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THE PANAMA CANAL.

Part I.

PLAN RECOMMENDED BY THE ISTHMIAN CANAL COM-
MISSION OF 1899-1901, AND INFERENTIALLY
ADOPTED BY CONGRESS IN THE ACT
APPROVED JUNE 28, 1902.

PUBLISHED BY
OFFICE OF ADMINISTRATION
ISTHMIAN CANAL AFFAIRS.

WASHINGTON:
GOVERNMENT PRINTING OFFICE.

1905.

ISTHMIAN CANAL COMMISSION.

THEODORE P. SHONTS, *Chairman.*

CHARLES E. MAGOON, *Governor Canal Zone.*

REAR-ADMIRAL MORDECAI T. ENDICOTT, U. S. NAVY.

BRIG. GEN. PETER C. HAINS, U. S. ARMY, RETIRED.

COL. OSWALD H. ERNST, CORPS ENGINEERS, U. S. ARMY.

BENJAMIN M. HARROD, C. E.

JOHN F. STEVENS, *Chief Engineer.*

THE PANAMA CANAL
CANAL ZONE
DEPARTMENT OF OPERATION
AND MAINTENANCE
SPECIAL ENGINEERING DIVISION

CABLE ADDRESS:
"PANCANAL, PANAMA"

Diablo Heights, C. Z.,
December 31, 1947.

Major General Thomas B. Larkin, C.E., U.S. Army,
The Quartermaster General,
Department of the Army,
Washington 25, D. C.

Dear General Larkin:

Forwarded herewith are Plates 7 and 8 for inclusion in
the Folio of Plates, supplementing the Governor's Report under
Public Law 280, 79th Congress, 1st Session.

Very truly yours,

James H. Stratton
JAMES H. STRATTON,
Colonel, C.E.,
Supervising Engineer.

Enclosures:
Plates 7 & 8

*Tour: Thanks for your nice letter
We are living in your old house and
enjoying it very much.
The first chance I have, when in
Washington, I will call at your
office.*

Jimmy

THE PANAMA CANAL
CANAL ZONE
DEPARTMENT OF OPERATION
AND MAINTENANCE
SPECIAL ENGINEERING DIVISION

Licilio Velazquez, Jr.
December 31, 1941

Cable Address
"PANAMA, PANAMA"

Major General Thomas W. ...
The Department of ...
Department of the Army
Washington 25, D. C.

Dear General ...

Forwarded herewith are ...
the ... of ...
the ... of ...
the ... of ...

Very truly yours,

James H. ...
Colonel, U. S. A.
General Staff Officer

Enclosure:
Page 7 of 8

LETTER

FROM THE

ISTHMIAN CANAL COMMISSION

TO THE

ADVISORY BOARD OF ENGINEERS

UPON

PLANS FOR THE PANAMA CANAL,

SUBMITTING DATA.

SEPTEMBER 1, 1905.

WASHINGTON:
GOVERNMENT PRINTING OFFICE.
1905.

LETTER FROM THE ISTHMIAN CANAL COMMISSION TO THE ADVISORY BOARD OF ENGINEERS.

THEODORE P. SHONTS, *Chairman.*

CHARLES E. MAGOON, *Governor, Canal Zone.*

REAR-ADMIRAL MORDECAI T. ENDICOTT, U. S. Navy.

BRIG. GEN. PETER C. HAINS, U. S. Army, retired.

COL. OSWALD H. ERNST, *Corps of Engineers, U. S. Army.*

BENJAMIN M. HARROD, *Civil Engineer.*

ISTHMIAN CANAL COMMISSION,
Washington, D. C., September 1, 1905.

THE BOARD OF ENGINEERS ADVISORY UPON PLANS FOR THE PANAMA CANAL,
Washington, D. C.

GENTLEMEN: In accordance with the directions of the President dated April 1, 1905, the Isthmian Canal Commission has the honor to lay before you physical data concerning the Isthmus of Panama, and to solicit your opinion as to the best plan to be followed in the completion of the Panama Canal within reasonable limits of cost and time.

As you are aware this question has been the subject of prolonged and elaborate studies for many years by numerous able engineers. A vast amount of labor has been expended in the collection of information concerning the physics of the Isthmus and in digesting it and formulating it into plans for the canal. The results of all these labors, both in the field and in the office, down to a recent date, are given in the reports of two distinguished commissions, viz, the comité technique, of which the report is dated at Paris November 16, 1898, and the American Commission of 1899-1901, of which the report is dated at Washington November 16, 1901. A careful perusal of these reports and examination of the maps and documents which accompany them will afford as satisfactory a view of the entire subject, at the dates when they were written, as can now be given. They have been reprinted, each in a separate pamphlet, and in that form are now handed to you, marked Part I and Part II, respectively.

During the last year additional surveys and observations have been made upon the Isthmus, the results of which are laid before you. It may be stated here in general terms that the information which they furnish does not involve any radical change in the plans previously favored. Among the observations alluded to may be included the experience of the last year in excavating the Culebra Cut, from which some of our engineers have drawn unwarranted conclusions as to the probable cost of the work. There is nothing in this experience to justify the belief that the unit prices used in previous estimates were too high, or that the estimate of the time required for completing the work was too liberal. Nevertheless, this experience has been used as an argument in favor of a sea-level canal, which plan had been condemned by the two commissions mentioned above. It becomes necessary to consider once more the sea-level scheme. The principal information available for a decision as to the merits of that project has been printed in the form of a third pamphlet, which is now handed to you, marked Part III, and in which will be found the more important results of the recent surveys.

These three pamphlets are commended to your careful consideration. With the large map (scale $\frac{1}{50000}$), a copy of which is also handed to you, it is hoped that a fair idea may be obtained of the conditions on the Isthmus and of the relative merits of the three plans proposed. There are on file here many other maps, reports, and drawings, any or all of which will be placed at your disposal should there be any point which requires further elucidation.

The plan described in the first pamphlet is the one which was adopted by Congress—at least by inference—in the act approved June 28, 1902. It is the plan under which the work is now progressing, and under which all work of construction has been done since the United States acquired the property. It closely resembles the plan of the comité technique, described in the second pamphlet, in many essential particulars, but differs from it in the height of the Bohio dam, and the important results which flow therefrom. The advantages which its authors expected to derive from this change were:

1. To take fuller advantage of the topography of the country, by which it was possible to make the Gigante spillway automatic instead of mechanical, and adequate for the discharge of the greatest floods, with only one channel to the sea instead of two.

2. To increase the distance of lake navigation from 7 to nearly 13 miles.

3. To reduce the estimated cost of the canal by about \$15,000,000.

The disadvantage of the change is the somewhat greater difficulties in constructing the higher dam and the locks of greater lift—difficulties, however, which are by no means insuperable.

A disadvantage which the two plans have in common is that the rapid developments of naval architecture make it difficult to determine the proper dimensions of the lock chambers. It is to be considered, however, that up to the present time such development has not been greatly hampered by deficient depth in the harbors of the world, and that development hereafter will have that obstruction to contend with. Moreover, it is not possible to dispense with locks entirely. Even with the sea-level canal a tide lock will be required at the Panama end.

In addition to the plan above mentioned, a pamphlet has been prepared by Mr. Lindon W. Bates, which gives in outline a sketch of a plan proposed by him, which is interesting on account of its novelty, and is therefore laid before you. It does not give detail enough for a close analysis, nor for estimates of cost. To obtain this, extensive additional surveys, to occupy at least a year's time, would be necessary.

A paper has been submitted to the President by Mr. P. Bunau-Varilla, which explains a method by which a canal constructed at first with locks may be subsequently altered to a sea-level canal. This paper also is submitted for your consideration.

These last two documents are described in a fourth pamphlet, marked Part IV, which is now handed to you.

It is to be noted that the law by which Congress ordered the construction of an isthmian canal contained the following proviso, viz:

The President shall then, through the Isthmian Canal Commission hereinafter authorized, cause to be excavated, constructed, and completed, utilizing to that end as far as practicable the work heretofore done by the New Panama Canal Company, of France, and its predecessor company, a ship canal from the Caribbean Sea to the Pacific Ocean. Such canal shall be of sufficient capacity and depth as shall afford convenient passage for vessels of the largest tonnage and greatest draft now in use, and such as may be reasonably anticipated, and shall be supplied with all necessary locks and other appliances to meet the necessities of vessels passing through the same from ocean to ocean; and he shall also cause to be constructed such safe and commodious harbors at the termini of said canal, and make such provisions for defense as may be necessary for the safety and protection of said canal and harbors.

The Commission expects to visit the Isthmus of Panama, sailing from New York during the last week in September, the exact date to be fixed hereafter. You are cordially invited to accompany them.

This method of presenting the subject to you by offering several well-digested plans has been adopted because it seemed to be the method by which all essential information could be conveyed in the most condensed possible form. It is needless to say that the Commission desired your opinion not only upon these plans, but upon any variation of them, or upon any entirely different plan which may suggest itself to you. It requests your views as to what plan it is most expedient, all things considered, for the United States to follow in the completion of the Panama Canal.

Yours, very respectfully,

T. P. SHONTS, *Chairman.*

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PLAN RECOMMENDED BY THE ISTHMIAN CANAL COMMISSION OF 1899-1901, AND INFERENTIALLY ADOPTED BY CONGRESS IN THE ACT APPROVED JUNE 28, 1902.

EXTRACTS FROM THE REPORT OF THE COMMISSION DATED NOVEMBER 16, 1901,
WITH EXPLANATORY NOTES.

The old Panama Canal Company began its work without adequate knowledge of the physical conditions at the Isthmus. It inaugurated at an early day some of the surveys and examinations required to supply the deficiency, and some of these it maintained as long as it continued to exist. Additional surveys were made by the liquidator, and very extended additional surveys and observations have been made by the new company. The information relating to the topography, hydrography, and geology of the Isthmus is now much more complete than is usual before the inauguration of an engineering enterprise in a new country. The canal company spared no trouble or expense in laying it all before the Commission. The most important maps, drawings, and documents were lithographed or printed and systematically arranged for the use of the Commission, copies being furnished for each member. Many other documents were supplied in manuscript. In all, some 340 documents, many of them elaborate studies, were furnished. These supplied essentially all the data required for the preparation of plans and estimates, though further information was desired as to the foundation upon which the great dam at Bohio must be built, and as to the area of the Chagres River drainage basin. This additional information was obtained by the field parties of this Commission. It was necessary also for the purpose of this investigation to verify the French data. Independent lines of levels, measurements of distances, borings, soundings, and hydrographic observations made by its own parties, supplemented by personal observation, enable this Commission to state that the data furnished by the canal company are essentially correct.

The circumstances under which the Commission approaches the study of a plan for the canal differ from those of the French engineers in two important particulars. The question of the time required for completion is of less vital importance, since a new concession from the Colombian Government must be obtained in any event, and since the cost of financing would be much diminished if the United States should provide the funds; that question would not be decisive against a plan which is otherwise preferable. In a plan prepared for a government seeking the permanent development of its possessions, and content to receive its returns in an indirect way and at a future time, the

Plan for the United States differs from that for a commercial corporation.

canal must have dimensions which will permit the passage of the largest ships now afloat or likely to be constructed. For a time such ships may be exceptional and the canal revenue derived from them may be small. A plan prepared for a commercial corporation investing capital from which an immediate and direct revenue is desired would probably exclude such exceptional ships, and the dimensions given the canal—at least in the beginning—would be less than in the former case.

One of the greatest natural difficulties to be encountered in the construction of a ship canal on the Panama route lies in the control of the Chagres River. That stream is about 145 miles long and has a drainage area above Bohio of about 875 square miles. Above Obispo it is in general a clear-water stream flowing over a bed of coarse gravel; but sand, clay, and silt in moderate quantities appear in the lower portions of its course. It flows through a mountainous country, in which the average annual rainfall is about 130 inches. A maximum rainfall has been observed of over 6 inches in twelve hours. Its discharge at Bohio varies from a minimum of about 350 cubic feet to a possible maximum of 136,000 cubic feet per second. The excessive rainfall and the precipitous slopes of the valley give to the river a torrential character. On December 1, 1890, it rose at Gamboa 23 feet in sixteen hours, its discharge, which was about 9,000 cubic feet per second at the beginning of the rise, increasing in the same time to six or seven times that volume. This is the most violent change of which there is definite record, but similar changes of somewhat less violence are not uncommon. The admission of a stream of this character to the canal would create conditions intolerable to navigation unless sufficient section of prism be provided to reduce the current to an unobjectionable velocity.

If a sea-level canal be constructed, either the canal itself must be made of such dimensions that maximum floods, modified to some extent by a reservoir in the Upper Chagres, could pass down its channel without injury, or independent channels must be provided to carry off these floods. As the canal lies in the lowest part of the valley, the construction of such channels would be a matter of serious difficulty, and the simplest solution would be to make the canal prism large enough to take the full discharge itself. This would have the advantage, also, of furnishing a very large canal, in which navigation under ordinary circumstances would be exceptionally easy. It would involve a cross section from Obispo to the Atlantic, having an area of at least 15,000 square feet below the water line, which would give a bottom width of about 400 feet. The quantity of excavation required for such a canal has been roughly computed, and is found to be about 266,228,000 cubic yards. The cost of such a canal, including a dam at Alhajuela and a tide lock at Miraflores, near the Pacific end, is estimated at not less than \$240,000,000. Its construction would probably take at least twenty years. This Commission concurs with the various French commissions which have preceded it since the failure of the old company in rejecting the sea-level plan. While such a plan would be physically practicable, and might be adopted if no other solution were available, the difficulties of all kinds, and especially those of time and cost, would be so great that a canal with a summit level reached by locks is to be preferred.

In the case of a canal with locks the problem of controlling the floods is very much simplified, but a new one is introduced—that of supplying the summit level with water. The quantity of water required for the operation of the canal will vary with the amount of traffic and the size of the vessels carrying it. Assuming 10 lockages per day for vessels of about 3,000 tons each, an annual traffic of about 10,000,000 tons will be accommodated, which is greater than the amount to be expected at the opening of the canal. Ten lockages will require 35,127,960 cubic feet per day, or 406 cubic feet per second, assuming that four of these lockages are for the full-size lock and six of them for the reduced size, using the intermediate gates. The loss from evaporation is assumed to be 6 inches per month. The area of the lake to be proposed hereafter is 38.5 square miles, or 1,073,318,400 square feet. The loss from evaporation over this area is 536,659,200 cubic feet in a month, or 207 cubic feet per second. The loss from leakage at the lock gates is estimated at 250 cubic feet per second. To this has been added 200 cubic feet per second for power and other contingencies.

Adding these amounts together, the total amount required to operate the canal for a traffic of 10,000,000 tons per annum is found to be 1,063, or, in round numbers, 1,070, cubic feet per second. The average annual discharge of the Chagres is far in excess of this, being about 3,200 cubic feet per second, but there is a well-defined dry season when the daily discharge is often less. A deficiency during the months of February, March, and April is to be apprehended and must be provided for, though it does not always occur. For use during these months some of the surplus waters of the other months must be stored. The minimum average discharge at Bohio for any month covered by the records is that for March, 1891, when it was 600 cubic feet per second, or 470 cubic feet less than the amount required. If water enough be stored to supply this deficiency, supposing it to exist continuously for three months, provision will be made against a state of affairs worse than any that has ever been known or is likely to occur. A deficiency of 470 cubic feet per second for ninety days gives an aggregate deficiency of 3,654,720,000 cubic feet, for which storage room must be provided. In a lake having an area of 38.5 square miles it corresponds to a depth of 3.4 feet.

The greatest flood which has occurred since the occupation of the isthmus by the Panama Railroad (which covers a period of fifty years), and so far as known the greatest which ever occurred, was that of November 18, 1879. No measurement was made of its volume, but the height which it reached at Bohio is stated upon the authority of Mr. Sosa, a Colombian engineer, to have been 39.3 feet above low water. A comparison of this height with that reached by floods of which the volume was measured (see Appendix D) leads to the conclusion that the maximum discharge at Bohio at the highest point of the flood in 1879 might have been as much as 136,000 cubic feet per second. In reaching this conclusion one of the assumptions is that there was no change in the size of the waterway between 1879 and the dates of the later floods, and that if the same quantity of water had been flowing at the later dates as in 1879, it would have reached the same height. Inasmuch as the size of the waterway was much increased subsequently to 1879 by the excavations of the old company, this assumption gives a result which is certainly not too low. In this, as in all other cases of doubt, the assumptions have been made such as to err on the safe side, if at all. The other greatest floods of which there are records are those of 1885, with a height at Bohio 33.8 feet; 1888 with height 34.7 feet; 1890 with height 32.1 feet, and 1893 with height 28.5 feet. The last two were measured, the maximum discharge in 1890 being 74,998 cubic feet per second, and in 1893, 48,975 cubic feet. Thus it appears that the floods in which the discharge exceeds 75,000 cubic feet per second are of rare occurrence. If the works be so designed that such a flood would produce no currents which would interfere with navigation, and that a flood of 140,000 cubic feet per second, while it might temporarily suspend navigation, would not injure the structure of the canal, ample provision will be made for the flood control of the Chagres.

No location suitable for a dam exists on the Chagres River below Bohio, and while this location is not without difficulties it has the great advantage that about 3 miles southwest of the dam, near the head of the Rio Gigante, the tributary of the Chagres, there exists an excellent site for a spillway, by which the discharge from the lake can be kept well away from the dam and accessory works, and may be made extremely large without inconvenience either to the canal itself or to the country below the lake. The height of this spillway would regulate the height and area of the lake. After careful consideration of the requirements for flood control and for storage against deficiency in the dry season, and also of the effect upon the amount of excavation required for the canal through the continental divide, the Commission has decided to fix this height at 85 feet above mean tide, and to make the spillway a fixed weir 2,000 feet long. The area of the lake at this height is 38.5 square miles, or 1,073,318,400 square feet. Using coefficient 3.5 in the weir formula, it is computed that with a depth of 5 feet over its crest the weir will discharge 78,260 cubic feet per second. In reaching elevation 90 the area of the lake will be enlarged to about 43 square miles and it will impound over 5,680,000,000 cubic feet of water. The quantity of water discharged over the weir while the lake is rising from elevation 85 to elevation 90, assuming circumstances of flow similar to those observed in the flood of 1893, is computed to be about 4,000,000,000 cubic feet. (See Appendix E.) The total quantity

of water impounded and discharged before the lake will rise above elevation 90 is therefore nearly 10,000,000,000 cubic feet. It provides for unimpeded navigation during all floods not exceeding 75,000 cubic feet per second. The velocity of the currents in the narrowest part of the lake would not exceed 2 feet per second. Floods may occur, however, which will cause the lake to rise above elevation 90. From the data available it is not possible to compute with precision the exact height which a flood may hereafter attain, but the extreme possible effect of a flood discharging 140,000 cubic feet per second for a prolonged period would be to raise the water over the spillway to 92.5 feet. All great floods are of short duration, and such a flood is absolutely without precedent, being as improbable as any other convulsion of nature. The crest of the dam has, however, been placed at 100 and the top of the lock walls and gates at 94, to make them entirely safe from overflow by even such a flood, the ill effect of which would be limited to the temporary obstruction of navigation by swift currents in the narrowest part of the lake, where the velocity might reach 5 feet per second. Under extreme conditions the lake might be lowered to 82 to provide water for operating the canal during the dry months. The excavations will be so adjusted as to give a depth of 35 feet at that level.

This provision for the storage of water for use in the dry season is ample for a traffic of 10,000,000 tons per annum in vessels of the size now in common use. It will be equally ample for a much larger tonnage if, as seems probable, the size of vessels continues to increase. For example, the number of vessels which passed the Suez Canal in 1900 was 3,441, against 3,389 in 1890, while the gross tonnage in 1900 was 13,699,238, against 9,749,129 in 1890. The number of vessels in 1900 was less than in 1898, while the total tonnage was greater. The annual flow of the Chagres and the topography of the country are favorable, however, to a very large increase of the supply, if that be found desirable in the future. A reservoir can be constructed at Alhajuela with a capacity for storing an additional volume of water four times that now provided for daily consumption.

The overflow of Lake Bohio will discharge through the Gigante spillway into Peña Blanca Swamp, thence through natural and artificial channels to the Chagres River below Gatun, and thence through that river to the sea, being kept out of the canal in the lowlands by levees where necessary.

The canal, as thus projected, may be described as follows (see pls. 21, 22, 23): Beginning at the 6-fathom line in Limon Bay, a channel 500 feet wide at bottom, and with side slopes 1 on 3, is excavated, curving gently to the left upon a radius of 6,560 feet, until it reaches a point just inside the jetty constructed by the old Panama Canal Company. Here it changes direction to the right upon a curve of 3,280 feet radius, and is then conducted upon a straight line for a distance of 2,000 feet to a point 2.39 miles from deep water in the bay. For about a mile this wide channel is inside the shore line, forming a narrow but well-protected harbor. Near the apex of the second curve the bottom width is increased to 800 feet for a length of 800 feet, to provide a turning basin. The estimated cost of this entrance and harbor is \$8,057,707, of which \$1,936,991 is for work outside the jetty. The annual cost of maintenance is estimated at \$30,000.

From the inner end of the harbor the bottom width of the canal is 150 feet, the side slopes of 1 on 3 being retained for 1.86 miles through the swamp, after which they are reduced to the standard used in firm earth, and are kept at that standard for a distance of 12.56 miles farther to the Bohio locks. The length of this level measured from the inner end of the harbor is 14.42 miles. Its estimated cost is \$11,099,839, including \$151,347 for levees to exclude flood waters and \$299,000 for the lower approach, 1,200 feet long, to the lock.

At Bohio is located a double flight of locks, having a total lift varying from 82 feet at the minimum level of the lake to 90 feet at the maximum, 41 to 45 to each lock, the normal lift being 85 feet. These locks are on the location adopted by the French company. They are shown on Pl. 24 and are of the type adopted for both the Nicaragua and Panama canals and described elsewhere in this report. The estimated cost of this flight of double locks, four lock chambers in all, is \$11,567,275, including excavation.

Above the locks the canal enters the artificial lake formed by the Bohio dam and known as Lake Bohio. For the first 7 miles it is a broad, deep body of water, affording room for anchorage, as well as navigation. Beyond this some light excavations are necessary. At the upper end the channel will be enlarged to provide for the flood discharge of the Chagres, being given a minimum section of 42,000 square feet. The length of the channel in Lake Bohio is 12.68 miles from the locks to the point where the canal leaves the Chagres. The section extends ninety-three hundredths of a mile farther, to the point where it enters the cut through the divide. The estimated cost of this section is \$2,952,154, including \$434,400 for the upper approach to the Bohio locks.

Near the entrance to the summit cut will be placed a pair of gates 100 feet wide, so that if it should become necessary to draw off the water from the summit cut the level of Lake Bohio would not be affected. These gates will be at the site of a lock proposed by the French company near Obispo, with a foundation on hard rock. The estimated cost of these gates, including masonry and excavation, is \$295,434.

The summit cut is 7.91 miles long from the Obispo gates to the Pedro Miguel locks. The highest point is about 5 miles from the Obispo gates, where the bottom of the canal at the axis is 286 feet below the natural surface of the ground. This is the famous Culebra cut, though the name has often been applied only to the mile of heaviest work. There is a little very hard rock at the eastern end of this section, and the western two miles are in ordinary materials. The remainder consists of a hard indurated clay, with some softer material at the top and some strata and dikes of hard rock. In fixing the price it has been rated as soft rock, but it must be given slopes equivalent to those in earth. This cut has been estimated on the basis of a bottom width of 150 feet, with side slopes of 1 on 1. While the cut would probably not be finished with this uniform slope, this furnishes as correct a basis of estimate as can now be arrived at. The entire cut will be lined with masonry walls, finishing at elevation 92, 2 feet above high water, these walls having nearly vertical faces and furnishing benches 38 feet wide on either side of the canal, on one of which the Panama Railroad will be laid, while it is probable that a service track will be placed on the other.

Much has been said about the instability of the Culebra cut; in point of fact, there is a clay in the upper portion of the deep cut which flows readily when saturated, but which will give little trouble if thoroughly drained; probably nine-tenths of the material would naturally be classed as hard clay of stable character; it would weather somewhat, and the surface might require some repairing with concrete in bad places, a practice common in deep cuttings in Europe. This clay disintegrates rapidly in water, and for this reason the canal prism should be confined between masonry walls. With the provision made for broad benches on each side, on which any slight slides would be arrested, it is believed that no trouble will be experienced. The estimated cost of the 6.02 miles of heavy work is \$41,940,480, and of the entire 7.91 miles between the Obispo gates and the Pedro Miguel locks, \$44,414,460, including the upper approach to these locks. It would probably take eight years to excavate this section of the canal.

The amount of excavation in this section is 43,237,200 cubic yards. The concentration of so large an amount of excavation in so small a space is without precedent. The engineer will recognize at once that thorough organization and tools specially adapted to the work are required. Fortunately there is ample ground on which to deposit the spoil both north and south of the divide. The method of conducting the work in general principles and in detail should be thoroughly worked out before actual execution is begun. No work has ever been undertaken on which the highest class of practical engineering talent could produce so great economies as in this great concentrated excavation. Its cost has been estimated at 80 cents per cubic yard; bad management might easily increase this to a dollar, and it is not impossible that with a carefully considered equipment the cost might be reduced to 60 cents.

The Pedro Miguel locks (see pl. 25) will be similar to the Bohio locks, the aggregate lift varying from 54 to 62 feet. There is an excellent rock foundation here. The estimated cost of these locks, including an adjacent dam, is \$9,081,321.

A level 1.33 miles long extends from the Pedro Miguel locks to the last lock, which is at Miraflores. The normal elevation of the surface of the water is 28. The estimated cost of this section is \$1,192,286, including \$388,880 for lock approaches at each end.

At the end of this level will be located the Miraflores lock (see pl. 25), with a lift varying from 18 feet at high tide to 38 feet at mean low tide. There is a good rock foundation for this lock. A spillway will be required to regulate the height of this level. The estimated cost of this lock and spillway is \$5,781,401.

For 4.12 miles beyond the Miraflores lock the canal extends through a low swamp country through which the Rio Grande runs. Occasional rock is found here, but the material is generally very soft and the canal has been estimated for a bottom width of 150 feet with slopes of 1 on 3.

This brings the canal to a point known as La Boca where the Panama Railroad Company has constructed a large and substantial wharf. A dredged channel 200 feet wide with slopes of 1 on 3 will extend from this point 4.41 miles to the 6-fathom line in Panama Bay. The first 2 miles of this dredged channel are through flats which are bare at low water, where there is a considerable amount of submerged rock. The total cost of this section from the lock to deep water is estimated at \$12,427,971, of which \$1,464,513 is for work outside of La Boca. The cost of maintenance of this channel is included in that of the canal. No separate estimate for maintaining a harbor at Panama is submitted, because it is a natural roadstead, not requiring expenditure.

The Bohio dam is the most important structure on the line, being of great magnitude, of vital necessity to the scheme, and offering many difficulties of construction. The Commission has devoted much time to the procurement of full and reliable information concerning the foundation upon which this dam must rest, and to a study of the various types of structures which might be adopted.

The borings made by the French engineers upon and near the line of the dam as furnished to the Commission were 21 in number. In the central part of the valley they did not go down to rock. In this case the Commission decided to do more than verify the data furnished to it, and caused a large number (86) additional borings to be made. With the exception of seven, which were abandoned before completion on account of accidents to the apparatus or unusual difficulties of soil, all of these borings reached rock. They show a variety of materials—hard clay, soft clay, sand, gravel, and some mixtures of sand, clay and gravel in varying proportions. These materials are found in beds of varying shape and thickness, not distributed with uniformity and not arranged according to any general law from which can be deduced the character of the soil at points other than those actually examined. In every section constructed from the borings, strata of greater or less dimensions are found, which are permeable by water. How far these extend and whether or not they communicate with the surface of the ground above the site of the dam are points about which information can not be obtained in advance with certainty. If a dam be built with permeable strata under it there will probably be leakage, but what the amount of this will be is a question about which there is room for much difference of opinion. It would seem probable to many that the leakage will not be sufficient to endanger the water supply, and that an earthen dam is therefore feasible, but it is evident that here is a danger to be avoided if possible. A masonry dam founded throughout upon the rock, or an earth dam with a masonry core going down everywhere to rock, would close the valley completely and would leave no question open as to its future efficiency.

In its preliminary report the Commission based its estimates on a masonry dam. The examinations of the ground had not at that time been completed. So far as they had progressed they showed a site where a masonry dam seemed the most suitable, but it was subsequently found that the depth to rock upon that site was at least 143 feet below sea level at the deepest part. It was considered best to avoid, if possible, so great a depth of foundation. A site was found a few hundred feet farther downstream where the length of the dam would be considerably greater

than at the former site, but the greatest depth to rock revealed by the borings was only 128 feet^a below sea level. The line runs from a point near the railroad station at Bohio, on the east side of the river, straight across to the rocky hill on the west side. (See pl. 26.) On the east side the rock is at the surface practically from the water in the river to the end of the dam. On the west side the bank above low water is composed either of pure clay or of clay mixed with sand, while below low water are found irregular beds of sand and sandy clay. The physical features of the location admit of the construction of an earth embankment with a heavy masonry core carried down to bed rock throughout the length of the structure. For reasons of economy that type of dam is preferable to one wholly of masonry upon the new site, and is now adopted.

It is proposed to sink the foundation of the core wall by the pneumatic process at all points where the foundation bed is lower than about 30 feet below mean sea level. This requires the pneumatic process to be used through a length of 1,314 feet, of which about 310 feet is at the maximum depth of 128 feet below the sea level. Where the foundation bed is above elevation — 30, cofferdams are to be used. This involves the use of cofferdams through a length of 324 feet, the foundation at sea level being extended 78 feet at the easterly end and 246 feet at the westerly end of the pneumatic work. The cofferdams extend to a height 8 feet above sea level. Above elevation 8 all operations would be carried on by the ordinary methods of dry work.

The width of the dam at the top is 20 feet, and its total length is 2,546 feet. The elevation of the top is 100 feet above mean sea level, affording a superelevation of the dam of 8 feet above the highest possible water in the lake and 10 feet above the usual high water. Its total height above the lowest part of the foundation is 228 feet. The earth faces of the dam are designed to have mean slopes of one vertical to three horizontal, and to be broken by three terraces, each 6 feet wide. It is necessary to pave only the upstream face, but it is probable that both faces would be heavily riprapped with the rock spoil from the lock excavation near the westerly end. The masonry core is 30 feet thick at and below elevation — 30. From that level it tapers to a thickness of 8 feet at top.

Material for the heavy fill required is found in the immediate neighborhood. The local conditions are such that not less than seven-eighths of the work could be completed without interfering with the natural flow of the Chagres. When it becomes necessary for the completion of the dam to divert the river, the unfinished Gigante spillway and, later on, the finished locks at Bohio may be employed as diversion channels. A temporary dam would be required to turn the water through these outlets at suitable stages. This temporary dam may be placed either at the site of the permanent dam and finally be buried in it, or at some suitable point higher upstream. The cost of the Bohio dam is estimated at \$6,369,640. This estimate is higher than any which has hitherto been made for this dam. It is possible that before actual construction a better location can be found and the cost reduced. A dam on the French location, with masonry core carried to rock, would contain less than half the material in the dam for which estimates have been made, but for a length of 170 feet the foundation would be deeper than anywhere on the adopted location, the maximum being 146 feet below mean tide.

The Gigante spillway, which is a structure of considerable magnitude, is very simple. There is a good rock foundation at or above tide level for the entire length of this spillway. It will consist of a dam entirely of concrete with a crest at elevation 85, terminating in an apron at elevation 65, with a solid foundation below this level, the apron being everywhere below the present surface of the ground. The foundation, below elevation 65, will be put in first and before the flow of water through the present river at the site of the Bohio dam is checked. Plans for this spillway are shown on plate 27. The estimated cost, including the channel ways immediately above and below it, is \$1,209,419.

^a Borings made in 1904 developed the fact that the maximum depth to rock on the line here recommended is 168.3 feet instead of 128 feet, and that the depth upon the line previously recommended is 163.6 feet instead of 143 feet. The latter is the line selected by the Comité Technique in its report of November 16, 1898. These great depths preclude the method of construction proposed in the report, viz, a core wall of masonry sunk to bed rock by the pneumatic process, and they make the French location the better of the two. They do not otherwise affect the merit of the plan. The core wall was not designed to give stability to the dam, but to make it water-tight, particularly in the subfoundation. This must be accomplished by other methods. Probably the simplest and easiest method will be by grouting with liquid cement. The new steel sheet piling offers another simple method. Still other methods have been proposed; they are feasible, but less simple than those mentioned.

The water after passing over the spillway will flow across country about a mile to the Peña Blanca swamp. The elevation of the surface of this swamp is now 22.3, so that the water will have a fall of 62.7 feet in this mile. The swamp is separated from the line of the canal by a ridge, of which the lowest part is at elevation 33. It can be filled to elevation 31 without inconvenience to the canal. Its outlet will be placed at elevation 11. Its area at that level is 0.447 square mile and at elevation 31 it is 1.186 square miles. Its storage capacity between those two levels is 455,000,000 cubic feet.

From the Peña Blanca the water will be discharged into the Agua Clara swamp by an artificial channel, located about a mile from the canal, cut through the ridge which separates the two swamps. This channel will have a bottom width of 500 feet and a depth of 20 feet when the water in Peña Blanca swamp stands at elevation 31. Its length measured on the bottom is 5,676 feet and the maximum depth of cutting is 63 feet, much of it rock. The bottom has a fall of 2.2 feet from one swamp to the other. This channel will discharge 75,000 cubic feet per second with a mean velocity of 7.5 feet per second, or 5 miles per hour, and this it can attain if the difference of level between the two swamps be 2.1 feet. With a head of 7.3 feet it can discharge over 140,000 cubic feet per second, at a velocity of about 9.55 miles per hour. Its cost is estimated at \$2,448,076.

The present surface of the Agua Clara swamp is at elevation 11.5, and its area at that level is about 2.9 square miles. Farther on are other swamps—the Vino Tinto and the Bruja. The ground separating them from each other and from the line of the canal is low, affording a wide area for the spread of flood waters, until the neighborhood of Gatun is reached. Here the width of the valley contracts and its direction changes, so that an artificial channel becomes necessary to divert the Chagres. A channel was cut here by the old canal company, but it is of inadequate dimensions, and in order to avoid some rock excavation it was given an awkward shape. A new channel will be cut on a straight line, part of which will be an enlargement of the present one. It will have a width at bottom of 650 feet, and will be excavated to a depth 8.5 feet below the datum plane, which will give it an area of cross section of about 10,000 square feet when the surface of the water is at elevation 7. Its length measured on the bottom is 6,955 feet, and the maximum depth of cutting is 25 feet, much of it rock. With a head of 11 feet it can discharge 140,000 cubic feet per second at a velocity of about 9.55 miles per hour. Its cost is estimated at \$1,929,982.

The canal in the low region above and below Gatun must be protected from overflow by levees, their total length aggregating about 5.4 miles. The height to which these levees should be carried can not be determined with accuracy from the present data, and must be fixed from observations of floods hereafter. As in all other cases of doubt, a height has been adopted which will err, if at all, upon the safe side. For the purpose of estimate the height has been placed at elevation 25. The width on top is 13 feet and the side slopes 1 on 2. It is probable, however, that the levees will be used as spoil banks for the material dredged from the canal, and that their dimensions, except as to height, will much exceed those here given.

From Gatun the overflow from Lake Bohio and all tributaries below the lake on the west side of the canal will find its way to the sea through the Chagres River, which the canal here leaves.

The only tributary on the east side for which any special provision need be made is the Gatun. A diversion channel intended to take a portion of the water of the Chagres was constructed by the old company along the east side of the canal to Boca Grande back of Colon. It cuts across the Gatun near the town of the same name, and while no longer required for the Chagres, it is available as a new channel for the Gatun. It was designed to carry a discharge of 17,600 cubic feet per second, which is much in excess of the maximum discharge of the Gatun. Some work must be done on it, especially at the crossing of the Panama Railroad, where the piers for a new bridge have been built. The cost of putting this channel into service is estimated at \$100,000.

From Bohio to the Obispo gates the Panama Railroad must be rebuilt for 15.5 miles on a

new location, with a bridge across the Chagres below Gamboa. An estimate made from approximate profiles indicates that the cost of this diversion will not exceed \$75,000 a mile, or \$1,162,500. From the Obispo gates the railroad will be carried for 6 miles on the bench formed by the retaining wall on the east side of the Culebra Cut, these 6 miles being estimated to cost \$10,000 a mile, which includes only track laying, ties, and ballasting. Beyond this will be a mile of light work estimated at \$25,000, while the main track will have to be raised for 2 miles farther at a cost of \$20,000. Combining these figures, the total cost of the diversion of the Panama Railroad becomes \$1,267,500.

Summing up the several figures already given, the total estimated cost of completing the Panama Canal is as follows:

Total estimated cost.

	Miles.	Cost.
Colon entrance and harbor.....	2.39	\$8,057,707
Harbor to Bohio locks, including levees.....	14.42	11,099,839
Bohio locks, including excavation.....	.35	11,567,275
Lake Bohio.....	13.61	2,952,154
Obispo gates.....		295,434
Culebra section.....	7.91	44,414,460
Pedro Miguel locks, including excavation and dam.....	.35	9,081,321
Pedro Miguel level.....	1.33	1,192,286
Miraflores locks, including excavation and spillway.....	.20	5,781,401
Pacific level.....	8.53	12,427,971
Bohio dam.....		6,369,640
Gigante spillway.....		1,209,419
Peña Blanca outlet.....		2,448,076
Chagres diversion.....		1,929,982
Gatun diversion.....		100,000
Panama Railroad diversion.....		1,267,500
Total.....	49.09	120,194,465
Engineering, police, sanitation, and general contingencies, 20 per cent.....		24,038,893
Aggregate.....		144,233,358

The total amount of excavation is 94,863,703 cubic yards, exclusive of excavation for the Bohio dam and the Gigante spillway.

The location of the canal is, in general, the same as that proposed by the French company. Its total length, from 36 feet depth in the Atlantic to 36 feet depth in the Pacific, is 49.09 miles. The distance from the inner end of the harbor enlargement at Colon to the shore end of the bay channel at La Boca is 42.3 miles, of which 11 miles is the broad channel of

Length and curvature. Lake Bohio. The alignment is exceptionally good, the sharpest curve having a radius of 6,232 feet, except one at the entrance to Colon Harbor, which has a radius of 3,280 feet, but where the bottom width is from 500 to 800 feet. The total curvature in the entire length of the canal is $771^{\circ} 39'$, distributed as follows:

Number of curves.	Length.	Radius.	Total curvature.	Number of curves.	Length.	Radius.	Total curvature.
	<i>Miles.</i>	<i>Feet.</i>	<i>° /</i>		<i>Miles.</i>	<i>Feet.</i>	<i>° /</i>
One	0.88	19,629	14 17	Two	1.67	6,562	77 00
One	.48	13,123	11 04	One.....	.73	6,234	35 45
Four.....	4.22	11,483	111 32	One.....	.82	3,281	75 51
Fifteen.....	11.61	9,842	355 50				
Four.....	2.44	8,202	90 20	Total	22.85		771 39

The time required to pass through the canal after completion will vary with the size of the vessel and with the number of other vessels. For the purpose of comparison the time has been carefully computed for a ship 400 feet long, 50 feet beam, and 24.5 feet draft, or what may be called an average-sized ship. The open sea speed is taken at 12.5 statute miles, or about 11 knots per hour. The reduced speed in various parts of the canal and the delays caused by lockages and by passing other vessels have been obtained from observation of the practical working of the Sault Ste. Marie and the Manchester canals. They are as

Time of transit.

follows: In canal sections having a bottom width 150 feet, speed on tangents 8 miles per hour, on curves 7 miles per hour; in Panama Bay channel, speed 9 miles per hour; in Lake Bohio, speed on tangents 10 miles per hour, on curves 9 miles per hour; in harbors and harbor entrances, speed on tangents 10 miles per hour, on curves $8\frac{1}{2}$ miles per hour; all statute miles. The delay caused by lockages is 3 hours and 58 minutes, and that caused by meeting other vessels 1 hour and 14 minutes. From these data the time of transit through the canal is computed to be 11 hours and 14 minutes. A full discussion of this subject will be found in Appendix G.

The plan recommended by the Commission is, in its general outlines, the same as the second plan of the French engineers, the one preferred by them, except for the time required for construction. The principal difference is in the height given to the Bohio dam and the important consequences which result therefrom. A marked feature of the Commission's plan is its simplicity. The increase in the depth and area of Lake Bohio renders it possible to receive the full flood discharge of the Chagres directly into it without impeding navigation and at the same time to take full advantage of favorable topographical features of the country in the subsequent discharge of the surplus waters. The Alhajuela dam becomes unnecessary for flood control, and its construction may be deferred until additional storage capacity is required as the result of a large increase in the traffic of the canal in the future. The outlet of Lake Bohio becomes a single fixed weir instead of two weirs with regulating gates and with two separate channels to the sea. A great reduction also results in the amount of excavation required to cut through the continental divide. There is a material reduction of cost.

Advantages of the Commission's plan.

DIMENSIONS AND UNIT PRICES.

In fixing the dimensions of the canal, it is necessary to consider carefully the dimensions of the ships which will use it; the prevailing as well as the exceptional types of the present day, and the probable developments of the near future. If the dimensions adopted are too large, the cost of the work will be augmented unnecessarily; if too small, the canal will not fulfill its intended purpose.

The greater part of the world's commerce by sea is carried on by ships of moderate size. Lloyd's Register for 1900-1901 contains the names and dimensions of 16,264 steam vessels of all kinds, of which about 8,900 are more than 200 feet long on the keel. This number may be taken as approximately the number of cargo vessels. Only 421 seagoing ships have as great a beam as 50 feet, and only about 800 would require a lock more than 400 feet long. Until recently the larger ships were employed mainly on the North Atlantic route, and the largest were fast passenger ships not adapted to any other trade. The building of large freight ships with more or less passenger accommodations has now become a marked feature of the development of shipbuilding. In the years

Dimensions of ships now
in use.

Recent increase in ship
dimensions.

1897 to 1900 there were put in service on the North Atlantic route several ships of this type, about 600 feet long over all, 63 to 65 feet beam, and drawing, when fully loaded, 30 to 32 feet. These were followed by the White Star ship *Celtic*, recently built, said to be 698 feet 8 inches long over all, and 75 feet 4½ inches beam. On other routes ships of similar dimensions are being introduced, the largest now in use being the White Star ships of the *Afric* class, built for the British colonial service via the Cape of Good Hope, which are about 550 feet long between perpendiculars, 64 feet beam, and have a load draft of 32½ feet, and the Cape mail steamship *Saxon*, built for the Union Company, which is 585 feet 6 inches long over all, and 64 feet beam. The ships now building at New London for the Great Northern Railway Company are particularly noteworthy, being designed for the trans-Pacific trade; they are to be 630 feet long and 73 feet beam. They will be in the same class as the *Celtic*.

The steady growth in ships' dimensions, the introduction of large ships on so many different routes, and the undoubted fact that for long routes and for freight which can be quickly handled

Depth of canal.

at the ports the large ship is the more economical carrier, gives these ships an importance in determining canal dimensions much greater than their relative number or aggregate tonnage would indicate. A ship drawing 32½ feet in salt water, which is no longer exceptional, will draw nearly 1 foot more in fresh water, and requires for safe navigation not less than 35 feet of water in the canal. This depth is therefore fixed as the minimum in all the channels.

In fixing the width of locks and prism it is not necessary to take into account the fast passenger ships of the North Atlantic routes. Such a trade is not likely to develop through the Isthmus. Limiting the inquiry to freight or combined freight and passenger ships like those mentioned, it will be noted that the maximum beam of 73 to 76 feet is found

Beam of commercial ships.

in very few ships; excepting these, the greatest is 63 to 65 feet, which is found in quite a numerous class. If the canal were intended for commercial uses only, it might be questioned whether dimensions should be fixed for the extreme beam of 75 feet or more, with the added cost of construction and minor disadvantages, but the imperative requirement that the

anal shall afford a passage for the largest war ships makes it necessary to provide for a beam considerably greater. The broadest ships building for the United States Navy are those of the *Virginia* class, which have a beam of 76 feet and 2½ inches. The broadest battle ship afloat is the Italian ship *Regina Margherita*, recently launched, which has a beam of 78.2 feet. While the increase in beam of war ships has for some years been less rapid than that of commercial ships, it is unmistakable. For convenience in operating the locks the width should be 2 or 3 feet greater than the beam of the ship. The width is therefore fixed at 84 feet with a view to provide for some further increase in beam of ships.^a

The largest ships of war are shorter than commercial ships of like beam, and a clear length of lock chamber of 600 feet would be sufficient for any war ship now afloat or building. In order to make the canal practicable for the largest existing commercial ships, and also to provide for a considerable increase in size, the only additional expense to be incurred in the building of the canal, after providing for war-ship requirements, is to increase the length of the locks. This added cost is so small in comparison with the advantage gained that it is unquestionably judicious to incur it, and the length is therefore fixed at 740 feet in the clear.^a

The width of the canal has been fixed after careful consideration of the dimensions adopted and experience gained elsewhere. The Suez Canal had originally a bottom width of 22 meters (72 feet 2 inches). It had been intended, when the work was projected, to make this width 44 meters (144 feet 4 inches) between the Bitter Lakes and the Mediterranean, and 64 meters (209 feet 11 inches) between the Bitter Lakes and the Red Sea, but the resources of the company proved insufficient to carry out the work on this plan. The width finally adopted proved inconveniently small, and it has since been increased to about 115 feet. At the same time the depth has been increased from 26 feet 3 inches to 27 feet 10 inches. The ratio of present width to depth is about 4:1. In the Manchester Canal the depth is 26 feet, but is to be increased to 28 feet. When this is done the bottom width will be about 114 feet, and the ratio of width to depth will also be about 4:1. The Amsterdam Canal is at present 36 meters (118 feet 1 inch) wide at the bottom and 8.5 meters (27 feet 10 inches) deep, giving a ratio of width to depth of 4:2. These dimensions are to be increased to 50 meters (164 feet) width and 9.8 meters (32 feet 2 inches) depth, with a ratio of 5:1. The bottom width of 150 feet, which has been adopted by the Commission for the canal sections of the Isthmian routes, gives a ratio of width to depth of 4:3, which is slightly greater than at the Suez, Manchester, and Amsterdam canals, and considerably less than the enlarged Amsterdam Canal will give.

Mention should be made of the Kiel Canal, which has the bottom width first given to the Suez Canal, viz, 72 feet, and has a depth of 29.5 feet, giving a ratio of width to depth of 2:5. This has not been taken into account in the preceding comparison, for the reason that the width is clearly shown to be insufficient for commercial purposes by experience at Suez. The Kiel Canal was built primarily for an outlet to the North Sea for the German navy, and is adapted for the possible transfer of war ships rather than for the convenience of commercial ones. Few commercial ships of large size traverse it. The average tonnage of the vessels passing in 1899 was about 100 tons, which may be compared with the average of nearly 4,000 tons for the ships passing through the Suez Canal during the same year.

The side slopes of the Isthmian Canal sections vary with the materials. In soft earth or sand they are taken 1 on 3 below water and 1 on 2 above water; in firm earth, 2 on 3 below a berme 10 feet wide 6 feet under water and 1 on 1 above such berme. The 1 on 1 slopes are to be protected by paving from the berme to 6 feet above water. In rock the sides are vertical from the bottom to a berme 5 feet

^a Two vessels are now under construction—the new Cunarders—of which the dimensions are: Length over all, 800 feet; beam, 88 feet, and extreme draft, loaded, 36 feet. Enlarged dimensions of the lock chambers and of the cross section of the canal prism in the Culebra Cut will no doubt be required.

above water, with slopes of 4 on 1 in hard rock and 2 on 1 in soft rock above such berme, the berme being of such width that the extended slopes would intersect the bottom of the canal at the foot of the vertical sides. In several places a slope of 1 on 1 is used, as in the Culebra Cut, on account of the peculiar nature of the material.

Where channels are excavated in open water and the sides will be submerged, the width is made greater. In Panama Bay the bottom width is to be 200 feet, with side slopes of 1 on 3, but at mean tide the width 35 feet below water will be 260 feet and at high tide 320 feet. In the artificial harbor at Colon it will be 500 feet, with turning places 800 feet wide.

The channel widths above given are for straight sections. On curves of less than 12,000 feet radius, in channels less than 500 feet wide, the width is increased at the rate of 1 foot for each 200 feet reduction of radius, the widening on a curve of 6,000 feet radius being 30 feet. This is an arbitrary allowance. It is the same as the allowance in Kiel Canal for a radius of 5,000 feet; less than in the Kiel Canal for radii under 5,000 feet and more for radii over 5,000 feet.

As already stated, the locks are to have a clear length of 740 feet and a width of 84 feet between the side walls. The depth over the head wall and over the miter sills at the lower end of the locks, which fix the available depth for ships, is to be 35 feet, the same as in the prism of the canal. The miter sills at the head of the locks are placed 1 foot lower, the slightly greater safety thus afforded for these sills being secured by merely exchanging 1 foot in height of gate for 1 foot in height of miter sill wall and without appreciable cost. In order to give the required clear length, all single locks and the upper locks of combined systems are to be made 788 feet long from quoin to quoin. The lower locks of combined systems will be 793 feet from quoin to quoin, the greater length being due to the greater thickness of the cross wall at the middle gates.

Twin locks are provided in every case. This will insure uninterrupted navigation if one lock at any locality is closed for repairs. To facilitate making these repairs with the least possible delay, guard gates of the ordinary miter form are placed at both ends of every lock or flight of locks, those at the foot opening downstream. When repairs to the lock are needed, these gates can be closed and the lock pumped out immediately, thus avoiding the delay of building cofferdams or the uncertainties attending the use of caisson gates. This provision is not usual, but has been adopted for all the locks of the St. Marys Falls canals, and its utility has been frequently demonstrated there. These gates are supported by extensions of the lock walls. The extreme length of the masonry is 1,031.5 feet for a single lock and 1,829.5 for a flight of two locks.

While these locks provide for the passage of the largest ships anticipated in the near future, it is realized that the larger part of the sea traffic of the present day is carried on by much smaller ships.

Smaller locks than those adopted could be operated more quickly and would effect a material aggregate saving of time to the greater part of the traffic and reduce the amount of water consumed. The width of the locks can not be reduced without excluding the large ships. It is practicable, however, to provide a shorter lock by the introduction of intermediate gates, whereby two smaller chambers can be obtained and some of the advantages secured of having a small lock for small ships. By the arrangement shown in the plans the whole chamber can be used, the intermediate gates remaining open; by using these gates in connection with the upper lock gates, a chamber is provided having a clear length of 292 feet, and by using them in connection with the lower lock gates, a chamber is provided having a clear length of 400 feet, sufficient for most of the freight ships now in use. With two locks combined in a flight, only one of the smaller chambers in each lock is available, the intermediate gates being so placed that either the full

length or the 400-foot chamber can be used. It may be found expedient in construction to make the length of the reduced chamber 450 feet.

All the locks on both routes will have rock foundations. The rock varies greatly from hard to soft and partially disintegrated. The poorest will carry safely the imposed load, but will permit slow seepage for considerable distances, and will offer little resistance to abrasion. The floors of the locks are protected by inverts of concrete, the thickness being greater in soft rock. In the vicinity of the lock gates the floor is of sufficient thickness to sustain the full hydrostatic head.

The walls are also to be of concrete throughout, except the quoins, the tops of the miter-sill walls, and exposed angles at the inlets and outlets of the culverts. The use of concrete for the construction of locks is of comparatively recent introduction. The locks of the Manchester Canal are perhaps the most notable example, the great mass of masonry being concrete, granite being used only for quoins, copings, and exposed angles, and brick burned to the point of incipient vitrification for facing the walls above low water and for culvert linings. The moist, warm climate of the Isthmus is particularly favorable for concrete.

In the locks of greatest lift the velocity of water in the culverts will reach 50 to 60 feet per second, which would severely test any masonry, even of the best brick or cut stone. Even in locks of the smaller lifts some kind of protection for the surfaces of the culverts will probably be necessary. As a basis for estimates a lining of cast iron of a minimum thickness of 1 inch is provided where the extreme head of water in the culverts exceeds 30 feet and a lining of vitrified brick for smaller heads.

The gates are designed of steel of the ordinary miter form. They are based on actual designs of gates of nearly equal dimensions prepared under the direction of the United States Board of Engineers on Deep Waterways.

In designing the locks the varying height of rock at each of the lock sites has been taken into account. The details of the studies concerning the stability of the walls, as well as of the loss of water by leakage, etc., are given in Appendix A.

In order to facilitate the movement of ships into the locks, as well as to afford a safe place for ships to tie up while awaiting lockage, a vertical approach wall 1,200 feet long is provided on one side of the canal at each end of every lock or flight.

Unit prices have been fixed by agreement of all the members of the Commission, on the principle that, whatever differences of opinion or circumstances may exist, they are not enough to interfere with a fair and close comparison of the different routes. These prices are as follows:

Removal of hard rock, per cubic yard	\$1. 15
Removal of soft rock, per cubic yard	. 80
Removal of earth, not handled by dredge, per cubic yard	. 45
Removal of dredgable material, per cubic yard	. 20
Removal of rock, under water, per cubic yard	4. 75
Embankments and back filling, per cubic yard	. 60
Rock in jetty construction, per cubic yard	2. 50
Stone pitching, including necessary backing, per square yard	2. 00
Clearing and grubbing, per acre	100. 00
Concrete, in place, per cubic yard	8. 00
Concrete, in place, per cubic yard	8. 00
Finished granite, per cubic yard	60. 00
Brick in culvert lining, per cubic yard	15. 00
All metal in locks, exclusive of machinery and culvert linings, per pound	. 075
All metal in sluices, per pound	. 075
Cast iron in culvert lining, per pound	. 04
Allowance for each lock chamber for operating machinery	50,000. 00
Additional allowance for each group of locks for power plant	100,000. 00

Price of timber in locks, per M. B. M.	\$100. 00
Sheet piling in spillways, per M. B. M.	75. 00
Bearing piles in spillways, per linear foot.....	. 50
Average price of pneumatic work for the Bohio Dam, below elevation —30, per cubic yard.....	29. 50

It has been determined to add 20 per cent to the estimates of the cost of construction to cover expenses that will be incurred for engineering, police, sanitation, and general contingencies. The prices are based on efficient organization and thorough equipment, with the understanding that while the work would be vigorously handled it would not be so driven as to call for unnecessary duplication of machinery. The cost of the equipment and plant will be large, but it will be distributed over a very large work.

APPENDIX A.
A STUDY OF LOCKS FOR NICARAGUA AND PANAMA ROUTES.

By Mr. S. H. WOODARD.

The following contains a general description of the methods of calculation of the stresses and stability of the walls, floors, and various other parts of the locks and memoranda of the assumptions made, the stresses computed, the dimensions determined, and the estimates of cost of the locks designed for the proposed Nicaragua and Panama canals.

The designs and computations were made under the direction of the Commission's Committee on Locks.

The general drawings (pls. 24, 25, 64, 65, 66) show the principal features of the locks. They are designed to be constructed entirely of concrete with the exception of the gate sills and quoins of the gate recesses.

Twin locks were designed in all cases for both the Panama and Nicaragua canals. Estimates were made, however, of the cost of building a single lock system in such a way that the second lock could be added at some future time by simply building a floor and one side wall.

The general dimensions of the locks are: Clear length, 740 feet; clear width, 84 feet; and depth, 35 feet.

The total length of the lock masonry depends upon the kind, number, and arrangement of the gates, as well as upon the clear length required.

The lateral dimensions of the lock walls depend upon the height of the walls and upon local conditions.

LOCK GATES.

The United States Board of Engineers on Deep Waterways have recently completed a somewhat exhaustive study of steel lock gates. In their investigation of the various plans for a canal from the Great Lakes to tide water it was necessary for them to make estimates of cost of several hundred gates for locks varying in width from 60 feet to 80 feet and lifts varying from zero to 50 feet.

From the bills of material of a large number of detailed designs an empirical formula was developed which expressed the weight of a gate in terms of the width of the lock and depths of the water on the two sides of the gate. Although the formula is purely empirical, and is strictly applicable only within the limits of the data from which it was derived, it checks quite closely the weight of gates 100 feet wide. As the width of the proposed Panama and Nicaragua locks is only 4 feet greater than the widest lock from which the formula was derived, it is believed that the formula is applicable to them. Probably the difference between the weights given by the formula and those that would be obtained if complete designs were made of the gates for the cases in hand would not be greater than that resulting from the designs of two equally competent men.

It was decided by the Committee on Locks to make use of the work of the United States Board of Engineers on Deep Waterways as a basis of estimate, and to design the lock masonry with a view to the use of the same type of gate as was developed by that board. This type is the steel mitring gate, with horizontal framing, straight on the downstream side and curved on the other. The rise of sill is one-fifth the width of the lock, or, in other words, when the gate is closed the line which joins the centers of the quoin and miter posts makes an angle with the

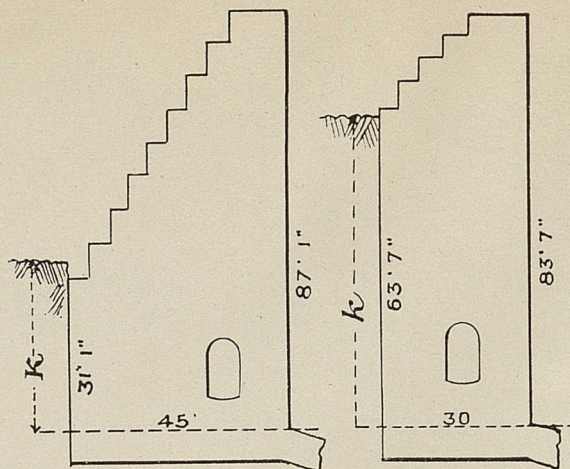


Fig. 1

Fig. 2

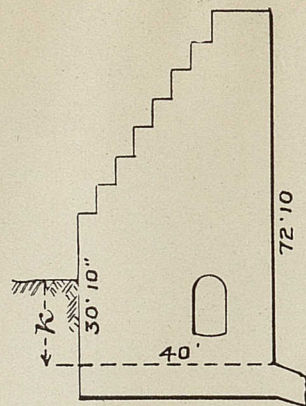


Fig. 3

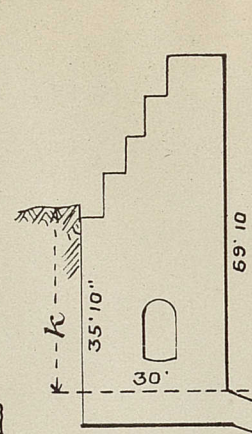


Fig. 4

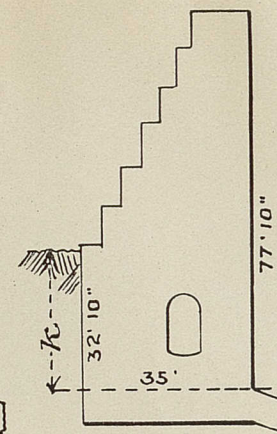


Fig. 5

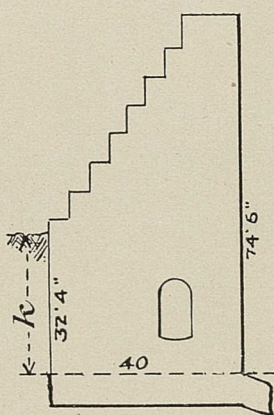


Fig. 6

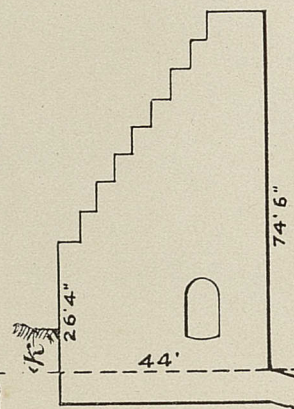


Fig. 7

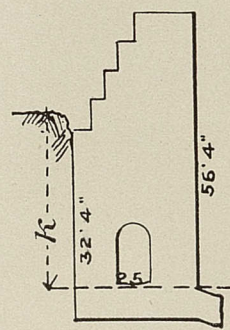


Fig. 8

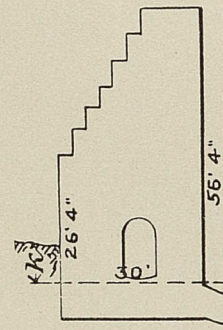


Fig. 9

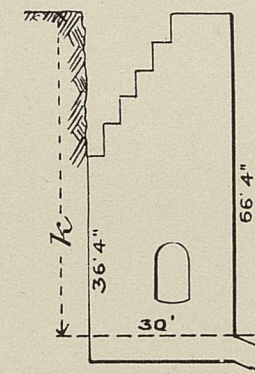


Fig. 10

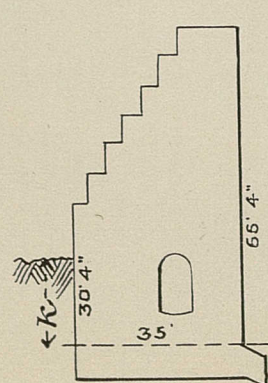


Fig. 11

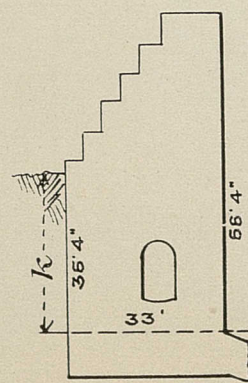


Fig. 12

Sections of Side Walls
OF PANAMA & NICARAGUA LOCKS

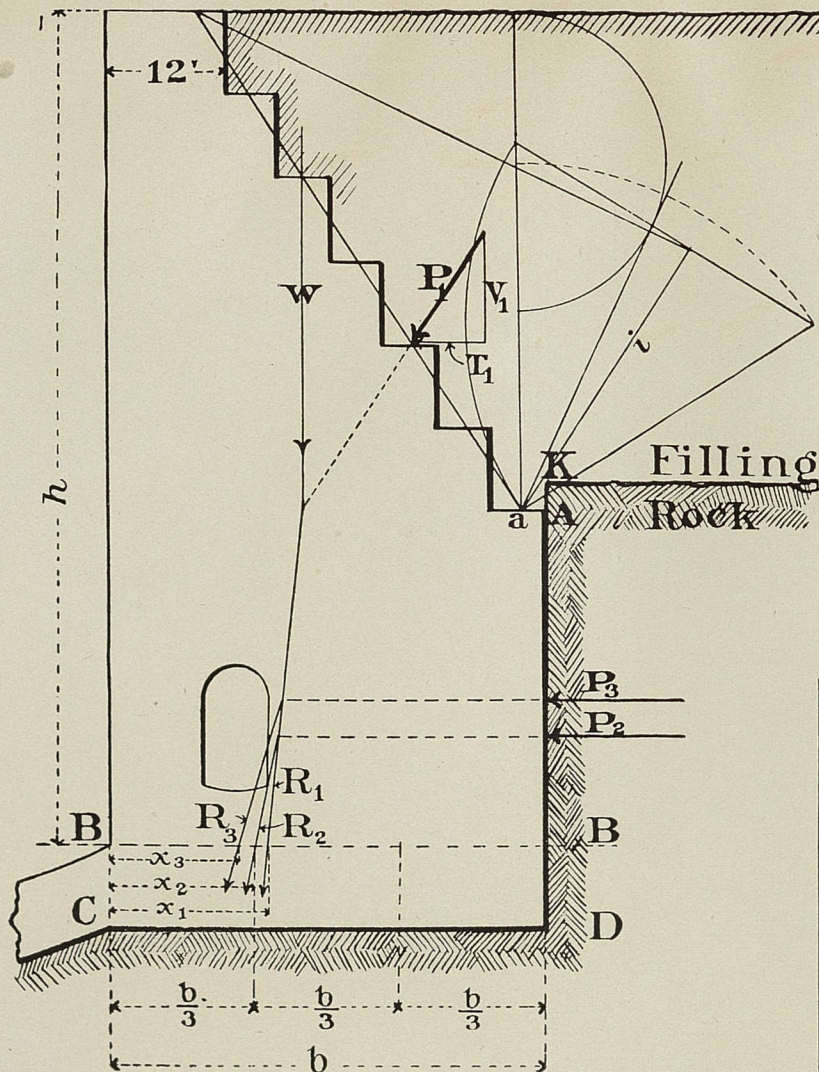


Fig. 13

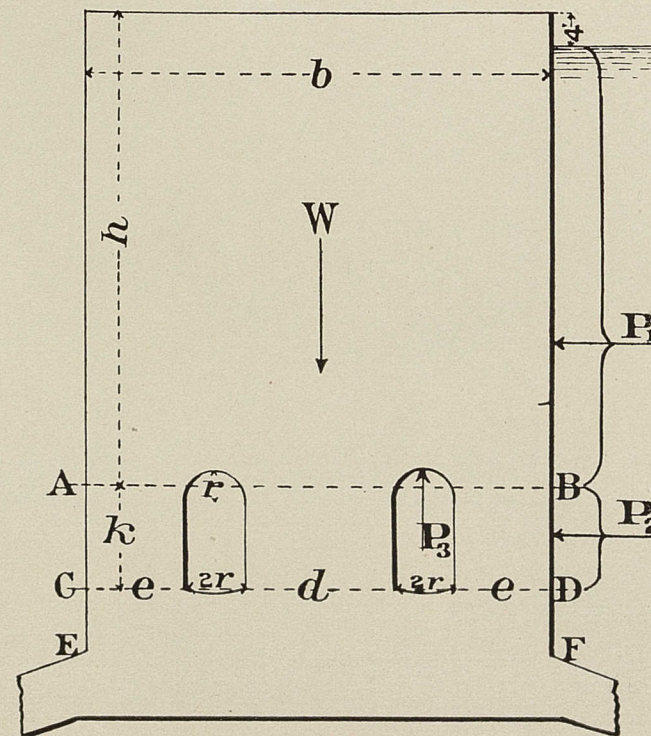
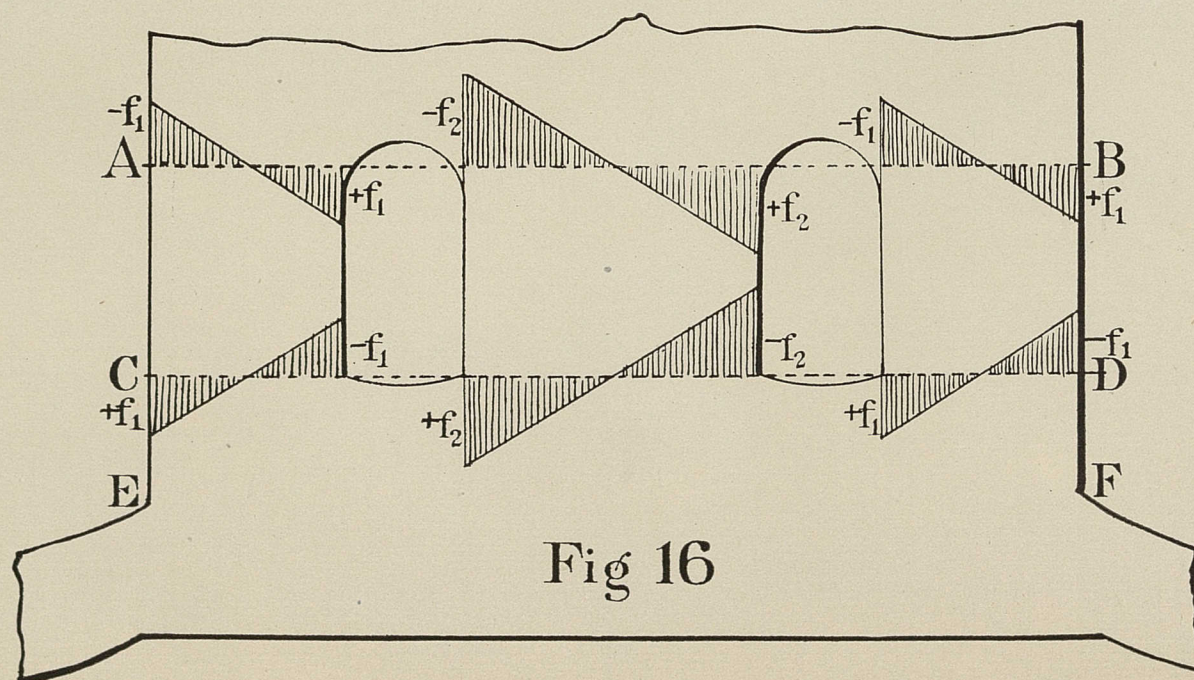
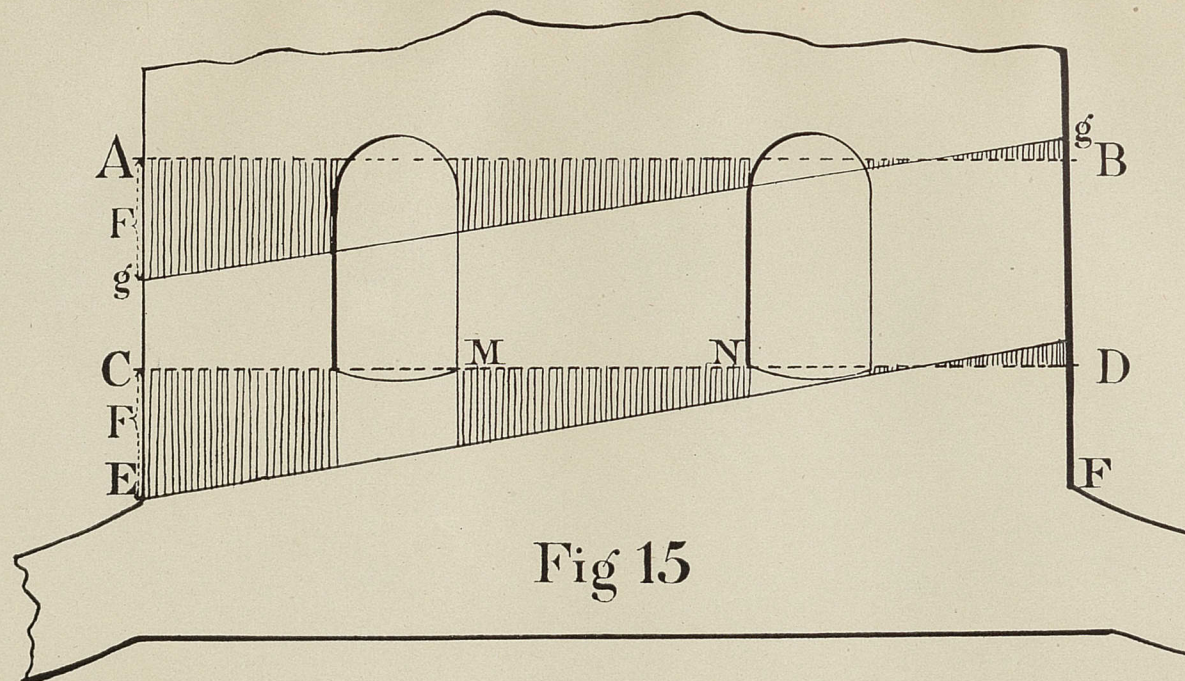
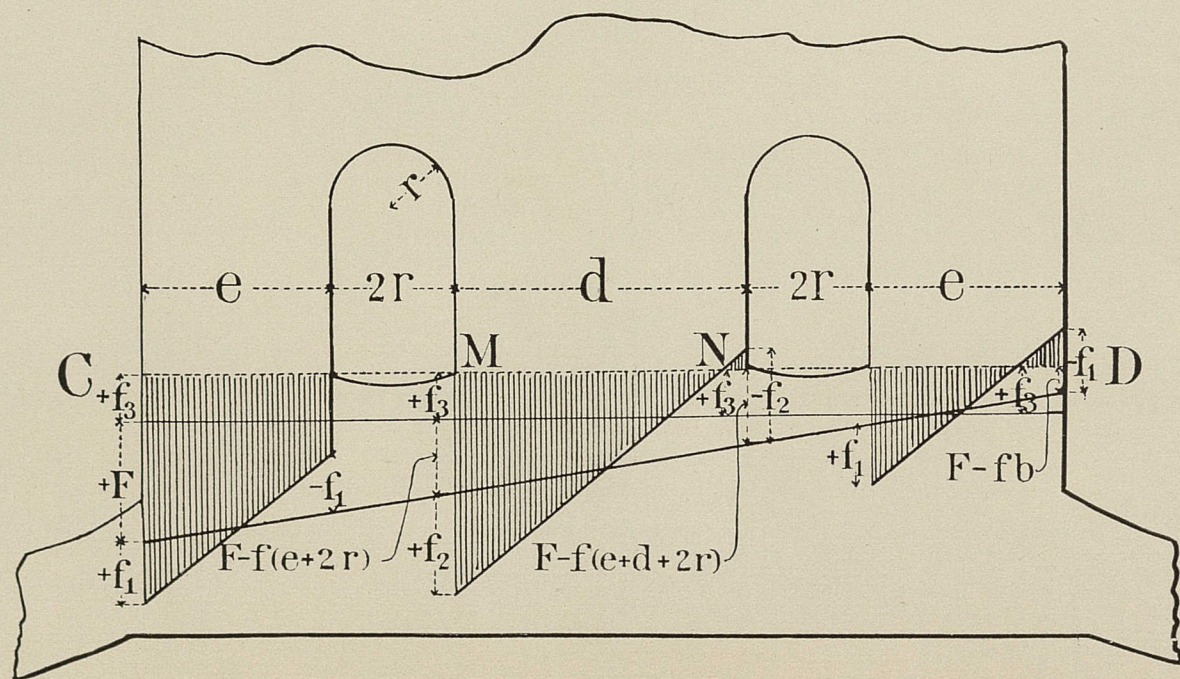
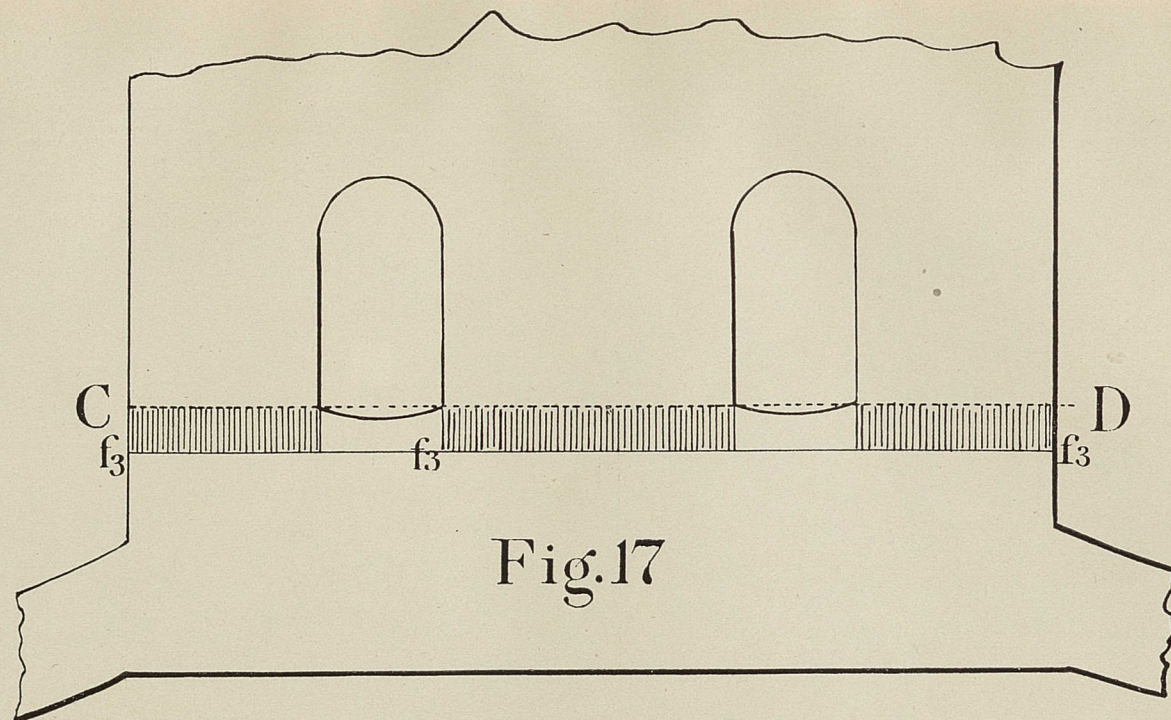


Fig.14





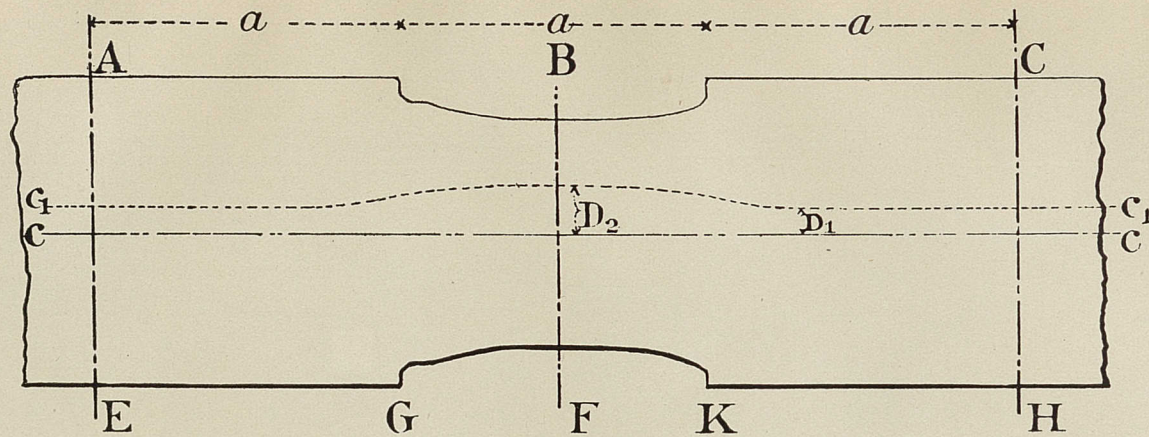


Fig. 20

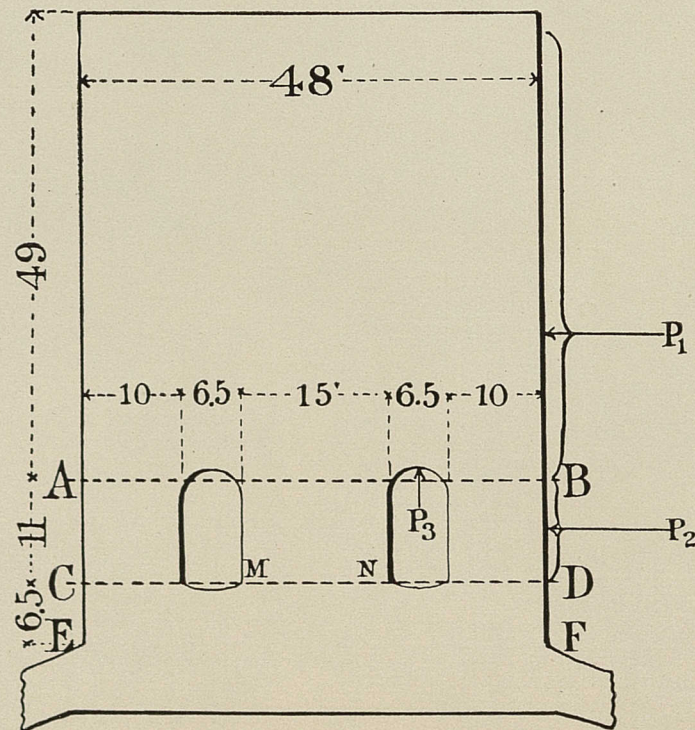


Fig. 19

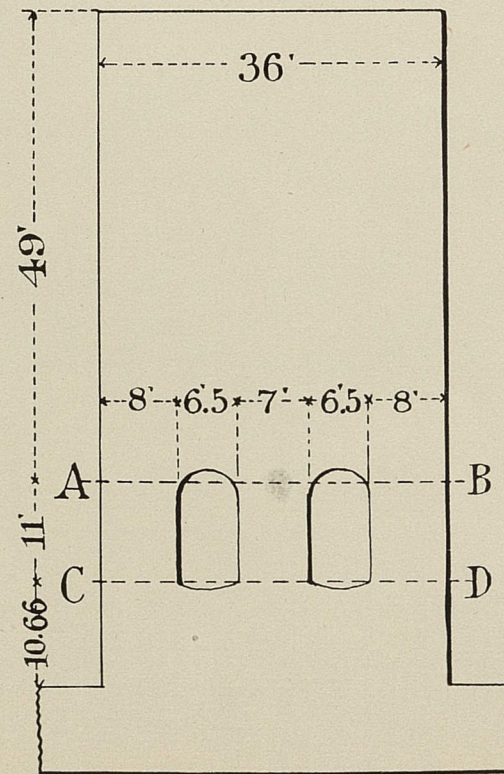


Fig. 21

normal to the side wall the tangent of which is 0.4. With these gates it is necessary to make the distance between centers of hollow quoins of upper and lower gates 48 feet greater than the required clear length of lock.

In general, the gates are sheathed on the upstream side only, but in a few cases, where the gates are extremely heavy, they are made partially buoyant by sheathing the downstream face below the level of the lower pool.

Each lock is provided with upper and lower gates and an intermediate gate, which is to be used as a lower gate when passing small vessels. The intermediate gate forms a lock chamber with a clear length of 400 feet. Besides these gates each lock or flight of locks is provided with upper and lower guard gates. To provide room for the gates the total length of the masonry becomes 1,031.5 feet for the single locks and 1,829.5 feet for a flight of two.

The formula (see Report of United States Board of Engineers on Deep Waterways, June, 1900) for the weight of lock gates is:

$$W = [(.000533 D + .0276) b^2 + (-.064 D - 1.876) b + 1.93 D + 51.75] H^2 + [(-.06 D + .557) b^2 + (10.022 D - 97.16) b - 352.44 D + 5848.3] H + (2.296 D - 33.215) b^2 + (-315.89 D + 5658.6) b + 13577 D - 200100.$$

In which:

W=the weight in pounds of the structural steel in one leaf of a single skin mitering gate with horizontal framing and straight on the downstream side.

b=breathth of the lock in feet.

H=the maximum head of water in feet which can act on the gate, or the maximum difference of elevation of water surfaces on the two sides of the gates.

D=the "depth on sill" or the depth in feet of the sill below the surface of the water on the downstream side of the gate.

In all the Panama and Nicaragua locks $b = 84$ feet. Substituting this in the equation, it becomes:

$$W = (.315 D + 88.9) H^2 + (66 D + 1617) H + 3243 D + 40900.$$

By substituting 35 for D in the above we get:

$$W = 99.925 H^2 + 3937 H + 154400$$

which gives the weight of one leaf of any lower gate of the Panama or Nicaragua routes.

By making $D=0$ we get:

$$W = 88.9 H^2 + 1617 H + 40900,$$

which gives the weight of one leaf of any upper lock gate or guard gate. Many of the upper gates are not ordinarily subject to so severe a condition as to be without water pressure on the downstream side, but it has been assumed that the occasion may arise when they will be used as guard gates.

The above formulas were used to make the estimates for all the gates, with the exception of the gates between the upper and lower locks of the Bohio and Pedro Miguel flights of locks on the Panama route. This gate is subject to a head of 84 feet at Bohio and 62 feet at Pedro Miguel. These heads are considerably greater than those for which the formula was developed, and in these two cases special estimates were made.

SIDE WALLS.

The masonry of all the locks rests upon a rock foundation, varying in quality from very hard to soft, quite porous, badly disintegrated rock. The poorest is, however, considered to

be a very good foundation so far as its ability to sustain the weight of the lock masonry is concerned.

Figures 1 to 12 show sections of the side walls of all the locks of both the Panama and Nicaragua routes.

The backing of the walls is, in most cases, the natural rock for a part of their height, above which filling of clay extends to the top of the wall.

The critical condition of the wall evidently arises when the lock is empty. At this time the forces acting on the wall are the weight of the masonry itself and the thrust of the clay back fill. There is, also, the possibility of there being water pressure acting along the joint between the masonry and the natural rock.

Except at the gates all side walls are made 12 feet wide on top and stepped off on the back, attaining their full width usually at the surface of the rock, but in no case is A B, fig. 13, less than 25 feet.

The angle of repose of the back filling above the rock surface is assumed to be 25° , and its weight 100 pounds per cubic foot. The masonry is assumed to weigh 140 pounds per cubic foot.

In fig. 13, P_1 is the resultant pressure of the filling.

This was determined in all cases by the graphical method given in Greene's Trusses and Arches, Part II, page 187. The horizontal and vertical components of P_1 are T_1 and V_1 . The horizontal component T_1 was checked in each case by Rankine's formula,

$$T_1 = \frac{w d^2}{2} \times \frac{1 - \sin a}{1 + \sin a}$$

in which

T_1 = the horizontal thrust of back filling in pounds.

W = the weight per cubic foot of filling.

d = depth of filling.

a = angle of repose of filling.

Both the analytical and graphical methods used are based on the theory of conjugate pressures and neither take account of the friction between the filling and the back of the masonry.

Although none of the theories for determining earth pressure is thoroughly established by actual observations of the intensities of pressure back of walls, it has been pretty well demonstrated by a great number of designs that the theory of conjugate pressures furnishes a safe working hypothesis.

R_1 is the resultant of the thrust of the filling and the weight of the masonry. All walls are made wide enough to make R_1 cut B B within its middle third.

If the conditions are such that A B will surely be a water-tight joint, then R_1 will be the resultant of all the forces acting above B B. In many of the locks of both the Panama and Nicaragua routes the natural rock back of the walls is quite soft and more or less porous. Provision has, therefore, been made for a certain amount of water pressure acting along A B in addition to the forces noted above.

The amount of the water pressure which should be considered as acting along A B depends upon the character of the rock and becomes a matter of judgment, and has been decided in each case by the Committee on Locks. In most cases it has been assumed that 50 per cent of the greatest possible water pressure might act. In other words, it is assumed that 50 per cent of the area along A B is an open joint and is connected with the upper pool. Under this assumption the resultant is, in some cases, allowed to fall a little outside the middle third of B B. No general rule was followed, however, each case being decided by itself.

TABLE I.

Lock.	Number of figure.				Lock.	Number of figure.			
	Wall A.	Wall B.	Wall C.	Wall D.		Wall A.	Wall B.	Wall C.	Wall D.
Upper lock, Bohio	1	1	1	1	Nicaragua—Continued.				
Lower lock, Bohio	2	2	2	2	No. 3	9	9	9	9
Upper lock, Pedro Miguel	3	3	3	3	No. 4	6	7	7	7
Lower lock, Pedro Miguel	4	4	4	4	No. 5	10	10	10	10
Miraflores	5	5	5	5	No. 6	11	11	11	12
Nicaragua:					No. 7	12	10	11	10
No. 1	7	6	6	7	No. 8	11	10	11	10
No. 2	8	8	9	9					

TABLE II.

Figure.	K	Forces assumed.	$x - \frac{b}{3}$	f_1	f_2	f_3
1	31.1	P_1	+ 0.5	+21,200	+ 760	
		P_1+P_2	+ .4	+22,600	+ 600	+22,700
		$P_1+.5 P_3$	- 1.1	+23,600	- 1,600	+23,800
		P_1+P_3	- 1.6	+25,900	- 3,900	+32,400
2	63.6	P_1	+ 3.1	+15,300	+ 7,000	
		P_1+P_2	+ 1.8	+18,200	+ 4,100	
		$P_1+.5 P_3$	0	+22,300	0	
		P_1+P_3	-10.0	+38,800	-16,500	infinite.
3	15.8	P_1	+ .1	+18,300	+ 200	
		P_1+P_2	+ .5	+19,100	+ 600	+19,300
		$P_1+.5 P_3$	- 1.3	+20,200	- 1,700	+20,500
		P_1+P_3	+ .6	+16,800	+ 1,000	
4	35.8	P_1	- .6	+18,900	- 1,100	+19,000
		P_1+P_2	- 3.0	+24,000	- 5,200	+25,000
		$P_1+.5 P_3$	- 6.5	+29,300	-11,500	+50,000
		P_1+P_3	- .3	+20,200	+ 560	+20,400
5	29.8	P_1	- 1.2	+21,800	- 2,160	+22,000
		P_1+P_2	- 3.1	+24,600	- 4,960	+26,200
		$P_1+.5 P_3$	- 5.5	+28,900	- 9,260	+37,500
		P_1+P_3	+ .5	+18,200	+ 800	
6	32.3	P_1	+ .2	+19,800	+ 400	+19,400
		P_1+P_2	- 1.6	+21,400	- 2,400	+21,600
		$P_1+.5 P_3$	- 3.7	+24,700	- 5,700	+26,400
		P_1+P_3	0	+18,700	0	+18,700
7	0	P_1	+ 1.3	+12,200	+ 2,200	
		P_1+P_2	- .5	+15,200	- 800	+15,300
		$P_1+.5 P_3$	- 1.6	+17,200	- 2,800	+17,800
		P_1+P_3	- 4.5	+23,200	- 7,800	+31,000
8	32.3	P_1	+ 1.3	+12,300	+ 2,000	
		P_1+P_2	- .5	+13,600	+ 700	
		$P_1+.5 P_3$	- .2	+14,600	+ 300	+14,600
		P_1+P_3	+ 1.4	+14,500	+ 2,500	
9	15.0	P_1	+ .2	+16,600	+ 400	
		P_1+P_2	- 1.8	+19,600	- 2,600	+20,200
		$P_1+.5 P_3$	- 4.6	+24,700	- 7,700	+31,400
		P_1+P_3	- 2.2	+17,200	- 200	+17,300
10	36.3	P_1	- 1.3	+18,800	- 1,800	+19,200
		P_1+P_2	- 1.8	+19,600	- 2,600	+20,000
		$P_1+.5 P_3$	+ .6	+16,100	+ 1,000	
		P_1+P_3	- .7	+18,100	- 1,000	+18,400
11	15.0	P_1	- 1.0	+18,500	- 1,400	+18,900
		P_1+P_2	- 2.5	+21,000	- 3,900	+22,000
12	25.0	P_1				
		P_1+P_2				
		$P_1+.5 P_3$				
		P_1+P_3				

Tables I and II give most of the results developed in regard to side walls of all the locks.
In Table I—

- A is the side wall on the right-hand side of the lock, looking downstream, between the upper and intermediate gates.
B is the side wall opposite A.
C is the side wall on the right-hand side between the intermediate and lower gates.
D is the side wall opposite C.

In Table II—

- K = the height in feet of rock above springing line of invert which was used in computing the stresses in the wall. The actual height is usually from 1 to 10 feet greater.
 x = the distance in feet from the face of the wall to the point where the resultant cuts the base of the wall.
 b = the breadth of the wall at its base in feet.
 f_1 = the stress per square foot at the toe of the wall if no rupture occurs at the back.
 f_2 = the stress per square foot at the back of the wall.
 f_3 = the stress per square foot at the toe, if the wall should crack at the back along B B.
 P_2 = the total thrust of water pressure acting along A B, if A B is an open joint and connected with the lower pool.
 P_3 = the total thrust of water pressure acting along A B, if A B is an open joint and connected with the upper pool.
B B, fig. 13, is considered to be the base of the wall.

In Table II the position of the resultant and the stresses which would be developed if the force P_1 and the greatest possible water pressure P_3 should act along the back of the wall are given. This has been done, not because it was considered that such a condition is likely to ever exist, but to show that in nearly every case the wall would not fail if it did exist.

A retaining wall must fail by sliding, by overturning, or by crushing at its toe accompanied by overturning.

A trial shows that there is no danger of any of the walls under discussion failing in the first manner.

It might be possible, with a material sufficiently strong, for a wall to fail by simply overturning about its toe. Practically, however, unless the foundation bed fails, it is impossible that this should occur in a large masonry wall without crushing the masonry. The condition of forces that would make such a failure possible requires the resultant to cut the plane of the base outside the wall. This does not occur in any of the walls even if it should be assumed that the full water pressure acts along the back.

As to the danger of failing in the third manner, by crushing at the toe, the maximum pressure is given in the table in the column headed f_3 . With the exception of the walls shown by figs. 2, and possibly 4, the stresses are not so high as to indicate any danger of actual failure, even if it were assumed that the greatest possible water pressure acted between the back of the wall and the natural rock. In the case of the two exceptions the rock back of the walls is very good, and is more than 1,000 feet from the canal above the locks, making it practically impossible that any great amount of water should be confined behind the walls. In none of the walls is it believed that more than a small percentage of the maximum pressure can act behind them.

It may seem that the critical horizontal section might be either above B B at the culverts or below B B at the bottom of the wall. Trial shows that B B is the critical section in all cases. There are two cases, however, where B B passes through the culvert opening, and the stresses for these two walls (see figs. 8 and 9) were computed by the method to be given later under the head of middle walls. The stresses given in Table II are, however, those computed by the same method as used for all the other side walls.

At the gates 5 feet is added to the bottom width of the side walls to make up for the reduction in width caused by the gate recesses. The top width at the gates is made uniformly 30 feet to provide room for the gate anchorage, the valve wells, and the machinery for operating both the valves and gates.

MIDDLE WALLS.

The critical condition for the middle wall occurs when one lock is empty and the other is full.

If it were not for the presence of the culverts in the middle wall it would be a comparatively simple matter to proportion the wall. It would then be a rectangular dam, for which the analysis of stresses is simple.

In the present case the culverts transform the middle wall into a block of masonry resting upon three columns, and acted upon by horizontal forces. There are very few precedents to confirm the results of the analysis of such a case and to guide the judgment. The theory ought to be the more carefully examined for that reason.

There is no short cut in the problem, such as the "middle-third" rule, and there seems to be no way of proceeding, therefore, other than to make as careful an analysis of the stresses as possible, and design the wall accordingly.

Fig. 14 represents a typical cross section of the middle wall.

At first it is not evident whether C D or E F is the critical horizontal section. For all the Panama and Nicaragua locks it is found, by trial, that C D is the critical section.

Let fig. 14 represent one foot length of wall acted upon by the forces from water pressure, P_1 P_2 P_3 .

Let

W = the weight of a prism of masonry of height h , breadth b and thickness of 1 foot,

F = maximum intensity of pressure at A,

f = rate of change of intensity of pressure along A B,

x = any distance from A toward B,

b , d , e , r , k and h be as indicated in fig. 14.

If the masonry above AB were supported by an invariable plane at AB and acted upon by the forces W , P_1 and P_3 there would be a uniformly varying stress along AB as indicated by fig. 15.

Proceeding under this assumption and taking moments about A we have:

$$(1) \quad \frac{Wb - P_1(h-4)}{3} - P_3(e+d+3r) - \int_0^e (F-fx)xdx - \int_{e+2r}^{e+2r+d} (F-fx)xdx - \int_{b-e}^b (F-fx)xdx = 0$$

Again for equilibrium of vertical forces:

$$(2) \quad W - P_3 - \int_0^e (F - fx) \, dx - \int_{e+2r}^{e+2r+d} (F - fx) \, dx - \int_{b-e}^b (F - fx) \, dx = 0$$

By integrating and substituting limits two simultaneous equations are obtained from which F and f may be found and these give the intensity of pressure at any point along AB .

The wall between AB and CD is divided into three parts by the culverts, which must act as columns to sustain the reactions just found and by their action as beams they must sustain the horizontal thrust $P_1 + P_2$.

Let T be the total horizontal thrust acting at the top of these vertical beams. If we consider that half of P_2 is carried to the top of the beam, which is sufficiently accurate for this purpose, we may write:

$$T = P_1 + \frac{1}{2} P_2$$

Let

T_1 = the horizontal thrust sustained by each of the two outside beams,

T_2 = the horizontal thrust sustained by the middle beam,

I_1 = moment of inertia of the horizontal section of each of the two outside beams,

I_2 = moment of inertia of the horizontal section of the middle beam,

M_1 = maximum bending moment in outside beams,

M_2 = maximum bending moment in middle beam,

f_1 = maximum intensity of stress resulting from bending outside beams,

f_2 = maximum intensity of stress resulting from bending of middle beam.

As the deflection of the vertical beams or columns are all equal to each other, the thrust T will divide itself between them in proportion to the moments of inertia of their horizontal sections.

Then

$$\frac{T_1}{T_2} = \frac{I_1}{I_2} = \frac{e^3}{d^3}$$

but

$$T = 2 T_1 + T_2$$

from which we get

$$(3) \quad T_1 = \frac{e^3 T}{2e^3 + d^3}$$

and

$$(4) \quad T_2 = \frac{d^3 T}{2e^3 + d^3}$$

If we consider these beams fixed at both ends with a point of contraflexure midway between, the maximum bending moment is at the end, and

$$M_1 = \frac{1}{2} T_1 k$$

and

$$M_2 = \frac{1}{2} T_2 k$$

Substituting values of T_1 and T_2 from equations 3 and 4,

$$M_1 = \frac{e^3 k T}{4 e^3 + 2 d^3}$$

and

$$M_2 = \frac{d^3 k T}{4 e^3 + 2 d^3}$$

$$f_1 = \pm \frac{M_1 e}{2 I_1}$$

$$f_2 = \pm \frac{M_2 d}{2 I_2}$$

Substituting the values just found for M_1 M_2 I_1 I_2 in the above we get:

$$(5) \quad f_1 = \pm \frac{e^3 k T}{4 e^3 + 2 d^3} \times \frac{12}{e^3} \times \frac{e}{2} = \pm \frac{3 e k T}{2 e^3 + d^3}$$

$$(6) \quad f_2 = \pm \frac{d^3 k T}{4 e^3 + 2 d^3} \times \frac{12}{d^3} \times \frac{d}{2} = \pm \frac{3 d k T}{2 e^3 + d^3} = \frac{d}{e} f_1$$

The distribution of these stresses, due to bending, are indicated graphically in fig. 16.

If there were no bending in the three columns, the uniformly varying intensity of pressure along AB, due to the forces acting above AB, would be transferred direct to the section CD as indicated in fig. 15.

If there were bending only in the three columns, the stress along CD would be as indicated in fig. 16.

If there were no horizontal forces acting above AB and no bending in the columns, there would be a uniform pressure along CD due to the weight of the columns themselves.

As all three of these conditions exist, the stress along CD will be a composite of what it would be if each condition existed alone.

Let f_3 be the stress on CD due to the weight of the columns of masonry between AB and CD. Taking the weight of concrete at 140 pounds per cubic foot:

$$f_3 = 140 k.$$

Fig. 17 indicates the distribution of the stress f_3 .

The shaded portion of fig. 18 indicates the stress along C D due to the three components.

The maximum compression will occur at either C or M (fig. 18) and the maximum tension will occur at either N or D, according to the relative magnitudes of e and d .

- (7) The stress at C = $F + f_1 + f_3$
- (8) The stress at M = $F - f_1(e + 2r) + f_2 + f_3$
- (9) The stress at N = $F - f_1(e + 2r + d) - f_2 + f_3$
- (10) The stress at D = $F - f_2b - f_1 + f_3$

No account has been taken of the portholes leading from the culverts to the lock. They pass diagonally through the horizontal section C D, and will no doubt weaken the wall somewhat, but the critical point of the wall is near the gates where there are fewest portholes. They may therefore be neglected.

The application of equations 1 to 10 will be made to a concrete example a little later.

If it is assumed that each linear foot of the wall resists simply the thrust of the water which falls upon it, then P_1 and P_2 (fig. 14 and equations 1 and 2) may be easily found and substituted, and the stresses computed. A complication is, however, introduced by the thinning of the wall at the gate recesses, which results in throwing part of the thrust at the gate recesses into the wall at each side.

Fig. 20 is a plan of the middle wall, say, at the intermediate gates. Fig. 21 is a section of the wall at the gate recesses.

The water pressure, along E H, fig. 20, is uniform, but the resistance is not, and instead of deflecting the same amount at all points under the uniform thrust, the deflected position of the center line will be some curve, as $c_1 c_1$. If the portion of the wall between G and K, fig. 20, has any strength as a horizontal beam, it must have reactions at its ends, which the rest of the wall must sustain.

It is impracticable to attempt to determine exactly the amount that $c_1 c_1$ will vary from a straight line.

It will be assumed for the present that the deflection of the wall at the gate recesses is twice that of the wide portion of the wall or, in fig. 20, $2 D_1 = D_2$.

The reason for making this assumption will appear later.

It is evident that there must be a certain length of wall which will resist all the thrust which falls directly upon it, without assistance from the rest of the wall. Let A C E H, fig. 20, be that length of wall, which is assumed to be 144 feet long, or three times as long as the gate recesses. This assumption is purely arbitrary, but is, no doubt, safe, for the wall considered as a horizontal beam is strong enough to distribute any inequalities of loading over such a short distance. On the other hand, the thrust at the gate recesses could not be considered to be distributed over a length much greater than this, certainly not at the lower gates, for the wall of full section extends only 40 feet below the hollow quoin of the gate. Also, it would hardly be wise to design a length of more than 144 feet of wall which should require support from shear at its ends.

Fig. 19 is a section of the wall of full width, of Lock No. 8, Nicaragua route, and fig. 21 is a section at BF.

The problem is to find what will be the thrust on the narrow wall and on the wide.
Let

S_1 = the horizontal thrust resisted by 1 foot length of wall of full width,
 S_2 = the horizontal thrust resisted by 1 foot length of wall at the gate recesses,
 D_1 = deflection of B, fig. 19, from a vertical line through D,
 D_2 = deflection of B, fig. 21, from a vertical line through D,
 T_1 = thrust at B, fig. 19, resisted by outside column,
 T_2 = thrust at B, fig. 21, resisted by outside column.

Then from Equation 3,

$$T_1 = \frac{e^3 S_1}{2e^3 + d^3}$$

And from the general formula for deflection of a beam,

$$D_1 = \frac{T_1 k^3}{24 E I}$$

in which E is the modulus of elasticity and I is the moment of inertia of the beam's section.

Applying this to the case of the middle wall of Lock No. 8, Nicaragua route,

$$T_1 = \frac{1000 S_1}{2000 + 3375} = .186 S_1$$

and substituting values of k and I ,

$$D_1 = \frac{1331 T_1}{24 \times 83.33 E} = .383 \frac{T_1}{E} = .0714 \frac{S_1}{E}$$

In the same way,

$$T_2 = \frac{512 S_2}{1024 + 343} = .375 S_2$$

and,

$$D_2 = \frac{1331 T_2}{24 \times 42.66 E} = 1.299 \frac{T_2}{E} = .487 \frac{S_2}{E}$$

Now, by assumption, $2 D_1 = D_2$,
then,

$$.1428 S_1 = .487 S_2$$

or,

$$S_2 = .293 S_1.$$

If S is the horizontal thrust of water pressure per lineal foot of wall, then

$$96 S_1 + 48 S_2 = 144 S$$

or

$$2 S_1 + S_2 = 3 S$$

Substituting the value of S_2 obtained above,

$$2.293 S_1 = 3 S$$

and

$$S_1 = 1.307 S.$$

This shows that about 60 per cent of the thrust upon the thinner wall would be transmitted to the thicker wall.

By computing the deflection of the wall as a horizontal beam acted upon by a load of 60 per cent of the thrust upon the thinner wall, it is found that the deflection D_2 is greater than $2 D_1$; therefore the error in assuming $D_2 = 2 D_1$ was on the safe side.

From this it would follow that the main or full-width wall should be proportioned to resist a horizontal thrust above the middle of DB (fig. 19) greater than the water pressure, and in the case of the wall in hand $P_1 + \frac{1}{2} P_2$ (fig. 19) should be made 1.3 times the water pressure. It is probable that the thrust against the narrow wall will be distributed over a greater length than 144 feet, which would make the average thrust in the wide wall less than 1.3 times the water pressure. The wall adjacent to the gate recess will, however, receive more than the average thrust, and it seems reasonable to proportion it for a load of 1.3 times the water pressure. This was the basis on which the wall was designed.

It is not maintained that the above analysis gives exact values. Masonry is not perfectly homogeneous nor perfectly elastic, and it is doubtful if the internal stress can be so closely predicted for any large body of masonry. It is believed, however, that the analysis gives a fair and safe indication of what should be provided for, and in the absence of any precedent must be depended upon to guide the judgment in designing the walls.

The results seem reasonable, as the unaided judgment would not lead to the reduction of the dimensions of the wall between AB and CD (fig. 21), and the walls are not much greater than would be required if there were no culverts.

To show the application of formulas 1 to 10, take Lock No. 8 of the Nicaragua Canal, the case shown by figs. 19, 20, and 21:

$$\begin{aligned} P_1 &= \frac{1.3 \times 45 \times 45 \times 62.5}{2} = 82,400 \text{ pounds.} \\ P_2 &= 1.3 \times 50.5 \times 11 \times 62.5 = 45,200 \text{ pounds.} \\ P_3 &= 45 \times 6.5 \times 62.5 = 18,200 \text{ pounds.} \\ W_1 &= 49 \times 48 \times 140 = 336,000 \text{ pounds.} \\ T &= P_1 + \frac{1}{2} P_2 = 105,000 \text{ pounds.} \\ h &= 49 \text{ feet.} \\ b &= 48 \text{ feet.} \\ c &= 10 \text{ feet.} \\ d &= 15 \text{ feet.} \\ r &= 3.25 \text{ feet.} \\ k &= 11 \text{ feet.} \end{aligned}$$

Substituting the above quantities in equations (1) and (2), we have—

$$\begin{aligned} 6,195,000 - 840 F + 27,828 f &= 0 \\ 317,800 - 35 F + 840 f &= 0. \end{aligned}$$

Solving these equations we get—

$$\begin{aligned} f &= 187 \\ F &= 13,600 \text{ pounds per square foot.} \end{aligned}$$

Substituting in equations (5) and (6) we get—

$$\begin{aligned} f_1 &= \pm \frac{3 \times 10 \times 11 \times 105,000}{2 \times 1,000 + 3,375} = \pm 6,450 \text{ pounds per square foot} \\ f_2 &= \frac{15 f_1}{10} = \pm 9,675 \text{ pounds per square foot} \\ f_3 &= 11 \times 140 = 1,540 \text{ pounds per square foot} \end{aligned}$$

Substituting in equations (7), (8), (9), and (10), we have—

The stress at C = $13,600 + 6,450 + 1,540 = 21,590$ lbs. per square foot.

The stress at M = $13,600 - 16.5 \times 187 + 9,675 + 1,540 = 21,735$ lbs. per square foot.

The stress at N = $13,600 - 31.5 \times 187 - 9,675 + 1,540 = -430$ lbs. per square foot.

The stress at D = $13,600 - 48 \times 187 - 6,450 + 1,540 = -280$ lbs. per square foot.

Trial should also be made of the shear along CD (fig. 14). The coefficient of friction required to resist sliding is $\frac{P_1 + P_2}{W_1 + 140 k (d + 2e)}$

For Lock No. 8, Nicaragua route, this is $\frac{127,600}{389,900} = .33$. As the coefficient of friction between surfaces of concrete, probably, is not less than 0.6, it is evident that the above wall is safe.

In order to be sure that no mistake has been made in taking CD (fig. 14) as the critical horizontal section, instead of EF, trial was made of EF.

It was found that the resultant of all forces above EF falls 4.2 feet inside its middle third, giving a maximum pressure at E of 13,000 pounds per square foot and a minimum at F of 4,700 pounds per square foot.

Whether CD or EF will be the critical section depends upon the width of the wall and the height DF, the size of the culverts being the same.

If the distance between CD and EF remains the same, then the wider the wall the stronger section CD becomes in proportion to section EF. The width that would make EF the critical section is very nearly reached in the Bohio Locks, in which case the middle wall is only 2 feet wider than would be required if there were no culverts.

LOCK FLOORS.

The lock floors are inverted arches, the upper surfaces of which have a radius of 112 feet, and are 7 feet below grade at the center of the lock. This is true of all the locks of both routes.

All floors have a rock foundation, but in some cases the rock is very seamy and porous, for which reason provision has been made for a certain amount of water pressure acting underneath the floor.

As will be seen from the general drawings, the floors have been made thicker near the gates than in the rest of the lock. The floor at the gates was made thick enough to resist, with a moderate internal stress, the uplift of the water, under the supposition that the lock is empty and the water under the floor has free communication with the upper pool. The thickness of the floor not in the neighborhood of the gates was then arbitrarily made somewhat less than at the gates. The amount was always determined by judgment, depending upon the nature of the rock.

It is confidently believed that the rock is impervious enough to permit very little water pressure below the floors, and that to provide for 50 per cent of the total is very conservative.

In no case, however, have the floors been made thin enough to allow failure even should the maximum possible water pressure act under them. The only effect would be that higher stresses would occur in the concrete than would ordinarily be allowed.

As a case typical of all the locks, take the floor of Lock No. 1, Nicaragua route. At the gates the floor is 14 feet thick. Its bottom is at an average of 91 feet below the upper pool. The water pressure under the floor is then $62.5 \times 91 = 5,690$ pounds per square foot. The weight of the floor is $14 \times 140 = 1,960$ pounds per square foot. The resultant lift under the floor when the lock is empty is then $5,690 - 1,960 = 3,730$ pounds per square foot. The radius of the center line of the arch is 119 feet.

Considering one linear foot of the floor, and assuming that the center of pressure lies at the middle of the arch ring, the total thrust in the arch is $T = 3,730 \times 119 = 444,000$ pounds. The pressure per square foot will be $\frac{444,000}{14} = 31,700$ pounds.

The line of thrust may be assumed to coincide with the center line of a circular arch if the load is fluid pressure and the skew-backs are invariable. The first condition may be assumed to be satisfied in the present case, although when the lift of the water is modified by the weight of the floor the resultant is not distributed exactly as fluid pressure, and it is possible that the rock may not be equally porous at all points, resulting in unequal loading.

The walls of the lock form the skew-backs of the floor arch, and they are not invariable. Every time the lock is emptied the thrust on the back of the walls deflects them forward, which deflection is transmitted to the invert at its springing line, changing the position of the center of pressure in the arch.

Assuming that the angular displacement along BC, fig. 13, is the same as that along BB, then the rate of change of stress along BC will be the same as that along BB. From Table II it is seen that the maximum difference of stress on the two sides of the side wall B of Lock No. 1, Nicaragua route, is $24,700 + 5,700 = 30,400$ pounds per square foot. The rate of change of stress between the two sides of the wall is $\frac{30,400}{40} = 760$.

It is assumed that the line of thrust would coincide with the center line of the arch ring if there were no deflection of the side wall.

Let

$$\begin{aligned} f_1 &= \text{stress at upper side of floor} \\ f_2 &= \text{stress at lower side of floor} \end{aligned}$$

then

$$f_1 - f_2 = 14 \times 760 = 10,640$$

The total thrust in the arch equals

$$7 (f_1 + f_2) = 444,000$$

or

$$f_1 + f_2 = 63,300$$

We have, then, the two simultaneous equations,

$$\begin{aligned} f_1 - f_2 &= 10,640 \\ f_1 + f_2 &= 63,300 \end{aligned}$$

From which

$$\begin{aligned} f_1 &= +36,970 \text{ pounds per square foot.} \\ f_2 &= +26,330 \text{ pounds per square foot.} \end{aligned}$$

This case is typical of all the locks. The maximum compression in the floors at the gates is always less than 40,000 pounds per square foot.

MITER SILLS.

The miter sills, as designed, consist of stone arches 3 feet thick and of varying depth. The rise is in all cases one-fifth the span of the arch, or 16.8 feet in 84 feet.

Each arch is faced with a timber-bearing piece.

It will be noticed from the drawing that the sills of the lower and intermediate lock gates are straight and that all others are curved. Where the sills are straight the bearing piece on the gate is placed upon the straight downstream face of the gates, but where the sills are curved the bearing pieces are placed beneath the gate, and follow a curve approximately parallel to the upstream side of the gate.

This construction is made necessary by the fact that if the bearing were upon the downstream face of the gate and the full area of the bottom of the gate exposed to the head of water which acts on the upper side, the upward lift would be so very much greater than the weight of the gate that it would be in danger of being lifted from its seat. The weight of the lower lock gates is great enough to allow of the placing of the bearing piece on the downstream face and thus allowing the sills to be straight.

APPROACH WALLS.

On the right-hand side of the canal as the dock is approached is placed a quay wall 1,200 feet long. Vessels may tie up to this wall and wait for the opportunity to use the lock, if waiting should be necessary, and in position to pass directly into the lock.

CULVERTS AND VALVES.

The culverts for filling and emptying the lock chambers are contained in the side walls. There are two for each lock, having a combined sectional area of 150 square feet. They are 12 feet 7 inches high and 6 feet 6 inches wide. The roof is a semicircular arch, and the floor is an inverted arch with a radius of 9 feet 4 inches.

Each culvert is connected with the lock by 18 ports 2 feet high and 4.5 feet broad. These ports are to be lined with cast iron.

In the flights and in single locks of a greater lift than 30 feet the culverts are also to have cast-iron linings. The culverts of single locks of less than 30 feet lift are to be lined with hard-burned brick.

A second set of culverts is provided at the lower end of the Miraflores lock and Lock No. 8 of the Nicaragua route. These extra culverts are made necessary by the difference in density between fresh and salt water.

When the lock is being emptied a point will be reached when the pressure from within at the mouth of the culvert openings will be balanced by an equal pressure from without.

If the water in the lock is fresh and there is salt water in the pool below the lock, then a balance will be reached with the water surface higher within the lock than outside.

Let

h_1 = depth of sill below water surface inside the lock,

h_2 = depth of sill below water surface outside the lock,

d = difference in elevation of sill and center of culvert opening.

If 62.5 pounds per cubic foot be taken as the weight of fresh water and 64.2 pounds per cubic foot be taken as the weight of salt water, then for a balance of pressure at the culvert openings

$$62.5 (h_1 - d) = 64.2 (h_2 - d)$$

or

$$(11) \quad h_1 = \frac{64.2 h_2}{62.5} - \frac{1.7 d}{62.5} = 1.027 h_2 - .027 d$$

The water pressure on the upstream side of one leaf of a gate is:

$$(12) \quad P_1 = \frac{46 \times 62.5 h_1^2}{2} = 1,440 (1.027 h_2 - .027 d)^2$$

The water pressure on the downstream side of one leaf of a gate is:

$$(13) \quad P_2 = \frac{46 \times 64.2 h_2^2}{2}$$

The centers of the openings into the lock of the main culverts are 2 feet above the sill. The maximum and minimum depths of the sill below the water in the lower pool at the Miraflores lock are 35 and 55 feet.

Making $d=2$ feet and $h_2=35$ feet, we have:

$$h_1=35.89 \text{ feet, and } P_1-P_2=48,800 \text{ pounds.}$$

Making $d=2$ and $h_2=55$, we get:

$$h_1=56.43 \text{ feet, and } P_1-P_2=148,700 \text{ pounds.}$$

These two values of (P_1-P_2) are the maximum and minimum additions to the normal resistance that the gate engines would be called upon to overcome to open the gate.

No doubt the above will be considerably modified by the dilution of the water in the lower pool by the fresh canal water, but this dilution can not be relied upon to entirely overcome the difference in pressure on the two sides of the gate; therefore the auxiliary culverts are introduced.

For these $d=30$ feet.

Substituting in equations (14) and (15), $d=30$ and $h_2=35$, we get:

$$\begin{aligned} P_1 &= 1,777,600 \text{ pounds.} \\ P_2 &= 1,806,000 \text{ pounds.} \end{aligned}$$

P_2 is greater than P_1 . There will therefore be an unbalanced force of $P_2-P_1=28,400$ pounds tending to open the gates. In other words, the gates may be opened before the water in the lock has reached the level of the lower pool.

Substituting in equations (14) and (15), $d=30$ and $h_2=55$, we get:

$$\begin{aligned} P_1 &= 4,454,700 \text{ pounds.} \\ P_2 &= 4,466,700 \text{ pounds.} \\ P_2-P_1 &= 12,000 \text{ pounds, unbalanced thrust on the down-} \end{aligned}$$

stream side of one leaf of lower gate.

The Bohio locks and Lock No. 1 of the Nicaragua route are so far away from the ocean that it is believed it will not be necessary to introduce the second culvert into them.

The culvert valves are to be of the Stoney sluice type, consisting of a simple framed door or shutter, suspended from cables which pass over sheaves above to a counterweight. The valve moves vertically against a train of free rollers at each end. This type of valve has been used very successfully on the Manchester and other canals.

TIME OF FILLING AND EMPTYING LOCKS.

Let

A = the horizontal area of lock chamber in square feet.

a = the sectional area of culverts in square feet.

h = lift of lock in feet.

g = acceleration of gravity in feet per second.

c = a coefficient depending upon the construction of the culverts.

The time in seconds for filling or emptying a single lock or for filling the upper or emptying the lower lock of a flight is:

$$(14) \quad t_1 = \frac{2A\sqrt{h}}{ca\sqrt{2g}}$$

For filling one lock of a flight from another of the same size, the time in seconds is:

$$(15) \quad t_2 = \frac{A\sqrt{h}}{ca\sqrt{g}}$$

The United States Board of Engineers on Deep Waterways made a study of the value of c for culverts like those designed for the Isthmian Canal locks, and they decided upon a coefficient of 0.75 to be used in formula (14) and 0.66 to be used in formula (15).

The following table gives the time computed on this basis for filling and emptying all the locks of both the Panama and Nicaragua routes:

TABLE III.—*Giving time for filling and emptying locks.*

	Average lift of lock.	Area of lock chamber.	Sectional area of culverts.	Formula used.	Coeffi- cient used.	Time.
PANAMA ROUTE.						
	<i>Feet.</i>	<i>Sq. ft.</i>	<i>Sq. ft.</i>			<i>m. sec.</i>
Filling upper or emptying lower lock at Bohio	42.5	67,812	150	14	0.75	16 19
Filling lower or emptying upper lock at Bohio	42.5	67,812	150	15	.66	13 07
Filling upper or emptying lower lock at Pedro Miguel	28.0	67,812	150	14	.75	13 14
Filling lower or emptying upper lock at Pedro Miguel	28.0	67,812	150	15	.66	10 39
Filling or emptying lock at Miraflores	29.0	67,892	150	14	.75	13 24
NICARAGUA ROUTE.						
Filling or emptying Lock No. 1	36.0	67,892	150	14	.75	14 56
No. 2 or 3	18.5	67,892	150	14	.75	10 42
No. 4	35.0	67,892	150	14	.75	14 48
No. 5	26.5	67,892	150	14	.75	12 49
No. 6, 7, or 8	28.5	67,892	150	14	.75	13 17

USE OF WATER FOR LOCKAGE OF VESSELS.

Let

Q=a lock full of water=horizontal area of lock chamber times lift.

D=displacement of vessel.

N=number of locks in a flight.

The amount of water required to lock a vessel depends upon the condition—whether full or empty—that the vessel finds the lock, and also upon the direction—whether up or down—in which the vessel is passing.

There are four combinations of condition of lockage. Following is a statement of each of the four combinations of condition, with a formula giving the amount of water which must be used for each condition. The formulas are general and are for flights of locks. For a single lock make $n=1$.

First. The ascent of a vessel next after one has descended, for which case the volume of water passing the upper gate is: $V=NQ+D$.

Second. The descent of a vessel next after one has ascended, for which case the volume of water passing the upper gate is: $V=-D$.

Third. The ascent of a vessel following one which has ascended, for which the volume of water passing the upper gate is

$$V=Q+D.$$

Fourth. The descent of a vessel following one which has descended, for which case the volume of water passing the upper gate is

$$V=Q-D.$$

All of these formulas contain the uncertain term D. There are, however, two considerations, either one of which will cancel D.

In both formulas for ascending vessels D has the plus sign, and in both formulas for descending vessels D has the minus sign. The principal use of the formulas is to obtain the amount of water drawn from the summit level, and as all vessels which ascend to the summit must again descend, it is evident that in getting the total amount of water used the displacement of the vessel will be canceled.

The second consideration is that the first two conditions are equally likely to occur. Therefore for vessels passing in alternate directions through a lock the volume of water used per vessel is

$$V=\frac{NQ+D-D}{2}=\frac{NQ}{2}.$$

The chances are also equal that the third and fourth conditions will occur. Therefore for vessels passing locks in succession the volume of water used per vessel is:

$$(16) \quad V=\frac{Q+D+Q-D}{2}=Q.$$

It is not so clear that there will be equal chances that vessels will pass the locks alternately and in succession in a canal with a system of double locks with quay or tie-up walls at the ends arranged as they are in the designs for the Isthmian Canal locks.

The quay walls are arranged for the greatest conveniences of vessels passing in succession, all upbound vessels using one lock of a pair and all down-bound vessels using the other, and if the number of vessels should approach at all closely the capacity of the locks no doubt the lockage would be arranged in that manner. Even if the number of vessels should be small it is probable that for the sake of convenience all vessels will be made to pass in succession. It is assumed, therefore, that such will be the case. The assumption is on the side of safety. However, let us assume for the moment that the chances that the vessels will pass the locks in succession and alternation are equal. In that case the volume of water used per vessel is:

$$(17) \quad V = \frac{1}{2} \left(\frac{NQ}{2} + Q \right) = \frac{N+2}{4} Q.$$

On the Panama route there is a flight of two locks on each side of the summit level. Substituting 2 for N in equation (17) we get $V=Q$, or the same as by equation (16). In other words, on the Panama route the order of the lockages makes no difference in the amount of water required.

It is assumed that there will be 10 lockages per day, which, with an average vessel of 3,000 tons, would be a traffic of approximately 10,000,000 tons per year. It is also assumed that 4 vessels per day use the long lock and 6 use the short lock.

The horizontal area of the full-length lock chamber at Bohio and Pedro Miguel is 67,812 square feet, and of the short lock chamber is 38,652 square feet.

The average lift at Bohio is 42.5 feet, and at Pedro Miguel 28 feet.

The average flow from the summit level of the Panama route to supply water for lockage will be:

$$\frac{(4 \times 67,812 + 6 \times 38,652)(42.5 + 28)}{86,400} = 411 \text{ cubic feet per second.}$$

The horizontal areas of the locks of the Nicaragua route are 67,392 square feet for the full-length chamber and 38,232 square feet for the short chamber. The average lift of Lock No. 4 will be 35 feet and of Lock No. 6, 28.5 feet.

It should be noted at this point that the average lift of Lock No. 5 of the Nicaragua route is not so great as that of Lock No. 6, and that Lock No. 6 will, therefore, require more water than Lock No. 5 would ordinarily pass down to it. To provide for this a by-pass is placed in the middle wall of Lock No. 5, as shown in the drawing. In the computation to determine amount of water required from the summit level Lock No. 6 is used in the place of No. 5. Assuming that 4 vessels per day use the long lock chamber and 6 use the short chamber, and that vessels pass in succession, then the average flow from the summit level of the Nicaragua route required to supply water for lockage will be:

$$\frac{(4 \times 67,392 + 6 \times 38,232)(35 + 28.5)}{86,400} = 367 \text{ cubic feet per second.}$$

LEAKAGE AT LOCKS.

The leakage at a lock is, of course, very uncertain in amount, varying as it does with the inaccuracy of fitting. The probable maximum can, however, be obtained. There are only two points at which serious leakage will occur, the culvert and the gates.

From observations on the Weitzel lock, at the St. Mary's Falls Canal, in June, 1898, the leakage of culvert valves was determined for heads of 1.75 to 16.75 feet. The valves were of iron, turning on horizontal axes in iron frames not closing against seats.

The coefficients of discharge, c , in the formula

$$Q = cA\sqrt{2gh},$$

were nearly constant at 0.50 for all heads up to 16.5 feet, inclusive. The coefficient for 16.75 feet head was 0.68. There was, however, probably an error of observation.

For the Isthmian Canal we have Stoney valves with seats 13 by 7 feet. If it be assumed that the valves run one-sixteenth inch clear of the seats, then the area of opening for four valves will be:

$$A = \frac{40 \times 12}{144 \times 16} \times 4 = .85 \text{ square foot.}$$

By careful fitting the leakage around the quoin posts can be made very small. There should be no leakage through the gates. If the gates are too short or much worn there may be

an opening between the miter posts at the bottom. This would hardly be allowed to become as much as 1 inch before refitting, and would not extend to the full height of the gates.

If the gates were too long one would usually close against the sill and the other would be free from it. If the gates, taken together, were seven-sixteenths of an inch too long there would be an opening of about 1 inch between one of the gates and the sill at the miter, decreasing to zero at some point between the miter and the quoin. Such an excess length would be produced by a rise of temperature of 52° F., or might be caused by a piece of wood caught between the gates. Such a variation of temperature is impossible at the Isthmus, while the accident of an obstruction between the gates could cover only a small part of the time.

It is believed, therefore, that if it be assumed that there will always be an opening of 1 inch at the miter for one leaf of the gate, decreasing to zero at the quoin, ample provision will thereby be made for both the leakage at the sill and any that may occur at the quoins or miter posts.

The area of opening at the gates of a pair of locks in which each leaf is 47 feet long is:

$$\frac{47 \times 12 \times 2}{144 \times 2} = 3.92 \text{ square feet.}$$

The total area of opening producing leakage is, therefore, $3.92 + .83 = 4.75$ square feet, and the leakage is:

$$Q = .5 \times 4.75 \sqrt{2gh} = 19.06 \sqrt{h}.$$

It will be assumed that there will be a constant head producing leakage equal to the lift of the lock.

This is very nearly true for all single locks. For the flights of locks at Bohio and Pedro Miguel there will be times when the head on the valves will be twice the lift of each lock; but, on the other hand, there will be times when lockages will occur at long intervals and each lock chamber will partially fill by leakage. Then the head on the valves may be only two-thirds the lift of each lock. What the relative frequency and effect of these conditions will be can not be told, and, as stated before, it will be assumed that the head on the valves is constant and equal to the lift of the lock.

From formula 18 we get: At Bohio Locks, for $h=42.5$, $Q=124$ cubic feet per second; at Pedro Miguel, for $h=28$, $Q=101$ cubic feet per second; total supply required for leakage on Panama route, 225 cubic feet per second. At Lock No. 4, Nicaragua, for $h=35$, $Q=113$ cubic feet per second; at Lock No. 6, Nicaragua, for $h=28.5$, $Q=102$ cubic feet per second; total supply required for leakage on Nicaragua route, 215 cubic feet per second.

For operating machinery and lighting it is assumed that 300 horsepower will be required at each lock, or 600 horsepower for a flight of two locks. Assuming an efficiency coefficient of

75 per cent for water wheels, there will be required at Bohio $\frac{600 \times 550}{85 \times 62.5 \times .75} = 83$ cubic feet per

second; at Pedro Miguel $\frac{600 \times 550}{56 \times 62.5 \times .75} = 117$ cubic feet per second; total for Panama route, 200

cubic feet per second. At Lock No. 4, Nicaragua, $\frac{300 \times 550}{35 \times 62.5 \times .75} = 100$ cubic feet per second; at

Lock No. 5, Nicaragua, $\frac{300 \times 550}{26.5 \times 62.5 \times .75} = 133$ cubic feet per second; total for Nicaragua route, 233 cubic feet per second.

Summing up the total average flow of water from the summit level to supply water used at locks, we have:

	Cu. ft. per sec.
Panama route:	
Water for lockage	411
Water to supply leakage	225
Water for power	200
Total	836
Nicaragua route:	
Water for lockage	367
Water to supply leakage	215
Water for power	233
Total	815

PANAMA ROUTE.

Estimate of cost of double locks, exclusive of excavation.

	Unit price.	Bohio Locks (Commission's plan, lift from elevation - 0.5' to elevation + 90').		Pedro Miguel Locks (Commission's plan) and Tiger Hill route (lift from elevation + 28' to elevation + 90').		Miraflores Locks (Commission's plan) and Tiger Hill route (lift from elevation - 10' to elevation + 30').		Obispo Guard Gates (Commission's plan), Tiger Hill route, and French plan.	
		Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.
Concrete.....cu. yds..	\$8.00	956,100	\$7,648,800	710,000	\$5,680,000	408,100	\$3,264,800	9,020	\$72,160
Cut stone.....do....	60.00	5,300	318,000	4,900	294,000	3,400	204,000	220	13,200
Structural steel.....lbs..		12,727,000		9,912,000		7,024,000		707,700	
Steel castings.....do....		2,060,000		1,660,000		1,180,000		86,000	
Forgings.....do....		36,400		36,000		26,000		2,600	
Bronze.....do....		12,600		12,600		9,000		1,000	
Metal in gates.....do....	.075	14,836,000	1,112,700	11,620,600	871,550	8,239,000	617,920	797,300	59,800
Timber.....M ft. B. M..	100.00	98	9,800	98	9,800	70	7,000	7	700
Brickwork.....cu. yds..	15.00					5,700	85,500		
Cast iron.....lbs..	.04	14,625,000	585,000	14,625,000	585,000	506,000	20,240		
Machinery and plant.....			300,000		300,000		200,000		
Total.....			9,974,300		7,740,350		4,399,460		145,860
<i>Approach wall, above lock.</i>									
Concrete.....cu. yds..	8.00	39,000	314,400	38,397	307,176	28,440	227,520		
Piles.....lin. ft..	.50	240,000	120,000						
Total cost upper approach wall.....			434,400		307,176		227,520		
<i>Approach wall, below lock.</i>									
Concrete.....cu. yds..	8.00	34,000	272,000	20,170	161,360	26,660	213,280		
Piles.....lin. ft..	.50	54,000	27,000						
Total cost lower approach wall.....			299,000		161,360		213,280		

	Unit price.	Tiger Hill Locks, Tiger Hill route (lift from elevation - 0.5' to elevation + 26.25').		Bohio Locks, Tiger Hill route (lift from elevation + 26.25' to elevation + 90').		Bohio Locks (French plan, lift from elevation - 0.5' to elevation + 67').		Pedro Miguel Locks (French plan, lift from elevation + 15.66' to elevation + 67').	
		Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.
Concrete.....cu. yds..	\$8.00	320,850	\$2,566,800	725,000	\$5,800,000	734,400	\$5,875,200	612,430	\$4,899,440
Cut stone.....do....	60.00	3,000	180,000	5,000	300,000	4,952	279,120	4,848	290,880
Structural steel.....lbs..		4,982,000		10,181,000		10,044,000		9,121,000	
Steel castings.....do....		1,020,000		1,690,000		1,724,000		1,648,000	
Forgings.....do....		26,000		36,000		36,400		36,400	
Bronze.....do....		9,000		12,600		12,600		12,600	
Metal in gates.....do....	.075	6,037,000	452,775	11,919,600	893,970	11,817,000	886,270	10,818,000	811,350
Timber.....M ft. B. M..	100.00	60	6,000	84	8,400	98	9,800	98	9,800
Brickwork.....cu. yds..	15.00	5,700	85,500						
Cast iron.....lbs..	.04	506,000	20,240	14,625,000	585,000	14,625,000	585,000	14,625,000	585,000
Machinery and plant.....			200,000		300,000		300,000		300,000
Total.....			3,511,315		7,887,370		7,953,390		6,896,470
<i>Approach wall, above lock.</i>									
Concrete.....cu. yds..	8.00	28,440	227,520	27,400	219,200	27,400	219,200	23,497	187,976
Piles.....lin. ft..	.50			240,000	120,000	90,000	45,000		
Total cost upper approach wall.....			227,520		339,200		264,200		187,976
<i>Approach wall, below lock.</i>									
Concrete.....cu. yds..	8.00	23,400	187,200	34,000	272,000	34,000	272,000	14,000	112,000
Piles.....lin. ft..	.50	48,000	24,000	54,000	27,000	54,000	27,000		
Total cost lower approach wall.....			211,200		299,000		299,000		112,000

PANAMA ROUTE—Continued.

Estimate of cost of double locks, exclusive of excavation—Continued.

	Unit price.	Miraflores Locks (French plan, lift from elevation -10' to elevation +15.66').		Miraflores Locks, sea-level canal (lift from elevation 0.0' to elevation +10').	
		Quantity.	Cost.	Quantity.	Cost.
Concrete.....cu. yds.	\$8.00	312,800	\$2,502,400	293,372	\$2,346,976
Cut stone.....do.	60.00	3,136	188,160	2,426	145,460
Structural steel.....lbs.		6,012,000		3,940,000	
Steel castings.....do.		1,112,000		768,000	
Forgings.....do.		26,000		20,800	
Bronze.....do.		9,000		7,200	
Metal in gates.....do.	.075	7,159,000	536,925	4,736,000	355,200
Timber.....M ft. B. M.	100.00	70	7,000	56	5,600
Brickwork.....cu. yds.	15.00	5,700	85,500	5,700	85,500
Cast iron.....lbs.	.04	506,000	20,240	506,000	20,240
Machinery and plant.....do.		200,000		200,000	
Total.....			3,540,225		3,159,076
<i>Approach wall, above lock.</i>					
Concrete.....cu. yds.	8.00	24,660	197,280	24,660	197,280
Piles.....lin. ft.	.50				
Total cost upper approach wall.....			197,280		197,280
<i>Approach wall, below lock.</i>					
Concrete.....cu. yds.	8.00	26,660	213,280	26,660	213,280
Piles.....lin. ft.	.50				
Total cost lower approach wall.....			213,280		213,280

Estimate of cost of single locks, exclusive of excavation.

	Unit price.	Bohio Locks (Commission's plan, lift from elevation -0.5' to elevation +90').		Pedro Miguel Locks (Commission's plan) and Tiger Hill route (lift from elevation +28' to elevation +90').		Miraflores Lock (Commission's plan) and Tiger Hill route (lift from elevation -10' to elevation +30').		Obispo Guard Gates (Commission's plan), Tiger Hill route (French plan).	
		Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.
Concrete.....cu. yds.	\$8.00	663,100	\$5,304,800	491,000	\$3,928,000	284,125	\$2,273,000	9,020	\$72,160
Cut stone.....do.	60.00	3,400	204,000	3,200	192,000	2,200	132,000	220	13,200
Structural steel.....lbs.		6,324,000		4,916,000		3,488,000		707,700	
Steel castings.....do.		1,030,000		830,000		590,000		86,000	
Forgings.....do.		18,200		18,200		13,000		2,600	
Bronze.....do.		6,300		6,300		4,500		1,000	
Metal in gates.....do.	.075	7,378,500	553,390	5,770,500	432,790	4,035,500	307,160	797,300	59,800
Timber.....M ft. B. M.	100.00	49	4,900	42	4,200	35	3,500	7	700
Brickwork.....cu. yds.	15.00					4,275	64,125		
Cast iron.....lbs.	.04	10,968,750	438,750	10,968,750	438,750	379,500	15,180		
Machinery and plant.....do.		200,000		200,000		150,000			
Total.....			6,705,840		5,195,740		2,944,965		145,860

Approach walls same as for double locks.

	Unit price.	Tiger Hill Locks, Tiger Hill route (lift from elevation -0.5' to elevation +26.25').		Bohio Locks, Tiger Hill route (lift from elevation +26.25' to elevation +90').		Bohio Locks (French plan, lift from elevation -0.5' to elevation +67').		Pedro Miguel Locks (French plan, lift from elevation +15.66' to elevation +67').		Miraflores Lock (French plan, lift from elevation -10' to elevation +15.66').	
		Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.	Quantity.	Cost.
Concrete.....cu. yds.	\$8.00	220,885	\$1,767,080	505,000	\$4,040,000	508,650	\$4,069,200	421,980	\$3,375,840	215,075	\$1,720,600
Cut stone.....do.	60.00	2,000	120,000	3,000	180,000	3,196	191,760	3,118	187,080	2,035	122,100
Structural steel.....lbs.		2,518,000		5,070,000		5,022,000		4,560,000		3,006,000	
Steel castings.....do.		510,000		851,000		862,000		824,000		556,000	
Forgings.....do.		13,000		18,200		18,200		18,200		13,000	
Bronze.....do.		4,500		6,300		6,300		6,300		4,500	
Metal in gates.....do.	.075	3,045,500	228,410	5,945,500	445,910	5,908,500	443,137	5,408,500	405,637	3,579,500	268,462
Timber.....M ft. B. M.	100.00	30	3,000	42	4,200	49	4,900	49	4,900	35	3,500
Brickwork.....cu. yds.	15.00	4,275	64,125							4,275	64,125
Cast iron.....lbs.	.04	379,500	15,180	10,968,750	438,750	10,968,750	438,750	10,968,750	438,750	379,500	15,180
Machinery and plant.....do.		150,000		200,000		200,000		200,000		150,000	
Total.....			2,347,795		5,308,860		5,347,747		4,612,207		2,348,967

APPENDIX D.

REPORT ON HYDROGRAPHY OF PANAMA CANAL ROUTE, MADE FOR THE ISTHMIAN CANAL COMMISSION BY ARTHUR P. DAVIS, CHIEF HYDROGRAPHER.

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7. Hydrographs of Chagres River at Gamboa.

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9. Rating curves of floods.

WASHINGTON, D. C., *March 15, 1901.*

Rear-Admiral J. G. WALKER,
President Isthmian Canal Commission.

SIR: I have the honor to transmit herewith a report on the hydrography of the Panama Canal route. It includes a full account of the work prosecuted by me under your authority, and also the most important portions of the results obtained by the Panama Canal Company, together with a discussion of the same and the conclusions derivable from all the information at hand.

Yours, with respect,

ARTHUR P. DAVIS, *Chief Hydrographer.*

THE HYDROGRAPHY OF PANAMA.

The hydrographic problem of the Panama Canal project requires a knowledge of the magnitude and habit of the flood discharge of the Chagres River and also in a minor degree of the tributaries of the Chagres and of the Rio Grande on the southern end of the line. It also requires a knowledge of the minimum flow of the Chagres River considered as a feeder to the summit level and the locks of the canal. Incidental to these matters the determination of rainfall is important, considered both as a source of water supply and as a hindrance to construction.

Some observations upon these points were taken by the old Panama Canal Company, and since the organization of the new company the records have been much more thorough and complete of the discharge of the Chagres at Gamboa and Bohio. Since April, 1899, measurements have also been made at Alhajuela, where it is proposed by the canal company to construct a reservoir to serve as a regulator of the floods of the river and to store water for the use of the canal.

The measurements of rainfall taken by the company were made at points where stream measurements were being made, and at La Boca, the southern end of the canal line, this record, however, being very short. These data are supplemented by rainfall records kept at Colon by the Panama Railway Company. This is the oldest record of rainfall in the vicinity of the Panama Canal, and is of great value in the study of hydrographic problems connected therewith.

The work undertaken by this Commission consists mainly in an examination of the observations and results already obtained by the company, of a verification of their methods by actual field observations, and of an extension of the rainfall information, as far as possible, over the basin of the Chagres River.

These operations were undertaken in November, 1899, and placed in charge of Mr. W. W. Schlecht, whose description of methods is appended to this report. In July and August, 1900, the writer made a personal inspection of the records of the old and new companies at the office of the Panama Canal Company in Paris, obtaining many details of value regarding the methods and data employed in arriving at the conclusions published by the company. It was found that the data obtained by the old company prior to the organization of the new was very fragmentary and incomplete, considerable periods being entirely skipped. The most serious lack of information was with respect to the magnitude of the great floods that have been observed upon the Isthmus, especially the maximum flood of 1879. The observations taken by the new company are far more complete and satisfactory, but, unfortunately, they do not include any flood discharge as great as those that occurred under the régime of the old company.

Every facility for the investigation of these records was extended by the Panama Company, and every possible assistance in the investigation was courteously rendered by Gen. Henry L. Abbot, of the comité technique.

THE CHAGRES RIVER.

The main trunk of the Chagres is formed by two principal branches. The Pequeni rises near the Caribbean coast, flows nearly south until it meets the Chagres proper a short distance above Alhajuela. The general course of the Upper Chagres from its source is southwest, and it continues in the same general direction after receiving the waters of the Pequeni to the mouth of the Obispo, where the proposed canal line leaves its valley. Its course is then westward to Tavernilla and from that point to its mouth its course is nearly northwesterly. Its total length is over 120 miles without including minor bends, although the width of the Isthmus at its mouth is less than 50 miles. The various drainage areas are about as follows:

	Sq. miles.
Total area of Chagres drainage basin.....	1,400
Upper Chagres (above Pequeni).....	300
Rio Pequeni.....	175
Above Alhajuela.....	505
Between Alhajuela and Gamboa.....	130
Above Gamboa.....	635
Rio Obispo.....	38
Between Gamboa and Bohio.....	245
Above Bohio.....	880
Between Alhajuela and Bohio.....	375
Below Bohio.....	520
Lower Gatun.....	145

The district below Bohio, drained by the Chagres and its tributaries, consists mainly of low hills and swamps. The river is sluggish, the effect of the tide being plainly perceptible at Bohio at low water, although the tidal fluctuations of the Caribbean are very slight. Above this point there are some modifications, the declivity being greater and the country higher and less swampy. Several rapids occur in this portion of the river, and above Obispo rapids are frequent and the river swift. The Upper Chagres is flanked by steep, rocky hills clothed with luxuriant vegetation, rapids are frequent, the water is everywhere swift and is very clear except in times of freshet. The stream is sinuous in many parts and bordered frequently by rock cliffs, lending variety to the scene, which is everywhere one of surpassing beauty. In some cases the convex curve of the stream with its swift current has undermined the limestone cliffs, leaving overhanging ledges and producing caverns of considerable extent. A photograph of one of these is shown in fig. 3.^a

Measurements of discharge of the Chagres were made during 1900 at all three of the stations maintained by the company, namely, Alhajuela, Gamboa, and Bohio, to check the methods and results for the information of the Isthmian Canal Commission. A comparison of these results with those submitted by the company may be made from the following tables. It will be seen that the company's results are much the smaller. This is partly accounted for by the disagreement in gauging results, as obtained by current meters, and by taking 80 per cent of the velocity of surface floats; but this explanation does not apply to Bohio, where the gauging results are

^aNot here reproduced.

practically in agreement. The principal reason may be formed by an examination of the rating curves as plotted in fig. 6.

It will be seen that the "general" curve, which is the one used for most of the computations made by the company, gives much lower values for all gauge heights than indicated by the measurements of discharge, and, in addition to this, the measurements themselves are too small, at least for Alhajuela and Gamboa. The fact that the results obtained by the float method were too small was pointed out by General Abbot in his note "Sur le régime du Chagres," page 6, where he remarked that—

In these calculations the velocities measured are to be multiplied by 0.8. This is too great a reduction if we take account of the fact that the floats are to be liberated on all the river, and not limited to the center. According to my opinion, a coefficient of 0.9 would be sufficient.

If the factor 0.9 were used in reducing the float measurements, as suggested by General Abbot, they would very nearly agree at Alhajuela and Gamboa with the current-meter measurements, but at Bohio the agreement would not be so close.

It is important to note, however, that the results obtained and put forth by the company are very conservative, the error being on the side of safety when considering the low-water flow, and as a different formula, called the "special" formula, is used for floods at Gamboa and Bohio, this error does not enter into them, and, as will be seen by the diagram, the special formulæ agree closely with the observations.

Detailed descriptions of the stations and a discussion of methods will be found in Mr. Schlecht's report, appended hereto, page 49.

ALHAJUELA STATION ON CHAGRES RIVER.

This is the point on the Chagres, about 11 miles above Gamboa, where the Panama Canal Company proposes to build a high masonry dam to impound water for the use of the canal and to assist in the storage of floods. The dam site is about 7 miles, by the course of the river, below "Dos Bocas," the junction of the two main branches of the Chagres, the gauging station being about half a mile below the dam site.

The company established the station on the 15th day of April, 1899, and still maintains it. The results given by the company are summarized as follows:

Estimated monthly discharge of Chagres River at Alhajuela.

[Drainage area, 505 square miles.]

OBSERVATIONS OF THE PANAMA CANAL COMPANY.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall in basin.
	Maximum.	Minimum.	Mean.		Second- feet per square mile.	Depth in inches.	
1899.							
April 15-30	1,024	600	706	22,400			<i>Inches.</i>
May	28,420	494	1,590	97,760	3.15	3.63	
June	4,200	1,165	2,150	127,930	4.26	4.75	
July	15,640	1,235	2,220	136,500	4.40	5.07	
August	27,430	1,660	3,280	201,680	6.50	7.49	
September	11,160	1,450	2,400	142,800	4.75	5.30	
October	9,990	1,590	2,650	162,940	5.25	6.05	
November	28,660	1,520	2,930	174,350	5.80	6.47	
December	11,160	1,380	2,260	138,960	4.47	5.15	
The year	28,660		2,430	1,205,320		43.91	

OBSERVATIONS OF THE NEW PANAMA CANAL COMPANY.

1900.							
January	11,150	1,020	1,590	97,760	3.15	3.63	<i>Inches.</i>
February	1,130	565	812	45,100	1.60	1.67	2.27
March	777	425	530	32,590	1.05	1.21	.38
April	2,050	318	565	33,620	1.12	1.25	.32
May	13,770	388	1,340	82,400	2.65	3.06	4.06
June	7,730	880	1,550	92,230	3.07	3.42	11.88
July	18,180	1,230	2,190	134,660	4.33	4.99	16.57
August	20,400	1,520	2,820	173,400	5.58	6.43	14.64
September	11,650	1,270	2,150	127,930	4.26	4.75	13.01
October	24,780	1,730	3,210	197,680	6.36	7.33	15.60
November	14,680	1,620	2,965	176,430	5.87	6.55	16.29
December	20,540	1,130	2,290	140,800	4.53	5.22	14.92
The year	24,780	318	1,840	1,334,300	3.64	49.51	3.76

Estimated monthly discharge of Chagres River at Alhajuela—Continued.

FROM OBSERVATIONS OF THE ISTHMIAN CANAL COMMISSION.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall in basin.
	Maximum.	Minimum.	Mean.		Second- feet per square mile.	Depth in inches.	
1900.							<i>Inches.</i>
January.....	14,535	1,230	1,941	119,350	3.84	4.43	2.27
February.....	1,365	652	916	50,870	1.81	1.88	.38
March.....	849	497	608	37,380	1.20	1.38	.32
April.....	2,032	395	631	37,550	1.25	1.39	4.06
May.....	17,460	476	1,588	97,640	3.14	3.62	11.88
June.....	10,395	1,025	1,434	115,080	3.83	4.27	16.57
July.....	21,950	1,814	3,148	193,560	6.23	7.18	14.64
August.....	27,400	2,157	4,094	251,730	8.11	9.35	13.01
September.....	17,065	1,720	3,101	184,520	6.14	6.85	15.60
October.....	31,740	2,710	4,889	300,610	9.68	11.16	16.39
November.....	20,670	2,223	4,504	268,010	8.92	9.95	14.92
December.....	28,020	1,396	3,286	202,050	6.50	7.49	3.76
The year.....	31,740	395	2,567	1,858,350		68.95	113.80

Ratio of run-off to rainfall = 60 per cent.

COMPUTED FROM RATING TABLE OF ISTHMIAN CANAL COMMISSION, USING THE GAUGE HEIGHTS OF THE PANAMA CANAL COMPANY AS THE ARGUMENT.

1901.							
January.....	5,070	850	1,165	71,630	2.31	2.66	
February.....	3,530	560	766	42,540	1.52	1.58	
March.....	2,290	415	541	33,260	1.07	1.23	
April.....	3,040	265	509	30,290	1.01	1.13	
May.....	35,460	815	2,050	126,050	4.06	4.68	
June.....	21,280	990	2,360	140,430	4.67	5.21	

GAMBOA STATION ON CHAGRES RIVER.

This is the station of longest record of any on the Isthmus. It is near the point where the canal line leaves the valley of the Chagres; the river valley ascending from this point to the northeast, while the canal line proceeds to the southeast to cut through the continental divide to the Pacific. It was at one time proposed to construct a dam at this point, and the canal company began observations here, but a careful examination of the records in Paris failed to bring to light any continuous record of river stages prior to 1892, but a number of discharge measurements for 1889-1891 were obtained, and are given herewith. A graphic exhibit of the flow from 1893 to 1898 is shown in figure 7, and a comparison with Alhajuela and Bohio for 1899 and 1900 is shown in figure 8.

Annual summary of discharge measurements made by the Panama Canal Company at Gamboa.

Year.	Discharge in second feet.			Total in acre-feet.	Run-off.		Rainfall at Colon.
	Maximum.	Minimum.	Mean.		Second- feet per square mile.	Depth in inches.	
							<i>Inches.</i>
1892	42,000						
1893	51,100	759	3,610	2,612,607	5.69	77.14	131.89
1894	26,190	530	3,770	2,732,284	5.94	80.68	153.69
1895	27,990	565	2,880	2,083,740	4.53	61.55	151.54
1896			2,880	2,093,910	4.53	61.83	131.51
1897	42,190		3,880	2,806,070	6.11	82.84	138.03
1898	28,770	777	3,000	2,169,630	4.72	64.05	115.55
1899	26,190	706	2,580	1,870,500	4.06	55.23	133.02
1900	23,720	388	2,360	1,705,100	3.72	50.35	108.01

a Mean rainfall on basin.

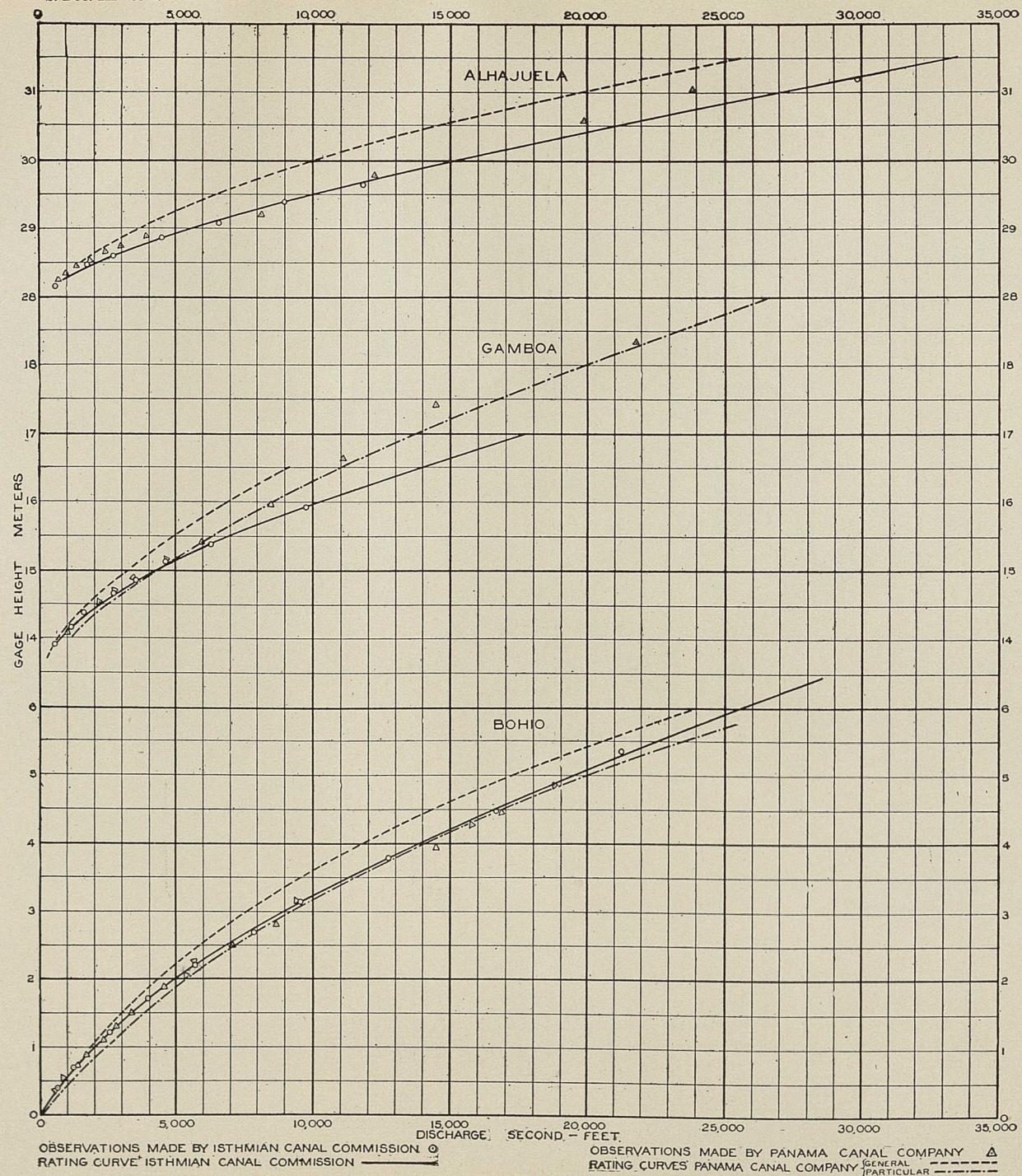


FIG. 6.—COMPARISON OF GAGING CURVES, SHOWING THE REASON FOR THE DISCREPANCY IN THE RESULTS OBTAINED BY THE PANAMA CANAL COMPANY AND THE ISTHMIAN CANAL COMMISSION.

THE PANAMA CANAL.

39

Estimated monthly discharge of Chagres River at Gamboa.

[Drainage area, 635 square miles.]

OBSERVATIONS OF THE NEW PANAMA CANAL COMPANY.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall at Colon.
	Maximum.	Minimum.	Mean.		Second- feet per square mile.	Depth in inches.	
1892.							
May.....	28,770	2,345	5,311	326,561	8.36	9.64	<i>Inches.</i> 18.03
June.....	17,930	2,260	3,590	213,620	5.65	6.30	16.97
July.....	27,320	2,232	7,114	437,423	11.20	12.91	21.77
August.....	25,910	3,531	5,950	363,084	9.37	10.80	15.98
September.....	7,275	3,178	4,476	266,341	7.05	7.87	16.26
October.....	19,840	3,284	5,848	359,580	9.21	10.62	6.6
November.....	42,000	3,990	6,715	399,570	10.57	11.80	26.30
December.....	25,200	2,860	5,968	366,958	9.40	10.84	11.30
The year.....	42,000			2,733,137			
1893.							
January.....	2,892	1,865	2,293	140,991	3.61	4.16	1.73
February.....	3,563	1,423	1,993	110,686	3.13	3.26	3.82
March.....	6,512	759	1,134	69,727	1.79	2.06	1.81
April.....	23,102	1,201	3,190	189,818	5.02	5.60	8.07
May.....	26,620	1,497	3,045	187,229	4.80	5.53	6.56
June.....	6,039	2,147	3,200	190,410	5.04	5.62	12.32
July.....	17,300	2,034	3,472	213,484	5.47	6.31	11.50
August.....	21,140	3,146	4,538	279,031	7.15	8.24	15.12
September.....	10,389	2,430	3,813	226,890	6.00	6.69	9.92
October.....	9,585	2,260	3,733	229,533	5.88	6.78	12.28
November.....	19,840	3,249	4,494	267,412	7.08	7.90	17.80
December.....	51,100	3,178	8,252	507,396	13.00	14.99	30.94
The year.....	51,100	759	3,610	2,612,607	5.69	77.14	131.89
1894.							
January.....	26,189	2,175	3,858	237,219	6.08	7.01	5.35
February.....	2,260	1,201	1,633	90,692	2.57	2.68	1.65
March.....	1,225	742	948	58,290	1.50	1.73	.35
April.....	2,317	572	760	45,223	1.20	1.34	2.16
May.....	13,278	530	1,358	83,500	2.14	2.47	9.85
June.....	7,190	798	2,180	129,719	3.43	3.82	12.24
July.....	18,222	1,349	4,199	258,186	6.61	7.62	19.10
August.....	14,804	2,670	3,864	237,588	6.09	7.02	23.03
September.....	18,462	2,486	4,402	261,937	6.93	7.73	18.78
October.....	6,039	2,860	4,690	288,370	7.38	8.51	12.40
November.....	21,780		4,530	388,560	10.28	11.47	23.66
December.....	32,510		10,620	653,000	16.72	19.28	25.12
The year.....	32,510	530	3,770	2,732,284	5.94	80.68	153.69
1895.							
January.....	14,610	1,910	3,530	217,050	5.56	6.41	3.86
February.....	3,350	1,160	1,480	82,200	2.33	2.43	1.89
March.....	3,210	850	1,200	73,790	1.89	2.18	2.09
April.....	14,120	565	1,130	67,240	1.78	1.99	21.73
May.....	12,430	1,760	3,210	197,380	5.06	5.83	16.77
June.....	25,560	1,690	2,930	174,350	4.61	5.14	9.25
July.....	21,000	1,410	2,860	175,850	4.50	5.19	17.09
August.....	19,340	2,050	3,740	229,960	5.89	6.79	14.13
September.....	12,140	600	3,140	186,840	4.95	5.52	12.09
October.....	27,990	1,690	4,590	282,230	7.23	8.34	16.46
November.....			3,280	195,170	5.17	5.77	20.47
December.....			3,280	201,680	5.17	5.96	15.71
The year.....	27,990	565	2,880	2,083,740	4.53	61.55	151.54
1896.							
January.....			2,220	136,500	3.50	4.04	4.02
February.....			1,130	65,000	1.78	1.92	1.30
March.....			630	38,740	.99	1.14	2.01
April.....			3,070	182,680	4.84	5.40	9.02
May.....			3,280	201,680	5.17	5.96	16.46
June.....			3,810	226,710	6	6.69	8.50
July.....			2,720	167,250	4.28	4.94	13.58
August.....			2,580	158,640	4.06	4.68	15.51
September.....			3,420	203,500	5.39	6.01	12.84
October.....			3,180	195,530	5	5.76	13.98
November.....			4,980	296,330	7.84	8.75	15.63
December.....			3,600	221,350	5.67	6.54	18.66
The year.....			2,880	2,093,910	4.53	61.83	131.51
1897.							
January.....			5,120	314,800	8.06	9.29	3.42
February.....			1,130	62,760	1.78	1.85	.04
March.....			777	47,780	1.22	1.41	.28
April.....			670	39,870	1.06	1.18	3.74
May.....	42,190	2,050	6,710	412,580	10.57	12.19	16.34
June.....	12,110	3,920	6,570	390,940	10.35	11.55	18.82
July.....	13,480	4,270	5,360	329,570	8.44	9.73	14.06
August.....			4,090	251,480	6.44	7.42	17.24
September.....			3,850	229,090	6.06	6.76	17.20
October.....	13,870	2,050	3,560	218,900	5.60	6.46	5.83
November.....	18,250	3,070	4,130	245,750	6.50	7.25	22.16
December.....	20,020	1,450	4,270	262,550	6.72	7.75	18.90
The year.....	42,190		3,880	2,806,070	6.11	82.84	138.03

^aThese means are obtained by averaging the discharge measurements.

THE PANAMA CANAL.

Estimated monthly discharge of Chagres River at Gamboa—Continued.

OBSERVATIONS OF THE NEW PANAMA CANAL COMPANY—Continued.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall at Colon.
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.	
1898.							
January.....	27,710	2,010	4,310	265,010	6.80	7.84	<i>Inches</i> 5.04
February.....	2,930	1,130	1,480	82,200	2.33	2.43	.35
March.....	2,540	777	1,020	62,720	1.60	1.84	1.58
April.....	28,770	777	1,840	109,490	2.90	3.24	4.72
May.....	9,920	1,340	2,470	151,870	3.89	4.48	12.83
June.....	11,650	1,380	2,860	170,180	4.50	5.02	16.38
July.....	18,960	2,330	4,240	260,700	6.68	7.70	21.89
August.....	14,510	2,470	4,130	253,940	6.50	7.49	10.91
September.....	7,270	1,660	2,540	151,140	4	4.46	10.24
October.....	20,470	1,340	3,460	212,750	5.45	6.28	11.38
November.....	27,600	1,020	5,190	308,830	8.17	9.12	12.28
December.....	5,790	1,690	2,290	140,800	3.60	4.15	7.95
The year.....	28,770	777	3,000	2,169,630	4.72	64.05	115.55
1899.							
January.....	22,060	1,730	3,490	214,600	5.50	6.34	6.93
February.....			950	52,760	1.50	1.56	6.49
March.....			950	58,410	1.50	1.73	1.26
April.....	6,740	706	1,060	63,070	1.67	1.86	1.43
May.....	20,470	706	1,910	117,440	3	3.46	13.90
June.....	14,120	1,310	2,470	146,970	3.89	4.34	6.41
July.....	14,120	1,340	2,820	173,400	4.44	5.12	27.68
August.....	26,190	1,870	4,240	260,700	6.68	7.70	14.80
September.....	11,750	1,660	3,180	189,220	5	5.58	16.55
October.....	13,730	1,840	3,390	208,440	5.34	6.16	15.04
November.....	23,890	1,870	3,740	222,550	5.89	6.57	14.49
December.....	13,200	1,620	2,650	162,940	4.17	4.81	9.04
The year.....	26,190	706	2,580	1,870,500	4.06	55.23	133.02
1900.							
January.....	12,920	1,060	1,690	103,910	2.66	3.07	2.09
February.....	1,165	600	810	44,980	1.28	1.33	.35
March.....	670	424	530	32,590	.83	.96	.29
April.....	1,870	388	565	33,620	.89	.99	3.94
May.....	12,320	424	1,340	82,400	2.11	2.43	11.15
June.....	10,020	918	1,940	115,440	3.06	3.41	15.94
July.....	16,910	1,480	3,350	205,980	5.27	6.08	14.47
August.....	20,620	1,620	3,565	219,200	5.61	6.47	12.43
September.....	16,060	1,460	3,070	182,680	4.83	5.39	14.69
October.....	23,720	2,050	4,520	277,920	7.12	8.21	15.78
November.....	17,540	1,910	4,060	241,590	6.40	7.14	13.86
December.....	21,920	1,160	2,680	164,790	4.22	4.87	3.02
The year.....	23,720	388	2,360	1,705,100	3.72	50.35	108.01

FROM OBSERVATIONS OF THE ISTHMIAN CANAL COMMISSION.

1900.							
January.....	14,030	889	2,026	124,570	3.19	3.66	2.09
February.....	1,420	726	970	53,870	1.53	1.59	.35
March.....	805	509	641	39,410	1.01	1.16	.29
April.....	1,745	443	637	37,900	1	1.11	3.94
May.....	15,900	492	1,615	99,300	2.54	2.91	11.15
June.....	12,550	1,205	2,426	144,360	3.82	4.25	15.94
July.....	21,980	2,058	4,104	252,350	6.62	7.62	14.47
August.....	26,300	2,222	4,692	288,500	7.39	8.51	12.43
September.....	20,060	1,899	3,750	223,140	5.91	6.59	14.69
October.....	29,900	2,795	5,846	359,460	9.21	10.61	15.78
November.....	22,760	2,461	5,415	322,220	8.53	9.52	13.86
December.....	27,900	1,509	3,380	207,830	5.82	6.13	3.02
The year.....	29,900	443	2,974	2,152,910		63.66	108.01

Ratio of run off to rainfall = 59 per cent.

COMPUTED FROM RATING TABLE OF ISTHMIAN CANAL COMMISSION, USING THE GAUGE HEIGHTS OF THE PANAMA CANAL COMPANY AS THE ARGUMENT.

1901.							
January.....	4,020	911	1,236	76,000	1.94	2.24	
February.....	2,960	560	817	45,370	1.27	1.32	
March.....	1,480	395	544	33,450	.86	.99	
April.....	2,760	278	498	29,630	.78	.87	
May.....	26,300	955	2,190	134,660	3.45	3.98	
June.....	18,310	1,205	2,500	148,760	3.94	4.40	

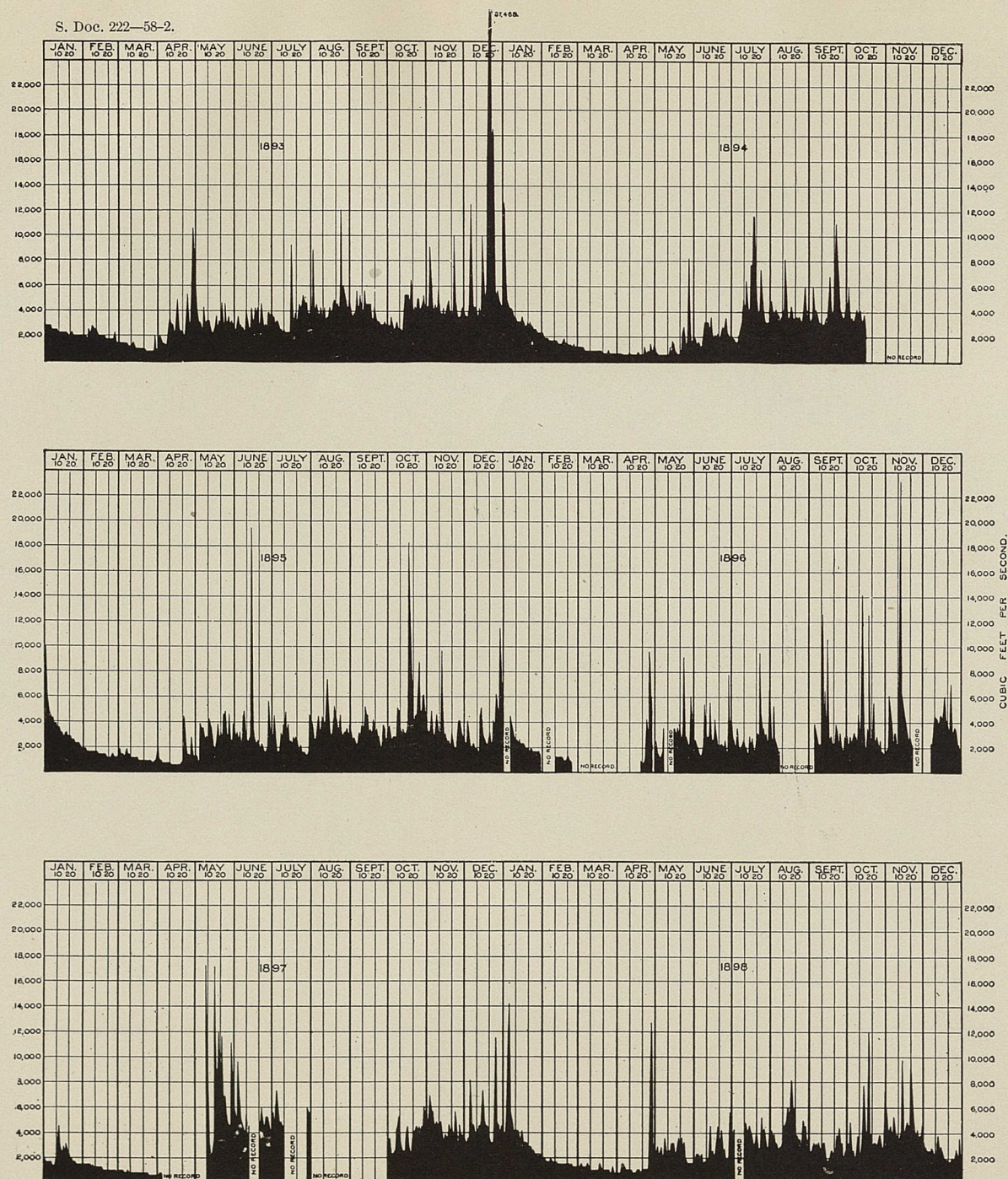


FIG. 7.—HYDROGRAPHS OF THE CHAGRES RIVER AT GAMBOA, PLOTTED FROM OBSERVATIONS OF THE PANAMA CANAL COMPANY.

BOHIO STATION, ON CHAGRES RIVER.

The present plans of both the Isthmian Canal Commission and the Panama Canal Company provide for a dam at Bohio. The company established a fluviograph here in 1895, and a record of discharge has been kept ever since.

Annual summary of discharge measurements made by the Panama Canal Company at Bohio.

Year.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall at Colon.
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.	
1895.....	27,750	710	4,510	3,263,810	5.12	69.52	<i>Inches.</i> 151.54
1896.....	28,950	600	4,220	3,062,960	4.80	65.28	181.51
1897.....	38,790	495	4,850	3,514,390	5.51	74.88	138.03
1898.....	27,890	780	3,960	2,865,730	4.50	61.07	115.55
1899.....	25,870	600	3,400	2,458,420	3.86	52.40	133.02
1900.....	23,860	490	3,520	2,551,810	4	54.38	α 108.67

α Mean rainfall on basin.

Estimated monthly discharge of Chagres River at Bohio.

OBSERVATIONS OF THE PANAMA CANAL COMPANY.

[Drainage area, 880 square miles.]

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall at Colon.
	Maximum.	Minimum.	Mean.		Second-feet per square mile.	Depth in inches.	
1895.							<i>Inches.</i>
January.....	20,721	2,290	α 4,550	279,770	5.17	5.96	
February.....	2,930	1,900	α 1,380	76,640	1.57	1.63	
March.....			α 1,060	65,180	1.20	1.38	
April.....	11,540	710	α 1,060	63,070	1.20	1.34	
May.....	13,830	1,380	α 4,170	256,400	4.74	5.46	
June.....	10,770	2,290	α 4,310	256,460	4.89	5.46	
July.....	19,980	1,910	α 4,590	282,230	5.22	6.02	
August.....	15,500	1,940	α 6,920	425,500	7.86	9.06	
September.....	21,320	2,540	α 5,790	344,530	6.58	7.34	
October.....	27,750	2,610	α 6,920	425,500	7.86	9.06	
November.....	18,640	4,020	α 6,070	361,190	6.90	7.70	
December.....	27,180	2,610	α 6,950	427,340	7.90	9.11	
The year.....	27,750	710	4,510	3,263,810	5.12	69.52	
1896.							
January.....	10,450	1,910	α 4,730	290,840	5.38	6.20	
February.....	2,470	1,020	α 1,550	89,160	1.76	1.90	
March.....	1,940	740	α 880	54,110	1	1.15	
April.....	25,030	600	α 1,690	100,570	1.92	2.14	
May.....	16,270	1,730	α 4,940	303,750	5.61	6.47	
June.....	13,240	2,080	α 4,200	249,920	4.77	5.32	
July.....	22,950	2,430	α 3,140	193,070	3.57	4.12	
August.....	11,930	2,650	α 3,500	215,200	4	4.61	
September.....			α 5,930	352,860	6.74	7.52	
October.....			α 6,350	390,450	7.22	8.32	
November.....	19,100		α 7,590	451,640	8.62	9.62	
December.....	28,950	3,880	α 6,040	371,390	6.86	7.91	
The year.....	28,950	600	4,220	3,062,960	4.80	65.28	
1897.							
January.....	5,370	3,390	α 2,050	126,050	2.33	2.69	
February.....			α 1,590	88,300	1.81	1.88	
March.....	1,380	600	α 710	43,660	1.81	.93	
April.....	14,160	495	α 880	52,360	1	1.12	
May.....	38,790	850	α 7,980	490,670	9.07	10.46	
June.....	12,910	3,460	α 4,200	249,920	4.77	5.32	
July.....	17,400	2,860	α 5,120	314,820	5.82	6.71	
August.....	18,460	2,820	α 7,270	447,010	8.26	9.52	
September.....	21,110	4,130	α 6,880	409,390	7.82	8.72	
October.....	17,540	3,390	α 7,480	459,930	8.50	9.80	
November.....	22,100	3,810	α 6,950	413,550	7.90	8.81	
December.....	22,100	3,670	α 6,810	418,730	7.74	8.92	
The year.....	38,790	495	4,850	3,514,390	5.51	74.88	
1898.							
January.....	27,250	2,260	5,650	347,400	6.42	7.40	
February.....	3,320	1,230	1,690	93,860	1.92	2	
March.....	2,400	810	1,090	67,020	1.24	1.43	
April.....	24,000	780	2,010	119,600	2.28	2.54	
May.....	8,510	1,450	2,680	164,790	3.05	3.52	
June.....	15,140	1,760	3,350	199,340	3.81	4.25	
July.....	23,860	2,890	6,570	403,970	7.47	8.61	
August.....	15,500	3,000	5,300	325,880	6.02	6.94	
September.....	8,540	2,150	3,210	191,000	3.65	4.07	
October.....	23,370	2,080	5,050	310,510	5.74	6.62	
November.....	27,890	3,810	7,840	466,510	8.91	9.94	
December.....	6,670	1,980	2,860	175,850	3.25	3.75	
The year.....	27,890	780	3,960	2,865,730	4.50	61.07	

α These means are obtained by averaging the discharge measurements.

THE PANAMA CANAL.

Estimated monthly discharge of Chagres River at Bohio—Continued.

OBSERVATIONS OF THE NEW PANAMA CANAL COMPANY.

Month.	Discharge in second-feet.			Total in acre-feet.	Run-off.		Rainfall at Colon.
	Maximum.	Minimum.	Mean.		Second- feet per square mile.	Depth in inches.	
1899.							<i>Inches.</i>
January	22,240	1,800	3,848	236,600	4.37	5.04	
February	2,540	1,340	1,730	95,960	1.98	2.06	
March	1,410	740	988	60,750	1.12	1.29	
April	3,280	635	1,090	64,860	1.24	1.38	
May	17,540	600	2,050	126,050	2.33	2.69	
June	5,470	1,910	3,140	186,840	3.57	3.98	
July	17,750	1,660	3,950	242,880	4.49	5.18	
August	25,870	3,040	5,720	351,700	6.50	7.49	
September	9,531	2,440	4,240	252,300	4.82	5.38	
October	16,130	2,720	4,940	303,750	5.61	6.47	
November	24,250	2,790	5,300	315,370	6.02	6.72	
December	15,280	940	3,600	221,360	4.09	4.72	
The year	25,870	600	3,400	2,458,420	3.86	52.40	

BASED ON DISCHARGE MEASUREMENTS MADE BY THE ISTHMIAN CANAL COMMISSION.

1899.							
January.....	21,830	1,900	4,150	255,170	4.72	5.44	6.93
February.....	2,810	1,300	1,795	99,690	2.04	2.12	6.49
March.....	1,420	665	943	57,980	1.07	1.23	1.26
April.....	3,670	570	1,057	62,900	1.20	1.34	1.43
May.....	17,440	540	1,900	116,830	2.16	2.49	13.90
June.....	6,440	2,030	3,500	208,260	3.98	4.44	6.41
July.....	17,730	1,710	4,372	268,820	4.97	5.73	27.68
August.....	24,520	3,370	6,425	395,060	7.30	8.42	14.80
September.....	10,225	2,680	4,858	289,070	5.52	6.16	16.55
October.....	16,285	2,990	5,511	338,860	6.26	7.22	15.04
November.....	23,320	3,080	5,732	341,080	6.51	7.26	14.49
December.....	15,430	2,210	3,950	242,880	4.49	5.18	9.04
The year.....	24,520	540	3,697	2,676,600	4.18	57.03	133.02

OBSERVATIONS OF THE NEW PANAMA CANAL COMPANY.

1900.							
January.....	10,410	1,480	2,120	130,350	2.41	2.78	2.44
February.....	1,380	810	1,060	58,870	1.20	1.25	.33
March.....	920	600	740	45,500	.84	.97	.32
April.....	1,765	490	706	42,010	.80	.89	3.78
May.....	9,672	490	1,620	99,610	1.84	2.12	10.45
June.....	10,840	1,340	2,860	170,180	3.25	3.63	15.67
July.....	21,180	2,010	5,610	344,940	6.38	7.36	15.28
August.....	19,240	2,440	5,260	323,420	5.98	6.89	12.36
September.....	18,070	2,260	4,770	283,830	5.42	6.05	14.33
October.....	23,860	3,180	7,000	430,410	7.95	9.17	15.48
November.....	20,300	3,740	6,600	392,730	7.50	8.37	15.10
December.....	22,870	1,800	3,740	229,960	4.25	4.90	3.13
The year.....	23,860	490	3,520	2,551,810	4	54.38	108.67

OBSERVATIONS OF THE ISTHMIAN CANAL COMMISSION.

1900.							
January.....	10,155	1,441	2,203	135,460	2.50	2.87	2.44
February.....	1,640	802	1,052	58,420	1.20	1.24	.33
March.....	877	570	681	41,870	.77	.88	.32
April.....	1,845	442	652	38,800	.74	.82	3.78
May.....	9,500	506	1,670	102,680	1.92	3.20	10.45
June.....	12,700	1,640	3,185	189,520	3.62	4.04	15.67
July.....	20,650	2,521	5,960	366,470	6.77	7.80	15.28
August.....	18,900	2,859	5,880	361,550	6.68	7.70	12.36
September.....	17,800	2,651	5,190	308,830	5.90	6.58	14.33
October.....	23,000	3,802	8,220	505,430	9.34	10.77	15.48
November.....	19,800	4,464	7,540	448,660	8.57	9.56	15.10
December.....	22,100	1,949	4,100	252,100	4.66	5.37	3.13
The year.....	23,000	442	3,880	2,809,790		60.83	108.67

Ratio of run-off to rainfall = 55 per cent.

COMPUTED FROM RATING TABLE OF ISTHMIAN CANAL COMMISSION, USING THE GAUGE HEIGHTS OF THE PANAMA CANAL COMPANY AS THE ARGUMENT.

1901.							
January.....	3,400	975	1,508	92,720	1.71	1.91	
February.....	2,680	618	937	52,040	1.06	1.10	
March.....	1,975	400	654	40,210	.74	.85	
April.....	2,443	255	521	31,000	.59	.65	
May.....	19,180	860	2,400	147,570	2.73	3.15	
June.....	13,010	1,390	2,890	171,970	3.28	3.66	

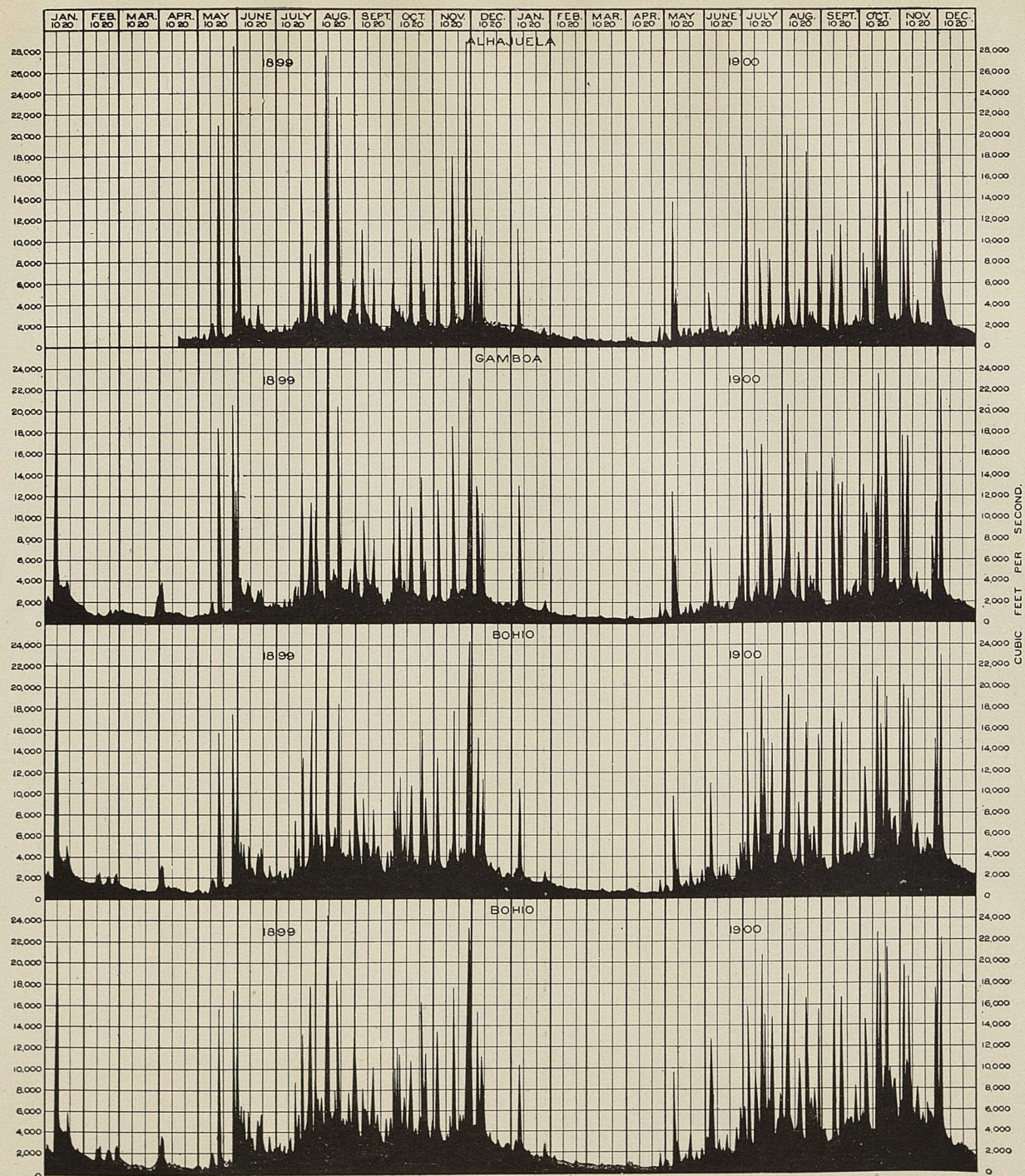


FIG. 8.—COMPARATIVE HYDROGRAPHS OF CHAGRES RIVER.

Alhajuela, Gamboa, and the upper Bohio diagram are drawn from observations of the Panama Canal Company; the lower diagram from observations of the Isthmian Canal Commission.

MINOR STREAMS ON THE ISTHMUS OF PANAMA.

In a minor degree it is required to know something of the volume and regimen of the Grande River, which is followed by the canal line from Paraiso to the Pacific. Also of the tributaries of the Chagres, which are to be conducted to the sea independent of the canal.

Observations on all of these were made in 1900 sufficient to show that no difficult problems are involved in their control. A detailed description of work and results on these streams will be found in Mr. Schlecht's report, page 49.

THE FLOODS OF THE CHAGRES RIVER.

The projected canal from Colon to Panama is 49 miles in length, and for more than half this distance it follows the valley of the Chagres. The magnitude of a flood discharge is, therefore, important in its relation to the works necessary for controlling and discharging the surplus waters without injury to the works or obstruction to navigation.

Although nearly the entire country is clothed with vegetation, much of which is very dense, the slopes are so precipitous and rock is so near the surface that the heavy tropical rainfall often produces sudden freshets in the river. A violent rainfall of a few hours converts the banks of the Chagres in the vicinity of Alhajuela and above into a series of small torrents and cascades, causing the river to rise very suddenly and flow with great velocity. It is this feature that lends such importance to the study of the floods of the Chagres in their relation to the proposed canal.

It is stated by the officers of the Panama Railway Company that previous to 1879 no freshet had occurred in the Chagres of magnitude sufficient to cause serious damage to the property of the railway company or to delay its operations. The flood of 1879 caused great damage to the railway and obstructed traffic for a considerable period, and the conclusion follows that it is far greater than any flood that had occurred since the construction of the railway was begun. This conclusion is verified by the testimony of the oldest inhabitant of the vicinity. Since 1879 sufficient attention has been paid by canal officials to render it certain that no flood equal in magnitude to that of 1879 has since occurred. The flood occurred in November, but we do not know how long it lasted, except that a violent storm raged for six days throughout the Isthmus, the precipitation amounting to 320 millimeters at Panama.

The only actual information existing other than that of a general character from the officials of the Panama Railway above referred to is the testimony of Mr. Sosa, a young Colombian engineer, who, by inquiry from the inhabitants or otherwise, decided upon certain points as high water of this flood in the neighborhood of Bohio. Mr. Sosa was unfortunately drowned in the sinking of *La Bourgogne*, and it is not known how accurate his information was, excepting that he expressed his entire reliance upon it. By his testimony the river reached an elevation of 12 meters above sea level at Bohio at its maximum stage. From the measurements made of the floods of less magnitude and from the large number of measurements of the river at its lower stage a curve has been plotted expressing the relation of discharge and gauge height at Bohio. The existing observations are in satisfactory accordance, and by extrapolating this curve upward to include the flood of 1879 we obtain a discharge for the river at Bohio for gauge height 12 meters of 3,860 cubic meters, or about 136,000 cubic feet per second. The curve and the data upon which it is based are shown in figure 9, page 44, and is submitted as being the best conclusion permitted. The plotted observations are those given by General Abbot in his note "Sur le régime du Chagres," pages 24 to 27. The results obtained by the company at Bohio, being in practical accordance with those observed by this Commission, are accepted without change.

In this connection three points should be remembered—first, the discharge for "cote" 12 may be greater than here indicated; second, the flood of 1879 may have reached a higher elevation than "cote" 12; third, it is possible that a larger flood than that of 1879 may occur in the future.

Under these circumstances it seems unsafe to consider the maximum flood wave to be expected on the Chagres at less than 140,000 cubic feet per second.

Four great floods occurred since 1879, of which the discharge at Gamboa has been measured or estimated by the Panama Canal Company as follows:

Date.	Gauge height.	Discharge maximum.	Mean for forty-eight hours.	Percentage.
	<i>Meters.</i>	<i>Cub. meters.</i>	<i>Cub. meters.</i>	
November 25, 1885	23.60	1,638	1,077	66
December 12, 1888	23.56	1,628	1,270	78
December 1, 1890	23.70	1,663	833	50
December 17, 1893	21.72	1,207	759	63

The flood of 1879 seems to have been longer sustained than any of the others, and it seems likely that the ratio of the mean discharge to the maximum should be somewhat greater than the others. General Abbot has assumed the mean discharge for forty-eight hours for this flood at 80 per cent of the maximum, and this appears to be approximately correct. Eighty per cent of 140,000 cubic feet per second is 112,000 cubic feet per second.

The area of Lake Bohio according to the plans adopted by this Commission are as follows:

Elevation.	Acres.	Square miles.
20 meters, 65.6 feet.....	14, 771	23. 08
25 meters, 82 feet.....	23, 542	36. 78
26 meters, 85.3 feet.....	24, 627	38. 48

The waste weir provided is to be at elevation 85 feet above sea level, or slightly less than 26 meters, and is to have a length of 2,000 feet.

This flow continued for forty-eight hours would result in a total discharge of 444,300 acre-feet, or about 9,300 acre-feet per hour. A sustained discharge of this amount would raise the lake to a head of about 6.6 feet over sill of the weir. This, however, may be taken as the extreme condition, which will never be exceeded.

LOW WATER OF THE CHAGRES RIVER.

The Chagres River being the one feeder to the summit level of the proposed canal, its mean and minimum flow is very important as indicating the amount of storage that must be provided for the operation of the canal during the dry season. The mean flow for the various months of the year, as determined by averaging all the monthly records at Gamboa in our possession since 1892, are as follows:

	Cubic feet per second.		Cubic feet per second.
January.....	3, 310	July.....	4, 010
February.....	1, 330	August.....	4, 080
March.....	900	September.....	3, 540
April.....	1, 530	October.....	4, 110
May.....	3, 180	November.....	4, 790
June.....	3, 280	December.....	4, 850

Averaging these quantities, we find the mean annual discharge is about 3,200 cubic feet per second. As the maximum consumption and loss by leakage and evaporation is but little over 1,000 cubic feet per second, we find that the month of March is the only month having a shortage in average years; and as this is the record for Gamboa, and the discharge at Bohio, where the dam is to be constructed, is somewhat greater, we may say that the requirements of the canal are less than the average discharge for any month in the year, and if one-third of the mean annual flow can be saved there will be ample for canal purposes.

But some years furnish less water than others, and to correctly solve the problem of the water supply for the canal we must consider the yield in the minimum year.

In all the observations at Bohio the lowest monthly means are for the months of March and April, 1891, as shown by the mean of eight measurements made in each of those months. More records, however, are at hand for Gamboa than for Bohio, and the lowest reading at the Gamboa fluvograph occurred in the spring of 1901, when for a period of about two and one-half months the fluvograph at Gamboa indicated a stage of less than 14 meters almost continuously, the lowest point reached being 13.72. This corresponds to a discharge of from 7 to 8 cubic meters per second.

In the examination of the records in Paris an elevation at Gamboa was found for the last three days of March, 1896, giving the stage as 13.1 meters. The rest of the month of March and the first half of April are entirely blank, no record having been kept. It is impossible that this could have been a fluvograph record, since the river would be dry before reaching so low an elevation at the location of the fluvograph, and the only plausible explanation is that given by the officers of the company that this record was on a rod a considerable distance below the fluvograph, and this statement is confirmed by the reports of discharge on these dates, which are 16.8, 16.8, and 16.9 cubic meters, respectively, leaving no doubt that at this time the river was above the stage reached in 1900. No other record gives a stage of river as low as that reached in 1900.

The year 1891, however, seems to have had a still lower dry-season flow. No fluvograph records were obtainable, except such as were given with the measurements of discharge, and these do not show any stage as low as that of 1900, but in so long a time the bed of the river

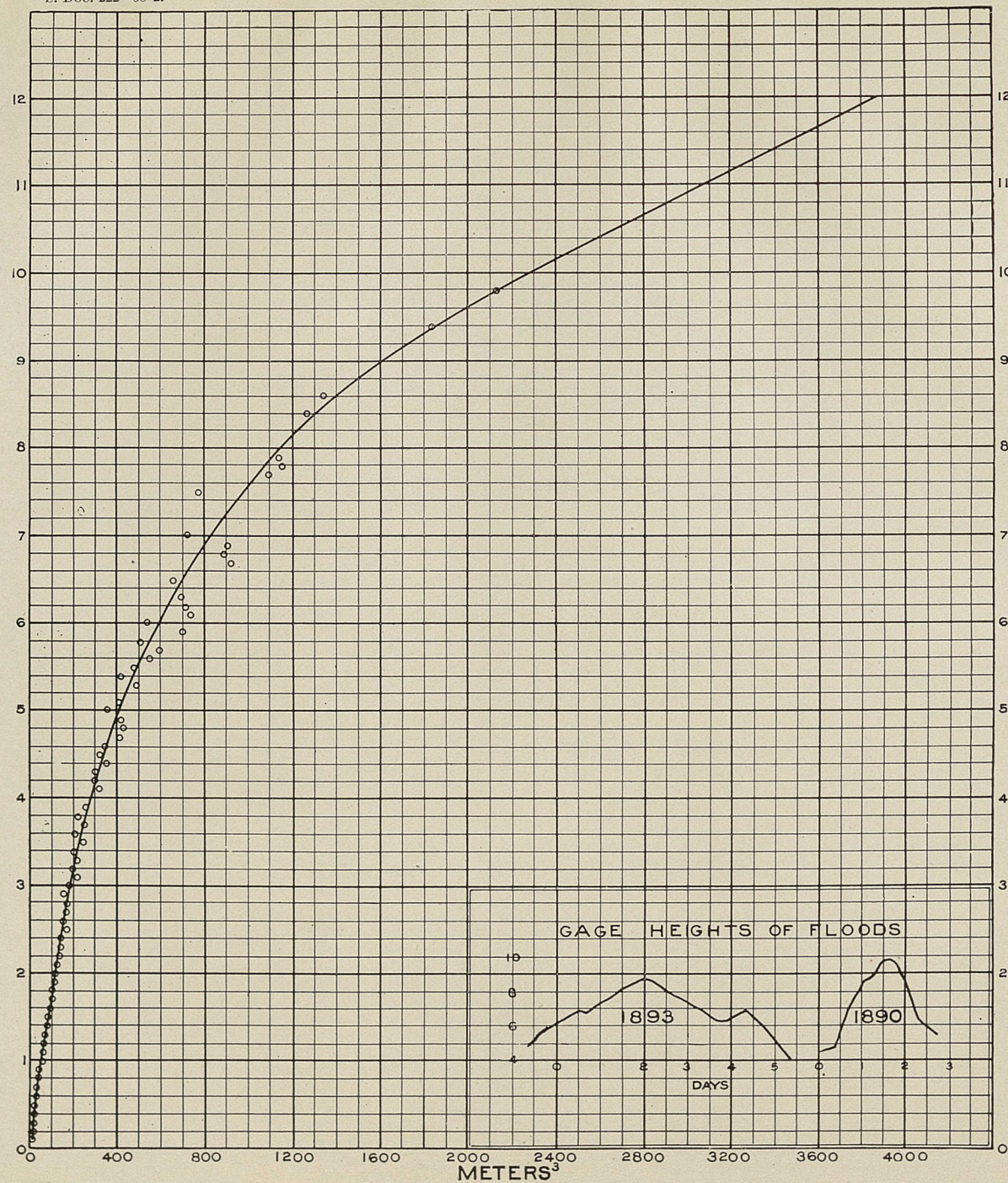


FIG. 9.—RATING CURVE FOR FLOODS OF THE CHAGRES RIVER AT BOHIO.

Constructed by the Isthmian Canal Commission from observations of the Panama Canal Company, 1890-1899

would be likely to change, and the measurements show that it has done so. Of actual gaugings we have (p. 230) 8 for February, 9 for March, and 8 for April. The measurement on the 1st of May shows low water, while that of May 4 exhibits a freshet.

The indications of the gaugings are that up to February 14 the discharge of the river was above the requirements of the canal, while from February 15 to May 3 they were below. This is the same period for which 8 measurements per month were made at Bohio. The means are 39 cubic meters per second for February, 17 for March, and 17 for April. This is the lowest discharge at Bohio of which we have record. It is, of course, not certain that a drier year has not occurred or will not occur, and in making the estimates it will be assumed that the mean of 17 cubic meters or 600 cubic feet per second is continuous for ninety days, from February 1 to May 1.

The requirements and losses of the canal in operation, as estimated by this Commission, are as follows:

	Cubic feet per second.
Lockage.....	406
Power.....	200
Leakage.....	257
Evaporation.....	207
Total.....	1,070

Applying this quantity, we have for the ninety days, from February 1 to May 1:

	Acre-feet.
Total leakage, lockage, and evaporation, 1,070 cubic feet per second.....	191,000
Less inflow.....	107,000
Deficit.....	84,000

Taking 24,000 acres as the area of the lake, this will lower the lake about 3.5 feet. An allowance of this amount is therefore ample for the most extreme case.

This brings the elevation of the lake to 81.5 feet, which may be regarded as the elevation reached in the minimum year of a long series, and would occur perhaps only once in a generation. If the consumption and loss should be less than 1,070 cubic feet per second the deficit would, of course, be diminished.

The dry season of 1900 would have brought the lake level to an elevation of about 82.5.

The possibility always remains of constructing a large reservoir at Albajuela, which will not only guard against all possibility of annoying currents in Lake Bohio, but will store sufficient water for use in the dry season so that the lake need never decline below the sill of the waste weir.

It is indeed true, as stated by General Abbot, that the Chagres is a river well adapted to the service of the proposed Panama Canal.

Discharge measurements made on Chagres River at Gamboa.

[Made by Panama Canal Company.]

Date.	Height.	Dis-charge.	Date.	Height.	Dis-charge.	Date.	Height.	Dis-charge.
1889.			1890.			1890.		
	Meters.	Sec. feet.		Meters.	Sec. feet.		Meters.	Sec. feet.
September 27.....	15.4	4,448	February 14.....	14.5	1,412	May 23.....	14.8	2,824
September 30.....	15.4	4,977	February 21.....	14.4	1,412	May 27.....	15.2	3,990
			February 27.....	14.3	1,236	May 30.....	14.8	2,753
October 2.....	15.7	5,154	Mean.....		1,353	Mean.....		5,271
October 5.....	15.8	5,613	March 1.....	14.3	1,270	June 3.....	15.4	4,483
October 9.....	15.1	3,954	March 6.....	14.2	1,094	June 6.....	15.6	4,518
October 12.....	15.4	5,330	March 11.....	14.2	882	June 7.....	16.8	10,307
October 15.....	15.0	3,459	March 19.....	14.2	882	June 10.....	16.4	8,472
October 19.....	15.9	6,107	March 27.....	14.4	1,341	June 13.....	14.8	2,471
October 23.....	15.4	4,977	Mean.....		1,094	June 17.....	16.8	11,120
October 26.....	15.1	3,600	April 8.....	14.2	812	June 20.....	15.5	4,730
October 30.....	15.2	4,165	April 10.....	14.1	600	June 24.....	15.1	3,636
Mean.....		4,707	April 14.....	14.1	600	June 27.....	15.2	3,777
November 2.....	15.4	5,366	April 17.....	14.1	565	Mean.....		5,946
November 6.....	15.2	4,236	April 19.....	14.1	600	July 1.....	15.4	4,483
November 9.....	15.4	4,942	April 22.....	14.1	586	July 7.....	15.2	3,954
November 14.....	15.0	3,812	April 25.....	14.1	777	July 8.....	15.2	3,636
November 16.....	15.4	4,765	April 29.....	14.4	1,306	July 8.....	17.2	11,331
November 20.....	15.2	4,660	Mean.....		731	July 16.....	15.6	5,013
November 22.....	17.1	11,614	May 3.....	17.6	14,470	July 18.....	15.4	4,518
November 30.....	15.1	3,600	May 6.....	14.8	3,530	July 22.....	15.2	3,777
Mean.....		5,374	May 9.....	14.4	1,165	July 25.....	16.6	8,930
1890.			May 12.....	15.8	6,707	July 29.....	15.0	3,000
January 13.....	16.8	9,531	May 16.....	16.1	8,013	Mean.....		5,404
Mean.....		9,531	May 20.....	15.2	3,990	August 1.....	14.8	2,400
						August 5.....	15.4	4,624

Discharge measurements made on Chagres River at Gamboa—Continued.

Date.	Height.	Dis-charge.	Date.	Height.	Dis-charge.	Date.	Height.	Dis-charge.
1890.	Meters.	Sec. feet.	1891.	Meters.	Sec. feet.	1891.	Meters.	Sec. feet.
August 8.....	15.3	4,377	January 27.....	14.6	1,341	July 17.....	14.7	1,977
August 12.....	15.8	5,720	January 30.....	14.5	1,341	July 21.....	15.0	3,000
August 14.....	16.2	7,554	Mean.....		2,051	July 24.....	14.7	1,588
August 19.....	15.3	4,306	February 3.....	14.5	1,236	July 28.....	15.5	4,483
August 22.....	15.2	4,060	February 6.....	14.5	988	July 31.....	14.7	1,906
August 26.....	15.9	6,954	February 10.....	14.4	1,130	Mean.....		3,793
August 29.....	15.3	4,448	February 13.....	14.5	1,024	August 4.....	14.9	2,683
August 31.....	17.1	12,320	February 17.....	14.4	847	August 7.....	15.2	3,777
Mean.....		5,676	February 20.....	14.4	928	August 11.....	14.9	2,577
September 5.....	15.4	4,977	February 24.....	14.3	709	August 14.....	14.7	2,259
September 9.....	15.2	4,130	February 27.....	14.3	709	August 18.....	15.0	3,142
September 12.....	15.6	5,330	Mean.....		946	August 21.....	14.8	2,330
September 13.....	17.7	15,890	March 3.....	14.3	720	August 25.....	15.0	3,354
September 16.....	16.0	6,920	March 6.....	14.3	681	August 28.....	14.7	2,120
September 19.....	15.5	5,083	March 10.....	14.3	745	Mean.....		2,780
September 21.....	16.3	8,225	March 13.....	14.3	582	September 1.....	15.4	4,130
September 26.....	15.4	4,342	March 17.....	14.2	540	September 4.....	15.1	3,000
September 30.....	15.9	6,460	March 20.....	14.2	551	September 8.....	15.3	4,342
Mean.....		6,817	March 24.....	14.2	600	September 11.....	14.8	2,259
October 3.....	15.4	4,518	March 27.....	14.2	434	September 15.....	14.7	2,083
October 7.....	15.4	4,660	March 31.....	14.2	515	September 18.....	14.7	1,518
October 10.....	15.9	5,825	Mean.....		596	September 21.....	15.7	4,590
October 14.....	15.3	3,742	April 3.....	14.2	572	September 29.....	14.8	2,330
October 17.....	15.2	4,024	April 7.....	14.1	568	Mean.....		3,032
October 21.....	15.1	3,600	April 10.....	14.1	547	October 2.....	14.9	2,316
October 24.....	17.1	10,307	April 14.....	14.1	441	October 6.....	14.7	2,120
October 25.....	18.4	17,960	April 17.....	14.1	614	October 9.....	14.6	1,870
October 31.....	15.3	3,777	April 21.....	14.2	522	October 13.....	15.1	3,390
Mean.....		6,483	April 24.....	14.1	541	October 16.....	16.4	6,848
November 4.....	15.2	3,777	April 28.....	14.1	473	October 20.....	15.4	4,095
November 7.....	15.1	3,565	Mean.....		535	October 23.....	15.6	4,836
November 11.....	15.4	4,412	May 1.....	14.2	512	October 27.....	15.5	4,695
November 14.....	15.2	3,990	May 4.....	18.2	18,285	October 30.....	15.4	3,954
November 18.....	15.0	3,018	May 8.....	14.5	1,701	October 31.....	17.0	10,555
November 22.....	15.6	5,048	May 12.....	14.1	618	Mean.....		4,468
November 25.....	15.4	4,589	May 15.....	14.1	512	November 3.....	15.1	3,142
November 29.....	15.1	3,248	May 19.....	15.0	3,036	November 6.....	15.5	4,377
Mean.....		3,956	May 22.....	14.4	1,423	November 10.....	15.4	3,636
December 2.....	18.0	14,296	May 26.....	14.7	2,404	November 13.....	15.2	3,565
December 5.....	15.6	5,471	May 29.....	14.8	2,580	November 17.....	14.9	2,294
December 9.....	15.6	6,178	Mean.....		4,524	November 19.....	19.2	26,475
December 12.....	15.2	3,600	June 2.....	14.7	2,051	November 24.....	15.3	3,848
December 16.....	15.4	4,554	June 5.....	14.7	2,365	November 27.....	17.5	12,637
December 19.....	15.5	5,295	June 9.....	14.7	2,230	Mean.....		7,497
December 23.....	15.4	4,836	June 12.....	14.3	1,097	December 1.....	16.9	9,955
December 26.....	15.4	5,013	June 16.....	14.5	1,750	December 4.....	15.6	4,907
December 30.....	15.2	3,990	June 19.....	15.8	6,080	December 8.....	17.5	12,885
Mean.....		5,915	June 23.....	14.7	1,860	December 11.....	15.1	2,930
1891.			June 26.....	14.4	1,377	December 15.....	15.0	2,790
January 2.....	15.1	3,353	June 30.....	14.8	2,602	December 18.....	14.8	2,153
January 6.....	14.9	2,612	Mean.....		2,379	December 22.....	14.8	3,083
January 9.....	15.1	3,177	July 3.....	14.5	1,518	December 27.....	14.7	1,730
January 13.....	14.7	1,836	July 7.....	14.6	1,871	December 29.....	14.6	1,590
January 16.....	14.7	1,765	July 8.....	17.5	11,967	Mean.....		4,670
January 20.....	14.6	1,482	July 13.....	16.0	5,825			
January 23.....	14.6	1,553						

RAINFALL.

Notwithstanding the fact that the isthmus is narrow and the cordillera low in the vicinity of Panama, there is a striking difference of rainfall between the Caribbean or northern and the Pacific or southern coast of the isthmus in this region. The contrast is similar, though not so striking, as that between the east and west coasts of Nicaragua. The mean rainfall at Colon as shown by a record of nearly thirty years is 130 inches, while that at Panama, though not so well determined, is only 66.8. The precipitation at Panama and vicinity is confined almost entirely to the months from May to November, inclusive, the rest of the year being the dry season. At Colon, though the rainfall is less during the dry season, there is still quite a precipitation, the mean for April being 4.54 and for January 3.73, while these two months are dry on the Panama side. February and March are less likely to yield any considerable precipitation, but no month is exempt from rainfall on the coast of the Caribbean. The upper portion of the Chagres drainage basin represents the mean between these extremes. (See map.)

The longest record of rainfall in the vicinity is that of Colon, which continues from March, 1881, to date, with the exception of a period of twenty-one months in 1888-89. This station, however, is not on the Chagres drainage, and the records which are in that basin are not so long. They are mainly at Gamboa and Obispo, with very short records at Bohio, Gorgona, and Alha-

juela. Short records also exist at Panama, at La Boca, the south end of the canal, and at Naos, an island in the Bay of Panama.

The long records of stream flow at Gamboa and Bohio, together with an approximate knowledge of the area of the drainage basin, and the observations of rainfall, constitute a valuable contribution to the knowledge of the ratio of rainfall to run off in the Tropics, the weakest point being the information on rainfall, the only records in the basin being confined to the lower valley, and being, moreover, very short. If a definite relation could be established between the long Colon record and the mean of the Chagres basin, it would increase the value of these data to an important extent. For this purpose five stations above Alhajuela were established in 1899, two at different points on Rio Pequeni, at Salamanca and Las Minas, one on the Upper Chagres at Santa Barbara, and two at high elevations above the river valley. Of the latter one is on a hill between the Chagres and Pequeni, 700 feet above the nearest point of the Chagres, and the other is between the Chagres and Puente, about 500 feet above the river. Salamanca and Las Minas were discontinued at the end of 1900. The others were discontinued October 30, 1900. Their relative location is shown on the map. These stations, considered with those lower in the basin, are taken as the mean for the basin, and compared with observations at Colon as follows:

Rainfall at Colon compared with the mean rainfall in the basin of the Chagres River.

Month.	Colon.				Basin.
	1872 (maxi- mum).	1884 (mini- mum).	1900.	Mean for 30 years.	1900.
	<i>Inches.</i>	<i>Inches.</i>	<i>Inches.</i>	<i>Inches.</i>	<i>Inches.</i>
January.....	3.57	3.39	6.06	3.56	2.44
February.....	.75	.39	.33	1.52	.33
March.....	.83	.39	1.06	1.55	.32
April.....	1.30	4.33	.75	4.42	3.78
May.....	21.43	10.16	12.25	12.27	10.45
June.....	22	10.32	11.65	13.46	15.67
July.....	19.90	15.59	16.81	16.92	15.28
August.....	19.97	13.27	17.04	15.53	12.36
September.....	16.20	9.37	9.37	12.77	14.33
October.....	30.32	8.66	16.33	14.20	15.48
November.....	19.11	7.05	20.28	20.56	15.10
December.....	13.12	3.62	12.49	12.49	a 3.59
Total.....	168.50	86.54	120.97	129.25	109.13

Mean rainfall on basin: 129.25, 109.13, 120.97, or 116.60 inches.

a To obtain a comparison for a complete year, the rainfall for December, 1899, is included, because during December, 1900, many stations in the basin were discontinued.

Monthly rainfalls.

[1862-1874, by Drs. White and Kluge, surgeons Panama Railroad Company; 1881-1898, by Panama Canal Company; 1898-1900, by Panama Railroad Company.]

COLON.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
1862.....										13.10	43.01	16.83	
1863.....	1.75	2.94	.85	4.70	13.09	15.32	25.76	10.34	15.54	11.22	17.59	15.21	134.81
1864.....	1.90	.77	.78	.44	15.87	8.78	16.54	13.37	17.85	12.83	17.90	16.40	123.43
1865.....	1.10	1.08	.02	3.89	9.22	16.85	9.61	18.39	8.55	9.69	22.16	5.58	106.14
1866.....	3.99	1.07	.21	4.07	14.76	12.17	16.72	12.72	18.82	15.04	21.72	8.42	129.71
1867.....	1.56	.80	.48	1.20	11.88	8.85	16.03	19.82	5.35	20.50			
1868.....	1.17	2.77	2.18	.87	7.24	18.11	20.60	12.50	16.16	13.13	21.58	3.72	120.03
1869.....	.83	.77	.49	5.04	6.72	10.66	18.22	14.02	8.98	14.82	24.13	10.10	114.78
1870.....	4.30	3.33	4.95	6.46	20.95	12.48	15.60	16.35	6.74	11.21	32.42	14.85	149.64
1871.....	15.42	.53	.05	1.52	1.68	7.70	23.27	11.56	8	12.58	12.38	4.94	99.58
1872.....	3.57	.75	.83	1.30	21.43	22	19.90	19.97	16.20	30.32	19.11	13.12	168.50
1873.....	6.33	.25	.13	2.18	3.92	13.20	12.50	10.69	10.91	14.30	11.77	.94	87.12
1874.....	5.33	1.34	3.94	18.02	8.92	15.87	13.62	17.28	8.22	16.65	20.62	7.89	137.70
1881.....			1.08	2.52	10.04	15.28	12.24	6.46	6.30	12.91	22.13	10.35	
1882.....	1.65	1.10	1.69	1.73	13.23	18.90	19.10	13.94	10.63	14.96	22.09	5.08	124.10
1883.....	1.85	.47	.55	1.77	11.85	10.08	13.39	25.43	11.14	16.77	11.10	10.94	115.34
1884.....	3.39	.39	.39	4.33	10.16	10.32	15.59	13.27	9.37	8.66	7.05	3.62	86.54
1885.....	.87	.59	.55	1.34	7.91	16.61	22.99	20.32	17.44	7.99	24.17	25.51	146.29
1886.....	2.13	5	9.17	1.58	13.15	16.38	11.10	12.20	7.52	14.33	21.89	22.72	137.17
1887.....	2.01	.67	.47	10.63	10.28	16.50	17.05	16.89	15.63	19.61	31.81	13.33	154.88
1888.....	.63	1.58	1.26										
1889.....													
1890.....	7.24	1.02	2.01	2.99	9.76	17.24	10.24	20.51	22.99	21.77	19.49	19.06	154.32
1891.....	2.52	.51	1.50	.51	23	7.99	14.02	15.98	17.48	17.48	19.49	4.25	124.73
1892.....	.98	2.01	3.98	5	18.03	16.97	21.77	15.98	16.26	6.69	26.30	11.30	145.27
1893.....	1.73	3.82	1.81	8.07	6.58	12.32	11.50	15.12	9.92	12.28	17.80	30.94	131.89
1894.....	5.35	1.65	.35	2.16	9.85	12.24	19.10	23.03	18.78	12.40	23.66	25.12	153.69
1895.....	3.86	1.89	2.09	21.73	16.77	9.25	17.09	14.13	12.09	16.46	20.47	15.71	151.54
1896.....	4.02	1.30	2.01	9.02	16.46	8.50	13.58	15.51	12.84	13.98	15.63	18.66	131.51
1897.....	3.42	.04	.28	3.74	16.34	18.82	14.06	17.24	17.20	5.83	22.16	18.90	138.03
1898.....	5.04	.35	1.58	4.72	12.83	16.38	21.89	10.91	10.24	11.38	12.28	7.95	115.55
1899.....	6.93	6.49	1.26	.43	13.90	6.41	27.68	14.80	16.55	15.04	14.49	9.04	133.02
1900.....	6.06	.33	1.06	.75	12.25	11.65	16.81	17.04	9.37	16.33	20.28	4.13	116.06
Mean.....	3.56	1.52	1.55	4.42	12.27	13.46	16.92	15.53	12.77	14.20	20.56	12.49	129.25

THE PANAMA CANAL.

Monthly rainfalls—Continued.

BOHIO.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
1896					15.63	8.54	5.55		25.51	13.35	17.05		
1897			3.07	8.11	13.54	14.10	15.83	25.20	17.48	26.02	19.57	22.05	
1898	12.36	1.26	3.03	10.59	14.61	19.76	34.96	38.31	13.31	28.23	21.81	6.38	204.61
1899	9.87	4.49	3.28	1.10	10.35	14.80	17.76	12.99	8.90	19.33	10.43	6.18	118.98
1900	7.06	.48	1.04	2.89	7.42	18.40	17.79	14.02	15.40	18.43	24.98	4.02	131.93
Mean	9.60	2.08	2.60	5.67	13.31	15.12	18.38	22.63	16.12	21.07	18.77	9.66	155.01

GORGONA.

1896					9.02	6.81	5.16	8.98	13.35	10.94	14.37		
1897		.08		2.60	25.12	13.54	9.65	16.93	15.98	14.41	7.16	7.91	
1898	3.42	.20	.00	1.38	5.04	4.37	18.50	19.88		7.72	9.61	3.94	
1899	3.78	2.01	3.31									4.58	
1900	2.63	.13	.20	3.15	7.07	12.48	19.72	11.04	11.25	11.93			
Mean	3.28	.60	1.17	2.38	11.56	9.30	13.26	14.21	13.53	11.25	10.38	5.48	96.40

GAMBOA.

1881						10.63	12.40	9.17	10.39	11.06	12.95	4.76	
1882			.51	1.50	15.47	6.26	10.12	9.88	11.81	8.15	11.81	1.38	
1883				2.60	9.68		11.02	6.54	15.94	4.13	10.04	7.01	6.30
1884		.71	.28	6.46	6.18	13.35	9.62	16.50	10.55	22.36	6.18	2.20	
1885	.20	.20	.00	1.38	11.06	10.35	9.06	15.51	16.10	9.33	13.23	11.06	97.48
1886	.55	1.06	.71	2.76	15.71	10.55	11.69	16.38	9.13	13.62	16.10	4.61	102.87
1887	2.20	.08	.28	6.85	11.02	19.45	14.02	19.17	11.50	14.88	24.06	16.28	136.19
1888	.12	.63	.35	1.26	20.47	11.93	3.27	10.24	12.28	9.57	16.18	16.34	102.64
1889	1.97	4.53	1.42	.00	4.37	9.10	7.28	10.51	11.42	13.07	8.70	3.35	75.72
1890	4.06	.35	2.36	3.03	13.27	11.65	10.43	15.35	8.90	21.41	9.92	4.29	105.03
1891	.63	.00	.35	2.13	7.48	9.29	6.06	8.50	10.47	15.71	10.67	6.38	77.67
1892	1.10	.67	2.56	4.72	16.81	8.54	13.98	14.33	13.74	11.10	10.24	6.58	104.37
1893	.67	1.06	.71	7.44	11.89	10.71	15.87	7.95	10.24	16.50	13.90	20.87	117.81
1894	1.46	.16	.04	1.34	10.94	8.78	10.08	8.42	15.16	15.28	10.67	8.27	90.60
1895													
1896					3.35	3.31	5.79						
1897	.20	.28		3.23	17.44	12.64	9.10	17.20	18.82	12.80	5.91	8.62	
1898	2.76	.12	.00	1.42	5.32	4.65	18.43	20.16	4.10	8.70	14.57	2.40	82.60
1899	5	1.73	1.34	1.42	8.54	8.78	9.45	10.95	13.46	7.95	8.70	2.68	80
1900	1.01	.16	.13	3.21	6.76	12.15	13.45	8.92	9.24	12.11	10.67	.79	78.60
Mean	1.67	.78	.71	2.99	10.88	10.17	10.35	13.06	11.19	12.98	11.74	7.06	93.58

BAS OBISPO.

1884	1.10	.35	.35	2.16	3.86	11.97	8.46	10.51	13.98	12.40	6.93	2.64	74.71
1885	.24	.12	.00	1.14	7.36	8.74	6.34	12.84	12.87	8.94	11.10	11.50	81.23
1886	.83	.87	.87	1.69	17.44	12.91	9.02	8.27	10.20	9.87	15.63	3.90	91
1887	2.28	.00	.12	3.19	10.16	15.43	10.87	8.03	10.98	7.36	14.06	15.79	98.27
1888	.16	.51	.32	.91	11.58	11.93	1.93	7.76	7.13	3.82	10.59	10.08	66.72
1889	.79	.32	1.10	6.50	4.57	9.84	6.43	8.98	13.46	13.23	8.54	3.78	77.53
1890	4.53	.12	2.56	2.56	13.07	14.96	9.88	12.16	13.03	4.50	6.54	7.95	92.16
1891	6.10	.00	.35	2.16	5.83	11.54	6.10	7.76	12.32	14.49	8.35	8.72	83.82
1892	.79	.63	2.56	4.76	15.16	10.51	13.10	11.93	17.36	10.28	11.02	6.97	105.07
1893	.55	1.34	.71	3.19	19.77	10.75	14.37	9.57	10.63	19.84	13.55	19.13	123
1894	1.97	.32	.20	1.38	11.89	10	10.59						
Mean	1.76	.42	.83	2.69	10.95	11.69	8.83	9.78	12.20	10.45	10.61	9.05	89.26

CULEBRA.

1883 ^a				1.26	11.50	10.32		10.71	5.47			5.04	
1884	.20	.04	.67	3.78			10.24	13.62	8.70	10.12	11.18	1.46	
1885	.28	.00	.00	1.61	15.39		4.29	5.35	11.38	10.63	7.20		
1886	.43	.55	.16	1.18	13.78	12.64	14.33	8.19	7.36	20.63	14.29	5.43	98.97
1887	3.15	.00	.04	3.50	7.91	9.96	6.10	8.74	7.61	9.65	12.13	7.95	76.77
1888	.24	.04	.51	.35	11.42	7.99	2.91	6.54	11.26	5.75	9.53	7.72	64.25
1894									12.13	13.50	13.23		
1895		.75											
Mean	.86	.23	.28	1.95	12	10.23	7.57	8.86	9.13	11.71	11.26	5.52	79.60

PANAMA.

1879	.04	2.52	5.71	5.55	10.28	6.46	7.91	7.24	9.02	9.50	19.21	.98	84.73
1880	1.89	.12	.16	1.61	4.45	5	9.88	11.46	7.91	11.81	6.46	5.51	66.26
1881	.16	.16	.35	3.23	10.35	13.78	7.20	4.49	8.94	9.68	9.72	2.48	70.55
1882	.00	.12	.00	.98	5.24	6.18	5.35	4.05	4.05	6.69	10.91	2.01	45.60
1894 ^b						7.48	12.72	6.42	7.32	11.51	6.73	8.27	
1895 ^b	1.38	.08											
1899 ^b									6.14	11.98	8.92	4.35	
1900 ^b	.76	.00	.00	2.24	11.21	8.91	15.81	5.99					
Mean	.70	.60	1.24	2.72	8.31	7.97	9.81	6.61	7.23	10.08	10.32	3.93	69.52

^a Observations at Emperador.^b Observations at La Boca.

Monthly rainfalls—Continued.

NAOS.

Year.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
1881.....											7.48	2.87	
1882.....	.00	.04	.04	.94	4.53	5.08	1.61	1.14	1.14	2.72	5.04	2.84	25.12
1883.....	.59	.00	.00	1.93	4.45	2.56	5.47	5.51	4.64	4.25	7.24	3.11	39.76
1884.....	2.01	.04	.00	.94	4.84	4.17	4.65	2.99	5.71	8.31	8.62	1.34	43.62
1885.....	.55	.20	.00	.91	2.56	5.63	2.64	5.51	4.17	9.96	4.37	4.92	41.42
1886.....	.00	.28	1.81	1.92	5.20	6.26	9.61	5.91	20.75	6.10	4.49	3.74	66.06
1887.....	.79	.04	.04	2.36	5.24	9.88	4.06	6.69	6.97	7.05	8.82	5.59	57.52
1888.....	.08	.00	.28	5.55	4.88	2.01	5.71	8.58	8.11	4.02	1.30		40.55
1889.....	.00	.00	.91	4.41	5.79	5.43							
Mean	.50	.08	.35	1.71	4.77	5.49	4.29	4.78	7.42	6.64	6.26	3.21	45.50

TEMPERATURE.

Monthly temperature observed by the Panama Canal Company, in degrees Fahrenheit.

Month.	La Boca.			Alhajuela.		
	Maximum.	Minimum.	Mean.	Maximum.	Minimum.	Mean.
1899.						
July.....	91	70	78.8	88	72	79.7
August.....	88	68	75	94	71	80
September.....	91	69	77	97	72	80.6
October.....	86	70	76.4	94	71	78.8
November.....	88	69	76.8	91	72	79.5
December.....	89	68	77.9	91	66	78.9
1900.						
January.....	88	68	77.7	90	66	76.8
February.....	89	70	77.5	92	66	79.5
March.....	90	68	79.7	98	64	79.7
April.....	90	68	80	94	64	80.9
May.....	90	73	80.9	94	68	78
June.....	89	74	80.4	94	70	78.9
July.....	88	73	79.3	90	70	77.3
August.....	91	72	80.3	90	70	78.4
September.....	91	72	80.6	92	70	76.4
October.....			79.4	89	69	77.7
November.....	88	72	78.8	89	69	77.3
December.....	89	73	80.2	90	68	78.1
The year	91	68	79.6	94	64	78.3

REPORT OF W. W. SCHLECHT, HYDROGRAPHER FOR PANAMA.

WASHINGTON, D. C., February 27, 1901.

SIR: I have the honor to submit the following report of my work on the Isthmus of Panama, performed under your direction:

Although during former years hydrographic records were collected by the Panama Canal Company on several streams, at present all investigations are confined to the Chagres River, which really presents the only important hydrographic problems relative to an interoceanic canal across the Isthmus of Panama, and as Alhajuela, Gamboa, and Bohio have at different times been considered as probable dam sites, these places have been selected as hydrographic stations. In order to obtain the daily mean gauge height and a record of the rapid fluctuations of the river, the canal company has installed continuous self-registering river gauges, or "fluviographs," at these points with an observer at each. The scale of the fluviograph record is 1 millimeter (vertical)=2 centimeters of rise; and 5 millimeters (horizontal)=1 hour, so that each centimeter of rise and each ten minutes of time may be easily read. It was at once apparent that no improvement on this means of obtaining the desired information could be introduced, so the records were simply checked each day at 6 a. m. and 6 p. m., and with very few exceptions no material error was ever found.

The measurements for the discharge are made by means of floats of 1 to 3 feet immersion and 2 to 3 inches diameter, the latter size being used only at times of flood, so all may be considered as surface floats, and a length of course of 60 meters at Alhajuela, and Gamboa, and 80 meters at Bohio. Cross-sections of the courses are taken 10 meters apart; and from these a table of mean sectional areas for different stages of the river is computed for each station. Following are the instructions to the observers concerning the method of obtaining the discharge, which are carefully followed:

The observations should be made in calm weather. The floats should have but a small portion exposed above the surface of the water. The float should be liberated a short distance above the first profile, so that at the moment it crosses that section it will have attained the velocity of the water; the observer will note the moment of passage at the first section, and then go to the lower section and note its passage at that section.

The discharge is then obtained by means of the formula $D=0.80 A V$; D =discharge; A =mean sectional area; V =the velocity as obtained by the observations on the floats.

It is possible to materially improve on this means of obtaining the discharge, especially as the coefficient 0.80 is arbitrary. Accordingly, permanent hydrographic stations were established at Alhajuela, Gamboa, and Bohio, with an observer and current-meter equipment at each.

ALHAJUELA.

The canal company's gauging station at Alhajuela is but a short distance below the proposed dam site. It has high, steep banks to each side and a width of about 250 feet, and at normal stage a mean depth of 4 or 5 feet. But it is on a slight bend, and the cross sections along the course differ considerably in area and conformation. Thus, at gauge height, 29 meters, we have 145, 135, and 133 square meters as the area at the upper end, middle, and lower end of the course. About 60 feet above the upper range a three-fourths inch cable is stretched across the river, with marks giving at normal stage the one-fourth, one-half, and three-fourths points of the width of the river. These marks are used to show the points at which to release the floats. The fluviograph is 200 yards below, and a secondary rod divided into centimeters is firmly set in the bank at the gauging station. The reading of this rod is used as the argument in the table giving the mean area.

On October 31, 1899, a No. 8 telegraph wire was stretched across the river about 60 feet above the canal company's cable. This was divided into 10-foot lengths, and was used to hold a boat in a fixed position while the velocity was being measured. This wire could be readily raised or lowered. While in use it was lowered so that a man in the bow could seize it and hold the boat in the desired position. While not in use it was raised 20 or 25 feet above the surface of the river—that is, beyond the reach of floods. A crutch, with a pulley, was fastened to the bow of the boat, over which the current meter was lowered. The crutch was fastened so that the pulley was at least 3 feet in front and 1 foot to one side of the cutwater. Thus the displacement of the boat could not affect the measured velocity.

The first current-meter measurement was made on October 31, 1899, with new small Price electric meter No. 35, which had been rated at Chevey Chase in July. Two simultaneous determinations of the discharge were made, the first by measuring the velocity at 0.6 depth below the surface at each station, and the second by measuring the velocity at each 1 foot of depth at the same stations and then taking the mean as the mean velocity at that station. The river was divided into twelve sections, varying from 10 feet wide near the banks, where the velocity across the stream changes rapidly, to 30 feet near the center, where it is more uniform.

Following are the results:

Fluviograph=28.565 meters.

First method.—Mean velocity=1.92 feet per second; discharge=2,280 second-feet.

Second method.—Mean velocity=1.90 feet per second; discharge=2,257 second-feet.

From the above we see that at Alhajuela the velocity at 0.6 depth below the surface gives practically the mean velocity in a section.

Following are specimen computations for discharge to show in detail the method used by the hydrographic office, Isthmian Canal Commission, and by the Compagnie Nouvelle du Canal de Panama:

Gauging made August 18, 1900, on Chagres River, at Athajuela, by the Isthmian Canal Commission.

[Time, 8.40 to 9.30 a. m. Current meter = large Price electric No. 94. Fluviograph, 28.51 meters.]

Distance from initial point.	Depth.	Depth of observations.	Time.	Revolutions.	Revolutions per second.	Velocity.	Section width.	Mean depth.	Area.	Discharge.
	<i>Feet.</i>		<i>Seconds.</i>				<i>Feet.</i>	<i>Feet.</i>	<i>Sq. feet.</i>	<i>Sec. feet.</i>
68 feet	R. B.					<i>a. 35</i>				
71 feet	2					.80	6	2	12	4
80 feet	4.1	2.5	100	23	.23	.80	11	4.1	45	36
90 feet	3.7	2.2	100	25	.25	.86	10	3.7	37	32
100 feet	4.7	2.8	100	32	.32	1.08	15	4.7	70	75
120 feet	4.2	2.5	100	28	.28	.96	20	4.2	84	81
140 feet	4.4	2.6	100	35	.35	1.18	20	4.4	88	104
160 feet	4.7	2.8	100	40	.40	1.35	20	4.7	94	127
180 feet	5	3	100	58	.58	1.97	20	5	100	197
200 feet	5.4	3.2	100	71	.71	2.38	20	5.4	108	257
220 feet	5.7	3.4	100	77	.77	2.56	20	5.7	114	292
240 feet	5.8	3.5	100	76	.76	2.53	20	5.8	116	293
260 feet	5.2	3.1	100	74	.74	2.47	20	5.2	104	257
280 feet	4.2	2.5	100	67	.67	2.26	15	4.2	63	142
290 feet	3.8	2.3	100	59	.59	2	10	3.8	38	76
300 feet	3.3	2	100	49	.49	1.66	10	3.3	33	55
310 feet	3	1.8	100	19	.19	.67	10	3	30	20
318 feet	2					<i>a. 30</i>	5	2	10	3
320 feet	L. B.									
Total							252		1,146	2,051

$2051 \div 1146 = 1.79$ feet per second = mean velocity.

a Estimated.

Gauging made August 18, 1900, on Chagres River, at Alhajuela, by canal company, by means of floats.

[Floats were 2-foot sticks, 1½ inches in diameter, weighted at one end so as to float vertically. Time, 8 a. m. Fluviograph, 28.50 meters. Length of course = 60 meters.]

No. of float.	Distance from right bank.	Time.			Computations.
		Upper range.	Lower range.	Elapsed.	
1	One-fourth width of river.....	<i>h. m. s.</i> 7 55 6	<i>h. m. s.</i> 7 57 15	<i>Seconds.</i> 129	Area = 101.45 m². 60 102.2 = .472 = velocity.
2	do.....	7 55 14	7 57 26	132	
3	One-half width of river.....	8 4 4	8 5 23	79	
4	do.....	8 4 11	8 5 32	81	
5	Three-fourths width of river.....	8 11 17	8 12 56	99	
6	do.....	8 11 25	8 12 58	93	
				$\frac{613}{6} = 102.2 = \text{mean time,}$	
then $101.45 \times .472 \times 0.80 = 47.884 \text{ m}^3 \text{ per second, or } 1,690 \text{ second-feet.}$					

The company's result was 1,690 cubic feet per second, being only 82 per cent that of this Commission, which was 2,051. Since the gaugings were made at almost the same time and gauge height, the above discrepancies are greater than they should be. The ratio was almost constant in all the simultaneous observations at Alhajuela, and is explainable as follows: The coefficient of 0.80 is too small, considering that floats are released at three sections. Owing to the curvature of the channel the currents are not parallel to the bank, and floats Nos. 1 and 2 always approach the right bank and at times cross the lower range within 10 feet of the bank. The second float at times passes the first float, although liberated about ten seconds later. Generally a distance between ranges of 60 meters is too long, as the error introduced by the more erratic drift of the float is not balanced by the greater accuracy in the time unless the channel is straight and the cross section exceptionally uniform.

Although it is useless to introduce any degree of refinement in the computation of float measurements unless the course is exceptionally good, still the above method of obtaining the mean velocity is erroneous, as it is obtained by dividing the length of the course by the mean time. The true mean velocity is the mean of the velocities of the filaments which constitute the stream. It may be mathematically shown that unless the observed time of all the floats is the same the result obtained by the above method is always too small and that the error increases as the differences in the observed times increase. The method as employed introduces an error varying from 3 to 5 per cent. The balance of the discrepancy is mainly accounted for by the coefficient 0.80 being too small.

On December 26 Mr. R. G. DuBoulay was put in charge of all the hydrographic observations at Alhajuela, which included measuring the Chagres at Alhajuela and at Santa Barbara, the Gatuncillo, and the Chilibre rivers.

In all, 74 measurements of the Chagres River at Alhajuela were made, varying in gauge height from 28.12 meters to 31.39 meters. The first high flood of the year, that of July 3 and 4, cut into the crest of the rapids just below the fluviograph, thus changing the relation between gauge height and the discharge, so that two rating tables are necessary.

RATING TABLE FOR STATION ON CHAGRES RIVER AT ALHAJUELA.

This table is applicable only from October 1, 1899, to July 3, 1900:

Gauge height.	Dis-charge.	Gauge height.	Dis-charge.	Gauge height.	Dis-charge.	Gauge height.	Dis-charge.
<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>
28	355	28.75	3,440	29.50	9,105	30.25	15,985
28.05	355	28.80	3,775	29.55	9,530	30.30	16,475
28.10	455	28.85	4,120	29.60	9,960	30.35	16,965
28.15	560	28.90	4,470	29.65	10,395	30.40	17,460
28.20	675	28.95	4,825	29.70	10,835	30.45	17,955
28.25	815	29	5,185	29.75	11,280	30.50	18,455
28.30	985	29.05	5,550	29.80	11,730	30.55	18,950
28.35	1,185	29.10	5,920	29.85	12,185	30.60	19,450
28.40	1,410	29.15	6,300	29.90	12,645	30.65	19,950
28.45	1,655	29.20	6,685	29.95	13,110	30.70	20,450
28.50	1,920	29.25	7,075	30	13,580	30.75	20,950
28.55	2,200	29.30	7,470	30.05	14,055	30.80	21,450
28.60	2,495	29.35	7,870	30.10	14,535	30.85	21,950
28.65	2,800	29.40	8,275	30.15	15,015		
28.70	3,115	29.45	8,685	30.20	15,500		

This table is applicable only from July 4, 1900, to date:

Gauge height.	Dis-charge.	Gauge height.	Dis-charge.	Gauge height.	Dis-charge.	Gauge height.	Dis-charge.
<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>
28	355	28.90	4,990	29.80	13,090	30.70	23,710
28.05	355	28.95	5,380	29.85	13,680	30.75	24,320
28.10	455	29	5,775	29.90	14,180	30.80	24,935
28.15	560	29.05	6,175	29.95	14,740	30.85	25,550
28.20	675	29.10	6,575	30	15,310	30.90	26,165
28.25	815	29.15	6,980	30.05	15,890	30.95	26,780
28.30	990	29.20	7,390	30.10	16,475	31	27,400
28.35	1,200	29.25	7,810	30.15	17,065	31.05	28,020
28.40	1,445	29.30	8,240	30.20	17,660	31.10	28,640
28.45	1,720	29.35	8,680	30.25	18,260	31.15	29,260
28.50	2,025	29.40	9,130	30.30	18,860	31.20	29,880
28.55	2,355	29.45	9,590	30.35	19,460	31.25	30,500
28.60	2,710	29.50	10,060	30.40	20,065	31.30	31,120
28.65	3,080	29.55	10,540	30.45	20,670	31.35	31,740
28.70	3,455	29.60	11,030	30.50	21,275	31.40	32,360
28.75	3,835	29.65	11,530	30.55	21,880	31.45	32,980
28.80	4,215	29.70	12,040	30.60	22,490	31.50	33,600
28.85	4,600	29.75	12,560	30.65	23,100		

The table of "Monthly discharge" was then computed, using these rating tables with the daily mean gauge height, as given by the fluvograph, as the argument; but on days of flood—that is, days in which the river rises over gauge height, 29—the daily mean discharge is obtained by taking the mean of the discharge corresponding to the bihourly height. This is necessary, as the increment increases rapidly with equal increases of gauge height. This table will be found on page 235.

TRIBUTARIES BETWEEN ALHAJUELA AND GAMBOA.

The mean monthly discharge of all the tributaries between Alhajuela and Gamboa was obtained by subtracting the discharge at the former station from the discharge at the latter, month by month.

Estimated monthly discharge of tributaries to the Chagres River between Alhajuela and Gamboa.

[Drainage area, 130 square miles.]

Month.	Mean discharge.	Total.	Run-off.		Rainfall. ^a
			Per square mile.	Depth.	
1900.	<i>Sec. feet.</i>	<i>Acre-feet.</i>	<i>Sec. feet.</i>	<i>Inches.</i>	<i>Inches.</i>
January.....	85	5,226	0.65	0.75	1.41
February.....	54	2,999	.42	.44	.10
March.....	33	2,029	.25	.29	.08
April.....	6	357	.05	.06	3.66
May.....	27	1,660	.21	.24	8.46
June.....	492	29,276	3.78	4.20	14.97
July.....	956	58,782	7.35	8.46	16.58
August.....	598	36,769	4.60	5.30	9.74
September.....	649	38,620	4.99	5.74	13.22
October.....	957	58,840	7.36	8.47	12.68
November.....	911	54,210	7.01	7.82	12.01
December.....	94	5,780	.72	.83	.97
The year.....	407	294,548		42.60	93.88

^a Mean of observations at Alhajuela and Gamboa.

Ratio of run-off to rainfall = 45 per cent.

The chief tributaries between Alhajuela and Gamboa are the Chilibre and the Gatuncillo. Weekly gaugings of these were made, and their gauge height was read three times per week. These observations in a general way furnished the relation of the discharge of each one to the total increase of discharge between the stations, which is as follows:

	Per cent.
Chilibre.....	36
Gatuncillo.....	26
Other sources.....	38
Total.....	100

The highest observed gauge height on the Chilibre was 22 feet, giving a computed discharge of 3,700 second-feet. On the Gatuncillo the highest observed gauge height was 16 feet, giving a computed discharge of 2,500 second-feet.

The following experiments with rod floats were made at Alhajuela to find the relation between the surface velocities and the mean velocity in a vertical section:

Float experiments made August 22, 1900 (fluvograph = 28.81).

I. FULL-DEPTH ROD FLOATS OF 2 TO 3 INCHES UNIFORM DIAMETER.

Distance from right bank.....feet..	20	40	60	80	100	120	140	160	180	200	220
Length of floats.....do.....	7.5	5	5	5	3.5	3.5	3.5	3.5	3.5	3.5	2
Elapsed time.....seconds..	87	78	78	77	75	57	55	55	59	64	93
Velocity.....feet per second..	2.27	2.53	2.53	2.56	2.63	3.46	3.58	3.58	3.34	3.08	2.12

Mean = 2.88 feet per second.

II. FLOATS OF 1½ FEET LENGTH, 2 INCHES UNIFORM DIAMETER.

Distance from right bank.....feet..	20	40	60	80	100	120	140	160	180	200	220
Elapsed time.....seconds..	72	75	70	68	63	53	50	45	51	58	71
Velocity.....feet per second..	2.74	2.63	2.81	2.90	3.13	3.72	3.94	4.38	3.86	3.40	2.77

Mean = 3.30 feet per second.

Ratio = $2.88 \div 3.30 = 0.87$.

The discharge obtained by using 2.88 as the mean velocity is 4,040 second-feet, which compares favorably with discharges obtained by means of current meters.

CHAGRES RIVER AT SANTA BARBARA.

The gauge height of the Chagres River at Santa Barbara was read by C. Clauzel at 6 a. m. and 6 p. m. and at times of important fluctuations. The discharge measurements were made with current meters and surface floats. After comparing several float measurements with simultaneous current-meter measurements a coefficient of 0.85 was adopted to reduce the observed velocity of the floats to the mean velocity of the cross section. Thirty-five current-meter and 110 float measurements were made, from which the following rating table was computed:

Rating table, Chagres River at Santa Barbara.

Gauge height.	Dis-charge.	Gauge height.	Dis-charge.	Gauge height.	Dis-charge.	Gauge height.	Dis-charge.
<i>Feet.</i>	<i>Sec. feet.</i>	<i>Feet.</i>	<i>Sec. feet.</i>	<i>Feet.</i>	<i>Sec. feet.</i>	<i>Feet.</i>	<i>Sec. feet.</i>
1.0	-----	3.4	1,790	5.8	5,520	8.2	10,500
1.1	-----	3.5	1,915	5.9	5,715	8.3	10,720
1.2	-----	3.6	2,040	6.0	5,910	8.4	10,945
1.3	-----	3.7	2,170	6.1	6,110	8.5	11,170
1.4	-----	3.8	2,300	6.2	6,310	8.6	11,400
1.5	-----	3.9	2,430	6.3	6,510	8.7	11,640
1.6	-----	4.0	2,565	6.4	6,710	8.8	11,890
1.7	300	4.1	2,700	6.5	6,915	8.9	12,155
1.8	320	4.2	2,840	6.6	7,120	9.0	12,445
1.9	350	4.3	2,980	6.7	7,325	9.1	12,745
2.0	390	4.4	3,125	6.8	7,530	9.2	13,055
2.1	440	4.5	3,270	6.9	7,735	9.3	13,375
2.2	500	4.6	3,420	7.0	7,940	9.4	13,700
2.3	570	4.7	3,575	7.1	8,150	9.5	14,025
2.4	650	4.8	3,730	7.2	8,360	9.6	14,350
2.5	750	4.9	3,890	7.3	8,570	9.7	14,675
2.6	855	5.0	4,055	7.4	8,780	9.8	15,000
2.7	965	5.1	4,225	7.5	8,990	9.9	15,325
2.8	1,075	5.2	4,400	7.6	9,200	10.0	15,650
2.9	1,190	5.3	4,580	7.7	9,415	11.0	18,900
3.0	1,305	5.4	4,765	7.8	9,630	12.0	22,150
3.1	1,425	5.5	4,950	7.9	9,845	13.0	25,400
3.2	1,545	5.6	5,140	8.0	10,060		
3.3	1,665	5.7	5,330	8.1	10,280		

REMARKS.—The zero of the gauge rod is at elevation 170.8 feet.

Estimated monthly discharge of Chagres River at Santa Barbara.

[Drainage area, 300 square miles.]

Month.	Discharge.			Total.	Run-off.		Rainfall in basin.
	Maximum.	Minimum.	Mean.		Square mile.	Depth.	
1900.	<i>Sec. feet.</i>	<i>Sec. feet.</i>	<i>Sec. feet.</i>	<i>Acre-feet.</i>	<i>Sec. feet.</i>	<i>Inches.</i>	<i>Inches.</i>
January.....	8,360	700	1,205	74,090	4.02	4.63	1.34
February.....	1,020	390	542	30,100	1.81	1.88	.21
March.....	415	320	369	22,690	1.23	1.41	.10
April.....	8,570	300	430	25,590	1.43	1.60	5.52
May.....	16,625	310	1,003	61,670	3.34	3.88	13.15
June.....	6,510	610	1,338	79,620	4.46	4.98	21.51
July.....	19,875	802	1,900	116,830	6.33	7.31	13.53
August.....	18,250	1,020	2,348	144,370	7.83	9.02	11.22
September.....	13,055	965	1,717	102,170	5.72	6.38	16.08
October.....	33,600	1,365	3,601	221,420	12.00	13.84	17.00

GAMBOA.

The canal company's gauging station at Gamboa is about 100 yards above the mouth of the Obispo River, or about 300 yards below the proposed dam site. Although by no means an ideal station, on account of the tributary just below it, which may at times produce backwater, and also on account of the broken water produced by a whirlpool above it, still it is the best section within several miles to each side of it. The banks are fairly high, and the course straight. Floods rising over gauge height, 18 meters, will cover the left bank, and some water will flow through the digging for the canal, and thus will not be included in the discharge measured at the gauging station. It then becomes necessary to make the measurement at Gorgona, where, although the discharge of the Obispo is added, better results are obtained. The fluvograph, which is similar to the one at Alhajuela, is one-half mile above the gauging station. Discharge measurements are made with floats in the same way as at Alhajuela.

As the chief object of the work was to check the observations of the canal company, it was necessary to locate the station near theirs, although for current meter measurements this location is poor. Near the left bank the river is swift and shallow, and toward the right it is sluggish and deep, thus a large per cent of the discharge has a low velocity, which during low stages falls below 0.30 feet per second, and it is in these low velocities that the current meter works at the greatest disadvantage. The unnatural condition of the river is due to the spoil composed of large pieces of rock, which were deposited along the left bank during construction times.

On this account this station did not receive the same attention as Alhajuela and Bohio, and no permanent observer was stationed here, but the writer, with the assistance of R. McD. Geraty, made discharge measurements whenever time permitted. Thirty-three measurements were made in the same general way as at Alhajuela, ranging in gauge height from 13.85 to 16 meters, from which the rating table and estimated monthly discharge table were computed. (Table of estimated monthly discharge. See page 224.)

Rating table for Chagres River at Gamboa.

This table is applicable from October 1, 1899, to date.

Gauge height.	Discharge.	Gauge height.	Discharge.	Gauge height.	Discharge.	Gauge height.	Discharge.
<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>
13.5	-----	14.9	3,650	16.3	12,550	17.7	23,150
13.6	-----	15.0	4,115	16.4	13,290	17.8	23,930
13.7	250	15.1	4,605	16.5	14,030	17.9	24,710
13.8	395	15.2	5,135	16.6	14,775	18.0	25,500
13.9	560	15.3	5,690	16.7	15,525	18.1	26,300
14.0	745	15.4	6,280	16.8	16,275	18.2	27,100
14.1	955	15.5	6,905	16.9	17,025	18.3	27,900
14.2	1,205	15.6	7,550	17.0	17,780	18.4	28,700
14.3	1,480	15.7	8,215	17.1	18,540	18.5	29,500
14.4	1,775	15.8	8,900	17.2	19,300	18.6	30,300
14.5	2,090	15.9	9,605	17.3	20,060	18.7	31,100
14.6	2,425	16.0	10,330	17.4	20,820	18.8	31,900
14.7	2,795	16.1	11,070	17.5	21,590	18.9	32,700
14.8	3,205	16.2	11,810	17.6	22,370	19.0	33,500

BUENA VISTA AND BOHIO.

The canal company's gauging station is at Buena Vista, or about one-half mile above the proposed dam site. It is on a long, straight stretch of the river, of 220 feet width, with high, steep banks. The cross sections are very similar, and the velocities across the stream are very uniform.

It is almost an ideal station for discharge measurements, its sole defect being that at times of extreme low water the tides of the Atlantic may slightly affect the velocity.

The fluviograph is at Bohio and is the counterpart of the one at Alhajuela, but the tides are noticeable whenever the height of the river is less than 0.5 meter, so that below this stage a rating table may give slightly erroneous results, as the same gauge height at different times may not correspond to the same discharge. A gauge rod divided into centimeters is firmly set in the bank at the gauging station. The reading of this is used in obtaining the mean area of cross section.

In making discharge measurements the canal company has no means whatever to show the points at which to release the floats, this matter being left to the judgment of the boatmen who release the floats at points which they estimate as the one-quarter, one-half, and three-quarters of the width; still, as previously stated, the velocities across the streams vary slightly, so that the results are but slightly affected by any errors of judgment of the boatmen.

On December 27, 1899, Mr. Inocencio Galindo, jr., was put in charge of the hydrographic work at Bohio, which included weekly measurements of the chief tributaries between Gamboa and Bohio, such as the Caño Quebrada, Gigante, Frijolitos, Frijoles, and the Agua Salud. Occasional trips were also made to the Trinidad and Gatun rivers, at which times measurements were made.

Mr. R. McD. Geraty made frequent trips to this station to assist in and to check the work.

Eighty-six measurements for discharge were made, ranging in gauge height from 0.28 to 5.43 meters.

The summary of discharge will be found on page 225.

Rating table for Chagres River at Bohio.

This table is applicable only from December, 1899, to date.

Gauge height.	Discharge.	Gauge height.	Discharge.	Gauge height.	Discharge.	Gauge height.	Discharge.
<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>	<i>Meters.</i>	<i>Sec. feet.</i>
0.1	330	1.6	3,610	3.1	9,305	4.6	17,150
.2	490	1.7	3,930	3.2	9,760	4.7	17,730
.3	650	1.8	4,260	3.3	10,225	4.8	18,310
.4	820	1.9	4,600	3.4	10,700	4.9	18,890
.5	1,015	2.0	4,950	3.5	11,185	5.0	19,470
.6	1,230	2.1	5,310	3.6	11,680	5.1	20,060
.7	1,465	2.2	5,680	3.7	12,185	5.2	20,650
.8	1,715	2.3	6,060	3.8	12,700	5.3	21,240
.9	1,975	2.4	6,440	3.9	13,225	5.4	21,830
1.0	2,235	2.5	6,830	4.0	13,765	5.5	22,420
1.1	2,495	2.6	7,220	4.1	14,315	5.6	23,020
1.2	2,755	2.7	7,620	4.2	14,870	5.7	23,620
1.3	3,020	2.8	8,025	4.3	15,430	5.8	24,220
1.4	3,305	2.9	8,440	4.4	16,000	5.9	24,820
1.5		3.0	8,865	4.5	16,570	6.0	25,420

The following current meter observations were made on July 31 at Buena Vista to test the assumption that the velocity at 0.6 depth below the surface is the mean of the velocities in a vertical section:

Distance from right bank.					
50 feet.		110 feet.		160 feet.	
Depth of observation.	Velocity.	Depth of observation.	Velocity.	Depth of observation.	Velocity.
<i>Feet.</i>	<i>Ft. per sec.</i>	<i>Feet.</i>	<i>Ft. per sec.</i>	<i>Feet.</i>	<i>Ft. per sec.</i>
1.0	3.75	1.0	3.85	0.5	3.69
2.0	3.75	2.0	3.72	1.5	3.63
3.0	3.72	3.0	3.66	2.5	3.63
4.0	3.63	4.0	3.59	3.5	3.53
5.0	3.56	5.0	3.53	4.5	3.50
6.0	3.52	6.0	3.43	5.5	3.43
7.0	3.43	7.0	3.40	6.5	3.24
8.0	3.21	8.0	3.24	7.5	3.08
9.0	2.91	9.0	3.05	8.5	2.61
9.7	2.35	9.5	2.34		
Mean	3.44		3.44		3.37
0.6 depth	3.52		3.43		3.45

The width of the river at the time of the observations was 214 feet, the gauge height 2.26 meters, and the total depth at the stations in order from the right bank was 9.9, 9.9, and 9.1 feet.

As there is less than one-half foot below the last measurement, one-half weight is given to these in obtaining the mean, except in the last case, where there is also but one-half foot above the first measurement. These observations were made from a boat. The meter was lowered to the specified depth, and the revolutions for two consecutive fifty seconds were noted; if these did not agree, the revolutions for fifty additional seconds were noted, and the mean of these was then used as an argument in the rating table to obtain the velocity.

TRIBUTARIES BETWEEN BOHIO AND GAMBOA.

The largest tributary between Bohio and Gamboa is the Caño Quebrada; minor tributaries in order of their size are the Obispo, Gigante, Frijoles and Frijolitos, and Agua Salud. By subtracting the mean monthly discharge of the Chagres at Gamboa from the discharge at Bohio the discharge of all the tributaries was obtained.

Estimated monthly discharge of tributaries to the Chagres River between Gamboa and Bohio.

[Drainage area, 245 square miles.]

Months.	Mean discharge.	Total.	Run-off.		Rainfall. ^a
			Per square mile.	Depth.	
1900.	<i>Sec. feet.</i>	<i>Acre-feet.</i>	<i>Sec. feet.</i>	<i>Inches.</i>	<i>Inches.</i>
January.....	177	10,890	0.72	0.83	3.57
February.....	82	4,550	.33	.34	.26
March.....	40	2,460	.17	.20	.46
April.....	15	900	.06	.07	3.08
May.....	55	3,380	.22	.25	7.08
June.....	759	45,160	3.10	3.46	11.01
July.....	1,856	114,120	7.57	8.73	16.99
August.....	1,188	73,050	4.85	5.59	11.33
September.....	1,440	85,690	5.88	6.56	11.96
October.....	2,374	145,970	9.69	11.17	15.27
November.....	2,125	126,440	8.68	9.68	17.82
December.....	720	44,270	2.94	3.39	2.40
The year.....	908	656,880		50.27	101.23

^a Mean of observations at Bohio, Gorgona, and Gamboa.

Ratio of run-off to rainfall=50 per cent.

The tributaries named were gauged on an average of once per week, and the following table depends upon these data:

Rivers.	Maximum measured discharge.	Minimum measured discharge.	Mean.	Per cent of total discharge. ^a
	<i>Sec. feet.</i>	<i>Sec. feet.</i>	<i>Sec. feet.</i>	
Caño Quebrada.....	1,640	17	335	37
Obispo.....	1,177	1	90	10
Gigante.....	281	0	65	7
Frijoles and Frijolitos.....	219	5	55	6
Agua Salud.....	90	3	33	4
Other sources.....			330	36
Total.....			908	100

^a Per cent of the total inflow between Gamboa and Bohio contributed by each stream.

DRAINAGE AREA OF THE CHAGRES RIVER.

The drainage area of Chagres River above Bohio is only approximately known and depends upon the following data:

Totten's map of the Isthmus of Panama gives that portion of the divide lying between latitude 9° 0' and 9° 20' N., longitude 80° 30' and 80° 50' W. It also gives the divide of the Rio Boqueron, i. e., the northwestern extremity of the drainage area. The survey of the Mandinga River by this Commission gives the eastern divide at longitude 79° 15' W.

The survey of the Chagres River above Santa Barbara gives the northeastern extremity of the drainage basin at latitude 9° 25' N., longitude 79° 15' W. It also gives an idea of the size and general direction of the tributaries.

The northern and southern portions of the divide, not included in Totten's map, have been approximately determined from the Atlantic and Pacific oceans, and in these parts the greatest probable error exists.

SEDIMENT OBSERVATIONS.

Observations to determine the amount of sediment carried by the Chagres River were made at Alhajuela and Bohio, by means of water samples representing the mean condition of the river.

Months.	Alhajuela.		Bohio.	
	Mud.	Solid matter.	Mud.	Solid matter.
1900.				
June.....	<i>Cu. yds.</i> 75,900	<i>Cu. yds.</i> 15,180	<i>Cu. yds.</i> 152,720	<i>Cu. yds.</i> 30,540
July.....			763,800	152,760
August.....	406,100	81,230	580,000	116,000
September.....	203,300	41,660	396,800	79,360
October.....	485,040	97,000		

Five cubic yards of mud are assumed to equal 1 cubic yard of solid matter.

TRINIDAD AND GATUN RIVERS.

Below Bohio the major part of the Chagres flows through the almost completed cut for the canal, its old channel being in many places choked by vegetation. In this stretch it receives the waters of two large tributaries, the Trinidad and the Gatun. Frequent trips to these were made and current meter measurements were made each time. No gauge rods were set on these streams, because if they were set less than 10 miles up, the fluctuations of the Chagres would materially influence the rod reading, the slope of the rivers being very slight, and the time taken in these trips would be longer than the time taken in making a gauging. The inflow between the mouth and station would also be omitted.

Discharge of the Gatun and Trinidad rivers.

GATUN.

Date.	Dis-charge.	Date.	Dis-charge.	Date.	Dis-charge.	Date.	Dis-charge.
1899.		1900.		1900.		1900.	
November 17.....	1,213	February 22.....	160	April 23.....	117	July 5.....	945
December 9.....	688	February 26.....	186	April 26.....	237	July 9.....	1,559
December 29.....	442	March 1.....	109	May 3.....	71	July 12.....	916
		March 6.....	197	May 14.....	65	July 16.....	962
1900.		March 12.....	142	May 18.....	225	July 19.....	729
January 11.....	359	March 15.....	111	May 21.....	240	July 25.....	923
January 19.....	218	March 19.....	164	May 30.....	217	August 2.....	922
January 25.....	259	March 23.....	133	June 7.....	315	August 24.....	1,021
February 1.....	209	March 26.....	178	June 11.....	348	September 5.....	420
February 6.....	216	April 5.....	130	June 14.....	285	September 11.....	1,111
February 8.....	198	April 12.....	88	June 18.....	214	September 18.....	888
February 12.....	160	April 17.....	133	June 21.....	225		
February 15.....	151	April 20.....	98	June 28.....	616		

Maximum.....	Second-feet.
Minimum.....	1,559
Mean.....	65
	528

TRINIDAD.

Date.	Dis-charge.	Date.	Dis-charge.	Date.	Dis-charge.	Date.	Dis-charge.
1899.		1900.		1900.		1900.	
November 17.....	1,687	February 15.....	291	May 10.....	463	July 16.....	1,594
December 9.....	2,371	February 19.....	299	May 14.....	244	July 19.....	2,255
December 29.....	896	February 22.....	264	May 18.....	610	July 25.....	2,295
		February 26.....	358	May 21.....	620	August 2.....	2,144
1900.		March 1.....	365	May 24.....	533	August 7.....	1,375
January 8.....	1,210	March 6.....	368	May 30.....	1,094	August 24.....	1,464
January 11.....	744	March 15.....	143	June 7.....	2,373	August 30.....	2,521
January 15.....	705	March 19.....	242	June 11.....	1,379	September 5.....	1,403
January 19.....	574	March 23.....	221	June 14.....	993	September 8.....	1,283
January 22.....	599	March 26.....	304	June 18.....	931	September 11.....	2,120
January 25.....	569	April 12.....	204	June 21.....	1,358	September 18.....	2,607
January 29.....	555	April 17.....	226	June 28.....	1,537	September 21.....	1,474
February 1.....	631	April 20.....	220	July 2.....	1,337	September 27.....	2,256
February 6.....	439	April 23.....	225	July 5.....	1,577		
February 8.....	492	May 1.....	261	July 9.....	1,574		
February 12.....	450	May 3.....	286	July 12.....	1,868		

Maximum.....	Second-feet.
Minimum.....	2,607
Mean.....	143
	1,110

The "mean" is the mean of the monthly means.

HYDRAULICS OF THE CHAGRES RIVER AT ALHAJUELA AND BOHIO.

At Alhajuela and Bohio the fluvigraph rods and gauge rods were read each time a gauging was made; the difference in these simultaneous readings in connection with the distance between the rods, the measured discharge, and several cross sections of the river furnished the slope, the mean hydraulic radius, and the mean velocity. From these data the value of the coefficient for roughness "N" in "Kutter's formula" was computed.

At Alhajuela the distance between the rods is 200 meters; the bed of the river is small gravel and the banks are steep and free from weeds and bushes. At Bohio the distance between the rods is 1,335 meters; the bed is clay, with small gravel and free from detritus; the banks are high but are covered with weeds and bushes.

ALHAJUELA.

Gauge height.	Mean area.	Mean discharge.	Mean velocity.	Hydraulic radius.	Slope.	"N."
28.15	815	580	0.71	3.84	0.000025	0.027
28.28	910	953	1.05	4.16	.000050	.026
28.51	1,075	1,928	1.80	4.74	.000095	.023
28.68	1,200	2,926	2.44	5.17	.000144	.022
28.84	1,320	4,040	3.01	5.58	.000211	.023
29.46	1,785	9,361	5.20	7.23	.000350	.020
31.01	3,000	25,872	8.62	11.50	.000892	.026

BOHIO.

1.59	1,340	2,560	1.91	5.90	0.000097	0.025
2.50	2,000	6,100	3.05	8.58	.000142	.024
3.19	2,500	8,930	3.58	10.42	.000165	.026
4.42	3,400	14,600	4.29	13.10	.000210	.029
5.50	4,040	21,020	5.20	14.70	.000277	.029

RIO GRANDE.

From Culebra to the Pacific Ocean the line of the canal follows the valley of the Rio Grande; therefore an idea of the floods and the general characteristics of this stream becomes valuable. Accordingly, on October 25, 1899, two gauge rods were set at Pedro Miguel, the first being 250 yards above and the second 30 yards below the mouth of the Pedro Miguel River. The lower rod could not be set farther downstream on account of the high tides of the Pacific Ocean, the tidal effect at Miraflores being at least 5 feet.

These rods were read at 6 a. m. and 6 p. m., and additional readings were taken during floods.

Estimated monthly discharge of Grande River at 250 yards above the mouth of the Pedro Miguel.

[Drainage area, 10 square miles.]

Month.	Discharge.			Total.	Run-off.		Rainfall.
	Maximum.	Minimum.	Mean.		Per square mile.	Depth.	
	<i>Sec. ft.</i>	<i>Sec. ft.</i>	<i>Sec. ft.</i>	<i>Acre-ft.</i>	<i>Sec. ft.</i>	<i>Inches.</i>	<i>Inches.</i>
1899.							
November	410	10.0	35.0	2,082	3.50	3.90	8.67
December.....	65	3.0	13.0	800	1.30	1.50	2.34
1900.							
January.....	37	1.0	3.0	184	0.30	0.35	0.56
February.....	1	0.5	0.7	39	0.07	0.07	0.09
March.....	2	0.0	0.2	12	0.02	0.02	0.00
April.....	1	0.0	0.3	18	0.03	0.03	2.73
May.....	210	0.6	11.0	676	1.10	1.27	10.66
June.....	650	3.0	44.0	2,618	4.40	4.91	10.30
July.....	550	5.0	114.0	7,010	11.40	13.14	25.04
August.....	283	2.0	50.0	3,074	5.00	5.76	6.52
September.....	660	11.0	38.0	2,261	3.80	4.24	6.75
October.....	370	30.0	60.0	3,690	6.00	6.92	8.68
The year.....	660	0.0	31.0	22,464		42.11	82.34

Ratio of run-off to rainfall=0.51 per cent.

Estimated monthly discharge of Grande River at 30 yards below the mouth of the Pedro Miguel.

[Drainage area, 19 square miles.]

Month.	Discharge.			Total.	Run-off.		Rainfall.
	Maximum.	Minimum.	Mean.		Per square mile.	Depth.	
1899.	<i>Sec. ft.</i>	<i>Sec. ft.</i>	<i>Sec. ft.</i>	<i>Acre-ft.</i>	<i>Sec. ft.</i>	<i>Inches.</i>	<i>Inches.</i>
November	790	27.0	81.0	4,820	4.26	4.75	8.67
December.....	100	12.0	26.0	1,600	1.37	1.58	2.34
1900.							
January.....	50	3.0	7.0	430	0.37	0.43	0.56
February.....	3	0.0	1.0	56	0.05	0.05	0.09
March.....	3	0.0	0.2	12	0.01	0.01	0.00
April.....	2	0.0	0.5	31	0.03	0.03	2.73
May.....	310	1.0	24.0	1,476	1.26	1.45	10.66
June.....	1,550	10.0	105.0	6,250	5.53	6.17	10.30
July.....	1,850	21.0	295.0	18,140	15.53	17.90	25.04
August.....	1,550	12.0	142.0	8,730	7.47	8.61	6.52
September.....	1,550	18.0	80.0	4,760	4.21	4.70	6.75
October.....	800	34.0	112.0	6,885	5.90	6.80	8.68
The year.....	1,550	0.0	74.0	53,190		52.48	82.34

Ratio of run-off to rainfall=0.637 per cent.

PRECAUTIONS FOR ACCURACY.

The following precautions were taken so that any constant errors in the results might be detected: The current meters were rerated. Simultaneous gaugings with two meters were frequently made, which would show whether rerating was necessary. The meter at one station was at times exchanged for the meter at another. The gauging section was occasionally moved several hundred yards up or down stream, thus any local conditions giving erroneous results, such as diagonal currents, etc., could be detected. To eliminate errors of personal equation the observers were changed.

Rating the meters.—At Obispo there is a pool formed in the excavation for the “Obispo lock.” It is 130 feet long, 100 feet wide, and over 12 feet deep. It has no current except after a heavy rainfall. A railroad trestle spans it across its longest dimension, and, as a push car is readily obtainable, we have excellent conditions for rating meters.

An outrigger was fastened to the car, from which the meter was freely suspended so as to be at least 3 feet below the surface of the water. The car was then pushed at a uniform velocity over a measured course of 80 feet and the time and revolutions were noted. Thus the relation between the revolutions per second and velocity per second was obtained. Forty or fifty trips were made at velocities varying from 0.30 feet per second to 9 feet per second, each trip being at a uniform velocity; thus any change in the relation between the revolutions per second and the velocity per second was obtained.

The following table shows the care taken to reduce the error due to the rating table as far as practicable to a minimum; it also shows the effect of constant and ordinary use:

Summary of rating tables for current meters.

Meter No.	93.			94.		47.	
	Dec. 16, 1899.	Mar. 8, 1900.	July 7, 1900.	Mar. 20, 1900.	Oct. 11, 1900.	Dec. 16, 1899.	Sept. 6, 1900.
Revolutions per second.	Velocity (feet per second).						
0.50	1.72	1.72	1.66	1.69	1.71	1.25	1.21
1.00	3.42	3.36	3.37	3.25	3.35	2.38	2.40
1.50	5.02	4.94	4.88	4.75	4.90	3.53	3.60
2.00	6.62	6.52	6.33	6.25	6.45	4.68	4.80
2.50	8.22	8.12	7.78	7.77	8.00	5.83	6.02
3.00	9.82	9.72	9.23	9.32	9.55	6.98	7.32
3.50	11.42	11.32	10.68	10.87	11.10	8.13	8.62
4.00	13.02	12.92	12.13	12.42	12.65	9.28	9.92

Meters Nos. 93 and 94 are “large Price electric meters,” and No. 47 is a “small Price electric meter.”

RAINFALL.

Rainfall observations are made by the Panama Canal Company at Bohio, Gamboa, Alhajuela, and Panama, and the Panama Railroad Company keeps a record at Colon. These records give a good idea of the amount and distribution of the rainfall along the canal line, but it was also deemed desirable to obtain the mean rainfall on the basin of the Chagres River and its relation to one of the old records. With this object in view rain gauges were set in the Upper Chagres and Pequeni basins, so distributed with reference to altitude and distance up the river as to give practically the mean rainfall; but, owing to the numerous and almost impassable rapids on the upper Chagres, it was impracticable to set these very far up. Following is a description of the location of the rain gauges referred to Dos Bocas, which is the junction of the Chagres and Pequeni rivers.

SANTA BARBARA, RIO FEA, AND RIO PUENTE.

The observations at these stations were made by C. Clauzel. Santa Barbara is on the Chagres River, 3 miles ESE. of Dos Bocas, at an elevation of 225 feet above sea level. The observations were made at 6 a. m. and 6 p. m.

Rio Fea gauge is on a hill between the Chagres and the Rio Fea; it is 4.5 miles NE. of Dos Bocas, at an elevation of 850 feet. It was observed four times per week.

Rio Puente gauge is on a hill in the Puente River basin, 4 miles SE. of Dos Bocas, at an elevation of 610 feet. Observations were made each day at 10 a. m.

SALAMANCA AND LAS MINAS.

The rainfall at these stations was observed by Simon Bolivar. Salamanca is on the Pequeni 5.5 miles N. of Dos Bocas at an elevation of about 200 feet. Observations were made at 6 a. m. and 6 p. m.

Las Minas is 12 miles NNE. of Dos Bocas at an elevation of about 250 feet. Observations were made three times per week.

The results of these observations are appended herewith.

Very respectfully,

W. W. SCHLECHT, *Hydrographer.*

Mr. A. P. DAVIS,

Hydrographer, Isthmian Canal Commission, Washington, D. C.

Rainfall for 1899-1900.

GORGONA.

Day.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.
1	0.00	0.00	0.00	0.00	0.00	0.25	0.56	0.80	0.00	0.05	1.15
2	.00	.30	.00	.03	.00	.00	.18	.30	.10	.03	.12
3	.29	.10	.00	.00	.00	.00	.00	.00	.61	.00	.46
4	1.39	.00	.00	.00	.30	.00	.48	.35	.09	.00	.40
5	.00	.00	.00	.45	.00	.00	1.40	.00	.07	.00	.38
6	.05	.10	.00	.00	.00	.06	1.70	.00	.00	.60	.50
7	.00	.00	.00	.00	.00	.15	.30	.15	.64	.56	.00
8	.04	.00	.00	.00	.00	.10	.00	.10	.33	.00	.00
9	.00	.00	.00	.00	.00	1.20	.00	.78	.26	.10	.00
10	.00	.05	.00	.00	.00	.00	.46	.55	.00	2.78	.18
11	1.58	.35	.00	.02	.00	.85	.00	.72	.80	.00	.00
12	.35	.00	.05	.00	.00	.00	.30	2.40	.30	.00	1.36
13	.00	.00	.00	.00	.00	.00	.20	.05	.88	.15	.05
14	.10	.25	.00	.00	.00	.00	.00	.10	1.30	.30	1.40
15	.00	.00	.00	.00	.00	.35	.90	3.45	.00	.70	.00
16	.03	.00	.00	.00	1.00	.00	.22	.00	.25	2.45	.50
17	.00	.05	.00	.10	.00	.00	.50	2.87	.00	.10	.10
18	.00	.00	.03	.00	.00	.00	.55	.35	.00	.90	.00
19	.00	.00	.00	.00	.00	.00	.10	.00	1.43	.18	.24
20	.00	.00	.05	.00	.00	.00	2.25	.64	.20	.00	.05
21	.00	.00	.00	.00	.00	.00	.06	.12	.60	.00	.68
22	.00	.00	.00	.00	.00	.00	.00	.30	.25	.70	.18
23	.00	.00	.00	.00	.00	.00	.30	.60	.00	.17	.14
24	.00	.00	.00	.00	.00	.02	.00	.20	.25	.05	.70
25	.25	1.28	.00	.00	.00	.22	1.35	.00	.15	.00	.77
26	.24	.00	.00	.00	1.10	.18	.00	.25	.00	.13	.32
27	.00	.00	.00	.00	.00	.20	.35	.20	.23	.95	1.57
28	.00	.10	.00	.00	.00	.00	.12	.00	2.10	.35	.28
29	.26	.00	.00	.00	.00	.74	.30	.74	.03	.00	.00
30	.00	.05	.00	.00	.30	.00	.10	1.70	.05	.00	.40
31	.00	.00	.00	.05	.00	.40	.00	2.90	.12	.00	.00
Total	4.58	2.63	.13	.20	3.15	7.07	12.48	19.72	11.04	11.25	11.93

THE PANAMA CANAL.

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Rainfall for 1899-1900—Continued.

SANTA BARBARA, ON CHAGRES RIVER.

Day.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.
1		0.00	0.00	0.00	0.09	0.41	2.91	0.00	0.03	0.62	0.52
2		.03	.02	.00	.00	1.19	.25	.00	.78	.00	.67
3		.06	.00	.00	.22	.00	1.05	.91	.05	.00	.86
4		.02	.00	.02	.00	.00	1.32	.16	.78	.00	.19
5		.06	.00	.00	.01	.15	1.65	.00	.09	.00	.05
6	1.50	.57	.01	.00	.00	.07	.72	.15	.08	.14	.00
7	.45	.02	.03	.01	.00	.14	.06	.30	.12	.02	.06
8	.20	.01	.00	.00	.00	.23	.04	.00	.04	1.59	.03
9		.05	.00	.00	.00	2.36	.25	.66	.00	2.79	.04
10		.01	.01	.04	.00	.15	.02	.08	.00	.02	.12
11		.01	.00	.00	.00	1.00	.68	.60	1.21	.24	.15
12		.01	.00	.00	.00	.03	.02	.04	1.75	.00	1.49
13		.05	.00	.00	.00	.04	.57	.00	.92	.72	1.82
14		.01	.00	.00	.00	.65	.38	2.86	.02	.46	.30
15	.04	.00	.00	.00	.00	.00	.10	.25	.00	.68	.05
16	.00	.00	.00	.00	.00	.02	.65	.80	.00	.10	.17
17		.00	.00	.02	.00	.00	.85	.60	.00	.06	.02
18		.00	.01	.00	.00	.11	.10	.02	.00	.01	.76
19	.02	.00	.06	.00	.00	.29	.39	.90	.06	.77	.20
20	.00	.00	.02	.00	.00	.00	.37	.64	1.25	.24	2.07
21	.04	.00	.00	.00	.00	.00	.00	.18	.06	2.33	.15
22	.00	.03	.00	.00	.00	.38	.37	.78	.26	.41	.12
23	.01	.01	.00	.00	.00	.65	.08	.19	.95	.48	.01
24	.02	.11	.00	.00	.00	.93	2.10	.16	2.38	.03	.71
25	.03	.22	.00	.00	.18	.27	2.32	.22	.22	.18	.31
26	.16	.08	.00	.00	3.48	.00	.31	.58	.07	3.01	.18
27	.06	.00	.00	.00	.01	.86	.63	.03	.00	.72	.75
28	.02	.02	.00	.00	.00	.45	2.08	.85	.92	.02	.00
29	.00	.00		.00	.33	.19	.10	.72	.57	.02	.00
30	.01	.08		.00	1.11	1.03	2.17	.01	.02	.00	
31	.01	.00		.00		2.07		.96	.02		
Total.....	2.62	1.41	.16	.09	5.43	13.67	22.54	13.65	12.65	15.66	11.80

RIO PUENTE.

Day.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.
1		0.01	0.00	0.00	0.06	0.64	0.80	2.06	0.04	0.95	0.74
2		.05	.02	.00	.00	.00	.08	.00	3.28	.10	.40
3		.05	.00	.00	.09	.26	.24	.10	.70	.00	1.10
4		.12	.00	.00	.00	.00	.86	.67	1.20	.00	1.00
5		.00	.00	.02	.02	.00	1.72	.00	.30	.03	.37
6		.03	.01	.00	.00	.02	3.82	.17	.14	.46	.05
7	1.03	.30	.03	.00	.00	.02	.10	.33	.80	.05	.08
8		.01	.00	.00	.00	.37	.04	.00	1.12	.10	.03
9		.03	.00	.00	.00	.56	.02	.83	.02	1.44	.05
10		.00	.01	.05	.00	1.74	.18	.05	.00	1.52	.14
11		.02	.00	.00	.00	.91	.17	.63	.03	.02	.17
12		.00	.00	.00	.00	.42	.27	.36	.90	.05	.05
13		.00	.00	.00	.00	.04	.02	.00	3.21	.03	2.10
14	.50	.00	.00	.00	.00	.13	1.13	.02	1.61	1.64	1.90
15		.00	.00	.00	.00	.42	.03	3.20	.00	.60	.30
16	.05	.00	.00	.00	.00		.07	.10	.00	.95	.25
17		.00	.00	.01	.00	.02	.70	.94	.00	.60	.15
18		.00	.00	.00	.00	.00	1.03	.99	.00	1.40	.05
19		.00	.05	.00	.00	.20	.04	1.05	1.15	.90	.75
20	.00	.00	.00	.00	.00	.90	.93	.43	.00	.22	.50
21	.00	.00	.00	.00	.00	.00	.02	1.30	1.56	.25	2.05
22	.00	.02	.00	.00	.00	.00	.18	.99	.06	1.42	.12
23	.00	.02	.00	.00	.00	.00	.08	.60	.23	.38	.08
24	.00	.00	.00	.00	.00	.65	.04	.10	.55	.52	.00
25	.00	.13	.00	.00	.00	.10	3.16	.11	1.35	.04	.40
26	.08	.15	.00	.00	1.05	.38	.46	.75	.10	.40	.60
27	.10	.02	.00	.00	.00	.00	.70	.00	.15	1.32	.70
28	.02	.00	.00	.00	.00	.03	.74	.05	.72	.11	1.02
29	.03	.00		.00	.37	.60	.98	1.08	.03	.00	.00
30	.00	.00		.00	.02	.87	.74	1.20	.76	.00	
31	.00	.06		.00		.83		.03	.08		
Total.....	1.81	1.02	.12	.08	1.61	10.11	19.35	18.14	20.09	15.50	15.15

THE PANAMA CANAL.

Rainfall for 1899-1900—Continued.

SALAMANCA, ON PEQUENI RIVER.

Day.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.
1	0.00	0.10	0.00	0.02	0.14	0.35	0.96	1.80	0.00	0.55	0.34	1.10	0.15
2	1.10	.10	.21	.01	.01	.04	.47	.00	.69	.00	1.50	1.31	1.18
3	.00	.11	.01	.00	.26	.00	.06	.72	.50	1.35	.42	.04	.33
4	1.50	.02	.00	.05	.02	.00	.42	.37	1.21	.00	.23	.47	.19
5	.09	.30	.00	.00	.00	.97	1.43	.08	.06	.42	.00	.56	.00
6	.02	1.09	.10	.03	.00	.22	.07	.39	.10	.51	1.01	2.09	.00
7	.20	.04	.02	.02	.00	.09	.12	.10	.00	.00	.24	1.43	.00
8	.01	.01	.00	.00	.00	.67	.00	.01	.00	1.85	.01	.08	.00
9	.02	.05	.00	.05	.00	2.59	.45	1.27	.07	3.32	.15	.27	.00
10	.00	.04	.00	.04	.00	.06	.67	.67	.10	.00	.03	.00	.00
11	.08	.00	.00	.00	.00	.36	.00	.35	.00	1.22	.77	.02	.27
12	.02	.00	.00	.01	.00	.37	.00	.01	.02	.00	.11	1.57	.00
13	.00	.00	.00	.00	.00	.14	.20	.04	.43	.73	1.56	.43	.00
14	.20	.00	.00	.00	.00	.06	.00	1.16	.01	1.84	.40	.00	.02
15	.03	.07	.00	.00	.00	.04	.02	.17	.00	.86	.38	.00	.18
16	.00	.01	.00	.00	.00	.33	.00	.00	.00	.40	.62	.03	.00
17	.10	.02	.00	.04	.12	.00	.89	.00	.00	.00	.07	1.28	.22
18	.02	.00	.00	.06	.00	.97	.00	.03	.00	.00	1.56	.00	.00
19	.00	.00	.14	.05	.00	.60	.00	.21	.30	.73	1.19	.00	.00
20	.05	.00	.02	.00	.00	.04	.00	1.14	1.85	.47	2.27	.00	.07
21	.07	.05	.00	.00	.00	.00	.45	.27	.08	.35	.02	1.53	.27
22	.00	.07	.00	.00	.00	.00	.06	.09	.19	.13	.30	.41	.00
23	.00	.09	.00	.00	.00	.00	.00	.05	.14	.80	.00	.06	.00
24	.02	.01	.00	.00	.00	.37	.00	.10	1.71	.31	.02	.00	.00
25	.05	.60	.00	.00	.87	.38	.00	.01	1.01	1.84	1.00	.00	.00
26	.23	.28	.00	.00	1.10	.00	.09	.18	.00	.13	.00	.20	.03
27	.49	.03	.00	.00	.04	.06	.14	1.09	.00	.38		.10	.01
28	.05	.03	.00	.00	.00	.70	.67	.07	1.24	.00	2.00	.88	.00
29	.20	.00		.00	.27	.61	.87	.07	.20	.02		.77	.00
30	.04	.08		.00	.39	.47	.00	.00	.06	.60		.14	.00
31	.00	.00		.00		1.99		.01	.00				.00
Total	4.59	3.20	.50	.38	3.22	12.15	8.37	10.46	9.97	18.81	16.20	14.77	2.92

PEDRO MIGUEL.

Day.	Dec.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.
1	0.00	0.00	0.00	0.00	0.00	0.25	0.22	0.00	0.00		1.23
2	.00	.15	.00	.00	.00	.00	.10	.00	.00		.45
3	.17	.00	.00	.00	.00	.00	.17	.35	.43	0.79	.53
4	.08	.00	.00	.00	.00	.00	.30	.60	.44	.00	.78
5	.00	.00	.00	.00	.04	.00	1.21	.00	.10	.02	.45
6	.55	.10	.00	.00	.00	.65	1.30	.05	.10	.51	.02
7	.00	.00	.00	.00	.00	1.52	1.45	.03	.40	.02	.21
8	.51	.00	.00	.00	.00	.49	.00	.20	.00	.00	.01
9	.80	.00	.00	.00	.00	1.22	.00	.78	.00	1.12	.02
10	.00	.00	.00	.00	.00	.45	.20	.61	.00	.01	.00
11	.00	.00	.00	.00	.00	.00	.00	.62	.00	.00	.02
12	.00	.00	.09	.00	.00	.00	.07	1.10	.20	.00	.45
13	.12	.00	.00	.00	.00	.00	.00	.02	1.50	.05	.39
14	.06	.00	.00	.00	.00	.00	.16	.00	.00	.02	.87
15	.00	.00	.00	.00	.00	.73	.00	3.00	.15	.15	.27
16	.05	.00	.00	.00	.12	.00	.00	.00	.00	.67	.78
17	.00	.00	.00	.00	.00	.00	.04	1.31	.00	.19	.02
18	.00	.00	.00	.00	.00	.22	1.57	1.10	.10	.09	.05
19	.00	.00	.00	.00	.00	.25	.05	1.00	.45	.04	.03
20	.00	.00	.00	.00	.00	.95	1.69	1.20	.00	.22	.03
21	.00	.00	.00	.00	.89	.00	.15	1.35	.00	.80	.51
22	.00	.00	.00	.00	.65	.06	.00	1.10	.00	.16	.01
23	.00	.00	.00	.00	.00	.49	.14	2.50	.00	.08	.42
24	.00	.00	.00	.00	.00	.00	.04	.25	.00	.71	.00
25	.00	.10	.00	.00	.10	.27	.23	.57	.00	.08	.15
26	.00	.12	.00	.00	.48	.28	.02	2.10	.50	.20	.09
27	.00	.00	.00	.00	.00	.21	.82	.70	.50	.25	.35
28	.00	.00	.00	.00	.20	.00	.00	.00	1.50	.25	.01
29	.00	.00		.00	.00	.97	.26	1.40	.15	.30	.00
30	.00	.09		.00	.25	.64	.11	2.00	.00	.02	.28
31	.00	.00		.00		1.01		.10	.00		.25
Total	2.34	.56	.09	.00	2.73	10.66	10.30	25.04	6.52	6.75	8.68

Rainfall for 1900.

ISTHMUS OF PANAMA.

Stations.	Jan.	Feb.	Mar.	Apr.	May.	June.	July.	Aug.	Sept.	Oct.	Nov.	Dec.	Total.
Colon	6.06	0.33	1.06	0.75	12.25	11.65	16.81	17.04	9.37	16.33	20.28	4.13	116.06
Bohio	7.06	.48	1.04	2.89	7.42	18.40	17.79	14.02	15.40	18.43	24.98	4.02	131.93
Gorgona	2.63	.13	.20	3.15	7.07	12.48	19.72	11.04	11.25	11.93			78.60
Gamboa	1.01	.16	.13	3.21	6.76	12.15	13.45	8.92	9.24	12.11	10.67	.79	82.34
Pedro Miguel	.56	.09	.00	2.73	10.66	10.30	25.04	6.52	6.75	8.68	a 8.67	a 2.34	109.19
Alhajuela	1.82	.04	.03	4.10	10.16	17.79	19.73	10.56	17.20	13.26	13.36	1.14	113.18
Campañas	1.54	.05	.11	2.66	8.57	22.12	12.78	12.49	13.55	15.34		a 2.06	
Santa Barbara	1.41	.16	.09	5.43	13.67	22.54	13.65	12.65	15.66	11.80	b 13.50	a 2.62	116.22
Rio Fea	1.26	.26	.11	5.60	12.64	20.48	13.41	9.79	16.51	21.98		a 1.81	
Rio Puente	1.02	.12	.08	1.61	10.11	19.35	18.14	20.09	15.50	15.15		a 1.81	
Salamanca	3.20	.50	.38	3.22	12.15	8.37	10.46	9.97	18.81	16.20	14.77	2.92	100.95
Las Minas	4.90	1.17	1.21	4.39	12.53	10.91	12.43	15.01	9.90	19.91	16.63	7.23	116.22

a For 1899.

b Estimated.

APPENDIX E.

WASTE-WEIR DIMENSIONS AND DISCHARGE FOR LAKE BOHIO.

The determination of the dimensions of the wastew weir for Lake Bohio depends upon the simultaneous discharge of the Chagres River into the lake and the outflow from the latter through the weir during floods. It becomes necessary, therefore, to establish formulæ, or curves, exhibiting certain results arising from the conditions which would attend the concurrent flood flow into and out of Lake Bohio. These formulæ, or curves, necessarily do not represent the actual conditions with mathematical exactness, but they approximate the natural conditions of the lake closely enough to determine with confidence the dimensions of the wastew weir and the variation of the head of water on it during a given period of variation of flood in the Chagres River.

The discharge of the waste way will not be through a thin-edge weir, but over a rounded masonry lip or crest, for which the coefficient is not precisely known. A careful consideration of the results of experiments with rounded crests, including those of Bazin, appears to indicate that the value of 3.5 for the weir coefficient is, on the whole, justifiable for the present case, and it will be used in the following investigations. It may be a little too large for small heads or a little too small for the greater heads used.

In deducing the following formulæ it will be assumed that during the period of flood considered the discharge of the river per unit of time (per second) increases at a uniform rate until the maximum stage is reached; then that it remains at that stage for a definite time, and subsequently decreases at a uniform rate, generally not the same as the rate of rise. As a matter of fact, the uniform rate of increase is the only one of the two of practical importance in this investigation, and it is to be determined from data supplied by observation of actual floods.

Obviously, water may begin to flow over the crest of the wasteway at any stage of the river at its point of entrance into the lake, depending on the proximity of prior floods and the draft recently made on the waters of the lake. The stage of the river, therefore, taken to coincide in time with the beginning of flow over the rounded crest of the wastew weir, must be a matter of judgment in view of known data.

The notation to be used is the following:

- t_1 =time in seconds measured from beginning of flow over weir.
- T_1 =value of t_1 at instant the river reaches its greatest elevation.
- v_1 =discharge of the river in cubic feet per second at any time t_1 .
- V_1 =discharge of the river in cubic feet per second at the time T_1 .
- r =rate (uniform) of variation of discharge of river per unit of time.
- b =length of weir in feet.
- h =head of water on weir in feet; and H =greatest value of h .
- a =uniform rate of variation of h per unit of time, so that $h=at_1$ and $H=aT_1$; $T>T_1$.
- V =greatest discharge over weir in cubic feet per second; V may or may not be equal to V_1 .
- e =discharge of river per unit (second) of time when lake is at elevation of crest of weir.
- Q =general value of discharge over weir in cubic feet per second. V is the greatest value of Q .
- c =weir coefficient, taken equal to 3.5.
- D =total discharge of weir in cubic feet in time T (seconds).
- A =area in square feet of surface of the lake.
- L =total storage in cubic feet in reservoir or lake above the elevation of the crest when head on the weir is H feet.

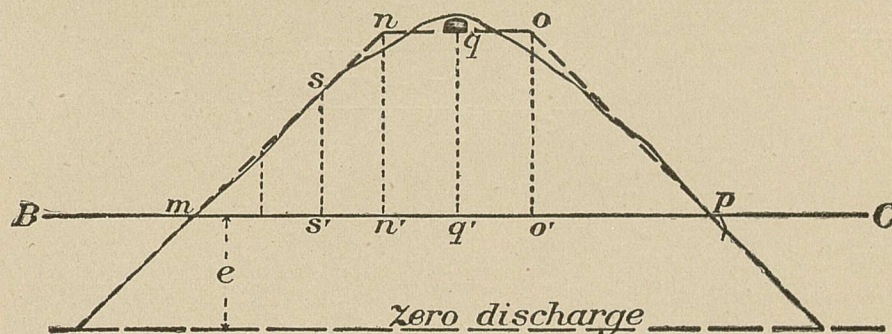


FIG. 1.

The full line of figure 1 represents the fluvograph record of the rise of the Chagres River during a flood at Bohio. Inasmuch as this discussion must cover all the water flowing into Lake

When h is small the second number of equation (7) is positive, and negative when h is large. This shows that the curve, of which h is the ordinate and t_1 the abscissa, is convex toward t_1 in the vicinity of 0 and concave in the upper part of the curve, as indicated in figure 2.

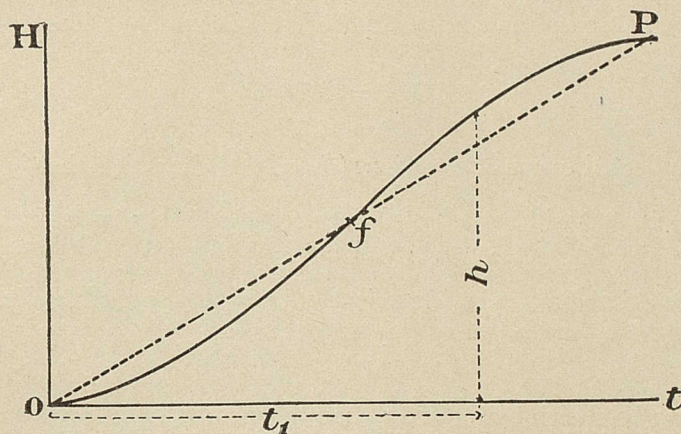


FIG. 2.

If the flood remains at its maximum stage a sufficient length of time the curve will become horizontal at some point P and remain so, or the curve may become horizontal after the maximum stage has passed, but in the latter case it will not remain horizontal.

After the flood reaches its maximum stage the unit discharge $V_1 = e + rT_1$ may remain constant for a definite length of time. In that case the rise of the lake surface, i. e., the increase of h , will follow a different law from that shown by equation (5). By writing V_1 for $(e + rT_1)$ in equation (6):

$$\frac{dh}{dt_1} = \frac{V_1 - 3.5 bh^{3/2}}{A} \dots \dots \dots (8)$$

or

$$dt_1 = A \frac{dh}{V_1 - 3.5 bh^{3/2}} \dots \dots \dots (9)$$

This equation may readily be integrated.

If $k = \left(\frac{V_1}{-3.5b} \right)^{2/3}$ the integral will take the form:

$$\frac{t_1}{A} = \frac{1}{3(3.5b)k} \left[\frac{1}{2} \log \frac{k^2 - kh^{1/2} + h}{(k + h^{1/2})^2} + \sqrt{3} \tan^{-1} \left(\frac{2h^{1/2} - k}{k\sqrt{3}} \right) \right] + C \dots \dots \dots (10)$$

By giving known values to t_1 and h the value of the constant of integration, C , can be determined for any given case.

Equation (6) will enable that portion of the curved line to be constructed showing the relation between h and t_1 during the rising stage of the river, since $\frac{dh}{dt_1}$ is the tangent of the inclination of the curve to the coordinate t_1 . Similarly, equation (8) by the same method of tangents, will enable that portion of the curved line to be constructed belonging to the highest stage of the river when that stage holds constant for a suitable length of time.

These investigations are subject to one source of criticism when applied to so large a reservoir as Lake Bohio, because it is implicitly assumed that the surface of the lake remains horizontal at all times, whatever may be the rate of inflow or outflow. In this case the point of inflow may be fifteen or more miles from the discharging weir, giving a sensible slope to the lake surface and causing the head on the weir to increase for a considerable time after the river ceases to rise, or possibly after the river begins to fall.

In the application of the preceding analysis to the computations for the Bohio weir the curves OMKS and O'M'K'S' in figure 3 have been constructed, OM and O'M' belonging to the rising stage of the river, and MK and M'K' to the constant maximum stage. S and S' are the

If it is desired that the head on the weir shall not exceed H , then from equation (12) the length of the weir must be at least:

$$b = \frac{V}{c H^{3/2}} \quad \dots \quad (19)$$

The most simple expression for the time T is given by equation (16):

$$T = \frac{5}{2} \frac{D}{H^{5/2}} \frac{V^{2/3}}{(cb)^{1/3}} \quad \dots \quad (20)$$

In equation (20), however, the discharge D is not generally known, and it will be necessary to use equation (18) in order to determine T .

In the following computations it will be assumed that the unit discharge v_1 of the river is 17,657 cubic feet (500 cubic meters) per second at the beginning of discharge over the weir, i. e., $v_1 = e$.

In making the application of the preceding formulæ to the problem presented at Lake Bohio it will be necessary to consider first a head of water of 5 feet on a waste weir 2,000 feet long, in connection with a flood discharge in the Chagres River increasing uniformly from 17,657 cubic feet (500 cubic meters) per second to 112,000 cubic feet (3,170 cubic meters) per second during a period of 26.43 hours, or 95,145 seconds, and remaining at that maximum stage for 14.25 hours, or 51,300 seconds, the sum of the two periods being 146,445 seconds. The rate of discharge after the maximum period has passed is not needed. The surface area of Lake Bohio at the elevation of the crest of the wastew weir is about 38.5 square miles, or 1,073,318,400 square feet. At an elevation of 5 feet higher that surface area becomes about 43 square miles, or 1,198,771,200 square feet. The product of half the sum of these two areas by 5 gives the volume of water impounded in the lake with a 5-foot head on the weir as

$$L = 5,680,224,000 \text{ cubic feet.}$$

The other data required are:

$$v_1 = 17,657 \text{ cubic feet per second} = e.$$

$$V_1 = 112,000 \text{ cubic feet per second.}$$

$$T_1 = 95,145 \text{ seconds.}$$

$$c = 3.5 \text{ and } b = 2,000 \text{ feet.}$$

Hence by equation (12):

$$Q = V = 3.5 \times 2,000 \times (5)^{3/2} = 78,262 \text{ cubic feet.}$$

Introducing these various quantities in equation (18) there will result:

$$T = 126,009 \text{ seconds.}$$

$$= 35 \text{ hours.}$$

As this period is 5.68 hours less than 40.68 hours, the total time of rising and remaining at the maximum height, it is probable that under the conditions of the flood assumed the head of water on the weir would rise a little above 5 feet, but a little only.

The total amount discharged over the weir while the head increases to 5 feet is by equation (16):

$$D = 3,944,711,745 \text{ cubic feet.}$$

If a flood discharge of 140,000 cubic feet be assumed in the river it is interesting to determine approximately how long it would have to continue in order to raise the head on the weir to 7 feet. By equation (12) the discharge over the weir (2,000 feet long) with a head of 7 feet is:

$$Q = V = 129,642 \text{ cubic feet per second.}$$

$$\text{As before, } v_1 = 17,657 \text{ cubic feet per second.}$$

$$\text{Also, } V_1 = 140,000 \text{ cubic feet per second.}$$

$$\text{Again, assume } T_1 = 95,145 \text{ seconds.}$$

At an elevation of 7 feet above the crest the area of the lake surface is 1,249,394,752 square feet, while at the elevation of the crest the area, as before, is 1,073,318,400 square feet. The half sum of these areas multiplied by 7 gives the volume stored in the lake above the crest as:

$$L = 8,129,496,032 \text{ cubic feet.}$$

These quantities inserted in equation (18) will give:

$$T = 158,307 \text{ seconds.}$$

$$= 43.98 \text{ hours.}$$

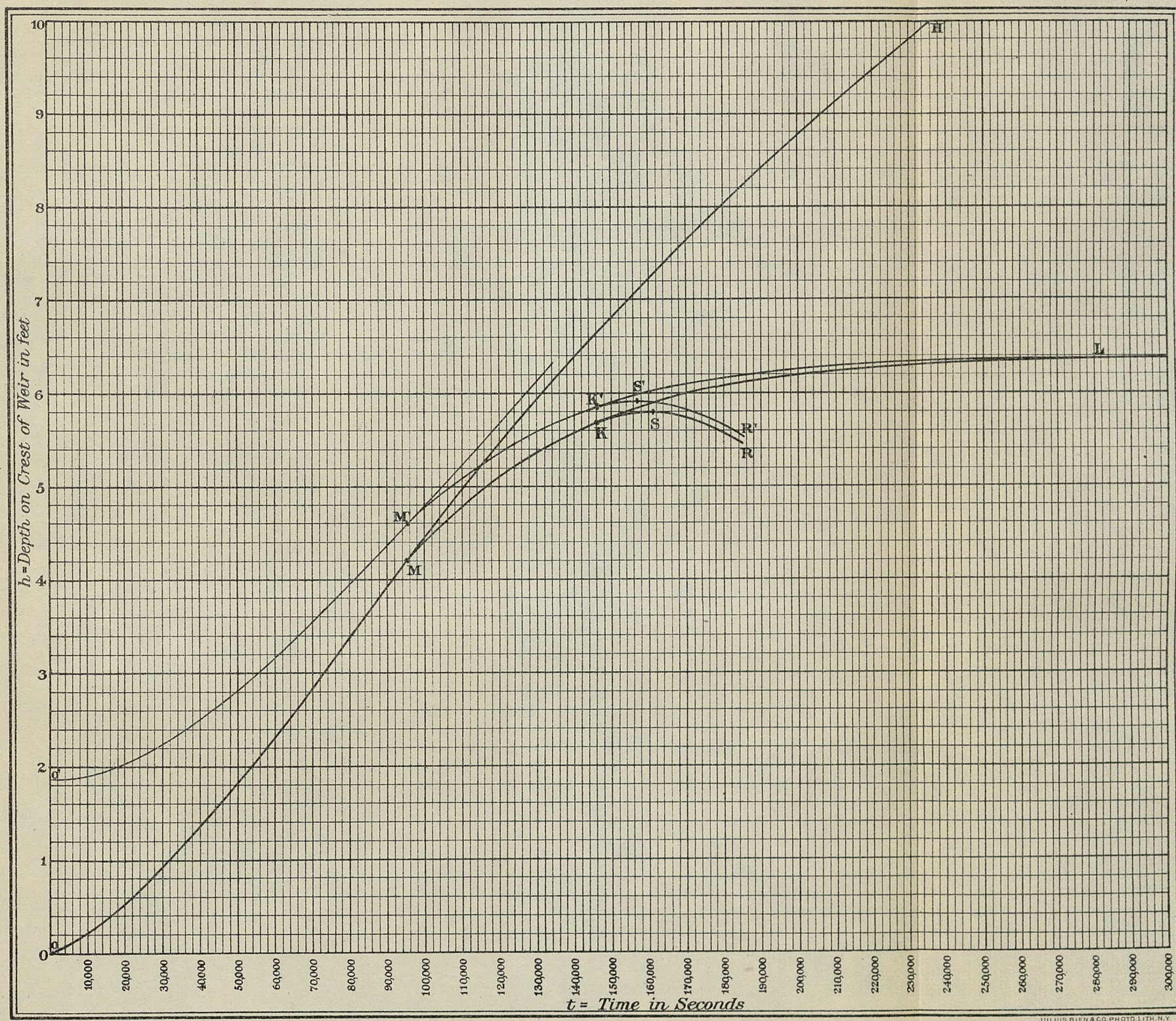
As the period of increasing discharge has been taken at 26.43 hours (95,145 seconds), it is thus seen that the maximum discharge of 140,000 cubic feet per second would have to be maintained for probably eighteen hours or more in order to produce the head of 7 feet on the weir crest. This is a contingency so excessively remote as to be practically impossible. The total discharge over the weir while the head rises from 0 to 7 feet is, by equation (16):

$$D = 8,007,498,655 \text{ cubic feet.}$$

The general conditions assumed in the preceding applications of the formulæ established are, in the main, essentially the same as those taken by M. Choron in his fifth hypothesis, on page 54 of his Notes Techniques.

The curve OMKS of figure 3 shows the relation between the head on the weir and the time from the beginning of flow over the weir for the first of the preceding set of computations. It indicates that the greatest head on the weir would be 5.8 feet. This is a little too high, on account of the neglect of the actual slope of the surface of the lake, as has already been observed. The approximate computations show that the greatest head would be a little over 5 feet. It is probable that 5.5 feet is about right. The curve and computations practically confirm each other.

The O'M'K'S' exhibits the relation between head on the weir and time, and the assumption that there is a discharge over the weir of 17,657 cubic feet per second, when there is an equal discharge of the river, followed by the same flood as before. The greatest head on the weir is but one-tenth of a foot greater than in the previous case, showing that the difference in assumptions has no material effect on the maximum head, either as to amount or time. The volume of water discharged during the period of increasing head would be materially greater in the latter case than in the former.



APPENDIX G.

TIME REQUIRED FOR TRANSIT THROUGH AN ISTHMIAN CANAL.

The time required for the passage of a ship through the proposed isthmian canal will depend upon the speed attainable and permissible in the various sections and the delays occasioned by lockages and by meeting other ships. Attainable speed here means the speed limited by the power of the ship. From this deductions are to be made in some cases to give the permissible speed.

This discussion will embrace four type ships, as follows:

Attainable and permissible speed.

Type ships.

	Length.	Beam.	Draft.	I. H. P.
	<i>Feet.</i>	<i>Feet.</i>	<i>Ft. in.</i>	
Type A.....	400	50	24 6	1,900
Type B.....	400	50	32 0	2,300
Type C.....	540	60	32 0	3,300
Type D.....	650	70	32 0	4,500

The draft given refers to fresh water. The power is sufficient to give the ship a speed of $12\frac{1}{2}$ miles per hour in open sea. The miles referred to in this discussion are statute miles.

The area of the midship section of each ship is assumed to be 96 per cent of the inclosing rectangle. The block coefficient is taken as 0.8. This expresses the ratio of displacement to the product of length of keel, beam, and draft. The mean cross section is therefore $0.8 \times \text{beam} \times \text{draft}$.

Coefficients.

No experiments on the comparative speeds attainable by large seagoing ships in open sea and in restricted waterways, carried on in a scientific and comprehensive manner, are known to have been made. The movement of small boats in small canals at low speeds has been the subject of extended experiment and careful study. The most notable of these have been made recently in France and Germany. They have been summarized and discussed by Mr. Elnathan Sweet in a paper accompanying the report of the engineer and surveyor of New York on the proposed barge canal from Lake Erie to the Hudson River. These experiments were carried out in great number and detail, but on boats of such different model, of so much smaller dimensions, and at such moderate speeds that they can not be applied directly to the problem of speeds of large seagoing ships through an isthmian canal. However, the analysis of the problem by Herr R. Haack, who conducted the experiments in Germany, is suggestive and useful in the present discussion.

Available data.

Experiments in France and Germany summarized by Mr. E. Sweet.

While a ship moves forward its length in a canal, a volume of water equal to its displacement must move back past the ship.

Back current.

Let A = wet cross section of canal, square feet.

a = mean cross section of ship, square feet.

$$r = \frac{A}{a}$$

V = speed of ship past a fixed land point, in miles per hour.

v = velocity of backward current past the moving ship in miles per hour; approximately it may be thus expressed:

$$v = \frac{a}{A-a} V = \frac{V}{r-1}.$$

Approximate formula for backward current.

This is the expression ordinarily used. Herr Haack points out the important fact that it is not the correct measure of v . The passage of a boat through a canal is

accompanied by a depression of the water surface which begins at or a little ahead of the bow and continues past the boat some distance astern. The water which moves past the boat while the latter passes a fixed point is not only the boat's displacement, but also the volume measured by this depression into the length of the boat and width of the canal. Taking this movement also into consideration, Haack proceeds to deduce an expression for the relation between V , v , and the resistance of the boat.

Let $\triangle A$ = reduction of wet cross section of canal caused by sinking of water surface during passage of boat.

R = resistance of boat.

L = length of boat in feet.

w = weight of 1 cubic foot of water.

C = coefficient to be determined by experiment.

Then $A - \triangle A - a$ = area of cross section of water remaining between the boat and the bottom and sides of the canal.

$(\triangle A + a)V$ = volume of water displaced in a unit of time.

$(A - \triangle A - a)v$ = volume flowing in the section $A - \triangle A - a$ past a given point in a unit of time.

These volumes are identical, hence

$$(A - \triangle A - a)v = (\triangle A + a)V$$

and

Correct formula
for backward
current.

$$v = \frac{\triangle A + a}{A - \triangle A - a} V \quad . \quad . \quad . \quad . \quad . \quad (1)$$

Which is the correct value of v .

The weight of water kept in motion by the movement of the boat is

$$w (A - \triangle A - a) L$$

and Haack gives the following expression for the work required for moving it during a unit of time:

$$C w (A - \triangle A - a) L v \quad . \quad . \quad . \quad . \quad . \quad (2)$$

and places this equal to the work of the resistance of the boat during a unit of time; that is,

Tractive re-
sistance.

$$R V = C w (A - \triangle A - a) L v \quad . \quad . \quad . \quad . \quad . \quad (3)$$

The determination of the value of the coefficient requires the measurement of the sinking of the water surface and the tractive resistance R . Mr. Sweet does not give the value of C as determined by Haack, or state whether it is constant or variable except indirectly by showing the curves of resistance for the boats used in the experiments. For the reasons already given these curves are not applicable to the present case. There is needed a series of experiments with large ships in restricted waterways approximating closely the ships and channels of the future isthmian canal.

In the absence of such experiments the inquiry resolves itself first into the estimation of the speed attainable in a restricted channel by a ship whose speed in open sea is known. This requires a consideration of the various causes of retardation encountered by a ship when it passes from broad, deep water into such a channel.

Backward cur-
rent past a ship
in a canal.

When a ship moves upstream in a broad, deep river its speed through the water is essentially the same as in open sea. Its speed past a fixed point will be its open-sea speed less the velocity of the current. In this respect the backward current in a canal is analogous to the river current and is to be deducted from the open-sea speed of the ship.

Observations
at St. Clair Flats
Canal.

The determination of the velocity of the backward current requires, as Haack points out, and as indicated in formula (1), that the depression of the water surface between the ship and the sides of the channel be known. Haack observed this carefully in his experiments, but no record has been found of such observations during the passage of a large ship through a ship canal. The writer therefore proceeded to make a limited series at the St. Clair Flats Canal. This series included 29 ships. One of these was rejected because its dimensions are not given in the Marine Register. Six were rejected because they were without cargoes, and the observations on them little pertinent. The observations consisted of: First, measurements at intervals of 5 seconds from a fixed horizontal plane, to the water surface before, during, and after the passage of a ship, covering the period when the surface of the water varied from the normal; second, a record of the time taken by the ship in passing; third, sound-

ings across the canal opposite the observation point to determine its cross section; fourth, measurement by rod floats of the normal current in the canal.

The St. Clair Flats Canal is a channel without locks, 296 feet wide and about 7,200 feet long. The sides are vertical with timber revetments. The depth, which was dredged to 16 feet for about 20 feet from the sides and to 20 or 21 feet in the remainder of the channel, has increased opposite the point of observation to 26 feet at the center. The point of observation was 2,280 feet from the upper end of the canal.

The observed time of passing, with the length of ship taken from the Marine Register, gave the speed of the ship, which was corrected for the normal current in the canal. For the calculation of the ratio r_1 , which is the quotient resulting from dividing the area of the cross section of the canal by the midship section of the ship, the coefficient of the ship's midship cross section is assumed to be 0.96, the same as for the type ships. This is probably too large for the smaller ships, but substantially correct for the larger ones.

The measurements of the water surface were made about 1 foot out from the vertical side of the canal. They may not represent exactly the mean water surface between the ship and sides of the canal, but it is believed the difference is not great. While a ship was approaching, the water surface at the point of observation remained nearly or quite normal until the bow was within 25 to 100 feet of the cross section of the canal passing through the observing station. The water then began to fall, attained its maximum depression a little aft the center of the ship, then began to rise and again attained the normal 50 or 100 feet behind the ship.

These distances varied with different ships and are to be taken only in a general way.

For the calculation of the average depression of water surface opposite the passing ship, the distances between observations were plotted as abscissas and the depression of the water surface as ordinates, the total distance plotted being the keel length of the ship. This gave a longitudinal section during the time of the ship's passing. The area of this section divided by the keel length gave the average depression.

The results of the observations are given in the following table:

TABLE I.—Results of observations at St. Clair Flats Canal during passage of ships.

Name of ship.	Keel.	Beam.	Draft.	r_1 .	Depression of water surface.		V.	v.
					Maxi- mum.	Mean.		
	<i>Feet.</i>	<i>Feet.</i>	<i>Feet.</i>		<i>Foot.</i>	<i>Foot.</i>	<i>Miles.</i>	<i>Miles.</i>
Manistique.....	157	31	12.00	19.15	0.21	0.15	5.78	0.31
V. Swain.....	187	33	13.50	15.99	.25	.20	6.66	.43
Pueblo.....	225	36	16.50	11.99	.58	.50	8.24	.82
New York.....	268	36	16.50	11.99	.97	.80	10.07	1.17
W. H. Sawyer.....	201	37	13.50	14.26	.21	.17	6.71	.48
Choctaw.....	266	38	17.75	10.56	.67	.46	7.28	.79
Vega.....	301	38	18.00	10.40	.42	.31	7.20	.74
Spokane.....	311	38	17.50	10.71	.83	.58	8.66	1.00
Frontenac.....	270	40	18.00	9.89	.83	.66	7.42	.94
Philip Minch.....	275	40	17.50	10.17	.42	.30	6.81	.71
Seneca.....	290	40	16.00	11.13	.75	.57	8.41	.90
Masaba.....	292	40	17.50	10.17	1.00	.83	8.13	1.12
Roman.....	296	40	17.75	10.03	.75	.53	7.06	.84
Iroquois.....	242	41	17.00	10.22	.63	.44	7.95	.89
Livingstone.....	281	41	17.50	9.93	.92	.72	8.23	1.07
T. W. Palmer.....	281	41	19.00	9.14	.58	.44	7.77	.96
Hudson.....	288	41	15.50	11.21	.54	.39	8.62	.86
Niagara.....	266	42	17.00	9.98	.42	.31	7.40	.79
America.....	274	42	16.50	10.28	.42	.27	7.38	.75
Manola.....	282	42	18.50	9.17	.46	.32	6.34	.74
S. R. Kirby.....	294	42	18.00	9.42	.67	.60	6.35	.82
Presque Isle.....	406	50	19.00	7.50	.92	.68	7.04	1.15
Mean.....				11.06	.62	.46	7.52	

In observations on ships of such different types as those in the preceding list, moving at such different rates of speed, great apparent eccentricity in result should be expected. Much of this, however, will disappear if the ships are classified with respect to r_1 and averages taken as in the next table.

TABLE II.

Value of r_1 .	Number of ships averaged.	Mean value of r_1 .	Mean depression of water surface.	Mean speed of ship (V) per hour.
7.5	1	7.5	Foot. 0.68	Miles. 7.04
9-10	6	9.59	.51	7.25
10-11	8	10.32	.46	7.56
11-12	4	11.58	.56	8.83
14-20	3	16.47	.17	6.72

These results are plotted in figure 1, attached hereto, with r_1 as abscissas and depressions (d) as ordinates. A line to represent the relations is shown, which passes reasonably near all the points. The point $r_1=11.58$, $d=0.56$, should lie considerably below the line, because it corresponds to the largest value of V; and the point $r_1=16.47$, $d=0.17$, should lie above the line, because it corresponds to the smallest value of V.

The same figure shows the depressions which have been used for this discussion for the isthmian canal. They exceed those given by the observations as far as the latter extend, and should do so, because the calculated speeds in the discussion for the large channels, where r_1 is large, exceed those observed in the St. Clair Flats Canal. For values of r_1 less than 7.50 the assumed depressions have for basis only the writer's judgment in interpreting the experiments. For $r_1=1$ it is obvious the depression would equal, theoretically, the normal depth of water in the canal. Between the point and the point corresponding to $r_1=7.50$ there may be material difference of opinion as to the location of the depression curve.

Increased head resistance. The backward current is only one of the new conditions set up when a ship passes from open sea into a restricted channel. The depression of the water surface must increase the head resistance to the ship's motion. In the observations at the St. Clair Flats Canal the measurements of depression were made, as before stated, at the side of the channel where the depression occurred in advance of the ship, which passed at a distance of 100 to 175 feet from the observing station. As far as could be judged by watching the movement of the ship, without actual measurement, this depression does not occur immediately in front of the ship. It appears probable that the excess in height of water against the bow over the height against the stern may be taken as safely equal to the mean depression of the water's surface, or d . The pressure and horsepower required to overcome it may be calculated as follows:

Let h = excess of elevation of water against bow, compared with elevation at stern, in feet. This is taken equal to the mean depression d .

a_1 = midship section of ship in square feet.

w = weight of 1 cubic foot of water in pounds.

P = opposing pressure in pounds.

$V_1 = \frac{5280}{3600} V$ = speed of ship in feet per second.

H = horsepower of ship's engines.

H_1 = horsepower required to overcome end pressure.

Then

$$P = Chwa_1 \quad \dots \quad (3)$$

in which C is a coefficient depending on the ship's model. It will be taken here as $\frac{1}{6}$.

The horsepower required to overcome P when the ship moves at the rate V_1 is

$$H_1 = \frac{PV_1}{550} = \frac{Chwa_1}{550} V_1 = \frac{1}{6} \frac{hwa_1}{550} V_1 \quad \dots \quad (4)$$

and the power remaining to overcome the other resistances is $H - H_1$. The speed of the ship will therefore be reduced. For moderate speeds the horsepower required to move a ship varies approximately as the cube of the speed. If V_2 represents the speed in miles per hour which the ship would attain in the restricted channel if there were required no deduction for head resistance,

$$\begin{aligned} H : H - H_1 &:: V_2^3 : V^3 \\ V^3 &= \frac{H - H_1}{H} V_2^3 \quad \dots \quad (5) \end{aligned}$$

It might seem that the deductions for backward current and increased head resistance were all that are required to give the speed of a ship in a canal. No allowance, however, has yet been made for the changed conditions in the mass of water near the ship, such as eddies, which would not occur to the same extent in deep water, increased friction of the water on the channel bed, etc.; some of the conditions unknown or imperfectly understood, but consuming more or less of the power of the ship's engines. No deduction has yet been made which would be applied to a ship passing through shoal water of unlimited width, but it is well known that there is a material reduction of speed in wide, shoal water. The rate of this retardation doubtless increases with the speed of the ship. A few comparisons have been made of the speed of ships in the deep water of Lake Huron and the shoal water of Lake St. Clair, where the depth of water exceeds by 2 or 3 feet the usual draft of loaded ships. The loss of speed in the shoal water is about 15 per cent, while using the same steam as in deep water. There is probably in this case no appreciable backward current; the water to fill the void behind the moving ship can reach it more quickly by a lateral movement from the unlimitedly wide water. The observed reduction of speed is probably caused almost wholly by the work done in forming eddies, overcoming friction between the water and the bed of the lake, etc., and should be taken into account in calculating the attainable speeds to be expected in a restricted channel. This should apply, however, only when the draft of the ship is within 2 or 3 feet of the depth of the channel. It should be reduced for type A, which will have 5.5 feet more clearance under the keel than the other types. It should also be reduced for all types in those channels where, on account of tides, as in Panama Bay, or on account of storage, as in the summit levels of both routes, the clearance usually exceeds 3 feet by a considerable amount; it is deemed best, however, not to attempt to draw narrow distinctions, but to apply the reduction of 15 per cent to ships of types B, C, and D in all channels of whatever depth. For type A a reduction of 10 per cent is applied for shoal-water effect. It should obviously be less than for the ships of deeper draft, but the difference of 5 per cent may be too great or too small. These percentages are applicable only when the open-sea speed of the ship is 10 to 13 miles per hour.

Shoal water deduction.

These several reductions have been applied to the normal speed of each of the type ships in each of the different channels of both routes, with the results given in Table III. In order to show more clearly how the results are obtained the calculation will be given for a single case:

Channel—standard for firm earth.

Top width, 269 feet.

Side slopes at top, 1 on 1.

A = area of cross section = 7,189.5 square feet.

Ship, type B, H. P. = 2,300 = H.

a_1 = midship section = $50 \times 32 \times 0.96 = 1,536$ square feet.

a = mean section = $50 \times 32 \times 0.8 = 1,280$ square feet.

Depression of water surface assumed to be 1.2 feet.

$\triangle A = (269 - 1.2) 1.2 = 321$ square feet.

$\triangle A + a = 1,601$ square feet.

$A - \triangle A - a = 5,588$ square feet.

v = backward current = $\frac{1601}{5588} V = 0.287 V$.

Speed of ship in open water 12.5 miles per hour.

Deduction for shoal water 15 per cent 1.875 miles per hour.

Speed in wide shoal water 10.625 miles per hour.

Since the head resistance depends on the speed of the ship as finally deduced, it is necessary to assume a value of V .

Assume $V = 7.9$ miles per hour.

$= 11.59$ feet per second.

$v = 0.287 V = 2.267$ miles per hour.

Deduct this from speed of ship in wide shoal water.

$10.625 - 2.267 = 8.358$ = speed of ship without deduction for increased head resistance = V_2 .

For head resistance

$$P = \frac{1}{6} \times 1.2 \times 62.5 \times 1,536 = 19,200 \text{ pounds.}$$

Horsepower required to overcome it

$$= \frac{19200}{550} \times 11.59 = 405 = H_1.$$

$$H - H_1 = 1895.$$

From equation (5).

$$V^3 = \frac{1895}{2300} \times (8.358)^3 = (7.84)^3.$$

$$V = 7.84.$$

Since this does not agree with the assumed value of V , a new trial must be made. The correct value is 7.85.

In the calculations for the following table the depth of water in the channel in the summit level of the Panama Canal is taken as 38 feet; in the summit level of the Nicaragua Canal, 39 feet; in the Pacific section of the Panama Canal, 45 feet, and in all other excavated channels, 35 feet. Allowance has been made in the deepened San Juan for the river section outside of the deepened channel by assuming side slopes of 1 on 4. In the excavated channel in Lake Nicaragua the channel section is computed as having side slopes of 1 on 5.

TABLE III.—*Calculated speeds through isthmian canal channels by ships having sufficient power for a speed of 2.5 miles per hour in open sea.*

Section.	Area of wet section.	Type ship A.				Type ship B.			
		r_1 .	De- pres- sion of water sur- face.	V.	v .	r_1 .	De- pres- sion of water sur- face.	V.	v .
	<i>Square feet.</i>		<i>Feet.</i>	<i>Miles per hour.</i>	<i>Miles per hour.</i>		<i>Feet.</i>	<i>Miles per hour.</i>	<i>Miles per hour.</i>
Canal in rock, summit level excepted, Nicaragua	5,250	4.46	1.3	8.28	2.38	3.42	1.7	7.07	2.93
Canal in rock, west divide, Nicaragua	5,700	4.90	1.2	8.52	2.17	3.71	1.6	7.33	2.67
Canal in rock, Culebra cut, Panama	5,966	5.07	1.1	8.64	2.07	3.88	1.5	7.47	2.55
Canal in firm earth	7,189.5	6.11	.9	8.96	1.83	4.68	1.2	7.85	2.25
Canal in sand or silt	8,925	7.59	.8	9.28	1.53	5.81	1	8.18	1.84
Channel in Panama Bay	15,075	12.82	.6	9.97	.94	9.81	.7	9.14	1.08
Channel in San Juan River, deepened	15,834	13.47	.6	9.97	.94	10.31	.7	9.15	1.08
Channel in Lake Nicaragua, deepened	19,305	16.41	.5	10.18	.75	12.57	.6	9.37	.90
Harbors and entrances, 500 feet wide	21,175	18.02	.5	10.24	.69	13.79	.6	9.49	.83

Section.	Area of wet section.	Type ship C.				Type ship D.			
		r_1 .	De- pres- sion of water sur- face.	V.	v .	r_1 .	De- pres- sion of water sur- face.	V.	v .
	<i>Square feet.</i>		<i>Feet.</i>	<i>Miles per hour.</i>	<i>Miles per hour.</i>		<i>Feet.</i>	<i>Miles per hour.</i>	<i>Miles per hour.</i>
Canal in rock, summit level excepted, Nicaragua	5,250	2.85	2.1	6.53	3.55	2.44	2.4	6.01	4.18
Canal in rock, west divide, Nicaragua	5,700	3.09	1.9	6.87	3.22	2.65	2.2	6.39	3.79
Canal in rock, Culebra cut, Panama	5,966	3.24	1.8	7.02	3.07	2.77	2.1	6.55	3.62
Canal in firm earth	7,189.5	3.93	1.5	7.32	2.70	3.34	1.8	6.94	3.21
Canal in sand or silt	8,925	4.84	1.2	7.95	2.24	4.15	1.4	7.61	2.63
Channel in Panama Bay	15,075	8.18	.7	9.03	1.27	7.01	.8	8.85	1.49
Channel in San Juan River, deepened	15,834	8.59	.7	9.06	1.25	7.36	.8	8.85	1.48
Channel in Lake Nicaragua, deepened	19,305	10.47	.6	9.30	1.04	8.98	.7	9.13	1.21
Harbors and entrances, 500 feet wide	21,175	11.49	.6	9.44	.94	9.85	.7	9.27	1.12

Speeds plotted
and curve
drawn.

The speeds in the foregoing table are plotted on figure 2 as ordinates with values of r_1 as abscissas. Two curves appear, one for type A, the other for types B, C, and D. The curve for type A is an asymptote to a horizontal through $V = 11.25$. The

curve for types B, C, and D is an asymptote to a horizontal through $V=10.625$. For the larger channels neither of these curves can be much in error. At the other extremity both curves must pass through the point where $V=0$ and $r_1=1$. Tests are needed at intermediate points, and particularly where the value of r_1 corresponds with the standard sections in earth.

All steamships are run under check while passing through the St. Clair Flats Canal where, on account of the crowded navigation, more than one hundred vessels passing per day, involving frequent meetings of ships in the canal, speeds are limited by regulations to 8 miles per hour. The estimated traffic through the isthmian canal is about ten ships per day, and the meetings can be provided for more judiciously by reducing the speed of both ships at meeting points, and in the narrower channels by tying up one of them.

Information has been sought regarding the amount of checking, measured by the revolutions of the engines, ordinarily made by ships passing the St. Clair Flats Canal. In open water, at moderate speeds, the speed of a ship varies almost directly with the number of revolutions; in other words, with moderate speeds, the percentage of slip of the propelling wheels is nearly constant. As the speed is increased beyond that suitable for the model of the ship, the slip increases, and at high speeds the increase in speed would be much less than the increase in the number of revolutions. In a canal increase of slip becomes appreciable at a lower speed than in open water. If the number of revolutions were increased greatly, the increase of speed would be materially less; but if, on the other hand, the number of revolutions were increased a small amount, the increase in speed would be in corresponding proportion. Inquiries of shipowners and navigators elicited the statement that on approaching the canal the revolutions were reduced 20 to 25 per cent. With no more reduction than this it might be concluded, without serious error, that with the ordinary power of the engines the speed through the canal would be increased in like proportion. If this rule be applied to the average results given in Table II, assuming the reduction in revolution before entering the canal to have been 20 per cent, the speeds of 7.04, 7.25, 7.56, 8.83, and 6.72 miles observed would become 8.80, 9.06, 9.30, 11.04, and 8.40 if the full power of the engines were used. These are plotted in figure 2, in the points G, H, I, K, and L. The very low value of the observed speed in the last case is due to the fact that two of the three ships entering into this average were towing other ships, one having three, the other two. If the open-sea speed of these ships is less than $12\frac{1}{2}$ miles per hour, which is probable, as few lake freighters equal it, and if allowance were made for this, the points G, H, etc., would plot a little higher.

The following observations have been taken on four ships for the purposes of this study. They were of the largest class now navigating the lakes:

1. The time and number of revolutions across Lake Huron. The distance being known, the speed and number of revolutions per minute become known.
2. While in the open lake the steam was throttled to the same extent as during the passage through the St. Clair Flats Canal. The number of revolutions were taken each minute for five minutes. It is assumed that the speed was reduced in proportion to the number of revolutions.

3. The time passing through the St. Clair Flats Canal was noted. The length of the canal being known, the speed in miles per hour can be calculated. The comparison of the speed under check in Lake Huron with the speed through the canal with the same steam pressure and valve opening furnishes the percentage of loss due to the special resistance in the canal. If the increase of speed with the engines, developing full power, were in the same proportion in the canal as in open sea, the speed to be expected in the canal could be easily determined. The process used here is as follows:

Ship No. 1.—When running at full speed the rate was 11.24 miles per hour; number of revolutions per minute, 74.42. When running under check, with same steam opening as when passing through the canal, the number of revolutions was 56 per minute, and the speed would be $\frac{56}{74.42} \times 11.24 = 8.46$ miles per hour. In the canal the speed was 6.01 miles per hour. The loss of speed due to canal resistance is then $\frac{8.46 - 6.01}{8.46} = 0.29$, or 29 per cent.

Speed through
St. Clair Flats
Canal limited by
regulations.

Reduction of
speed in St. Clair
Flats Canal.

Observations
on Lake Huron
and in St. Clair
Flats Canal.

If the full power of the engines had been used with the same proportionate loss of speed on account of canal resistance the speed in the canal would have been $11.42(1 - 0.29) = 7.98$ miles per hour.

If the power of the ship had been sufficient to propel it 12.5 miles per hour instead of 11.24, the canal speed with full power of engines and the same assumptions as to increase of speed would have been $12.5 \times (1.00 - 0.29) = 8.88$ miles. For this ship $r_1 = 8.50$. The speed is plotted at M, Fig. 2.

Ship No. 2.—Full speed in open lake, 11.51 miles per hour. Revolutions per minute at full speed, 71.43. Revolutions per minute in open lake while under check, 43. Speed in canal with same steam, 4.88 miles per hour. Following the same method as before, the calculated speed with full power of engines capable of moving the ship $12\frac{1}{2}$ miles per hour in open sea would be 8.81 miles per hour. In this case $r_1 = 7.79$. The point is plotted at N.

Ship No. 3.—Full speed in open lake, 10.61 miles per hour. Revolutions per minute in open lake at full speed, 78.35. Revolutions per minute in open lake while under check, 59. Speed in canal with same steam, 6.75 miles per hour. Calculated speed in canal with full power of engines capable of moving the ship $12\frac{1}{2}$ miles per hour in open lake, 10.56 miles per hour. In this case $r_1 = 7.68$. The point is plotted at V.

Ship No. 4.—Full speed in open lake, 13.13 miles per hour. Revolutions per minute in open lake at full speed, 84.79. Revolutions per minute in open lake while under check, 63.2. Speed in canal with same steam, 7.92 miles per hour. Calculated speed in canal with full power of engines capable of moving the ship $12\frac{1}{2}$ miles per hour in open lake, 10.12 miles per hour. In this case $r_1 = 8.14$. The point is plotted at W.

There is considerable discrepancy in these results, two of the points (M and N) falling near the curve of calculated speeds, but a little below it, the other two falling considerably above it. Considerable variation would be produced by difference in ship models. The variations may also be due to inaccuracies in observations, particularly in regard to the exact steam opening while the engines were under check. The assumption that increase in number of revolutions will give the same proportion of increase in speed in a canal as in open sea is not exactly true, although the error will not be great, with the channel, increase of revolutions, and speed taken here. If the channel were smaller the error would be greater. Another consideration to be taken into account is the fact that the ships passing the St. Clair Flats Canal have more water under the keel than assumed for types B, C, and D in the standard earth sections. For the summit-level sections, however, the clearance will be about the same as in the St. Clair Flats Canal.

Information was obtained from the manager of one of the steamship lines giving for two ships the number of revolutions of the engines in open lakes and in the canal. It may be assumed without great error, in these cases, that the efficiency of the engines in producing speed was reduced in inverse proportion. The data and calculations are as follows:

Ship No. 5.—Revolutions per mile in open lake, 404; in canal, 530. Loss in efficiency of engines for producing speed $= 1 - \frac{404}{530} = 23.8$ per cent. Calculated speed in canal with engines capable of moving the ship $12\frac{1}{2}$ miles per hour in open lake $= 12\frac{1}{2} \times (1 - .238) = 9.52$ miles per hour. In this case $r_1 = 7.61$. The point is plotted at X.

Ship No. 6.—Revolutions per mile in open lake, 346; in canal, 457. Loss in efficiency of engines for producing speed $= 1 - \frac{346}{457} = 24.3$ per cent. Calculated speed in canal with engines capable of moving the ship $12\frac{1}{2}$ miles per hour in open lake $= 12\frac{1}{2} \times (1 - .243) = 9.46$ miles per hour. In this case $r_1 = 9.42$. The point is plotted at Y.

These points fall near the curve for type A. These calculations are also based on the assumption that speed would vary directly as the number of revolutions, both in open lake and in the canal, which, as above stated, is somewhat in error.

These data, taken collectively, appear to confirm the calculated speeds in the part of the curve near M and N; but the data being inexact and the basis of the calculations involving some error, it seems to be judicious to adopt for the present discussion a little lower speeds in the vicinity of these points. To the right of these points the possible error is less, the speeds being definitely known for wide shoal water.

Calculated
speeds may be
too high.

To the left of the given points the data are not so good and the possibility of material error is greater. The St. Clair Flats Canal is the only one in the United States traversed at speed by deep-draft ships, but the channel is so wide (296 feet) and the value of r_1 so great that the observations do not apply directly when r_1 is less than 7.5. In the canal sections of the isthmian waterway, the value of r_1 will vary for the type ships from 7.59 to 2.44. (See Table III.) Systematic observations on such channels with large ships are much needed.

The best opportunities for such observations would be afforded at the Suez Canal, for at this waterway are found a channel of moderate cross section and a heavy traffic with large ships. Only two observations made there are available. The first one is reported by Sir Charles Hartley (Minutes of Proceedings, Inst. C. E., vol. 141, p. 169), who cites the case of the steamship *Austral* on a passage made in 1885:

Observations at
Suez Canal.

On the straight reaches of the canal, between Lake Timsah and Suez, the ship only made 5 knots an hour with 43 revolutions of her screw. Whereas, in the open sea, with the same number of revolutions, she made 11 knots.

There was a tide of half a knot per hour in her favor in the canal. The speed of 11 knots per hour is almost identical with that assumed for the type ships. Deducting from the observed speed for the tidal current, the corrected speed in the canal would be 4.5 knots, or 5.17 statute miles, per hour. If this speed was observed between Lake Timsah and the Bitter Lakes, the value of r_1 would be 2.95. The point thus determined is plotted at P in figure 2. If the speed was noted on the larger section between the Bitter Lakes and Suez, r_1 would be 3.25, and the point thus determined is plotted at Q. The clearance under the keel of the *Austral* was less than assumed for the type ships in the isthmian canal. The reduction of speed was therefore somewhat greater than would be sustained by the latter. The second observation at Suez is reported by Mr. Lionel B. Wells (M. P. I. C. E., vol. 141, p. 208) as follows: In 1898 he had visited the Suez and traveled along the canal thence to Ismailia. The passage occupied from 5.30 p. m. until 11.45 p. m., the average speed, irrespective of stoppages, being therefore more than 6 knots per hour. The steamship was 320 feet long, 39 feet wide, and drew about 21 feet, her gross tonnage being 2,260 tons; he was told that 11 knots was good speed at sea. In the canal 8 knots per hour were made at times, for the regulation as to maximum speed (5.33 knots per hour) was disregarded. The tide was against the ship, but the velocity of current is not stated. There is the same uncertainty here as in Hartley's observations as to the section of the canal where the speed of 8 knots was observed. The enlargement of the canal to the dimensions reported in 1899 is supposed to have been completed. The points for the two sections are plotted at R and S in figure 2 without allowance for the opposing current. The ship had nearly the same clearance under the keel as type A will have in the isthmian canal. The observation is therefore applicable to that type, and the points are found to fall near its attainable speed curve.

Referring to the Kaiser Wilhelm (Kiel) Canal, the chief constructing engineer, Mr. Fölscher, states (M. P. I. C. E., vol. 141, p. 207):

Observations
at Kiel Canal.

Vessels capable of making 16 knots per hour in the open sea and having an immersed midship section of about 1,400 square feet only made between 5 and 6 knots per hour for the same expenditure of power when passing through that canal, which had a cross-sectional water area of 4,520 square feet. * * * Vessels having an immersed midship section of between 645 and 753 square feet, being about one-sixth of the cross-sectional water area, traversed the canal without undue expenditure of power and without causing objectionable waves at a speed of about 6½ knots per hour.

The speed for the ship of 1,400 square feet section is plotted at T, and for the smaller ship last mentioned at U, fig. 2.

The foregoing data from the Suez and Kiel canals indicate that for values of r_1 considerably less than 7, the calculated speeds for types B, C, and D are too high. The material reduction of speed when the ship's keel is near the channel bed has been taken into account in the preceding pages. An analogous reduction of speed must be incurred when the ship's sides approach the sides of the channel, and this may not have been fully measured by the allowances made for backward current and depression of water surface. A new curve for these types is therefore drawn, which will coincide with the so-called "calculated attainable speed curve" in very wide shoal water, would pass a little below G, H, I, M, and N, where the calculated speeds are certainly

not much in error, and thence to the left, where the data are less definite, deviates more widely, passing below T and a little above P and Q. It is deemed proper to give greater weight to G, N, and M than to U. The new curve is marked "Type ships B, C, and D. Final curve for attainable speeds."

The calculated curve for type A is well supported for values r_1 of 7 to 8 by Mr. Wells's observation in the Suez Canal. It is deemed judicious, however, to draw a new curve for this type also, with corresponding reduction of speed, because the data applicable are not quite so full.

The final curve for types B, C, and D may be further tested by the St. Clair observations. Referring to the group of four ships in Table II having r_1 between 11 and 12, the record shows that all of them had speeds of more than 8 miles per hour, the mean being 8.83. It may be accepted as certain that these ships ran under check and that their average sea speed is less than $12\frac{1}{2}$ miles per hour. The observed speed in the canal is plotted at O in figure 2 and falls but little below the final curve; that is to say, the speed by the curve was nearly equaled by ships of less average power than assumed running under check. The ships of this group are not remarkable in any way. They are medium-sized freight ships, and it happened that they were checked less while passing the canal than the other groups.

Speed limit
fixed by back-
ward currents.

In the smaller channels, where formed in earth, the attainable speeds are not permissible because of the injurious results to the canal banks. This injury is caused mainly by wave wash, but to some extent by backward currents. While the waves produced by the passage of a ship are not directly proportional to the backward current, there seems to be no better way of fixing a speed limit than by reference to the current. In the Suez Canal speeds are permitted which produce backward currents exceeding 4 feet per second. In the Amsterdam Canal the largest ships are limited to speeds causing a backward current of about 2.3 feet per second. In both cases the banks are of sandy material, easily moved. In the Manchester Canal the largest ships are permitted a speed of 6 miles per hour, causing a backward current of 3.3 feet per second. These currents, it should be said, are calculated from the approximate

formula $v = \frac{V}{r_1 - 1}$, in which v is the velocity of back current, V the speed of the ship, and r_1 the quotient obtained by dividing the wet cross section of the canal by the ship's midship section. The actual velocities are a little greater, and in the Suez and Manchester canals they are further augmented by tidal currents. Tides and floods will cause currents in the Isthmian Canal, augmenting the calculated backward current, which, it is assumed for this discussion should not exceed 3 feet per second, or 2.05 miles per hour. The resulting limiting speeds are shown in figure 2. It will be noted that this line replaces that part of the final speed curve concerning which any question could plausibly be raised.

Finally, speeds are to be read from the diagram, as follows:

Type.	Earth chan- nels, from line—	Rock chan- nels, from line—
Ship A.....	a d e	g d e
Ships B, C, D.....	a b c	f b c

Reduction of
speed on curves.

If the width of the channel were not increased on curves and the speed of the ship were not reduced, the difficulty of steering and the risks to ships would be greater than on tangents. The projects for the isthmian canal provide for a widening on curves which varies inversely with the radius of curvature, and possibly this may fully compensate. It is believed, however, that a reduction of speed would be made on approaching any curve. If practicable so to make it, such a reduction might vary directly with the degree of curvature and inversely with the width, but a schedule thus made would be too complex for practical use. Since the curves on the Panama route are generally of larger radius than on the Nicaragua route, the reduction of speed need not be so great. A reduction of speed of 20 per cent in the narrow sections of the Nicaragua route and of 15 per cent in the narrow sections of the Panama route would seem to be sufficient. In the San Juan River and the harbor entrances, where the width is greater but the curvature sharp, the reduction may be taken 15 per cent; in Lake Nicaragua and Panama Bay, where the curvature is slight, the reduction may be 10 per cent.

TABLE IV.—Schedule of speeds proposed as a basis for calculating the time of transit through the Panama and Nicaragua canals.

[Speeds in statute miles per hour.]

	Tangents or curves.	Type A.	Type B.	Type C.	Type D.
Rock cuts:					
East and west of summit level, Nicaragua.....	{Tangents.....	7.5	6.0	5.0	4.5
	{Curves.....	6.0	5.0	4.0	3.5
West divide, Nicaragua.....	{Tangents.....	7.5	6.0	5.5	4.5
	{Curves.....	6.0	5.0	5.0	4.0
Culebra cut, Panama.....	{Tangents.....	8.0	6.5	5.5	5.0
	{Curves.....	6.5	5.5	4.5	4.0
Canal section in firm earth.....	{Tangents.....	8.0	7.0	6.0	4.5
	{Curves <i>a</i>	7.0	6.0	5.0	4.0
	{Curves <i>b</i>	6.5	5.5	4.5	3.5
Canal section in sand and silt.....	{Tangents.....	8.5	7.5	7.0	6.0
	{Curves <i>a</i>	7.0	6.5	6.0	5.0
	{Curves <i>b</i>	6.5	6.0	5.5	5.0
Channel in Panama Bay.....	{Tangents.....	9.5	8.5	8.5	8.0
	{Curves.....	8.5	8.0	7.5	7.5
San Juan River, where deepened.....	{Tangents.....	9.5	8.5	8.5	8.0
	{Curves.....	8.0	7.5	7.0	7.0
Lake Nicaragua, where deepened.....	{Tangents.....	10.0	9.0	8.5	8.5
	{Curves.....	9.0	8.0	7.5	7.5
Lake Bohio.....	{Tangents.....	10.0	10.0	10.0	9.5
	{Curves.....	9.0	9.0	9.0	8.5
San Juan River, where not deepened.....	{Tangents.....	10.0	9.5	9.0	9.0
	{Curves.....	8.5	8.0	7.5	7.5
Lake Nicaragua, where not deepened.....	{Tangents.....	12.5	12.5	12.5	12.5
	{Curves.....	10.0	9.0	9.0	8.5
Harbor and harbor entrances.....	{Tangents.....	8.5	7.5	7.5	7.0
	{Curves.....	8.5	7.5	7.5	7.0

a Panamá.*b* Nicaragua.

The foregoing schedule applied to the two routes gives the times of transit across the Isthmus shown in Tables V and VI, but without allowance for delays at meeting points and at locks.

TABLE V.—Time of transit through the Panama Canal without allowance for meetings or lockages.

	Tangents or curves.	Distance.	Type A.	Type B.	Type C.	Type D.
		<i>Statute miles.</i>	<i>Hours.</i>	<i>Hours.</i>	<i>Hours.</i>	<i>Hours.</i>
Canal section:						
Firm earth.....	{Tangents.....	10.76	1.35	1.54	1.80	2.39
	{Curves.....	6.85	.98	1.14	1.37	1.71
Sand and silt.....	{Tangents.....	3.48	.41	.46	.50	.58
	{Curves.....	2.50	.36	.38	.42	.50
Culebra cut.....	{Tangents.....	3.00	.37	.46	.55	.60
	{Curves.....	3.02	.46	.55	.67	.75
Channel in Panama Bay.....	{Tangents.....	3.90	.41	.46	.46	.49
	{Curves.....	.51	.06	.07	.07	.07
Colon Harbor and entrance.....	{Tangents.....	.39	.04	.04	.04	.05
	{Curves.....	2.00	.24	.27	.27	.29
Lake Bohio.....	{Tangents.....	6.18	.62	.62	.62	.65
	{Curves.....	6.50	.72	.72	.72	.76
Total.....		49.09	6.02	6.71	7.49	8.84

TABLE VI.—Time of transit through the Nicaragua Canal without allowance for meetings or lockages.

	Tangents or curves.	Distance.	Type A.	Type B.	Type C.	Type D.
		<i>Statute miles.</i>	<i>Hours.</i>	<i>Hours.</i>	<i>Hours.</i>	<i>Hours.</i>
Canal section:						
Firm earth.....	{Tangents.....	17.99	2.25	2.57	3.00	4.00
	{Curves.....	10.45	1.61	1.90	2.32	2.99
Sand or silt.....	{Tangents.....	19.93	2.34	2.66	2.85	3.32
	{Curves.....	8.39	1.29	1.40	1.53	1.68
Rock, west divide.....	{Tangents.....	4.79	.64	.80	.87	1.06
	{Curves.....	3.89	.65	.78	.78	.97
Rock, exclusive of west divide.....	{Tangents.....	2.94	.39	.49	.59	.65
	{Curves.....	2.32	.39	.46	.58	.66
San Juan River:						
Where deepened.....	{Tangents.....	12.58	1.32	1.48	1.48	1.57
	{Curves.....	13.77	1.72	1.84	1.97	1.97
Not deepened.....	{Tangents.....	5.15	.52	.54	.57	.57
	{Curves.....	7.86	.92	.98	1.05	1.05
Lake Nicaragua:						
Where deepened.....	{Tangents.....	26.69	2.67	2.97	3.14	3.14
	{Curves.....	1.56	.17	.19	.21	.21
Not deepened.....	{Tangents.....	42.27	3.38	3.38	3.38	3.38
	{Curves.....	2.04	.20	.23	.23	.24
Harbors and entrances.....	{Tangents.....	1.04	.12	.14	.14	.15
	{Curves.....	1.04	.12	.14	.14	.15
Total.....		183.66	20.58	22.81	24.69	27.61

To obtain the full time of transit the time consumed by lockage and the delays at meeting points are to be added.

Time consumed
by lockage.

The time required for lockage is here taken to be the delay caused by a lockage. It is the period beginning when the ship begins to slacken speed on approaching a lock, and ending when it has acquired full speed after leaving it, less the time that is required to pass over the same distance at full canal speed. The delay while waiting for the locking of another ship will be considered further on.

The channel in the vicinity of the lock is taken to be of standard canal section in firm earth. While this is not true in every case, the resulting error is not material. The approaching ship must reduce speed, and be under perfect control, while yet at a considerable distance from the lock. Its movements while approaching, passing, and leaving the lock are supposed to be as follows:

1. The speed of the approaching ship is to be reduced to 1.7 miles per hour (150 feet per minute) at a point 700 feet distant from the lock-gate quoin. This reduction of speed is to be made at the rate of 1 mile per hour while moving 500 feet.

Observations on a large number of lake freight ships show that when the engines are stopped the speed is reduced at an average rate of 1 mile per hour while moving less than 400 feet. (See report U. S. Board of Engineers on Deep Waterways.) A like increase of speed occurs when the engines are again started.

2. The ship is to continue at the speed of 1.7 miles per hour until its stern is within 150 feet of the lock-gate quoin (200 feet, if moving down stream) and then is to come to a stop in the next 200 feet and during two and two-thirds minutes, backing the wheel if necessary.

Observations on 35 ships entering the lock of the St. Marys Falls Canal show that a stop from a speed of 125 feet per minute is made in 100 feet and in 1.4 minutes.

3. The time required to open or close a pair of lock gates is taken at two minutes.

At the Ymuiden lock, on the Amsterdam Canal, the observed time was one and one-third minutes.

4. The time required for filling and emptying the several locks is given in the following table, VII. (See Appendix A for details of calculation.)

TABLE VII.—Time required for filling and emptying locks.

	Filling.	Empty- ing.
<i>Panama route.</i>		
	<i>Min. sec.</i>	<i>Min. sec.</i>
Lower lock at Bohio.....	13 7	16 19
Upper lock at Bohio.....	16 19	13 7
Upper lock at Pedro Miguel.....	13 14	10 39
Lower lock at Pedro Miguel.....	10 39	13 14
Miraflores lock.....	13 24	13 24
<i>Nicaragua route.</i>		
Lock No. 1.....	14 56	14 56
Lock No. 2.....	10 42	10 42
Lock No. 3.....	10 42	10 42
Lock No. 4.....	14 43	14 43
Lock No. 5.....	12 49	12 49
Lock No. 6.....	13 17	13 17
Lock No. 7.....	13 17	13 17
Lock No. 8.....	12 18	12 18

5. A ship leaving the lock is to acquire a speed of 1.7 miles per hour while moving 300 feet, requiring four minutes, and is then to gain speed at the rate of 1 mile per hour while moving 500 feet, until full canal speed is attained.

Observations on 30 ships leaving the lock of the St. Marys Falls Canal gave as mean results an acquired speed of 1.7 miles per hour while moving 200 feet in two and one-third minutes, and then an increase of speed of 1 mile per hour while moving 360 feet. The observations extended only until a speed of 3 miles per hour was attained.

These rules applied to the Bohio locks result in the time interval given in the next table.

TABLE VIII.—*Time required to pass Bohio locks.*

[In minutes.]

	Type A.	Type B.	Type C.	Type D.
Reducing speed to 1.7 miles per hour	8.8	8.0	7.2	5.5
Moving at 1.7 miles per hour until stern is 175 feet from lock-gate quoin	6.2	6.2	7.1	7.8
Coming to full stop in first lock	2.7	2.7	2.7	2.7
Closing lock gates	2.0	2.0	2.0	2.0
Filling (or emptying) first lock	13.1	13.1	13.1	13.1
Opening second gates	2.0	2.0	2.0	2.0
Moving into second lock:				
Attaining speed of 1.7 miles per hour	4.0	4.0	4.0	4.0
Proceeding at this speed 300 feet	2.0	2.0	2.0	2.0
Coming to full stop in second lock	2.7	2.7	2.7	2.7
Closing second gates	2.0	2.0	2.0	2.0
Filling (or emptying) second lock	16.3	16.3	16.3	16.3
Opening third gates	2.0	2.0	2.0	2.0
Moving out of lock and attaining full speed:				
Attaining speed of 1.7 miles per hour	4.0	4.0	4.0	4.0
Increasing to full canal speed	8.8	8.0	7.2	5.5
Time required to traverse this distance at full canal speed	76.6	75.0	74.3	71.6
Time lost at locks	12.6	12.8	13.0	13.6
Distance traversed	64.0	62.2	61.3	58.0
	8,886	7,886	6,886	5,386

These are the net delays if all movements are made without loss of time. In practice this would not be possible. In 1894 the lock at the St. Marys Falls Canal was worked nearly to its full capacity, and the tonnage through it was about 0.62 of the theoretical capacity of the lock. Probably ships were awaiting lockage 90 to 95 per cent of the time. If this is correct, about 50 per cent should be added to the calculated time of lockage to obtain the time used in actual work. This allowance is adopted for the present discussion. The resulting time lost by lockages is given for the two routes in Tables IX and X.

TABLE IX.—*Time lost by lockages on the Panama route.*

	Type A.	Type B.	Type C.	Type D.
Bohio locks	64.0	62.2	61.3	58.0
Pedro Miguel locks	58.5	56.7	55.8	52.5
Miraflores lock	36.4	34.8	34.1	31.3
Total	158.9	153.7	151.2	141.8
Add 50 per cent for sundry delays	2.65	2.56	2.52	2.36
Total delays for lockages	1.32	1.28	1.26	1.18
	3.97	3.84	3.78	3.54

TABLE X.—*Time lost by lockages on the Nicaragua route.*

	Type A.	Type B.	Type C.	Type D.
Atlantic slope:				
Lock No. 1	37.9	36.3	35.6	32.8
Lock No. 2	33.7	32.1	31.4	28.6
Lock No. 3	33.7	32.1	31.4	28.6
Lock No. 4	37.7	36.1	35.4	32.6
Pacific slope:				
Lock No. 5	35.8	34.2	33.5	30.7
Lock No. 6	36.3	34.7	34.0	31.2
Lock No. 7	36.3	34.7	34.0	31.2
Lock No. 8	35.3	33.7	33.0	30.2
Total	286.7	273.9	268.3	245.9
Add 50 per cent for sundry delays	4.78	4.57	4.47	4.10
Total delay for lockages	2.39	2.28	2.23	2.05
	7.17	6.85	6.70	6.15

TABLE XII.—*Number of meetings for each ship in the canal sections of the Panama route and delays therefrom.*

	Type A.	Type B.	Type C.	Type D.
Number of hours traversing sections of 150 feet, bottom width, excluding meetings ...	3.93	4.53	5.31	6.53
Average number of meetings	1.91	2.21	2.62	3.31
Delay in transithours..	.65	.77	.97	1.42

TABLE XIII.—*Number of meetings for each ship in the canal sections of the Nicaragua route and delays therefrom.*

	Type A.	Type B.	Type C.	Type D.
Number of hours traversing sections of 150 feet, bottom width, excluding meetings.....	9.56	11.06	12.52	15.32
Average number of meetings	4.64	5.40	6.17	7.78
Delay in transithours..	1.58	1.89	2.28	3.35

The delays to a ship in transit on account of meetings in the wide channels is calculated in the same manner as for the canal sections.

TABLE XIV.—*Delay at a meeting point in the channel in San Juan River, Lake Nicaragua, or Panama Bay.*

[In minutes.]

	Type A.	Type B.	Type C.	Type D.
Elapsed time while reducing speed to 4 miles per hour.....	3.9	3.2	2.3	0.7
Elapsed time while running 2,000 feet at 4 miles per hour.....	5.6	5.6	5.6	5.6
Elapsed time while regaining speed	3.9	3.2	2.3	.7
Total elapsed time.....minutes..	13.4	12.0	10.2	7.0
Total distance traversed.....feet..	6,000	5,000	4,000	2,500
Time required to traverse this distance at full allowed speed.....minutes..	8.5	8.1	7.6	6.4
Delay to shipminutes..	4.9	3.9	2.6	.6
.....hours..	.08	.07	.04	.01

TABLE XV.—*Number of meetings for each ship in the channel excavated in Panama Bay and delays therefrom.*

	Type A.	Type B.	Type C.	Type D.
Number of hours traversing channel	0.47	0.53	0.53	0.56
Average number of meetings.....	.20	.23	.23	.24
Delays in transithours..	.02	.02	.01	.00

TABLE XVI.—*Number of meetings for each ship in the channel excavated in the San Juan River and Lake Nicaragua.*

	Type A.	Type B.	Type C.	Type D.
Number of hours traversing channel	5.84	6.43	6.75	6.84
Average number of meetings	2.50	2.76	2.86	2.87
Delays in transithours..	.20	.19	.11	.03

It is not likely that a full working force will be provided for both sets of locks, the duplicate locks being provided to enable repairs to be made to either separately, without closing the canal to navigation, and as insurance against delay to navigation in case of accident to either lock. It is assumed, however, that both sets will be in use, when in order. If a ship should arrive while another is being locked it will have to wait until the other is out of the way, or until some of the working force can be spared to attend to it. Except at the initial locks, ships following each other will be spaced by previous lockages, and so will delay each other but little. Meetings, however, are as likely to occur at locks as anywhere else in proportion to the time consumed there. If the second ship arriving were obliged to wait until the first one was entirely clear of the lock and approach, the average delay, when one occurred, would be one-half the time required for a lockage (not the time lost by lockage).

Meetings at locks.

Average delay to each ship.

Only one of two meeting ships would suffer this delay and the average loss of time to each ship would be one-fourth the time required for a lockage. In practice, however, the delay would probably be a little less because, after the first ship has been tied up in the lock, a part of the force could be spared to begin operating the duplicate lock. The time thus saved would provide for all delays to ships following each other. The full allowance will therefore be made here for meeting ships and none for following ships. The number of meetings is the total delay for lockages as obtained from Tables IX and X, by applying formula (7). The time required for passing the locks is the calculated time plus the allowance of 50 per cent for sundry delays. The time lost by a meeting at the Bohio locks is estimated as shown in Table XVII.

TABLE XVII.—*Delay caused by a meeting at Bohio locks.*

[Interval between arrival of ship at a point 700 feet from the lock-gate quoin and the point when in leaving the lock it has attained a speed of 1.7 miles per hour. This is taken to be the maximum delay which a second ship would meet. Data from Table VIII.]

[In minutes.]

	Type A.	Type B.	Type C.	Type D.
Moving at 1.7 miles per hour until stern is 175 feet from lock-gate quoin	6.2	6.2	7.1	7.8
Coming to full stop in lock	2.7	2.7	2.7	2.7
Closing lock gates	2.0	2.0	2.0	2.0
Filling (or emptying) first lock	13.1	13.1	13.1	13.1
Opening second gates	2.0	2.0	2.0	2.0
Moving into second lock	8.7	8.7	8.7	8.7
Closing second gates	2.0	2.0	2.0	2.0
Filling (or emptying) lock	16.3	16.3	16.3	16.3
Opening third gates	2.0	2.0	2.0	2.0
Attaining speed of 1.7 miles per hour	4.0	4.0	4.0	4.0
Maximum delay to waiting ships	59.0	59.0	59.9	60.6
Average delay to waiting ships	30.0	30.0	30.0	30.0
Average delay to both meeting ships	15.0	15.0	15.0	15.0
Average delay with 50 per cent added	22.0	22.0	23.0	23.0

From the foregoing table the average delay to all ships at the Bohio flight of two locks would be fifteen minutes. In a similar way the average delay at other locks is calculated, with the following results:

TABLE XVIII.—*Delays caused by meetings at each of the locks or flights of the Panama route.*

[In minutes.]

	Type A.	Type B.	Type C.	Type D.
At Bohio locks	15.0	15.0	15.0	15.0
At Pedro Miguel locks	13.0	13.0	14.0	14.0
At Miraflores lock	8.0	8.0	8.0	8.0
Total	36.0	36.0	37.0	37.0
Mean	12.0	12.0	12.0	12.0
Mean with 50 per cent added	.2	.2	.2	.2
Mean with 50 per cent added	.3	.3	.3	.3

TABLE XIX.—*Delays at meetings at each of the locks of the Nicaragua route.*

[In minutes.]

	Type A.	Type B.	Type C.	Type D.
At Lock No. 1.	8.0	8.0	8.0	8.0
At Lock No. 2.	7.0	7.0	7.0	7.0
At Lock No. 3.	7.0	7.0	7.0	7.0
At Lock No. 4.	8.0	8.0	8.0	8.0
At Lock No. 5.	7.0	7.0	8.0	8.0
At Lock No. 6.	8.0	8.0	8.0	8.0
At Lock No. 7.	8.0	8.0	8.0	8.0
At Lock No. 8.	7.0	7.0	8.0	8.0
Totals	60.0	60.0	62.0	62.0
Mean	7.5	7.5	7.8	7.8
Mean with 50 per cent added	.12	.12	.13	.13
Mean with 50 per cent added	.18	.18	.19	.19

These delays for each meeting multiplied by the number of meetings give the following tables, XX and XXI:

TABLE XX.—*Number of delays while awaiting lockage on the Panama route, and delays therefrom.*

	Type A.	Type B.	Type C.	Type D.
Number of delays per transit.....	1.89	1.83	1.80	1.69
Time of each delay.....hours..	.30	.30	.30	.30
Total delay.....	.57	.55	.54	.51

TABLE XXI.—*Number of delays while awaiting lockage on the Nicaragua route, and delays therefrom.*

	Type A.	Type B.	Type C.	Type D.
Number of delays per transit.....	3.23	3.09	3.02	2.78
Time of each delay.....hours..	.18	.18	.19	.19
Total delay.....	.58	.56	.57	.53

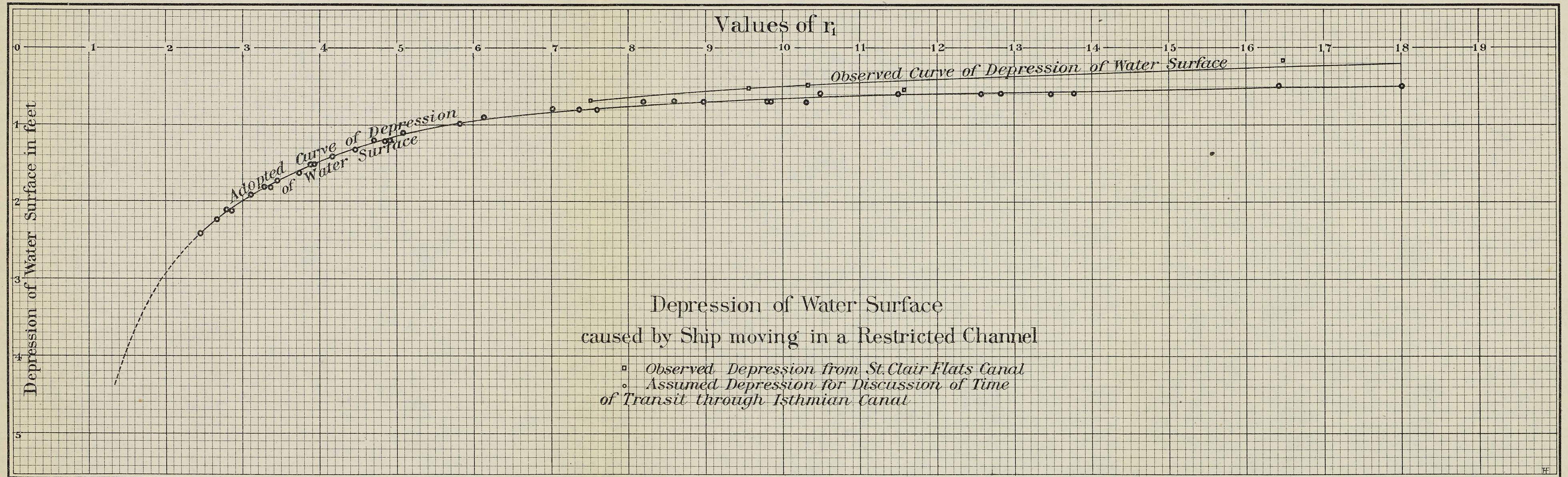
The several time intervals are summed up in the following table, giving the total time of transit by each route:

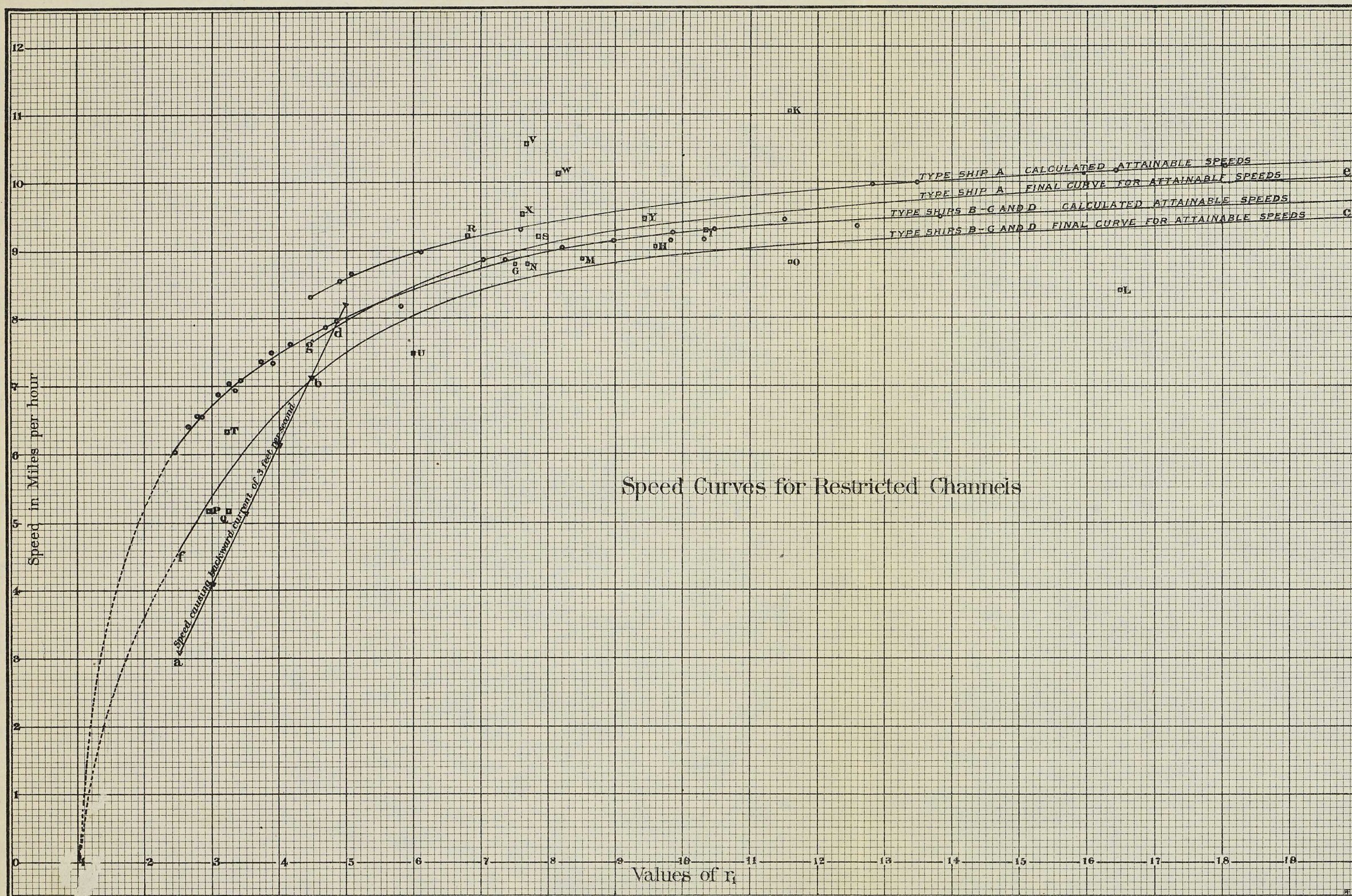
TABLE XXII.—*Time of transit across isthmus.*

[In hours.]

	Type A.	Type B.	Type C.	Type D.
<i>Panama route.</i>				
Time without allowance for lockage or meetings.....	6.02	6.71	7.49	8.80
Lockages.....	3.97	3.84	3.78	3.54
Meetings in canal section.....	.65	.77	.97	1.42
Meetings in channel in Panama Bay.....	.02	.02	.01	.00
Meetings at locks.....	.57	.55	.54	.51
Total time of transit.....	11.23	11.89	12.79	14.27
<i>Nicaragua route.</i>				
Time without allowance for lockage or meetings.....	20.58	22.81	24.69	27.61
Lockages.....	7.17	6.85	6.70	6.15
Meetings in canal sections.....	1.58	1.89	2.28	3.35
Meetings in channels in San Juan River and Lake Nicaragua.....	.20	.19	.11	.03
Meetings at locks.....	.58	.56	.57	.53
Total time of transit.....	30.11	32.30	34.35	37.67

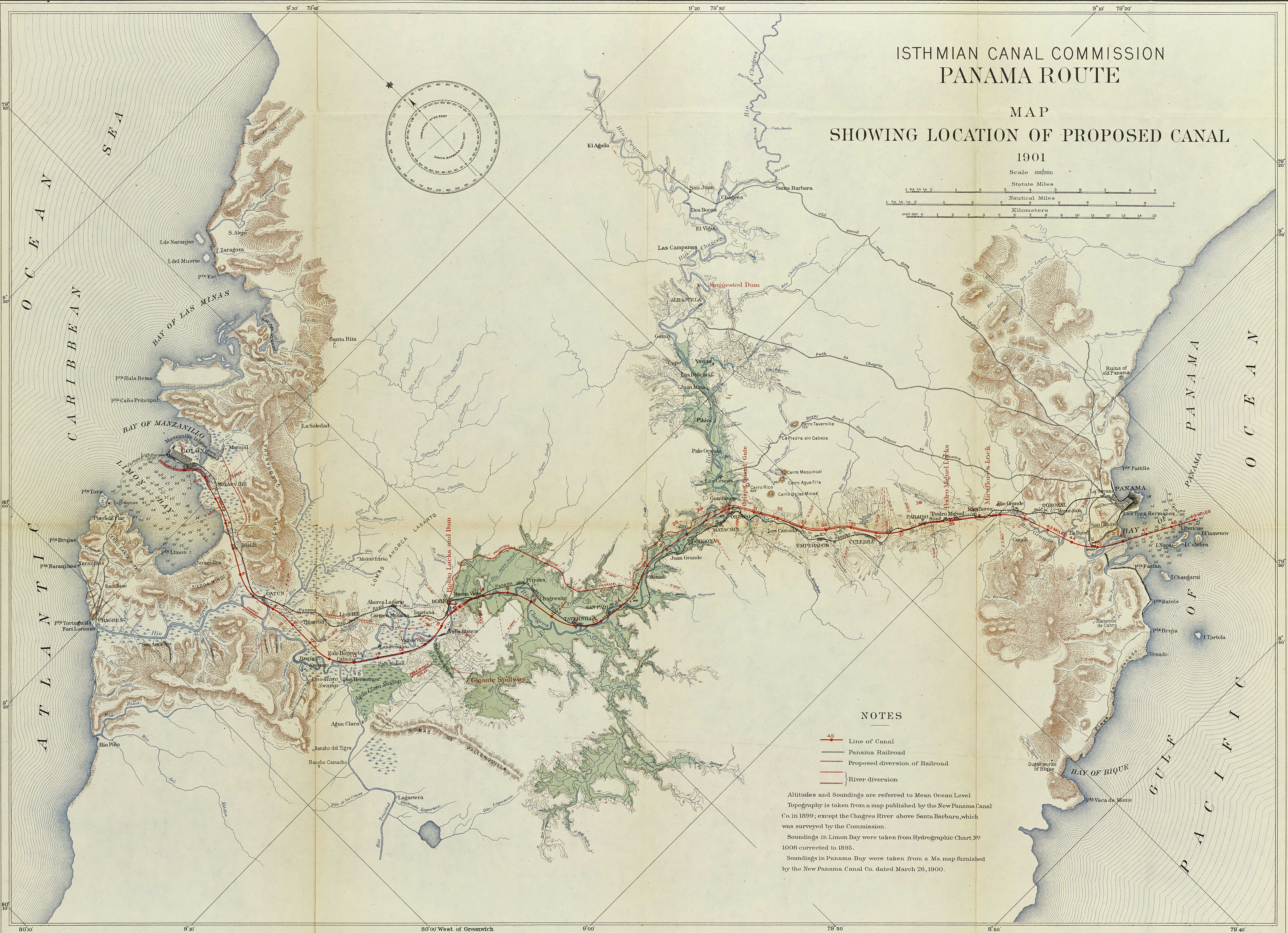
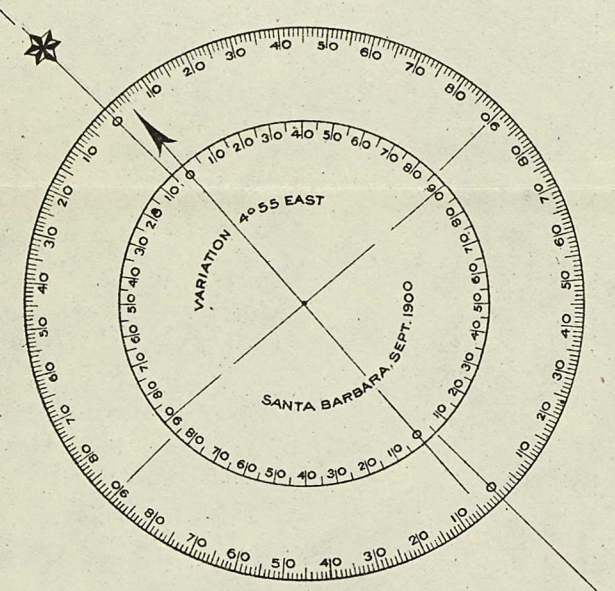
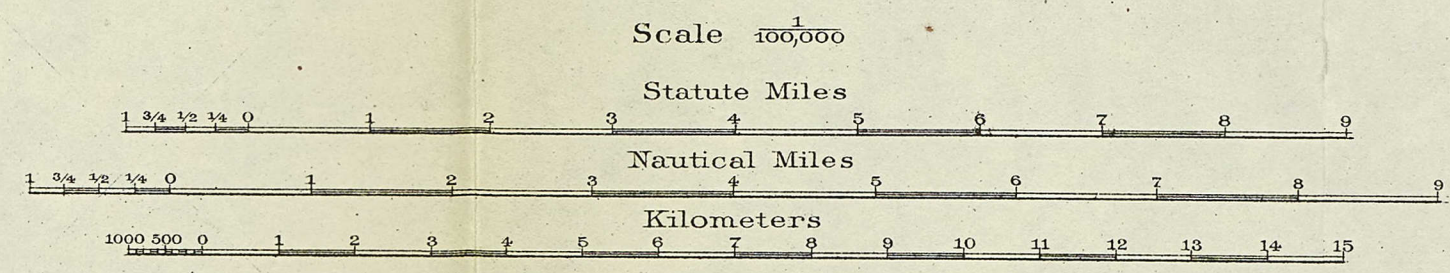
These estimates are for ships having a sea speed of $12\frac{1}{2}$ statute miles per hour. It is assumed that passing places, where ships can tie up, will be provided at intervals of 5 miles in all canal sections, and that the route will be so efficiently lighted, either by fixed lights along the route or by lights carried by the ships, or by both, as experience may indicate, that navigation may be continued by night as well as by day. Should the ships be powered for a less sea speed the time of transit will not be increased in simple inverse proportion, nor could the time of transit be much reduced if the sea speed were considerably greater. If no provisions be made for passing places, or if the lighting be inadequate, the estimates do not apply.





ISTHMIAN CANAL COMMISSION PANAMA ROUTE

MAP SHOWING LOCATION OF PROPOSED CANAL 1901



NOTES

- Line of Canal
- Panama Railroad
- Proposed diversion of Railroad
- River diversion

Altitudes and Soundings are referred to Mean Ocean Level
Topography is taken from a map published by the New Panama Canal Co. in 1899; except the Chagres River above Santa Barbara, which was surveyed by the Commission.
Soundings in Limon Bay were taken from Hydrographic Chart No 1008 corrected to 1895.
Soundings in Panama Bay were taken from a Ms. map furnished by the New Panama Canal Co. dated March 26, 1900.

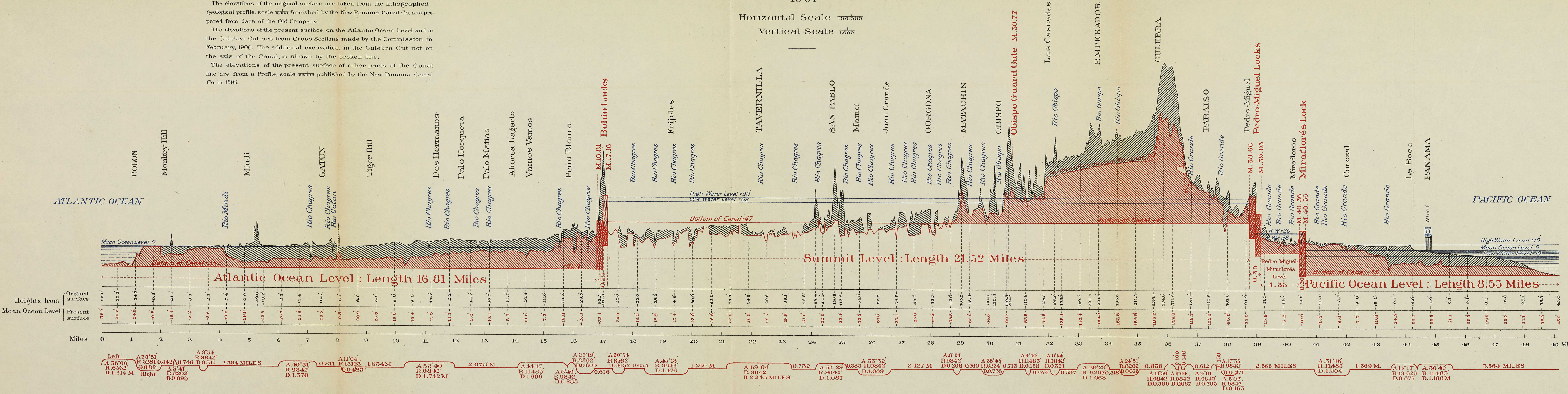
ISTHMIAN CANAL COMMISSION
PANAMA ROUTE

PROFILE
OF
PROPOSED CANAL
1901

Horizontal Scale $\frac{1}{100,000}$
Vertical Scale $\frac{1}{1,000}$

NOTE.

The Profile is taken on the Axis of the Canal.
The elevations of the original surface are taken from the lithographed geological profile, scale $\frac{1}{200,000}$, furnished by the New Panama Canal Co. and prepared from data of the Old Company.
The elevations of the present surface on the Atlantic Ocean Level and in the Culebra Cut are from Cross Sections made by the Commission in February, 1900. The additional excavation in the Culebra Cut, not on the axis of the Canal, is shown by the broken line.
The elevations of the present surface of other parts of the Canal line are from a Profile, scale $\frac{1}{200,000}$ published by the New Panama Canal Co. in 1899.

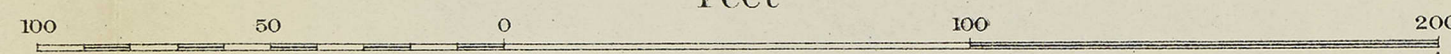


ISTHMIAN CANAL COMMISSION PANAMA ROUTE

CROSS SECTIONS

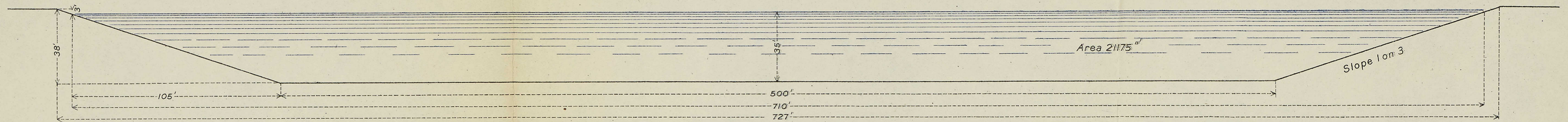
Scale $\frac{1}{500}$

Feet

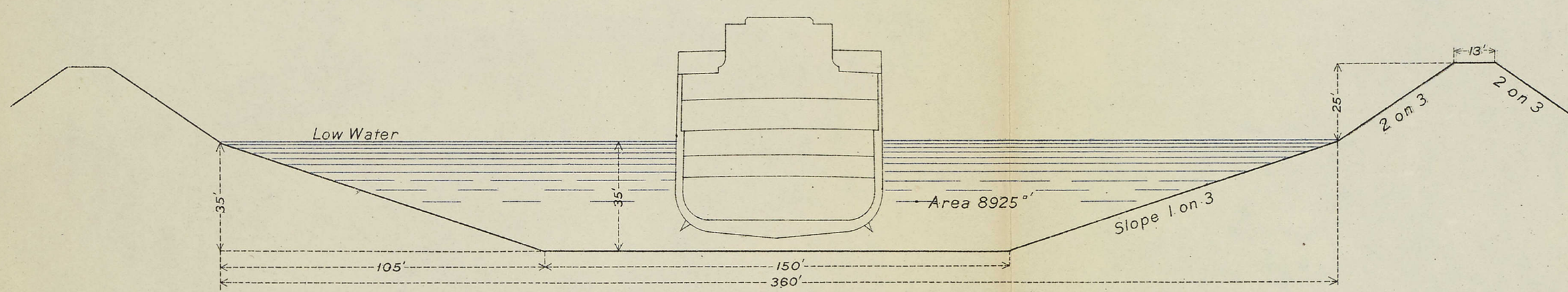


1901

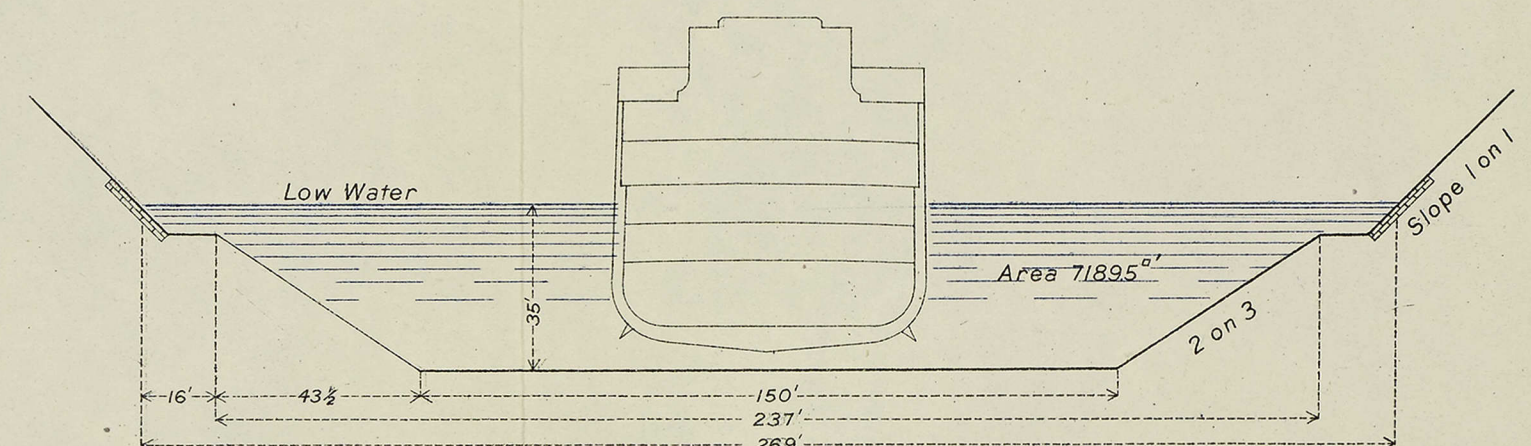
HARBOR AT COLON



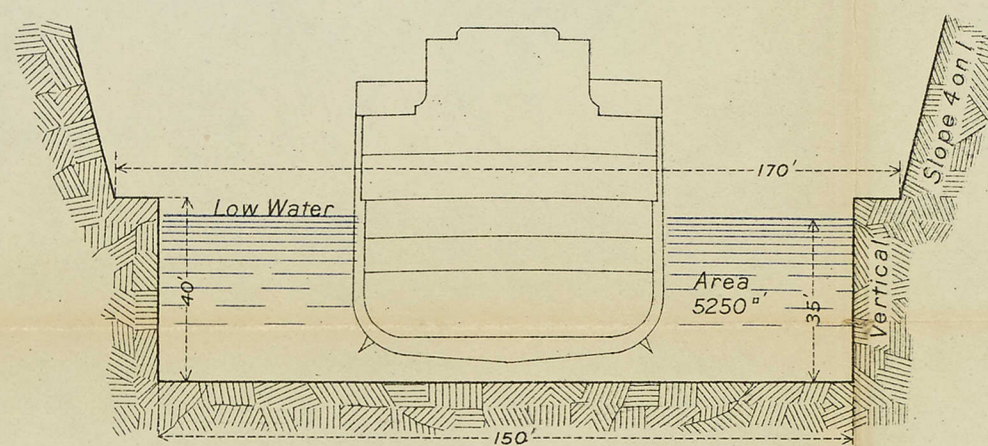
SILT



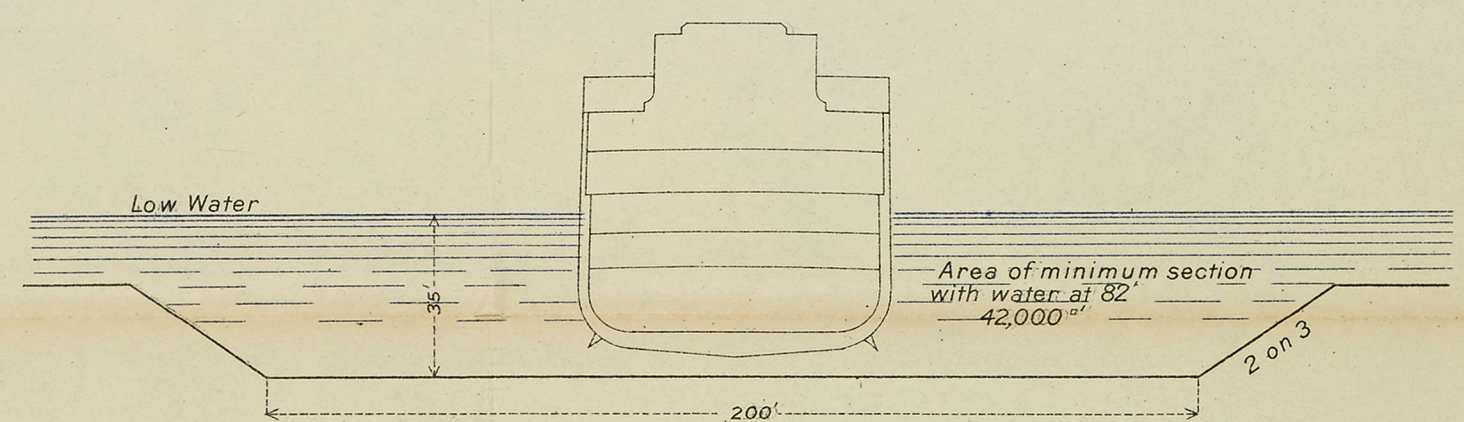
FIRM EARTH



ROCK



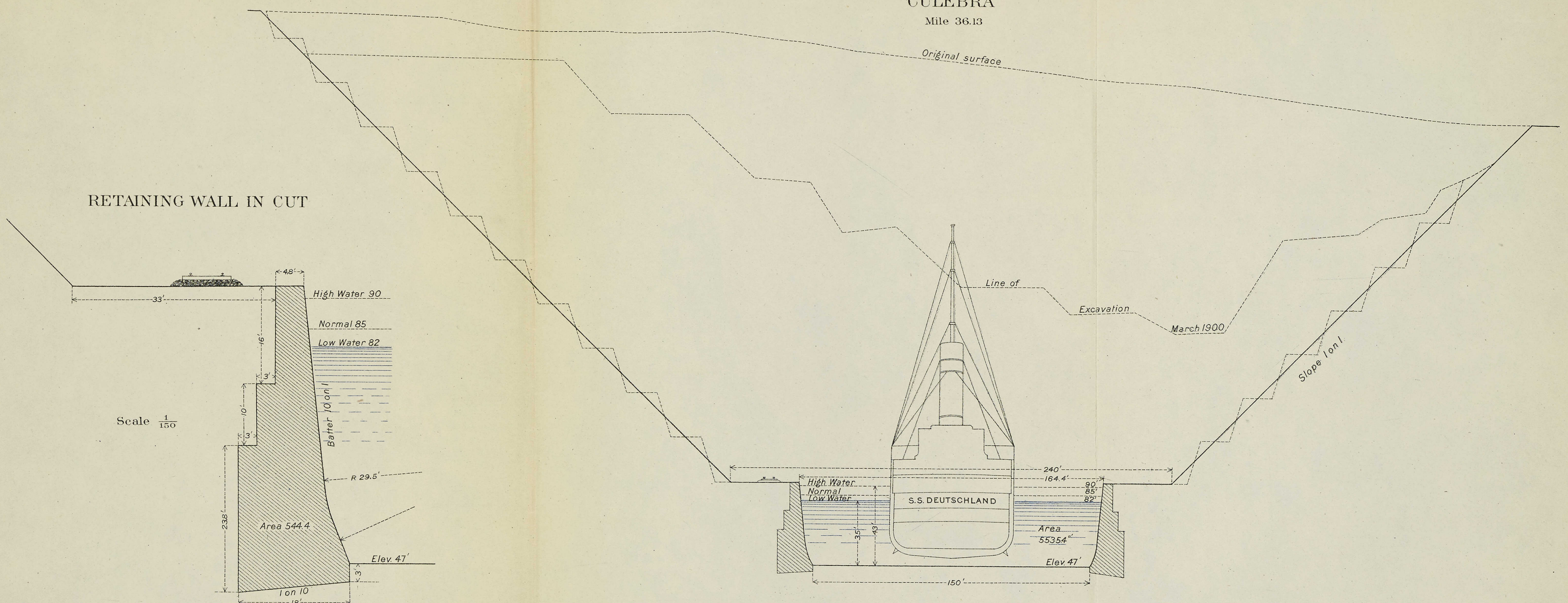
LAKE BOHIO
Submerged Section



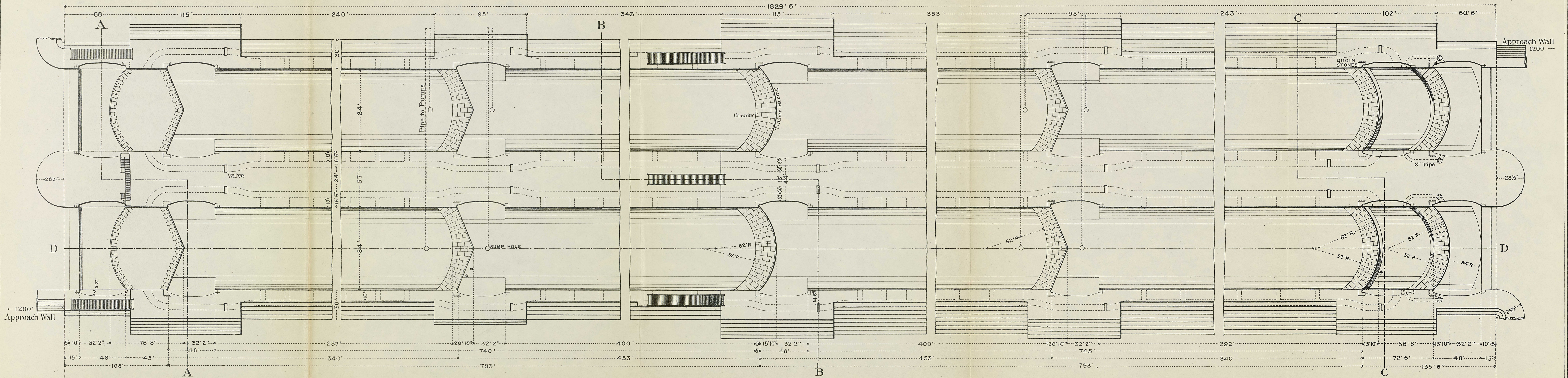
CULEBRA
Mile 36.13

RETAINING WALL IN CUT

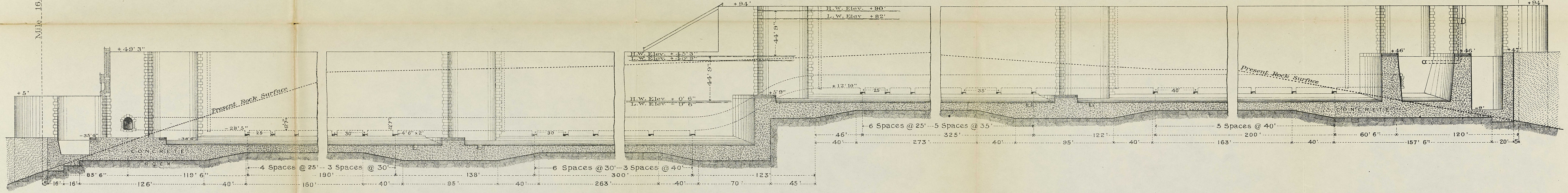
Scale $\frac{1}{150}$



PLAN



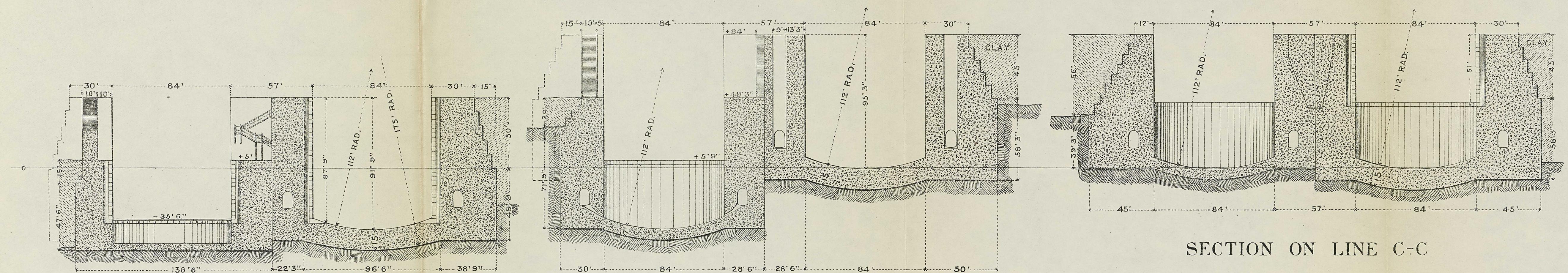
SECTION ON LINE D-D



SECTION ON LINE A-A

SECTION ON LINE B-B

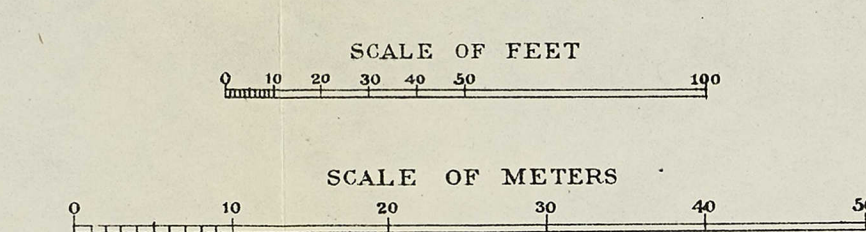
SECTION ON LINE C-C



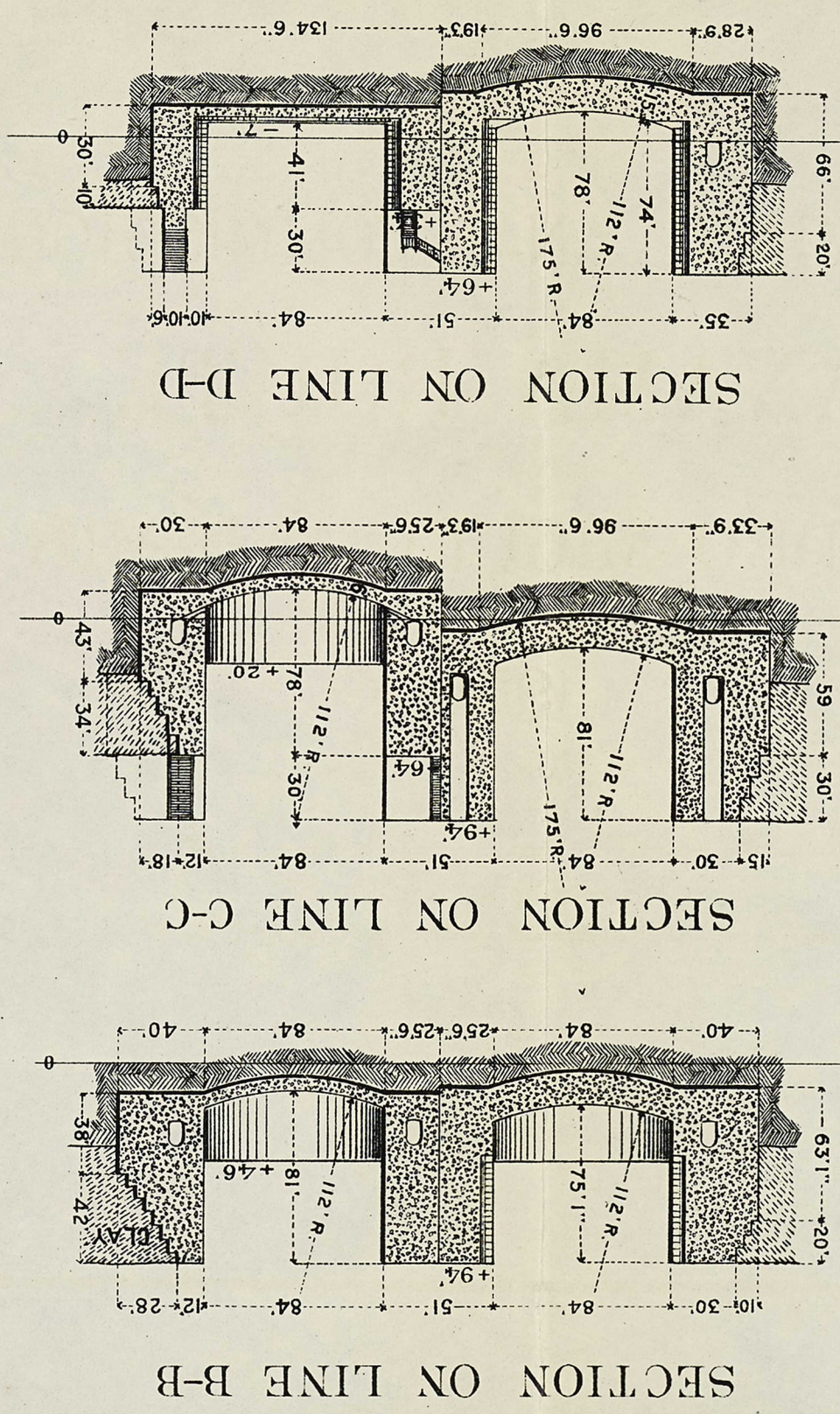
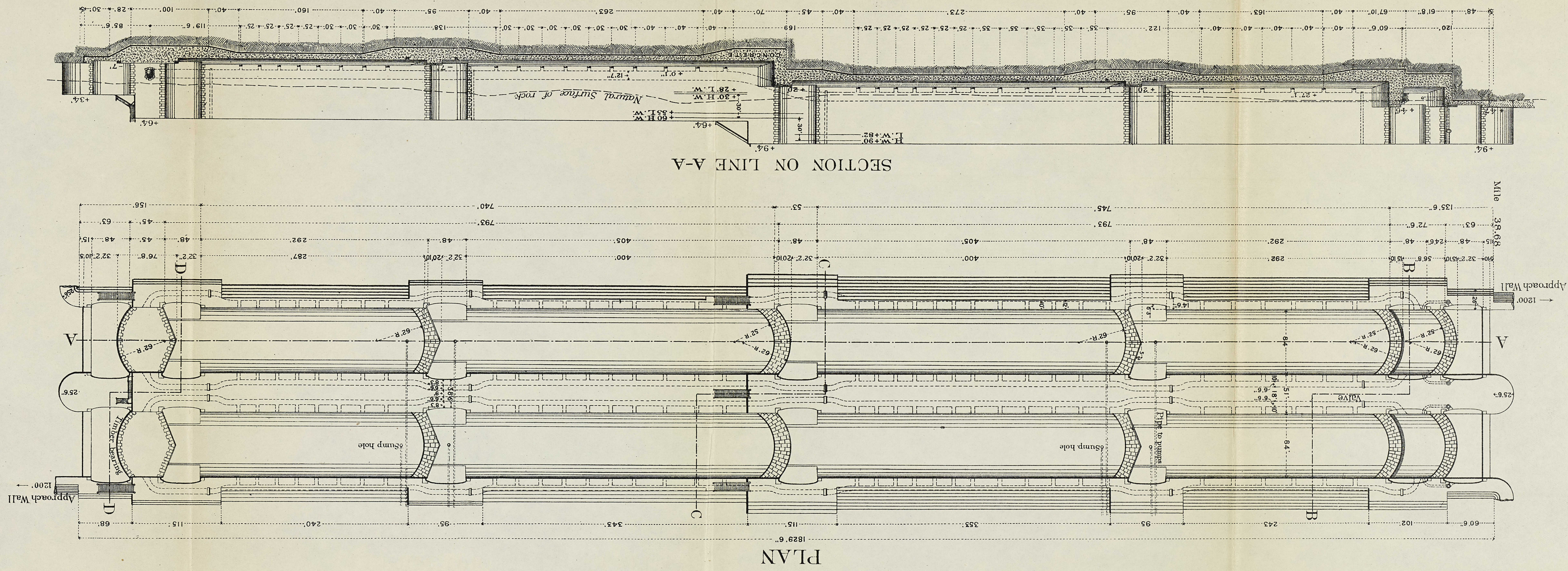
ISTHMIAN CANAL COMMISSION
PANAMA ROUTE
LOCKS AT BOHIO

1901

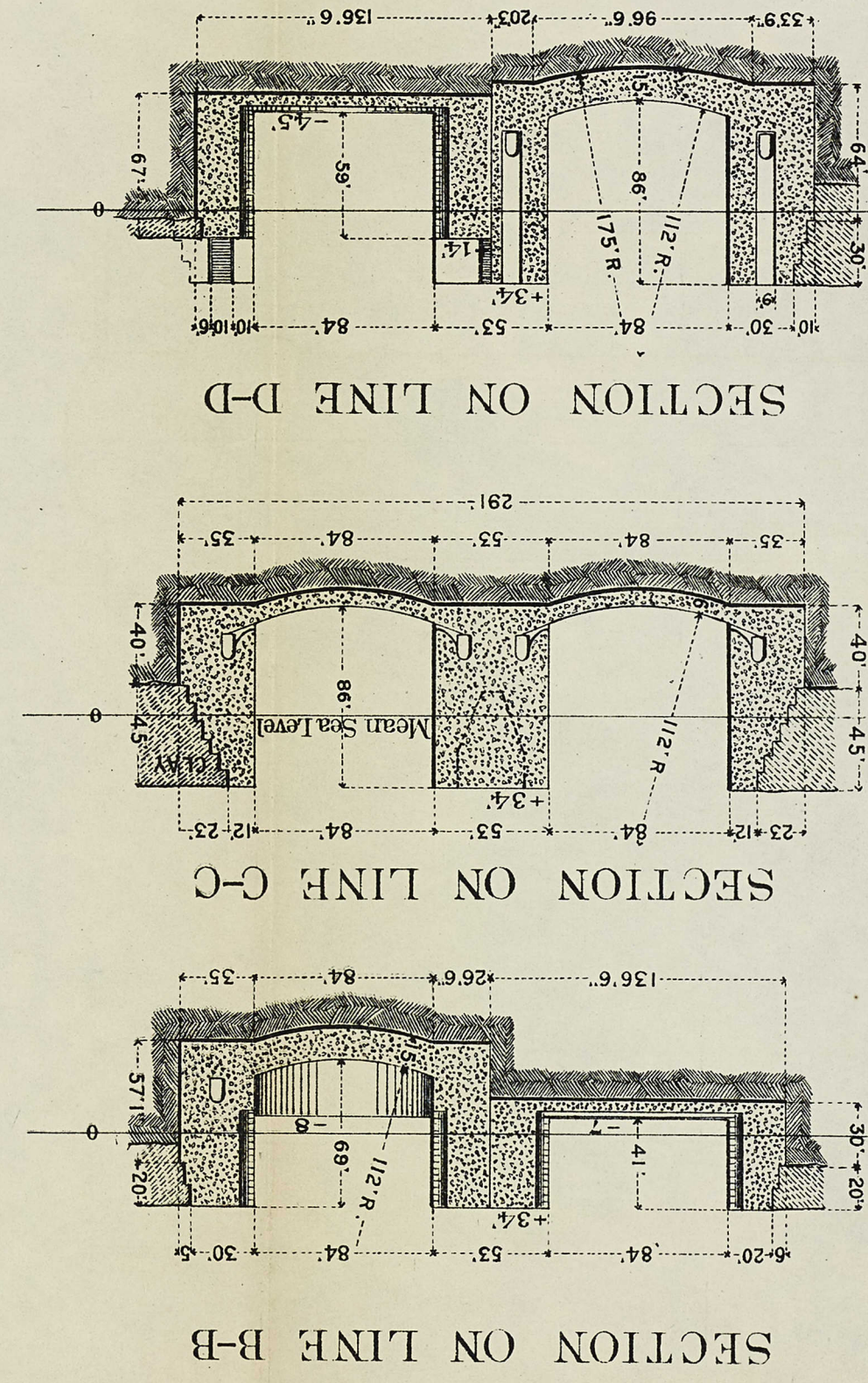
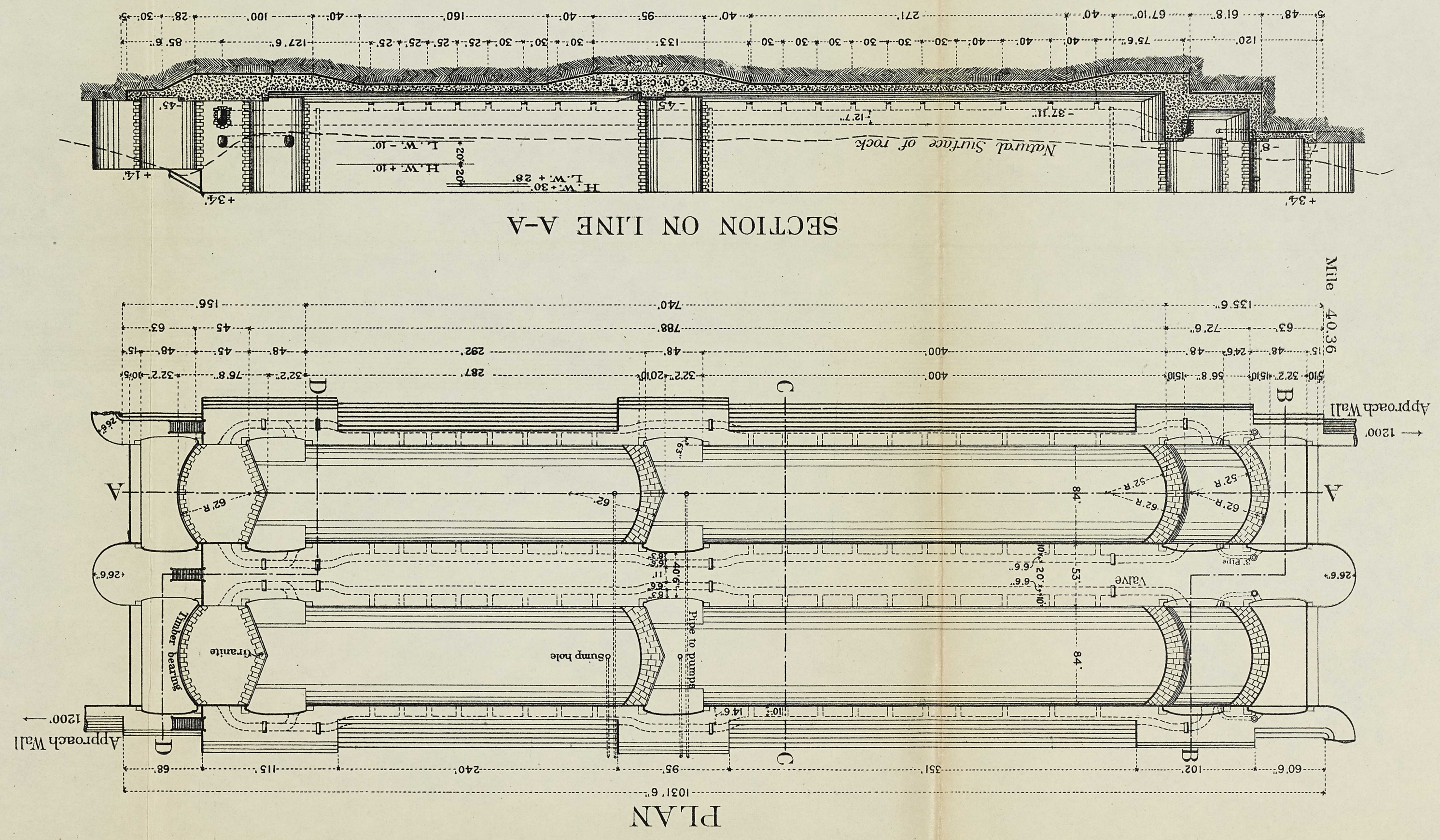
SCALE 1/480



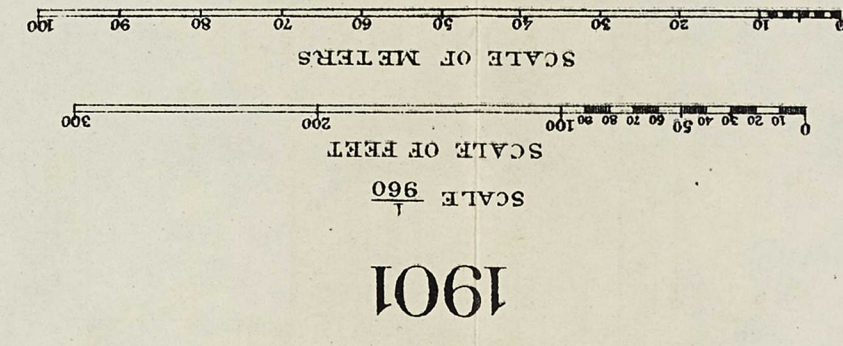
LOCKS AT PEDRO MIGUEL



LOCK AT MIRAFLORES



ISTHMIAN CANAL COMMISSION
PANAMA ROUTE
LOCKS AT PEDRO MIGUEL AND MIRAFLORES



SECTION ON LINE X-Y FIG.3



Scale $\frac{1}{5000}$

SCALE OF FEET

500 1000

SCALE OF METERS

100 300 500

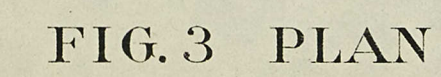
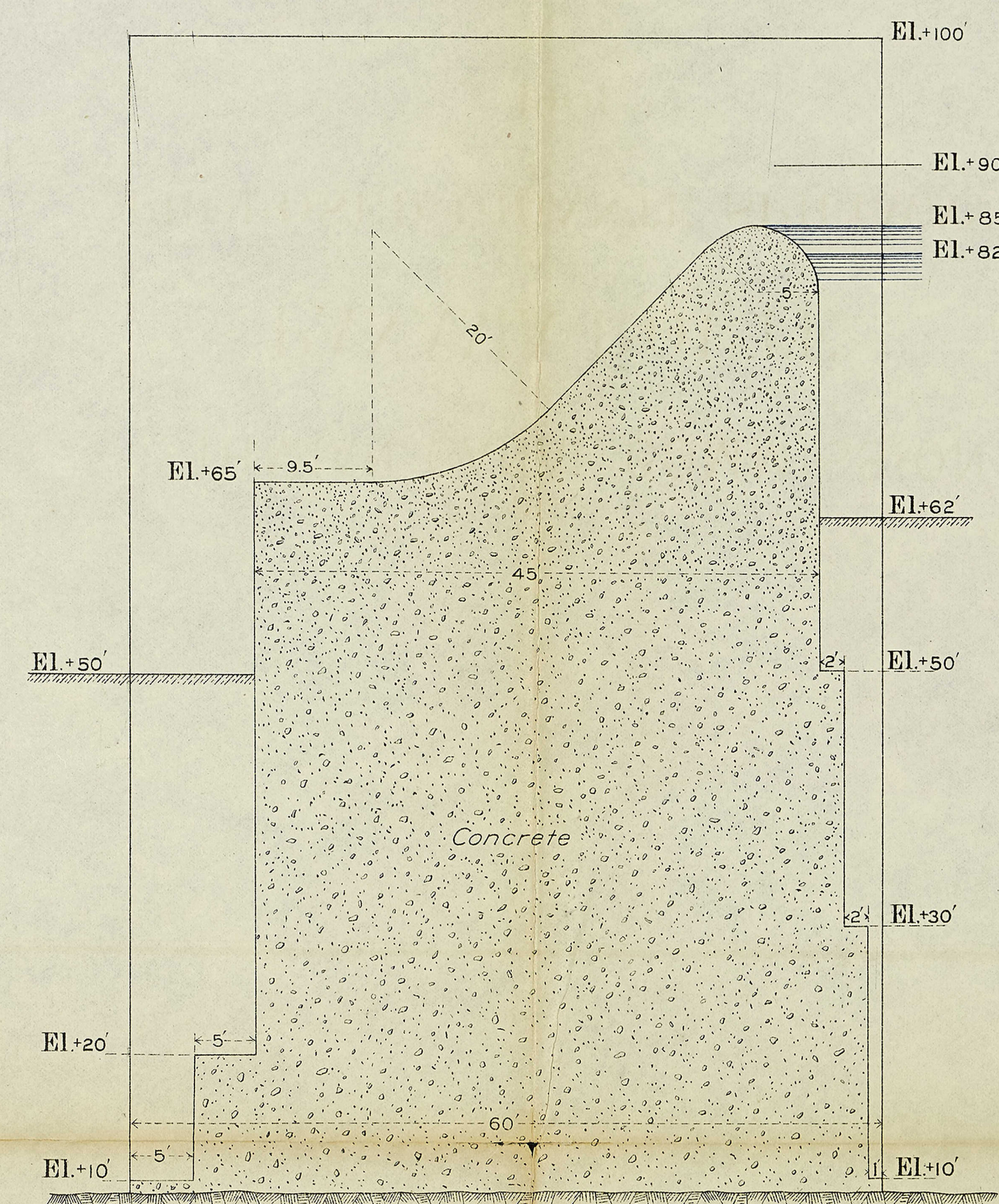


FIG. 3 ELEVATION

ISTHMIAN CANAL COMMISSION
PANAMA ROUTE
PROPOSED GIGANTE SPILLWAY
1901

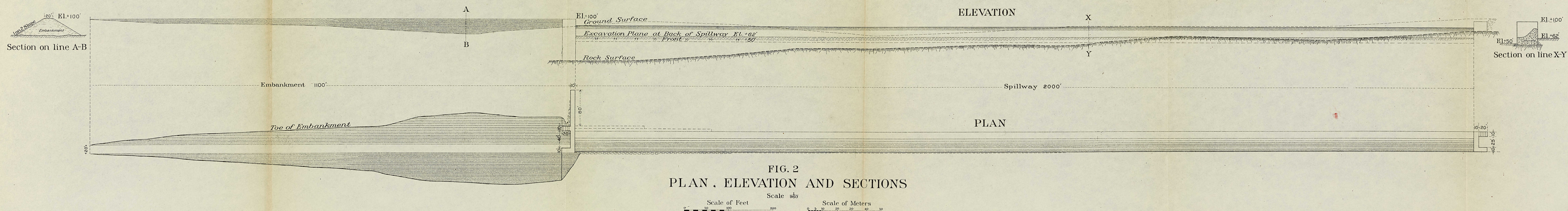


Enlarged Section
Scale 120
FEET
METERS



FIG. 1
LOCATION OF SPILLWAY

Scale 5000
SCALE OF FEET
0 500 1000
SCALE OF METERS
0 100 200 300



ISTHMIAN CANAL COMMISSION PANAMA ROUTE

MAP SHOWING LOCATION OF PROPOSED CANAL

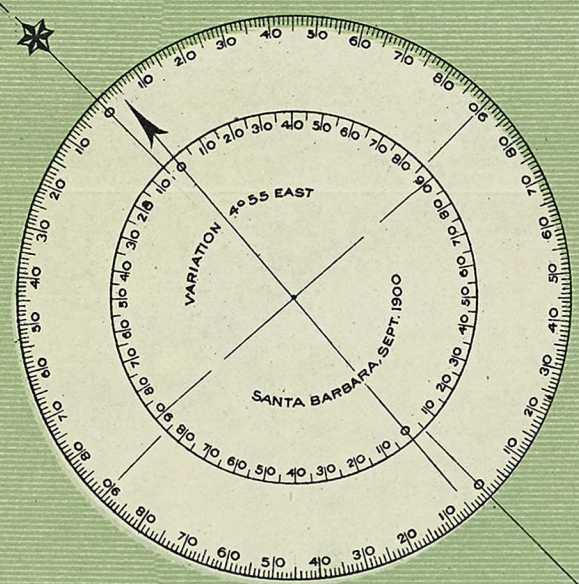
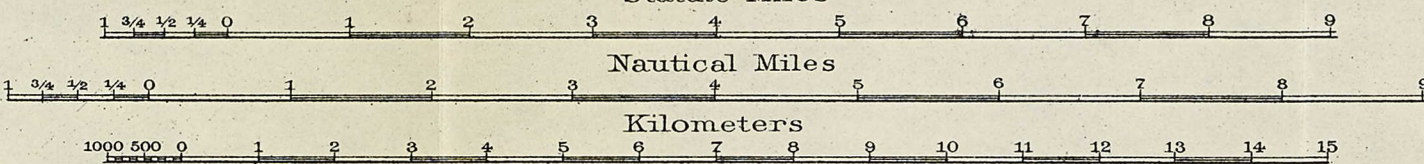
1901

Scale 1:100,000

Statute Miles

Nautical Miles

Kilometers



**ZONES OF
MEAN ANNUAL RAINFALL
IN INCHES.**

ARTHUR P. DAVIS,
CHIEF HYDROGRAPHER.

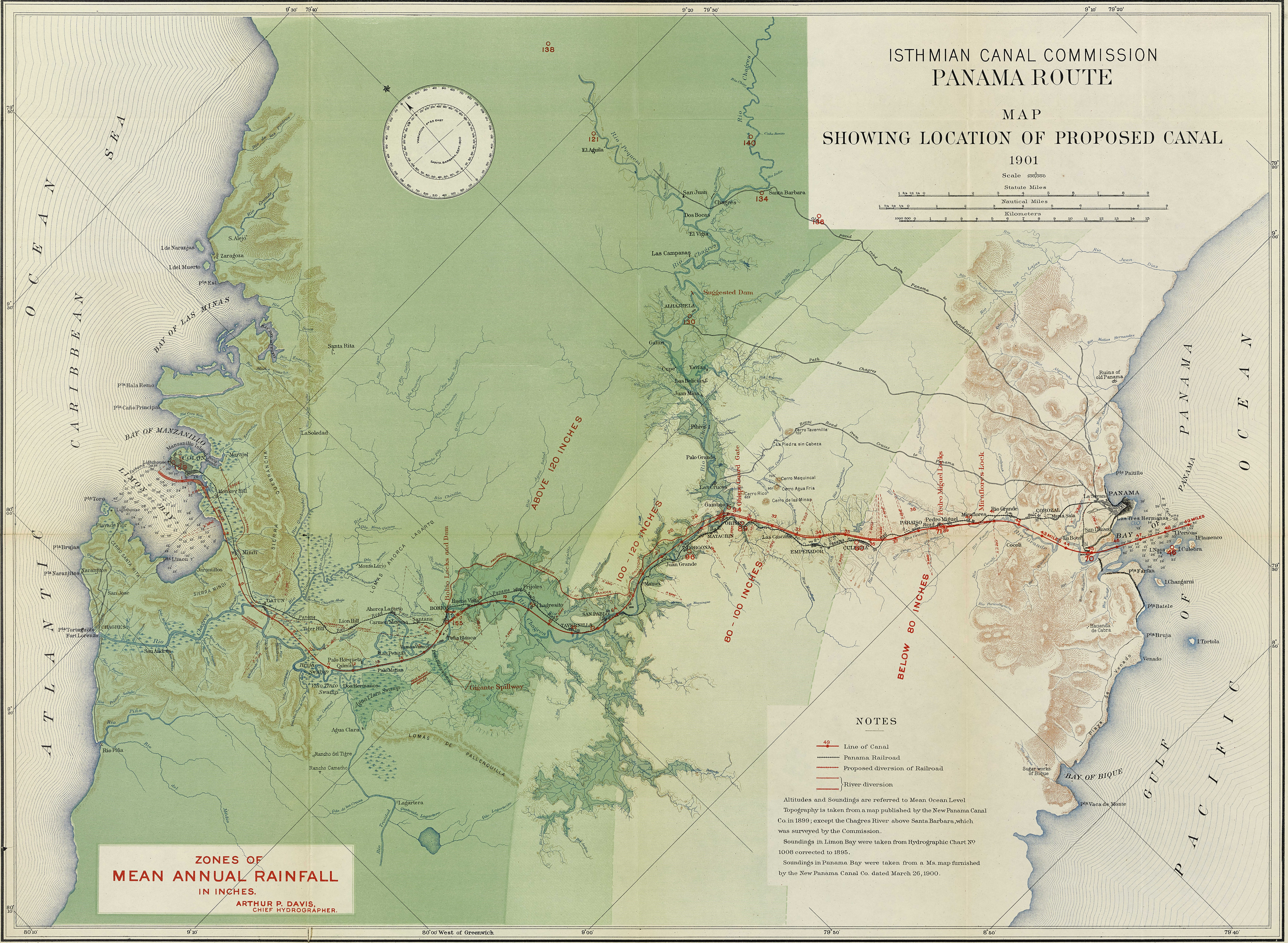
NOTES

- 49 — Line of Canal
- Panama Railroad
- Proposed diversion of Railroad
- River diversion

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1008 corrected to 1895.

Soundings in Panama Bay were taken from a Ms. map furnished
by the New Panama Canal Co. dated March 26, 1900.



67.
1905
9-
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