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CHARLES D. WALCOTT, DIRECTOR

UNDERFLOW TESTS

IN THE

DRAINAGE BASIN OF LOS ANGELES RIVER

BY

HOMER HAMLIN

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DEPARTMENT OF THE INTERIOR,
UNITED STATES GEOLOGICAL SURVEY,
RECLAMATION SERVICE,
Washington, D. C., March 28, 1904.

SIR: I have the honor to transmit, for publication in the series of Water-Supply and Irrigation Papers, a manuscript entitled "Underflow Tests in the Drainage Basin of Los Angeles River," by Mr. Homer Hamlin.

The report describes the conditions under which ground water usually occurs in arid regions and the fluctuations in the water level due to rainfall and other causes. The methods used in obtaining this information are of interest, and the paper will contribute valuable material to the important subject of underground waters and their use in the arid regions.

Very respectfully,

F. H. NEWELL,
Chief Engineer.

Hon. CHARLES D. WALCOTT,
Director United States Geological Survey.

UNDERFLOW TESTS IN THE DRAINAGE BASIN OF LOS ANGELES RIVER.

By HOMER HAMLIN.

INTRODUCTION.

The purpose of this report is to assemble in one publication the results of a series of underflow tests made in the drainage basin of Los Angeles River in 1902 and 1903 by the United States Geological Survey.

This report briefly describes the conditions under which ground water usually occurs, especially in arid regions, and the fluctuations in its water level due to rainfall and to sinking flood waters.

The method of testing used was invented by Prof. Charles S. Slichter, and is fully described in "The Motions of Underground Waters."[a] Up to the present few investigators have used this method.

An attempt has been made to describe briefly the method of sinking test wells and the machinery designed and used during this investigation. The various devices used in testing and the arrangement of the instruments, the methods of testing found most satisfactory, the results obtained at each of the testing stations, and the amount of underflow supposed to pass the Huron street section, are fully described.

GROUND WATER.

By ground water is meant water percolating beneath the surface of the earth. The original source of all ground water is rainfall. Part of the rain soaks into the ground and percolates downward until it reaches a level where the interstices of the rocks, sand, and gravel are already saturated; another part is absorbed by growing plants or evaporated from the surface of the ground; and still another part runs off in surface streams.

[a] Slichter, Charles S., The motions of underground waters: Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902.

WATER TABLE.

The upper surface of the water-soaked zone in pervious ground is called the water table or water plane. From the surface of the ground to the water table the ground is damp but not saturated, and all contributions of surface water continually tend to sink downward, while below this level the ground is completely saturated; that is, all of the open spaces between the rock particles are full of water. The water table is the level at which water is struck in a well. In artesian belts the water is under pressure, being confined by overlying impervious strata, and when such strata are pierced it rises to the level of the outcrop of the water-bearing formation. The water table is not a level plane like the surface of a lake, except rarely in inclosed basins, but usually has a slope toward the main drainage lines of the region, to which the ground water slowly flows. The slopes usually follow in a general way the surface topography, but are much flatter.

FLUCTUATIONS OF WATER TABLE.

As the ground below the water table is completely saturated, any contributions of water from the surface must result either in raising the water table or in a lateral movement of the ground water toward some outlet. Usually both phenomena occur, the lateral flow being a result of and following the rise in the water table. In humid regions the supply from rainfall is so nearly constant throughout the seasons that the position of the water table is practically fixed, and the rate of lateral movement does not change appreciably from year to year. In arid regions much of the ground water under the plains and valleys is supplied by the streams which pour down from adjacent mountain ranges. When such streams leave the mountain canyons they soon sink in their débris cones of sand, gravel, and bowlders; in such localities the water table is sometimes 200 or 300 feet below the surface. When in flood such streams discharge enormous quantities of water, which sinks and is added to the ground water of the region. The immediate result is the raising of the water table, often 25 to 50 feet, over the region where the sinking occurs. The ground water afterwards flows outward in a generally horizontal direction from these regions and the water table is gradually lowered. In arid regions the water table rises sometimes many feet during rainy seasons and gradually sinks in dry years or during the dry season.

VELOCITY OF UNDERFLOW.

The slowly moving ground water beneath a stream bed or valley is usually designated "the underflow." In order to estimate accurately the amount of ground water passing a given section, it is necessary to know, among other factors, the rate of movement or velocity of the underflow through the pervious beds below. This is conveniently measured by the method invented by Professor Slichter.[a] Briefly the method is as follows:

A group of four or five wells arranged as shown in figs. 9 and 10 is sunk at the locality where the underflow is to be tested. Well A is placed upstream, or in the direction from which the underflow is supposed to come, and the wells B, C, D, and E are spaced downstream at a uniform distance from well A. All of these wells are of small diameter (drive pipe, 1¼ to 2 inches) and have from 4 to 8 feet of perforated screen, usually ordinary well points, at the lower end. These perforations allow the percolating water to pass through the pipe driven into the pervious stratum to be tested. The upper well, A, is charged with a chemical, usually sal ammoniac (NH_4Cl), which dissolves in the water passing through the well and is carried along by the underflow to one or more of the lower wells—which ones depends upon the direction of the underflow. The arrival at one of the lower wells of water containing sal ammoniac in solution is detected by means of electrical instruments.

METHODS AND APPARATUS USED IN UNDERFLOW TESTS.

In September, 1902, the writer was placed in charge of experiments to determine, if possible, the amount of underflow passing through the narrows of Los Angeles River at Huron street, Los Angeles, Cal.

Velocity measurements were begun under direction of Prof. Charles S. Slichter, with the apparatus invented by him. As the work progressed and tests at greater depths were made it was found necessary to modify this apparatus to suit local conditions.

LOCATION OF TEST WELLS.

The first step in testing the velocity of an underflow is to determine, approximately, the direction in which the underflow moves, and the second is to drill the test wells for measuring it. If the locality chosen is in a narrow valley or canyon bounded by rock walls, it is obvious that the underflow must be down the general trend of the valley. If, however, the tests are to be made in a wide valley or plain, it is advisable to ascertain the extent and slope of the water table in the vicinity.

If there are wells near, they should be located by some of the ordinary methods of surveying, and levels referred to some permanent bench mark should be run and the surface elevation at each well determined. The depth to water should be measured and the elevation of the water table computed. It is sometimes necessary to sink additional test wells, which should be located as just described. The data

[a] Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, pp. 48-51.

should be platted on a map and contours, or lines of equal elevation, of the water table drawn, and the slope of the water table determined. The movement of the ground water in general is down the slope of the water table.

Fig. 21 (p. 42) is a map of the narrows of Los Angeles River at Huron street, Los Angeles, Cal., showing the extent and slope of the water table at that locality. The broken contour lines show the form of the water table in December, 1902; the dotted contour lines the form in June, 1903. The rise in the water table was due to the sinking flood waters of the Arroyo Seco, which enters Los Angeles River a short distance below. Its channel is usually a dry wash of sand and bowlders, but was in flood during March and April, 1902, and the map shows plainly the sudden rise in the ground water due to the sinking of the storm waters.

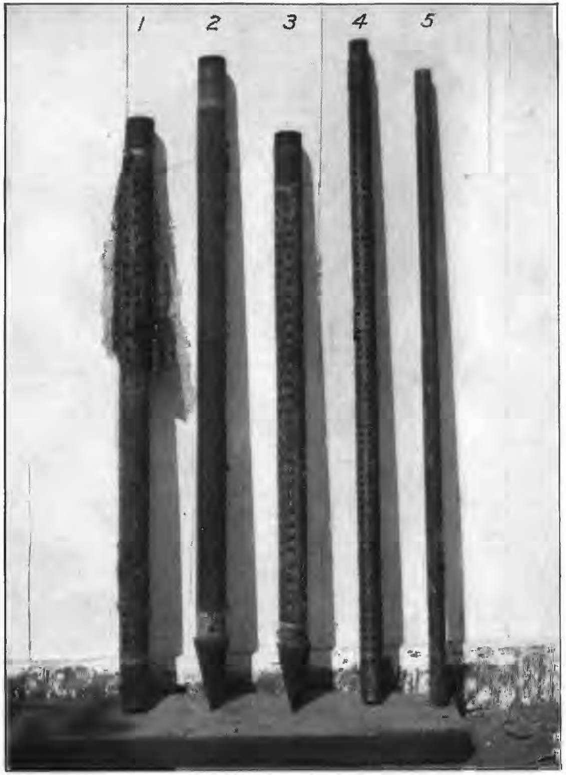
A practical conclusion to be drawn from this map is that in order to avoid the disturbing influence of the fluctuations of a tributary stream the test wells should be located a considerable distance either above or below the mouth of such a stream.

When the slope of the water table and the approximate direction of the underflow have been ascertained, wells for testing the velocity should be sunk in groups arranged as shown in figs. 9 and 10 (pp. 24 and 25). They may be sunk by any of several methods, depending on the locality, depth to the water table, coarseness of material, etc.

HAND DRIVING.

For shallow wells in unconsolidated material ordinary well points, as shown in figs. 2 and 3 of Pl. I, *A*, may be used. These are screwed on a length of standard or extra strong wrought-iron pipe and simply driven into the porous strata. Two men using a heavy wood maul, as shown in Pl. I, *B*, can drive such wells to a considerable depth, and, in such cases, this is a rapid and economical method. It is not possible, however, to collect samples of the material penetrated, and the porosity must be estimated from the amount of water yielded when the wells are pumped. This method was used in the first velocity test made in Los Angeles River, but was abandoned as unsatisfactory when deeper tests were attempted.

A portable hand rig is sometimes used to drive well points. Ordinarily two men can drive as many feet a day with a maul as with the hand rig in unconsolidated sands, unless the wells are deep. When used in combination with the hydraulic-jet method of sinking wells, such a rig will sink a pipe to a considerable depth, depending on the material penetrated, but when the material is coarse a point is soon reached beyond which it is impossible to drive any style of small pipe, as it is telescoped or battered by the hammer.



A. WELL POINTS

B. DRIVING WELL POINTS.

WELL-DRILLING RIG.

When wells are to be sunk to a considerable depth, 100 to 150 feet for instance, in very coarse material, some kind of machinery is necessary. A rapid and economical method is a modification of the hydraulic-jet process of deep-well drilling by combination with driving. This method consists of three distinct processes—(1) driving the pipe; (2) chopping up and washing out the sand and gravel that enters the bottom of the pipe as it is driven downward; (3) turning or rotating the pipe to keep the hole straight and to reduce the frictional resistance to driving. A rig equipped for this work, with the machinery assembled, is shown on Pl. II. It consists of the derrick for hoisting the heavy drivepipe, a hammer for driving the pipe, wash rods and chopping bit for hydraulicking, a pump for forcing water down the wash rods under pressure, and a rotator for turning the pipe. The whole is driven by a gasoline engine with a hoisting attachment. This machinery was designed to meet the local conditions of sinking wells to a depth of 100 to 150 feet in very coarse sharp sand, gravel, and bowlders. It has proved economical and satisfactory in actual use for several months.

MACHINE DRILLING.

When starting a well, a hole is first dug to a depth of 4 or 5 feet and a 10-foot length of drivepipe with a shoe firmly screwed on is inserted vertically, passing up through the rotator (a), as shown on Pl. III, *A*. It must be plumbed carefully, for if it is not driven straight the wells will not be properly spaced at the bottom, and this will introduce errors into the velocity measurements. About 13 to 15 feet of wash rod, with a chopping bit attached, is then inserted in the drivepipe, projecting about 3 feet above the top. The drive head (c) is then screwed on the drivepipe with the wash rods projecting up through the hole in the center. The buffer block (e) and hammer (f) should then be slipped over the wash rods which serve as a guide for the hammer, and the water swivel (g), with the hose attached, should be screwed to the top of the wash rods. The ropes for hoisting and driving and the sprocket chain for driving the rotator, etc., are attached as shown on Pl. II, *A*, and Pl. III, *A*.

The rig is operated as follows: Drive the pipe a few feet by tapping lightly with the hammer, then start the pump and force water down the wash rods and through the chopping bit. The rods should be churned up and down like a churn drill until the drivepipe is clear of sand to the bottom of the shoe. The chopping bit can be operated with, or independent of, the rest of the driving apparatus by a line passing over the middle sheave of the hoisting block. When the well is shallow the chopping can be done by hand, but when deep it is necessary to use the winch or winding drum on the engine. The water swivel is made with a projecting base larger than the hole through the hammer (see fig. 7). By means of this device it is possible to drill with the chopping bit and at the same time drive the pipe with the hammer. The length of wash rods is so adjusted that the water swivel is 6 inches to 1 foot above the top of the hammer, when it rests on the drive head. As the ordinary stroke of the hammer is 1 to 2 feet, the wash rods will be raised from 6 inches to 1 foot at each stroke. When the hammer is dropped the rods drop with it until the chopping bit strikes the sand in the hole, while the hammer still falls and almost instantly strikes the drive head. This is done automatically, the only attention required from the driller being the gradual lowering of the wash rods as the pipe is driven downward. The water which is forced out in jets through the chopping bit washes the pulverized sand and gravel up through the drivepipe to the surface. The drivepipe should be turned from three to five times per minute by an ordinary rotator such as is used with hydraulic-well rigs. The rotator should be run from the engine by a sprocket chain passing over sprocket wheels, as shown on Pl. II, *B*, and Pl. III, *A*. When the wells are shallow two men with chain tongs can usually turn the pipe.

This method is particularly adapted to sinking wells in coarse sand and bowlders. The drivepipe should be double, extra-strong, steel pipe to secure the strength and stiffness necessary to prevent bending when forced through coarse materials, and its internal diameter should be at least 2¼ inches. It is sometimes very difficult to pull a long string of drivepipe, especially to start it. The only practicable way is to use jackscrews, as shown on Pl. III, *B*, or use block and tackle with the engine at the same time turning the pipe with the rotator. Below is a more detailed description of machinery used.

APPARATUS.

ENGINE.

In the selection of an engine, consideration must be given to the cost of fuel, transportation facilities, weight of machinery, etc. When fuel is expensive and water scarce, a gasoline engine will be found economical. For ordinary drilling when the depth of the wells does not exceed 150 or 200 feet, a 6 to 8 horsepower engine will furnish the necessary power. A very convenient type of engine is shown on Pl. II, *B*. It is a vertical gasoline engine geared to a hoisting drum, which is controlled by means of a friction clutch and brakes. The drum is used to raise the hammer and wash rods, for hoisting pipe, etc. From ten to twenty blows per minute can be struck with the hammer in driving pipe. The small winch on the engine is used for pulling

A. WELL-DRILLING RIG.

B. WELL-DRILLING MACHINERY.

pipe, moving heavy weights, moving the engine, etc. The engine is belted to a force pump, which supplies water for hydraulicking, and is connected to the rotator by a sprocket chain. The engine shown in the cut is but 3 horsepower and is too light for deep wells, but has proved very economical in fuel consumption, using but 3 to 4 quarts of gasoline per day in ordinary drilling; in addition it has required but little attendance, is light, and easily moved.

PUMP.

The pump should be a suction force pump, capable of delivering water under a pressure as high as 150 pounds per square inch, with a discharge of 20 to 50 gallons per minute. It should be provided with a safety valve to prevent excessive pressure. The pump must be able to draw a supply from one to two driven wells, when the water table is within reach. In other cases it will be necessary to haul water for hydraulicking—an expensive and troublesome method.

Connection between the pump and the wash pipe, as shown in Pl. II, *A* and *B*, and Pl. III, *A*, is made by means of 25 to 30 feet of flexible hose of good quality to withstand high pressure. Ten to 15 feet of 2-inch suction hose will be needed to connect the pump with the drive wells from which the wash water will usually be pumped.

LIGHT DRIVEPIPE.

For shallow wells driven by hand in unconsolidated material the ordinary standard or extra-strong pipe will answer for drivepipe. It can usually be pulled and used several times. The outside couplings of such pipe greatly increase the frictional resistance in driving and pulling, and a considerable percentage of pipe is sure to be left in the ground. Extra-strong pipe is sometimes fitted with flush-joint couplings, but so much metal is cut away at the joint that they are frail and will not withstand hard driving, breaking at the joint when the pipe does not go down straight.

HEAVY DRIVEPIPE.

For deep wells the drivepipe should be what is known to the trade as double-extra-strong steel pipe with joints flush inside and out. The inside diameter of the drivepipe must be at least 2¼ inches. The nearest commercial size to this is known as 3-inch pipe; its actual diameter is 3½ inches outside and 2.284 inches inside, the thickness of its shell 0.608 inch and its weight 18.56 pounds per linear foot. Rings of tough Norway iron of the same diameter and thickness as the pipe, and 6 or 8 inches long, should be welded on both ends of each length of pipe, in which the flush-joint coupling should be turned. A taper thread with butt joints, as shown in fig. 1, has been found satisfactory and stands much hard usage. This form of thread has the advantage of a firm bearing throughout its entire length when it is screwed up tight. The pipe should be finished in lengths of exactly 10 feet with a few additional short lengths of 5 feet. A 1½ to 2 foot length (*b* on Pl. III, *A*.) should also be provided with a 1½-inch hole drilled through it about midway of its length to allow the wash water and sand to run out; the drive head should be screwed on the upper end of this short length. Such a drivepipe is stiff and heavy enough to stand repeated blows from the hammer. The fact that there are no outside couplings make it far easier to drive and pull than the ordinary pipe.

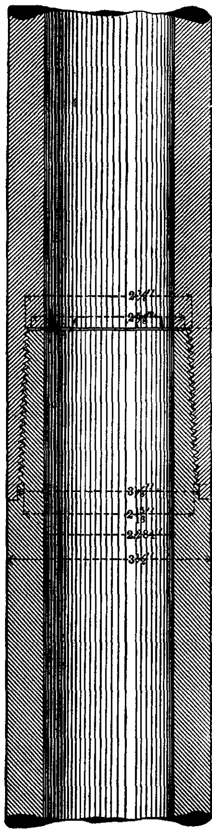


FIG. 1. Heavy drivepipe.

SHOE.

The lower end of the drivepipe must be protected from injury by a shoe. The form shown in fig. 2 has been found satisfactory. It should be turned out of tough steel, the outside diameter being a trifle greater than the outside of the drivepipe, and the inside diameter the same. If it is to be used with a rotator the cutting edge should be notched and well tempered, as it will then cut its way downward more readily. The threads should be the same as on the drivepipe, so that it will fit on any length.

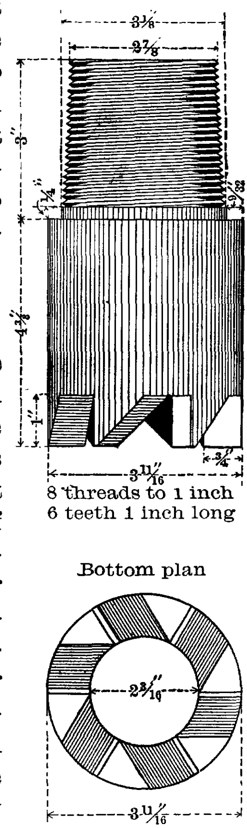
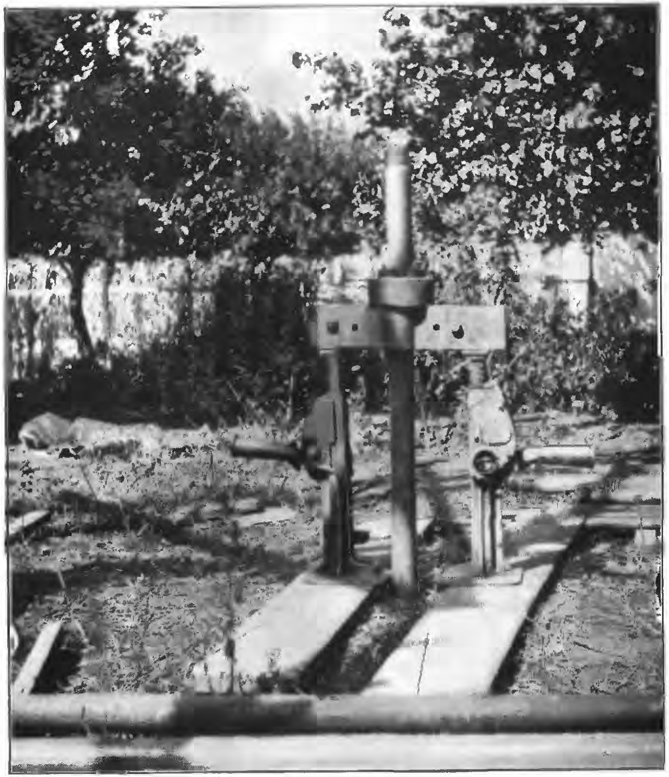


FIG. 2.—Drive shoe.

DRIVE HEAD.

The drive head, as shown in fig. 3, should be made of steel, accurately turned to fit the drive pipe on which it is screwed, and strong enough to withstand long-continued driving. The wash rods pass up through the hole in the center of the head, serving as a guide for the hammer (see *c* on Pl. III, *A*). The four holes drilled in the side are

A. ROTATOR AND DRIVING MACHINERY.



B. JACKSCREWS.

sockets for rods used in screwing on the head or in handling the pipe. A bail for lifting can be sprung into these holes, if desirable, but an eyebolt passing through the hole in the center and loosely fastened with a nut and washer inside the drive head will act as a swivel and be found stronger and more convenient when pulling pipe, hoisting, etc.

BUFFER BLOCK.

A wooden block should be placed between the hammer and the drive head to prevent the destruction of both. A convenient form is shown

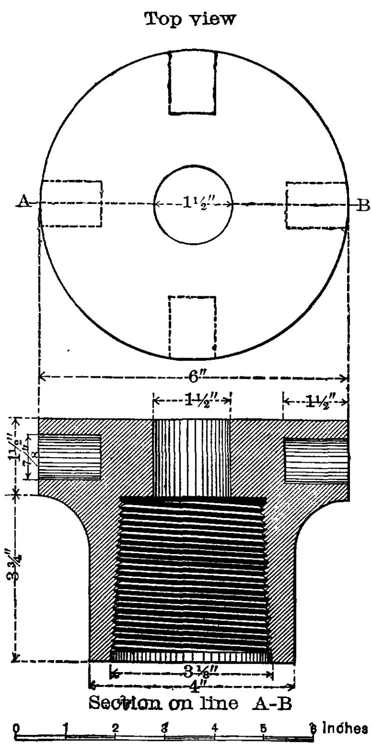


FIG. 3.—Drive head.

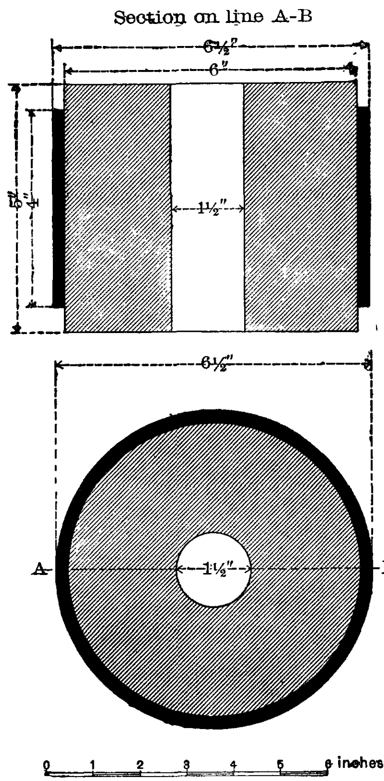


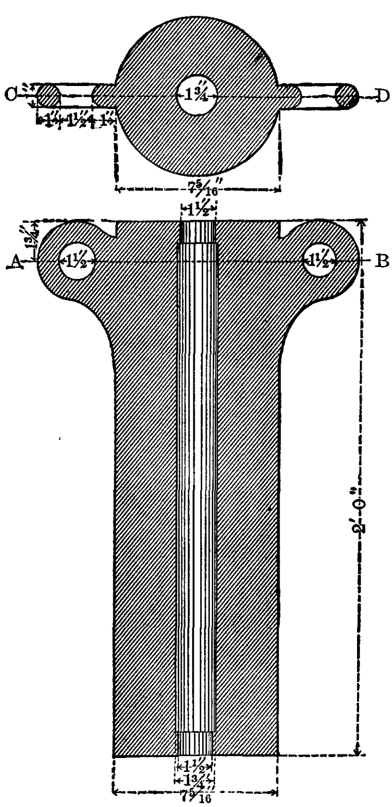
FIG. 4.—Buffer block.

in fig. 4. It is simply an oak or other hardwood block with the grain of the wood vertical, securely bound with an iron band. A hole should be bored through the middle and the block may then be slipped over the wash rods and rest on the drive head, as shown at e on Pl. III, *A*.

HAMMER.

A hammer of the form shown in fig. 5 requires no guides, but slides up and down on the wash rods. It may be made of cast iron with projecting ears for attaching the hoisting ropes. When it is necessary to add a length of drive pipe, the hammer can be raised off the wash rods, swung to one side, and lowered to the ground. It is raised and lowered independently of the wash rods by means of two ropes, which pass up through the outside sheaves of the hoisting block at the apex of the derrick and thence to the winding drum of the engine, as shown on Pl. II, *A*, and at f on Pl. III, *A*.

Section on line A-B



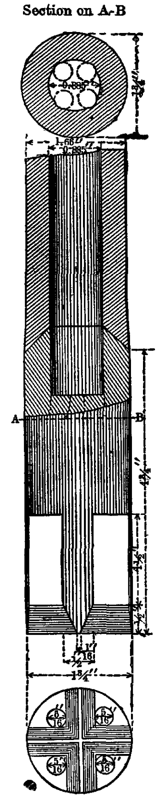
Section on line C-D

FIG. 5.—Hammer.

WASH RODS.

The wash rods should be made of what is known as double-extra-strong 1-inch pipe, the diameter being 1.315 inches outside and 0.587 inch inside, the thickness of the shell 0.364 inch, and the weight 3.65 pounds per linear foot. The rods should have rings of Norway iron on each end and the same style of taper thread as the large drive pipe shown in fig. 1. When finished, the rods should be exactly 10 feet in length, but several 5-foot and 2-foot pieces should also be on hand. If deep wells are to be drilled, heavier wash rods will be necessary. This will require larger holes than those figured in the drive head, buffer block, and hammer.

Section on A-B



Bottom plan showing beveled edges of chopping bit.

FIG. 6.—Chopping bit.

CHOPPING BIT.

The chopping bit should be made of best tool steel. A drawing of the bit in use is shown in fig. 6. It should be securely welded to a 5-foot length of wash rod instead of being screwed to it, otherwise when drilling in coarse material or bowlders it will probably be broken off and lost in the well. The cut shows the bit welded to a length of double-extra-strong 1¼-inch pipe, commercial size.

The edges of the star bit should not be drawn down too thin, and must be well tempered to withstand hard usage, for when drilling the impact of the heavy string of wash rods, when dropped several feet, must be borne by the chopping bit. The holes through the bit should be of ample size to deliver enough water to raise the pulverized sand and gravel to the surface and still must be small enough to give the issuing jet a high velocity.

WATER SWIVEL.

A swivel connection between the hose and the wash rods is necessary to permit turning the chopping bit, making connections, etc. The form shown in fig. 7 has been found convenient. It should be strong and well made, as the hammer engages with the lower part of the swivel in raising the wash rod. As sand and gravel in the drive pipe sometimes pack around the drill and rods, the pull on the swivel may be considerable, perhaps enough to stop the engine, in which case the wash rods must be jarred loose by upward blows with the hammer.

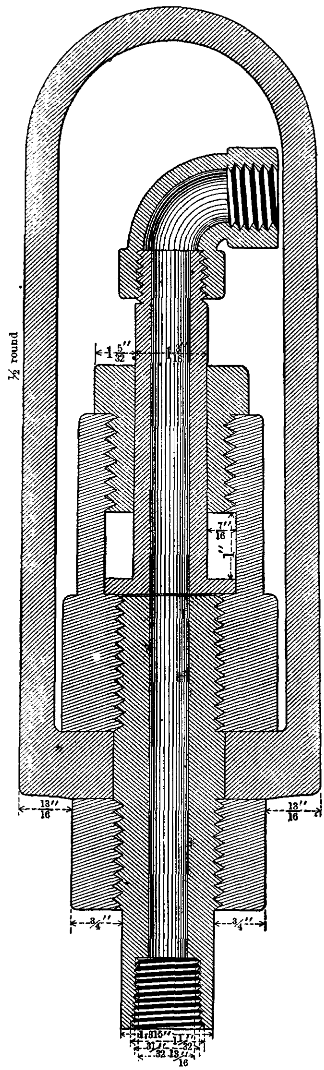


FIG. 7.—Water swivel.

DERRICK.

The derrick may be made of three timbers, about 4 by 6 inches, and 30 to 36 feet long, as shown in Pl. II, *A*. Holes are bored through the top and the timbers loosely connected by a 1-inch bolt, from which is suspended a long clevis, to which the hoisting blocks are attached. Such a tripod can be quickly raised, lowered, or adjusted over a well. Slats nailed on the timbers serve as a ladder for reaching the top.

BLOCKS.

The lines from the hammer should be run up over the outside sheaves of a triple block, and then to the winding drum of the engine, as shown in Pl. II, *A*. The line from the wash rods is run over the center sheave and then fastened to a cleat on one leg of the tripod. By this arrangement the rods can be easily raised or lowered by hand, or suspended at any height.

A double block should be provided for use with the triple block in pulling the drive pipe, moving the engine, etc.

WELL POINTS.

When wells are shallow and the material to be penetrated is soil or unconsolidated sand, it is economical to drive ordinary 1¼ or 2 inch points. These are usually in 4-foot lengths and are made by perforating standard or extra-strong wrought-iron pipe with seven rows of oblong holes three-eighths by one-half inch. Around the outside of the perforated pipe is wrapped a fine brass screen of No. 35 wire with about 2,500 meshes per square inch; over the gauze is wrapped a perforated brass screen with 28 one-eighth inch holes per square inch, the whole being securely soldered to the perforated pipe along the vertical seam. The gauze strains out the fine sand and silt, but allows the water to enter the well, and the brass screen is put on to protect the gauze. Fig. 1 of Pl. I, *A*, shows such a well point with the screen and gauze partly removed. This style of point serves for both a well and for part of the testing apparatus as described on page 26.

When the material is compact, or contains cobblestones or bowlders, these well points fail, either by bending, by breaking, or by stripping off the outside screen and gauze. If the material contains stones large enough to deflect the pipe, it will bend and either break in driving or in pulling; in fact, it is almost impossible to pull a crooked pipe, as it will break at one of the couplings. When the material is compact and coarse the frictional resistance on the screen will often strip it entirely off of the pipe, leaving it as shown in figs. 1 or 3 of Pl. I, *A*. To overcome these difficulties the experiment of placing a fine screen on the inside of the perforated drive pipe was tried and found quite satisfactory. Perforated sheet brass, known to the trade as No. 26 gage, No. 1, with 400 holes, about 0.02-inch in diameter per square inch, should be rolled into a tube slightly smaller than the inside diameter of the perforated pipe (fig. 5 of Pl. I, *A*), securely soldered along the vertical seam and shoved inside the perforated pipe, where it is held in place by the couplings above and below. It is largely protected from injury and can be withdrawn and cleaned if necessary. Such a well point is shown in fig. 4 of Pl. I, *A*. The only objection to this style of point is that the holes in the outside tend to fill with sand and clay when the pipe is pulled.

JACKSCREWS.

The jackscrews for pulling pipe should be strong, of at least 32 tons capacity each. A clamp of heavy flat iron, a pulling ring, and steel wedges or dogs with notched edges should be provided for hold-

ELECTRODES AND WELL.

ing the ring in place when the pipe is being pulled. The arrangement of the jackscrews, etc., is shown on Pl. III, *B*.

PLACING WELL POINTS FOR UNDERFLOW TESTS.

As noted above, ordinary well points are driven into the pervious strata where it is proposed to test the underflow.

When the heavy drivepipe is used the procedure is somewhat different. The pipe is driven by machinery to bed rock, or to the desired depth, and thoroughly cleaned from sand, silt, etc. A length of well screen similar to that shown in fig. 4 of Pl. I, *A*, is screwed to a sufficient length of 1¼-inch standard pipe and lowered to the bottom of the hole. The drivepipe is then pulled up, leaving the well screen and 1¼-inch pipe in the ground with the screen at the proper depth. The drivepipe is used to sink another well, in which is placed another screen, etc., until all the well screens of the group are in place. The well screens and small pipe, whether set by the first or the second method, now form part of the testing apparatus, being a portion of the electric circuit.

TESTING APPARATUS.

The various instruments at first used in testing the velocity of underflow were made from designs furnished by Prof. Charles S. Slichter. The modifications of the original apparatus and the methods developed are the result of several months' experimenting by the writer, and may be of value to investigators in the future.

WIRE.

The wire should be a good quality of rubber-insulated copper, about No. 14. Single wires are more convenient than a cable or a twisted lamp cord. Care must be used and the wire examined occasionally to see that the insulation is not worn through, for if it is there will probably be a short circuit between the well casing and the wire. It should be kept rolled into open coils, and not kinked or twisted, as such usage tends to break the insulation.

ELECTRODES.

The electrodes are brass rods from one-fourth to five-sixteenths inch in diameter and about 4 feet long, usually nickel plated (Pl IV). They should be heavy, of solid metal throughout, as it is difficult to sink a light electrode to the bottom of a deep well. The wire should be attached to the electrode by some such device as that shown in fig. 8, this being necessary in deep wells. It should not be bent or kinked where attached to the electrode, as it may break and leave the electrode in the bottom of the well. The electrodes are insulated from the well casing by means of wooden blocks (see fig. 8), which should be boiled in paraffin to make them impervious to water. The electrodes will need frequent plating, as the action of the electric current and of the sal ammoniac quickly corrodes the nickel coating.

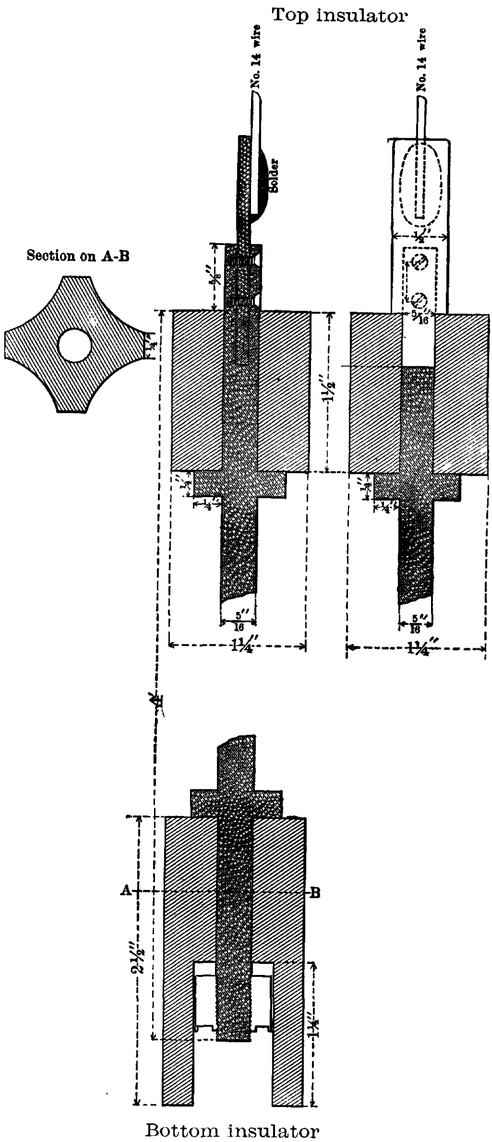
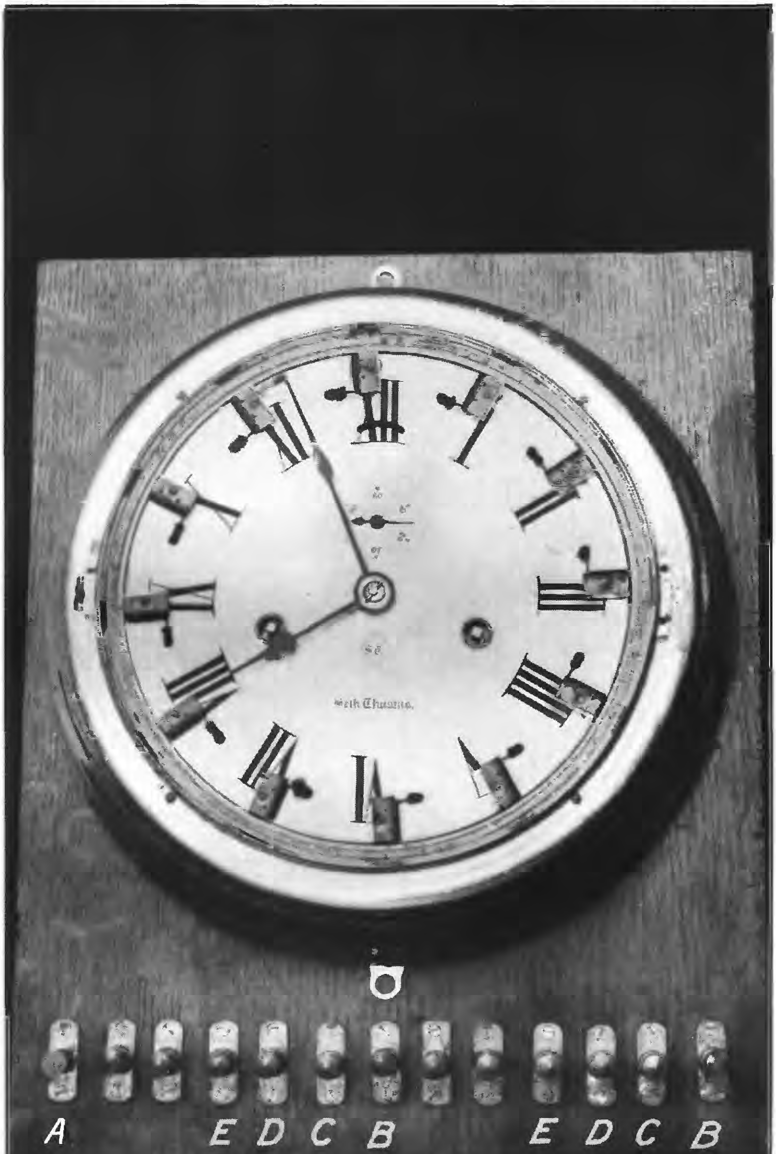


FIG. 8.—Electrodes.

SWITCH CLOCK.

A switch clock found satisfactory in actual use is shown in Pl. V. It is an ordinary eight-day clock with electrical devices added. The minute hand of the clock carries a steel spring, tipped with platinum, which, at five-minute intervals, is brought into contact with platinum strips, each of which is connected with a separate insulated wire passing down through the clock case and thence to the proper binding post. Each contact closes a circuit through one of the test wells. The platinum strips are laid in slits cut in lugs of hard rubber. The strips, the projection of which beyond the edge of the lug is adjustable, are held in place by screws which pass through narrow slots. The hard-rubber lugs are permanently fastened to the clock dial by screws from the back. They are placed at each hour on the dial or at five-minute intervals as measured by the minute hand.

In operating, the spring on the minute hand is carried over the hard-rubber lug, which is so placed that the spring is slightly raised. When the tip of the spring reaches the sharp edge of the lug it suddenly snaps down on the platinum strip and at once closes the circuit. It is



SWITCH CLOCK.

pulled across the platinum strip by the motion of the hand and when it reaches the edge snaps away from the strip, suddenly opening the circuit. The time the circuit is closed can be regulated by adjusting the width of the strip. Five to ten seconds is about the proper interval. It is important that the circuit be opened and closed suddenly, otherwise the recording pen, full of ink, may move back and forth on the chart several times and probably obliterate the record.

The current from the positive pole of the battery enters the clock at one of the binding posts, *B*, *C*, *D*, *E*, etc., and passes through the insulated wire to the minute hand, thence to the clock frame and out to the binding post marked *A*. By the movement of the minute hand the circuits to the various wells are progressively closed. The twelve switches on the dial permit of thirty hourly records with four lower wells and a thirty-minute interval between records, as shown on Pls. V, *A*, and VI, *B*, or of records at intervals of twenty minutes with three lower wells and a ten-minute interval between records. It is also evident that in the first case two groups of wells, or in the second case three groups of wells, can be tested hourly at the same time, with a ten-minute interval between tests.

RECORDING AMPERE METER.

The recording ampere meter (Pl. VII, *A*) was made by the Bristol Company from designs furnished by Prof. Charles S. Slichter. The instrument keeps a continuous record for twenty-four hours, after which the chart must be changed. The chart is moved at a uniform rate by clockwork. The recording pen is actuated by the electric current sent through the wells at fixed intervals by the switch clock. The electric apparatus is very simple, consisting of a stationary solenoid through which the current passes. A thin disk of soft iron is attached to a nonmagnetic shaft which passes through the solenoid and is supported at its opposite ends by steel knife-edge springs. The recording pen is secured directly to the steel springs and partakes of its angular motion as the disk is attracted by the solenoid. The result is a record such as shown on Pl. VI, *A* and *B*.

If an ordinary ampere meter is used the switching must be done by hand, as described on page 27.

BATTERY.

The electric battery should be of ample capacity. Dry cells known as large-size Columbia, 3½ inches in diameter and 8 inches high, have been found satisfactory. The plan and wiring of the battery for use with a recording ampere meter only are shown in fig. 9, and for use with either a recording or ordinary ampere meter are shown in fig. 10.

The battery consists of 12 large-sized dry cells connected in sets of two in parallel. These parallel sets are connected up in series, and by means of the double-throw switches the electromotive force of the battery can be varied from 1.5 to 9 volts, assuming the electromotive force of each cell to be $1\frac{1}{2}$ volts when new. One, two, three, or more sets of two cells connected in parallel can be used in one series, as the case requires, or if necessary either side of the battery can be cut out by the opening of one of the single-throw switches marked "A, A′." An ordinary ampere meter should, for convenience, be placed in the battery box. Fig. 10 shows such a battery connected up to a group of four wells and arranged for use with an ordinary ampere meter. The switches are set for measuring the current from casing of well B to the electrode of well B, with two sets of cells in series.

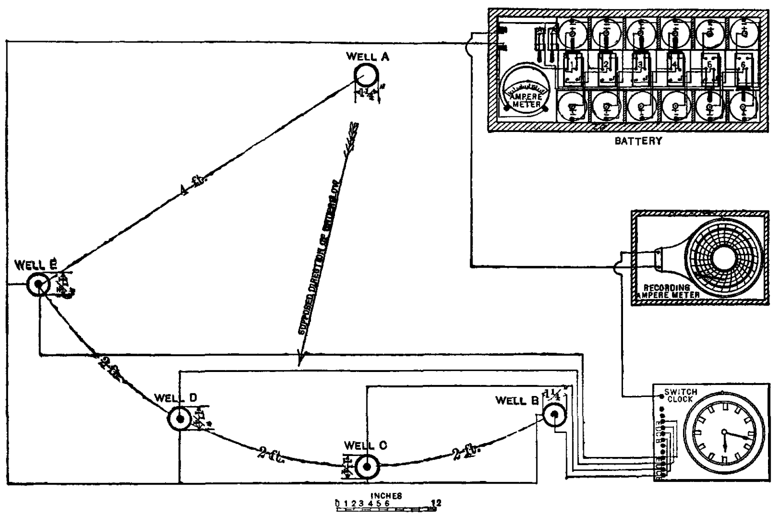


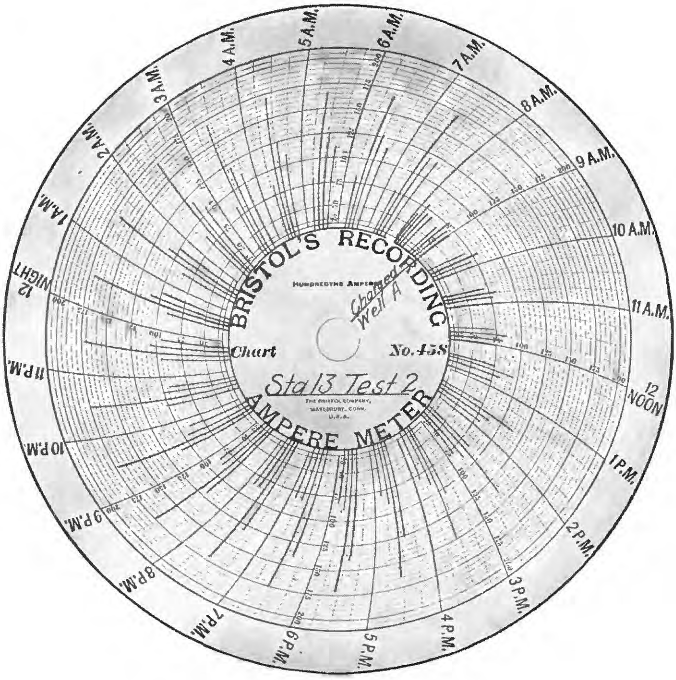
FIG. 9. Plan of test wells and apparatus.

METHOD OF MEASURING THE VELOCITY OF UNDERFLOW.

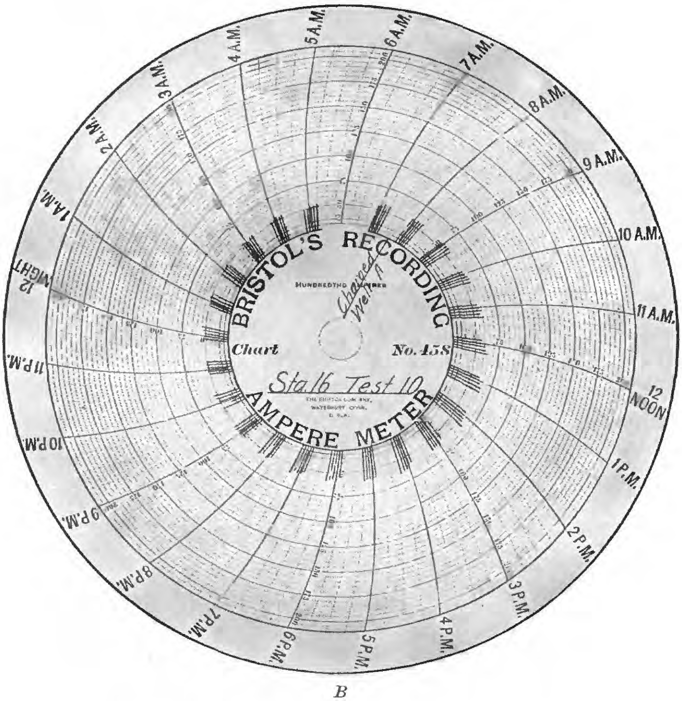
An outline of the method invented by Prof. Charles S. Slichter has been described on page 11. The following is a description more in detail of the writer's experience during several months' work at Los Angeles, Cal. An attempt has been made to describe the difficulties encountered and the methods developed during the progress of the work.

PUMPING.

After all the test wells have been sunk, as described on page 11, it is necessary before beginning a test to take a small sand pump and thoroughly clean the inside of the well points from sand and silt, and then pump water from each well until it clears. A small hand pump which can be readily moved from well to well is convenient for this



A



B

CHARTS USED WITH RECORDING AMPERE METER.

purpose. Pumping is often a tedious job, as the screens become clogged with silt and mud. In such cases a few buckets of clear water poured down the well will often open up the screen. Another way is to occasionally break the vacuum in the pump and let the water in the pipe fall back suddenly. When these methods fail, success may be attained by rotating the pipe with a pair of tongs and at the same time pouring clear water down the well.

If some of the wells pump freely and the rest will not start, it is almost certain that the screens are clogged, and if all else fails they may be allowed to stand a day or two. If none of the wells can be started, or after starting continue to pump fine sand and mud, the test may as well be abandoned, for in such cases the velocity will be too low to measure by this method. Testing should not be begun for a few hours after pumping, in order to allow the water plane to rise to its normal level.

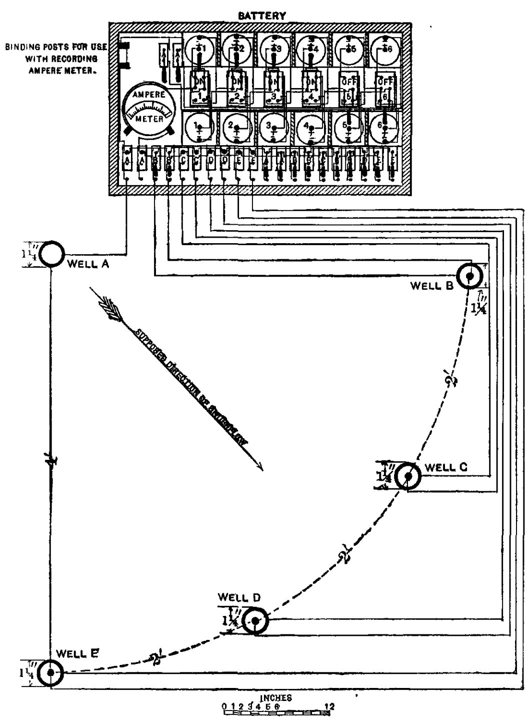


FIG. 10. Plan of test wells and apparatus.

WIRING.

The next step is to wire the wells. There are several ways in which this may be done, depending on the kind of electrical instruments used. If a recording ampere meter is used, the wiring should be as shown in fig. 9. A wire is run from the positive pole of the battery to the casings of lower wells B, C, D, E, etc., all of which may be connected by soldering the wire to the galvanized casing of each, thus insuring good connections. A wire is connected to the electrode for each well and run to the proper binding post on the switch clock; the electrodes should then be lowered to the bottoms. From the positive binding post of the switch clock a wire is run to the recording ampere meter, and from the ampere meter to the negative pole of the battery. An ordinary ampere meter (in the battery box) is put in the circuit to check the record of the recording instrument.

The current recorded is that which passes through the water between the casing of the wells and the electrode at the bottom of the well.

The construction of the clock, recording ampere meter, battery, etc., is described on pages 21 to 24.

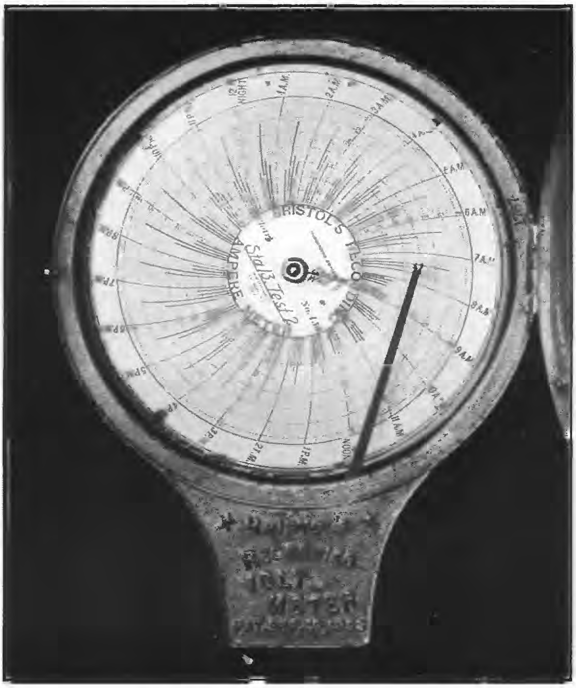
If an ordinary ampere meter is used the wiring will be quite different, as all switching must be done by hand. A battery arranged for such a testing and the wiring of a group of wells is shown in fig. 10. A wire should be run from the proper binding posts, marked A, B, C, D, and E, of the battery to the casing of the corresponding wells, and another from the binding posts, marked B′, C′, D′, E′, of the battery to the electrodes of the corresponding wells. The battery is so wired that when the switches A and B are closed, the ampere meter indicates the amount of current flowing through the ground from the casing of well B to the casing of well A. When the switches B and B′ (fig. 10) are closed, the ampere meter indicates the amount of current flowing through the water from the casing of well B to the electrode of well B. After all the wiring is done, the correctness of the various connections should be tested. If the circuit through one of the wells is closed and the electrode lifted out of the well, the amperemeter needle should fall back to zero, for raising the electrode out of the well opens the circuit. On the other hand, if the casing of the well is touched with the electrode, a strong deflection of the needle should be noted. By these tests an error in wiring may be detected.

CHARGING WELL A.

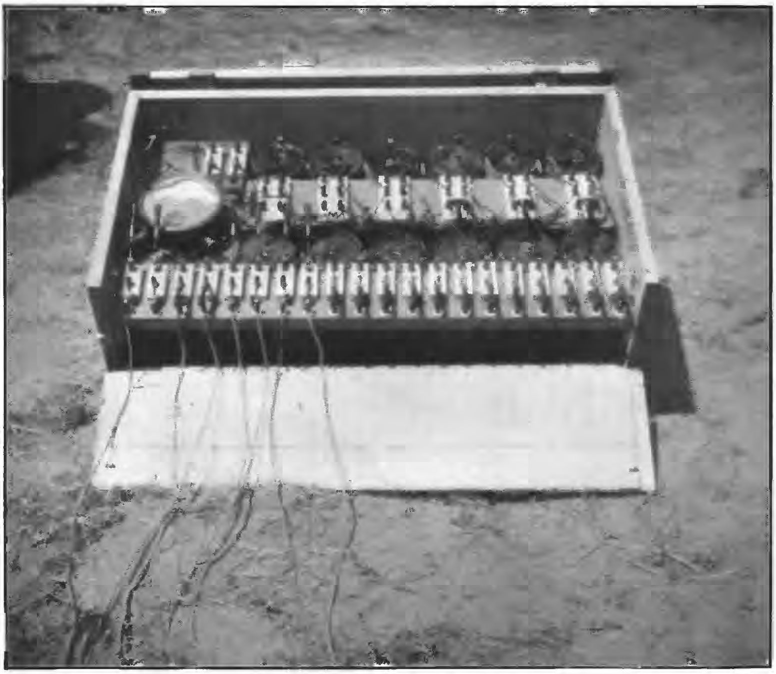
After all the connections have been made and tested, a new chart should be placed in the recording ampere meter, and the apparatus allowed to run for an hour or so, to get a record of the wells before well A is charged.

Various methods of charging wells have been tried, and the following has finally been adopted. The electrolyte used is sal ammoniac (NH_4Cl). This should be placed in a small wooden or paper tub (it rusts iron quickly) and enough water poured on to cover it. A bucket of perforated sheet brass, similar to that used for the inside screen of the well points, about 4 feet long and small enough to readily slip down the inside of the well pipes, should be provided. This should be placed in the tub of wet sal ammoniac and filled with water and the salt by means of a funnel and dipper. The water will drain away into the tub and leave the wet sal ammonia filling the bucket, which should at once be lowered to the bottom of the well.

It is difficult to get a bucket of dry sal ammoniac down a deep well on account of the contained air. In addition, the escaping air causes the formation on the water of a sticky foam, which is carried down by



A. RECORDING AMPERE METER.



B. BATTERY.

the next bucket and clogs the screen. The experiment of pouring a hot saturated solution of sal ammoniac down the well pipe was tried, but was condemned. It will either crystallize out when the water cools and leave the pipe filled solidly with the salt, or, if the strata be pervious, will be forced through to the lower wells almost immediately. In any case, undissolved sal ammoniac should not be poured down a well, as it is almost impossible to clean it out for the next test.

The following rule is now observed in charging wells: Put down one bucket of wet sal ammoniac every fifteen minutes for two hours, then one bucket an hour for six hours. Of course, if at any time the electrical instruments show that the sal ammoniac has reached the lower wells, charging should be stopped at once.

DETERMINATION OF VELOCITY OF UNDERFLOW.

When recording instruments are used, it is only necessary to examine them occasionally to ascertain when the sal ammoniac reaches the lower wells. Its arrival is indicated by an increased deflection of the recording pen, as shown by the chart in Pl. VI, *A*. When the velocity is great the deflection is sudden, but when it is low the increase is not so marked. When there is no underflow the deflection of the pen remains the same throughout the test, as shown in Pl. VI, *B*.

When an ordinary ampere meter is used, the circuits to the various wells must be closed by hand at frequent intervals, varying from ten minutes to two hours, depending on the velocity of underflow. When it is probable that the velocity is great, readings must be made often; but if no flow is detected for two or three hours, the time between observations may be longer. The ampere meter must be read and the time and readings recorded in a notebook. From the notes an ampere curve should be platted (fig. 11), from which an estimate of the velocity of flow can be made. The sudden rise in the curve indicates the arrival of the sal ammoniac at the lower well. This rise is primarily due to the fact that the solution of water and sal ammoniac is a better conductor of electricity than is water alone, and when this solution flows into and mingles with the pure water in the well, the rise in the current, due to lessened resistance, can at once be noted.

The time of passage of the solution from well A to one of the lower wells is usually calculated from the hour when charging was begun to the hour when the rising ampere curve reverses, or, in other words, at the point of the maximum rate of increase in the ampere curve. This method corrects for the diffusion or spread of the sal ammoniac in the water and for the error due to the fact that the water in the upper well can not be charged quickly.

The distance between the wells being known and the time of passage of the solution having been noted, it is a simple matter to compute the velocity or rate of flow. This may be expressed in any unit of measure desired. In this report the rate of flow is expressed in feet per day.

When wells have been sunk with the heavy drivepipe, as described on page 21, and the well screens inserted, the first test will evidently be at bed rock, or at the bottom of the hole. After the completion of

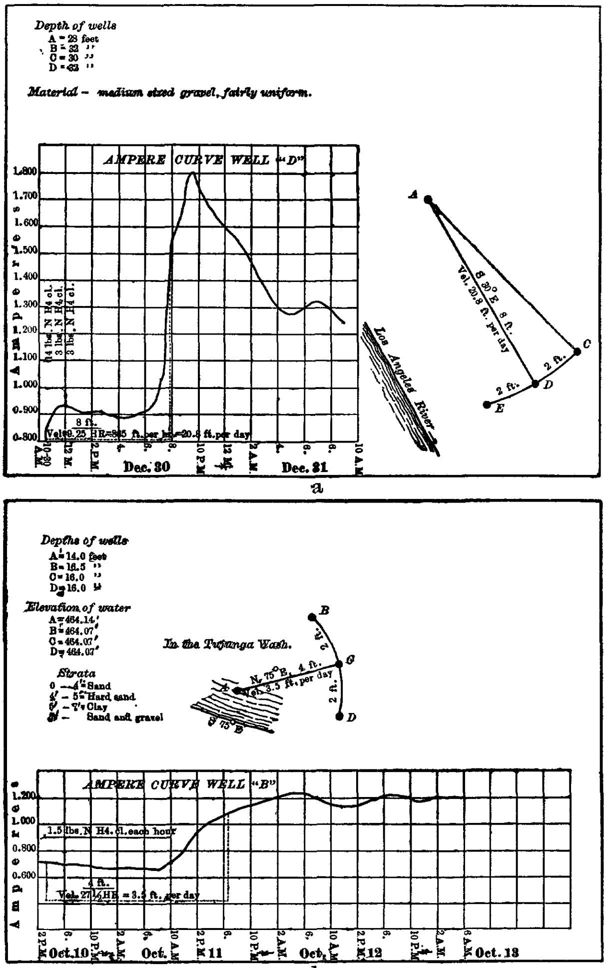


FIG. 11.—Ampere curves.

the first test, the pipe may be pulled up the length of the screens and another test made. This pulling up and testing may be continued to the level of the water table if desirable.

When well points are driven by hand, it is best to drive into the first stratum below the water table, test it, and then drive deeper, continuing the process as far as necessary.

COMPUTATIONS OF DISCHARGE.

In a given section wide differences in the velocity of underflow are to be expected, due to the various factors on which flow depends, such as slope of the water table, porosity, relative size of the sand grains, temperature, etc. It is evident that if the actual percentage of the open spaces or voids between the particles of sand and gravel of each square foot of a measured section was known, and the average velocity at which water flows through the ground had been measured, it would be possible to compute the amount of water passing a given line during a fixed interval of time; moreover, no further data would be necessary.

The difficulties met with when an attempt is made to compute the discharge from a section should be clearly set forth.

Natural sands, as dug from pits or the bed of a stream, consist of grains of different sizes and shapes. Such sand weighs more per unit of volume, as per cubic foot, than any of the various grades of sand sifted from the same bed and having grains of nearly uniform size. This proves that the natural sands contain a less percentage of voids, or, in other words, the porosity is less, than that of the sorted sands.

Experiments made by Prof. F. H. King[a] show that the porosity of unsorted sands from the Tujunga washes in San Fernando Valley, Los Angeles County (see fig. 12), varies from 31.42 to 42.28 per cent. The effective diameter of the sand grains was ascertained to range between 0.05542 and 0.4289 millimeters. The same writer says:

It appears to be generally true that well-rounded grains of nearly uniform diameter tend to give a pore space which lies between 32 and 40 per cent. The mean theoretical pore space for spherical grains of a single size is 36.795 per cent, and this is very close to the mean observed limit for the more simple sands of rounded grains.

A sample of coarse sand from Arkansas River Valley was tested by G. W. Stose in the following manner:

After being thoroughly dried it was saturated with water, and the quantity of water received was determined by weighing. The vessel containing it was then punctured at the bottom, so as to permit the water to drain away, and after a lapse of several days the loss of water was determined by another weighing. It was then found that the sand had received 29 per cent of its volume of water, but afterwards parted with only one-third of the water received, equivalent to 10 per cent of the whole volume of sand.[b]

Both of these samples had been removed from the original beds. The experiments show the probable range of porosity in clean sand of moderately uniform size.

Prof. Charles S. Slichter states that "the porosity of quartz sand will usually vary between 30 and 40 per cent, and that of clay loams

[a] King, F. H., Nineteenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1899, p. 212.
[b] Gilbert, G. K., Seventeenth Ann. Rept. U. S. Geol. Survey, pt. 2, 1896, p. 600.

between 40 and 50 per cent, depending upon the variety of sizes in the mixture and the manner of packing the particles."[a]

Very little is known regarding the porosity of naturally deposited and undisturbed beds composed of coarse gravels, coarse and fine sand, and silt. Theory indicates that such beds, having been deposited by the action of running water, or by the waves and currents of the ocean, are far less porous than sands of uniform size, the sorting action of the water tending to pack the material as closely as possible.

It is usual to assume that the porosity of sand is about 33 per cent, and if the grains are rounded, of nearly uniform size, and free from silt, the assumption is, without doubt, about correct.

The actual porosity in each case must be known before any reliable computation of discharge can be made. Concerning this factor, Professor Slichter says:

If two samples of the same sand are packed, one sample so that its porosity is 26 per cent and the other sample so that its porosity is 47 per cent, the flow through the latter sample will be more than seven times the flow through the former sample. If the two samples of the same sand had been packed so that their porosities had been 30 and 40 per cent, respectively, the flow through the latter sample would have been 2.6 times the flow through the former sample. These facts should make clear the enormous influence of porosity on flow, and the inadequacy of a formula of flow which does not take it into account.[b]

Evidently beds which consist of a mixture of bowlders, cobblestones, fine sand, and silt will have a less percentage of voids than graded sand. No flow can occur through bowlders or cobblestones, and if they make up 50 per cent of the volume, and the intervening spaces are filled with sand and silt of graded sizes, there is certainly far less than 33 per cent of voids in the mass. How much less must be determined by experiment.

It is exceedingly difficult in underflow tests to come to any satisfactory conclusion regarding the porosity of the various beds. This is due to the almost insurmountable difficulties in collecting representative samples of the material penetrated. If the testing is done with a small-size drive pipe, 2 to 4 inches, all the coarse material must be excluded, because it is impossible to get it into the pipe or to the surface. When the material is very coarse, it is obviously necessary to dig a well or shaft, or put down a large bore well, 12 inches or more in diameter. Both methods have decided limitations.

It is impracticable, by any reasonable expenditure of money, to dig a well much below the level of the water table. If a large-bore well is sunk, material below the water table must be brought to the surface in a sand bucket. The continual churning of the sand bucket under water washes the fine silt and mud out of the sand and mixes it with

[a] Slichter, Charles S., The motions of ground water: Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, p. 17.

[b] Slichter, Charles S., Nineteenth Annual Report U. S. Geol. Survey, pt. 2, 1899, p. 323.

the water in the well. This mixture of mud and water is partly bailed out with each bucket partly filled with sand. How much silt is washed out by the water at any particular point is, of course, impossible to say, but if one will catch the muddy water from the sand bucket and let it settle he will be convinced that it is usually no small amount. No one can tell exactly at what depth or how much of the sand contains silt. It is entirely possible that a bed of clay or silt a foot or more thick may be entirely mixed with the water in the well and no evidence of its presence be shown unless the driller notes that the water is more muddy than usual.

The writer has seen samples of sand taken from wells, carefully washed, cleaned from all silt, clay, etc., and then bottled up and exhibited as water sand. Any estimate of discharge based on such samples is utterly misleading.

Samples are often collected by catching the sand, etc., brought up by a hydraulic rig. Of course these samples are thoroughly washed. In porous gravel the wash water used in hydraulicking continually tends to flow away from the well, being under great pressure. That it carries the fine material near the hole away with it is proved by the fact that the sinking of the wash water when drilling in gravel can often be prevented by pouring clay down the well, which soon fills the voids in the surrounding gravel.

The hydraulic and drive-pipe method of sinking test wells has one great advantage over the sand bucket, in that the presence of a bed of clay or silt is shown at once by the muddy water which rises to the surface through the drive pipe. The depth at which such beds occur can by this means be ascertained.

Another uncertain factor is the unknown horizontal extent of the pervious beds. It can only be assumed that they extend from station to station, and in regions where the pervious beds have been deposited by torrential streams this assumption may lead to error, such gravel beds often being in the form of long trains of coarse material, the buried channel of the stream, and not in a wide extended sheet as supposed by many. To correct for this uncertainty, it is obviously necessary to sink test wells at close intervals along the cross section.

There is much uncertainty regarding the weight to be given velocity measurements. As a rule the velocities recorded are the maximums. Suppose test wells are set 4 feet in a pervious stratum, the upper two feet of which is composed of coarse sand 2 millimeters in diameter, the lower two feet composed of fine sand 0.4 millimeter in diameter, with a pressure gradient of 10 feet per mile. As computed from tables prepared by Prof. Charles S. Slichter,[a] the velocity of flow in the upper bed will be at the rate of 5,386 feet per year, and in the

[a] Slichter, Charles S., The motion of underground water: Water-Sup. and Irr. Paper No. 67, U. S. Geol. Survey, 1902, p. 29.

lower bed at the rate of 216 feet per year. In the first instance this is at the rate of 14.75 feet per day, and in the second at the rate of 0.59+ foot per day.

If the test wells are spaced 4 feet apart the chemicals used in charging the upper wells should pass through the coarse stratum and reach the lower well in about six and one-half hours, while it will take about one hundred and sixty-two and five-tenths hours for the same chemical charge to reach the lower well through the fine stratum. In such a case, the salt passing from the upper bed would enter the lower well and cause a sudden rise in the current, as recorded by the instruments. The sal ammoniac in solution, being much heavier than the ground water alone, will settle and fill the lower well. There is no doubt but that the result is correct for the upper 2 feet, but, as the lower well is full of the solution of sal ammoniac, there will be no means of ascertaining when the charge from the lower stratum reached the well, even if the test should be continued for six and three-fourths days, which is highly improbable. It is in fact impossible to know, in testing material naturally deposited, whether the flow is through the whole section tested or through one or two thin strata. The only practical method is to use short well points and test the ground at each few feet in depth.

The most promising field for this method of testing underflow is in regions where the pervious formations are uniform in composition and texture throughout wide areas, and where the water table is not subject to great fluctuations in level. If testing in mountainous regions or in the débris cones of torrential streams is attempted, the testing stations should be close together and tests made at many points and at various depths, to eliminate as much as possible the inaccuracies due to varying texture and porosity of the formations.

BASIN OF LOS ANGELES RIVER.

The drainage basin of Los Angeles River has an area of approximately 502.5 square miles.

The principal eastern tributaries of this stream rise on the west slopes of San Gabriel Mountains and flow down to San Fernando Valley in deep rocky canyons which have been eroded in the granitic rocks of the range (see fig. 12). These streams are torrents during the rainy season, but dwindle to mere rivulets in the summer. When in flood they transport a vast amount of detritus, sand, gravel, and bowlders to the plain below, and have buried the old drainage lines across the east end of San Fernando Valley beneath extensive detritus cones, into which the surface streams sink except in times of extraordinary floods.

The western tributaries of Los Angeles River rise in Santa Susanna Mountains, which bound San Fernando Valley on the north, and in

Santa Monica Mountains on the south. These streams are small, rarely reaching the eastern end of the valley. They flow over sandstones, shales, clays, etc., and are strongly impregnated with alkaline salts, in strong contrast to the pure water from the granitic range to the east.

After flowing southerly in a broad underground channel beneath the detritus in the east end of San Fernando Valley, the water of Los Angeles River is deflected easterly by the impervious rocks of Santa Monica Mountains. This barrier, taken together with the contraction of the underground channel, so obstructs the free percolation below that much of the ground water rises and flows as a surface stream again.

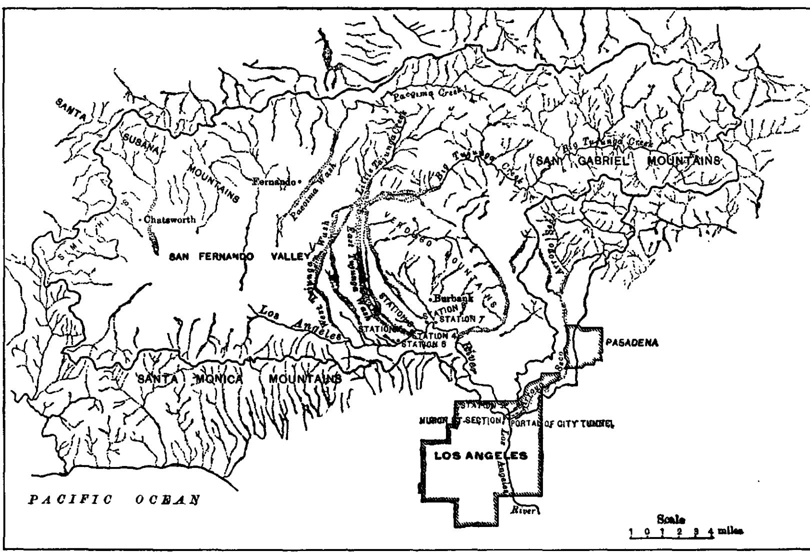


FIG. 12.—Drainage basin of Los Angeles River.

After rounding the eastern end of Santa Monica Mountains the river flows through a region of low, rolling hills in a valley which is about 2½ miles across at the widest place and narrows to half a mile at Los Angeles, 6 miles below. This valley was formerly deeper than at present, and, in common with San Fernando Valley above, has been partially filled with river detritus, which near the mountains is very coarse but becomes fine and contains beds of sand and silt toward the south. The gradual change from coarse to fine material in connection with the contraction of the underground channel without doubt causes the rise of ground water along this 6-mile section of the river. At the mouth of the Arroyo Seco a marked change occurs in the character of the stream detritus. This stream is steep-graded and torrential, heading on the precipitous slopes of San Gabriel Mountains (see fig. 12). After leaving the mountains it flows in a canyon cut across the hill country to its junction with Los Angeles River. It carries bowlders, coarse gravels, and sand into the river valley, and its wide-spreading débris cone at the mouth of the canyon has forced Los Angeles River to the southwest side of the old valley. There is evidence that this débris cone was once more extensive than now, and it without doubt then acted as a barrier to the flow of the river, thus perhaps accounting in a measure for the fine material in the valley immediately above. About 2 miles below Huron street the river flows out on an alluvial plain and the surface stream soon sinks.

Lower Los Angeles River, extending from Huron street 12 miles northwesterly, being supplied by underground water, is remarkably

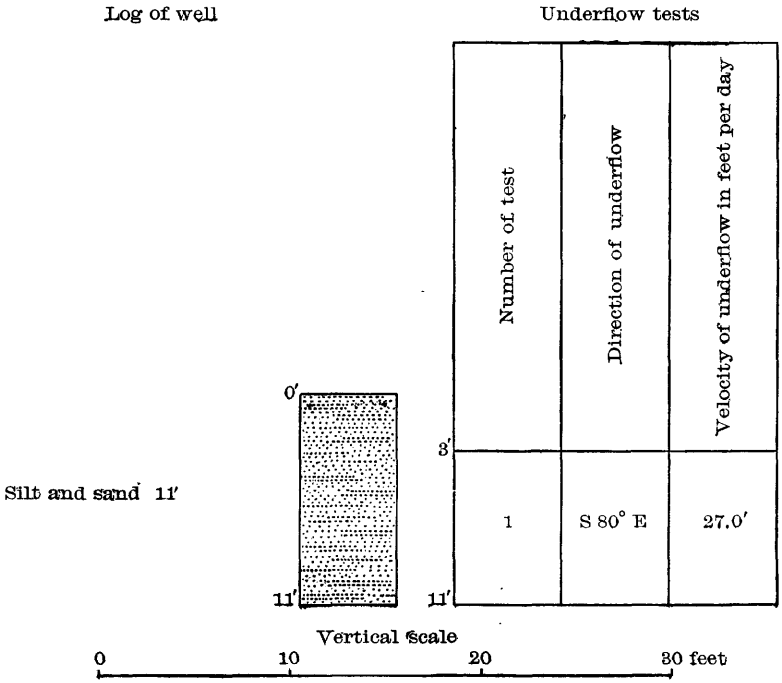


FIG. 13.—Underflow tests at station 1.

uniform in its discharge, little difference being noted throughout the year. The entire surface and part of the underflow is diverted by the city of Los Angeles for domestic use and irrigation.

TESTS IN 1902.

The following preliminary tests were made in Los Angeles River valley during September and October, 1902. The tests made at stations 1 and 2 were mainly for the purpose of adapting the apparatus to local conditions.

STATIONS AND RESULTS.

Station 1 is in Los Angeles River bed, at the Los Felis road bridge, near Huron street (see fig. 21). A group of well points was driven by hand to a depth of 11 feet. The velocity of the underflow was 27 feet per day between 3 feet and 11 feet, and the movement was S. 80° E., or away from the surface stream, which was actually losing water at this point. Later the water table was found to slope in the direction indicated by the underflow. The order of the strata, record of the wells, and the velocity measurements at this station are shown in fig. 13. Sixty-five feet to the east of this station another group of well points was driven to a depth of 11 feet and tested between 3 feet and 11 feet, but no underflow was detected. They were then driven to a depth of 19 feet, and tested between 11 feet and 19 feet, with the same

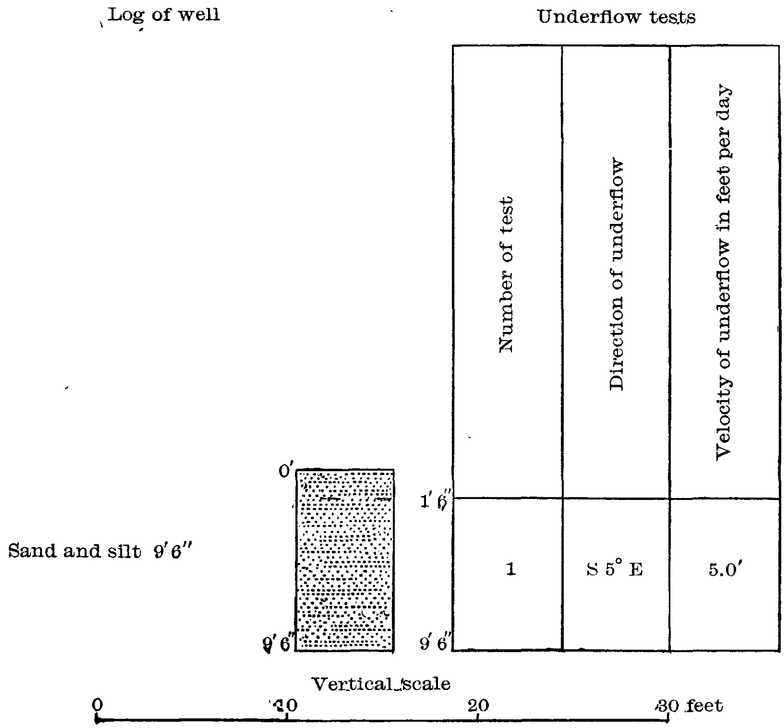


FIG. 14.—Underflow tests at station 2.

result. As these were driven wells no samples were collected, but the material appeared to be fine silt except near the surface, where it was somewhat sandy, and it is probable that the underflow at station 1 was through this top stratum.

In August and September, 1903, the Southern Pacific Railroad Company dug pits for bridge piers in the river bed about 100 feet east of this point. The material penetrated was mainly mud and silt, with a few irregular seams of sand. Only the sand beds were water bearing; they are from 2 to 4 inches thick.

Station 2 is on the west side of the river, 1 mile above the Huron street section, as shown in fig. 12. A group of well points was driven by hand to a depth of 9½ feet, and a test made between 1½ feet and 9½ feet, showing the direction of the underflow to be S. 5° E., velocity 5 feet per day. No samples of the strata pierced were collected, but the amount of water pumped indicated that the ground was not porous. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 14.

The testing apparatus was then moved upstream to San Fernando Valley, where the conditions of underflow and the extent of the ground

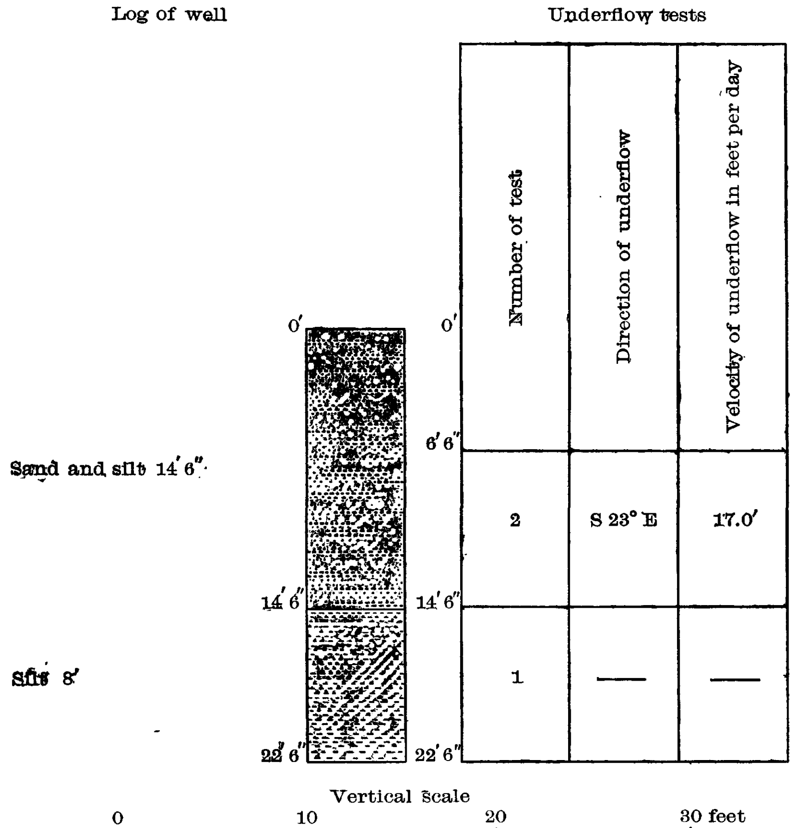


FIG. 15.—Underflow tests at station 3.

water have been thoroughly studied. The object of these tests was to determine, if possible, the relation of the slope of the water table to the direction and velocity of the underflow.

Station 3 is on the east bank of Tujunga Wash, near its junction with Los Angeles River, as shown in fig. 12. A group of well points was driven by hand to a depth of 14½ feet, and a test made between 6½ feet and 14½ feet, where the direction of the underflow was S. 23° E., velocity 17 feet per day. The wells were then driven deeper and a test made between 14½ feet and 22½ feet, but no underflow could be detected. No samples were collected. The underflow appeared to be along the gravel beds in the bottom of the wash.

The order of the strata, record of the wells, and the velocity measurements are shown in fig. 15.

Station 4 is on the east bank of Tujunga Wash, as shown in fig. 12. A group of well points was here driven by hand, and a test made between 6½ feet and 14½ feet, where the direction of the underflow was S. 13° E., velocity 4½ feet per day. These wells were driven

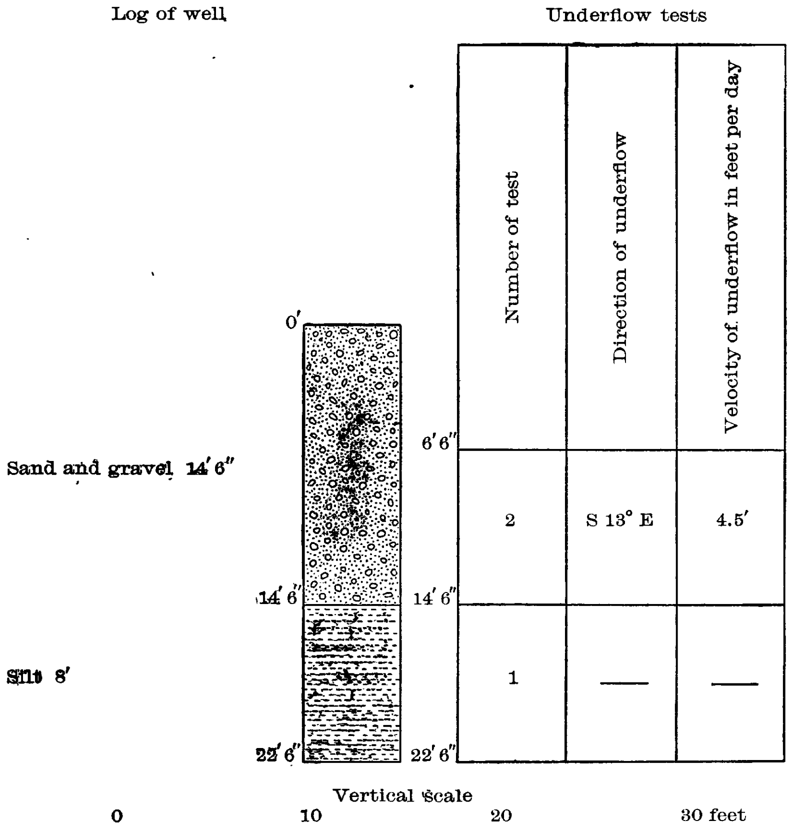


FIG. 16.—Underflow tests at station 4.

deeper, and a test was made as at station 3, but no underflow could be detected. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 16.

Station 5 is on Los Angeles River, near the road from Burbank to Cahuenga Pass, as shown in fig. 12. A group of well points was here driven by hand to a depth of 34½ feet, and a test made between 26½ feet and 34½ feet, where the direction of the underflow was S. 40° E., velocity 48 feet per day. The water from this depth rose to a point 6 feet above the surface, and it is possible that the high velocity was caused by leakage upward around the casing, which drew the charged water toward the well. Borings were made with a 2-inch auger at this station. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 17.

Station 6 is in Tujunga Wash, south of the road from Burbank to Cahuenga Pass, as shown in fig. 12. This group of wells was driven to a depth of 16 feet, and a test made between 8 feet and 16 feet, where the underflow was N. 75° E., velocity 3½ feet per day. Borings were made with a 2-inch auger at this station. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 18.

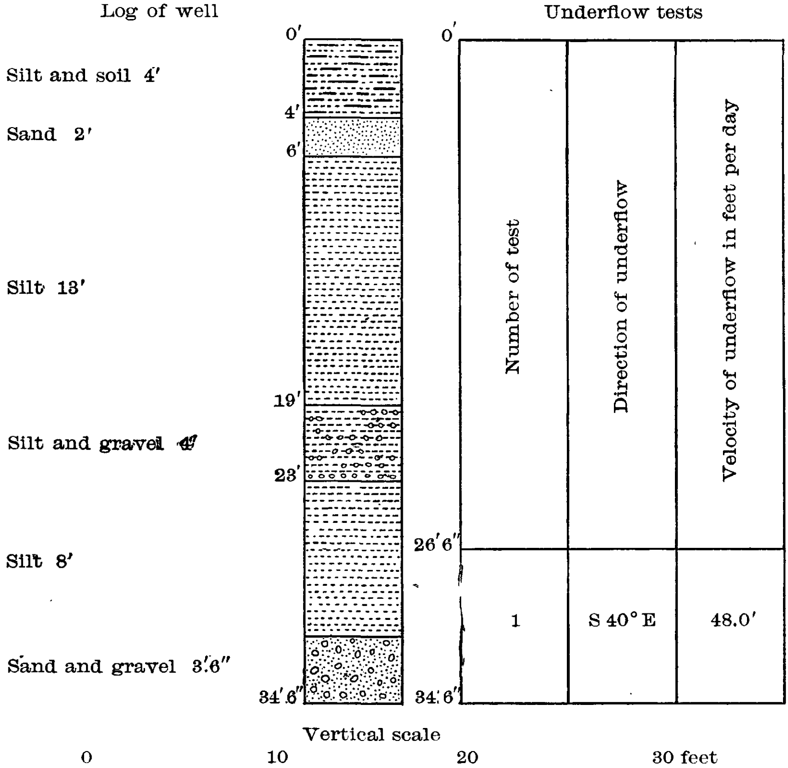


FIG. 17.—Underflow tests at station 5.

Station 7 is on the left bank of Los Angeles River, 1 mile south of Burbank, at the mouth of a broad sandy wash (see fig. 12). This group of wells was driven by hand to a depth of 8 feet, and a test made between 5 feet and 8 feet, where the direction of the underflow was due east, velocity 2½ feet per day. The underflow was away from the surface stream, and the slope of the water table was afterwards found by leveling to be in the direction indicated by the underflow. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 19.

Station 8 is on the left bank of Los Angeles River, 1 mile south of Burbank, at the mouth of a sandy wash (see fig. 12). This group of wells was driven by hand to a depth of 9 feet, and a test made between 1 foot and 9 feet, where the underflow was found to be N. 60° E., velocity 6.4 feet der day. The order of the strata, record of the wells, and the direction of the underflow are shown in fig. 20.

These tests indicate that in general the underflow is in the direction of the greatest slope of the water table, although there are marked variations. No relation could be traced between the slope of the water table and the velocity of the underflow. The conclusion drawn from the few tests made was that the underflow tends to follow the most

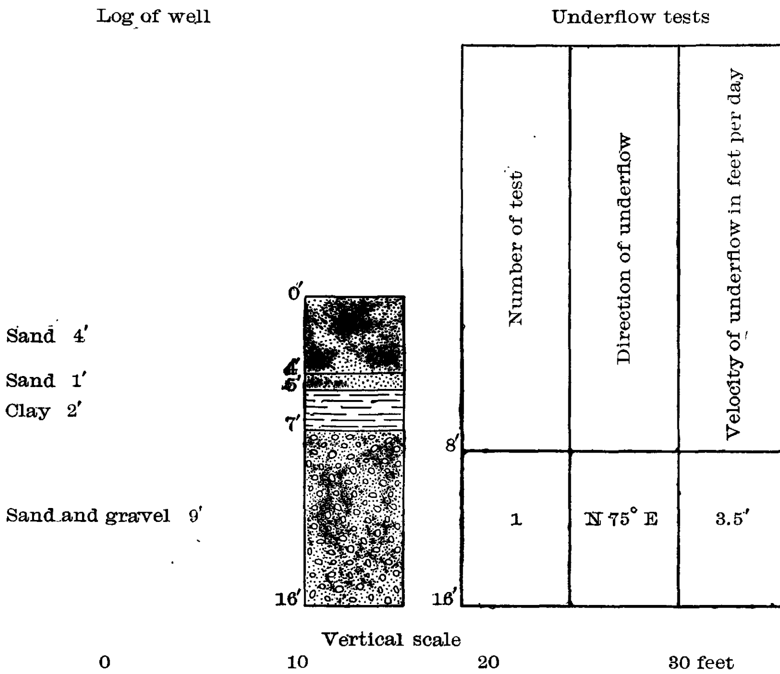


FIG. 18.—Underflow tests at station 6.

pervious strata, even if the extension of these beds varies considerably from the general slope of the water table. The maximum velocities are also found in such strata.

TESTS AT HURON STREET IN 1903.

Extensive tests were made at the narrows of the Los Angeles River along Huron street, Los Angeles, Cal., in 1903 (see fig. 21). This locality will be spoken of as the Huron street section. It was selected for the following reasons:

(1) In 1896 the city drilled several test wells to bed rock, furnishing valuable data regarding the cross section of the pervious gravel beds.

(2) The city water commissioners are now driving a tunnel in bed rock beneath the river channel, beginning about half a mile south of Huron street, to extend entirely across the valley (see fig. 12). It is the intention in the future to drill wells through the pervious gravels and the bed rock above the tunnel and by perforating them draw off the percolating ground water through the tunnel and pump it to the city reservoirs. It was believed that the amount of water intercepted by the tunnel would give an approximate check on the accuracy of the ground-water measurements.

(3) The locality is near shops at which tools and machinery could be repaired, or manufactured if necessary.

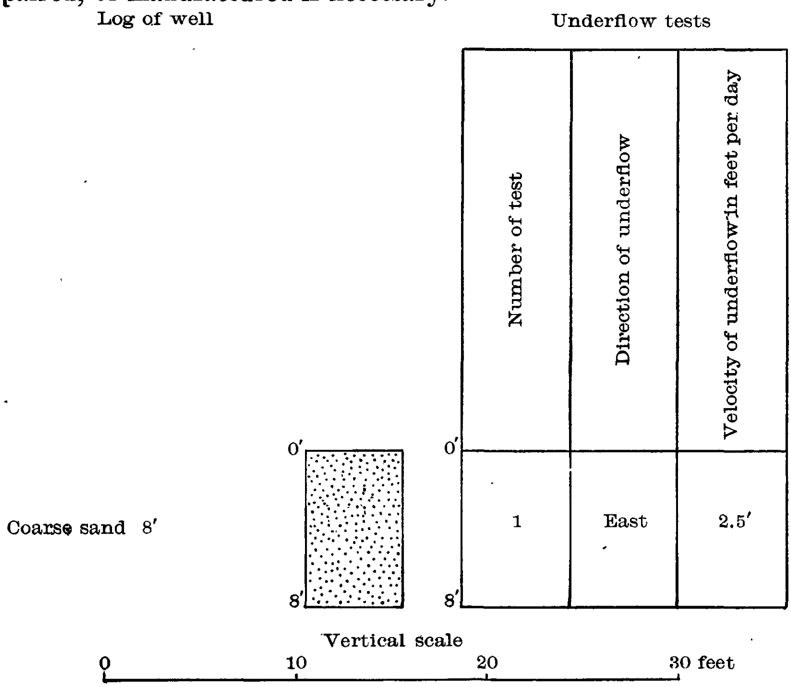


FIG. 19.—Underflow tests at station 7.

The surface profile of Huron street and the cross section of the pervious gravels below are shown in fig. 22. The points marked station 1 and stations 9 to 16 on both map and profile, figs. 21 and 22, represent groups of test wells sunk by the United States Geological Survey. The points numbered 1–7 represent test wells sunk by the city of Los Angeles in 1896, and the wells numbered 8–22 represent house wells in the vicinity. The underground cross section was platted from the information supplied by the various test wells and house wells in the vicinity. This representation of the underground conditions is necessarily incomplete, but no additional information is obtainable at present.

Attention is called to the fact that the pervious beds of sand and gravel in this section (fig. 22) fall into two distinct groups, the lower and most westerly filling the bottom of the old channel of Los Angeles River. The easterly group is without doubt the thinning edge of the Arroyo Seco débris cone. This is proved by the fact that the coarse débris from the Arroyo Seco can be traced directly to this place. The terrace at the east end of the section is also composed of the same material.

The tests of underflow point to the conclusion outlined above; in the lower gravel bed the direction of the underflow is down the general

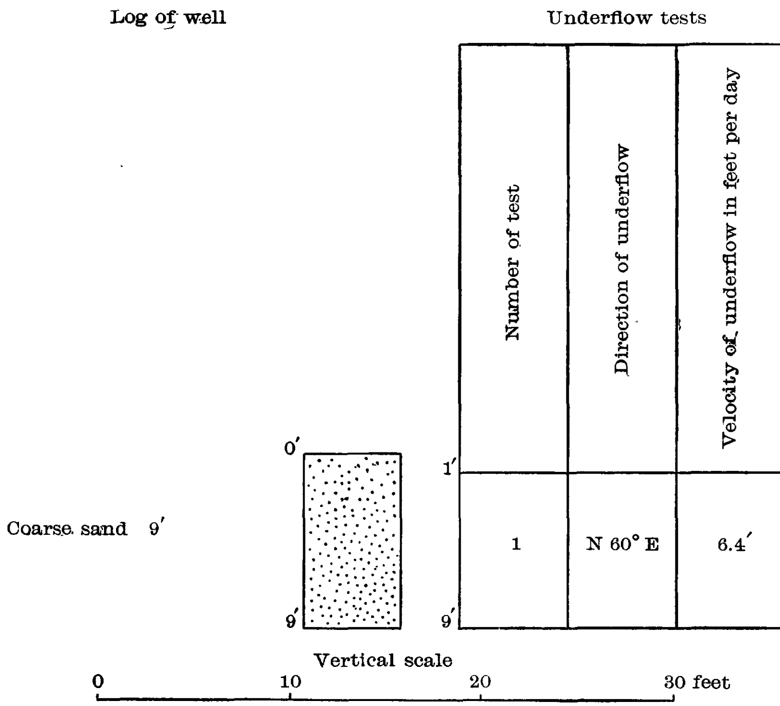


FIG. 20.—Underflow tests at station 8.

trend of the river valley, as shown by the arrows on the map at stations 1 and 9–11; while at stations 12, 13, 14, 15, and 16 the divergence from the trend of the valley is very marked—in fact, at station 15 the flow at present is up Los Angeles River Valley.

It is probable, however, that this reversion of the flow is due to the rise of the water table in the Arroyo Seco débris cone. Normally the underflow is probably from Los Angeles River down the slope of the water table, as indicated by the broken contours in fig. 21, which show the slope of the water table in December, 1902, while the dotted contours show the slope in June, 1903, the change being due, as above noted, to the floods of the Arroyo Seco. The portions of the cross section where measurable velocities were observed are diagonally cross hatched. The rest of the section appears to be practically impervious.

The depths at which the various tests were made and the direction

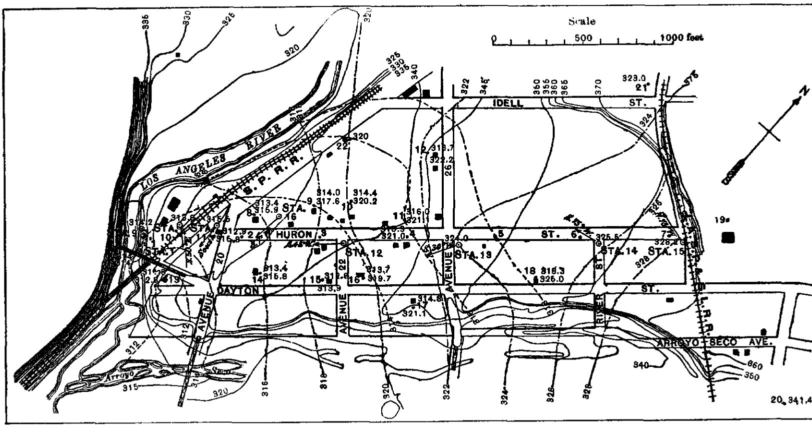


FIG. 21.—Map of Huron street and vicinity. Unbroken lines are surface contours showing topography; broken lines show form of water table in December, 1902; dotted lines show form of water table in June, 1903.

and velocity of the underflow measured are shown in figs. 23–30.

Attention is called to the general agreement in the slope of the water table and direction of the underflow.

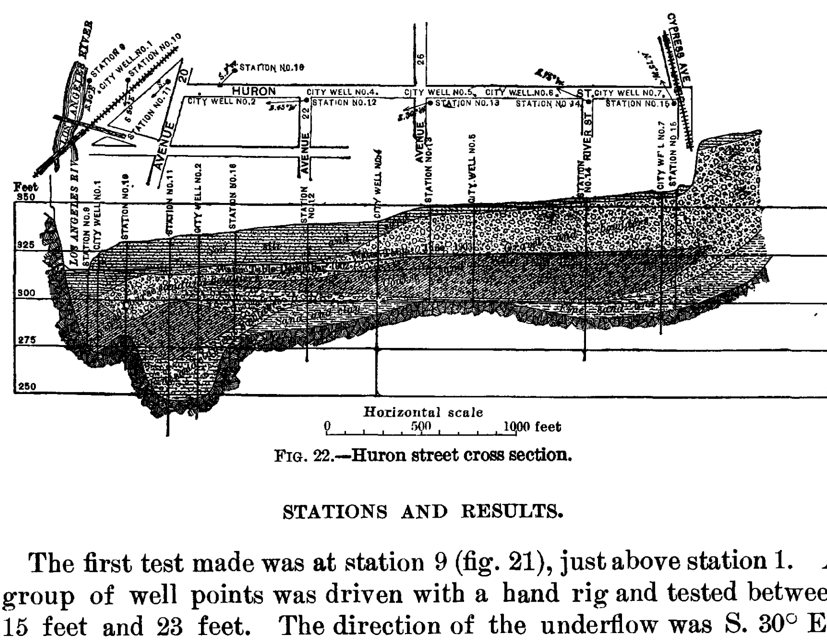


FIG. 22.—Huron street cross section.

STATIONS AND RESULTS.

The first test made was at station 9 (fig. 21), just above station 1. A group of well points was driven with a hand rig and tested between 15 feet and 23 feet. The direction of the underflow was S. 30° E., parallel with the surface stream, velocity 77 feet per day. The well points were then driven deeper and a test made between 22 feet and 30 feet, where the direction of the underflow was S. 30° E., velocity 20.8 feet per day. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 23.

The ampere curve of the second test at this station is shown in fig. 11, a. It illustrates what form of curve may be expected with high

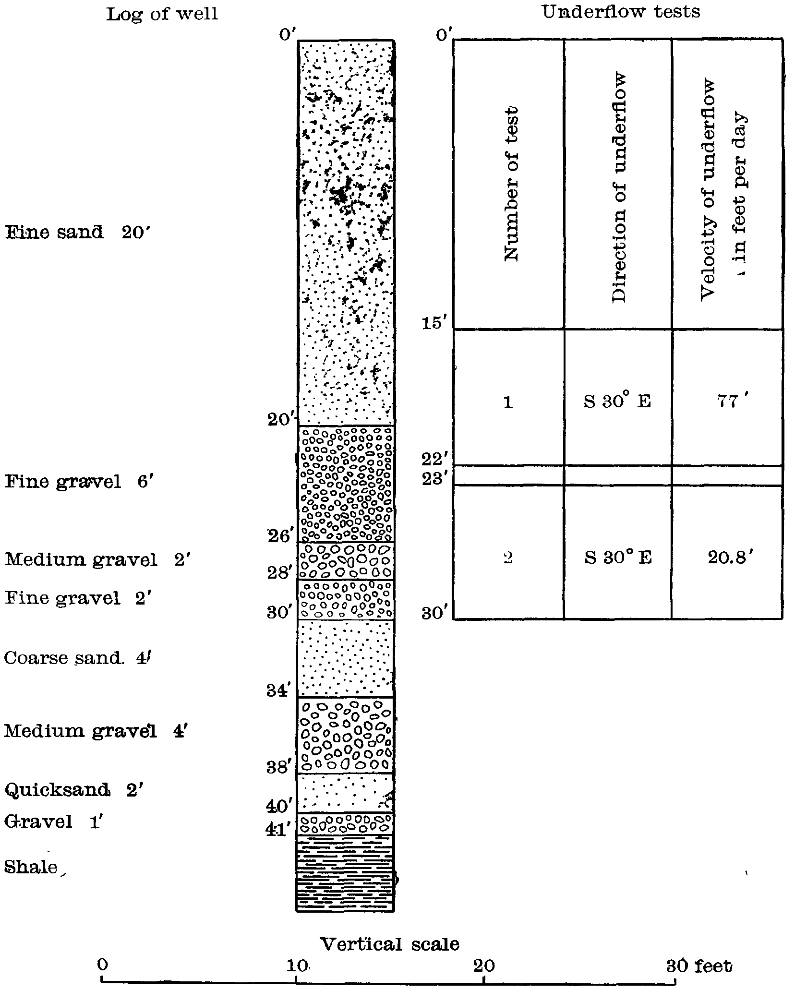


Fig. 23.—Underflow tests at station 9.

velocities and shows that observations at intervals of a few minutes must be taken to correctly delineate such a curve. This is an extreme case, however. A test well was sunk by the hydraulic-jetting method here and the record of the wells is based on the material washed out of the drivepipe. The pipes broke when it was attempted to drive them beyond about 30 feet and no deeper tests were made.

Station 10 is on the left bank of the river, as shown in fig. 21. This group of wells was sunk with a hand rig, using the heavy drive-

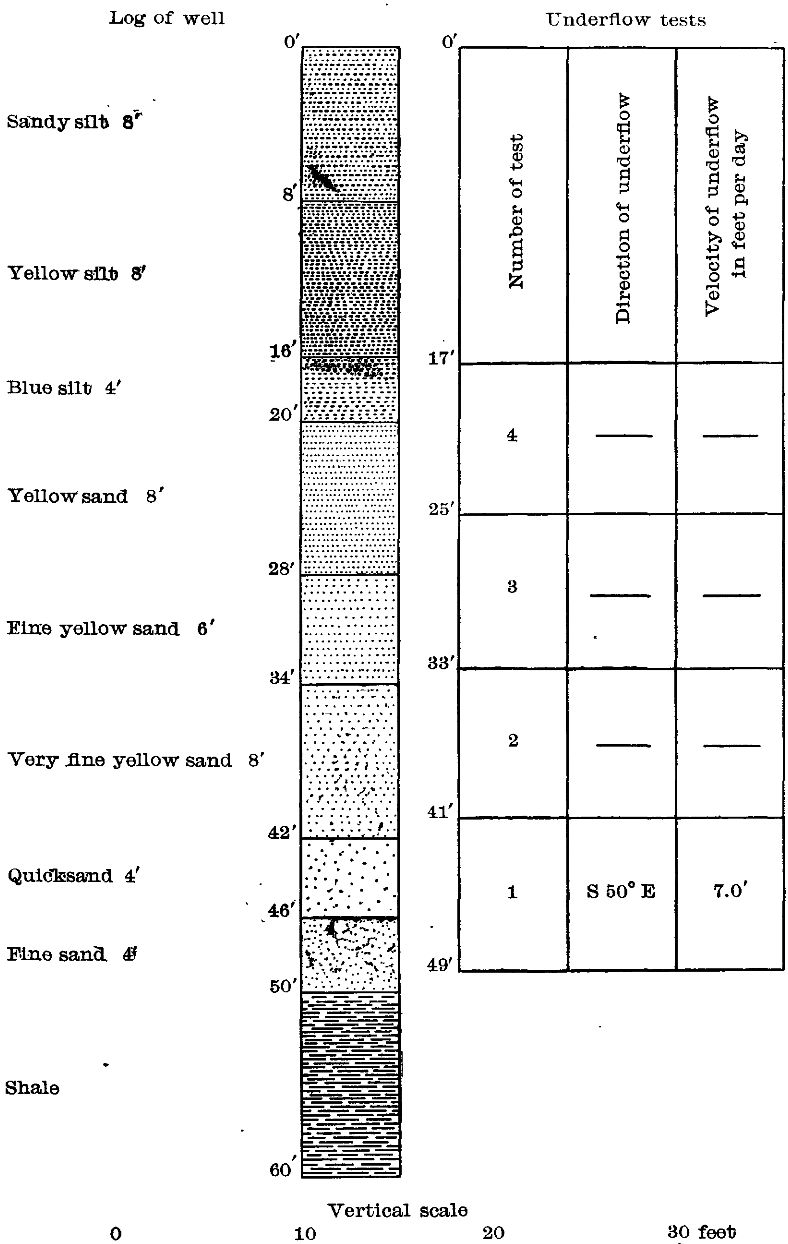
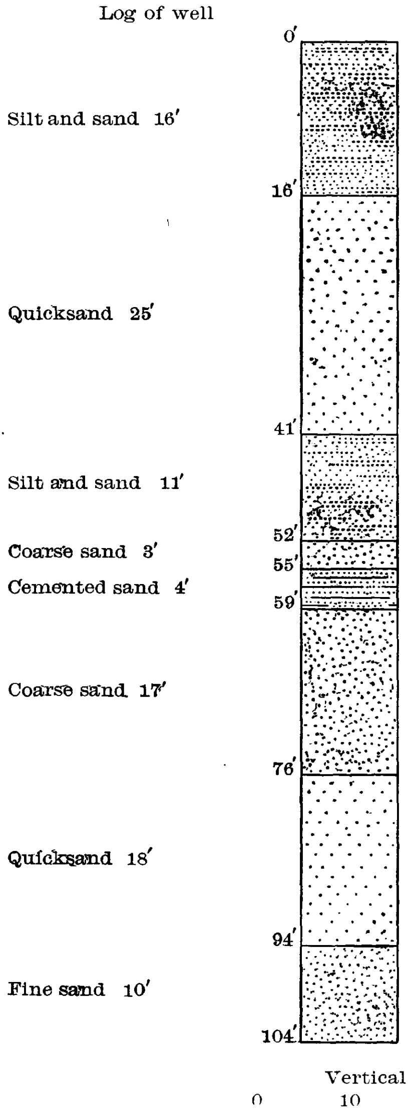


FIG. 24.—Underflow tests at station 10.

pipe described on page 15. The first test was made between 41 feet and 49 feet. The direction of the underflow was S. 50° E., velocity 7 feet per day. It should be noted that the flow was probably through the stratum between 46 feet and 50 feet. Tests were afterwards made betweeen 33 feet and 41 feet, between 25 feet and 33 feet, and between 17 feet and 25 feet. but no underflow was detected. The



Underflow tests

Depth	Number of test	Direction of underflow	Velocity of underflow in feet per day
0′–17′			
17′–25′	11	——	——
25′–33′	10	——	——
33′–41′	9	S 30° W	20.0′
41′–49′	8	S 30° E	16.0′
49′–55′	7	South	10.0′
55′–57′			
57′–63′	6	S 30° E	10.0′
63′–71′	5	——	——
71′–79′	4	——	——
79′–87′	3	——	——
87′–95′	2	——	——
95′–103′	1	——	——

FIG. 25.—Underflow tests at station 11.

material washed out of the drivepipe also indicated that the ground was practically impervious. The order of the strata, record of the wells, and the velocity measurements at this station are shown in fig. 24.

Station 11 is between the Southern Pacific Railroad and Avenue 20, as shown in fig. 21. One well of this group was sunk with the hand-rig and the rest by the well-driving machinery, as described on page 12. The first test was made between 95 feet and 103 feet, but the result was uncertain, the final conclusion being that there was no underflow at this depth. Other tests were made between 87 feet and 95 feet, between 79 feet and 87 feet, between 71 feet and 79 feet, and between 63 feet and 71 feet, but no underflow could be detected. The water from 63 feet to 104 feet was heavily charged with sulphureted hydrogen. A test was then made between 55 feet and 63 feet.

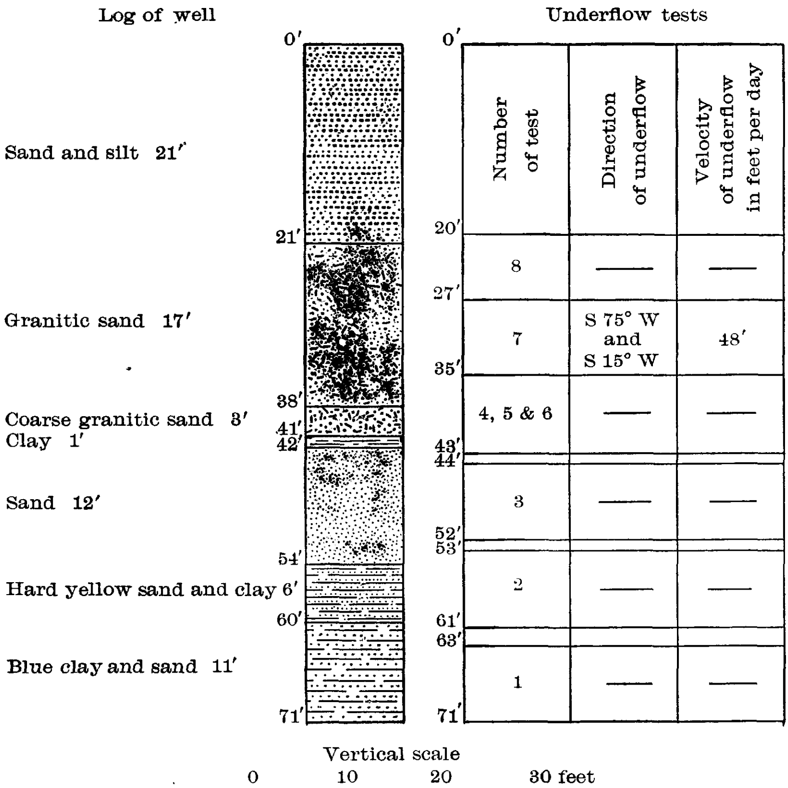


FIG. 26.—Underflow tests at station 12.

The direction of the underflow was S. 30° E., velocity 10 feet per day. Another test was made between 49 feet and 57 feet. The direction of the underflow was due south and the velocity 10 feet per day.

This was followed by a test between 41 feet and 49 feet, where the direction of the underflow was S. 30° E., velocity 16 feet per day. The next test was between 33 feet and 41 feet, where the direction of the underflow was S. 30° W., velocity 20 feet per day. Two more tests were made, one between 25 feet and 33 feet and another between 17 feet and 25 feet, but no underflow was detected. The order of the strata, record of the wells, and the velocity measurements at this station are shown in fig. 25. The wide divergence in the direction of the underflow at this station, amounting to 60°, indicates that the water follows the most open strata.

Station 12 is at the intersection of Huron street and Avenue 22, as shown in fig. 21. Tests were made between 63 feet and 71 feet, between 53 feet and 61 feet, between 44 feet and 52 feet, and between 35 feet and 43 feet, but no underflow was detected. The next test was made between 27 feet and 35 feet, where the direction of the underflow was S. 15° W., velocity 48 feet per day, and also S. 75° W., velocity 48 feet per day, the underflow here apparently being through two porous strata in which the directions of movement diverge 60°. The last

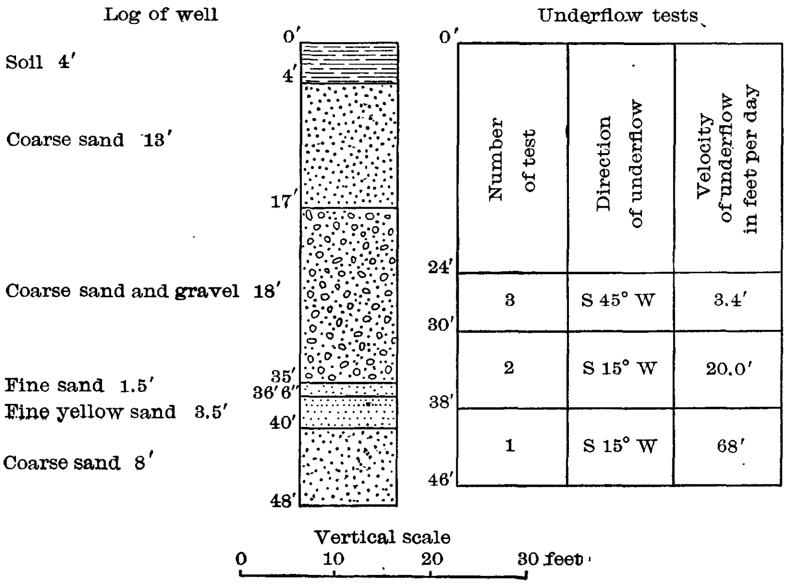


FIG. 27.—Underflow tests at station 13.

test was made between 20 feet and 27 feet, where no underflow was detected. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 26. Below 50 feet the water was impregnated with sulphureted-hydrogen gas.

Station 13 is at the corner of Huron street and Avenue 26, as shown in fig. 21. A test was made between 38 feet and 46 feet, where the direction of the underflow was S. 15° W., velocity 68 feet per day. The next test was between 30 feet and 38 feet, where the direction of the underflow was S. 15° W., and the velocity 20 feet per day. The last test was between 24 feet and 30 feet, where the direction of the underflow was S. 45° W., velocity 3.4 feet per day. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 27. Attention is called to the westerly direction of the underflow, due, without doubt, to the influence of the rising water table.

Station 14 is at the crossing of Huron street and River street, as shown in fig. 21. The first test was between 44 feet and 52 feet, where the direction of the underflow was S. 75° W., velocity 48 feet per day. The next test was between 36 feet and 44 feet, where the direction of the underflow was S. 75° W., velocity 32 feet per day. The

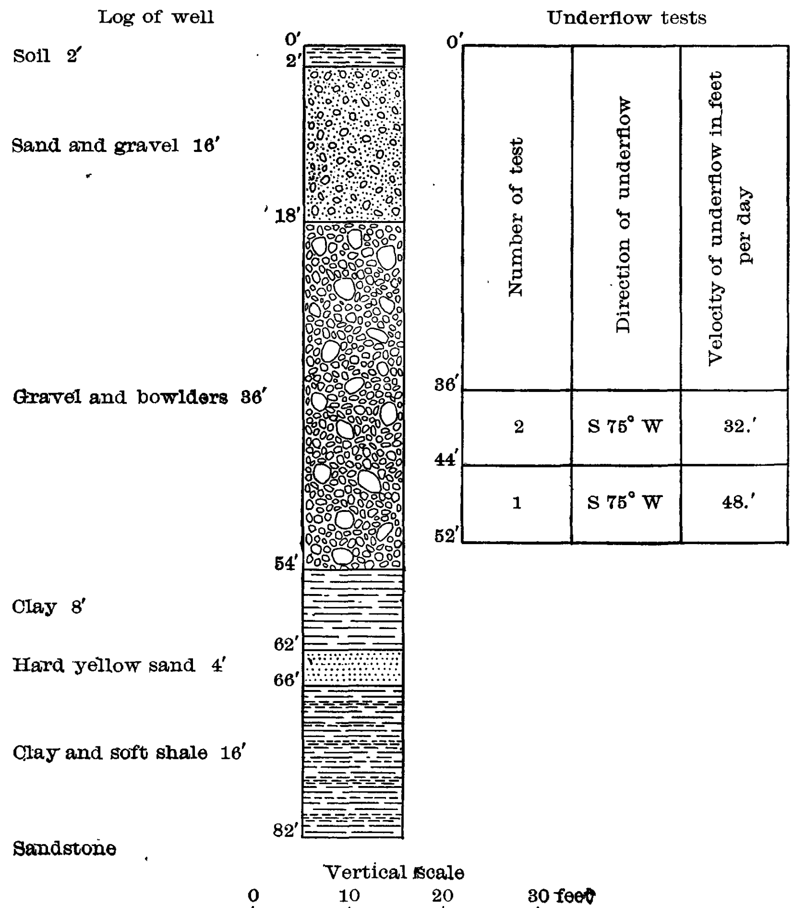


FIG. 28.—Underflow tests at station 14.

top of the well screen being at about the level of the water plane before the rise due to the floods of the Arroyo Seco, it was decided to leave the pipes in the ground and test the direction and velocity of the underflow as the water table gradually falls. That it will fall is proved by the fact that the wells above are now lowering at about 1 foot per week. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 28.

Station 15 is at the eastern termination of Huron street, as shown in fig. 21. The first test was made between 45 feet and 47 feet, but

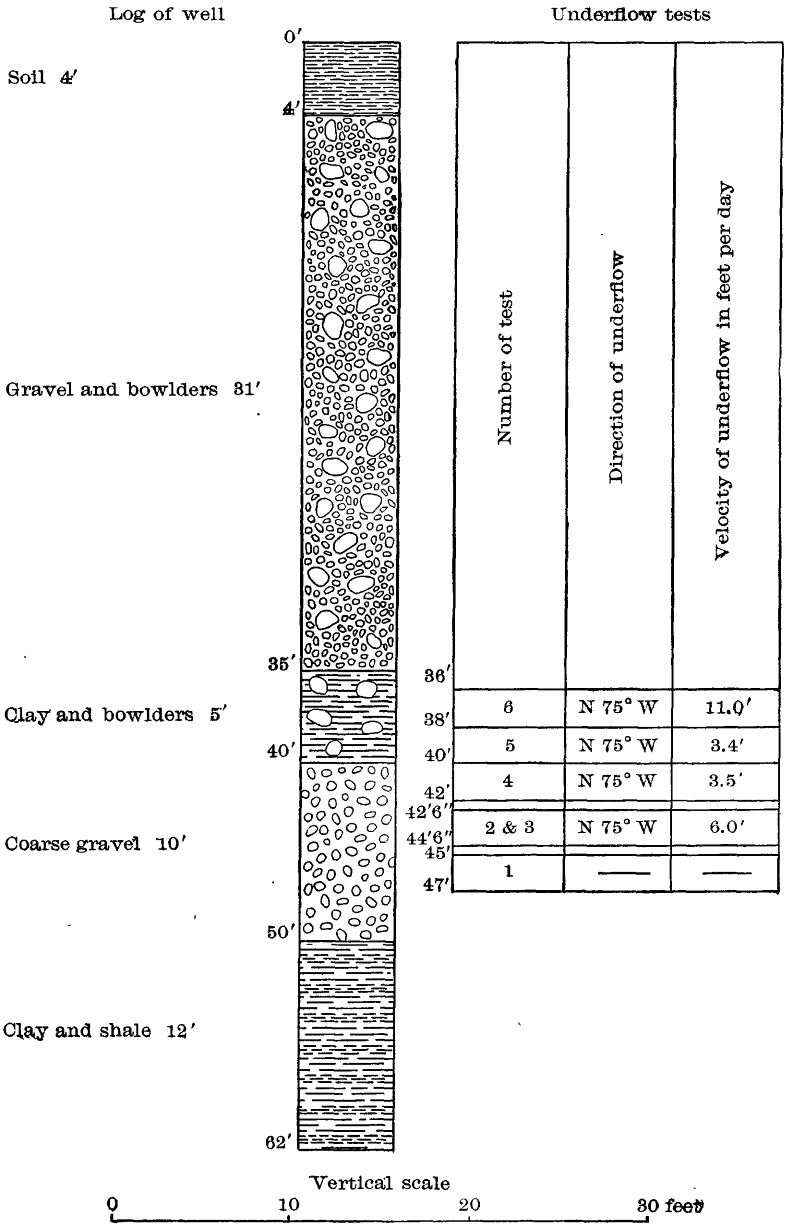


FIG. 29.—Underflow tests at station 15.

no underflow was detected. A test was then made between 42.5 feet and 44.5 feet, where the direction of the underflow was N. 75° W., velocity 6 feet per day. The next test was between 40 feet and 42 feet, where the direction of the underflow was N. 75° W., velocity

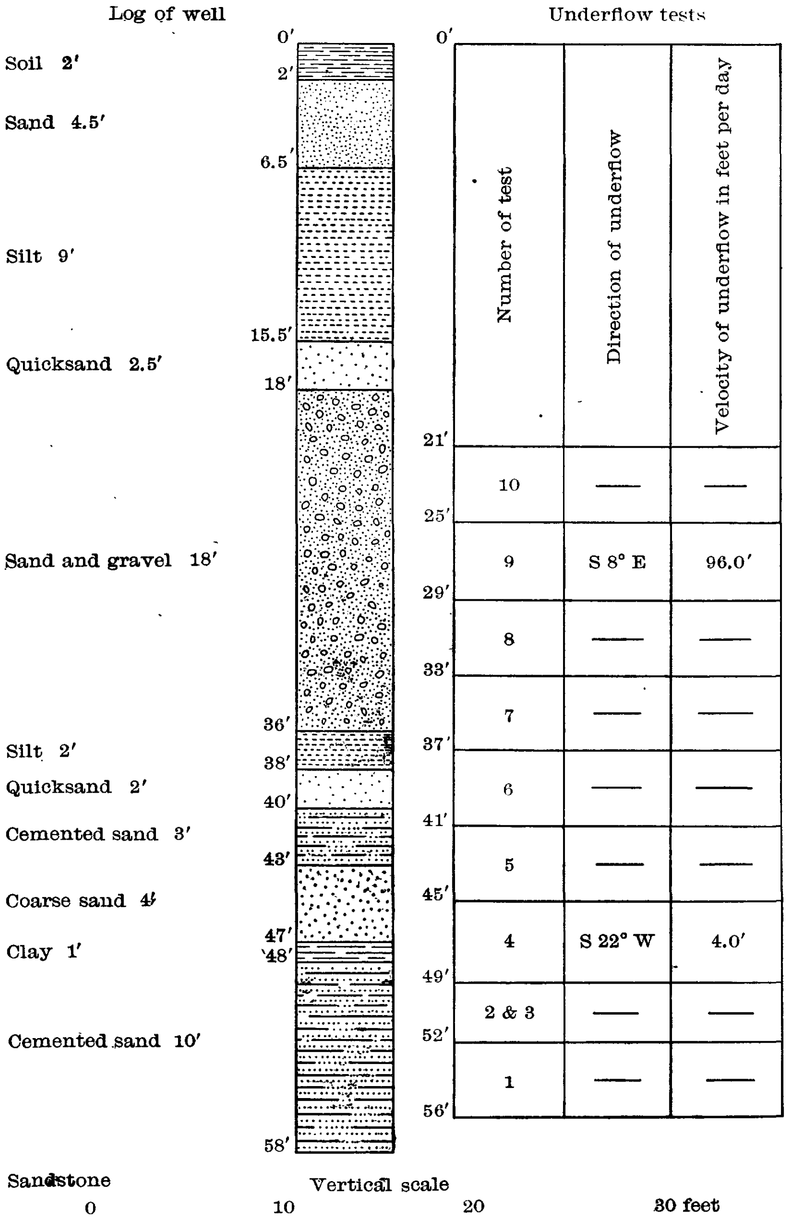


FIG. 30.—Underflow tests at station 16.

3.5 feet per day. Another test was made between 38 feet and 40 feet, where the direction of the underflow was N. 75° W., velocity 3.4 feet per day. The last test was made between 36 feet and 38 feet, where the direction of the underflow was N. 75° W., velocity 11 feet per day. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 29.

Station 16 is north of Huron street, between Avenue 20 and Avenue 22, as shown in fig. 21. Tests were made between 52 feet and 56 feet and between 49 feet and 52 feet, but no underflow was detected. The next test was made between 45 feet and 49 feet, where the underflow was S. 22° W., velocity 4 feet per day. Tests were then made between 41 feet and 45 feet, between 37 feet and 41 feet, between 33 feet and 37 feet, and between 29 feet and 33 feet, but no underflow was detected. A test was then made between 25 feet and 29 feet, where the direction of the underflow was S. 8° E., velocity 96 feet per day. The last test was made between 21 feet and 25 feet, but no underflow was noted here. The order of the strata, record of the wells, and the velocity measurements are shown in fig. 30.

Attention is directed to the probability of this well having penetrated both pervious areas in the Huron street cross section, as shown in fig. 22. All the ground tested here, except the two beds in which underflow was detected, seemed to be close-textured and practically impervious.

DISCHARGE AT HURON STREET.

Owing to the reversal in the normal direction of underflow, by the floods of the Arroyo Seco, it is not possible to estimate, except very approximately, the quantity of ground water passing the Huron street section. The observed facts are here presented, from which engineers may draw their own conclusions regarding the tests and value of the computations submitted.

The western and lower gravel bed, shown on the cross section, fig. 22, has a cross-sectional area of approximately 17,000 square feet, and the upper and eastern gravel bed has a cross-sectional area of about 20,700 square feet below the water plane of 1896. These figures are based on the results of the underflow tests, and on the assumption that all the cross section where flow was not detected has a velocity of underflow of 1 foot per day. The outline of the pervious sections was sketched from station to station.

The average of all the velocities observed in the western gravel bed is 20.6 feet per day, and the porosity is assumed to be 25 per cent. Using this data, the discharge from this portion of the section should be $17,000 \times 20.6 \times 0.25 = 87,550$ cubic feet per day. This discharge should be reduced by about 20 per cent to correct for the obliquity of flow across the section, $87,550 \times 0.8 = 70,040$ cubic feet, or 524,000 gallons per day, in round numbers.

The average of all the velocities observed in the eastern gravel bed is 30.9 feet per day. If it is assumed that under normal conditions the height of the water table is represented by the line marked 1896 in fig. 22, the area of the cross section through which flow takes place will be 30,700 square feet. The slope of the water table was not radically different in 1896 (the assumed normal condition) from what it was in 1903, when the underflow tests were made, but the direction of the flow in the eastern end of the section was deflected nearly or quite 90 degrees from the normal. It is assumed that since the degree of slope of the water table at these two periods is approximately the same, the velocity of underflow was not radically different. Under normal conditions, the direction of flow makes an angle of about 30 degrees with the line of Huron street. The computed discharge has been multiplied by 0.50, the sine of 30 degrees, to correct for the obliquity in the direction of the flow. The porosity is here assumed to be 25 per cent, as before. The discharge from this portion of the section should then be $30{,}700 \times 30.9 \times 0.25 \times 0.5 = 118{,}578$ cubic feet, or 889,000 gallons per day, in round numbers.

Although no flow was detected in the rest of the cross section, between the water table and bed rock, it can not be entirely impervious, and it is assumed that the average velocity is 1 foot per day. This is certainly a maximum, as many tests were continued long enough to detect as low a velocity as this.

If the average porosity is assumed to be 25 per cent as above, and the average direction of underflow is at an angle of 30 degrees with Huron street, the discharge from this section should be $60{,}000 \times 1 \times 0.25 \times 0.50 = 7{,}500$ cubic feet per day, or 56,000 gallons, in round numbers.

The total computed discharge from the Huron street section will then be 196,118 cubic feet per day, or 2.27 cubic feet per second.

The city tunnel has now been driven 1,163 feet under the river bed, one-fourth mile below. This tunnel yields about 7 second-feet when pumped for a short time at intervals of a few days. The supply from the tunnel has so far been drawn from the water stored in the gravels in the immediate vicinity, pumping not having as yet been continued long enough to demonstrate what the gravel beds will yield.

COST OF TESTS AT HURON STREET.

The following table shows the average cost of testing underflow at Los Angeles, Cal., for a period of two months, using the well-drilling machinery and testing apparatus described in this report:

Cost of underflow tests at Huron street, Los Angeles, Cal., from May 14 to July 14, 1903.

	Total expenses for 2 months.	Driving pipe.	Pulling pipe.	Testing.
Well driller	$150	$96	$22	$32
Laborers	100	64	14	22
Cook	70	40	10	20
Observer	150			150
Subsistence	120	50	20	50
Repairs	40	28	6	6
Chemicals	30			30
Fuel	8	6	2	
Supervision	150	60	30	60
Total	818	344	104	370

Pipe driven, 989 feet; cost per foot, $0.348.
Pipe pulled, 929 feet; cost per foot, $0.111.
Total number of underflow tests, 16; cost per test, $23.125.
Total average cost per test, including driving, pulling, etc., $51.125.

The well-drilling machinery described was constructed in Los Angeles from designs furnished the foundrymen and from machinery purchased at local supply houses.

Cost of well-drilling machinery.

Cost of drilling rig in Los Angeles	$870.00
Two hundred linear feet of 3-inch, double, extra-heavy drive pipe, with taper-joint couplings, at $4 per foot	800.00
Total	1,670.00

SUMMARY.

The following suggestions are based on the experience gained during the work at Huron street and in the San Fernando Valley:

(1) The location of the section where it is proposed to test the underflow should be carefully studied. It should be, if possible, in a straight stretch of the valley, and at some distance, either up or down, from large tributary streams.

(2) The form and slope of the water table should be ascertained and the line of test stations placed most advantageously.

(3) In order to secure accurate results, the testing stations should be close together along the line of the section.

(4) The well screens should be short and the ground should be tested at each 2 to 4 feet in depth, down to bed rock when possible.

(5) If possible, the porosity of the pervious beds should be determined.

(6) When making deep tests some form of drive pipe and screen such as described on the preceding pages should be used.

(7) Recording ampere meter and switch clocks should be used.

The discharge from a given section will undoubtedly be far less than anticipated, the popular tendency being to greatly overestimate the amount of underflow. Even if the results attained by this method of testing are not as accurate as desired, they are, nevertheless, of great value, as they enable investigators to compute, approximately, what could only be roughly estimated before.

INDEX.

O

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Reference.

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