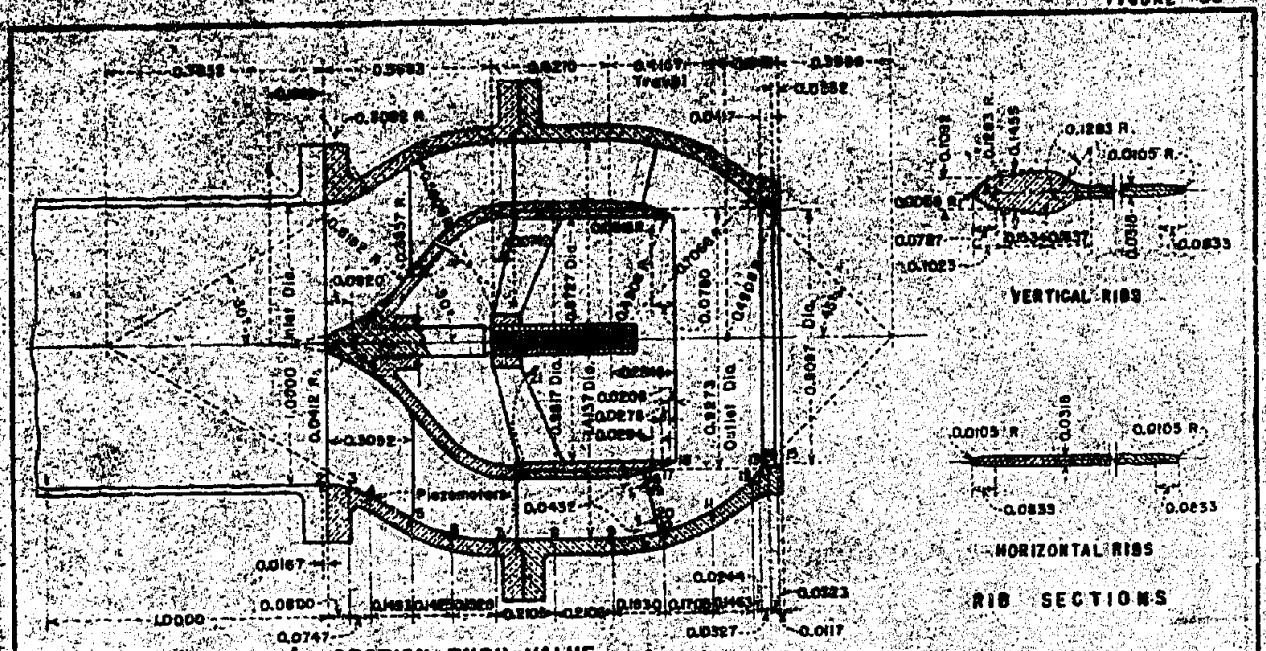
(Note: All dimensions related to half inlet diameter. Maximum unbalanced thrust in position 0.125 sq ft effective area & sq ft of balancing chamber. W-62A)



B. COEFFICIENT AND PRESSURE FACTOR CURVES
(DASHED CURVES FOR HEAD ON BODY - SOLID CURVES FOR NEEDLE)

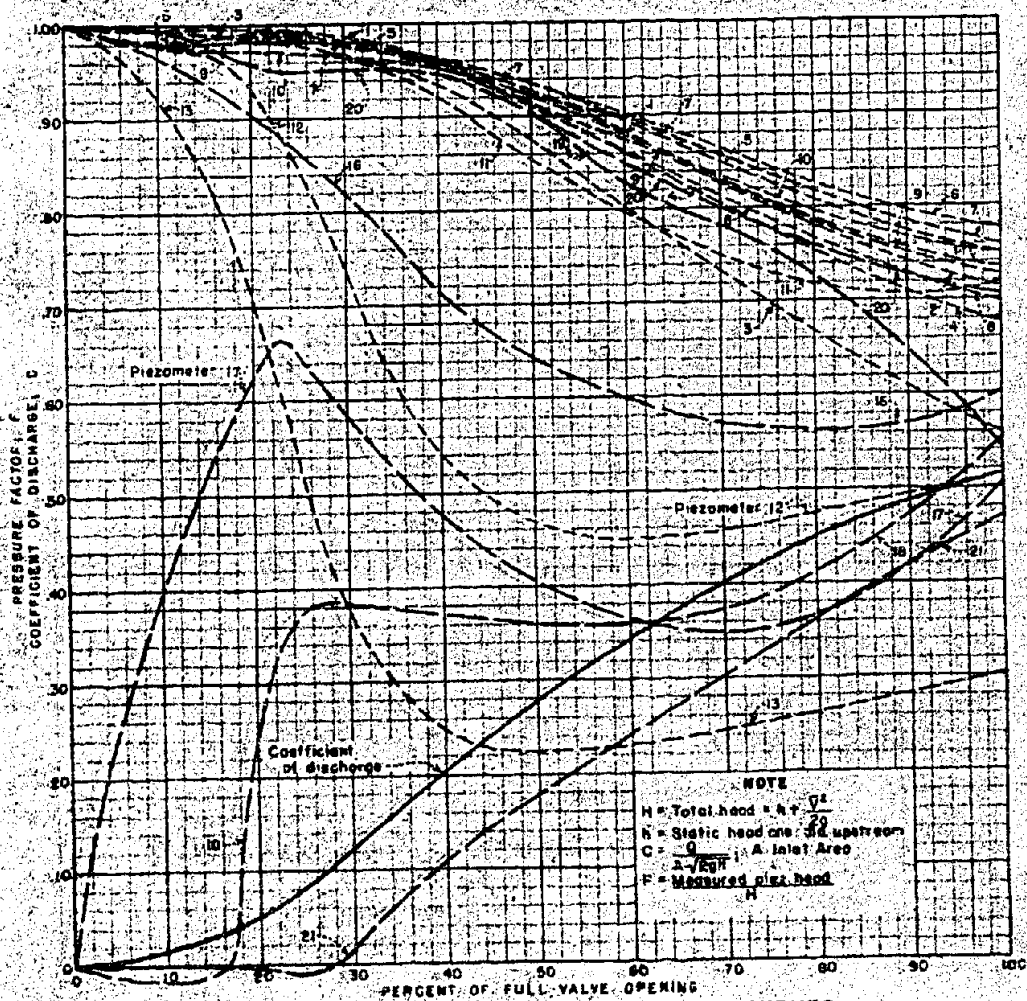
HOLLOW-JET VALVE for ANDERSON RANCH DAM
PRESSURES AND CHARACTERISTICS DETERMINED FROM 6-INCH HYDRAULIC MODEL
FINAL DESIGN

REV. 7-20-66
1-19-1965



A. SECTION THRU VALVE

Note - All dimensions related to a unit inlet diameter

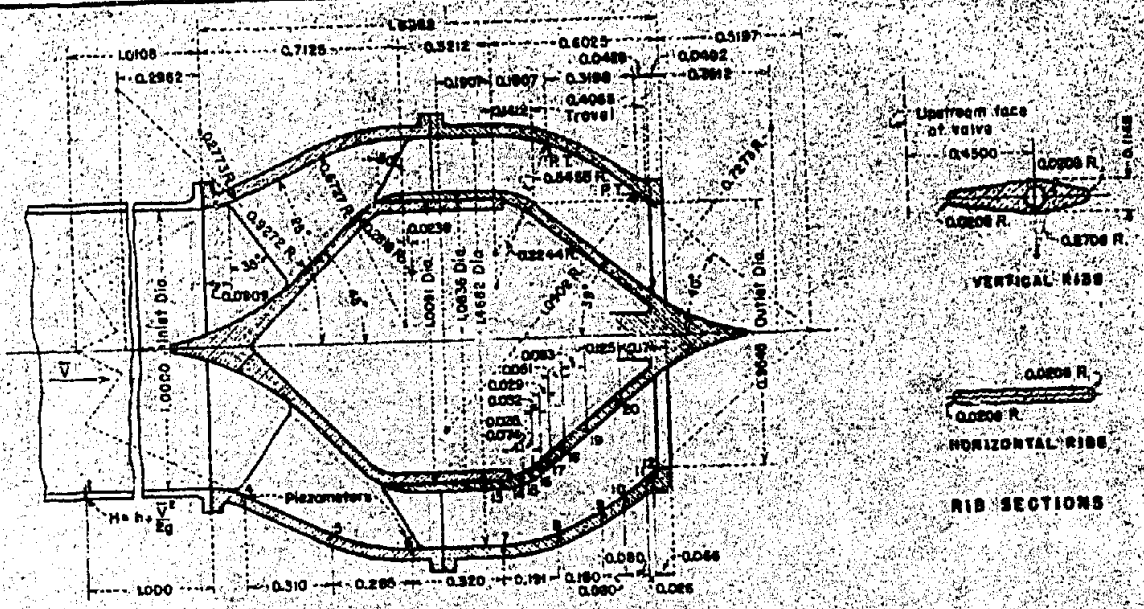


B. COEFFICIENT AND PRESSURE FACTOR CURVES

FRIANT DAM TUBE VALVE

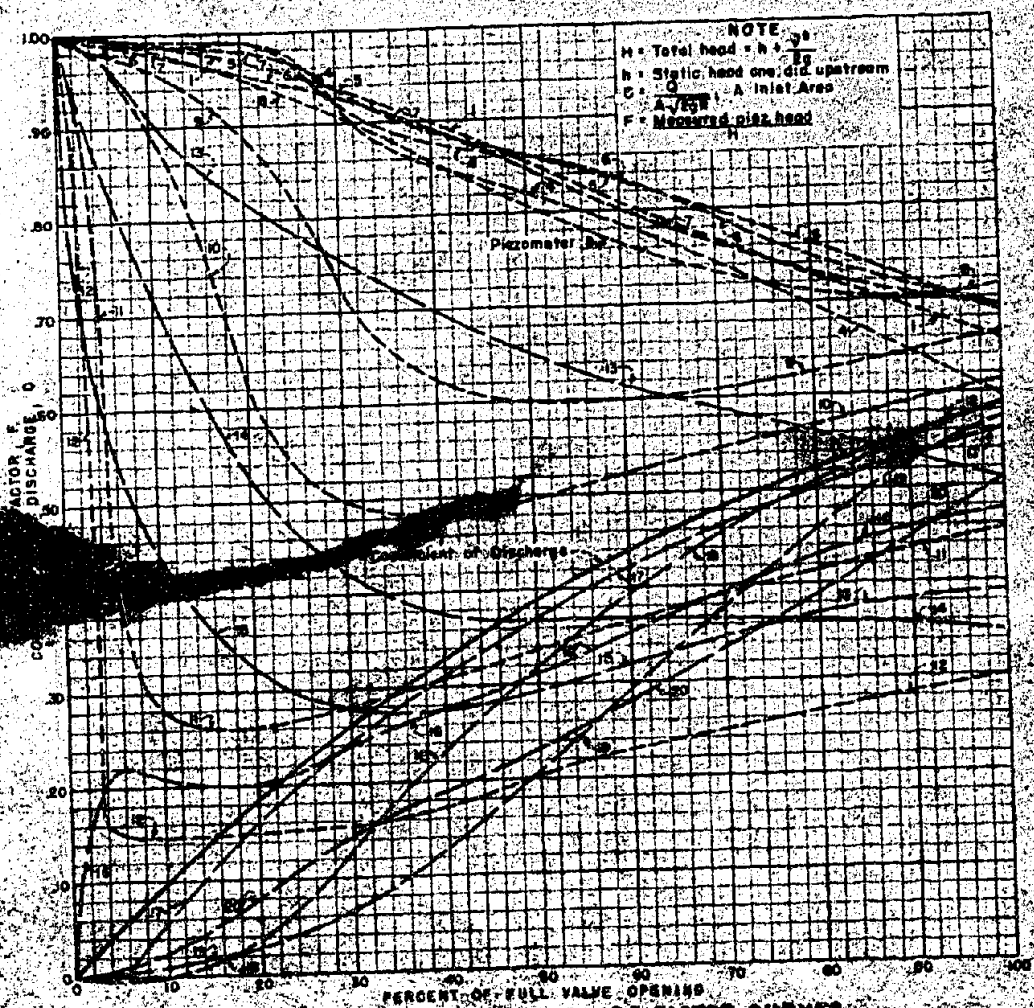
PRESSURES AND CHARACTERISTICS DETERMINED FROM 6-INCH HYDRAULIC MODEL
FINAL DESIGN

6-23-1948



A. SECTION THRU VALVE

Note - All dimensions related to a unit inlet diameter



B. COEFFICIENT AND PRESSURE FACTOR CURVES

FRIANT DAM NEEDLE VALVE

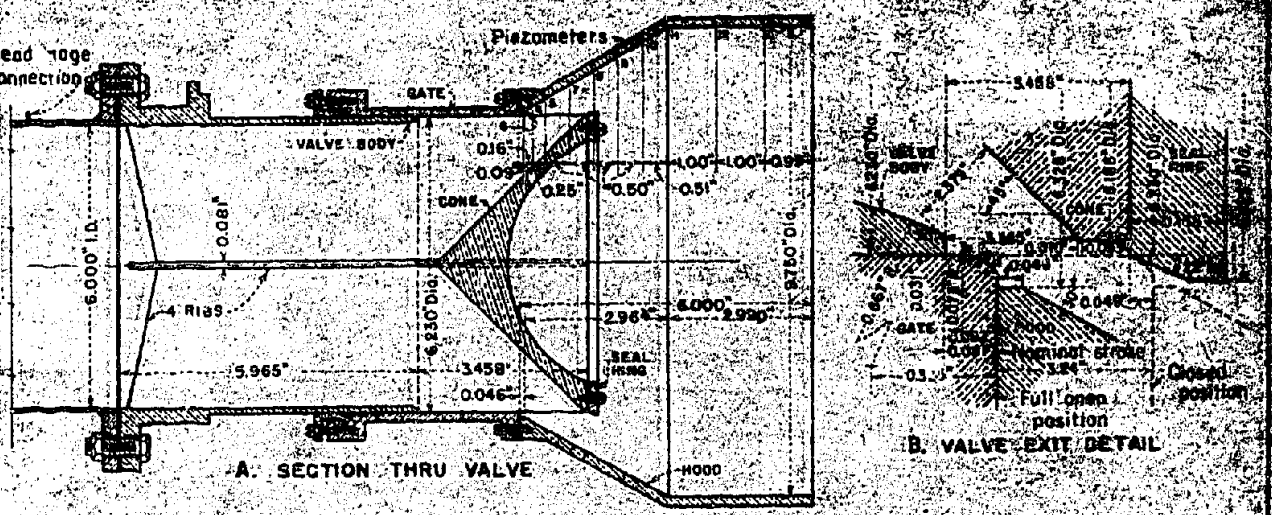
PRESSURES AND CHARACTERISTICS DETERMINED FROM 6-INCH HYDRAULIC MODEL
FINAL DESIGN

all existing installations, difficulties have been experienced with this valve due to low pressures and cavitation erosion. The model was constructed to aid in studying the alteration and the repair of existing design valves. The second and third valves shown on figure 31B are intended for installation at the end of outlet conduits. The Howell-Bunger valve possesses a high coefficient of discharge, table 1, but emits a widely dispersed spray at an angle of 45 degrees with the center line. If a hood is installed on the valve to reduce the spray it results in cavitation, vibration, and unbalancing of the valve. It will operate without negative pressures when used as shown in figure 31B and is the least expensive to build of the six valves. A valve cross section, coefficient of discharge curve, and pressure curves are shown for the Howell-Bunger valve operating with and without a hood on figure 35.

Butterfly valves will be used on the end of the outlet pipes at Ross Dam to operate under a head of 270 feet. The coefficient of discharge for the butterfly valve is 0.60 when operating completely open (table 1). The valves will operate without low negative pressures at this head but is balanced only in the fully open position; thus a fairly heavy operating mechanism will be required.

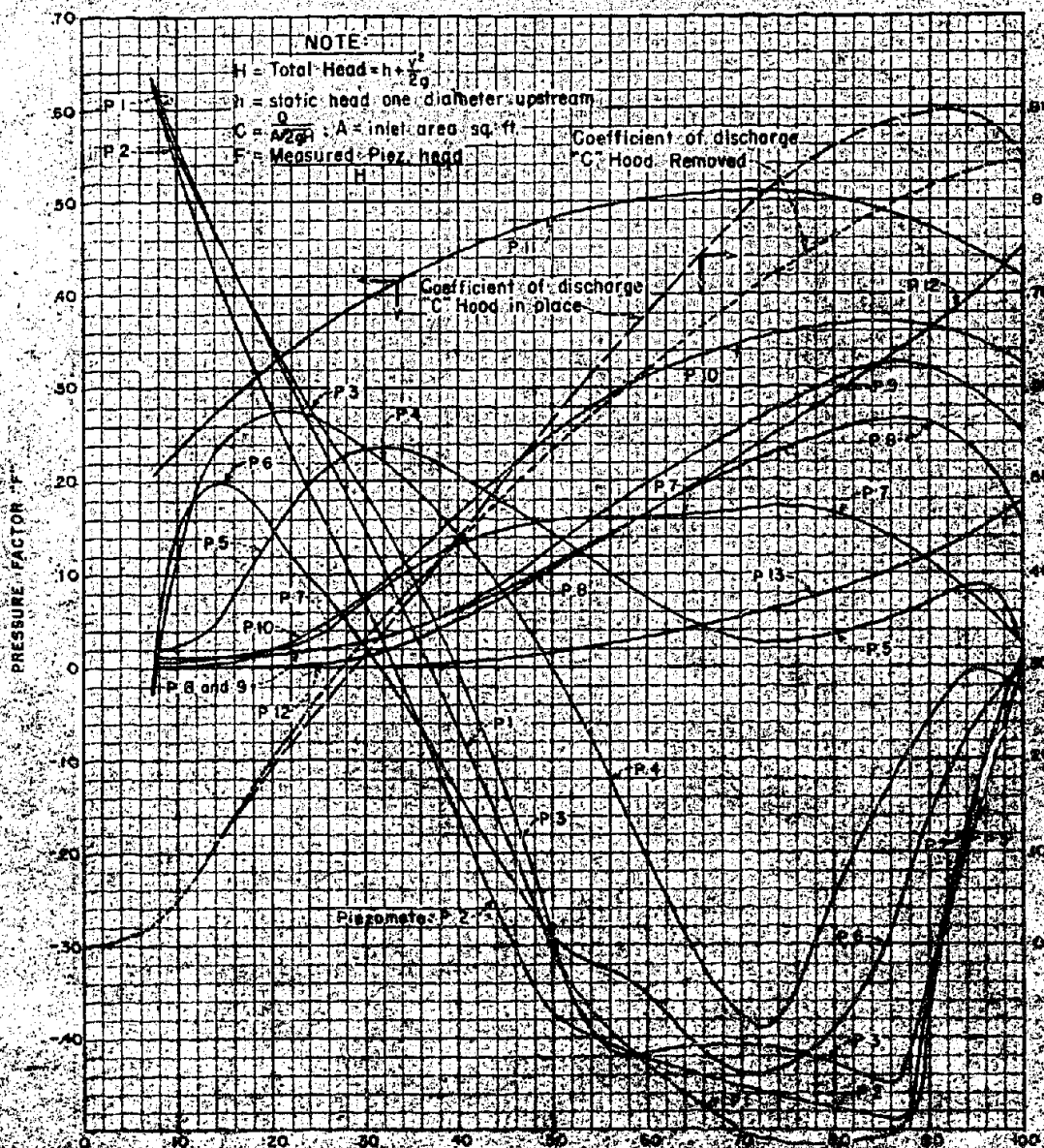
When gates or valves are installed in an outlet conduit, other than at the end and operated at partial openings and high velocities, cavitation is invariably the result unless aeration or ventilation is provided in the proper manner. References concerning the design of these valves can be found in table 1.

24. <u>Stilling basins for outlet works.</u> Stilling basins are required for most earth-dam and some masonry-dam outlets to aid in dissipating the energy in the flow before the water can enter the river downstream. These are very similar to the spillway stilling basins employed on earth dams, which have been discussed in Section 15. They are usually rectangular in cross section with horizontal aprons and utilize combinations of



A. SECTION THRU VALVE

B. VALVE EXIT DETAIL



C. COEFFICIENT AND PRESSURE FACTOR CURVES

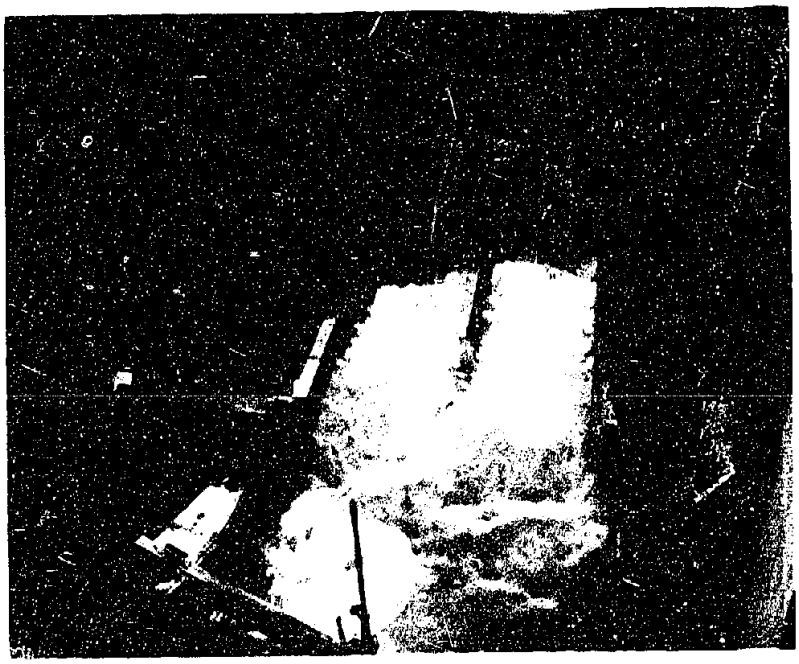
HOWELL-BUNGER VALVE STUDIES

DETAILS AND CHARACTERISTICS DETERMINED FROM 6-INCH HYDRAULIC MODEL

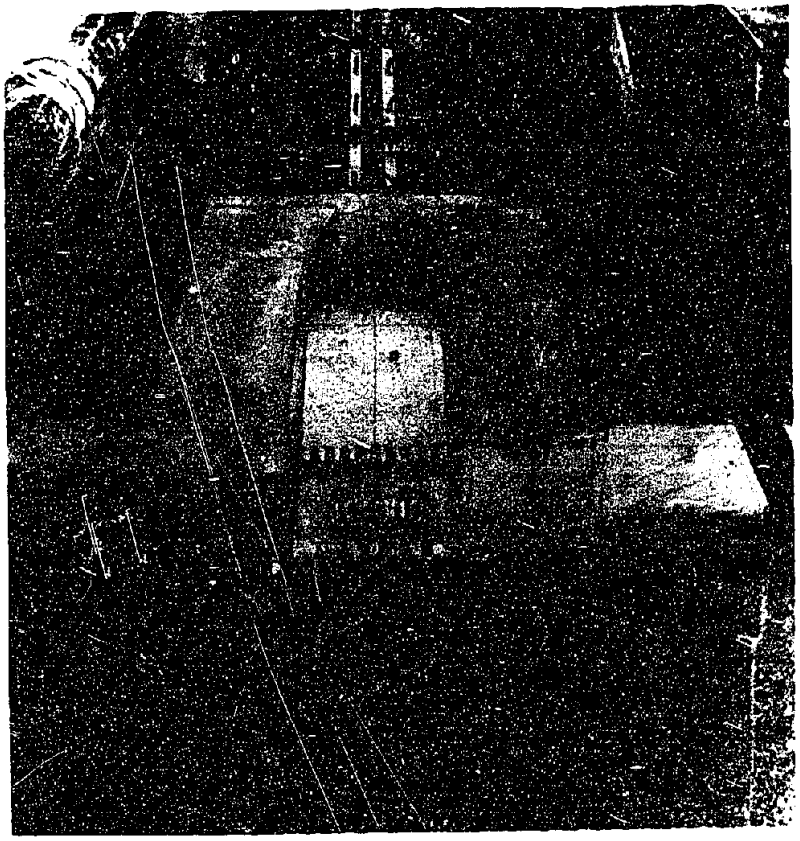
REVISED DESIGN

spreader teeth, baffle piers, and sills. Where more than one valve discharges into the pool, longitudinal dividing walls are provided so that unsymmetrical operation is permissible. The outlet stilling basin for the Friant Dam, river outlets, is shown on figure 36A. The valves are located above the maximum tailwater elevation. The structure consists of a parabolic floor on which the jets from the valves impinge lightly to spread the jets uniformly before entering the hydraulic jump. The stilling pool proper consists of vertical walls and a horizontal floor with a step at the downstream end. The dividing wall in the basin makes possible operation of one, two, or three valves without the production of objectionable whirlpools in the basin. With the spread of the jets from the valves accomplished such that the flow entering the jump approximates uniform depth, the stilling basin can be designed in the same manner as previously described for spillway stilling basins with horizontal floors.

In cases where tunnel outlets enter the river submerged or partially submerged, a stilling pool such as that shown for the Bull Lake outlet works, figures 30B and 36B, can be utilized. When this submerged condition exists, the hydraulic jump has a tendency to move in or out of the tunnel with a change in discharge. Present practice is to discourage the formation of the hydraulic jump in a tunnel. This condition was eliminated in the Bull Lake design by incorporating a hump in the stilling basin as shown in figure 30B. The hump serves to spread the jets emerging from the outlet tunnels before they enter the stilling basin proper. The hump consists of a simple reverse curve on the upstream side and a parabola designed to fit the shape of the trajectory for maximum discharge on the downstream side. The crest of the hump is ordinarily set considerably below the maximum tailwater elevation but sufficiently high to keep the tailwater from submerging the hump for any appreciable discharge from the outlet tunnels. The stilling basin proper is designed similar to those previously described with horizontal floor, spreader teeth, and



A - Valves and stilling basin for river outlet works at Friant Dam



B - Stilling basin and hump for outlet works at Bull Lake Dam

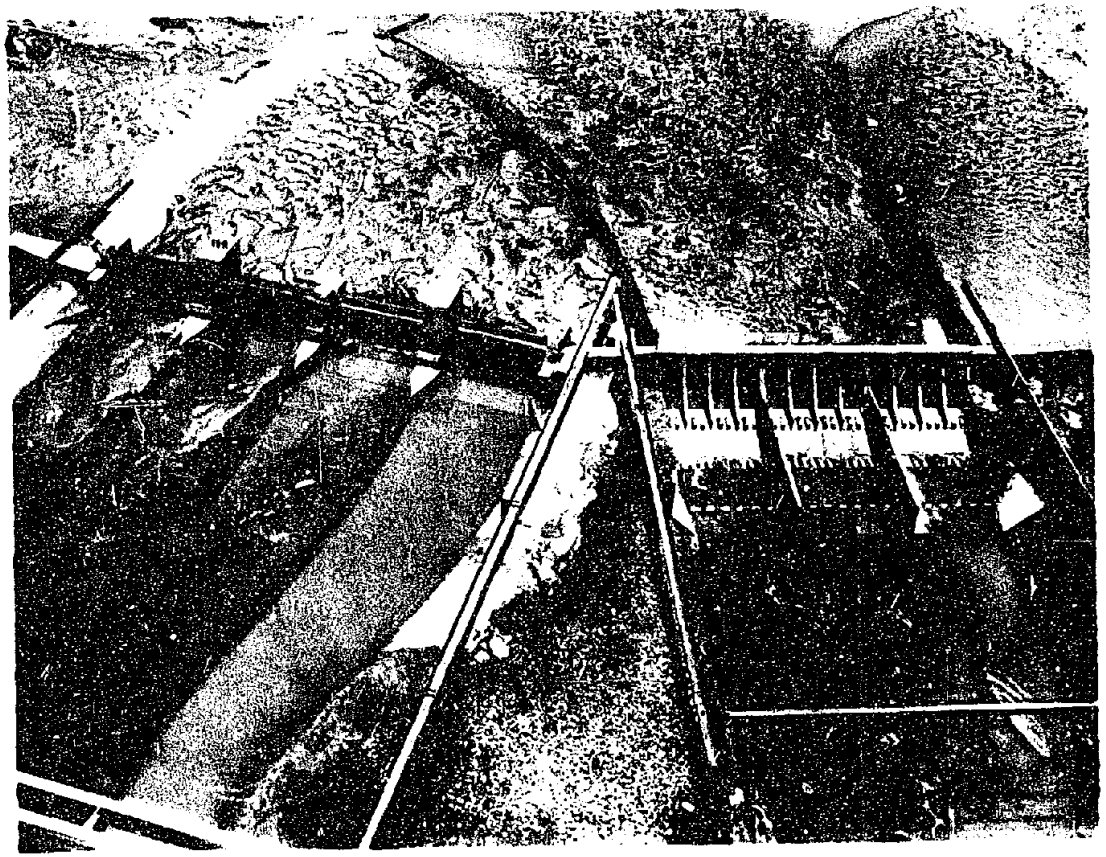
STILLING BASINS FOR OUTLET WORKS

baffle piers. Unsymmetrical operation of the gates or valves is usually permissible in this type of pool, without the provision of division walls, as spreading of the jets by the hump is very effective. Development of this type of outlet stilling basin is described on page 17 reference (32) and in TM-556 and HYD-37.

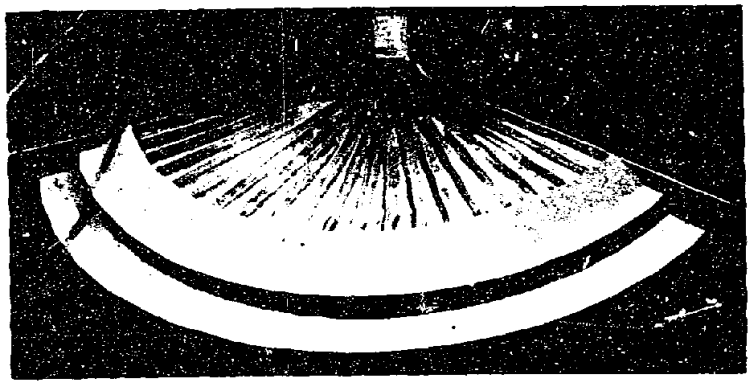
25. Sluiceways and canal headworks. Sluiceways and canal headworks are related in that the function of the sluiceway is to remove silt and bed material which otherwise would find its way into the canal headworks and thus interfere with proper operation of the canal. It can be observed from table 1, that the laboratory has investigated a number of models of canal headworks but only three sluiceways. The reason for this is that the majority of canal headworks in this country do not require desilting devices.

Figure 37A, shows a model of the All-American canal headworks on the left and the Imperial Dam sluiceway on the right. This structure is located on the Colorado River which carries a heavy load of fine silt. The purpose of the sluiceway is to systematically flush down the river as much of the fine bed deposit along the trashrack as possible. This aids in keeping a channel open to the headworks. Without a sluiceway, continual dredging would be required. The photograph shows only a small portion of the model which consisted of the entire Imperial Dam constructed on a 1:40 scale which occupied an area of several acres. Similarity as to movement of bed material in model and prototype is not well understood. A satisfactory analytical solution to a problem of this nature is a near impossibility. Experience combined with a model study, in spite of imperfect quantitative results, is the best guide in design. This type of model shows trends and tendencies in the movement of silt. It is an excellent aid in qualitative comparison of designs. In other words, the model can be depended upon to accurately depict the advantages or disadvantages of one design compared with another.

Figure 37B shows a model of the intake to the 13-mile tunnel which



A - All-American Canal Headworks at left and sluiceway on right, located at Imperial Dam on Colorado River



B - Headworks to Alva B. Adams Tunnel on Colorado-Big Thompson Project, Colorado

C - Headworks to Madera Canal at Friant Dam operating under 130 feet of head.

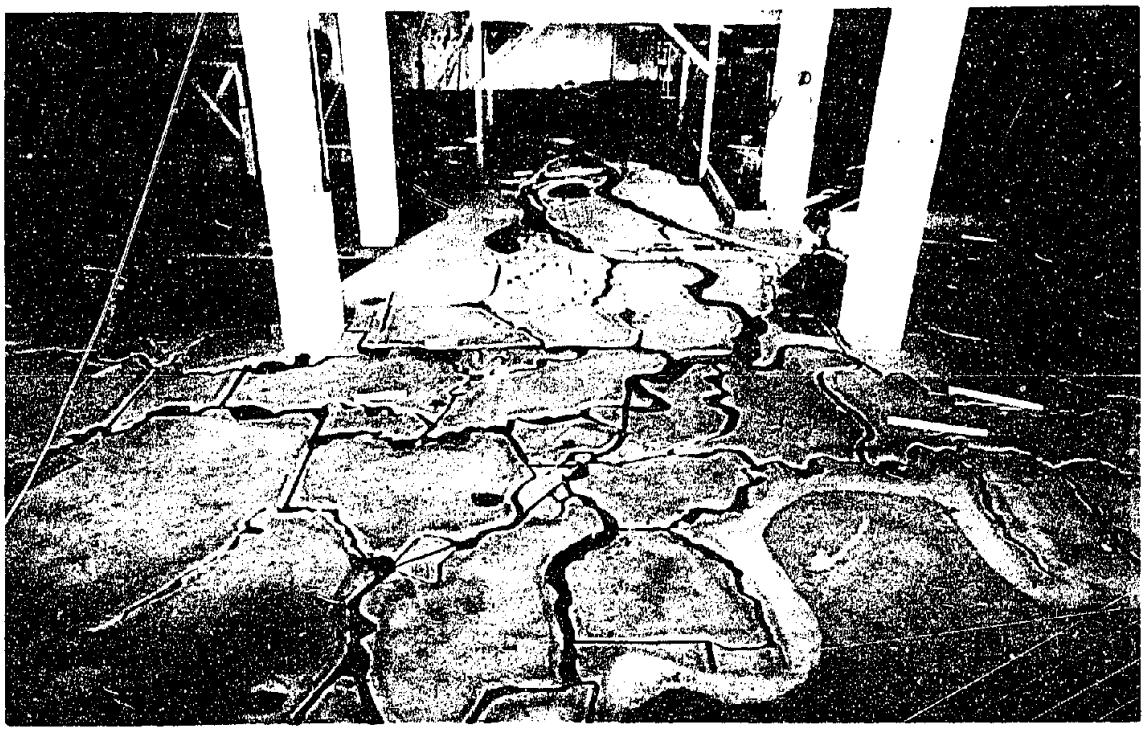
CANAL HEADWORKS

FIGURE 37

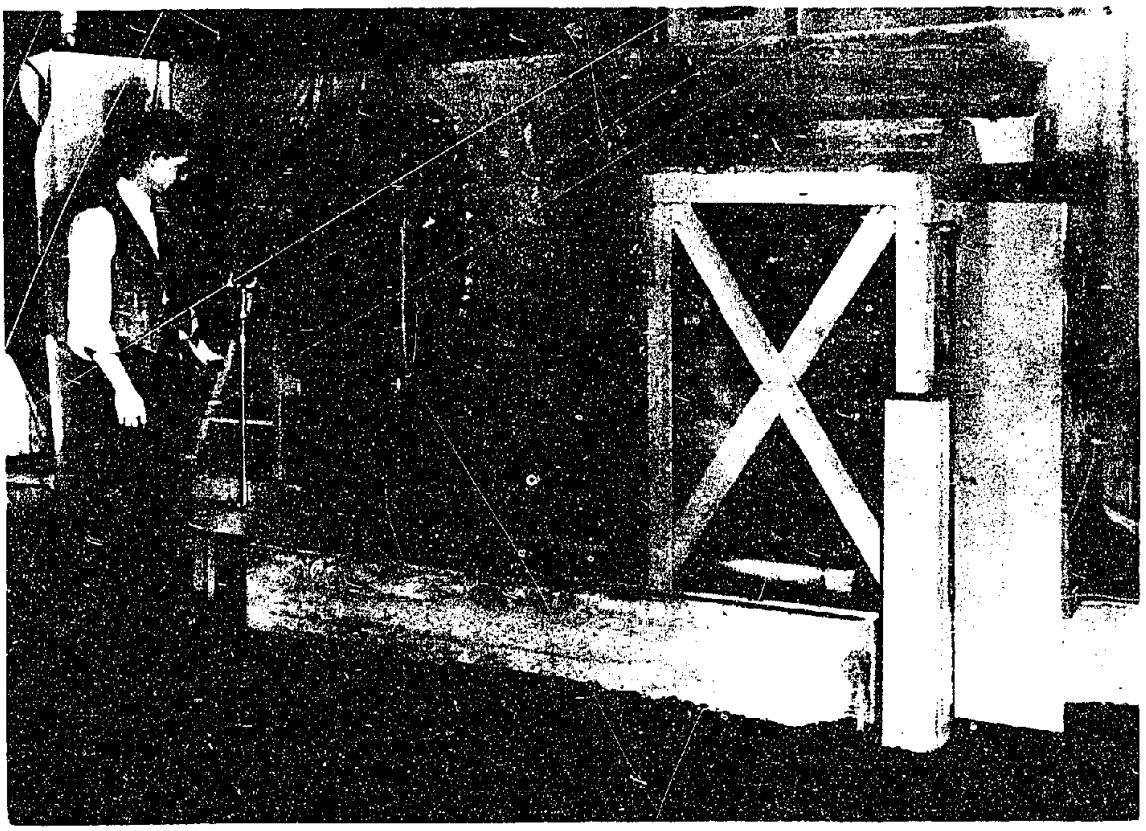
will convey water from Grand Lake, under the Continental Divide, to the Big Thompson River in Colorado. In this case a silt problem does not exist. Instead, reduction in head loss was of prime importance, thus the structure is exceedingly large for the discharge to be handled. The over-all loss for the intake amounts to less than one quarter of a foot, prototype. The numerous walls are required to support the roof as the structure will be underground and covered as a protection from cold weather. Nevertheless, the intake will operate on the free-flow principle.

A model of the Madera canal headworks at Friant Dam is shown on figure 37C. The headworks consists of two 78-inch needle valves, operating under a maximum head of 130 feet, discharging into a stilling basin at the upstream end of the Madera irrigation canal. This is very similar to the Friant Dam river outlets shown on figure 36A.

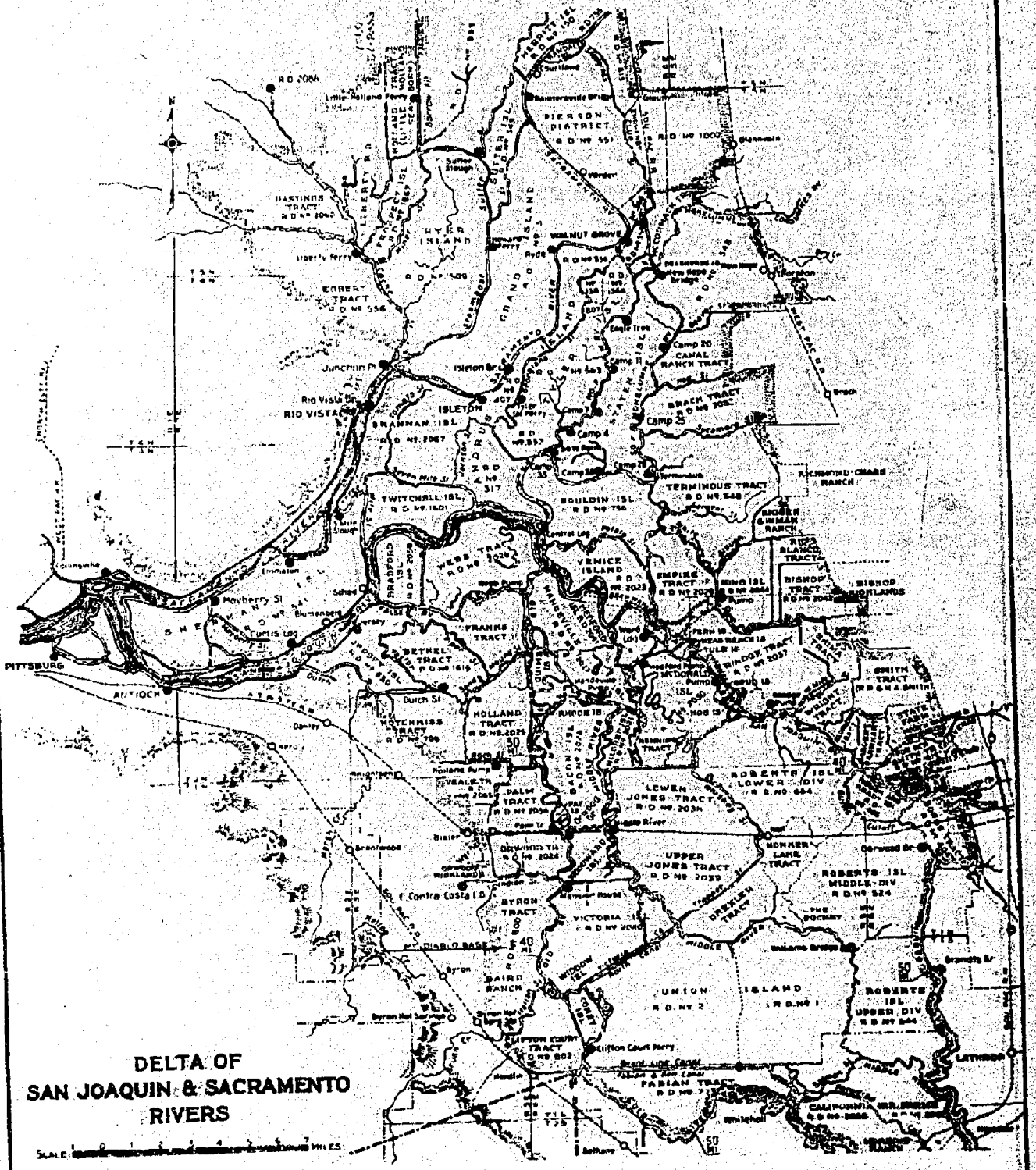
26. River models. Several types of river models have been constructed and tested in the laboratory, table 1, but perhaps the most interesting of these was a model of the Sacramento-San Joaquin River delta in California, constructed on a distorted scale of 1:4800 horizontally and 1:100 vertically. Photographs of the model are shown on figure 38 and a map of the delta region is included as figure 39. The model was constructed to study methods or transferring water from the Sacramento River, across the San Joaquin delta, to supply water to future pumping plants to the south. This transfer of excess water from the Sacramento Valley to the Delta Mendota pumping plant which will serve the San Joaquin Valley, is one of the ultimate objects of the Central Valley plan. The transfer is complicated by the fact that the flow of both rivers drops so low during the summer months of some years that ocean salinity is propogated upstream by the tides sufficiently far to seriously affect agricultural development in the Sacramento-San Joaquin delta. A closed channel was proposed to transfer this Sacramento River water across the delta, but a saving



A - Distorted model of Sacramento and San Joaquin River Deltas Horizontal scale 1:4800 and vertical scale 1:100



B - Tide producing maching for above model showing tide control cam at left and displacement float in center of photograph


DELTA OF
SAN JOAQUIN & SACRAMENTO
RIVERS
SCALE
MILES
PITTSBURG
ANTIOCH
RIO VISTA
ISLETON
WALNUT GROVE
Ryde
Courtland
Hood
Clarksburg
Freeport
Walnut Grove
Thornton
Lathrop
Stockton
Tracy
Byron
Brentwood
Oakley
Bethany
Lodi
Galt
Elk Grove
Sacramento
Rio Vista Br.
Junction Pt.
Isleton Br.
Hoyberry St.
Curtis Log
Dutch St.
Emmaton
Collinsville
Bacon Island
Bacon Isl. Ferry
Clifton Court Ferry
Tracy Blvd.
Mossdale
Lathrop
Stockton
Manteca
Ripon
Escalon
Lodi
Galt
Elk Grove
Sacramento
Walnut Grove
Thornton
Courtland
Hood
Clarksburg
Freeport
Byron
Brentwood
Oakley
Bethany
Tracy
Stockton
Lathrop
Manteca
Ripon
Escalon
Lodi
Galt
Elk Grove
Sacramento
Rio Vista
Isleton
Pittsburg
Antioch
Collinsville
Emmaton
Junction Pt.
Rio Vista Br.
Isleton Br.
Hoyberry St.
Curtis Log
Dutch St.

of approximately $25,000,000 would be possible if natural existing delta channels could be used for this purpose.

By means of Shasta Dam on the Sacramento River, enough storage will be provided to insure a minimum flow sufficient to provide for water use in the San Joaquin valley, for use in the delta area, and for keeping the salinity encroachment below a tolerable value in the delta. As inferred from the problems involved, the model study was extremely complex. The construction of the model was a major task in itself involving careful reproduction of the existing surveys and soundings of the channels. It was necessary to devise methods for obtaining tidal action, river flow and salinity intrusion. As it was essential that these phenomena act the same in the model as in the prototype, adjustments of tidal action and salinity intrusion were made by historical tests from data collected in the field since 1920. This done, it was then possible to explore methods of admitting fresh water from the Sacramento River into the delta.

Of particular interest in this study was the methods used for determining the concentrations of salinity and the proportionate water mixtures at any given point and the tide producing machine. Salt water was represented by coloring the model water with a biological stain "patent Blue" whose concentration could be quickly and accurately determined by a chemical analysis tool, the spectrophotometer. The tide-producing machine, which operated on the displacement principle is shown on figure 38B. A displacement float, shown in the channel, was actuated vertically by an hydraulic cylinder. The latter was controlled by a cam to reproduce tide gage records. The cam, represents the tides for a lunar month.

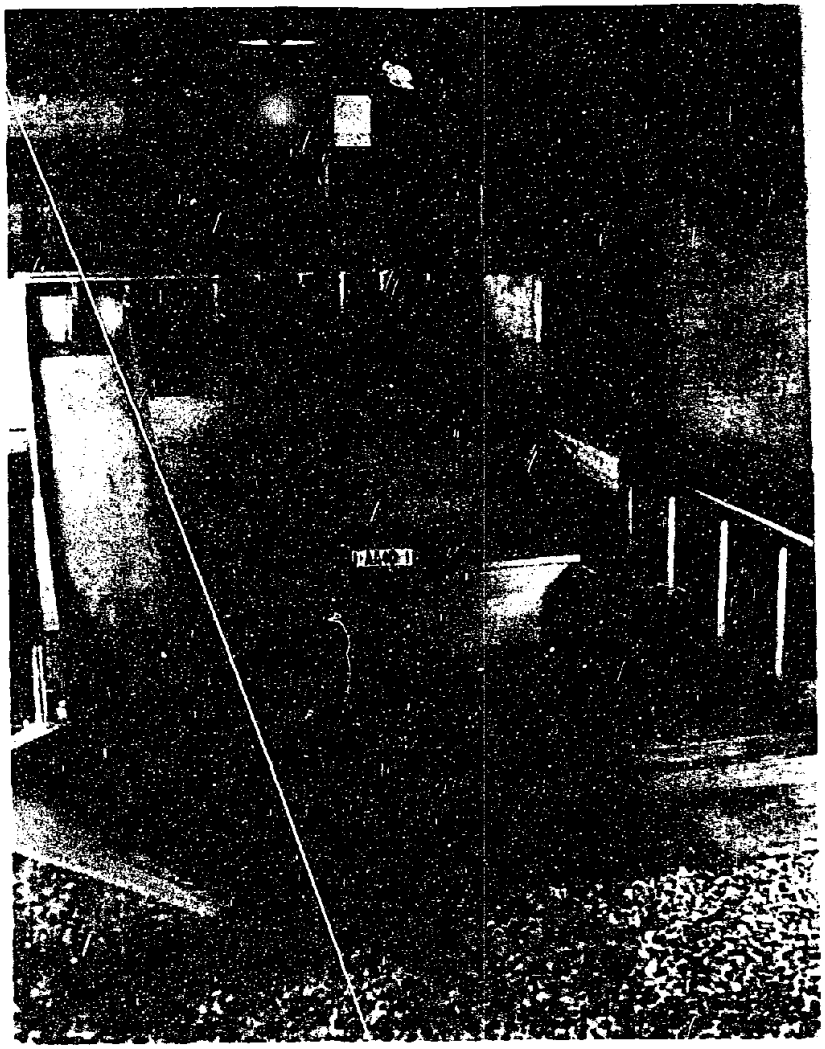
The model demonstrated that the transfer of water from the Sacramento River to the Delta Mendota pumping plant could be successfully accomplished by utilizing existing channels. Further details of the work on this model can be found in hydraulic laboratory reports HYD-142 and 155.

27. Canal structures. In many cases it is more difficult to solve a flow problem on a canal structure than on a spillway for a dam as canals often involve trapezoidal earth sections that require low steady velocities if scouring is to be prevented. Another difficulty frequently encountered on canal structures is that the drop in head between headwater and tailwater is not sufficient to allow a good hydraulic jump to form. As a result it is necessary to use combinations of spreader teeth, baffle piers or sills to dissipate the excess energy. Figure 40A shows a canal chute with trapezoidal stilling basin. The model was constructed to study means of altering existing structures which performed poorly. By means of large baffle piers and sills it was possible to improve the performance of the stilling basin to that shown in figure 40A. Nevertheless, there is a choppiness in the water leaving the stilling basin which is invariably detrimental to the earth sections of the canal downstream. This is characteristic of all trapezoidal stilling basins. It is no longer considered good practice to design canal structures with flat-sided trapezoidal stilling basins.

More recent designs of canal drop structures are shown by the models on figures 40B and 41A. The former is a power structure on the All-American canal with a drop on each side. The stilling basins for the two drops are rectangular in cross section with sloping aprons and sills. The action is much improved over that of the stilling basin of figure 40A. In spite of improved action in the stilling pool, some difficulty has been experienced in holding the riprap in place at the downstream end of the warped concrete walls in the prototype structure. From economic considerations the warps were made short and abrupt. The eddies produced by these short transitions tend to undermine the warped walls.

Figure 41A shows a model of Pilot Knob wasteway on the All-American canal. By means of a hump in the floor of the chute, just upstream from the change in grade, the flow is spread uniformly in the diverging section, thus flow enters the stilling basin at practically a uniform depth. The entire chute and stilling basin are rectangular in cross section and

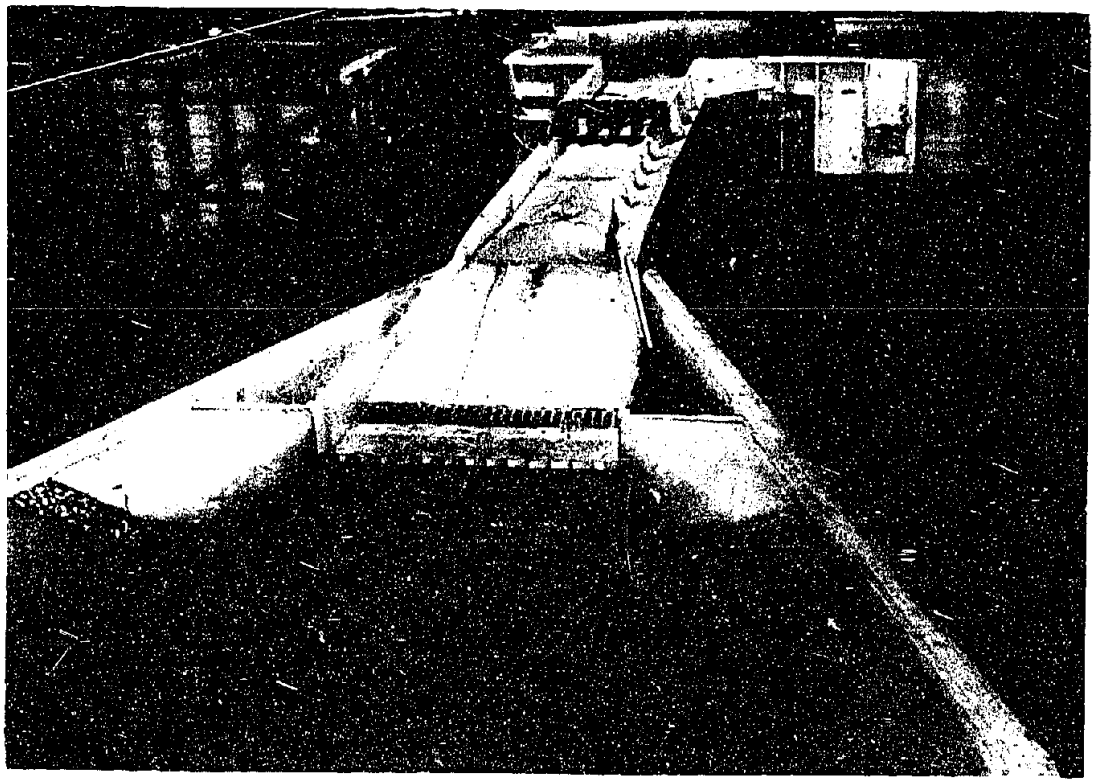
A - Canal chute with trapezoidal stilling basin



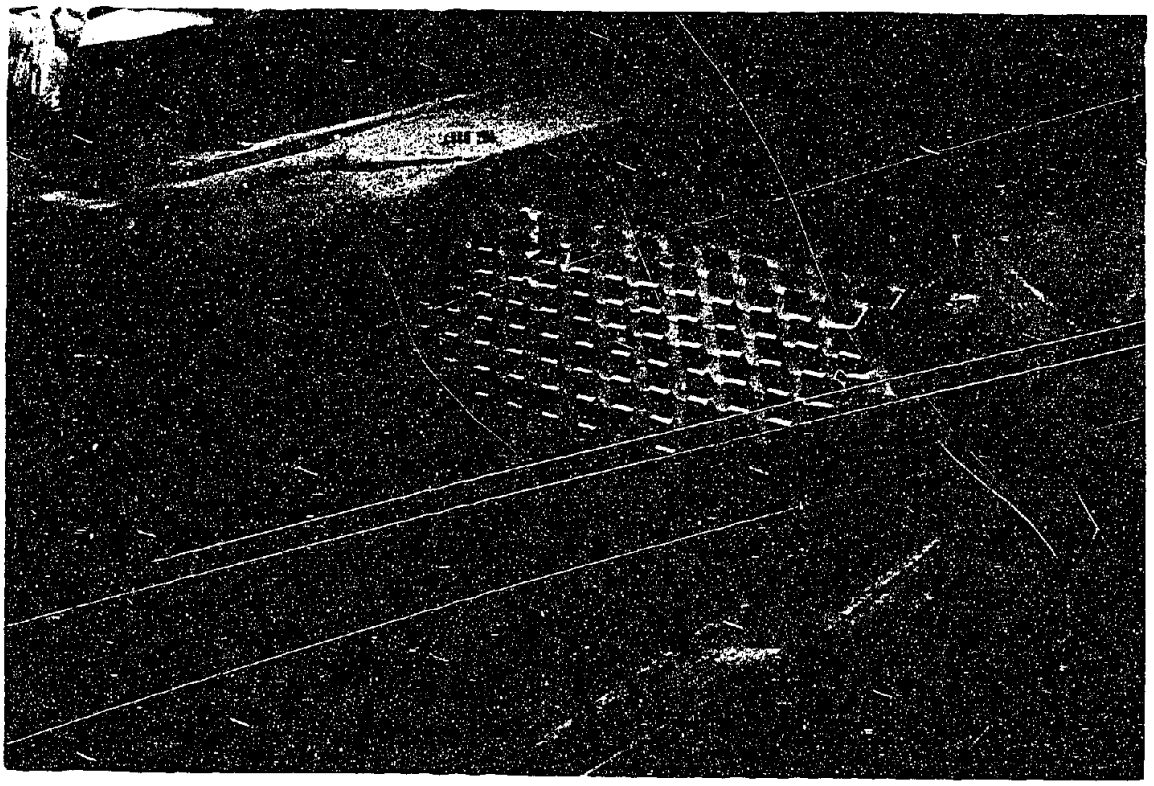
B - Power Drop No. 4 on All-American Canal

FIGURE 40

CANAL STRUCTURES



A - Pilot Knob Wasteway on All-American Canal.



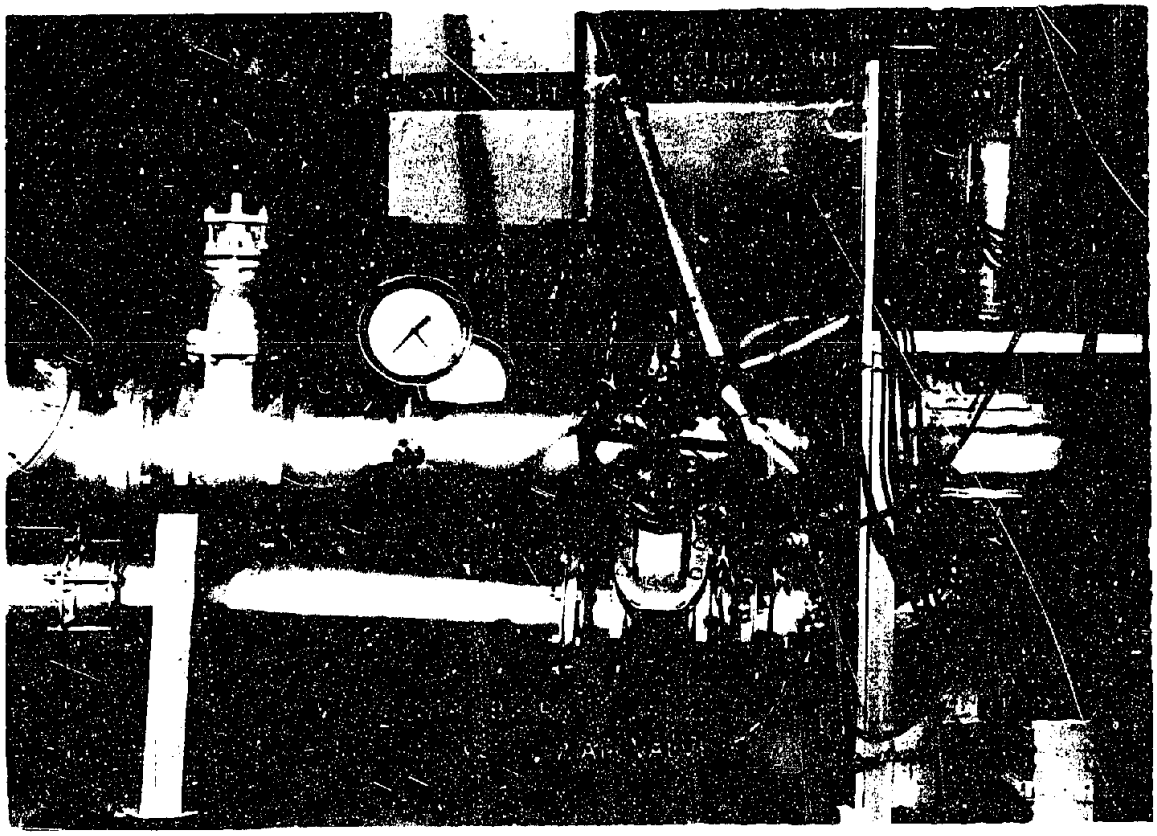
B - Typical Wash Overchute on Coachella Canal

CANAL STRUCTURES

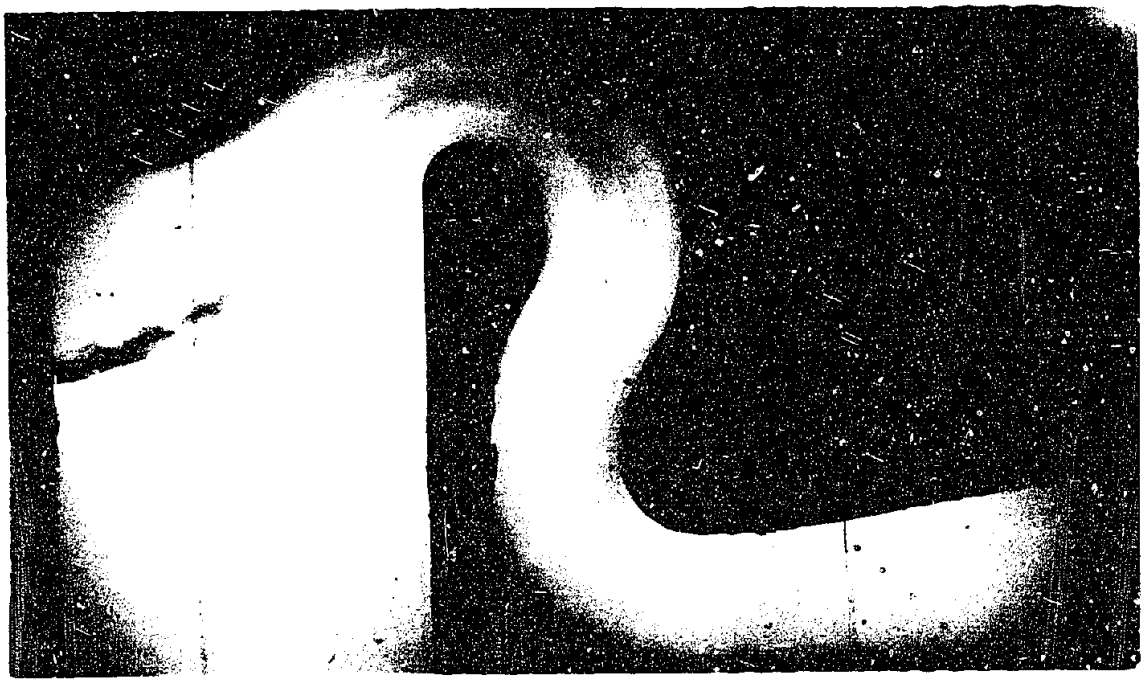
operate in an excellent manner. The paved warp surfaces downstream, connecting the rectangular section to the trapezoidal earth section, are fairly long and little difficulty has been experienced with erosion downstream.

The wash overchute shown in figure 41B is 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 are necessary for one canal. They are essential in the controlled operation of a canal, for without them tremenduous amounts of silt and debris would be deposited in the canal requiring removal and the additional water would overtop the banks of the canal causing washouts. The model shown in figure 41B is typical of a wash overchute founded on desert sand. It usually consists of a siphon on the canal, a slab of concrete paving downstream and appropriate training walls. The inclined portion extends some 20 to 30 feet underground and is studded with baffle piers. The extension underground provides for future retrogression of the channel downstream and the baffle blocks dissipate a maximum of energy before the water is released downstream. The photograph in figure 41B was taken after a run of long duration and the lack of scour demonstrates the effectiveness of this design. The energy dissipator for this type of overchute was developed in the hydraulic laboratory. Further references to model studies on canal structures can be found in table 1.

28. Models of pumps. The Granby pumping plant on the Colorado-Big Thompson project will house four pumps of approximately 6,000 horsepower each, which will be subject to frequent starting and stopping. Starting and stopping pumps of this size under full load would cause dimming of lights and interfere with the operation of automatic equipment throughout the vicinity. A model of one pump on a scale of approximately 1:8, figure 42A, was tested in an effort to alleviate this condition. The model showed that the power required on starting could be reduced to a permissible



A - Model of one of four pumps to be installed at Granby Dam, Colorado



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PUMPS AND SIPHONS

value by forcing compressed air into the suction tube of a pump and then starting the motor with the pump impeller rotating in air. Once started, the air would be released at a steady rate allowing the water to gradually submerge the impeller. This will decrease the power required on initial starting by about 25 percent. The valve on the discharge lines would then be slowly opened allowing the power to increase steadily to full load. The procedure would be exactly the opposite when stopping a pump. It was necessary to reduce power surges to a minimum as long transmission lines as well as long pump lines were involved. The model showed that power surges were directly related to pressure surges and vice versa. Should a surge be started from one source or the other, a self-induced regenerative oscillatory power surge might be established which could only be stopped by complete shutdown. A record of the model pump performance showing pressure and power surges during the starting and stopping operations was recorded by the laboratory oscillographic equipment. A complete record of the tests are reported in hydraulic laboratory reports HYD 113 and 150.

29. Low-head siphons. A fairly comprehensive series of tests was made in an effort to improve the design of low-head siphons. It is generally understood that a quick priming siphon will not necessarily be efficient, and an efficient siphon will not prime readily. The purpose of the tests was to develop a siphon that would prime readily and still possess the best efficiency obtainable by reproportioning the passages. A siphon was developed, figure 42B, which rates well as to both characteristics. The long sweeping curves result in a small increase in construction costs over ordinary types. It is felt however, that too much emphasis has been placed in the past on simple construction at the expense of hydraulic considerations. There are many siphons in existence which will not prime at the heads for which they were designed. When installed in canal structures, where the head is limited, these siphons never go into operation. The results of the model tests are available from reference (19) and Hydraulic Laboratory Reports HYD 108 and 62.

30. Miscellaneous problems. Many other diversified and interesting problems studied by the hydraulic laboratory, such as the supply of condenser cooling water from a river influenced by tides for the Antioch steam plant, and the prevention of ice formation on the Grand Coulee drum gates by bubbling compressed air through the water, are listed with references to available literature on table 1. In spite of the accumulation of data and information the laboratory facilities are taxed heavier as time progresses.

SIMILARITY RELATIONSHIPS BETWEEN MODEL AND PROTOTYPE

31. Dimensional models. Before a model can be properly constructed of a prototype design or model results interpreted in terms of the prototype structure, an understanding of the principles of similitude is necessary. Many treatise have been written on the subject but the majority of them become quite involved. After describing the previous models it is felt that this manual would not be complete without some explanation of the laws governing model and prototype performance relationships. This will be done briefly but incompletely.

A thorough knowledge of the principles of the mechanics of similitude is indispensable in hydraulic laboratory research. In planning a hydraulic model it is necessary to construct it with a definite purpose in mind and with an understanding of the various limitations involved. In fluid motion, the predominating forces that influence motion occurrences are: (a) gravitational forces, (b) friction between fluid particles or viscous forces, (c) capillarity or adhesive forces and (d) elastic forces. Theoretically, all of these are present, but in practice it is usually possible to neglect all but the two major forces. Inertial forces are omnipresent, and the choice of the other major forces must be made from the four listed above before it is possible to properly design a model. It is customary practice to designate the type of motion by

the ratio of the two major forces. These ratios are known as Froude's Reynolds', Weber's and Cauchy's laws, in the same order as listed above.

The following expression is Froude's law which expresses the ratio of inertia forces to gravitational forces in prototype and model;

$$\frac{v_p^2}{g_p L_p} = \frac{v_m^2}{g_m L_m}$$

in which v is velocity, g = acceleration due to gravity, and L is a length. Exact similarity exists, in this respect, if the two sides of the above expression are equal and no other forces are acting.

Reynolds' number is the ratio of inertia forces to viscous forces and is represented by the expression;

$$\frac{v_p L_p \rho_p}{\mu_p} = \frac{v_m L_m \rho_m}{\mu_m}$$

in which v is velocity, L is a linear dimension, μ = coefficient of viscosity of the fluid and ρ = density of the fluid.

Weber's number expresses the ratio of inertia forces to the forces due to surface tension.

$$\frac{v_p^2 L_p \rho_p}{\sigma_p} = \frac{v_m^2 L_m \rho_m}{\sigma_m}$$

in which σ = surface tension.

Cauchy's number is the ratio of inertia forces to the forces due to the elastic compression of the fluid and is expressed mathematically as follows:

$$\frac{v_p^2 \rho_p}{K_p} = \frac{v_m^2 \rho_m}{K_m}$$

in which K = the bulk modulus of elasticity of the fluid.

Complete similarity would exist between model and prototype were it possible to balance the four above expressions. It is, however, physically impossible to find a fluid for the model that possesses the exact values of density, viscosity, elasticity, and surface tension necessary to satisfy Froude's, Reynolds', Weber's, and Cauchy's expressions. In the majority of models, which are tested in this laboratory, open flow is prevalent and thus gravity forces predominate. The neglect of the other three major forces is responsible for inaccuracies only in the final result. It is possible to choose model scales and build and operate models such that some of the non-dominant forces will be negligible or compensating.

It is often a mistake to look for similar pictures of photographic accuracy in model and prototype as this is not always necessary. As long as it is possible to determine, from the behavior of the model, the corresponding behavior of the prototype with a fair degree of accuracy, the model is of value. A model designed for a certain class of phenonenon may reproduce these faithfully but it may err in the reproduction of others.

The laws of similitude apply not only to flow of fluids or gases, the design of ships, aircraft, and machinery but to the transference of various motion occurrences from one fluid to another. In other words it is possible by the principles of similitude to analyse flow conditions of a gas or oil, in a closed conduit, from knowledge of the flow of water or other fluid in another conduit. Similitude is kin- generally divided into three classifications, namely: geometric, kinematic, and dynamic similarity. The following relationships will be developed for the case in which gravity forces predominate and fluid friction, surface tension and elasticity effects are neglected.

Two objects are geometrically similar if the ratios of all corresponding dimensions are equal. This involves only similarity in form and these relationships can be derived from the expression;

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

in which L_m and L_p are corresponding linear dimensions in model and prototype respectively and L_r is known as the length ratio or linear scale ratio.

Geometric Similarity Relationships

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

Area $A_r = \frac{A_m}{A_p} = \left(\frac{L_m}{L_p}\right)^2 = \left(L_r\right)^2$

Volume $V_r = \frac{V_m}{V_p} = \left(\frac{L_m}{L_p}\right)^3 = \left(L_r\right)^3$

Hydraulic radius $r_r = \frac{L_m^2}{L_p} = L_r$

These physical properties are the same for all four laws, Froude, Reynolds, Weber and Cauchy.

Two systems are kinematically similar if the fluid motions are geometrically similar at all corresponding periods of time. Stating it another way the ratio of the time periods required for two similar motions must be constant throughout the system or

$$\frac{t_m}{t_p} = t_r$$

in which t_r is the time ratio or time scale.

By definition acceleration $a = \frac{L}{t^2}$,

and for vertical acceleration $a = g$

so $t = \sqrt{\frac{L}{g}}$, and $t_r = \sqrt{\frac{L_r}{g_r}}$.

Since $\rho = \frac{w}{g}$, in which ρ is density, w is specific weight and g is acceleration of gravity, the time ratio may be written

$$t_r = \sqrt{\left(\frac{L\rho}{w}\right)_r}$$

Furthermore, when two motion occurences are kinematically similar, they are dynamically similar if the ratios of the homologous forces which in any affect the motion of the homologous objects are equal to the ratios of the masses of these homologous objects.

The fundamental expression relating the forces and the motion is expressed as Newton's second law of motion:

$$F = ma$$

Expressing this in terms of the four dimensionless ratios L_r, t_r, m_r and F_r (length, time, mass and force)

$$F_r = m_r \frac{L_r}{(t_r)^2} = m_r \cdot \frac{L_r}{\left(\frac{L\rho}{w}\right)_r}$$

Since mass is the product of volume and density

$$m_r = \frac{m_m}{m_p} = \frac{V_m \rho_m}{V_p \rho_p} = (L^3 \rho)_r$$

and

$$F_r = (L^3 \rho)_r \cdot \frac{L_r}{\left(\frac{L\rho}{w}\right)_r} = (L^3 w)_r$$

From the foregoing expressions the kinematic and dynamic relationships will be developed according to Froude's law. When the fluid in model and prototype is the same, ρ_r and w_r are both unity and thus drop out of the above expressions. When the fluid is not identical in model and prototype, ρ_r and w_r are no longer unity and must appear in the similarity relationships.

Kinematic similarity Relationships

Time $\quad t_r = \sqrt{\left(\frac{L\rho}{w}\right)_r}$

Velocity $\quad v = \frac{L}{t} \qquad v_r = \frac{L_r}{\sqrt{\left(\frac{L\rho}{w}\right)_r}} = \sqrt{\left(\frac{Lw}{\rho}\right)_r}$

Acceleration $\quad a = \frac{L}{t^2} \qquad a_r = \frac{L_r}{\left(\frac{L\rho}{w}\right)_r} = \left(\frac{w}{\rho}\right)_r$

Discharge $\quad Q = \frac{L^3}{t} \qquad Q_r = \frac{L_r^3}{\sqrt{\left(\frac{L\rho}{w}\right)_r}} = \left(L^{5/2}\sqrt{\frac{w}{\rho}}\right)_r$

Roughness factor $\quad n = \frac{c}{v} \cdot r^{2/3} \cdot S^{\frac{1}{2}} \qquad n_r = \frac{c}{\sqrt{\left(\frac{Lw}{\rho}\right)_r}} L_r^{2/3} \cdot S_r^{\frac{1}{2}}$

or $\quad n_r = \left(L^{1/6}\sqrt{\frac{\rho}{w}}\right)_r$ where n is the

coefficient in the Manning formula $\quad v = \frac{1.486}{n} \cdot r^{2/3} \cdot S^{\frac{1}{2}}$.

Dynamic similarity Relationships

Mass $\quad m_r = (L^3 \rho)_r$

Force $\quad F_r = (L^3 w)_r$

Momentum $\quad M = mv \quad M_r = (L^3 \rho)_r \cdot \sqrt{\left(\frac{L\,w}{\rho}\right)_r} = \left(L^{7/2} \cdot \sqrt{\rho\, w}\right)_r$

Work $\quad W = FL \quad W_r = (L^3\, w)_r \cdot L_r = (L^4\, w)_r$

Power $\quad P = \frac{W}{t} \quad P_r = \frac{(L^4\, w)_r}{\sqrt{\left(\frac{L\,\rho}{w}\right)_r}} = \left(L^{7/2} \cdot w^{3/2} \cdot \rho^{-\frac{1}{2}}\right)_r$

Impulse $\quad I = F.t \quad I_r = (L^3\, w)_r \cdot \sqrt{\left(\frac{L\,\rho}{w}\right)_r} = \left(L^{7/2} \cdot \sqrt{\rho w}\right)_r$

The above relationships apply only for flow in which gravity forces predominate. Should vicous forces predominate, an entirely different set of relationships would hold for the kinematic and dynamic properties.

One of the difficulties in model design consists in making surfaces sufficiently smooth to represent the roughness factor, n, to the ratio shown. For large models, this is possible, but for small models the friction is seldom to scale. When the frictional resistance is not to scale in a model, all flow relationships are thrown out of scale by a corresponding amount. It is possible, however, to compensate for the excess friction in the model by increasing the slope S (40). Surface tension is another factor which can affect the accuracy of model results.

This is actually "adhesion," the tendency for the fluid to cling to boundary surfaces of the model. This factor is always present but its effect can be reduced to negligible proportions by constructing large models or employing a wetting agent such as aerosol. In both cases, the larger the model the more accurate will be the results obtained. Various factors, however, limit the size of a model such as space available, discharge capacity of the laboratory, accuracy of results anticipated, and final cost. Accuracy of results are usually sacrificed to some extent at the expense of other considerations.

32. Distorted models. Should the model in question be distorted, that is, the vertical scale be different than the horizontal scale, the similitude relations must be revised somewhat. Models of rivers and canal structures, where the length of reach is great in proportion to width and depth, are often distorted, otherwise the channels would be of insufficient cross section to obtain representative flow conditions. The principal limitation for a distorted model is that it should be designed such that conversions of kinetic energy to potential energy and the reverse will not be distorted. It is particularly important to remember that the hydraulic radius in a distorted model bears no definite relation to either scale ratio and therefore cannot be computed as a general expression. The roughness factor, n, also follows no definite law with respect to scale ratio as it is dependent on the hydraulic radius for solution. As a result, it is necessary to compute these two factors for each individual section of the river. The remaining factors which do bear definite relationships to the chosen scale ratios are listed. These are based on Froude's law and apply where the fluid in model and prototype is the same.

Relationships for Distorted Models

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

Vertical length $\frac{D_m}{D_p} = D_r$

Area in horizontal plane $A_r = \frac{A_m}{A_p} = \frac{L_m^2}{L_p^2} = L_r^2$

Area in vertical plane $A_r = \frac{A_m}{A_p} = \frac{L_m D_m}{L_p D_p} = L_r D_r$

Time $t = \frac{L}{v}$ and $t_r = \frac{L_r}{v_r} = L_r D_r^{-\frac{1}{2}}$

Discharge $Q = Av$ and $Q_r = A_r v_r = L_r D_r D_r^{\frac{1}{2}} = L_r D_r^{3/2}$

Volume $V_r = \frac{V_m}{V_p} = \frac{L_m^2 D_m}{L_p^2 D_p} = L_r^2 D_r$

Velocity $v = gD$ $v_r = \frac{v_m}{v_p} = \sqrt{\frac{g_m D_m}{g_p D_p}} = D_r^{\frac{1}{2}}$

providing $g_m = g_p$.

Other relationships must of necessity be computed for the specific model in question. The foregoing explanation of model-prototype similitude has been made in as simple a manner as possible. For more complete information on dimensional analysis and similitude the reader is referred to references (10) (22) and (28) of the bibliography.

CONSTRUCTION OF MODELS

33. Materials and methods of construction. Hydraulic models are commonly constructed of oiled wood, wax, sheet iron, concrete, bronze, aluminum, plastics, and other materials which are easily worked and fairly resistant to moisture. Alterations are usually foreseen in advance and the model constructed so that these can be made easily. A rather complete discourse on model construction and operation is available on pages 44 to 58 of reference (4), so a repetition of this phase of model work is unnecessary here. In addition, specific examples covering the construction, operation, and results of several types of models are included in pages 67 to 108, of the same source.

Model and prototype comparisons

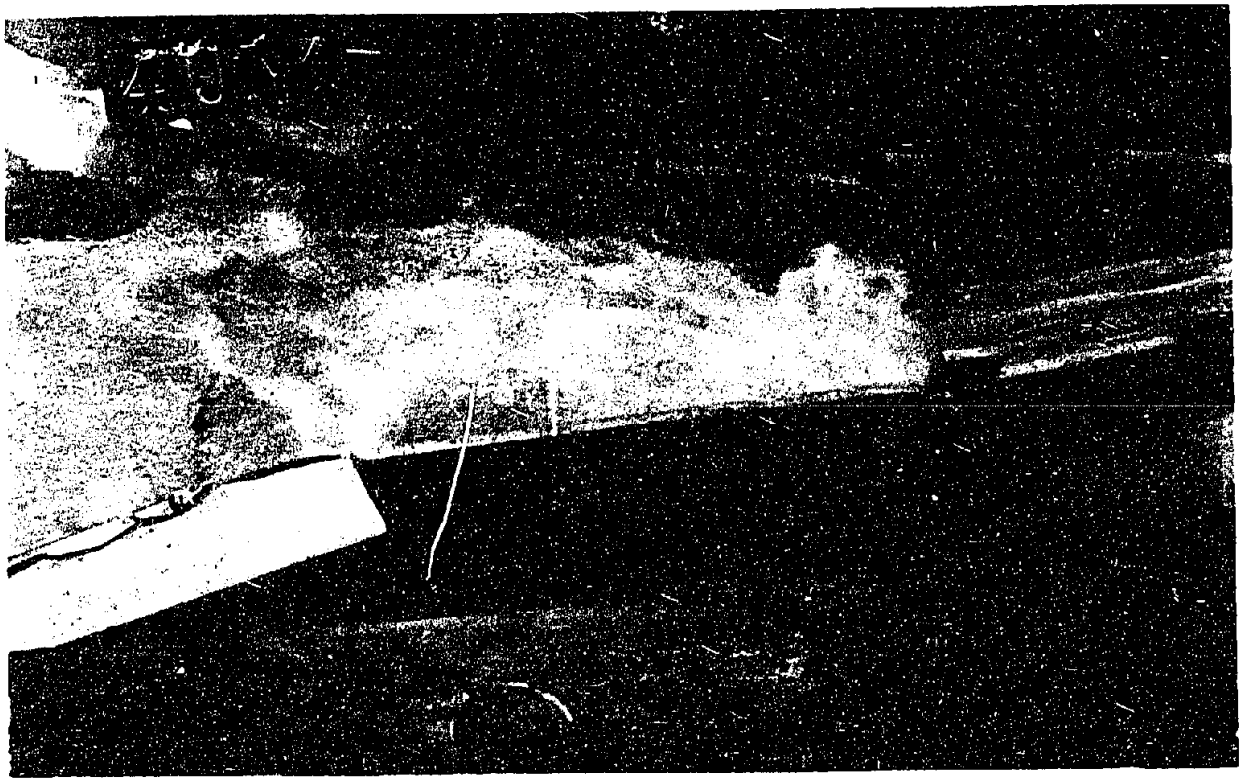
34. Difficulties involved in obtaining comparisons. Good model-prototype comparisons are few as it is difficult to reproduce exactly similar flow conditions in the prototype as were established in a model study. For example, spillways on the majority of western dams spill approximately once in ten years, and then the discharge is usually much less than the designed capacity of the structure. Floods generally occur in overcast weather making photography difficult, and the peak flow often occurs at night. To obtain satisfactory prototype photographs or measurements for comparison, therefore requires that someone familiar with the model study be equipped and on hand at the proper time and place. It is also necessary that a hydrographer be present to stream gage the river for determination of the discharge. To satisfy the foregoing conditions it is evident that obtaining prototype measurements on spillways, comparable with previous model tests, is the exception rather than the rule.

In the case of canal structures, where the discharge can be controlled, comparable model and prototype measurements can be obtained

with little difficulty. This is also true in the case of dam outlet works and closed conduit structures which are controlled by gates or valves.

35. Canal structures. Comparable model - prototype photographs of a canal chute stilling basin on the south canal of the Uncompahgre project in Colorado are shown on figure 43. The model, photograph A, was built to a 1:20 scale and is shown in operation at a discharge corresponding to 820 second-feet. Photograph B shows the prototype structure in operation at 825 second-feet for approximately the same tailwater condition. The grid on the wall of the prototype corresponds exactly with the grid on the model. A comparison of the water surfaces in the two stilling basins shows the action to be quite similar in model and prototype but the prototype surface is much rougher. Experience has shown that this is generally true in stilling basin operation where moderate or high velocities are involved. The hydraulic jump in the prototype is more violent than that in the model because the proportion of air in the former to that in the model is completely out of scale. Air is released from the water in explosive blasts in the prototype and this action is responsible to some for the larger surges. The air content of the water flowing through the model produces a very minor effect. This can be explained by the fact that flowing water will not insufflate air to any appreciable extent at velocities less than 20 feet per second. For an increase in velocity the percentage of air increases rapidly (41) (42). In this particular case the model velocity did not exceed 12 feet per second while the prototype velocity approached 55 feet per second. So a difference in the water surfaces in the two cases can be expected. In the design of high velocity structures additional freeboard, over that indicated by the models, is allowed for air entrainment.

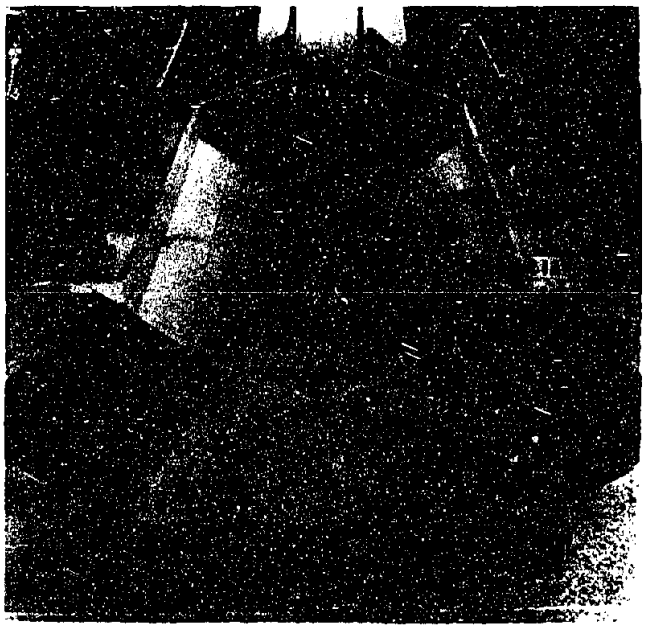
The photographs on figure 44 show another model-prototype comparison of flow in a trapezoidal canal stilling basin. Photograph A shows the general shape of the model basin and photograph B shows the model operating at a discharge corresponding to 1135 second-feet. Photograph C shows



A. Model on 1:20 scale discharging at 820 second-feet.

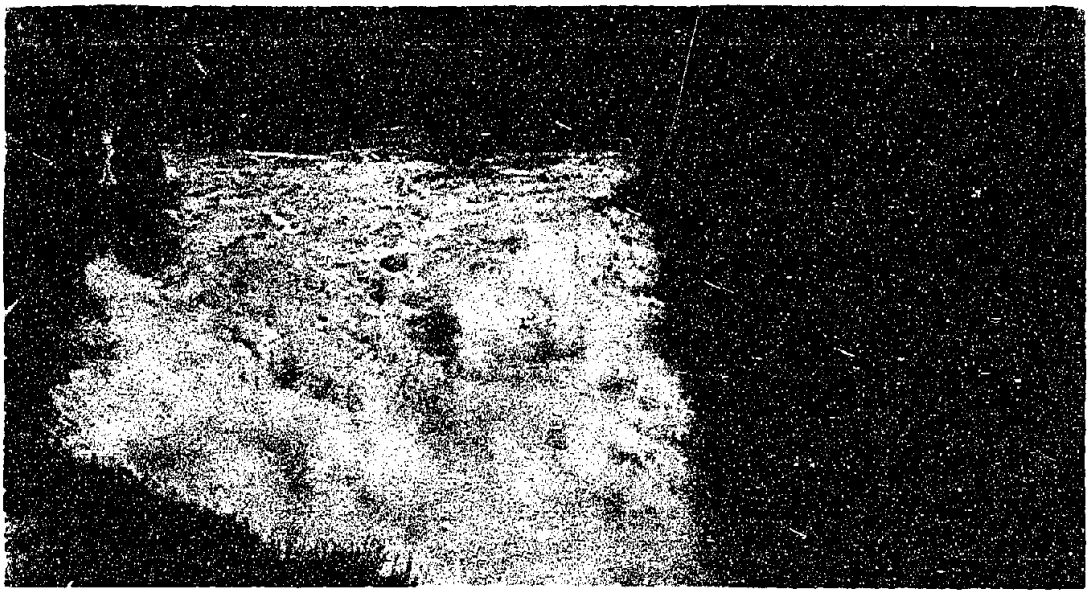
B. Prototype structure discharging at 841 second-feet.

MODEL AND PROTOTYPE COMPARISON OF SOUTH CANAL CHUTE ON UNCOMPAHGRE PROJECT, COLORADO



A. Model on 1:18 scale showing shape of stilling basin

B. Model discharging at 1135 second-feet



C. Prototype structure operating at 1160 second-feet for similar tailwater condition

MODEL AND PROTOTYPE COMPARISON OF A CANAL CHUTE ON THE SUN RIVER PROJECT, MONTANA

the prototype operating at 1100 second-feet for the same tailwater condition. The two comparable views were taken from opposite directions but it is clear that the flow in model and prototype is similar. In both cases the water surface was higher on the left than on the right wall. This is the only significant point that is clearly visible in this comparison. Unsymmetry and instability are characteristics of flow in trapezoidal stilling pools. The prototype photograph was taken previous to making alterations to the structure. The model was employed to study methods for redesign of the prototype.

Another case in which a canal structure was redesigned by means of a model study is described in detail, on pages 12 to 14, reference (13). A qualitative comparison between model and prototype is included for both the original and final designs.

36. Dam Spillways. A more striking example of air entrainment at very high velocities is shown by the photographs in figure 45. Photograph A shows a model of the Grand Coulee Dam on a 1:60 scale operating at a discharge corresponding of 250,000 second-feet and photograph B illustrates the prototype discharging at 200,000 second-feet. In the prototype the roller at the base of the dam is obscured by mist and spray. The roller was nevertheless present and much more violent than the model would indicate. This comparison illustrates a model limitation for which it is necessary to make allowances when interpreting model results. A model - prototype comparison for the Madden Dam Spillway can be obtained from reference (39).

37. Diversion dams. The photographs on figure 46 show a comparison of model and prototype of the Roosevelt Dam on the Salt River in Arizona. Photograph B was taken during a flood which occurred in 1937. Photograph A represents the same discharge of 30,000 second-feet on a 1:48 scale model of the same design. A careful examination of the photographs show identical flow conditions and these, it can be noted, vary

A. Model on 1:60 scale discharging 250,000 second-feet

B. Prototype structure discharging at 200,000 second-feet

MODEL AND PROTOTYPE COMPARISON ILLUSTRATING AIR ENTRAINMENT
GRAND COULEE DAM

Model on 1:48 scale discharging
30,000 second-feet

B. Prototype structure discharging approximately 30,000 second-feet.

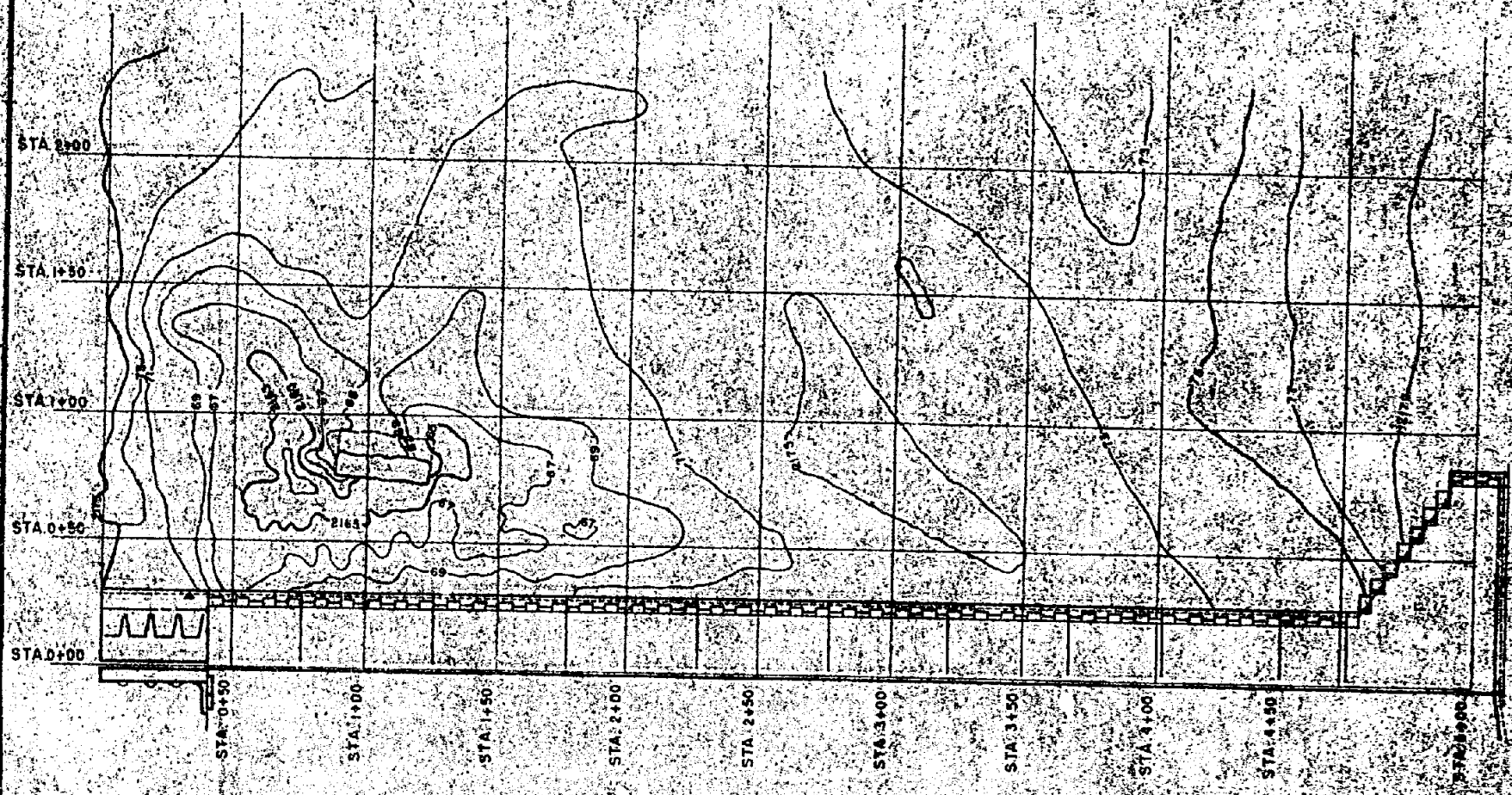
MODEL AND PROTOTYPE COMPARISON OF ROOSEVELT POWER DIVERSION DAM ON THE SALT RIVER, ARIZONA

FIGURE 46

over the length of the dam. In this case the fall, from headwater to tailwater amounts to only a few feet, thus air entrainment in the prototype was a minor factor.

Following the flood of 1937, detailed soundings were taken of the river bed downstream from the Roosevelt Power Diversion Dam. A contour drawing showing the scour in the river after abatement of the flood, is presented as figure 47. As the model was in tact at the time of the flood, it was possible to assimilate a corresponding flood on the model. A bed of fine sand was utilized both upstream and downstream from the model and the bed downstream was contoured after the run. The scour obtained in the model is shown on figure 48. Figures 47 and 48 present a very interesting comparison. The object immediately downstream from the left end of the structure is a section of old dam which was moved downstream and buried when the dam was partially demolished by a flood in 1916.

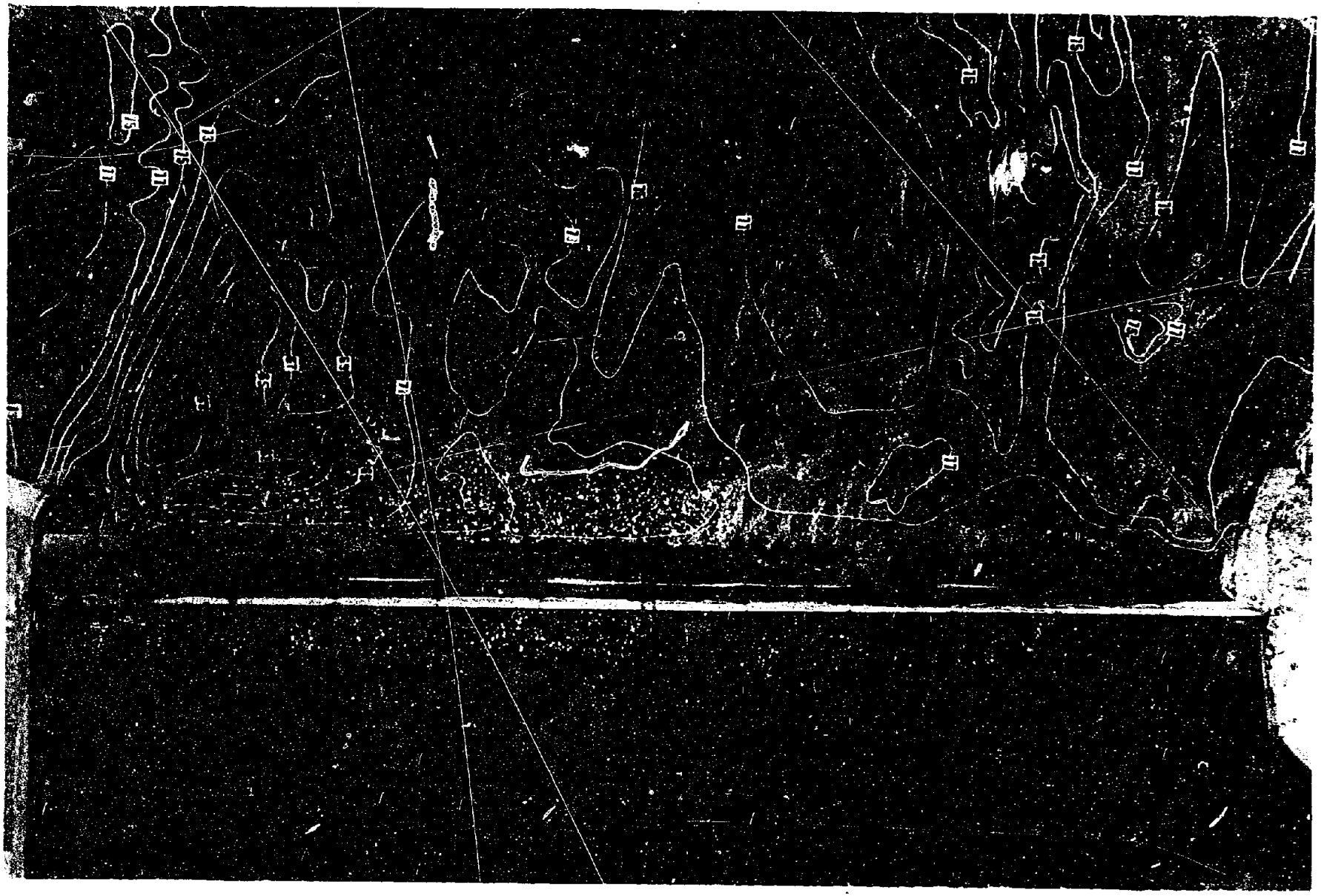
38. Outlet works. The best model - prototype comparisons are obtained from closed conduit structures where air entrainment does not enter the problem. A significant model and prototype comparison of pressures in one of the outlet conduits at Grand Coulee Dam is presented on pages 14 to 28, reference (13). A model - prototype comparison is also available on the Boulder Dam Outlet works (38). In addition an excellent model - prototype comparison for a hollow jet valve is reported in HYD-189.



POWER CANAL DIVERSION DAM
TOPOGRAPHY AND SECTIONAL ELEVATION
SHOWING EROSIAN OF RIVER BED DUE TO FLOOD
OF FEB. 7 AND 8, 1937

PLAN

0 10 20 30 40 50
SCALE OF FEET



MODEL OF ROOSEVELT POWER DIVERSION DAM SHOWING SCOUR IN RIVER CHANNEL DOWNSTREAM FOLLOWING A FLOOD OF 30,000 SECOND-FEET

FIGURE 48

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