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Investigation of vertical internal spillways

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INVESTIGATION OF VERTICAL

INTERNAL SPILLWAYS

BY

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ABSTRACT

This investigation resulted in the development of a means of predicting the discharge of water through an internal spillway type rockfill dam. From the data obtained during the various tests, equations were developed by which the coefficient of discharge for an internal spillway type rockfill dam can be determined when the core height, rock size and head causing flow are known. The equations give results with an error of less than 5%, which is adequate for engineering works.

ACKNOWLEDGMENT

The author wishes to express his sincere appreciation to Professor E. W. Carlton of the Civil Engineering Department for his guidance and counsel in the preparation of this paper.

The author is sincerely grateful to Professor C. D. Muir of the Civil Engineering Department, who was instrumental in the selection of the subject for this study, for his suggestions and help in the interpretation of the results.

TABLE OF CONTENTS

LIST OF ILLUSTRATIONS

LIST OF TABLES

CHAPTER I

INTRODUCTION

Rockfill dams are generally conceded to have had their beginning in California in the days of the Gold Rush. Since that time they have become rather common in occurrence, but due to the high cost of quarrying the rock itself, are seldom employed in areas where concrete or suitable earthfill materials are readily available. Yet today, as automation increases our ability to produce, and produce at lower costs, project sites and materials offer fewer and fewer limitations to construction of any particular type of structure at any particular location. This fact has been demonstrated on several occasions by the construction of concrete dams or rockfill dams in rather remote and practically inaccessible locations. The biggest factors in the elimination of problems of site location and construction materials have been more efficient methods and techniques of construction and the many improvements in design of construction equipment which have created the ability to complete the larger job with more speed and economy. It would perhaps be too positive of a statement to say explicitly where or when to use a rockfill dam, because every dam is an individual problem and should be considered in that light and designed accordingly. The type of material used would naturally be a design consideration, and on many occasions rockfill would be more desirable than concrete or some earthfill material, perhaps because of availability of material or purpose of construction. The number of sites where concrete or masonry dams may be constructed is decreasing, one reason being that foundation problems are more critical in concrete dams than in rockfill dams. Rockfill dams are particularly desirable where flash floods are frequent, where proper equipment and personnel are limited, and where economic limitations likewise might dictate or require this type of fill material. Certain combinations of geologic, topographic, and climatic conditions often found in the arid Southwest and in Mexico generate an ideal potential for flash floods and are the reasons why many rockfill dams are used in those areas.

During the period of construction of most dams, water is diverted or by-passed around or over the immediate area of construction by means of diversion tunnels, open channels, or sluiceways through the construction. The method of diversion, and the capacity or ability to divert water could be a very critical and expensive feature of construction in areas where flash floods occur, as the by-pass tunnels would not normally be designed to carry maximum flood flows. The expense of providing for diversion of maximum floods could make the cost of a project prohibitive, yet if overtopping of construction work occurred where a concrete or masonry dam or earthfill structure was being constructed, a serious problem could develop, one which might not only be costly in the loss of construction equipment, materials and time, but also costly in property damage and human life. In the case of rockfill dams the floods could be safely passed over the fill during construction with a minimum damage, if any, provided the dam is properly designed and protected in an appropriate manner, for it is general knowledge and a widely accepted fact that rockfill dams cannot be overtopped without risk of failure. Protection can be provided by laying a steel mesh mat on the downstream face of the dam, and securing the mesh to the dam itself by means of anchor bars, with hooks or plates on the ends running into the fill, such as have been used in the 'San Ildofonso' Dam in Mexico, located on the Prieto River about 15 miles upstream from the town of San Juan del Rio in the State of Queretaro.

An internal spillway type structure is a relatively new innovation in the field of hydraulic structures, and is unconventional in that the normal spillway structure familiar to most dams is missing. Rather than have the ordinary type of spillway, the rockfill itself serves as the spillway vehicle, the spillway section being the vertical area immediately above the centrally located impermeable core. Having the impermeable core located internally in the dam, rather than as a blanket on the upstream face of the dam might be a disadvantage because of its inaccessibility. However, no very serious problems should develop which could not be solved in a practical manner, and the advantages of such a location would offset any disadvantages. The primary advantage would be in the saving of costs, not only first costs but also maintenance costs, and time, factors which have very high monetary values.

Flows occur over the internal central core because the pool level above the dam is above the crest level of the core, and thus a certain head exists on the core. It is obvious that if flow is to occur over the core, this condition must exist, but this same condition which causes flow to occur from the pool above the dam to the basin below the dam is also associated with considerable energy dissipation during this process of water movement. When flow occurs over the ordinary spillway section, common to most dams, energy dissipation requires construction of baffle and stilling installations at the bottom of the spillway, particularly if the tailwater level (level of the water surface in the basin below the dam) is not high enough to cause a hydraulic jump, a natural phenomenon which is used extensively by engineers to dissipate the kinetic energy of the water at the bottom of a spillway. Considerable problems exist in the maintaining of the baffles and apron on which the baffles are located, but it has been shown in many instances that these energy dissipators are necessary. The internal spillway type structure has a built-in energy dissipator, due to the fact that the dissipation of energy associated with the fall of the water from the pool level to the tailwater level will occur in the rockfill itself without damage to the stability or structural strength of the dam. This feature is very likely the most unique and desirable characteristic of the internal type spillway.

The primary purpose of this study is to endeavor to establish some criteria for the design of internal spillway type rockfill dams. As no design criteria is presently available for such a structure, an attempt will be made to show that a relation exists between the coefficient of discharge, and the height of core and the head on the core; and that there is also a correlation between the size of rock, and the coefficient of discharge for the core. A vertical spillway section was chosen because of the known applicability of the standard weir equation ($Q = 2/3\ CL\ (2g)^{1/2} H^{3/2}$) to flow of water over the core, which in effect could be treated as a rectangular weir. A weir is an obstruction in a flow path, used as a means of metering flow, and which is probably the most used of all measuring devices for measuring flow in open channels. The standard equation for a rectangular suppressed weir was used as a means for comparing the results obtained in this study. This study is essentially a pioneering effort or basic approach to the development of criteria for the design of internal spillways for rockfill dams and certainly there will be many questions which will have to be answered by further investigation and study.

This problem was selected when the author first heard of the lack of information existing on internal spillways. The engineering merit of this subject has been demonstrated by the fact that several structures of this type have been constructed and partially studied in Australia. Any internal spillway type rockfill dam designed to date, has been designed with a general lack of knowledge and experience, and any information developed from this study will add materially to information and experience presently available.

CHAPTER II

REVIEW OF LITERATURE

The internal spillway type rockfill dam probably originated in Australia, for it is there that the only known investigations relative to this subject have taken place. The fact that water flows through a rockfill has long been a matter of knowledge, but the amount that could be safely passed through the fill is a matter of conjecture. "A knowledge of the conditions for maximum safe discharge with accurate means of calculation of its extent could enable rockfill dams to be built without conventional spillways, and would make it possible to forecast the behavior of floods during construction periods over partially completed dams."[1] The interest in this subject by J. F. Wilkins, an Australian engineer, prompted the first and only known investigations in this area. The investigations of the flow of water through rockfills were made on two prototype dams under construction; Laughing Jack Dam, a rockfill dam forty feet high and Wayatinah B Dam, a sixty five foot high rockfill structure, both located on the island of Tasmania off the southern coast of Australia. During construction of the Wayatinah B Dam, two floods passed over the partially completed structure, and although the second was of rather large proportion, the only damage that could be detected was a slight bulging of the rock at the toe of the downstream face of the dam. "The bulging was attributed to insufficient anchorage of steel

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1. WILKINS, J. K., Flow of Water Through Rock Fill and its Application to the Design of Dams (1955) New Zealand Engineering, p. 382-387.

at this point."[1] It is obvious in this type of structure, since overtopping of the dam during construction can be anticipated, that the downstream face of the dam must be protected against floods passing over the rock. In this study the method of protection used in dams found in Mexico, particularly the San Ildefonso Dam, was adopted. The method of protection was in the form of an "exposed mesh of 3/4 inch round reinforcing bars ... the center to center spacing being, for horizontal bars, one foot on centers and for sloping bars, four feet on centers, and wired together at the intersections. This steel mesh was thus anchored to the rockfill by thirteen foot steel bars, with hooks at both ends, spaced three feet vertically and four feet horizontally."[2]

There seems to be no reason why the methods of construction employed in Mexican dams cannot be applied to construction of rockfill dams in the United States. The Mexican rockfill dams are usually constructed in such a manner that during construction if floods occur and overtop the structure, they are not destroyed. The most important feature of these dams is the method of protection used to prevent failure of the downstream slopes. As mentioned above, this protection is an exposed mesh of reinforcing steel which has met with much success in reducing repair costs occurring when a dam is overtopped. Construction methods usually require the construction of costly by-pass tunnels or open channels to divert the maximum flood

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1. WILKINS, J. K., Flow of Water Through Rock Fill and its Application to the Design of Dams (1955) New Zealand Engineering, p. 382-387.

2. WEISS, ANDREW, Construction Technique of Passing Floods over Earth Dams (1951) Transactions, A.S.C.E., Paper 2461, p. 1158-1173.

flows expected to occur during the construction period. With the adoption of the Mexican system of construction, the maximum flood flows could safely be passed through and over a partially completed rockfill dam. Assuming that some part of the maximum flood flow expected would be diverted around the dam, the cost of the diversion works could be reduced by permitting any extreme flood flow to pass through and over the dam.

In 1958 the town of Osceola, Iowa found it necessary to increase the capacity of its water reservoir due to a natural growth in population. They achieved this means by constructing a loose-rock spillway low-head dam, and as no similar structure had been in known use at that time, the design and construction of this dam was completed without benefit of previous experience. The spillway "has a flat crest 100 feet long and sloping crests on each end making a total crest length of 124 feet."[3] The total cost of this spillway was $16,000 as compared to estimates of from $50,000 to $100,000 for the same capacity spillway of conventional concrete. Even though it is different than the internal spillway, the Osceola loose-rock spillway is the first known advance in that direction in the United States, and is an illustration of the fact that rockfill type structures might be very economical in many cases.

Design criteria for rockfill dams is very similar to those for earthfill dams, but there are some features which must have special consideration. One of the most important design problems

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3. BARR, DOUGLAS W. and R. W. ROSENE, Jr., Loose-Rock Spillway for Low-Head Dam (1958) Civil Engineering, Vol. 28, No. 4, p. 50-51.

for rockfill dams is the foundation, which must allow for a minimum of settlement. "It is therefore important that the structure be founded upon rock or upon well consolidated gravels or clays. Materials which tend to flow under load should be avoided as foundations."[4]

Rock used in the dam should be hard, durable, and able to withstand the weathering effects of air and water. Generally the fill material will be composed of quarry-run rock, and will be dumped on the fill without compaction. Dumped fills can lead to very serious problems and could lead to the ultimate failure of the structure, if possible settlement problems are overlooked during the design and construction. Some compaction will be attained through the medium of construction equipment, and high velocity streams from a 'hydraulic giant' sluicing the material over the fill can also help accomplish compaction, but the biggest compaction will come as the headwater rises behind the dam. "It is often desirable to sluice each layer during placement with water from high velocity nozzles, using a volume of water equal to two or three times the volume of rock. Sluicing of quarry-run rock provides point bearing of the larger sizes, as all smaller sizes are washed into the voids. This will provide a dense fill and minimize future settlement. Sometimes gravel is sluiced into the rockfill as layers are placed."[5] Rockfill dams achieve their watertightness by means of an impermeable membrane, generally constructed of concrete and laid in the form of a blanket

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4. Natural Resources Committee, Low Dams (1938), p. 153-154.
5. U. S. Department of the Interior, Bureau of Reclamation, Design of Small Dams (1960), p. 229.

on the upstream face of the dam. It is therefore obvious that only minor settlement or consolidation of the rock can be tolerated upon filling the reservoir. In most conventional dams, where an earth, concrete, steel or timber blanket rests on the upstream face, the settlement which occurs through compaction of the rock may cause the blanket to crack and possibly fail if this problem has not been considered.

The crest width of a rockfill dam is generally governed by height, but should be a minimum of ten to fourteen feet unless a roadway requires a greater width. "Side slopes for rockfills on the downstream side should have a minimum steepness of from 1 1/4 (horizontal) to 1 (vertical) to 1 1/2 to 1. Side slopes of 3/4 to 1 on upstream slopes are not uncommon where provision is made to prevent raveling and in some cases where a timber or concrete deck is used as a flow retarding element."[6] In the case of the internal spillway where no impermeable membrane is laid on the upstream face, the slopes of the upstream and downstream faces of the dam are governed by the type of equipment used and the economical problem of using more fill material. It would certainly be advantageous to maintain slopes as close to the angle of repose of the dumped rock as possible from an economical point of view, as steeper slopes will require less rock than flatter slopes.

The large number of dams found on natural water streams offer an excellent opportunity to measure the flow of water in the stream.

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6. Natural Resources Committee, Low Dams (1938), p. 154.

The coefficient of discharge for the dam must be determined first, in order that one of the many available flow formulas can be used. There are any number of weir formulas, such as the Bazin, Fteley and Stearns, or Smith and Francis, which could be used for determining the discharge, and it is impossible to say that any one of them is better than the others for general use. All of the formulas tend to approximate the general weir equation which is of the form, $Q = 2/3\ Cb\ (2g)^{1/2}\ H^{3/2}$, where b is the crest length, C the coefficient of discharge of the weir, H is the head over the crest of the weir, and g is the acceleration due to gravity, generally taken as 32.2 $\text{ft.}/\overline{\text{sec.}}^2$ Although g varies slightly with latitude and altitude, 32.2 $\text{ft.}/\overline{\text{sec.}}^2$ is sufficiently accurate for all hydraulic computations. The general weir formula was therefore used in this study because it closely approximates the spillway section under investigation.

The internal spillway with a centrally located impermeable vertical core, approaches the condition of a rectangular suppressed weir. Although there are many empirical formulas and any number of available coefficients of discharge to use in the formulas, there is no known relation existing for the variation of the discharge coefficient with rock size or weir height for flow through rockfill. An attempt has been made in this study to establish such a relation.

CHAPTER III

GENERAL DISCUSSION OF PROBLEM

The problem undertaken in this study was an attempt to establish a relation between the coefficient of discharge for an internal spillway type rockfill dam and the size of rock used in the dam, the head causing flow, and the core height. At the beginning of the project, a study of existing rockfill dams was made to ascertain the extent of the apparatus required for performing tests. A rather incomplete model study was initiated to determine the standard section, size of rock, head on the dam, and other information pertinent to the construction of the model. It was decided for the preliminary model to use only geometric similarity with a length scale ratio of 1/20. For the actual study a larger model was desired, but the scale ratio of 1:20 was retained due to space limitations and lack of larger collection and weighing equipment. The prototype dam investigated was five feet high, had a crest width of ten feet with twenty to thirty inch diameter rock used in the rock fill. A one foot crest length was selected for the channel section in the model, so as to be able to obtain fairly high flow rates and heads within the range of the length scale ratio. A wider channel section was eliminated since collecting and weighing equipment sufficiently large to handle the corresponding flows was unavailable, and a thinner section was eliminated to ease the handling of rock and construction of the model dam.

The rock selected for use was a crushed limestone rock quarried in the Springfield, Missouri area. The primary reason for its selection was immediate availability, as this rock was used in the materials testing laboratory and a stockpile was on hand. The results obtained from using crushed rock as opposed to 'pit-run' or other material would be practically the same. An important item to be considered is the shape of the rock, and all crushed rock would have the same general form. It was decided to use one size of rock for the tests with five different core heights, and the rock selected was 1 inch to 1½ inch in size.

This would approximate a rock size of twenty to thirty inches in diameter in the prototype and seemed to be a reasonable size to use. Cognizant of the fact that larger sizes of rock are sometimes used in the prototype, larger rock was used in a subsequent test to provide a comparison for such a condition. In trying to determine the effect of rock size on the discharge, four sizes of rock were used, ranging in diameter from 1/2 inch to 4 inches. It was thus felt that the comparison of extreme conditions would be studied.

Several tests were performed to determine specifically the effect of core height and rock size on the discharge of water through rockfills. Each core was first calibrated without rock to determine the coefficient of discharge of the core, rockfill was then placed and the coefficient of discharge again determined. The effect of rock size was determined in a like manner, using one core height and four sizes of rock.

Originally, it had been intended to conduct several other tests, but due to the added amount of time required to collect such additional data, it was decided to omit them. One of the tests omitted would have been the determination of the variation of the coefficient of discharge with the slope of the core. The idea of using a sloping core probably comes from the use of an impermeable blanket laid on the upstream face of the dam. In most conventional rockfill dams, an impermeable blanket is laid on the upstream face of the dam as a flow retarding element to provide a water tight structure. In some rockfill dams a sloping core has been used, located internally, but generally in close proximity to the upstream face. As far as the author has been able to determine, the slopes of practically all internally located cores were the same as the upstream face of the dam. It is obvious that changing the slope of the core will have the effect of changing the coefficient of discharge and therefore the amount of discharge over the core, since the shape of the streamlines are thus altered. If the slope of the core is increased downstream, the effect is to increase the flow rate, whereas increasing the slope upstream will have the reverse effect of decreasing the flow rate. It is a likely perception that there might be a particular slope and location of the core which would give the most efficient coefficient of discharge. The larger the discharge coefficient, the greater the flow rate will be, and the most efficient core slope will be the one which has the highest coefficient of discharge.

Another test originally intended to be conducted was the determination of the effect of changing downstream and upstream face slopes of the dam on the flow rate through the rock. Generally the slopes both upstream and downstream are kept as close to the maximum as possible, the maximum usually being close to the angle of repose of the dumped rock. Naturally several factors require that the slopes be flatter than the maximum, one being the type of equipment used in construction; another when an impermeable membrane is laid on the upstream face. When the upstream face of the dam is paved with concrete or various asphaltic mixes, the slope is usually made flatter in order to facilitate paving operations. It seems likely that whenever the slopes become flatter, especially the upstream slope, that the flow streamlines will change their pattern and when this occurs the discharge through the rock will be altered. To widen the crest would have a similar effect, as the streamline in the surface of flow would drop farther in passing through the rock; in other words, there would be a greater head loss through the fill. Any assumptions or explanations relevant to the above statements should be verified by investigation. Time did not permit such investigations in this study.

CHAPTER IV

TESTING APPARATUS

The tests conducted in this study were performed with the aid of a flume twelve feet long, one foot wide, and two and one-half feet high designed and built by the author. These dimensions were chosen after selecting a prototype dam, and a suitable length scale ratio to provide for a model which would best fit the space available. The prototype structure under investigation was 50 feet high, had a crest width of 10 feet, and had variable side slopes. A flume length of 12 feet would actually permit the use of slopes approximately three horizontal to one vertical. As the impermeable membrane was in the form of a centrally located vertical core wall 40 feet high in the prototype, the maximum head of water above the core wall of the dam would be 10 feet. Therefore heads in the magnitude of 10 feet above the core wall in the prototype are of interest in this study.

It is obvious that a weir of the size of the spillway section is capable of discharging a fairly high rate of flow, and that rather high velocities and considerable turbulence would be associated with the flow. It was therefore necessary to construct a stilling basin capable of handling the anticipated flows. The stilling basin was constructed from a tank 23 inches high, 29 inches wide and 69 inches long as shown in Figure 1. A section one foot wide and the full width of the tank was cut in one end, and a flexible connection was made to allow for securing the flume to the stilling tank (see Figure 2). Across the center of the tank was installed a baffling device, constructed of

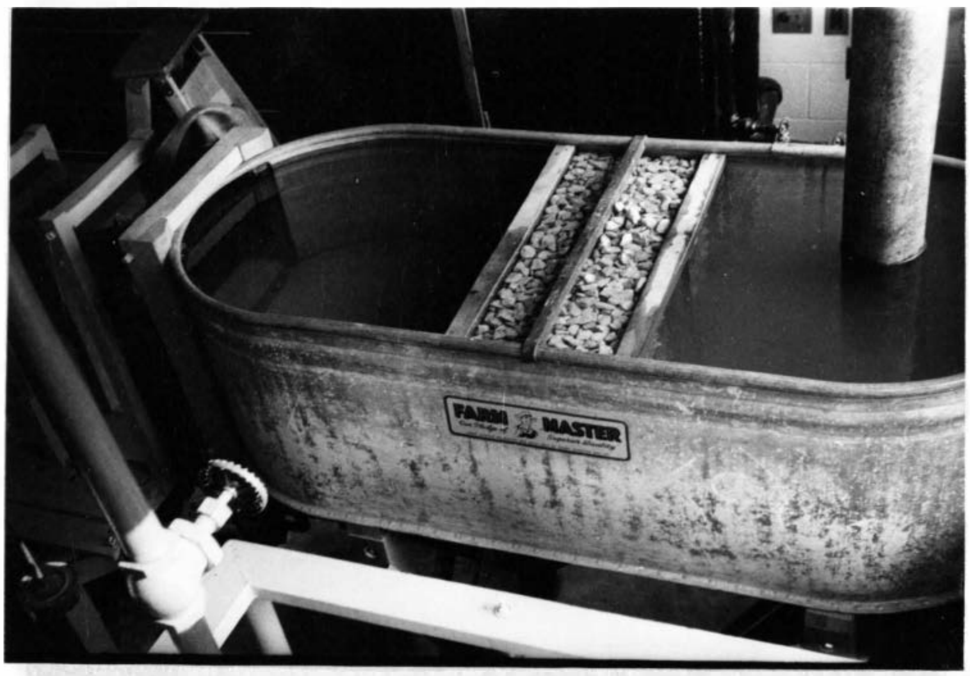


Figure 1

Stilling Basin showing Baffle

Figure 2

Flexible Connection Between Flume and Stilling Basin

3/4 inch to 1 inch rock to eliminate the turbulence associated with high rates of discharge. A drop of approximately one foot occurred at the connection of the stilling basin to the flume, and although this presented no problem when the higher cores were used, considerable turbulence existed when using the 12 inch and 15 inch cores. Therefore, another baffle was installed at this location, this baffle being an aluminum honeycomb sheet 1½ inches in thickness. The baffles in no way hindered velocity of approach, but they were a very effective means of reducing turbulence. The elimination or reduction to a minimum of turbulence was critical in this study because excessive turbulence could lead to erroneous readings of water head over the core during the various tests.

The first eight feet of the flume was constructed of ¼ inch clear plexiglas, while the last four feet was constructed of 3/8 inch marine plywood (see Figure 3). The floor of the flume was likewise constructed of 3/8 inch marine plywood, laid on a base of 3/4 inch exterior plywood. The flume was supported at three foot intervals by frames built of 4 x 4 posts cross-braced. Piezometer connections were made in the floor of the flume and spaced at different intervals along the floor. These piezometer tubes were ¼ inch plastic tubes which were connected to ½ inch acrylic tubes mounted as seen in Figure 4. The values of the piezometer readings during any test gave the information required to plot the hydraulic gradient for any flow. A dye was used in the piezometer tubes to allow for easier reading. Readings were taken to the 0.01 of an inch by means of a scale erected immediately behind the tubes as seen in Figure 3. The scale was leveled with the aid of an engineer's level.

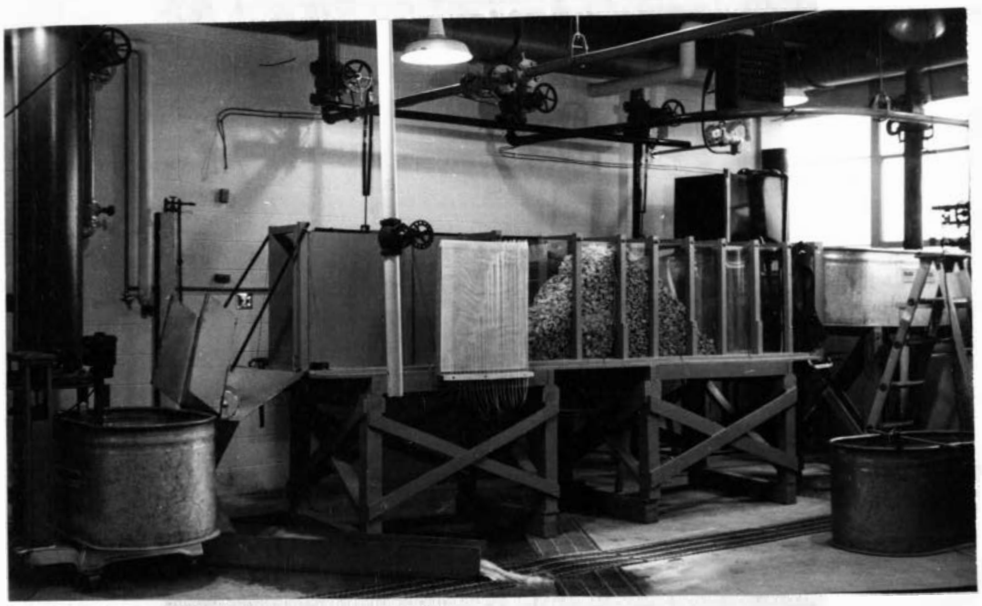


Figure 3

The Testing Apparatus

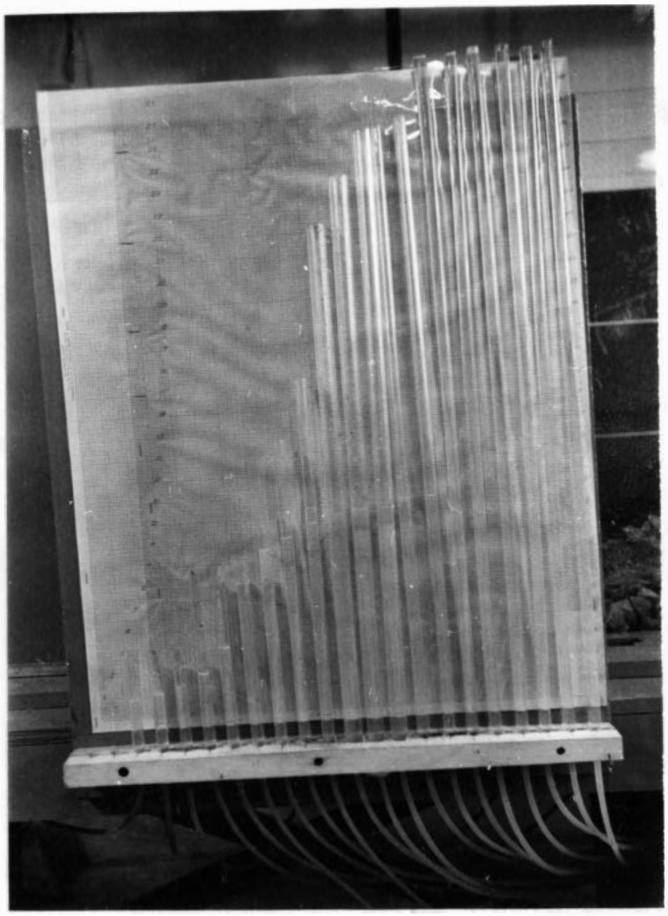


Figure 4

Piezometer Tubes for Determining Head

The water supply was obtained from a five inch pipe connected directly to a 12 inch feeder line and a gate valve in the line was used to control the flow rate. Upon leaving the flume, the water was directed into a line by-passing the weighing tank. When the by-pass scoop was closed, water was directed into the weighing tank, which was supported on a 1000 pound capacity scale as shown in Figures 5-7.

Figure 5

By-pass Scoop Open

Figure 6

By-pass Scoop Closed; Collecting a Volume of Water

Figure 7

Timing the Collection of a Sample

CHAPTER V

TEST PROCEDURE

Previous to the performance of the tests the equipment to be used was first calibrated. The scales used for weighing resultant flow quantities were calibrated by taking known weights, putting the weights on the scale, and noting the difference between the actual weight and observed weight. The scale used required no adjustment or correction as the variation in observed and actual weights was never greater than one pound over the entire range of weights necessary in the tests. The size of acrylic tubing used for piezometers was selected so that the effects of surface tension on the readings would be negligible. An error of 0.001 of a foot in the reading of the piezometer tubes could result from neglecting the effects of surface tension, and as the piezometer readings were taken to the nearest 0.01 of an inch, the error caused by surface tension effects on the piezometer readings were regarded as being insignificant. The effect of an error of 0.01 inch in a piezometer reading would result in a maximum error of less than 1% in the final calculated rate of discharge.

Two general tests were performed in this study; the calibration of the core with no rock in the flume, and the calibration of the core with a rockfill dam present (see Figures 8 and 9). The data collected in both types of tests was the same. The cores used were sawed from 3/8 inch marine plywood, and a metal cap was shaped to fit over the top of the core. This provided a core crest of approximately the same shape and the same characteristics for each core, and protected the wood crest of the core from disfiguration or damage. The cores were

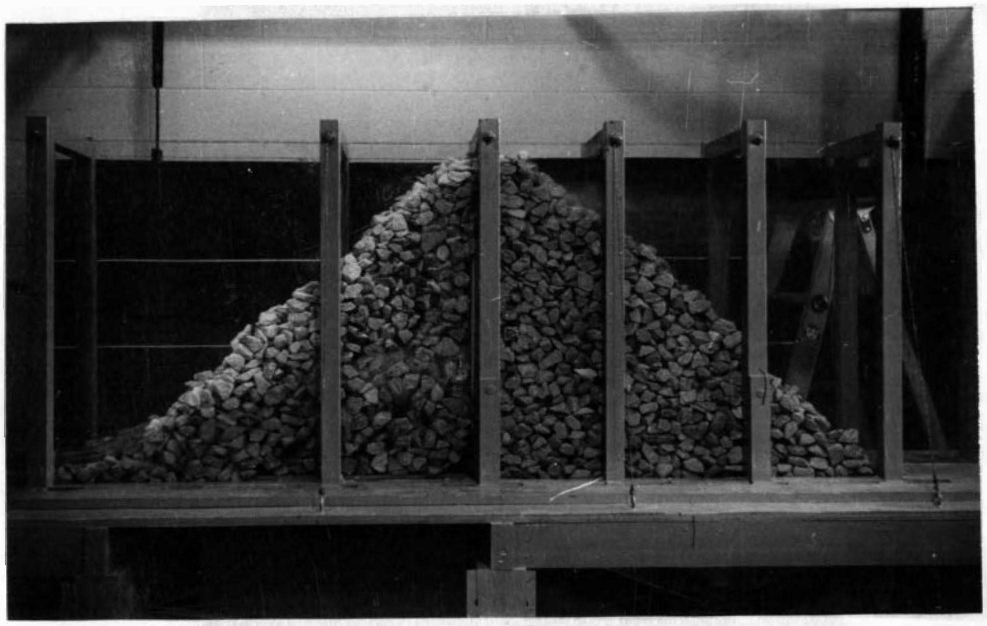


Figure 8

A Typical Dam Cross-section

Figure 9

The Rockfill Dam, looking down the length of the Flume placed in the flume, leveled horizontally and vertically plumbed, and then secured to the floor of the flume by screwing knee braces to the floor and to the core with wood screws. Diagonal braces made from reinforcing steel were used to maintain the vertical position of the core. After being tightly secured to the flume, a caulking compound was used around the edges of the core wall to furnish a water-tight seal between the core and flume. This same procedure was employed in all the tests.

When the rockfill was to be added, the core was first placed both on the upstream and downstream sides of the core wall, in layers of approximately six to twelve inches. The downstream fill was then compacted as much as possible by means of sluicing the fill with a stream of water from a nozzle connected to an ordinary 3/4 inch water hose. This afforded considerable compaction and permitted the use of higher heads of water over the core without necessitating protection to the downstream face of the dam. When using the smaller rock size this means of compaction was insufficient to keep the rock from washing away at the downstream toe of the dam. Therefore a wire mesh with 1/4 inch openings was laid over the downstream face, and secured to the floor of the flume and to a brace at the top of the flume (shown in Figure 10), and was adequate in preventing failures of this nature. A method of protection of this nature was used in the 'San Ildefonso' dam built in Mexico, and was very successful in preventing any failures of the rockfill on the downstream face. As mentioned above, the cores were first calibrated without rockfill in the channel.

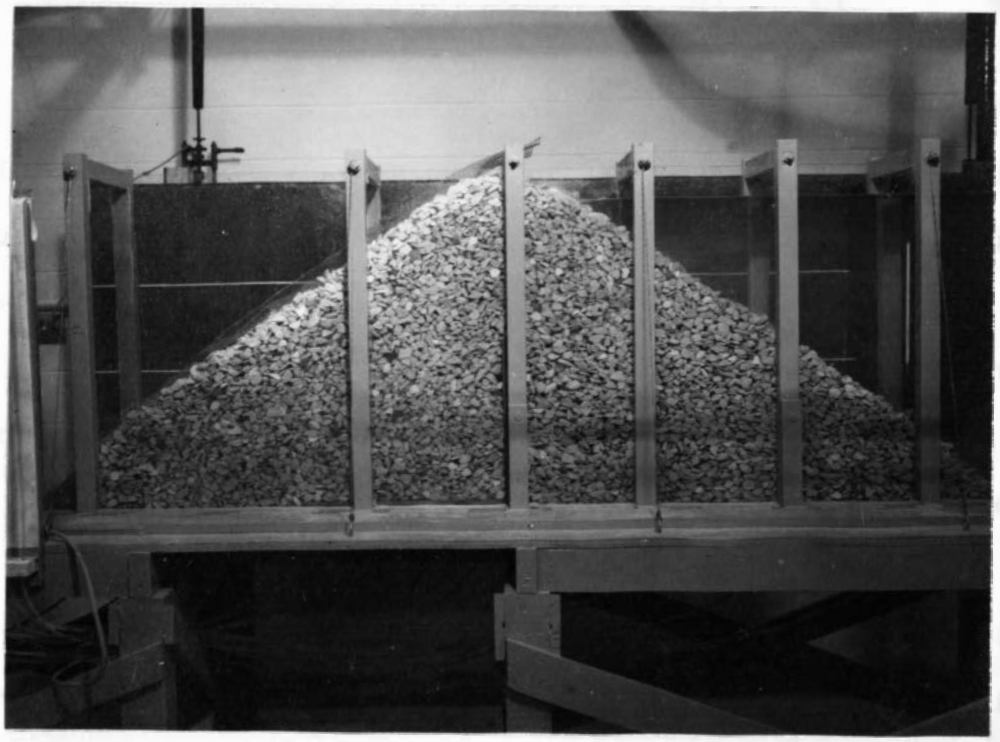


Figure 10

Wire Mesh on downstream Face of Dam

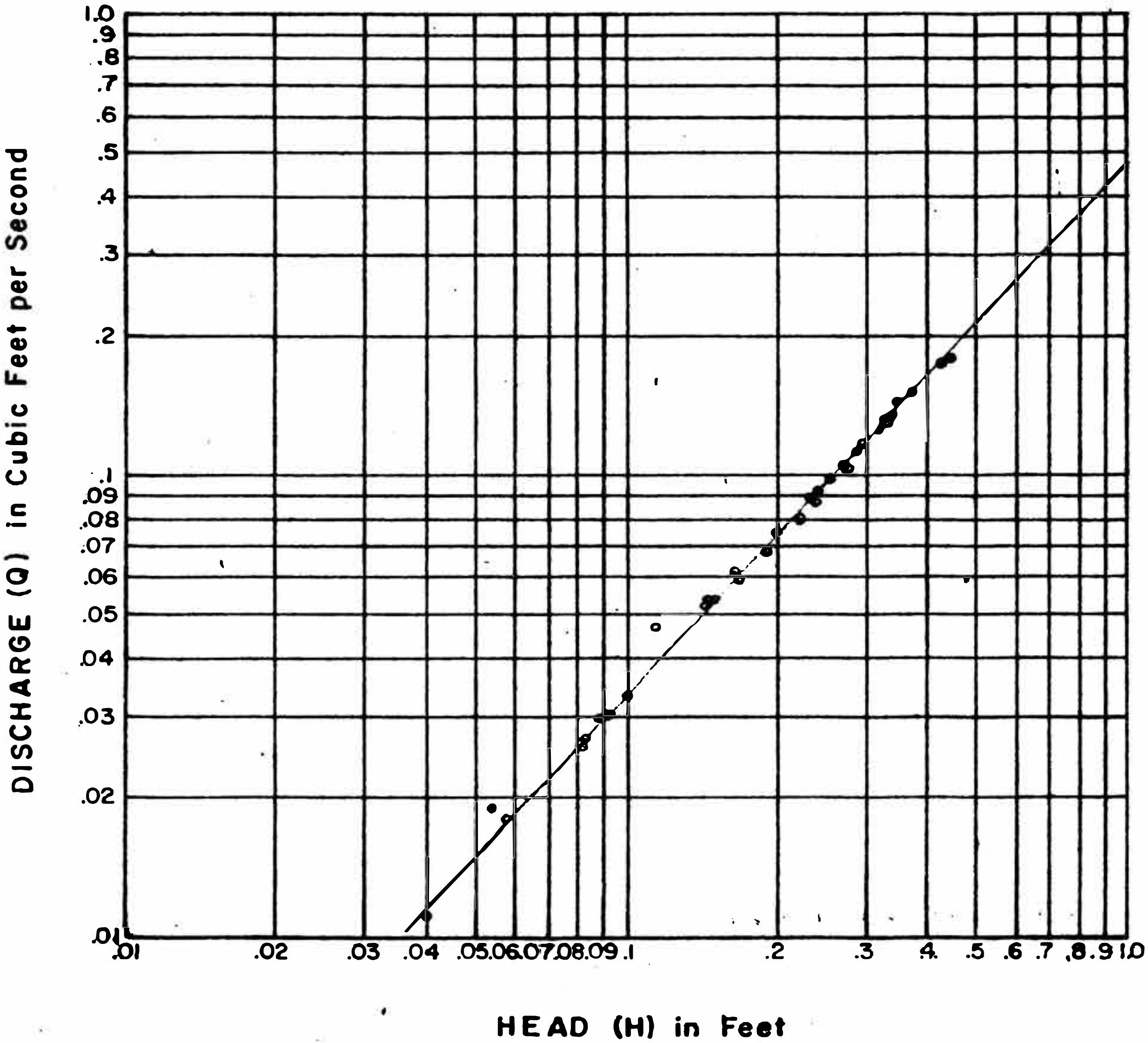
The procedure for each test was the same and the same type of data was collected. Before a test run was made, a period of five to ten minutes was required for the flow rate over the core to become established and constant. A constant flow rate could be ascertained by noting the readings of the piezometer tubes; when the head on the core had become constant, a constant flow rate was occurring. During each run the temperature of the water was taken and recorded in order that the specific weight of the water could be determined. Piezometer readings were taken for each increment of flow and the important feature noted was the variation in the readings at the same head for different core and rock sizes.

The weight of the empty collecting basin was first determined and recorded to the nearest 0.01 pound. The water collected during each run was timed with a stop watch reading to the nearest one-tenth of a second. In most cases a check run was made at the same head, and the average of the two runs used. The runs were performed with increasing increments of head, until the maximum head was attained, then the runs were repeated with decreasing values of head. The reason for this procedure was to eliminate the presence of surface tension causing a clinging nappe at low heads. These effects would tend to be present as water began flowing over the core, but would disappear at the higher heads. Therefore the purpose of this procedure is to have the same flow pattern with rising magnitudes of head as with falling magnitudes of head.

CHAPTER VI

TEST RESULTS

In performing the tests in this study it was first necessary to calibrate the core walls without rock in the channel, and the general weir equation was used for determining the coefficient of discharge over the cores. The general weir equation, of the form $Q = \frac{2}{3} ML\ (2g)^{1/2} \left[\left(H + \frac{V_1^2}{2g}\right)^{3/2} - \left(\frac{V_1^2}{2g}\right)^{3/2}\right]$, where L is the actual length of weir crest, H the observed head of water on the weir, $\frac{V_1^2}{2g}$ the velocity head in the approach channel, and M is an empirical coefficient relating the actual to the theoretical weir discharge, is easily derived and the derivation can be found in any textbook on fundamental hydraulics. It is not infrequent that the velocity of approach in a channel is negligible, and the above equation then reduces to the more simple form $Q = 2/3\ ML\ (2g)^{1/2}\ H^{3/2}$. This is an equation of the form $Q = CLH^n$; a form very frequently encountered in the field of hydraulics when attempting to obtain a relation between two variables as measured by a set of experimental data. In this study the data taken during each run was the weight of water, the time required to collect the water, the temperature of the water during collection and the head causing flow. The rate of discharge was computed from the data taken and was plotted against the head causing flow on log-log graph paper and resulted in a straight line relationship. It is apparent from the graphs constructed from the experimental data taken in this study (Figures 11 to 21) that the discharge of water over the core walls in the channel is proportional to the length



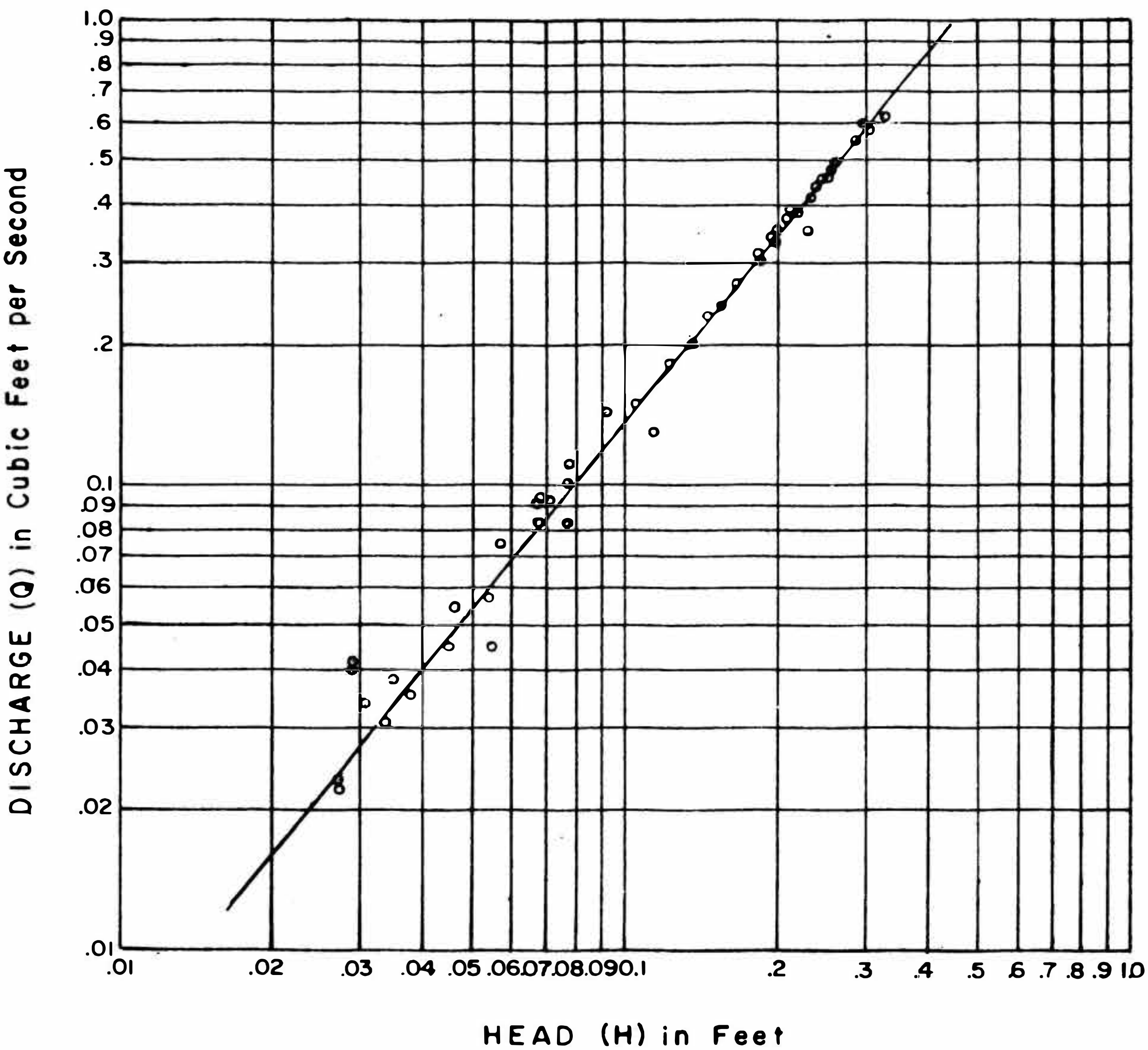
DISCHARGE (Q) Versus HEAD (H)

1" to 1 1/2" ROCK

24" Core Height

$Q = .468\ H^{1.149}$

FIGURE 11



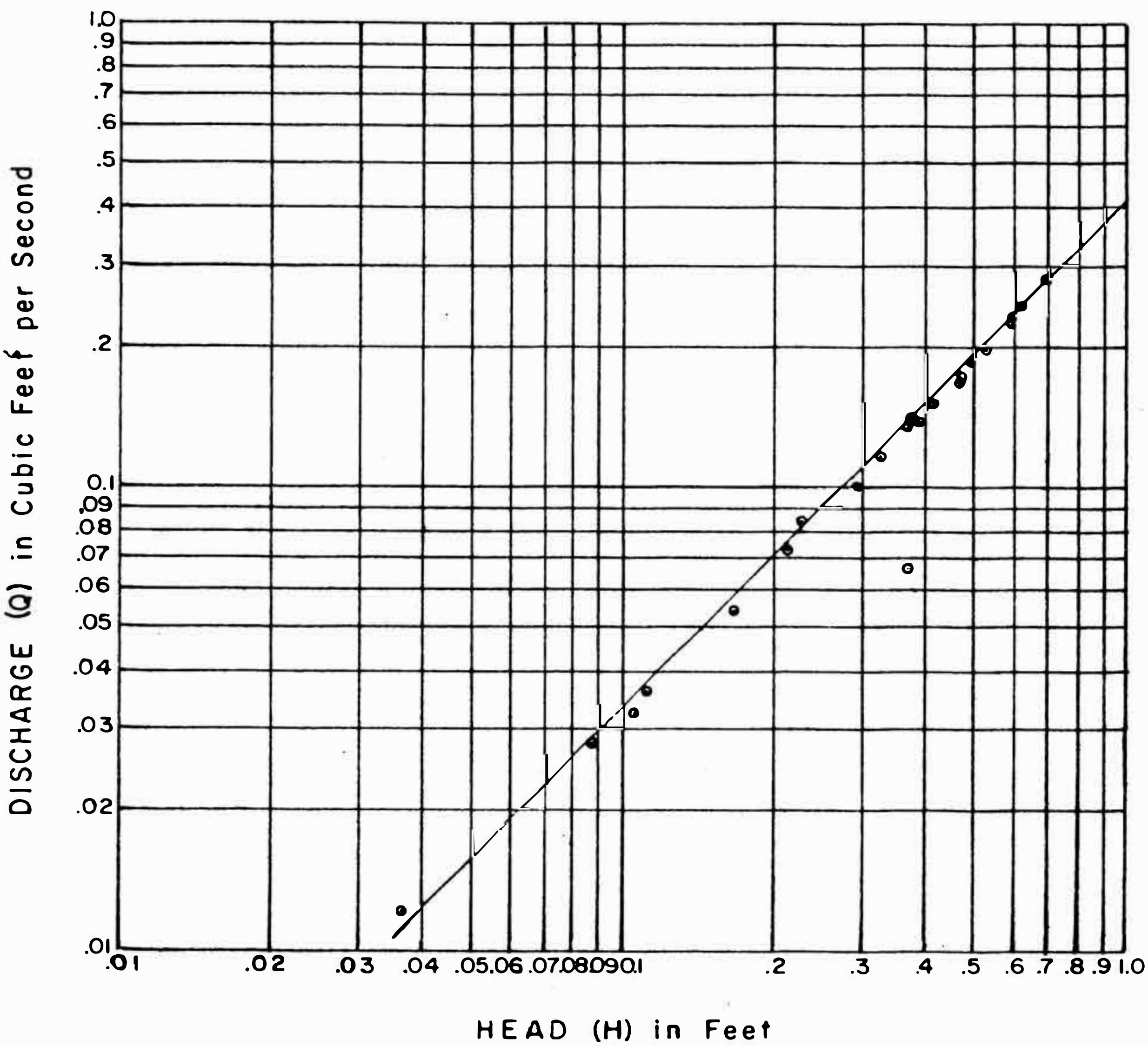
DISCHARGE (Q) Versus HEAD (H)

Without ROCK

24" Core Height

$Q = 2.82\,H^{1.330}$

FIGURE 12



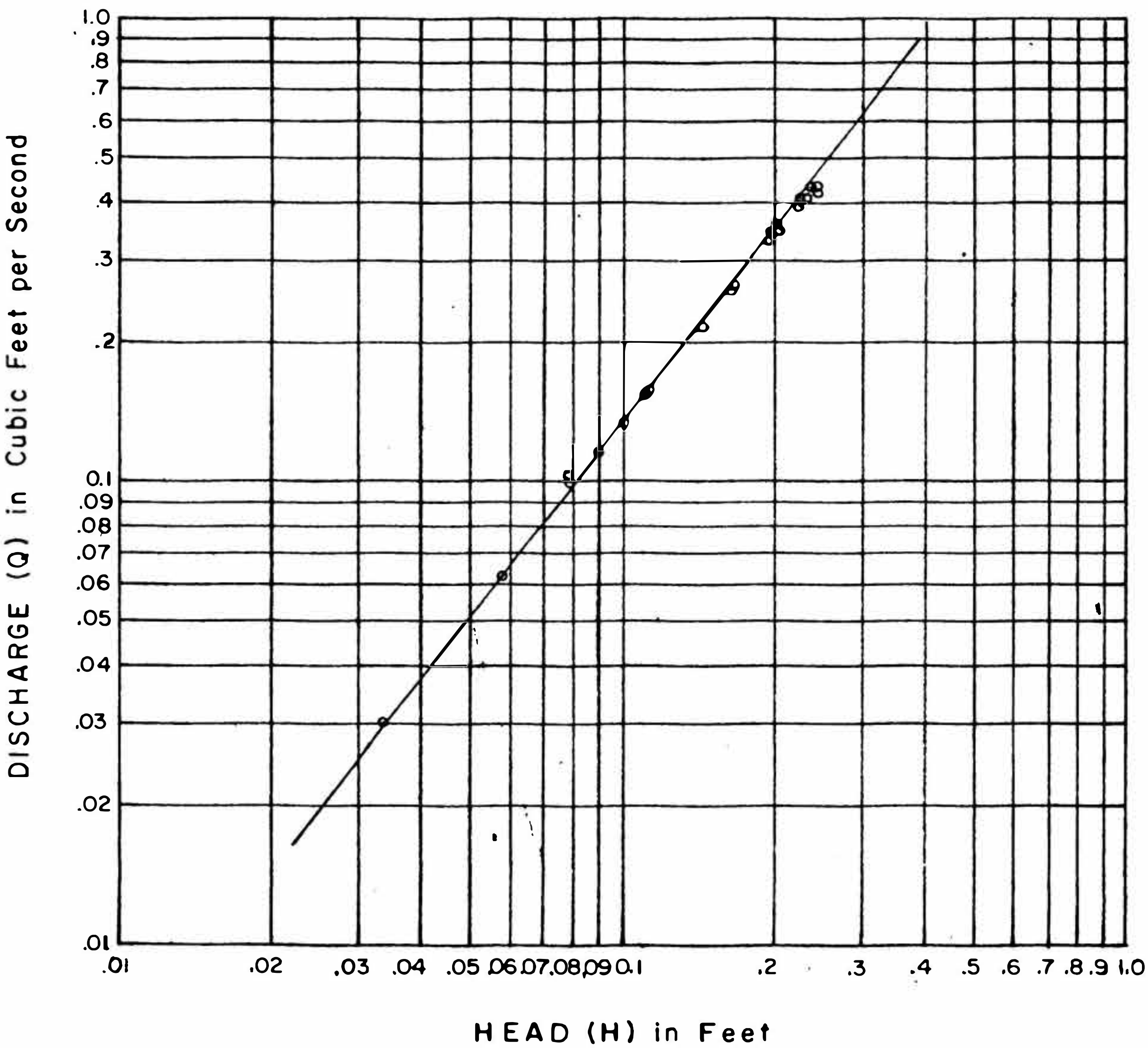
DISCHARGE (Q) Versus HEAD (H)

1" to 1 1/2" ROCK

18" Core Height

$Q = .406\, H^{1.120}$

FIGURE 13



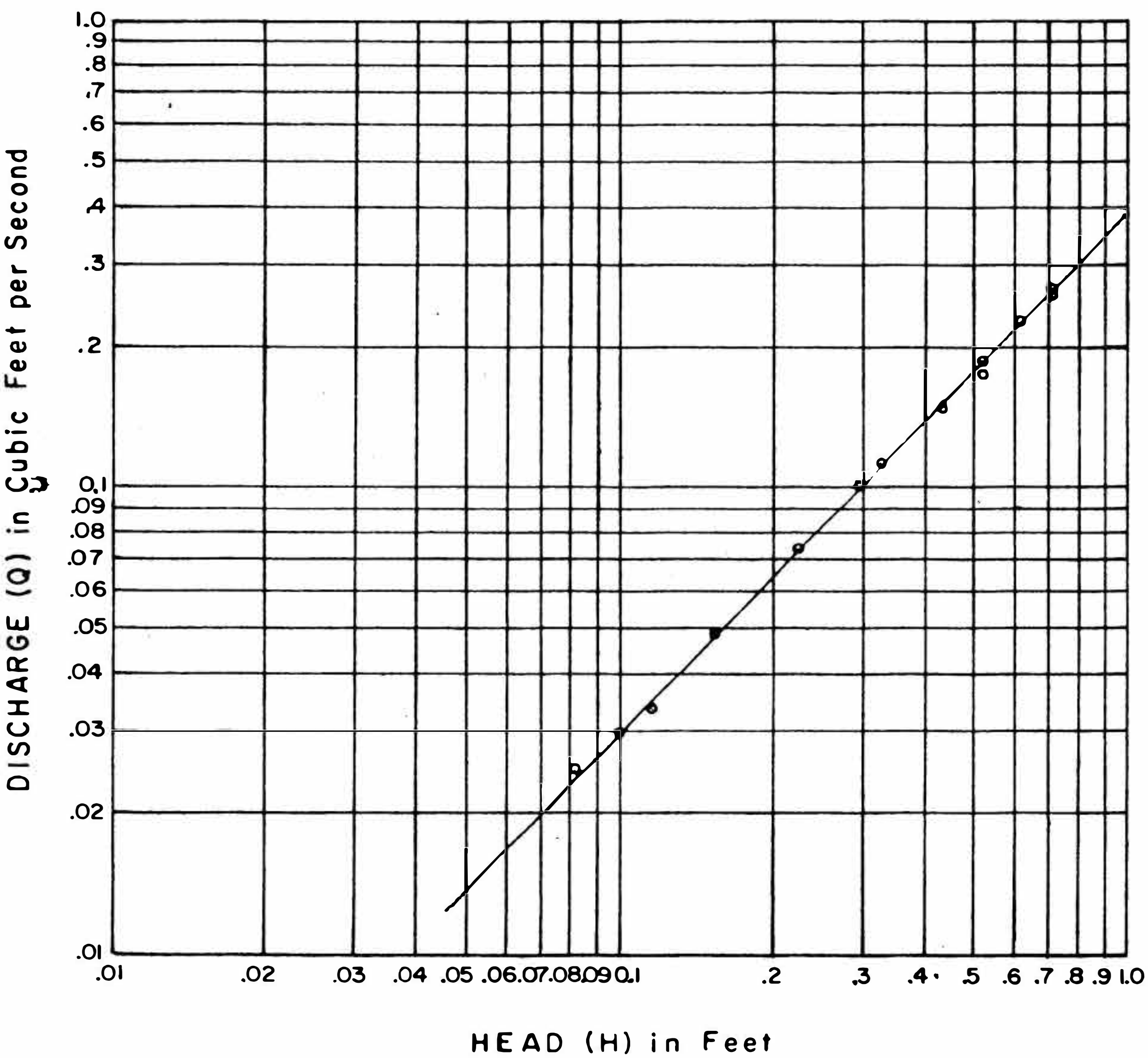
DISCHARGE (Q) Versus HEAD (H)

Without ROCK

18" Core Height

$Q = 3.15\ H^{1.373}$

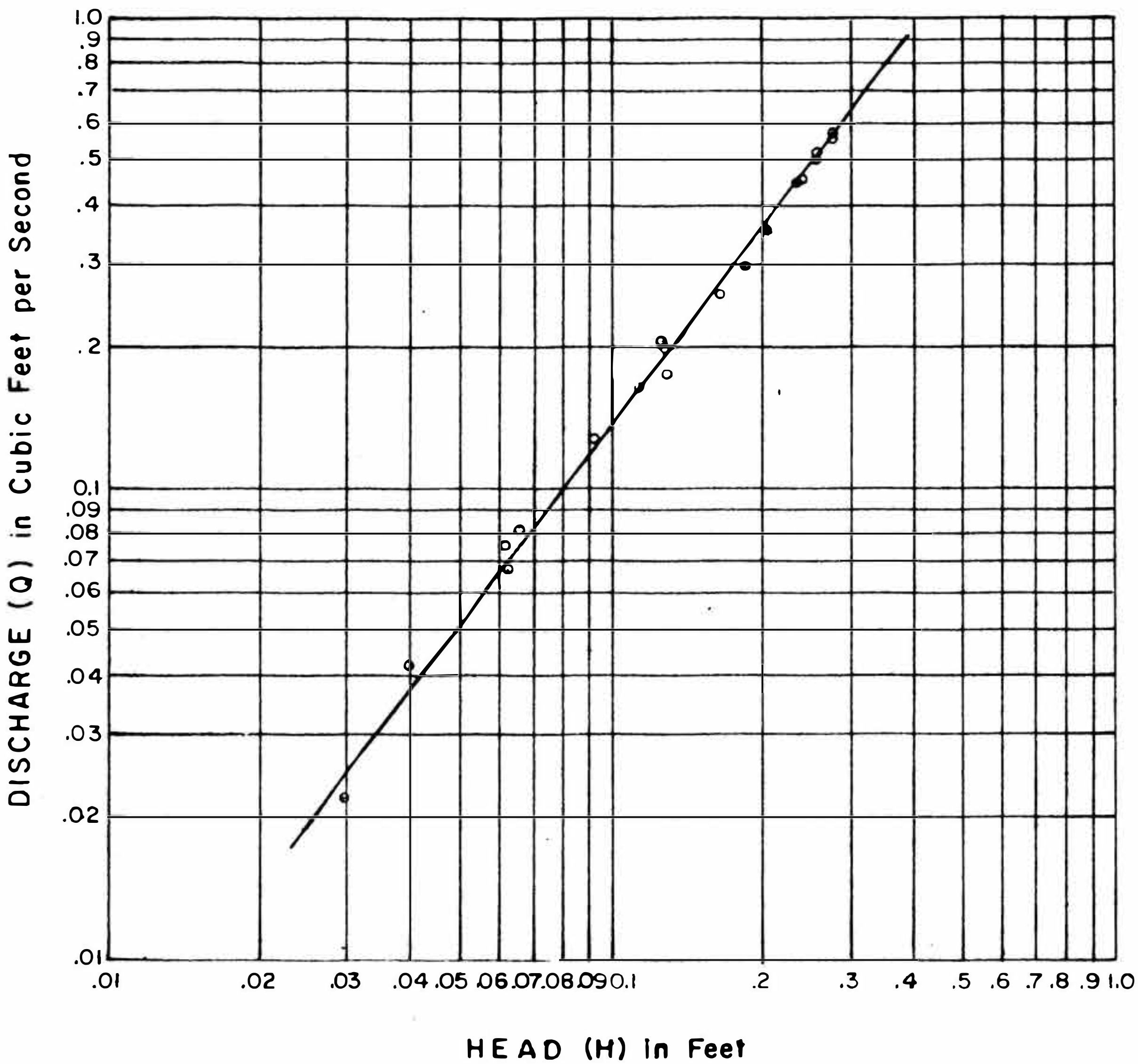
FIGURE 14



DISCHARGE (Q) Versus HEAD (H)

1" to 1 1/2" ROCK
15" Core Height
$Q = .385\ H^{1.107}$

FIGURE 15

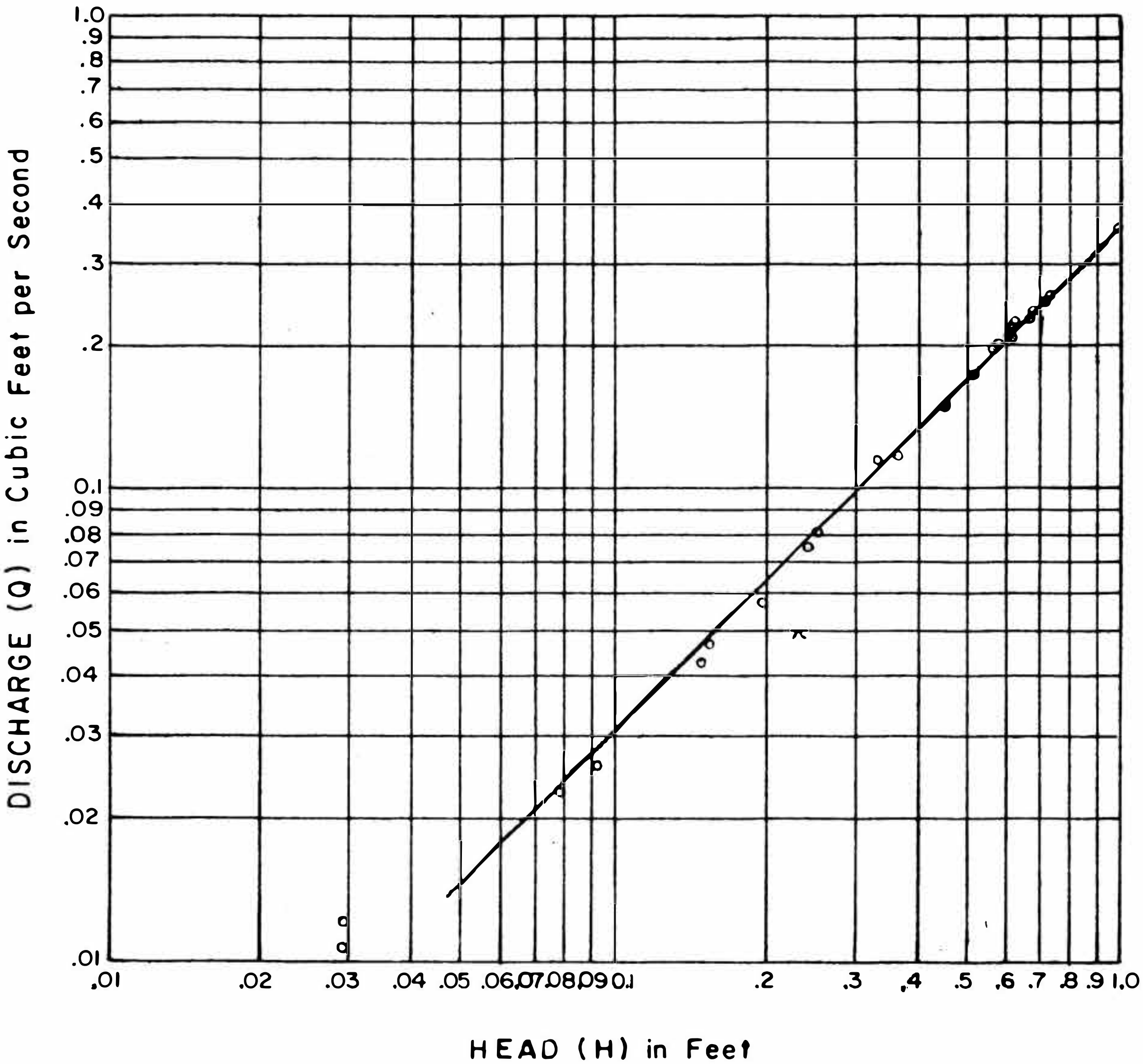


DISCHARGE (Q) Versus HEAD (H)

Without ROCK
15" Core Height

$$Q = 3.45\,H^{1.426}$$

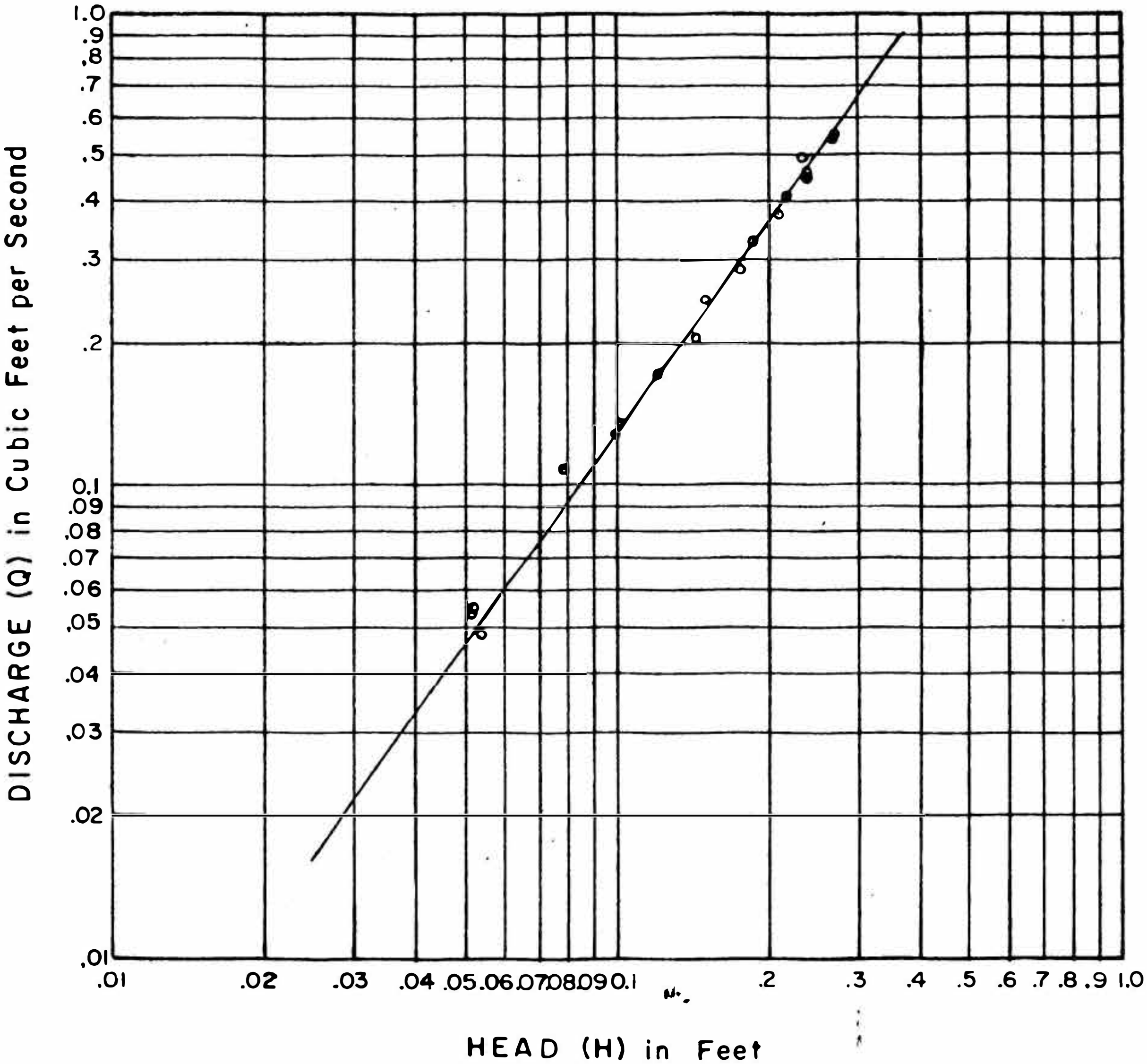
FIGURE 16



DISCHARGE (Q) Versus HEAD (H)

1" to 1 1/2" ROCK
12" Core Height
$Q = .354\,H^{1.070}$

FIGURE 17



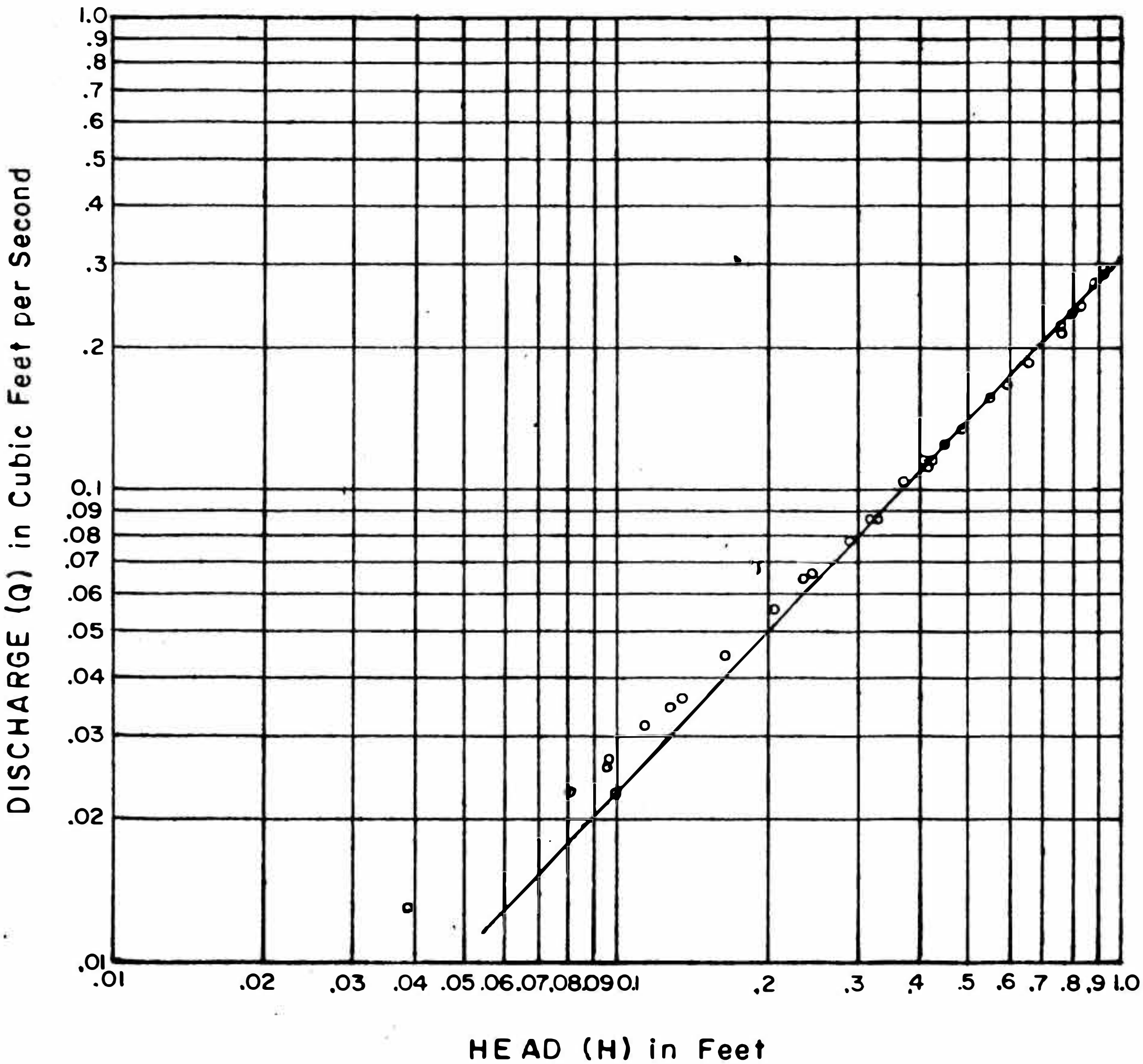
DISCHARGE (Q) Versus HEAD (H)

Without ROCK

12" Core Height

$Q = 3.87\, H^{1.478}$

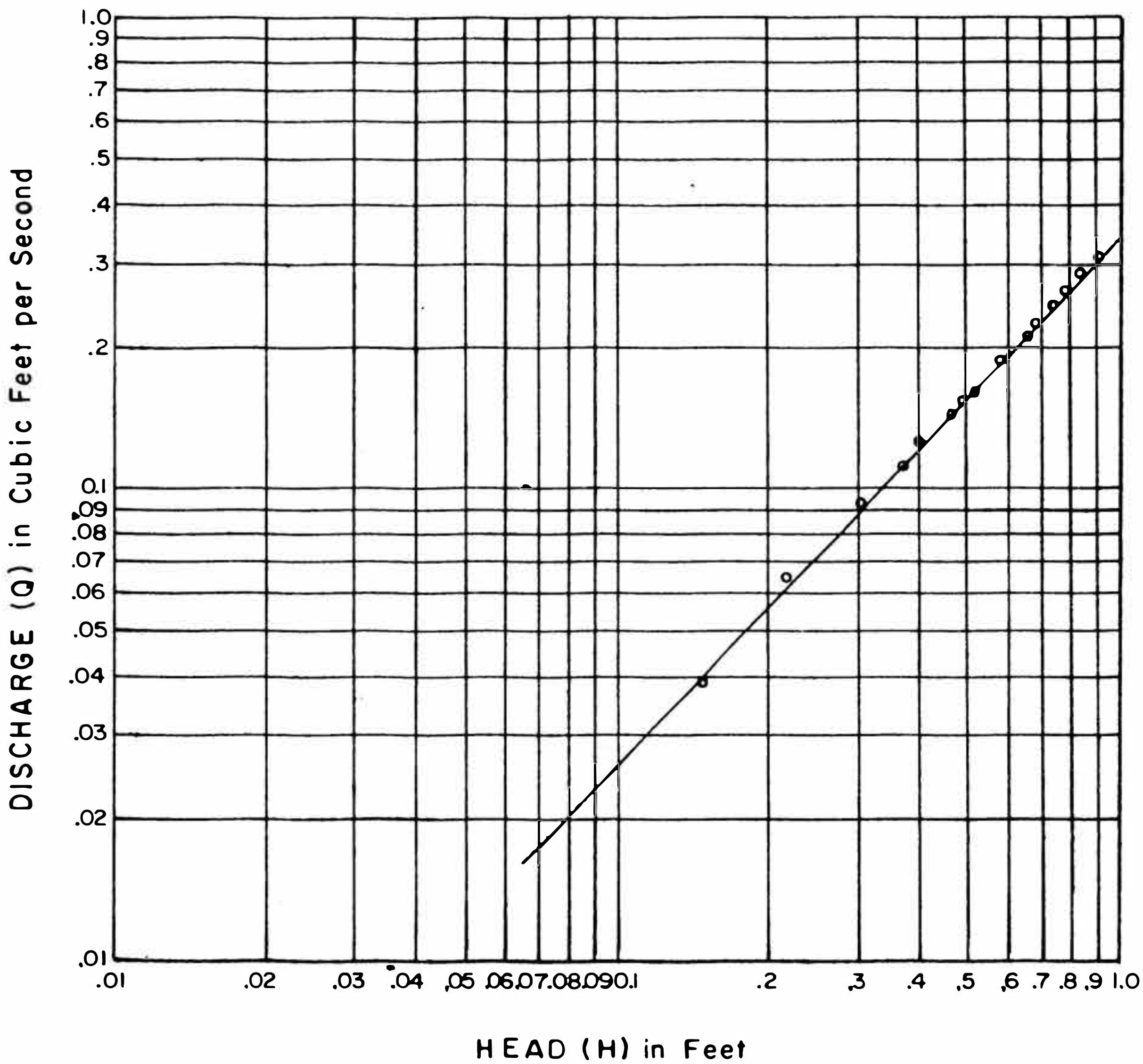
FIGURE 18



DISCHARGE (Q) Versus HEAD (H)

1/2" to 3/4" ROCK
18" Core Height
$Q = .302\ H^{1.120}$

FIGURE 19



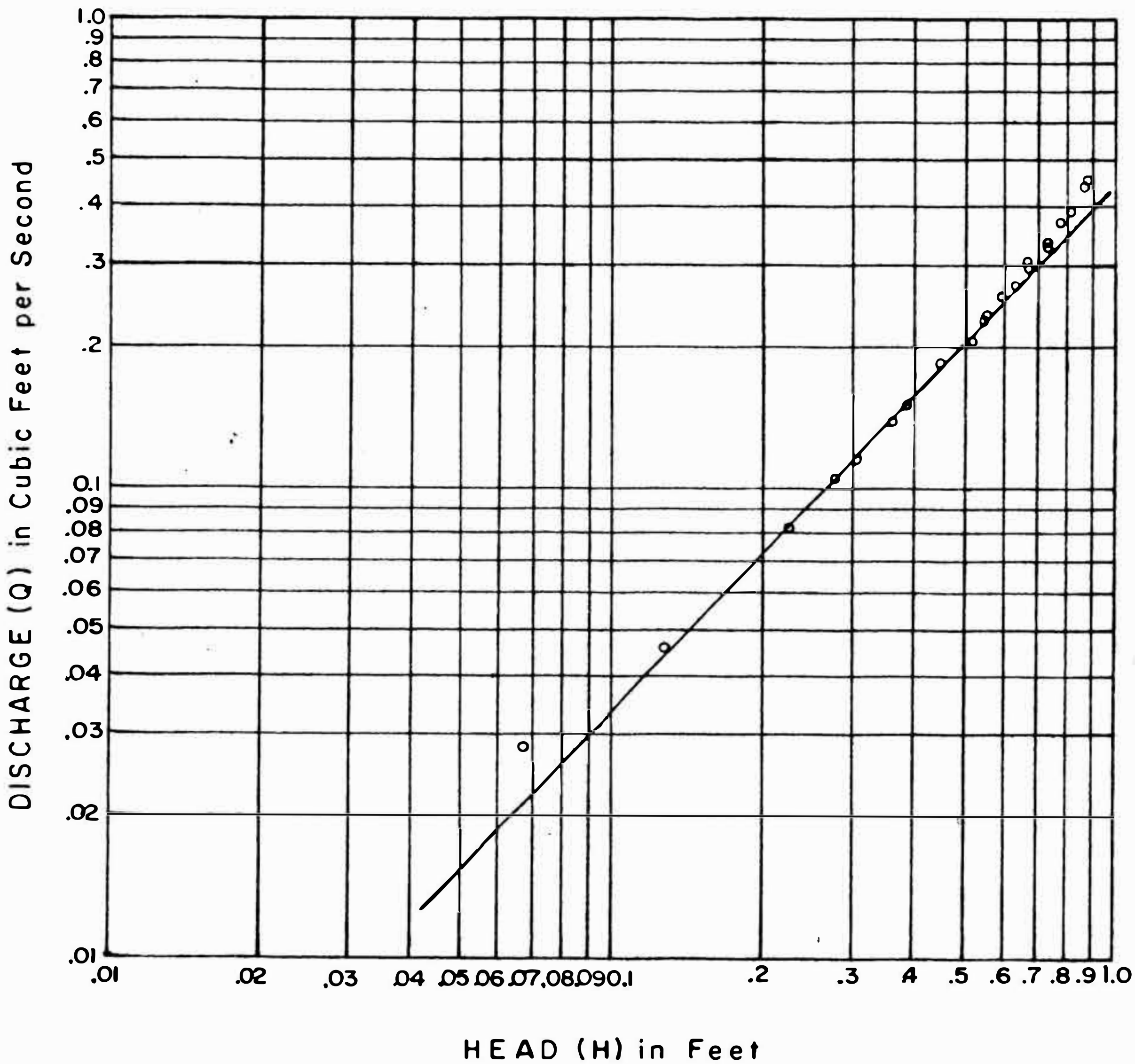
DISCHARGE (Q) Versus HEAD (H)

3/4" to 1" ROCK

18" Core Height

$Q = .339H^{1.120}$

FIGURE 20



DISCHARGE (Q) Versus HEAD (H)

1 1/2" to 4" ROCK
18" Core Height
$Q = .440H^{1.120}$

FIGURE 21

of weir crest, to the head on the core causing the flow, and is of the form $Q = KH^n$, familiar in the field of hydraulics.

It was decided to equate the relation between the discharge over the core and the head of water causing flow to the general weir equation since they were both of the same form. This allowed the calculation of the coefficient of discharge C in the equation $Q = CLH^{3/2}$, and of the coefficient of contraction M, in the equation $Q = 2/3\ ML\ (2g)^{1/2}\ H^{3/2}$. The coefficient of discharge C can be found in terms of the coefficient of contraction M by simultaneous solution of the two equations, and C is found to be 5.35 M. Thus by using this method of attack, the effect of changing core sizes and rock sizes could be evaluated and compared by observing the change in the coefficient of discharge.

In general the data taken gave very good results in that practically all the curves indicate that a linear relation exists between the rate of discharge and the head of water on the core raised to some power. It can be noted from the figures that some points vary somewhat from the plotted curves, especially at low heads. It is worthy to mention the fact that in the flow of water over a weir, surface tension will be present at low heads and small rates of discharge. It is reasonable to assume that the presence of surface tension at low heads is the cause for the variation of the plotted points in that region, and constitutes the reason for performing the tests with increasing and decreasing magnitudes of head. As heads in the range of ten feet are of interest in the prototype, a working range was established in this study including heads of the magnitude 0.5 foot up to 1.0 foot. Therefore the upper portions of the curve are of primary importance in this study, and the data in this range of values for discharge and head were used to construct the average curve.

An average curve was drawn through the plotted points and compared with the results obtained by using the method of least squares for fitting curves. There are three methods of curve-fitting used to obtain the best curve for a set of data, and the most appropriate method would depend upon the accuracy required. These three methods are the 'method of selected points', the 'method of averages', and the 'method of least squares'. The method of selected points requires the least work but is the least accurate of the three methods. The method of averages requires more computations than the method of selected points, but it gives greater accuracy. The method of Least Squares is the most laborious of the three methods, but it gives the most accurate results. Several of the curves (Figures 19 and 21) were checked by using the method of Least Squares, but since this method is such a long and arduous process, the data for the curves mentioned was written in a standard program (Least Squares Curve Fit) and run through a digital computer. The equation calculated by the computer in both instances was so close to the estimated average curve drawn through the points that it was decided to fit all the curves by estimating the average curve.

The development of the equations for the coefficient of discharge will be illustrated by means of an example, using Figure 11. The calculated values of the discharge (Q) were plotted against the head (H) causing flow, the dependent variable Q being plotted as the ordinate, and the independent variable H being plotted as the abscissa. An average curve was drawn through the plotted points, the y-intercept noted, and the slope of the curve calculated. The straight line relation present indicates an equation of the form

$$\log Q = \log K + n \log H$$

where K is the y-intercept at H = 1.0, and n is the slope of the curve. The equation of the plotted curve was therefore determined and was found to be

$$Q = 0.468\ H^{1.149}$$

The general weir equation states that

$$Q = 5.35\ MLH^{3/2}$$

and hence by equating the two equations for Q, the coefficient of contraction M for the weir can be calculated, and is found to be

$$M = \frac{0.0873}{H^{0.35}}$$

and since C = 5.35 M,

$$C = \frac{0.467}{H^{0.35}}$$

The coefficient of contraction M and therefore the coefficient of discharge C account for all the complexities of flow in this and other weir problems, such as non-uniform velocity distribution upstream, vortex action immediately ahead of the weir, non-parallel streamlines over the weir and thus through the spillway section, surface tension, viscosity, a clinging nappe, and all other factors which make weir flow a complicated process. The results of each run are tabulated in Table I.

TABLE I

P	Without Rock	With 1" to 1½" Rock	With ½" to 3/4" Rock	With 3/4" to 1" Rock	With 1½" to 4" Rock
24"	$Q = 2.82\ H^{1.330}$	$Q = 0.468\ H^{1.149}$			
18"	$Q = 3.15\ H^{1.373}$	$Q = 0.406\ H^{1.120}$	$Q = 0.302\ H^{1.120}$	$Q = 0.339\ H^{1.120}$	$Q = 0.440\ H^{1.120}$
15"	$Q = 3.45\ H^{1.426}$	$Q = 0.385\ H^{1.107}$			
12"	$Q = 3.87\ H^{1.478}$	$Q = 0.354\ H^{1.070}$			
24"	$M = \frac{0.526}{H^{0.170}}$	$M = \frac{0.0873}{H^{0.35}}$			
18"	$M = \frac{0.588}{H^{0.127}}$	$M = \frac{0.0757}{H^{0.38}}$	$M = \frac{0.0563}{H^{0.38}}$	$M = \frac{0.0632}{H^{0.38}}$	$M = \frac{0.0820}{H^{0.38}}$
15"	$M = \frac{0.643}{H^{0.073}}$	$M = \frac{0.0716}{H^{0.40}}$			
12"	$M = \frac{0.722}{H^{0.022}}$	$M = \frac{0.0660}{H^{0.43}}$			

Without rock

$$M = \frac{0.718\ (1/P)^{0.446}\ H^{0.349}}{H^{0.023}}$$

With rock

$$M = \frac{0.0658}{H^{0.428}\ (1/P)^{0.376}\ H^{0.231}}$$

With rock

$$M = \frac{0.152\ D^{0.336}}{H^{0.38}}$$

Without rock

$$C = \frac{3.84\ (1/P)^{0.446}\ H^{0.349}}{H^{0.023}}$$

With rock

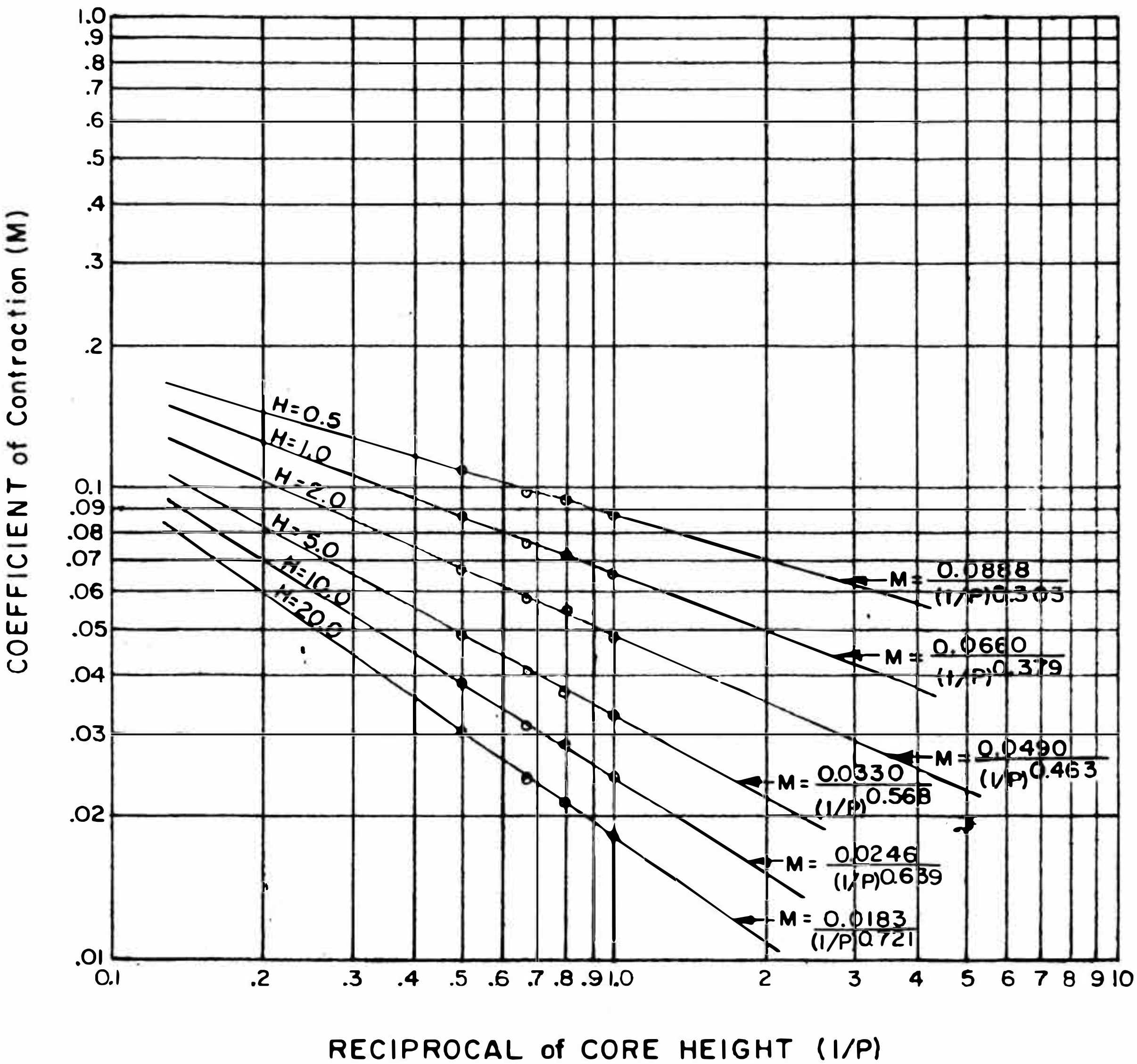
$$C = \frac{0.813\ D^{0.336}}{H^{0.38}}$$

With rock

$$C = \frac{0.352}{H^{0.428}(1/P)^{0.376}\ H^{0.231}}$$

It is seen from Table I that the coefficient of contraction varies with the height of core, the size of rock used, and the head causing flow. It was the intent of the author to establish a single relation between the variables involved; the core height, the size of rock, the head causing flow, and the coefficient of contraction. No simple equations could be established relating all four of the variables, so that two equations were found. The first equation related the height of core and the head of water to the coefficient of discharge, while the second related the size of rock used and the head of water to the coefficient of discharge.

The results shown in Table I indicate that the coefficient of contraction increases as the height of core decreases when there is no rock in the channel, but that the reverse is true when rock is present. In other words, in the process of replacing a simple weir in the channel by a rockfill dam, the slope of the plotted curve Q versus 1/P reverses direction from a positive slope to a negative slope (see Figures 22 and 23). This behavior is interesting and contrary to thoughts prior to this investigation, as the consensus of opinion of several consultants previous to this work was that as the height of core decreased, the rate of discharge over the core would increase at the same head. The belief that the rate of discharge at the same head increases when the core height decreases was substantiated when there was no rock present, but when the rockfill dam was constructed and the discharge through the dam observed, it was noted that the rate of discharge decreased as the core height decreased. This characteristic of flow can be explained by the presence



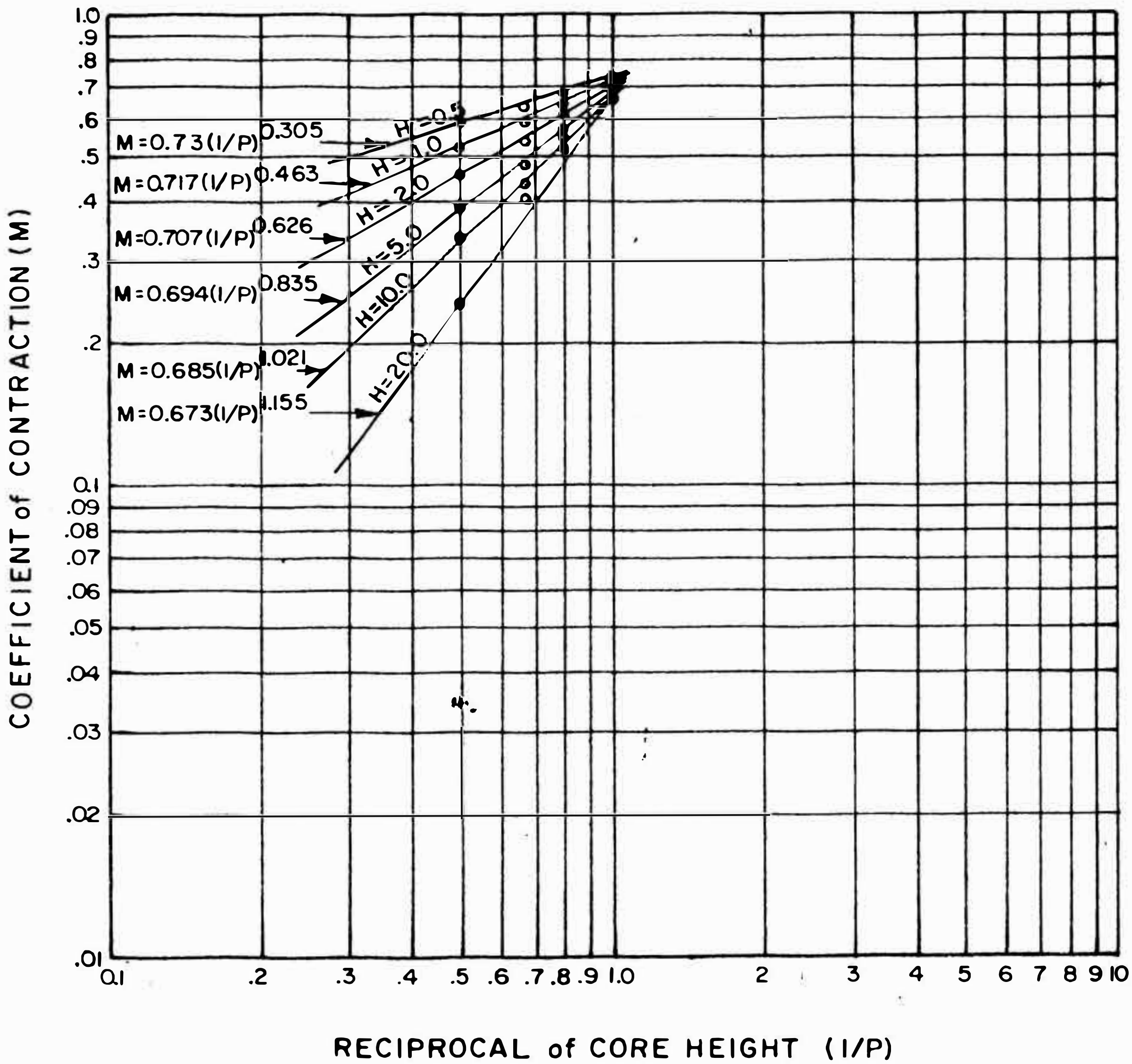
COEFFICIENT of CONTRACTION (M)

Versus

RECIPROCAL of CORE HEIGHT (1/P)

WITH ROCK

FIGURE 22

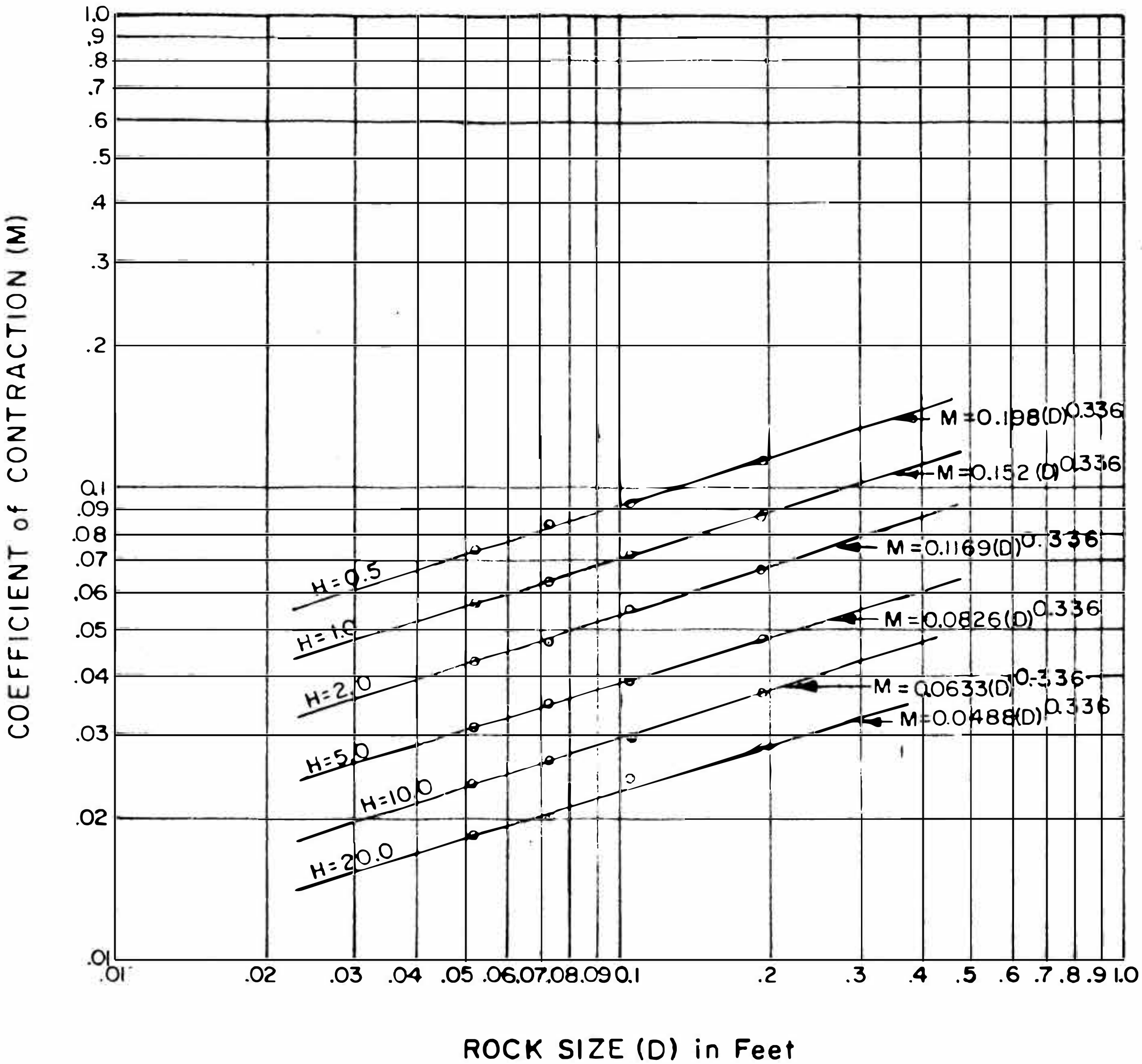


COEFFICIENT of CONTRACTION (M)
Versus
RECIPROCAL of CORE HEIGHT (I/P)
WITHOUT ROCK

FIGURE 23

of friction in the flow of water through rockfill. The effect of higher heads is evidently eliminated by the presence of friction when the water flows through a rockfill dam. It has been proven by the 'Darcy equation' that a loss of head or loss of energy occurs when water flows through a pipe line due to shear stress encountered at the pipe walls. If the openings or voids present in a rockfill dam which serve as conveyors for the water passing through the rockfill are considered to be a mass of water tubes or passages, it is readily apparent that friction losses are present. The friction loss through the rockfill will become greater as the rock size becomes smaller and as the core height decreases. Table I and Figure 24 both illustrate this fact. It must not be overlooked that there is a possibility of changing flow regimes, from turbulent to laminar flow, at low heads. The Reynolds Number was determined for one test run in this study by constructing a streamline pattern or picture of the flow, and from this pattern the flow was determined to be in the turbulent flow regime, which agreed with flow in the prototype where turbulent flow also existed.

Figure 24 represents a family of curves, the head (H) being the parameter and shows that at a constant head, the coefficient of contraction would increase with rock size. Also the fact that the intercepts of the curves change indicates that the coefficient of contraction decreases with increasing heads as the rock size remains constant. A possible explanation for this change would be the transition from a state where the rock controls flow to a state where the flow is governed by the impermeable core. This trend was not present where core heights were changed since the slopes of the curves of M versus 1/P were not constant with variable heads.



COEFFICIENT of CONTRACTION (M)

Versus

ROCK SIZE (D)

FIGURE 24

The piezometer readings used for establishing the hydraulic gradient and for determining the head of water on the core indicated that either changing the slope of the upstream face of the dam or widening the crest of the dam would affect the rate of discharge through the rockfill. The drop in the hydraulic gradient near the core was very pronounced at the higher heads when using the lower core heights and the larger rock size. When the slope of the upstream face of the dam is made flatter or the crest made wider, the water must travel farther and in a more random path. Therefore, the effects of frictional losses on the discharge through the rock as previously mentioned would cause a greater drop in the hydraulic gradient. The piezometer readings likewise gave some indication of the energy dissipation which occurred in the rockfill. Practically all of the dissipation of energy occurred in the immediate vicinity of the core wall, and was apparent by the large drop in the hydraulic gradient in this area.

On several test runs the downstream face of the dam began to bulge and rockfill started to 'kick-out' at the toe of the slope. This occurred at rather small heads when using the 1/2 inch to 3/4 inch rock. When the larger size rock was used this failure could be caused only by high heads. The 1½ inch to 4 inch rock used did not fail at the maximum head attainable in the testing apparatus, which would indicate that an effective size of rock exists which could withstand certain design flows without this type of failure. A gradation was run on the 1½ inch to 4 inch rock used in this study and it was found that 100.0% passed the 4 inch screen, 67% passed the 3 inch screen and 21.0% passed the 2 inch screen, giving an average size of 2.3 inches. This above gradation was capable of withstanding the force caused by the flowing water due to its mass and weight. When 'kick-out' failures of this nature occurred during the model study, they were eliminated by using a wire mesh laid over the downstream face of the dam as seen in Figure 10.

As previously mentioned, it was the purpose of this study to get a relation between the coefficient of discharge, the core height, the head causing flow, and the size of rock. Two equations were finally obtained, and both give accuracy well within the design limits required in engineering works. The curves of these equations were constructed by using the method of averages. In developing the equations for the coefficients of discharge, the coefficient of contraction (M) was plotted versus the reciprocal of the core height (1/P), the parameter being the head (H), as seen in Figure 22. Using the parameter H = 1.0, the equation for the curve will be determined by using the method of averages. It will be necessary to determine two constants; therefore the data is divided into two graphs and the sum of the residuals in each group set equal to zero.

$$\log M = \log a + b \log (1/P)$$

$$\log M - \log a - b \log (1/P) = 0$$

$$\sum \log M = n \log a + b \sum \log (1/P)$$

In the equation n is the number of observations in each group. The data is arranged in the following tabulation.

M	log M	1/P	log 1/P	H
0.0873	8.941-10	0.50	9.699-10	1.0
0.0757	8.879-10	0.667	9.824-10	1.0
0.0716	8.855-10	0.80	9.903-10	1.0
0.0660	8.819-10	1.00	10.000-10	1.0

From this data two equations are found:

$$17.820-20 = 2 \log a + (19.523-20)b$$

$$17.674-20 = 2 \log a + (19.903-20)b$$

Therefore b = -0.384, log a = -1.182, a = 0.0658

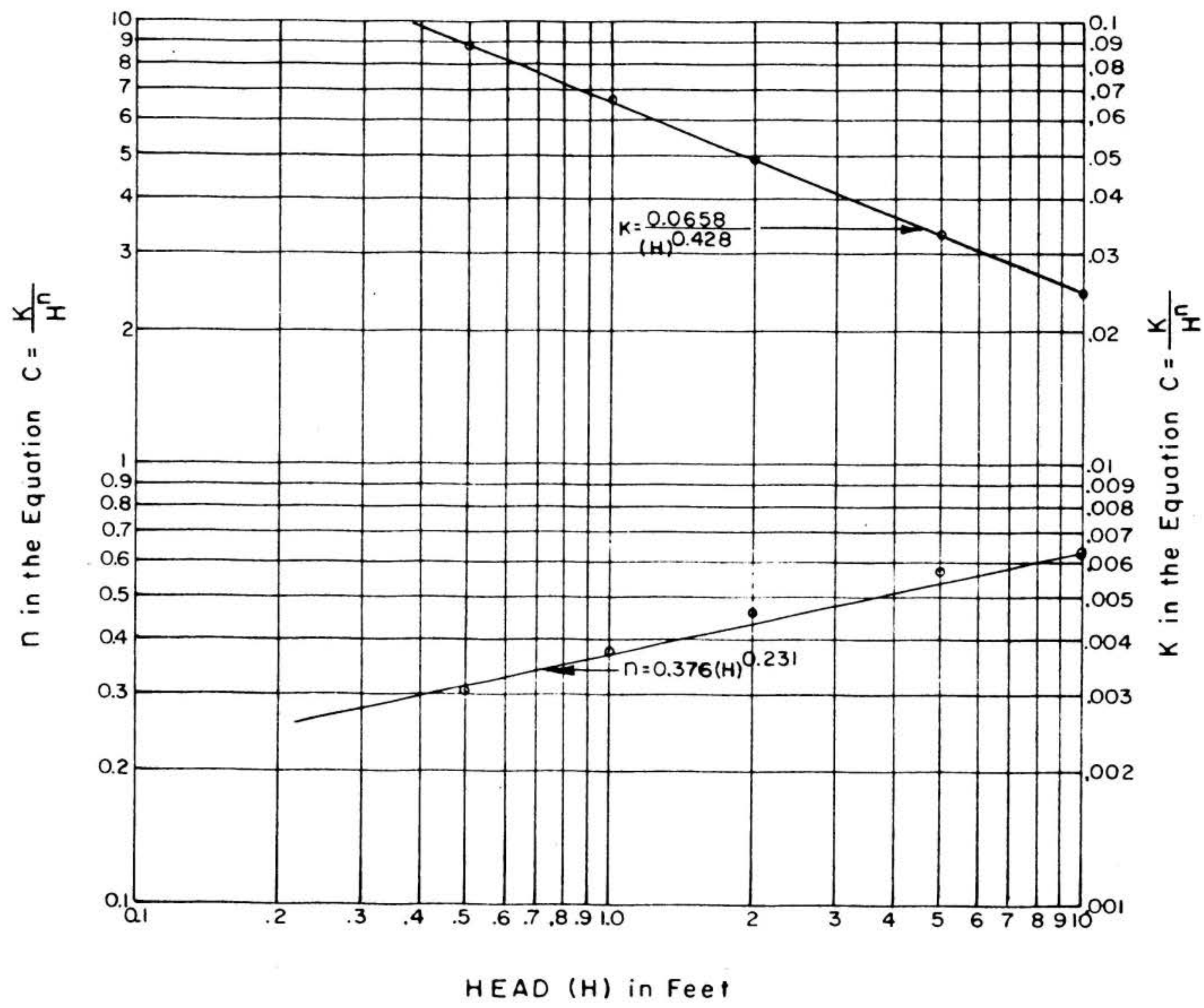
Hence log M = 1.182 - 0.384 log (1/P)

or $M = 0.0658(1/P)^{-0.384} = \dfrac{0.0658}{(1/P)^{0.384}}$

It can be seen from Figure 22 that the slope of each of the six curves is different. It was therefore decided to obtain a relation between the slopes of the curves and the head, and between the intercepts of the curves on the y-axis and the head. The six equations for M at six different heads, obtained in the same manner (method of averages) as outlined above, were of the general form:

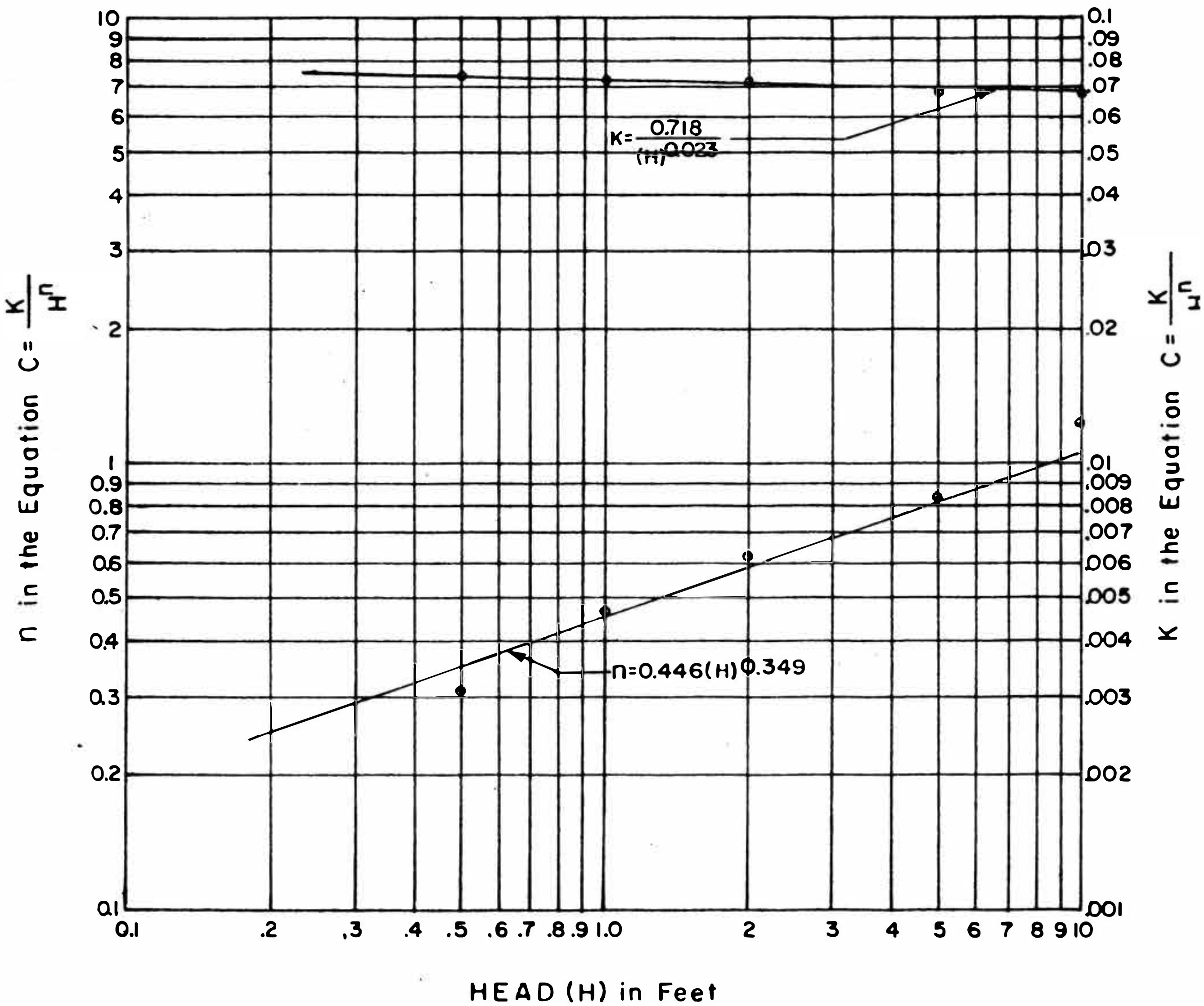
$$M = \frac{K}{(1/P)^n}$$

It was therefore decided to plot the constant K and the constant n in the above general equation versus the head (H) as shown in Figures 25-27. Both of these plots reveal a straight line variation between the independent and dependent variables. The method of averages was used to determine the average curve for the plotted points. The equation for K was found to be $K = \dfrac{0.0658}{H^{0.428}}$ while the equation for n was



n and K in the Equation $C = \frac{K}{H^n}$
Versus
HEAD (H)
WITH ROCK

FIGURE 25



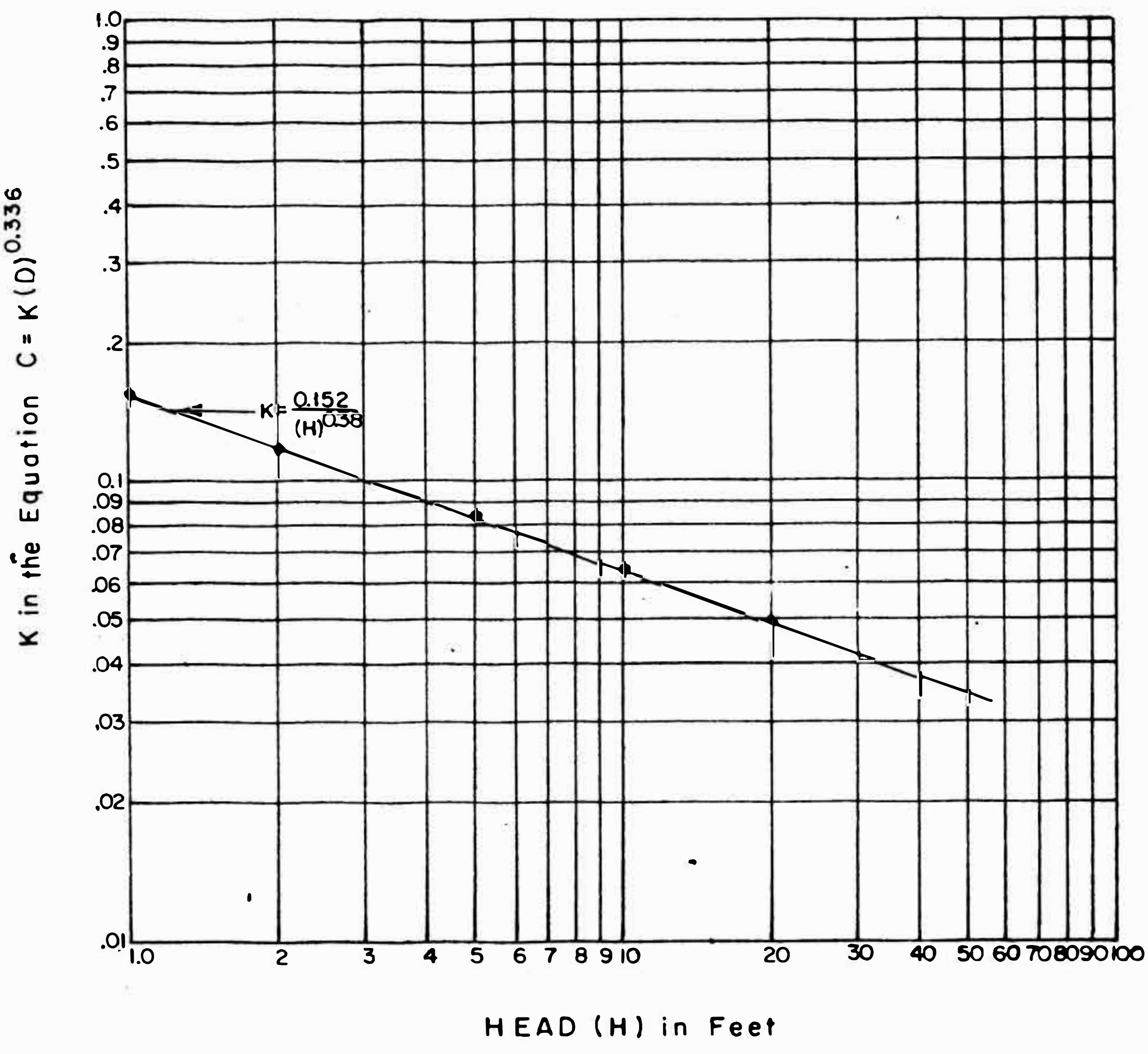
n and K in the Equation $C = \frac{K}{H^n}$

Versus

HEAD (H)

WITHOUT ROCK

FIGURE 26



K in the Equation $C = K(D)^{0.336}$
Versus
HEAD (H)
WITH ROCK

FIGURE 27

$n = 0.376\ H^{0.231}$. Combining these equations with the general form

$$M = \frac{K}{(1/P)^{n}}$$

$$M = \frac{0.0658}{H^{0.428}\ (1/P)^{0.376\ H^{0.231}}}$$

and $$C = \frac{0.352}{H^{0.428}\ (1/P)^{0.376\ H^{0.231}}}$$

It is therefore possible to predict a coefficient of discharge when the variables of head and core height are known. The equation for the coefficient of discharge in the absence of rock was determined in the same manner and found to be

$$C = \frac{3.84\ (1/P)^{0.446\ H^{0.329}}}{H^{0.023}}$$

For the variation of the coefficient of discharge with rock size, the equation was found to be

$$C = \frac{0.813D^{0.336}}{H^{0.38}}$$

These equations illustrate several points already mentioned, one being the reversal of the variation of the coefficient of discharge with the head and the core height with and without rock. It is also readily seen that the effect of the head on the coefficient of discharge is the same regardless of the size of rock used.

The accuracy of the equations for C is under a five percent error up to heads of twenty feet. In engineering work an error of five percent or less is considered to be adequate. In most cases the above equations give an accuracy within three percent, but it should be kept in mind that heads in the magnitude of ten feet in the prototype were of primary interest in this study, and the equations are valid over this range of heads.

CHAPTER VII

CONCLUSIONS

The results of this study produced information pertinent to the design of an internal spillway type rockfill dam. Several conclusions were drawn as a direct result of the work performed in this study. They are as follows:

1. The flow of water through a rockfill dam is characterized by the relation $Q = KH^n$. Although this equation is accurate for a certain 'working range', there appears to be a tendency toward variation from this curve at low heads and in some cases at high heads.

2. As the core height is decreased, the coefficient of discharge of an internal spillway type rockfill dam decreases. This is a particularly interesting fact, as it is a complete reversal of what happens when only the core wall is present in the channel.

3. At the same head, the discharge through an internal spillway is less than the discharge over the core when no rock is present. This indicates that the effects of friction on the flow rate through rockfill reduces the discharge.

4. As the size of rock increases, the coefficient of discharge increases, due to the fact that the voids or flow paths become larger.

5. The effect of the head (H) on the coefficient of discharge is constant as rock size increases.

6. Dissipation of energy occurs in the rock fill, and is a practical means of reducing first costs and maintenance costs associated with conventional concrete spillway structures.

7. When the head on the spillway section becomes excessively high, the downstream face of the dam must be protected from failure by 'kicking-out' of the rock fill at the toe. This protection can be in the form of a steel reinforcing mat laid on the downstream slope and anchored in the rockfill.

CHAPTER VIII

RECOMMENDATIONS

At the conclusion of any study or investigation of a particular problem, other areas needing further study are usually indicated, and the author recommends that further research be made in the following aspects of internal spillway type rockfill dams.

1. A series of tests should be made to determine the effect on the coefficient of discharge of varying the slope of the internal core wall. It is believed that sloping the core wall downstream will increase the flow through the rockfill, but this should be substantiated by further testing.

2. A study should be made on the effect of changing downstream, and particularly upstream, face slopes on the coefficient of discharge. This study could also include the investigation of the effect of increasing crest width on the discharge through rockfill.

3. Due to the turbulent flow which occurs through an internal spillway type rockfill dam, studies should be made to detect whether or not undermining of the core wall might possibly occur.

4. The possible derogatory effect from sedimentation clogging the spillway area could be investigated.

5. An attempt should be made to correlate the results of a study where various rock sizes are used with each change in core height.

TABLE II

DATA FOR CALIBRATION OF 24" WEIR SECTION - 1" to 1½" ROCK

CALIBRATION OF 24 INCH WEIR SECTION - 1" to 1½" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
620.50	541.10	62.32	0.018		0.058	0.058	0.009
616.60	540.35	62.32	0.018		0.058	0.058	0.009
635.00	217.60	62.31	0.047		0.117	0.117	0.022
647.50	222.70	62.31	0.047		0.117	0.117	0.022
642.25	166.80	62.31	0.062		0.166	0.166	0.028
687.75	180.50	62.31	0.061		0.166	0.166	0.028
690.00	124.50	62.31	0.089		0.233	0.233	0.040
692.75	125.00	62.31	0.089		0.233	0.233	0.040
699.75	100.70	62.31	0.112		0.284	0.284	0.049
705.50	101.20	62.31	0.112		0.284	0.284	0.049
692.75	82.50	62.31	0.135		0.340	0.340	0.057
666.00	79.15	62.31	0.135		0.340	0.340	0.057
672.50	60.70	62.31	0.178		0.429	0.429	0.073
674.25	60.95	62.31	0.178		0.429	0.429	0.073
658.50	69.30	62.31	0.152	NEGLIGIBLE	0.378	0.378	0.064
663.50	70.70	62.31	0.151		0.378	0.378	0.064
685.25	87.55	62.31	0.126		0.321	0.321	0.054
641.25	81.70	62.31	0.126		0.321	0.321	0.054
679.75	109.90	62.31	0.099		0.259	0.259	0.044
694.25	112.05	62.31	0.099		0.259	0.259	0.044
685.75	185.15	62.31	0.059		0.167	0.167	0.027
680.75	184.90	62.31	0.059		0.167	0.167	0.027
658.75	339.90	62.31	0.031		0.092	0.092	0.015
647.50	334.50	62.31	0.031		0.092	0.092	0.015
601.75	525.00	62.31	0.018		0.058	0.058	0.009
641.75	567.50	62.31	0.018		0.058	0.058	0.009
542.30	444.35	62.30	0.019		0.054	0.054	0.010
610.75	323.75	62.30	0.030		0.089	0.089	0.014

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1(ft./sec.)
613.80	182.65	62.30	0.054		0.144	0.144	0.025
615.85	146.10	62.30	0.068		0.190	0.190	0.086
622.25	114.60	62.31	0.087		0.236	0.236	0.039
631.00	96.75	62.31	0.105		0.271	0.271	0.046
637.40	79.00	62.31	0.130		0.327	0.327	0.056
684.00	76.15	62.31	0.144		0.342	0.342	0.061
669.00	59.20	62.31	0.181		0.446	0.446	0.074
676.30	83.00	62.31	0.131		0.337	0.337	0.056
660.25	101.05	62.31	0.105		0.275	0.275	0.046
662.25	130.50	62.31	0.081		0.221	0.221	0.036
655.00	196.10	62.31	0.054		0.150	0.150	0.025
658.00	390.50	62.31	0.027	NEGLIGIBLE	0.083	0.083	0.013
606.10	51.85	62.31	0.188		0.438	0.438	0.077
622.80	85.55	62.31	0.117		0.292	0.292	0.051
636.00	109.45	62.31	0.093		0.242	0.242	0.041
599.50	129.10	62.30	0.075		0.200	0.200	0.034
592.00	181.90	62.30	0.052		0.146	0.146	0.024
205.50	128.10	62.30	0.026		0.083	0.083	0.012
117.25	167.25	62.30	0.011		0.040	0.040	0.006

TABLE III

DATA FOR CALIBRATION OF 24" WEIR SECTION - WITHOUT ROCK

CALIBRATION OF 24 INCH WEIR SECTION - NO ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1(ft./sec.)
515.05	19.80	62.31	0.418	0.001	0.233	0.234	0.186
573.75	30.40	62.31	0.303		0.182	0.182	0.138
564.50	44.95	62.31	0.202		0.133	0.133	0.094
542.25	86.00	62.31	0.101		0.077	0.077	0.048
486.65	136.50	62.31	0.057		0.054	0.054	0.028
202.50	142.55	62.31	0.023		0.027	0.027	0.011
361.55	129.40	62.31	0.045		0.045	0.045	0.022
497.25	86.15	62.31	0.093	NEGLIGIBLE	0.071	0.071	0.045
591.35	63.40	62.31	0.150		0.106	0.106	0.071
624.25	41.50	62.31	0.241		0.155	0.155	0.112
667.50	35.10	62.31	0.305		0.183	0.183	0.139
658.50	28.55	62.31	0.370		0.210	0.210	0.167
646.50	24.55	62.31	0.423	0.001	0.233	0.234	0.188
624.25	120.85	62.31	0.083		0.067	0.067	0.040
534.00	244.25	62.31	0.035		0.038	0.038	0.017
384.70	68.90	62.30	0.090		0.067	0.067	0.043
392.50	58.20	62.31	0.108		0.077	0.077	0.052
376.75	42.45	62.31	0.142		0.092	0.092	0.068
419.95	36.90	62.31	0.183		0.122	0.122	0.086
486.40	33.60	62.31	0.232	NEGLIGIBLE	0.148	0.148	0.108
574.40	33.45	62.31	0.276		0.167	0.167	0.127
565.00	29.40	62.31	0.308		0.181	0.181	0.141
583.50	26.95	62.31	0.348		0.198	0.198	0.158
628.00	26.20	62.31	0.385	0.001	0.216	0.217	0.173
632.00	22.25	62.31	0.456	0.001	0.242	0.243	0.203
639.50	20.60	62.31	0.498	0.001	0.258	0.259	0.220
591.05	15.80	62.31	0.600	0.001	0.292	0.293	0.261
292.85	113.00	62.30	0.042		0.029	0.029	0.020

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs/cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1(ft./sec.)
268.50	105.10	62.31	0.041	NEGLIGIBLE	0.029	0.029	0.020
240.85	113.70	62.31	0.034		0.031	0.031	0.017
324.25	92.90	62.31	0.056		0.046	0.046	0.027
423.50	91.80	62.31	0.074		0.057	0.057	0.036
395.25	68.85	62.31	0.092		0.067	0.067	0.044
377.25	54.50	62.32	0.111		0.078	0.078	0.053
557.25	44.05	62.32	0.203		0.133	0.133	0.095
604.25	28.80	62.32	0.337		0.198	0.198	0.153
610.25	25.10	62.32	0.390	0.001	0.221	0.222	0.175
584.25	19.60	62.32	0.478	0.001	0.257	0.258	0.211
606.00	17.70	62.32	0.549	0.001	0.281	0.282	0.240
574.50	14.85	62.32	0.621	0.001	0.325	0.326	0.266
538.25	27.00	62.32	0.320	0.000	0.192	0.192	0.145
584.00	27.00	62.33	0.347	0.000	0.229	0.229	0.155
609.25	22.80	62.33	0.429	0.001	0.236	0.237	0.191
568.10	37.75	62.32	0.242	0.000	0.154	0.154	0.112
628.75	22.20	62.32	0.454	0.001	0.248	0.249	0.202
622.00	16.65	62.32	0.599	0.001	0.302	0.303	0.259

TABLE IV

DATA FOR CALIBRATION OF 18" WEIR SECTION - 1" to 1½" ROCK

CALIBRATION OF 18 INCH WEIR SECTION - 1" to 1½" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu.ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
625.00	185.90	62.31	0.054		0.168	0.168	0.032
654.45	195.80	62.31	0.054		0.168	0.168	0.032
610.50	115.10	62.32	0.085		0.246	0.246	0.049
633.25	118.80	62.32	0.086		0.246	0.246	0.049
630.00	76.40	62.32	0.132		0.372	0.372	0.070
625.50	75.50	62.32	0.133		0.372	0.372	0.071
660.25	62.50	62.32	0.170		0.465	0.465	0.086
661.50	62.55	62.32	0.170		0.465	0.465	0.086
659.75	46.35	62.32	0.228		0.596	0.596	0.109
675.50	47.65	62.32	0.228		0.596	0.596	0.108
657.25	38.20	62.32	0.276		0.698	0.698	0.125
659.75	38.45	62.32	0.275		0.698	0.698	0.125
665.50	46.30	62.32	0.231		0.593	0.593	0.110
649.25	45.30	62.32	0.230		0.593	0.593	0.110
674.50	55.50	62.32	0.195		0.521	0.521	0.096
669.50	55.10	62.32	0.195	NEGLIGIBLE	0.521	0.521	0.096
657.00	77.20	62.32	0.137		0.389	0.389	0.072
674.25	79.80	62.32	0.136		0.379	0.379	0.072
659.75	159.30	62.31	0.066		0.368	0.368	0.036
669.00	161.40	62.31	0.066		0.368	0.368	0.036
655.00	376.75	62.31	0.028		0.089	0.089	0.017
654.50	376.60	62.31	0.028		0.089	0.089	0.018
389.75	451.10	62.30	0.014		0.036	0.036	0.009
652.00	290.45	62.30	0.036		0.112	0.112	0.022
655.25	144.45	62.31	0.073		0.212	0.212	0.042
691.25	97.30	62.31	0.114		0.321	0.321	0.062
659.50	69.90	62.31	0.151		0.418	0.418	0.079

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu.ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
656.25	62.20	62.31	0.169	NEGLIGIBLE	0.462	0.462	0.086
677.00	47.00	62.31	0.231		0.594	0.594	0.110
663.75	42.90	62.31	0.248		0.633	0.633	0.116
660.50	57.70	62.31	0.184		0.496	0.496	0.092
659.00	106.00	62.31	0.100		0.291	0.291	0.056
659.25	336.30	62.31	0.032		0.104	0.104	0.020

TABLE V

DATA FOR CALIBRATION OF 18" WEIR SECTION - WITHOUT ROCK

CALIBRATION OF 18 INCH WEIR SECTION - NO ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
535.25	73.40	62.31	0.117	NEGLIGIBLE	0.090	0.090	0.073
630.00	86.20	62.31	0.117		0.090	0.090	0.074
630.50	64.25	62.31	0.158		0.112	0.112	0.097
639.00	65.45	62.31	0.157		0.112	0.112	0.097
617.50	45.75	62.31	0.217		0.143	0.143	0.132
617.75	45.60	62.31	0.217		0.143	0.143	0.132
623.00	37.75	62.31	0.265		0.166	0.166	0.159
630.75	38.05	62.31	0.266		0.166	0.166	0.159
644.50	29.05	62.31	0.356	0.001	0.204	0.205	0.208
668.50	29.75	62.31	0.361	0.001	0.204	0.205	0.211
613.00	24.90	62.31	0.395	0.001	0.223	0.224	0.229
621.50	24.90	62.31	0.401	0.001	0.223	0.224	0.232
626.00	23.00	62.31	0.437	0.001	0.242	0.243	0.250
616.50	23.20	62.31	0.426	0.001	0.242	0.243	0.244
619.00	22.80	62.31	0.436	0.001	0.238	0.239	0.250
603.00	23.40	62.31	0.414	0.001	0.225	0.226	0.239
607.00	23.30	62.31	0.418	0.001	0.229	0.230	0.241
634.25	30.20	62.31	0.337	0.001	0.196	0.197	0.198
612.00	28.70	62.31	0.342	0.001	0.196	0.197	0.201
609.25	37.65	62.31	0.260	NEGLIGIBLE	0.164	0.164	0.156
600.00	37.05	62.31	0.260		0.164	0.164	0.156
625.50	65.75	62.31	0.153		0.112	0.112	0.094
620.25	65.35	62.31	0.152		0.112	0.112	0.094
611.75	98.10	62.31	0.100		0.079	0.079	0.063
616.50	99.00	62.31	0.101		0.079	0.079	0.064
598.25	151.90	62.31	0.063		0.058	0.058	0.040
600.25	154.30	62.31	0.063		0.058	0.058	0.040
620.75	326.15	62.31	0.030		0.034	0.034	0.020
617.25	326.45	62.31	0.030		0.034	0.034	0.020

TABLE VI

DATA FOR CALIBRATION OF 15" WEIR SECTION - 1" to 1½" ROCK

CALIBRATION OF 15 INCH WEIR SECTION - 1" to 1½" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
645.30	409.45	62.31	0.025		0.083	0.083	0.019
627.25	399.80	62.31	0.025		0.083	0.083	0.019
704.00	112.65	62.31	0.100		0.292	0.292	0.065
660.75	105.75	62.31	0.100		0.292	0.292	0.065
661.25	70.45	62.31	0.151		0.428	0.428	0.090
660.25	70.60	62.31	0.150		0.428	0.428	0.089
665.00	57.05	62.31	0.187		0.519	0.519	0.106
672.75	57.60	62.31	0.187		0.519	0.519	0.106
660.75	47.20	62.31	0.225		0.618	0.618	0.120
663.25	47.40	62.31	0.225		0.618	0.618	0.120
663.00	39.80	62.31	0.267		0.712	0.712	0.136
664.00	39.85	62.31	0.267		0.712	0.712	0.136
663.50	41.00	62.31	0.260		0.704	0.704	0.133
661.75	41.00	62.31	0.259	NEGLIGIBLE	0.704	0.704	0.132
663.00	57.20	62.31	0.186		0.519	0.519	0.105
658.75	57.10	62.31	0.185		0.518	0.518	0.104
657.50	93.55	62.31	0.113		0.331	0.331	0.071
660.75	94.15	62.31	0.133		0.331	0.331	0.071
666.25	144.90	62.31	0.074		0.225	0.225	0.050
663.75	144.30	62.31	0.074		0.225	0.225	0.050
663.00	216.20	62.31	0.049		0.154	0.154	0.035
659.50	215.40	62.31	0.049		0.154	0.154	0.035
659.75	306.90	62.31	0.034		0.116	0.116	0.025
660.25	307.90	62.31	0.034		0.116	0.116	0.025

TABLE VII

DATA FOR CALIBRATION OF 15" WEIR SECTION - WITHOUT ROCK

CALIBRATION OF 15 INCH WEIR SECTION - NO ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1(ft./sec.)
653.25	249.20	62.30	0.042	NEGLIGIBLE	0.040	0.040	0.032
650.50	249.00	62.30	0.042		0.040	0.040	0.032
654.75	138.10	62.31	0.076		0.062	0.062	0.058
648.50	136.85	62.31	0.076		0.062	0.062	0.058
682.75	144.10	62.30	0.076		0.062	0.062	0.058
655.75	128.35	62.31	0.082		0.066	0.066	0.062
657.75	82.70	62.30	0.128		0.092	0.092	0.095
658.25	86.95	62.30	0.122		0.092	0.092	0.090
656.25	64.90	62.30	0.162		0.115	0.115	0.118
656.25	64.90	62.30	0.162		0.115	0.115	0.118
660.75	53.05	62.30	0.200		0.128	0.128	0.145
661.75	52.30	62.30	0.203		0.128	0.128	0.147
660.50	40.80	62.30	0.260	0.001	0.165	0.166	0.183
661.50	40.85	62.30	0.260	0.001	0.165	0.166	0.183
664.75	41.05	62.30	0.260	0.001	0.165	0.166	0.183
661.50	29.95	62.30	0.354	0.001	0.202	0.203	0.244
658.50	29.65	62.30	0.356	0.001	0.202	0.203	0.245
661.75	23.90	62.30	0.444	0.001	0.232	0.233	0.199
663.25	23.60	62.30	0.451	0.001	0.232	0.233	0.303
647.50	20.55	62.30	0.506	0.002	0.257	0.259	0.335
666.25	21.00	62.30	0.509	0.002	0.257	0.259	0.337
657.50	18.80	62.30	0.561	0.002	0.274	0.276	0.367
648.75	18.30	62.30	0.569	0.002	0.274	0.276	0.372
655.00	23.40	62.30	0.449	0.001	0.240	0.241	0.300
652.75	23.35	62.30	0.449	0.001	0.240	0.241	0.300
652.00	35.10	62.30	0.298	0.001	0.184	0.185	0.207

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (Cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
649.00	34.80	62.30	0.299	0.001	0.184	0.185	0.208
667.75	60.85	62.30	0.176		0.129	0.129	0.127
659.75	60.00	62.30	0.176	NEGLIGIBLE	0.129	0.129	0.128
653.25	156.40	62.31	0.067		0.063	0.063	0.051
662.75	160.40	62.31	0.066		0.063	0.063	0.050
417.50	309.55	62.31	0.022		0.029	0.029	0.017
416.25	310.70	62.31	0.022		0.029	0.029	0.017

TABLE VIII

DATA FOR CALIBRATION OF 12" WEIR SECTION - 1" to 1½" ROCK

CALIBRATION OF 12 INCH WEIR SECTION - 1" to 1½" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu.ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
231.25	309.90	62.26	0.012		0.029	0.029	0.012
231.75	310.10	62.25	0.012		0.029	0.029	0.012
671.00	231.25	62.27	0.047		0.154	0.154	0.041
665.25	228.65	62.28	0.047		0.154	0.154	0.040
654.00	182.80	62.30	0.057		0.196	0.196	0.048
658.00	185.10	62.30	0.057		0.196	0.196	0.048
657.25	129.90	62.30	0.081		0.258	0.258	0.064
660.25	130.45	62.30	0.081		0.258	0.258	0.064
660.00	92.35	62.30	0.115		0.350	0.350	0.085
653.50	91.20	62.30	0.115		0.350	0.350	0.085
665.00	70.90	62.31	0.151		0.450	0.450	0.104
658.75	70.20	62.31	0.151		0.450	0.450	0.104
677.50	62.80	62.31	0.173	NEGLIGIBLE	0.512	0.512	0.114
666.25	62.05	62.31	0.172		0.512	0.512	0.114
661.50	54.40	62.30	0.195		0.575	0.575	0.124
657.75	54.50	62.30	0.194		0.575	0.575	0.123
655.25	44.85	62.30	0.234		0.682	0.682	0.139
659.75	45.25	62.30	0.234		0.682	0.682	0.139
664.75	43.05	62.30	0.248		0.722	0.722	0.144
658.00	42.30	62.30	0.250		0.722	0.722	0.145
661.25	51.40	62.30	0.206		0.612	0.612	0.128
664.50	51.50	62.30	0.207		0.610	0.610	0.128
659.00	51.10	62.30	0.207		0.610	0.610	0.128
663.50	49.45	62.30	0.215		0.632	0.632	0.132
675.25	50.05	62.30	0.217		0.632	0.632	0.132
670.25	55.85	62.30	0.193		0.573	0.573	0.122
661.75	55.35	62.30	0.192		0.573	0.573	0.122

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu.ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
666.00	90.90	62.30	0.118	NEGLIGIBLE	0.362	0.362	0.086
667.25	91.10	62.30	0.118		0.362	0.362	0.086
657.25	141.05	62.30	0.075		0.245	0.245	0.060
659.50	141.70	62.30	0.075		0.245	0.245	0.060
667.00	248.55	62.30	0.043		0.150	0.150	0.037
662.00	248.05	62.30	0.043		0.150	0.150	0.037
656.50	411.45	62.30	0.026		0.092	0.092	0.023
652.50	411.20	62.30	0.026		0.092	0.092	0.023
640.50	444.20	62.31	0.023		0.078	0.078	0.021
630.50	440.20	62.31	0.023		0.078	0.078	0.021
127.00	371.00	62.31	0.006		0.029	0.029	0.005

TABLE IX

DATA FOR CALIBRATION OF 12" WEIR SECTION - WITHOUT ROCK

CALIBRATION OF 12 INCH WEIR SECTION - NO ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu.ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
605.00	177.90	62.31	0.055	NEGLIGIBLE	0.052	0.052	0.052
650.25	191.75	62.31	0.054		0.052	0.052	0.052
653.25	98.30	62.31	0.107		0.079	0.079	0.098
637.75	95.70	62.31	0.107		0.079	0.079	0.100
652.50	59.05	62.31	0.177		0.121	0.121	0.158
659.00	59.85	62.31	0.177		0.121	0.121	0.157
665.30	45.70	62.31	0.234		0.150	0.150	0.203
679.25	46.20	62.31	0.236		0.150	0.150	0.204
650.50	32.50	62.31	0.322	0.001	0.186	0.187	0.270
658.75	32.80	62.31	0.322	0.001	0.186	0.187	0.271
667.25	26.65	62.31	0.402	0.002	0.216	0.218	0.330
647.50	25.80	62.31	0.402	0.002	0.216	0.218	0.330
628.00	20.30	62.31	0.496	0.002	0.232	0.234	0.402
661.25	21.40	62.31	0.496	0.002	0.232	0.234	0.401
635.00	18.50	62.31	0.550	0.003	0.265	0.268	0.434
641.75	19.00	62.31	0.542	0.003	0.265	0.268	0.427
653.25	23.05	62.31	0.455	0.002	0.238	0.240	0.366
646.25	22.70	62.31	0.458	0.002	0.238	0.240	0.368
649.25	23.00	62.31	0.453	0.002	0.238	0.240	0.365
646.00	27.60	62.31	0.376	0.002	0.208	0.210	0.310
644.25	27.50	62.31	0.376	0.002	0.208	0.210	0.310
640.00	35.45	62.31	0.290	0.001	0.178	0.179	0.245
652.25	36.05	62.31	0.290	0.001	0.178	0.179	0.246
647.25	50.20	62.31	0.207	0.001	0.141	0.142	0.181
650.75	50.55	62.31	0.207	0.001	0.141	0.142	0.181
661.00	79.05	62.31	0.134	0.000	0.104	0.104	0.121
672.25	80.50	62.31	0.134	0.000	0.104	0.104	0.121
654.50	217.15	62.31	0.048	0.000	0.054	0.054	0.046
673.75	223.90	62.31	0.048	0.000	0.054	0.054	0.046

TABLE X

DATA FOR CALIBRATION OF 18" WEIR SECTION - ½" to 3/4" ROCK

CALIBRATION OF 18 INCH WEIR SECTION - 1/2" to 3/4" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
659.00	463.20	62.27	0.023		0.081	0.081	0.014
658.00	466.20	62.28	0.023		0.080	0.080	0.014
662.50	116.80	62.28	0.091		0.329	0.329	0.050
663.00	117.00	62.30	0.091		0.331	0.331	0.049
666.25	93.45	62.30	0.114		0.412	0.412	0.059
678.00	95.00	62.30	0.115		0.412	0.412	0.060
666.00	92.60	62.30	0.115		0.418	0.418	0.060
660.50	91.80	62.30	0.116		0.418	0.418	0.060
640.25	75.40	62.30	0.136		0.492	0.492	0.068
664.00	79.45	62.30	0.134		0.492	0.492	0.067
667.75	64.30	62.30	0.167		0.592	0.592	0.079
673.95	64.70	62.30	0.167		0.592	0.592	0.080
669.00	57.80	62.30	0.186		0.653	0.653	0.086
674.00	58.05	62.30	0.186	NEGLIGIBLE	0.653	0.653	0.086
668.75	49.20	62.30	0.218		0.752	0.752	0.096
666.50	49.00	62.30	0.218		0.752	0.752	0.096
666.15	43.40	62.30	0.246		0.834	0.834	0.105
668.50	43.60	62.30	0.246		0.834	0.834	0.105
668.25	39.20	62.30	0.274		0.892	0.892	0.114
665.00	38.95	62.30	0.274		0.892	0.892	0.114
666.75	48.20	62.30	0.222		0.752	0.752	0.098
661.75	47.60	62.30	0.223		0.752	0.752	0.099
672.25	68.70	62.30	0.157		0.549	0.549	0.076
660.25	67.80	62.30	0.156		0.549	0.549	0.076
675.00	84.60	62.30	0.128		0.459	0.459	0.065
659.25	82.50	62.30	0.128		0.459	0.459	0.065

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1(ft./sec.)
683.75	140.70	62.30	0.078		0.289	0.289	0.043
654.25	135.00	62.30	0.078		0.289	0.289	0.043
664.75	188.90	62.30	0.056		0.208	0.208	0.033
702.25	200.30	62.30	0.056		0.208	0.208	0.033
659.00	296.70	62.30	0.036		0.132	0.132	0.022
656.50	298.80	62.30	0.035		0.129	0.129	0.022
662.50	399.30	62.30	0.027		0.096	0.096	0.016
658.50	400.80	62.30	0.026		0.096	0.096	0.016
685.75	169.60	62.29	0.065		0.237	0.237	0.037
658.25	162.90	62.29	0.065		0.237	0.237	0.037
667.50	37.45	62.29	0.286		0.928	0.928	0.117
676.75	37.60	62.29	0.289		0.928	0.928	0.118
665.00	45.00	62.29	0.237		0.800	0.800	0.103
662.00	44.80	62.29	0.237		0.800	0.800	0.103
668.00	104.80	62.29	0.102		0.371	0.371	0.054
662.50	103.75	62.29	0.102	NEGLIGIBLE	0.371	0.371	0.054
696.75	128.05	62.29	0.087		0.320	0.320	0.048
662.25	121.90	62.29	0.087		0.320	0.320	0.048
664.75	159.00	62.29	0.067		0.248	0.248	0.038
659.25	158.00	62.29	0.067		0.248	0.248	0.038
662.75	239.80	62.29	0.044		0.165	0.165	0.109
711.50	259.60	62.29	0.044		0.165	0.165	0.109
660.25	331.80	62.29	0.032		0.115	0.115	0.076
660.50	333.00	62.29	0.032		0.115	0.115	0.076
366.00	472.30	62.29	0.013		0.039	0.039	0.026
562.75	727.00	62.29	0.013		0.039	0.039	0.026

TABLE XI

DATA FOR CALIBRATION OF 18" WEIR SECTION - 3/4" to 1" ROCK

CALIBRATION OF 18 INCH WEIR SECTION - 3/4" to 1" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
665.50	164.15	62.29	0.065		0.217	0.217	0.038
723.75	178.85	62.29	0.065		0.217	0.217	0.038
666.25	113.95	62.29	0.094		0.304	0.304	0.052
669.50	114.65	62.29	0.094		0.304	0.304	0.052
670.50	86.10	62.29	0.125		0.400	0.400	0.065
671.25	86.10	62.29	0.125		0.400	0.400	0.066
663.25	68.70	62.29	0.155		0.489	0.489	0.077
664.25	68.60	62.29	0.155		0.489	0.489	0.078
671.50	57.10	62.29	0.189		0.578	0.578	0.090
667.00	56.90	62.29	0.188		0.578	0.578	0.090
667.25	47.90	62.29	0.224		0.677	0.677	0.102
664.50	47.30	62.29	0.226		0.677	0.677	0.103
672.00	44.05	62.29	0.245		0.735	0.735	0.109
671.50	43.90	62.29	0.246	NEGLIGIBLE	0.735	0.735	0.109
666.50	40.45	62.29	0.264		0.788	0.788	0.115
670.25	40.55	62.29	0.265		0.788	0.788	0.115
666.25	40.20	62.29	0.266		0.788	0.788	0.116
671.75	37.20	62.29	0.290		0.848	0.848	0.123
668.25	37.00	62.29	0.290		0.848	0.848	0.123
669.50	34.35	62.29	0.313		0.904	0.904	0.130
666.50	34.00	62.29	0.315		0.904	0.904	0.130
664.75	50.10	62.29	0.213		0.656	0.656	0.099
667.75	50.45	62.29	0.212		0.656	0.656	0.098
665.75	65.90	62.29	0.162		0.518	0.518	0.080
665.25	66.05	62.29	0.162		0.518	0.518	0.080
664.75	74.55	62.29	0.143		0.462	0.462	0.072

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
669.50	75.00	62.29	0.143	NEGLIGIBLE	0.462	0.462	0.073
667.75	96.20	62.29	0.111		0.371	0.371	0.061
671.00	96.60	62.29	0.112		0.371	0.371	0.061
397.25	163.25	62.29	0.039		0.148	0.148	0.024
401.00	163.60	62.29	0.039		0.148	0.148	0.024

TABLE XII

DATA FOR CALIBRATION OF 18" WEIR SECTION - 1½" to 4" ROCK

CALIBRATION OF 18 INCH WEIR SECTION - 1½" to 4" ROCK

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1 (ft./sec.)
669.50	102.90	62.29	0.104		0.278	0.278	0.058
666.75	102.75	62.29	0.104		0.278	0.278	0.058
670.25	77.60	62.29	0.139		0.361	0.361	0.074
670.50	77.50	62.29	0.139		0.361	0.361	0.074
672.75	59.10	62.29	0.183		0.454	0.454	0.093
666.75	58.45	62.29	0.183		0.454	0.454	0.093
665.25	46.80	62.29	0.228		0.542	0.542	0.111
669.75	47.35	62.29	0.227		0.542	0.542	0.111
667.25	45.65	62.29	0.235	NEGLIGIBLE	0.554	0.554	0.114
671.25	45.80	62.29	0.235		0.554	0.554	0.114
675.80	39.80	62.29	0.273		0.627	0.627	0.127
662.75	38.80	62.29	0.274		0.627	0.627	0.128
678.75	32.75	62.29	0.333		0.725	0.725	0.149
651.75	31.15	62.29	0.336		0.725	0.725	0.150
656.25	26.85	62.29	0.392		0.815	0.815	0.169
665.25	27.30	62.29	0.391		0.815	0.815	0.168
658.75	23.50	62.29	0.450	0.001	0.888	0.889	0.187
661.75	23.85	62.29	0.445	0.001	0.888	0.889	0.186
662.25	28.70	62.29	0.370		0.788	0.788	0.161
666.75	28.80	62.29	0.372		0.788	0.788	0.162
663.50	35.85	62.29	0.297		0.672	0.672	0.136
657.75	35.10	62.29	0.301		0.672	0.672	0.138
652.00	40.75	62.29	0.257	NEGLIGIBLE	0.592	0.592	0.122
653.25	41.80	62.29	0.251		0.592	0.592	0.119
661.50	51.30	62.29	0.207		0.508	0.508	0.102
659.00	51.40	62.29	0.206		0.508	0.508	0.102

Weight of Water (lbs.)	Time (sec.)	Sp. Weight W (lbs./cu. ft.)	Q (cfs)	Velocity Head $h = V_1^2/2g$ (ft.)	Static Head H (ft.)	Total Head H + h (ft.)	Velocity V_1(ft./sec.)
663.25	70.45	62.29	0.151	NEGLIGIBLE	0.392	0.392	0.079
670.00	71.40	62.29	0.151		0.392	0.392	0.079
664.75	92.65	62.29	0.115		0.308	0.308	0.063
662.00	92.10	62.29	0.115		0.308	0.308	0.063
655.50	130.30	62.29	0.081		0.225	0.225	0.046
662.50	132.00	62.29	0.081		0.225	0.225	0.046
449.50	156.00	62.29	0.046		0.128	0.128	0.028
413.50	145.70	62.29	0.046		0.128	0.128	0.028
389.75	226.00	62.28	0.028		0.067	0.067	0.018
381.25	222.00	62.28	0.028		0.067	0.067	0.018

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VITA

Paul Rudolph Munger was born January 14, 1932 in Hannibal, Missouri. He is a son of Paul O. and Anna (Williams) Munger.

He received his elementary and high school education at Hannibal, graduating from high school in 1950.

In August 1952 he was inducted into the United States Army, and served two years, with overseas duty in France. He was separated from the Army in August 1954 and enrolled at Hannibal LaGrange Junior College in September 1954, as a pre-engineering student, where in June 1956 he received an Associate of Arts Degree.

Continuing his education he enrolled at the Missouri School of Mines and Metallurgy in September 1956. He received his Bachelor of Science Degree in Civil Engineering in June 1958.

In September 1958 he was employed as an Instructor in the Civil Engineering Department at the Missouri School of Mines and Metallurgy, at which time he began his graduate work.

His practical experience has consisted of summer employment with the Missouri State Highway Department during the summers of 1955, 1957 and 1958; with the National Park Service in Colorado during the summer of 1960; and as an Instructor in Civil Engineering at the Missouri School of Mines during the summers of 1959 and 1961.

He was married to Frieda A. Mette on November 26, 1954. They have two children, Amelia Ann born October 15, 1957, and Paul David born October 20, 1959.