AMERICAN SOCIETY OF CIVIL ENGINEERS

33 WEST 39TH STREET, NEW YORK, N. Y.

PANAMA CANAL

The Sea-Level Project

AMERICAN SOCIETY OF CIVIL ENGINEERS FOUNDED 1852

A Symposium

From PROCEEDINGS, April, 1948

Price $1.00 Net

PROCEEDINGS

OF THE

AMERICAN SOCIETY OF CIVIL ENGINEERS

VOL. 74 APRIL, 1948 No. 4

TECHNICAL PAPERS

AND

DISCUSSIONS

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Published monthly, except July and August, at Prince and Lemon Streets, Lancaster, Pa., by the American Society of Civil Engineers. Editorial and General Offices at 33 West Thirty-ninth Street, New York 18, N. Y. Reprints from this publication may be made on condition that the full title of paper, name of author, page reference, and date of publication by the Society are given.

Entered as Second-Class Matter, September 23, 1937, at the Post Office at Lancaster, Pa., under the Act of March 3, 1879. Acceptance for mailing at special rate of postage provided for in Section 1103, Act of October 3, 1917, authorized on July 5, 1918.

Subscription (if entered before January 1) $8.00 per annum. Price $1.00 per copy

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* Publication of closing discussion pending.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

PAPERS

PANAMA CANAL—THE SEA-LEVEL PROJECT

A SYMPOSIUM

Note.—Written comments are invited for immediate publication; to insure publication the last discussion should be submitted by September 1, 1948.

THE FUTURE AND THE PANAMA CANAL

By James H. Stratton,[1] M. ASCE

Synopsis

Investigations under Public Law No. 280 (Seventy-ninth Congress, First Session, approved December 28, 1945) disclose that only an Isthmian sea-level canal will meet the future needs of interoceanic commerce and national defense. The broad phases of the studies leading to this conclusion are described in this paper. Various engineering features of the studies of the recommended sea-level canal are described more fully in the other papers of this Symposium.

The Canal Zone offers the most economic site for either a lock or a sea-level Isthmian canal. The present lock canal could be improved at a cost of $129,983,000 to meet the needs of commerce for the remainder of the twentieth century. A lock canal designed to meet the future needs of commerce and constructed to have the maximum security feasible in this type of canal would require new locks and strengthened summit lake impounding dams. It would cost $2,307,686,000 and would still be deficient in resistance to modern weapons.

A sea-level canal at Panama constructed by the conversion of the existing lock canal could not be destroyed by enemy attack or sabotage. Only the atomic bomb could cause significant interruption in service, and then for not more than a few weeks. Navigation would be practicable in the sea-level canal even though tidal currents were not regulated. Nevertheless tidal regulation would be provided for greater safety to shipping. For this purpose a tidal lock and a navigable pass would be provided. The opening of the gates of the navigable pass at selected Pacific tidal stages would permit the routine operation of the canal as an open waterway. In the event of damage to the tidal-regulating structures, the gates of the navigable pass could be removed quickly and the canal could be operated thereafter as an open waterway. Serious slides could be prevented by appropriate flattening of the slopes in cut. The entire excavation to effect the conversion to a sea-level canal would be completed in advance of conversion; the major part would be in the dry. Part of the wet excavation would require the use of special dredges working to a depth of 145 ft. Construction interference with canal traffic would be negligible except for a period of a few days when the summit lakes are lowered. The Panama sea-level canal would be constructed in 10 years and would cost $2,483,000,000.

Introduction

The long history of the search for a passage to the east, and of the early efforts to promote a canal across the Isthmus of the Americas, provide a background of romance and color to the story of actual construction which opened

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in 1882 with the initiation of work on a sea-level canal at Panama by the first French Canal Company. After years of effort and mounting adversities when funds had run low, the French changed to the less costly high-level lock canal. Because the French believed that a sea-level canal ultimately would be necessary, plans for the locks were drawn to facilitate the transformation. The chapter of the story dealing with the French construction ended with financial failure and the transfer of all rights and interest in the Panama Canal to the United States.

The opening chapter of the tale of construction of the Panama Canal by the United States is one of controversy over the type of canal to be built. The records disclose that the selection of the lock canal was based on the advantages it offered in the earlier completion at a lesser cost.[2] The issues entering into the

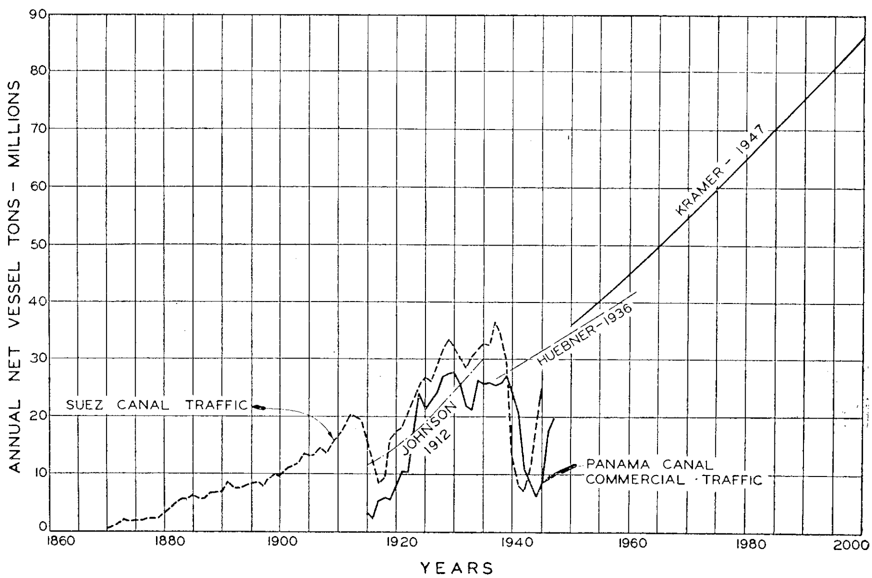


FIG. 1.—PANAMA CANAL TRAFFIC PREDICTIONS

selection have a particular interest now, in the face of the challenge presented by the "blockbuster" bomb, the guided missile, and the atomic bomb. When the present canal was planned the critical forms of attack were envisioned as naval gunfire directed against the locks and enemy forces moving overland to capture the canal intact.

The Seventy-ninth Congress expressed the temper of present concern for the security of the canal by passing Public Law No. 280 which was approved by President Harry S. Truman on December 28, 1945, and which provides:

> "Be it enacted by the Senate and the House of Representatives of the United States of America in Congress assembled, That the Governor of the Panama Canal, under the supervision of the Secretary of War, is hereby

[2] "Report of the Board of Consulting Engineers for the Panama Canal, 1906," U. S. Govt. Printing Office, Washington, D. C., 1906.

authorized and directed to make a comprehensive review and study, with approximate estimates of costs, of the means for increasing the capacity and security of the Panama Canal to meet future needs of interoceanic commerce and national defense, including restudy of the construction of additional facilities for the Panama Canal authorized by the Act approved August 11, 1939 (53 Stat. 1409). He shall also make such study without drafting plans or sketches as he may deem desirable to permit him to determine whether a canal or canals at other locations, including consideration of any new means of transporting ships across land, may be more useful to meet the future needs of interoceanic commerce or national defense than can the present canal with improvements. He shall report thereon to the Congress, through the Secretary of War and the President, not later than December 31, 1947."

Traffic History of the Canal

The volume of traffic through the Panama Canal has steadily increased since it was opened, despite the setbacks of wars and world depressions. With uninterrupted world prosperity a further growth of commercial traffic may be

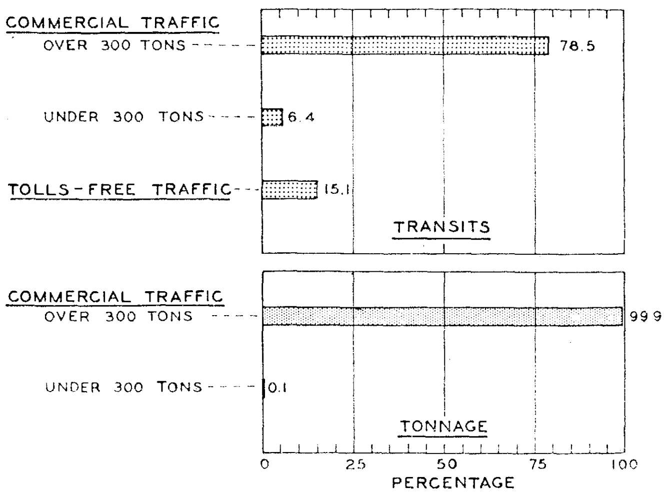


Fig. 2.—Predicted Daily Traffic

expected generally as predicted by Roland L. Kramer of the University of Pennsylvania at Philadelphia in recent studies which were made for use in the investigations under Public Law No. 280. Fig. 1 records past traffic and depicts the estimated and future commercial traffic, as projected by Professor Kramer.

Before the war the "tolls-free" traffic, consisting largely of vessels owned and operated by the United States, accounted for about 15% of the entire traffic. In 1945, the peak war traffic year, this rose to 78%. Predictions of future daily traffic are presented in Fig. 2. Since the canal was opened in 1914 it has transited 198,000 ships of which 142,000 were tolls-paying commercial craft. The percentage distribution of transits and tonnage is illustrated in Fig. 3.

War traffic through the canal in the period from 1941 to 1945 totaled nearly 17,000 transits. Had there been no canal during this period, it is estimated that the increased ship-operating costs and the cost of additional shipping and escort craft that would have been required to preserve the schedules made possible by the canal would have exceeded $1,500,000,000.

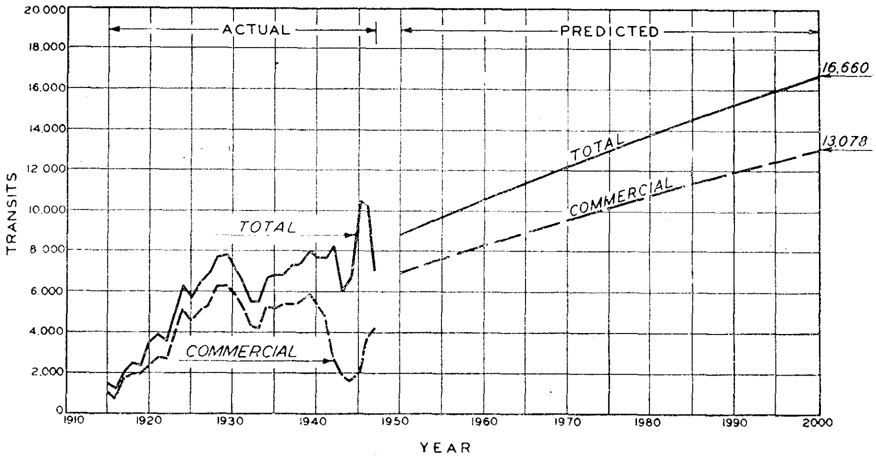


FIG. 3.—PANAMA CANAL TRANSITS

From its contribution in peace and war, it is clear that the future of the canal is the future of ships, provided the canal can be made safe against destruction. As Norman Padelford has stated:

> "* * * the Panama Canal is more than a commercial artery. * * * It is a focal point of national defense, a base of operations for the protection of the Hemisphere, an instrument of national influence."
>
> "* * * it seems safe to say that so long as any material part of the commodities of trade are carried in ships, and so long as sea power persists as a determining factor in the relationships of nations, so long certainly will use of the Panama Canal be sought by merchantmen and vessels of war. And so long will the Canal as a waterway remain essential to the United States. * * * No Panama Canal would exist today to pass great ships from ocean to ocean had it nor been for vision which saw beyond the limitations of existent realities. The hope for a more ordered future * * * lies in similarly transforming present difficulties through enlightened leadership and continued vision." [3]

THE PANAMA CANAL AS IT IS TODAY

The canal as a waterway has changed little since it was placed in service, except for the addition of the Madden Dam on the Chagres River in 1935 for the benefit of water supply for lockages, flood control, and power generation.

A plan and profile of the existing canal are shown in Figs. 4 and 5. It is estimated that by about 1960 the capacity of the canal will be inadequate to accommodate traffic without inflicting undesirable delays on peak traffic days.

[3] "The Panama Canal in Peace and War," by Norman J. Padelford, The Macmillan Co., New York, N. Y., 1942.

The delays thereafter will become more serious with the further growth of traffic unless additional capacity is provided.[4,5]

The restricting effect of the small locks (width 110 ft, length 1,000 ft) of the present canal on the design of Navy ships became intolerable with the approach of war and, in 1939, Congress directed the construction of a third set of locks, 140 ft wide and 1,200 ft long, which it was then thought would be adequate for all future needs. The new locks were designed to resist attack by

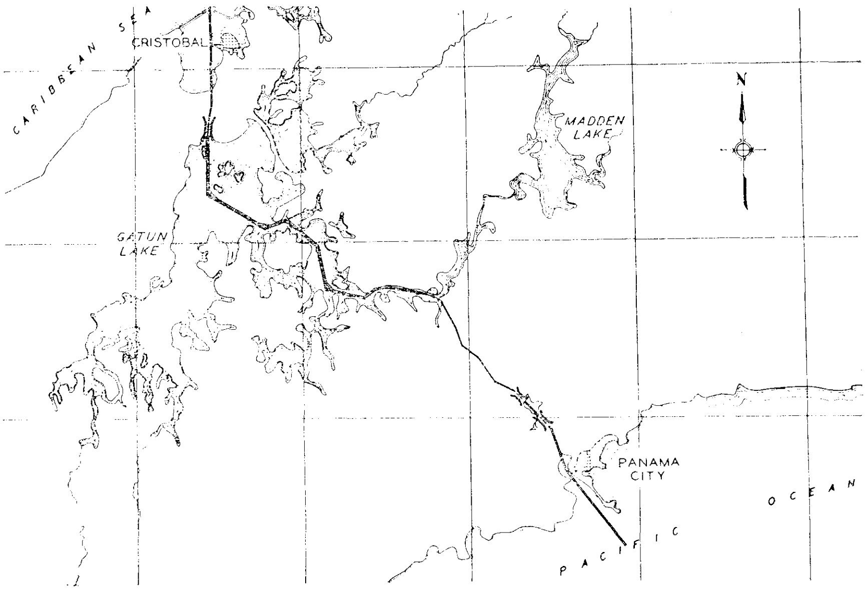


FIG. 4.—GENERAL PLAN, PANAMA LOCK CANAL

the largest aerial bomb then known to exist. Construction was suspended early in 1942, when it became apparent that the new locks could not be completed before the end of the war because of conflicting demands for men and materials. At that time, excavation for the Gatun and Miraflores Third Locks had been substantially completed, but excavation for the third lock at Pedro Miguel and work on actual lock construction had not commenced. Of the authorized expenditure of $277,000,000, approximately $75,250,000 was spent.

CAPACITY LIMITATIONS OF PRESENT CANAL

The present locks are expected to be adequate dimensionally for all commercial shipping for the remainder of the twentieth century except for ships of the "Queen" class which do not, and ordinarily would not, use the sea route through the canal. The limiting effect of lock size on the passage of naval ships is expected to become even more stringent in the future than it is at present.

[4] "The Isthmian Canal Situation," by Hans Kramer, *Transactions*, ASCE, Vol. 94, 1930, p. 406.

[5] "Additional Lock Facilities for Panama Canal, 1939," *House Report No. 494*, 76th Cong., 1st Session, 1939.

The individual locks of the present canal periodically are taken out of service a lane at a time for overhaul, thus the dependable or firm capacity of the canal is not that established by the twin lanes of locks but is that established by the single lane locks available in the extended periods of overhaul. Occasional night fogs require closing Gaillard Cut to traffic after midnight in the rainy season. The locks are overhauled in the dry season when round-the-clock operation of the canal is feasible, thereby avoiding the reducing effects of both lock overhaul and fog on the canal capacity.

Future Traffic Needs

The commercial canal tonnage predictions of Professor Kramer were transformed into expected future ship transits by the engineering staff employed on the studies by taking into account the trends in ship sizes, the expected character of future cargoes, and the expected proportion of transits under full and partial load, and in ballast, as evidenced by past experience. Thus, the transportation of 87,770,000 long tons of commercial cargo estimated by Professor Kramer for the year 2000 would require 13,078 ship transits. In the year 2000, commercial and tolls-free traffic would average 46 transits daily. On peak traffic days 69 transits could be expected. The year 2000 was selected to establish the period during which future traffic needs must be met if any major construction or reconstruction is undertaken.

By locking small ships in tandem and taking into account the sizes of ships estimated for the future, the peak load of 69 ships could be realized with 46 lockages using the present locks. With locks 200 ft wide and 1,500 ft long (which is the size now recommended by the Navy to meet its future needs), it would take 29 lockages to effect the passage of the 69 ships.

How Secure Is The Present Canal?

Although no one can say what course World War II would have taken had the Japanese followed up Pearl Harbor (Hawaii) with an attack on the Panama Canal, it is now clear that the locks could have been destroyed and the canal emptied into the sea had an attack been made and the defenses penetrated. The development of larger bombs and new weapons of both conventional and atomic types since Pearl Harbor leaves no doubt as to the vulnerability of the canal to enemy attack and sabotage. If the needs of national defense are to be met, the canal must be made secure against attack and sabotage.

New measures offering effective resistance to the penetration of defenses by rockets and guided missiles have not kept pace with the development of offensive weapons and indeed the prospects, for the future, of the defense are so gloomy it appears that the historic pattern of war may never be restored. The advantage now lies entirely with the aggressor who undertakes to destroy his enemy's ability to wage war by the wholesale destruction of his population and of his instruments of war before he can employ them. Thus, to live as a nation the United States must shield its own vital weapons so that it can take countermeasures to prevent exploitation of any initial advantages the enemy may gain by surprise attack. Prudence requires also that communications,

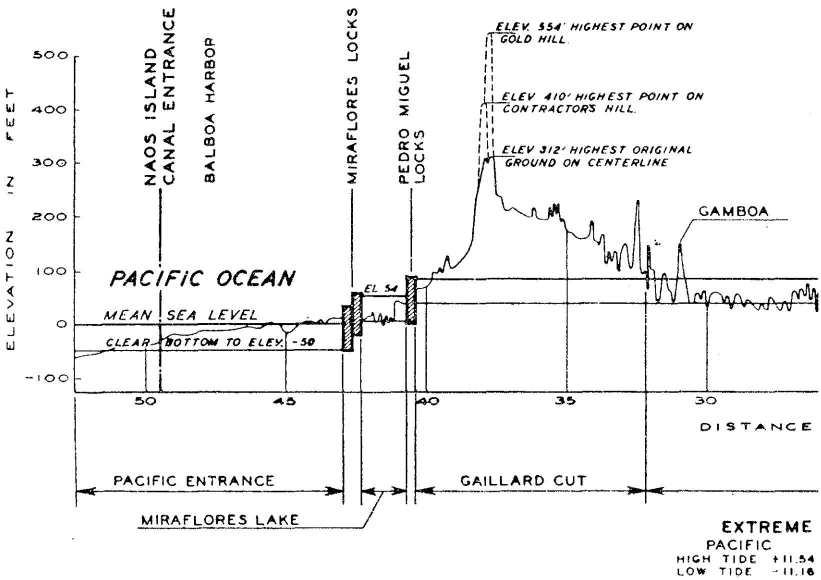


FIG. 5.—PROFILE,

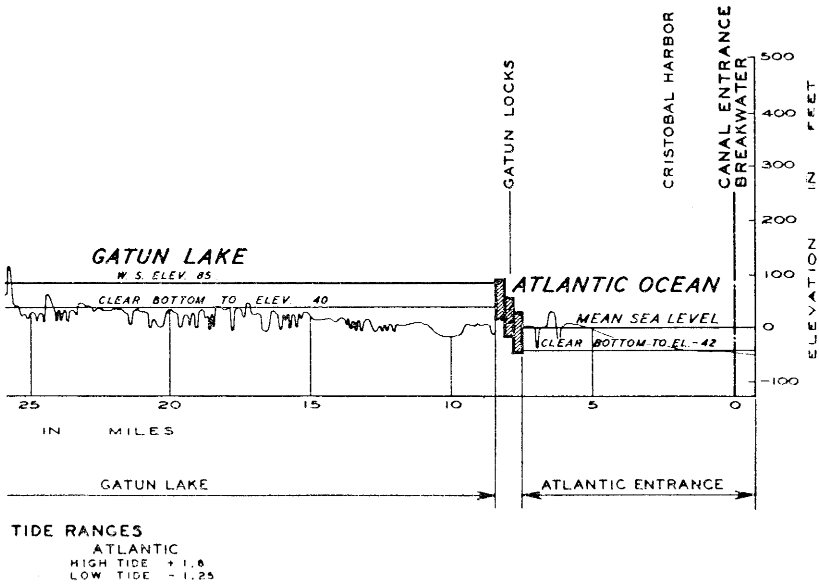
such as the Panama Canal, and essential war industries be planned so that the nation would not be hopelessly crippled if attacked.

The penetration of canal defenses by rockets, guided missles, or robot planes loaded with powerful conventional or atomic explosives launched from the air, from ship or submarine at sea, or from a land base, must be accepted as a possibility. A single sneak attack could destroy the lock gates of the present canal and drain Gatun Lake to the sea.

If the needs of national defense are to be met, steps must be taken to make the canal secure. No matter how resistant to attack the canal is made, however, an alert and modern defense is needed to warn the enemy that an attack will be costly in men and material, to prevent its capture, and to keep the enemy from having a free hand in sinking ships in transit and in making havoc of the auxiliary facilities of the canal.

Third Locks Project in Review

A new third set of locks to accommodate large naval vessels, no matter how strongly constructed to resist attack and sabotage, would not add to the overall security of the canal because the existing locks and impounding dams cannot be strengthened sufficiently to give them equivalent protection. If any lock is breached and Miraflores Lake or Gatun Lake is lost, the canal would be closed for months or even years for repairs and for the restoration of the lost lake. In the light of the threat of present weapons, the Third Locks Project would provide for only the peacetime needs of commerce, which, if that were the sole consideration, could be met by other means at a considerably lesser cost.



PANAMA LOCK CANAL

IMPROVEMENTS IN THE INTERESTS OF COMMERCE ONLY

The requirements of interoceanic commerce for the remainder of the twentieth century could be met by the elimination of lay-up of the locks for overhaul and by overcoming fog interference with traffic. The first of these is the more restrictive on the capacity of the canal, reducing it from 58 ships to 36 ships per day. Each lock is now overhauled every 4 years, repairs being undertaken every 2 years, alternating between the Atlantic (Gatun Locks) and the Pacific Locks (Miraflores Locks and Pedro Miguel Locks). While one lane of locks is under repair, the adjacent lane of locks is kept open to traffic; thus the availability of only a single operating lane of locks at one end or the other of the canal, during the period of overhaul (about 4 months), establishes the dependable canal capacity. Repairs are made in the dry season when there is no fog, and the canal is then operated 24 hours daily instead of 16 hours per day as is the case in the rainy season when fogs are of relatively frequent occurrence after midnight. Round-the-clock operation and careful scheduling of transits make it possible to hold the reduction in capacity due to overhaul of the locks to less than one half of the normal operating capacity of the canal.

Repairs to the lock gates and their mountings, and to the operating machinery and the filling culvert valves and fittings, are undertaken with the lock chamber dewatered, as are the cleaning and painting of all underwater metal parts. The lay-up and dewatering of the locks for repairs could be eliminated by providing new gate mountings of a special type and new type lock gates having buoyancy chambers to float them out and into position, thus effecting replacements in the matter of a few hours. Repairs to gates would be made in dry dock. Alterations to avoid lay-up of the locks for overhaul and certain channel improvements would raise the dependable capacity of the canal to 65 ships per day.

Fogs of the ground-radiation type which occur in Gaillard Cut in the rainy season (generally from May through December between midnight and daybreak) require closing the canal to traffic during these hours. Various methods of dispersing fog have been employed for the clearing of airfields, but present indications are that they would be too expensive for dispersing fogs in the canal. Developments in electronic aids to navigation offer the prospect of a cheap and fully reliable method of passing ships through a fogbound restricted channel. By the time traffic demands require 24-hour operation of the canal, these devices will be considerably improved and undoubtedly can then be adapted to the canal needs to increase its dependable capacity to 70 ships, which would be adequate for the remainder of the twentieth century.

The cost of the various improvements to extend the life of the present canal in the interests of commerce only would be $129,983,000. Any expenditure in excess of this amount can be justified only by the requirements of national defense.

A New Lock Canal For Security

Although it became apparent early in the studies that an increase in security to meet the needs of national defense could not be attained through the reinforcement of the locks of the present canal, this conclusion of itself did not eliminate the possibility of its attainment by the complete reconstruction of the existing lock canal or by the construction of a new lock canal elsewhere.

A survey of available reports of previous explorations and investigations and the current studies revealed thirty lock canal routes, of which many are merely alternate alinements of well-known routes:[6]

Route	Description	
1	Tehuantepec	
	Nicaragua, via Lake Nicaragua—	
2	Greytown-Fonseca Bay	Via Lake Managua
3	Greytown-Realejo	Via Lake Managua
4	Greytown-Tamarindo	Via Lake Managua
5	Greytown-Brito	
6	Greytown-San Juan del Sur	
7	Greytown-Salinas Bay	
	Nicaragua—	
8	Greytown-Salinas Bay	
	Panama—	
9	Chiriqui	
10	Chorrera-Lagarto	Alternate sea-level routes, Canal Zone and vicinity
11	Chorrera-Limon Bay	Alternate sea-level routes, Canal Zone and vicinity
12	Chorrera-Gatun	Alternate sea-level routes, Canal Zone and vicinity

[6] "U. S. Army Interoceanic Canal Report on Panama Canal and Nicaragua Canal, Sultan Report, 1932," *House Document No. 139*, 72d Cong., 1st Session, 1932.

Route	Description
13........	Panama parallel
14........	Panama sea-level conversion
15........	Panama Canal, route of the present lock canal
16........	San Blas
17........	Sasardi-Morti } Caledonia Bay routes
18........	Aglaseniqua-Asnati } Caledonia Bay routes
19........	Caledonia-Sucubti } Caledonia Bay routes
	Panama and Columbia; Tuyra River Routes—
20........	Tupisa-Tiati-Acanti
21........	Arquia-Paya-Tuyra
22........	Tanela-Pucro-Tuyra
23........	Atrato-Cacarica-Tuyra
24........	Atrato-Peranchita-Tuyra
	Colombia; Atrato River Routes—
25........	Atrato-Truando
26........	Atrato-Napipi
27........	Atrato-Napipi-Doguado
28........	Atrato-Bojaya
29........	Atrato-Baudo
30........	Atrato-San Juan

Excavation estimates based on existing maps, supplemented in some cases by field and air reconnaissance, resulted in narrowing the choice of routes for a lock canal to the eight listed in Table 1 and shown in Fig. 6.

TABLE 1.—COMPARISON OF LOCK CANAL ROUTES

ROUTE[a]		Length (miles)	Elevation of divide at crossing[b]	Channel excavation[c] (million cu yd)	Elevation of summit lake[b]	Lock lifts per lane[d]	Approximate cost (million dollars)	Action
No.	Place							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Tehuantepec	165	812	3,360	550	10	13,280	Eliminated because of higher cost.[e]
5	Nicaragua..	173	153	1,060	110	2	3,566	Eliminated on the same basis as route 1.[e]
9	Chiriqui....	55	5,000	....[f]	....[f]	....[f]	....[f]	Eliminated because of height of divide.[g]
15	Panama....	51	340	191	92	2	2,308	Retained for final study.
16	San Blas....	40	1,100	1,480	110	2	5,960	Eliminated because of excessive cost.
19	Caledonia...	63	1,100	1,110	110	2	4,751	Eliminated because of excessive cost.
23	Tuyra River	135	470	1,120	110	2	....[h]	Eliminated because of excessive cost.[g]
25	Atrato River	95	932	1,450	177	3	....[h]	Eliminated because of excessive cost.[g]

[a] See Fig. 6. [b] In feet above mean sea level. [c] Channels are 500 ft wide at a depth of 40 ft and 55 ft deep below low water level. [d] Two lanes of locks in each route, the lock chambers being 200 ft wide, 1,500 ft long, and 50 ft deep. [e] After full consideration of shipping benefits from shortening trade routes. [f] Details not developed. [g] Excessive excavation. [h] Not estimated.

None of the seven lock canal routes at locations remote from Panama offers compensating advantages in additional security or in savings through the shortening of trade routes over the Panama lock canal route.

The plan of development used in the comparative lock canal studies was the same in each case and is as described in the next section for a modernized lock canal at Panama having the maximum security feasible in this type of canal.

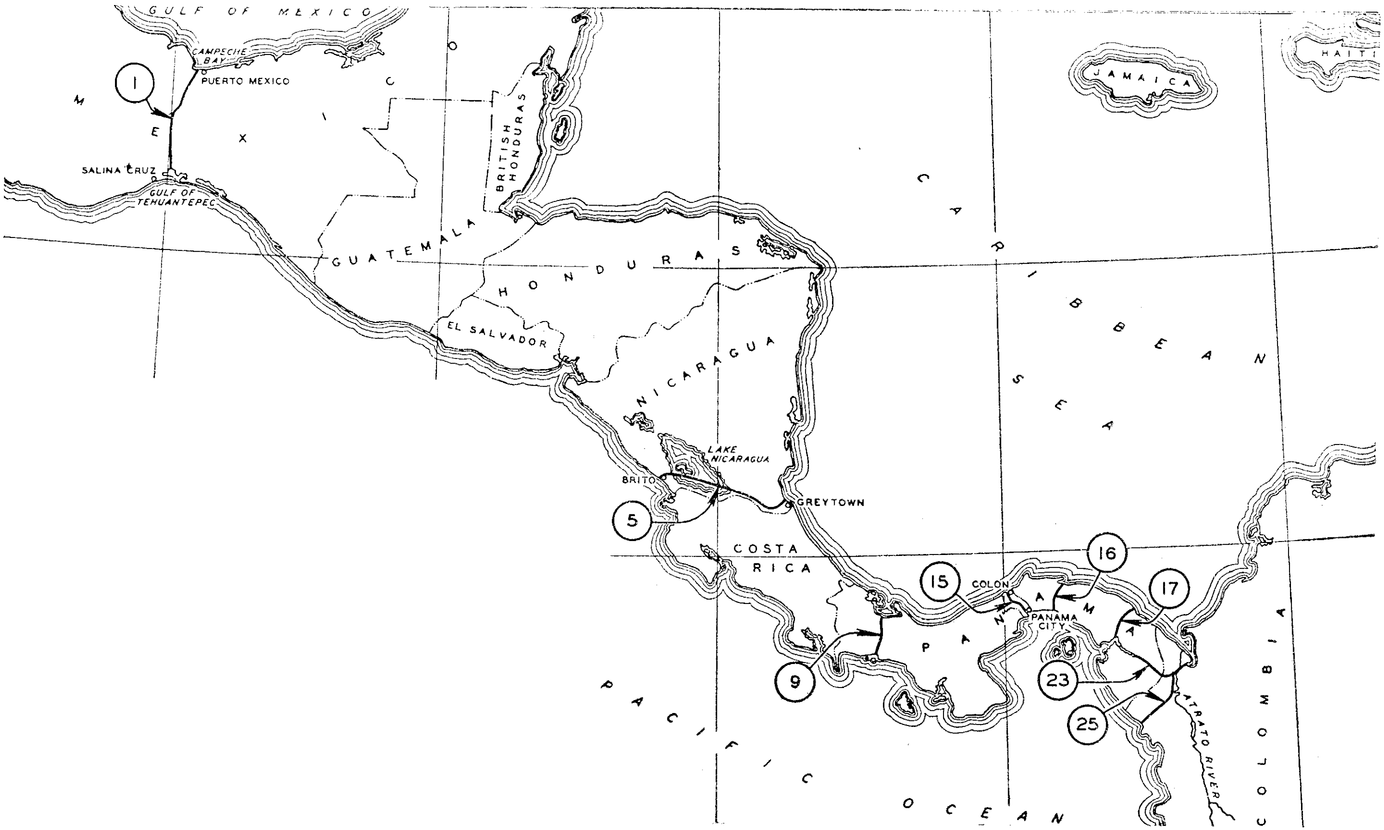


FIG. 6.—TRANSISTHMIAN CANAL ROUTES

A Modernized Panama Lock Canal

A reconstructed lock canal with all Pacific locks grouped at Miraflores to provide full lift to summit level would offer the greatest economy of construction and would create an anchorage area at the head of the Pacific locks that would facilitate the dispatch of vessels through Gaillard Cut. A similar arrangement of the Pacific locks was proposed by Adolphe Godín de Lépinay in 1879 and was advocated by the late W. L. Sibert, M. ASCE, in 1908. Colonel Sibert's suggestion was rejected because of the advanced state of the planning and construction, and doubts as to the foundations for three-lift locks at Miraflores, the site he proposed for the Pacific locks. The Miraflores foundations have since been found satisfactory for locks with full lift to summit level.

Two dispersed locks would be provided at Gatun and Miraflores with two lifts to attain summit level instead of three as at present. Chambers would be 200 ft wide, 1,500 ft long, and 50 ft over the sill. By raising Gatun Lake to El. 92, from its present maximum El. 87, additional storage for lockage water would be provided. This augmented water supply would not meet lockage demands until the year 2000 and supplementary pumping from the sea eventually would be necessary.

The dispersion of the locks and their armoring with concrete and steel to protect the lock machinery and the culverts would provide them with the highest practicable degree of protection. The lock gates do not lend themselves to protective treatment, except against the lightest type of aerial bomb. However, multiple sets of gates, well dispersed, would increase the difficulties of dissipating Gatun Lake. Certain of the gates would be housed in protected recesses when not in use. The closure dams adjoining the locks at Gatun and Miraflores would be of massive earth construction. The Gatun Dam spillway would be channeled in the rock abutment for maximum protection.

The cost of such a fully modernized lock canal at Panama would be $2,307,686,000.

Security considerations restrict public evaluation of the protective designs in relation to the various weapons that could be employed in an attack. It can be stated, however, that the lock canal cannot be made resistant either to atomic bombs or to modern conventional weapons. At best the protection that could be provided a lock canal would only increase the difficulties rendering it useless. The modernized locks could be breached by a determined enemy and the canal could thus be closed to traffic for the period required for reconstruction and for the capture of the tributary runoff to restore the summit lakes, which might require as much as 4 years. The extent of damage and the length of the period of traffic interruption would depend on the nature of the weapon employed and the intensity of attack. Radioactive contamination would make repairs to the locks difficult if not impossible. The lock type of canal, no matter how strongly constructed, would not increase security to meet the needs of national defense.

Sea-Level Canal Possibilities

The various sea-level canal route possibilities were narrowed down, in the same manner as were the lock canal routes, to the eight listed in Table 2, which includes the principal features, quantities of excavation, and the estimates of cost. The Panama route (Fig. 7) is the least costly and has the additional advantages of an operating and administrative establishment and of defenses

TABLE 2.—COMPARISON OF SEA-LEVEL CANAL ROUTES

Route[a] No. (1)	Route[a] Place (2)	Length (miles) (3)	Elevation of divide at crossing[b] (4)	Channel excavation[c] (million cu yd) (5)	Approximate cost[d] (million dollars) (6)	Action
1	Tehuantepec....	165	812	6,130	....[e]	Eliminated because of excessive cost.[f]
5	Nicaragua......	168	760	5,200	....[e]	Eliminated because of excessive cost.[f]
9	Chiriqui.......	55	5,000	68,800	....[e]	Eliminated because of height of divide.[f]
15	Panama........	46	410	1,069	2,483	Retained for final study.
16	San Blas.......	40	1,100	2,080	6,272	Eliminated because of excessive cost.
19	Caledonia......	59	1,100	1,880	5,132	Eliminated because of excessive cost.
23	Tuyra River....	135	470	2,140	....[e]	Eliminated because of excessive cost.[f]
25	Atrato River ...	95	932	1,810	4,594	Eliminated because of excessive cost.

[a] See Fig. 6. [b] In feet above mean sea level. [c] Channels are 600 ft wide at a depth of 40 ft and 60 ft deep below low water level. [d] Exclusive of tidal regulating structures, except at Panama. [e] Not estimated. [f] Excessive excavation.

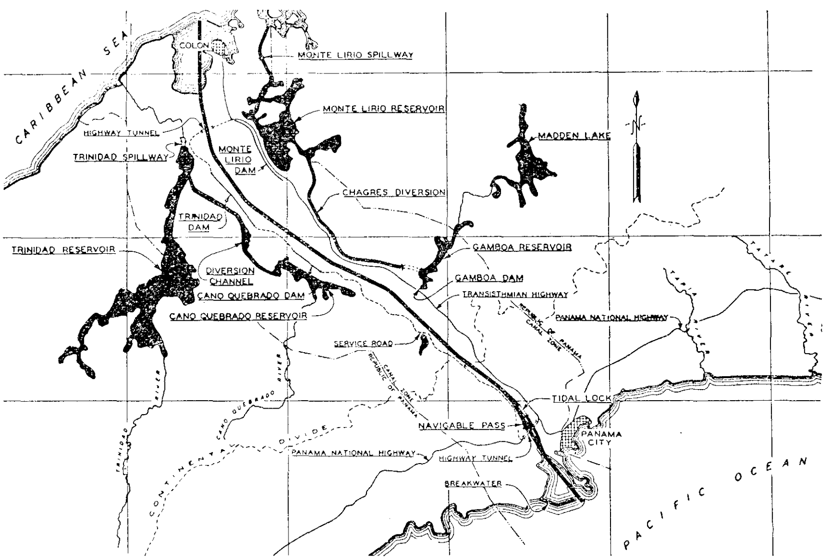


FIG. 7.—PLAN OF PANAMA SEA-LEVEL CANAL

already in place. These installations would have to be duplicated at any other sea-level route. The plan of development for a sea-level canal at the several routes was similar to that for a sea-level canal at Panama as described in the next section.

Plan of Development of a Sea-Level Canal at Panama

There are numerous possibilities for a sea-level canal in the Canal Zone and in the immediate vicinity;[7] the more favorable of these are shown in Fig. 8. The route that would cost the least and have other outstanding advantages is designated the Panama sea-level conversion route, since it follows generally the alinement of the present lock canal. The distinctive feature of a canal on the conversion route is that its construction would involve lowering the present canal to sea level, whereas this would not be the case if either the Panama parallel or one of the Chorrera canals were constructed.

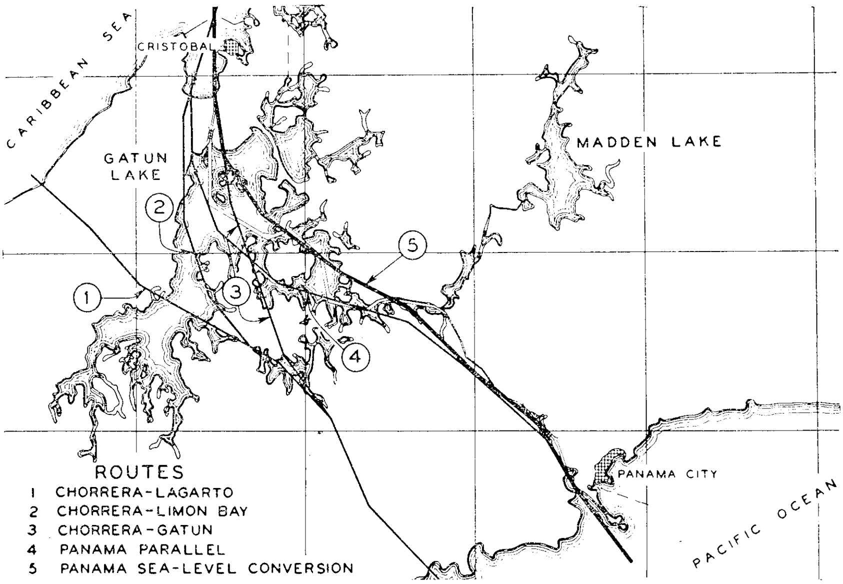


Fig. 8.—Routes in Canal Zone and Vicinity

In the Gatun Lake area, the Panama parallel canal would be separated from the existing lock canal by a barrier dam constructed from spoil material from the excavation for the new canal. From Gamboa south to Balboa Harbor it would follow an alinement separated from the present canal. Thus, the Panama parallel sea-level canal would be completely independent of the lock canal and, as would be the case if the Chorrera-Lagarto canal were constructed, both the existing canal and the new canal could be maintained and operated if this were thought necessary. There are no reasons arising either from capacity or security considerations that would justify the additional $800,000,000 for the Panama parallel sea-level canal. Any one of the Chorrera routes would be even more costly than the Panama parallel route and would be no more justified.

The alinement of the present canal and that of the proposed Panama sea-level conversion canal would be sufficiently separated at several beaches to make it possible to construct a substantial part of the latter in the dry, thus avoiding

[7] "Sea Level Plan for Panama Canal," by J. G. Claybourn, *Proceedings*, ASCE, February, 1947, p. 175.

interference with canal operations. The alinement improvements introduced in the conversion route would shorten the canal by 5.2 miles.

The Sea-Level Canal Channel

The channel of the Gaillard Cut in the present canal is deficient in width for two-way traffic involving large and unwieldy ships, and single directional transiting arrangements for such ships are in effect to avoid their encountering other ships in the cut. The delays and the inconveniences resulting from the special handling of this type of traffic have not thus far been seriously objectionable, but they would be with the further growth of traffic and this fact was borne in mind in designing the sea-level canal.

Currents up to 4.5 knots would be induced in a sea-level canal at Panama without tidal control by the Pacific tides which have a range up to 20 ft. Atlantic tides have a maximum range of 2 ft and would cause currents of 0.5 knot if the Pacific tidal effects were eliminated by control works at the Pacific entrance.

In establishing the design of the sea-level channel, a world-wide survey was made of comparable channels and canals. Several waterways having characteristics similar to the Panama sea-level canal, including the sea-level canals at Suez (Egypt) and Cape Cod (Massachusetts), were visited by members of the staff engaged on the studies. In addition, model investigations were made for The Panama Canal by the United States Navy at the David Taylor Model Basin at Carderock, Md., to determine, for both the lock and sea-level canals, the width, depth, and alinement requirements. For the sea-level channel determinations, various widths and depths of channel using both straight and bend channel sections were tested with currents up to 5 knots. Self-propelled, remote-controlled models of typical ships that transit the canal, operating at a wide range of ship speeds, were employed in testing the channels.

Two-way traffic of most of the ships that would be expected to transit the canal could be accommodated in a channel designed for the safe meeting of a standard "Liberty" ship by the largest naval craft or the largest commercial craft now afloat. The meeting and passing of the largest present-day naval craft and commercial vessels would be unusual and could be avoided by special transiting arrangements similar to those now in effect. Similar arrangements could be made for the transit of the largest ships expected to be built during the twentieth century.

In spite of considerations leading to the recommendation of the Governor of The Panama Canal (J. C. Mehaffey) that tidal currents in the canal be regulated, the channel was designed for safe navigation in currents up to 4.5 knots. This is made necessary by reason of the fact that the structures for the regulation of currents in the canal could be irreparably damaged by bombing and would have to be cleared from the channel. As the result of the studies, it was concluded that all shipping could safely transit the proposed Panama sea-level canal at any condition of current that would obtain up to the maximum of 4.5 knots; only an occasional unwieldy or low powered ship would be held for mean tide to avoid encountering high currents. Tug assistance could be provided such ships to avoid delays and to provide increased safety of transit when thought necessary.

The channel standards adopted as the result of the model studies and other investigations are given in Table 3, as are the controlling standards of the present canal.

TABLE 3.—COMPARISON OF CHANNEL DIMENSIONS

Description	Depth (ft)	Width (at a depth of 40 ft) (ft)	Cross-sectional area (minimum) (ft)	Minimum sight distance (miles)	Maximum deflection angle (degrees)	ANGULARITY	
						Total (degrees)	Per mile of canal length
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)
Present Panama lock canal	42	300	13,860	0.6	67[a]	598	11.7
Modernized Panama lock canal	55	500	28,400	0.6	67[a]	642	12.5
Panama sea-level canal	60	600	36,800	1.5	26	117	2.5

[a] This applies to Gatun Lake; in Gaillard Cut the maximum angle is 30°.

The survey of the world waterways disclosed that channel depths are generally considered inadequate and that better ship controllability would result with more water under the keel. The Carderock tests fully confirmed the latter conclusion. The lesser depth proposed for the projected lock canal results from the lesser ship speed (8 knots) that would be prescribed in the restricted channel of a lock canal. A ship speed of 10 knots was used to establish the depth of the projected sea-level canal. The width of channel is referenced in each case to the 40-ft depth below the low water surface, which approximates the draft of the large vessels of the future. This depth also establishes the point of revolution in setting bank slopes, thus minimizing variations in the surface width of the channel arising from differences in the slopes, which are fixed by the character of the bank materials.

The results of the model tests were interpreted with the assistance of the United States Navy and the pilots of the Panama Canal and of the Cape Cod Canal. The advice and counsel of the Panama Canal pilots on all problems dealing with navigation were invaluable in the conduct of the studies.

SEA-LEVEL CANAL MODEL INVESTIGATIONS

George B. Pillsbury,[8] M. ASCE, has developed a procedure for computing currents in sea-level canals which, when applied to the proposed Panama sea-level canal, yielded results that were closely confirmed by Boris A. Bakhmeteff, Hon. M. ASCE, by independent computations and by model investigations. A model of the proposed sea-level canal at an undistorted scale of 1 : 100 provided methods of accurately determining conditions of flow in the uncontrolled waterway at all ranges of tide, of establishing the design, and of testing the tidal-regulating structures. The maximum velocities in the unregulated

[8] "Tidal Hydraulics," by George B. Pillsbury, *Professional Paper No. 34*, Corps of Engineers, U. S. Government Printing Office, Washington, D. C., 1940.

canal 60 ft deep and 600 ft wide at 40-ft depth at various Pacific tides as computed and as observed in the sea-level canal model are given in Table 4.

TABLE 4.—VELOCITIES IN AN UNREGULATED SEA-LEVEL CANAL

Tidal range (ft)	Percentage of time tides are exceeded	Maximum Velocities, Uncontrolled Channel	
		Observed (knots)	Computed (knots)
(1)	(2)	(3)	(4)
6.....	99	2.1	2.1
10.....	80	2.7	3.0
13.....	50	3.3	3.5
16.....	20	3.8	4.0
20.....	2	4.4	4.5

TABLE 5.—SCHEDULE OF NAVIGABLE PASS OPENING IN A REGULATED SEA-LEVEL CANAL

Permissible current velocity (knots)	Hours per Day That the Navigable Pass Would Be Open for Tidal Ranges of:				
	6 ft	10 ft	13 ft	16 ft	20 ft
(1)	(2)	(3)	(4)	(5)	(6)
1........	8.5	4.5	3.5	2.9	2.3
2........	20.2	9.7	7.2	6.0	4.9
3........	24.0	24.0	14.9	11.6	9.1
4........	24.0	24.0	24.0	24.0	19.8
4.5.......	24.0	24.0	24.0	24.0	24.0

TIDAL REGULATION FOR THE SEA-LEVEL CANAL

The majority (eight out of thirteen) of the Consulting Board appointed by President Theodore Roosevelt in 1905 to consider various plans for a canal at Panama prepared by the Isthmian Canal Commission in voting for a sea-level canal stated with respect to tidal control:

> "The plan [sea-level] proposed by the Board for the Isthmian transit will have twin tidal locks near the Pacific terminus, which if disabled, one or both [by enemy attack or sabotage] would still be usable (after removal of wreckage) for a part of each day (the period of spring tides) in each lunar month, and probably throughout the whole twenty-four hours the remainder of the lunar month (neap tides)."

If tidal regulation were to be omitted, occasional ships would need to be held at the canal entrances for a favorable tide, or tug assistance would need to be provided for their transit. The Carderock tests and experience in other waterways are conclusive in demonstrating that relatively few modern ships would require special transiting arrangements; the majority, having reliable power and rudder control, would be capable of transiting the Panama sea-level canal safely in currents up to 4.5 knots. Nevertheless, because of the serious consequence of collisions with the canal banks (which would be largely of rock), it was decided to provide tidal regulation for added safety to shipping. The loss of tidal regulation as the result of bombing of the regulating structures is a definite possibility, and is accepted as a condition of operation during wartime.

If the control of the Pacific tides were to be provided by tidal locks and a barrier dam and these were to be damaged by the enemy, then the canal would either have to be closed for repairs or the structures cleared from the channel to permit use of the canal as an open waterway. In the latter case, there would be an abrupt transition from slack-water navigation to navigation in currents ranging up to 4.5 knots. Prolonged closure of the canal for repairs would be intolerable, which suggests that the tidal-regulating structures should be planned to permit the rapid clearing of a "channel-way" in case they are damaged. This could be done by excavating an auxiliary channel which would normally be closed by an earth barrier dam that could be rapidly blasted to clear the channel when the locks were rendered unusable by bombing. Obviously, an abrupt change from navigating a canal with currents completely regulated to one with currents of the order listed in Table 4 would be undesirable and this led to an investigation of other methods for providing tidal regulation.

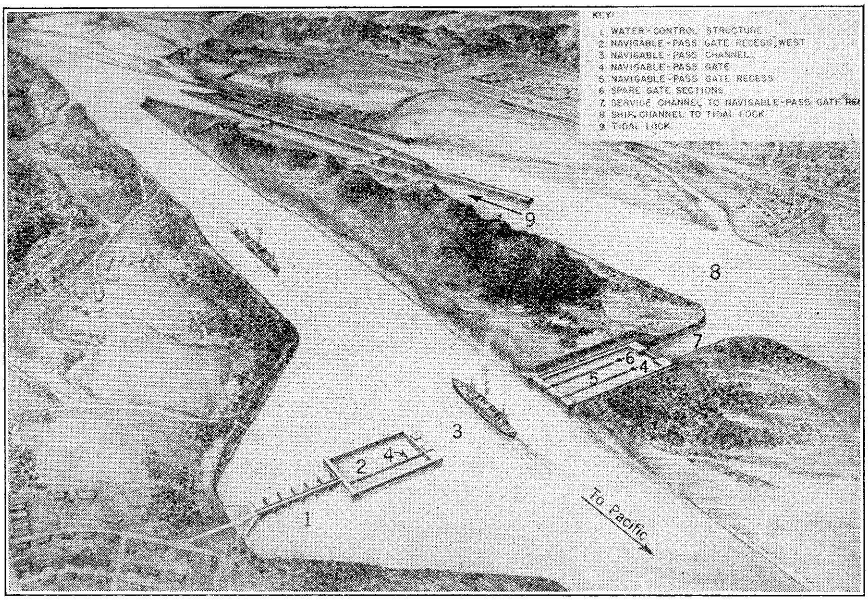


FIG. 9.—TIDAL-REGULATING STRUCTURES SHOWING THE NAVIGABLE PASS OPEN

A sudden transition from slack-water navigation to navigation in a completely unregulated waterway in the event of damage to the tidal lock could be avoided by supplementing the tidal lock with a navigable pass through which ships, as a matter of routine operation, could pass at any selected stage of the tide. The pass would have retractable gates (Fig. 9) which could be opened rapidly to permit navigation without the need for lockage under any set of conditions of current desired, from 0.5 knot (Atlantic tidal currents) to the maximum. The gates of the pass would be of steel construction, and in the event of their damage could be readily removed from the channel. Table 5 shows the hours daily that the pass would be open if the tidal currents were controlled to 1, 2, 3, 4, and 4.5 knots. These data were determined by computations and confirmed by sea-level model tests.

It is probable that, when the sea-level canal is first placed in operation, the navigable pass would be opened only at mean tidal stages to limit channel currents to low values. As operating experience is gained, the periods of opening of the pass gates would be extended to permit higher currents in the canal. Tentatively, currents in the canal would be limited to 2 knots; this value was selected after a survey of operating experience in other waterways and as the result of the United States Navy tests at Carderock. On this schedule for the opening of the pass gates, the canal would operate 32% of the total time as an open waterway.

A gated water-control structure would complement the navigable pass to assist in adjusting the water surface in the canal to extend the period of opening of the pass at each mean stage cycle of Pacific tide (Fig. 9.) The gates of the water-control structure would be open only when the pass is closed to traffic. The tidal lock could accept ships at all stages of the tide and would be the sole avenue of transit when the pass is closed.

The sea-level canal model was an invaluable aid in planning the location, the arrangement, and the hydraulic features of the separate elements of the tidal-regulating works and in developing the operating schedule of the navigable pass.

One important phase of the model study was that of relating the friction factor of the model to the friction factor of the prototype channel. This was accomplished by actual flow tests in Gaillard Cut with the culverts in the Pedro Miguel Locks wide open and discharging 22,000 cu ft per sec.

Slides in the Panama Sea–Level Canal[9]

A study of the major slides experienced in the period of the canal construction was of great assistance in comprehending the character and probable behavior of the materials to be encountered in any new excavation. These slides, as was stated by the late George W. Goethals, M. ASCE, in 1916, were the result of an attempt to fix uniform slopes throughout the length of the canal, regardless of height of cut and character of materials. General Goethals stated after the completion of the canal:

> "* * * With the geological formation changing so frequently and so suddenly both in the direction of the Cut and up and down there is no possibility of any uniformity in slopes. No uniformity of slopes could be maintained * * *."[10]

For a time after the first major construction slide took place, it was thought that the slopes would eventually stabilize themselves and that the volume of material ultimately to be removed would be less if the slides were allowed to run their course. This proved not to be the result; moreover, the pressing need for opening the canal led to flattening certain of the slopes to prevent further blockage of the channel. Fig. 10 represents for a specific location the design slopes now proposed for the Cucaracha formation, and shows the slope selected in the initial construction and the actual slope attained in this formation after failure.

Consideration was given in the design of slopes to dynamic loading that could be induced by bombing. A study of the resistances of the materials to

[9] "Panama Canal Slides," Rept. of the Committee of the National Academy of Sciences, *Memoirs*, National Academy of Sciences, Vol. XVIII, 1924.

[10] "The Dry Excavation of the Panama Canal," by George W. Goethals, *Transactions*, International Eng. Cong., San Francisco, Calif., 1915.

dynamic (transient) loadings was made at Harvard University in Cambridge, Mass., under an investigational contract. The studies of the resistances of the materials to transient loadings and the tests of their residual strengths after failure will be of particular interest to engineers. The transient-load

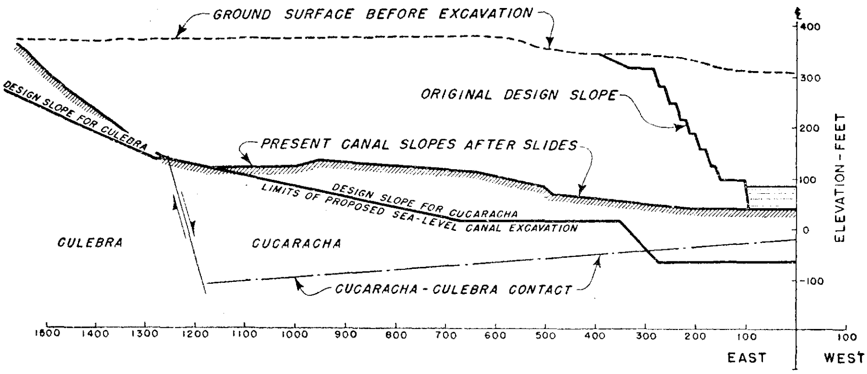


FIG. 10.—ORIGINAL AND PRESENT SLOPE DESIGN CRITERION (OLD CANAL STATION 1785+00)

testing apparatus developed at Harvard University offers promise as a valuable tool in developing fuller understanding of the behavior of soils and rocks under stress.

No allowance was made in the designs for slide failures resulting from dynamic loadings since it is unlikely that slides induced by the atomic bomb would fully block the canal. However, debris from cratering resulting from atomic bombing could block the channel, but widening the channel to overcome this possibility would require such a large amount of excavation that the risk of closure was accepted, since in any case the blocking materials, even though radioactive, could be removed in a few weeks at the outside. None of the conventional weapons could induce slides or throw up craters that would block the channel of either a sea-level or an improved lock canal.

CONVERSION TO SEA LEVEL

Studies of the conversion of the Panama lock canal to a canal at sea level prior to that under Public Law No. 280 contemplated the lowering of Gatun Lake in stages. One study planned for the lowering of Gatun Lake in seven stages. Philippe Bunau-Varilla,[2] the French engineer who negotiated the sale of the canal holdings of the French to the United States, urged the consulting board appointed by President Theodore Roosevelt to provide deep upper sills at each of the locks to facilitate the later conversion of the lock canal by stages to a sea-level canal. His plan was rejected on the grounds that major modification of the proposed twin locks would be inescapable in any case and that a third set of locks would be needed in order that two lanes of locks would be available for traffic at all times during the conversion period.

There is no question that the conversion of the existing locks would involve considerable risk both to shipping and to the integrity of the canal with only one lock lane available and that no plan should be accepted that would have less than two lanes of locks available to shipping throughout the period of conversion. If the lowering of the summit lakes to sea level by stages is undertaken, new special twin conversion locks of minimum construction with lift to El. 53 are preferable to previously considered arrangements involving the progressive alteration of the existing locks for each stage of lowering using a new third set of locks with full lift to El. 85 to insure two lanes being available at all times. The proposed special twin conversion locks would be placed in operation and the existing locks abandoned when the first stage of lowering of Gatun Lake from El. 85 to El. 53 is accomplished—or after the completion of excavation to prepare the channel to carry traffic with the summit lake at El. 53. The next stage of lowering of the canal would be to El. 22, and finally the lowering would reach sea level; in each case excavation to provide necessary depth of channel would precede lake lowering. The twin conversion locks have low upper sills to accommodate traffic at all stages of canal elevation from El. 53 to El. 0. This plan and others considered for the stage lowering of the summit lakes were finally abandoned because of the attractiveness and the cheapness of the plan for the single-stage lowering of the summit lakes.

The conversion of the lock canal to sea level by lowering the summit lakes in a single operation would require dredges capable of excavating to a depth of 145 ft. For this purpose a cutterhead-type suction dredge would be used to excavate the softer materials and a chain-bucket type dredge, to excavate blasted hard materials. The suction dredge would have a booster pump mounted in a well on its ladder to effect the lift of the materials. The chain-bucket dredge would be similar to the conventional type used in mining of gold and tin except that the 2-cu-yd buckets proposed are considerably larger than those ordinarily used to depths of the order proposed. Dredges of the chain-bucket type worked successfully to depths of 128 ft in the mining of gold; however, the buckets generally have not exceeded ½ cu yd. The Board of Consulting Engineers and others advising Governor Mehaffey on the current studies were unanimous in their opinion that deep dredging to effect single-stage conversion would be practical and economical.

In the deep-dredging method of conversion (single-stage lowering of the lakes), there would be a net saving of $130,000,000, principally through the omission of conversion locks. The actual lowering of the water surface to effect the conversion to sea level would be accomplished by progressive demolition of natural rock plugs and the removal of a temporary steel dam left to retain the Gatun and Miraflores lakes. The conversion could be effected with a traffic interruption of only 7 days.

Alinement improvements would permit a large part of the excavation to be done in the dry (750,000,000 cu yd) at a large saving in cost. There would be 318,000,000 cu yd of wet excavation classified as follows:

Cubic yards	Classification
9,400,000	Hard rock
97,700,000	Medium rock
46,600,000	Soft rock
165,000,000	Sands, gravels, and clays

Of this amount, approximately 132,000,000 cu yd would lie between depths of 85 ft and 145 ft below maximum water surface and would require the employment of special dredges.

Flood Control in the Sea-Level Canal

Unless tributary inflows into the sea-level canal are controlled, there would be interruptions in service and occasional hazard to navigation. The most essential control is that of the Chagres River, the major tributary; this would

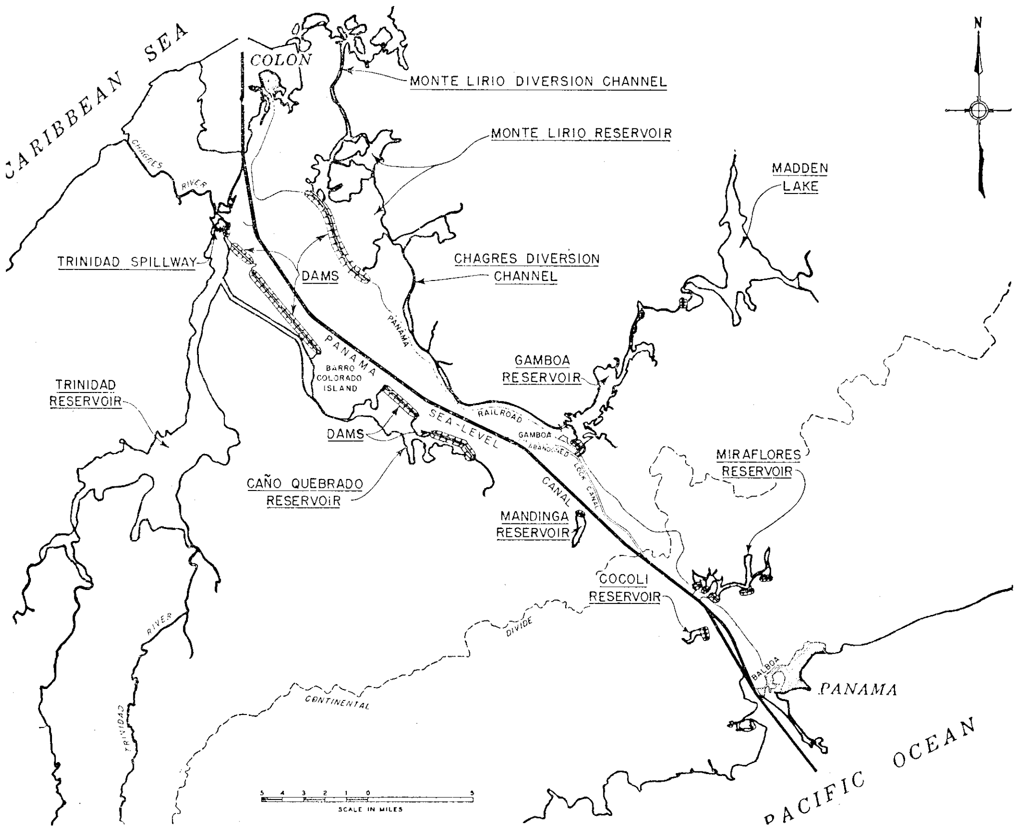


Fig. 11.—Sea-Level Canal Flood Control System

be accomplished by a dam constructed at Gamboa to operate in tandem with the upstream Madden Dam. The regulated outflows of the Chagres River would be excluded from the canal by diverting them to the sea through a short tunnel and a channel as shown in Fig. 11. The flows from all other tributaries on the east side of the canal north of the Chagres River would be intercepted and conducted to the Caribbean (Las Minas Bay) by a system of dams and the channels which convey the regulated flows from the Chagres. This control system, termed the "East Diversion," would have sufficient capacity to divert all floods up to 25% larger than the maximum flood of record. Emergency outlets into the canal are provided for larger floods.

"The West Diversion" system of dams, reservoirs, and channels would exclude the entry of flows into the canal from all the important tributaries west of the canal and north of the Continental Divide by diverting them into the channel of the lower Chagres through an outlet and spillway west of the present Gatun Dam.

South of the Continental Divide there are a number of small tributaries (maximum drainage area 13 sq miles) which could contribute undesirable flood inflows if not regulated. Diversion being impracticable because of the topography and the developments in the area, a system of regulating reservoirs as shown in Fig. 11 would be provided, the corresponding drainage areas being:

Reservoirs (see Fig. 11)	Square miles
East Diversion—	
Madden Lake	393
Gamboa Reservoir	127
Monte Lirio Reservoir	180
West Diversion—	
Caño Quebrado Reservoir	166
Trinidad Reservoir	322
Small Reservoirs—	
Mandinga	10.5
Cocoli	11.5
Miraflores	30
Uncontrolled	118
Total	1,358.0

The proposed combination of reservoirs and diversion systems would control 1,240 sq miles of the total of 1,358 sq miles of area tributary to the canal between Gatun and Balboa. Of the uncontrolled drainage areas, the largest would not exceed 6 sq miles. The flood contribution from the uncontrolled areas would neither inconvenience navigation nor be a hazard to ships in transit.

The flood-control dams would be constructed from excavation spoil except where the length of haul would make local borrow cheaper. The east and west diversion dams would be low structures of earth and rock spoil mounted on broad platforms of fill constructed of excavation spoil placed in the wet, thus insuring very conservative loadings where the foundations are muck and soft clays. The earth and rock flood-control structures would be readily repairable in case of damage by enemy bombing.

Sea-Level Canal Construction Plan

It is planned to complete the construction of the sea-level canal in 10 years. The construction plan adopted in the report to Congress provides for the use of large shovels and draglines (25 yd or larger) dumping into scows for the haul of dry excavation, totaling 750,000,000 cu yd, to disposal areas in Gatun Lake. Wet excavation to customary depths would be performed by conventional dredges, but for depths beyond the range of this equipment, special dredges capable of excavating to 145 ft below the water surface would be required. Vehicular tunnels under the canal would be constructed at the Atlantic and Pacific ends to aid construction and for cross-channel access thereafter. Total cost of the sea-level canal is estimated at $2,483,000,000.

Capacity of the Panama Sea-Level Canal

The capacity requirements for the year 2000 on peak days would be 69 transits. For planning purposes, ship speeds in the sea-level canal were established at 12 knots ground speed traveling with the current and 8 knots against current. A conservative spacing of 1.5 nautical miles between ships was selected after a survey of the practices of other waterways and with the advice of Panama Canal pilots. The daily capacity of the sea-level canal, based on a 16-hour operating schedule for both the tidal lock and the navigable pass, and assuming the latter would be opened to limit currents up to 2 knots, would be 116 transits daily. The average transit time in the sea-level canal would be 4.5 hours. This compares with an average transit time of 8 hours in the present canal.

Security of the Panama Sea-Level Canal

The tidal-regulating and flood-control structures of the sea-level canal would not be essential to its safe operation; hence their damage or destruction would result only in temporary interruptions to traffic. An adequate flood-warning system would insure against ships being caught in the channel at the time of incidence of large flood inflows in the event of damage to the flood-control structures. The flood-impounding structures being of earth and rock construction would be highly resistant to bombing but, because they are not absolutely vital to the operation of the sea-level canal, the costly treatment necessary to make them resistant to the atomic bomb would not be warranted. If damaged by bombing, the flood-control structures could be readily repaired and restored to service.

The tidal-regulating works could not be made resistant to modern weapons. Their loss by enemy action would be the loss of convenience to shipping and a lessening of safety in transit, particularly for ships with low power and poor rudder control. The risks in transit of such ships could be avoided by having them await a favorable tide or by providing them with tug assistance. The tidal lock, if heavily damaged, would be a mass of debris that would be difficult to remove. If the tidal lock were contaminated by radioactive particles as the result of atomic bombing, it would probably have to be abandoned. In either case the navigable pass could be used for transiting traffic. If the pass gates were damaged, they could be removed from the channel in a few days to a few weeks, depending on the extent of damage to the channel-way. Thereafter, the canal would operate as an open waterway.

None of the conventional weapons known today could induce slides or blockage of the channel by cratering that would close the sea-level canal to traffic. However, the canal could be blocked for limited periods as the result of an atomic weapon attack. It is estimated that a crater blockage of the canal would require a few weeks, at the most, for the clearing of a traffic lane, even though radioactive contamination would delay the initiation of the removal work. The shielding of the dredges and other excavation equipment to protect crews would be required, and this is considered practicable.

An attack employing conventional or atomic weapons could do great damage to the housing and administrative facilities of the canal and could result in great loss of life but, disastrous as the loss of life would be, the operation of the sea-level canal could go on uninterrupted since ships in an emergency could transit the canal through the navigable pass under the pilotage of their masters.

Acknowledgments

The papers of this Symposium summarize the major studies, except those pertaining to security, made by the staff of The Panama Canal and its consultants and collaborators. Those who contributed to the studies are too numerous to name, but acknowledgment of their contribution is made in the report of the Governor of The Panama Canal under Public Law No. 280. The report consists of text and a folio of eight plates, supplemented by eight annexes and twenty-one appendixes. Appendix 21 of the report contains a complete bibliography for the Isthmian Canal studies. Particularly valuable assistance was provided by the Bureau of Yards and Docks and the Bureau of Ships, Department of the Navy; the Chief of Engineers, Department of the Army; and the Atomic Energy Commission. All major features of the study were reviewed by a Board of Consulting Engineers composed of Boris A. Bakhmeteff and William H. McAlpine, Hon. Members, ASCE; Hans Kramer, John J. Manning, Hibbert M. Hill, and Joel D. Justin, Members, ASCE; and until his retirement in September, 1946, Ben Moreell, Hon. M. ASCE. The writer was in immediate charge of the studies under J. C. Mehaffey, the Governor, The Panama Canal, who was charged by Public Law No. 280 with the responsibility for the studies.

TRAFFIC AND CAPACITY

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SYNOPSIS

A forecast of future vessel movements through the Panama Canal and a study of the important characteristics of this traffic indicate that 16,660 vessels of all types may be expected to transit a Panama canal in the year 2000. The average daily traffic would be 46 vessels; the peak-day traffic for which the canal should be designed is 69 vessels. By about 1960, the capacity of the present canal, during periods of lock overhaul, would be insufficient to accommodate traffic without undesirable delays on peak days. With modifications principally to eliminate the periodic need for closure of one of the twin locks for overhaul, the capacity of the present canal could be increased to meet all commercial requirements until the end of the twentieth century. Because of the limited size of the lock structures, certain large naval and commercial vessels would be unable to transit the canal.

The construction of new and larger locks or the conversion of the present canal to sea level would create sufficient capacity to handle all traffic until well beyond the year 2000.

INTRODUCTION

The studies under Public Law No. 280, Seventy-ninth Congress, included investigations of the methods of increasing the capacity of the Panama Canal to meet the future needs of interoceanic commerce and national defense. This aspect of the studies required a prediction of the volume and characteristics of canal traffic at some future date which, for planning purposes, was selected as the year 2000. As stated in the first Symposium paper, the prediction was based on a recent evaluation of future Panama canal commercial traffic made by Professor Kramer of the University of Pennsylvania, who was engaged as a traffic consultant by The Panama Canal. The capacities of the present Panama Canal, of two improved Panama lock canals, and of a Panama sea-level canal were then analyzed and their capacities were compared to the estimated design peak-day demand for transits to determine when demand would exceed capacity. These comparisons provided an index of the date at which new facilities would be needed at Panama and of the relative adequacy of the proposed canal improvements.

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Ocean-Going Commercial Traffic

The largest group of vessels transiting the Panama Canal is that which pays tolls. It consists of merchant vessels, foreign naval vessels, privately-owned dredges, yachts, and similar craft. Tolls-paying vessels of more than 300 tons are considered to be ocean-going commercial vessels. In peacetime these make up about 78.5% of all transits, and carry nearly all the commercial cargo passing through the Panama Canal. Tolls-paying vessels of less than 300 tons are engaged principally in local trade and carry an insignificant volume of the commercial cargo.

Future Tonnage of Ocean-Going Commercial Traffic.—Professor Kramer's estimate of future Panama Canal traffic is concerned only with cargo carried in ocean-going commercial vessels. His forecast is based on the following assumptions:

a. No widespread wars or serious political disturbances will take place during the period covered by the forecast.

b. The effects of unforeseeable changes in the sciences, such as development of atomic power and of air transport, are not appraised.

c. Foreign trade will develop as the United Nations organization achieves stature and as trade restrictions are reduced.

d. The United States will maintain a substantial merchant marine in accordance with the mandates of the merchant marine acts of 1920, 1928, and 1936, and of the Ship Sales Act of 1946.

Professor Kramer's forecast, termed an "economic-statistical projection," correlates Panama Canal commercial traffic with the trend in United States national income as based on future United States population and future per capita income. In this forecast, an estimate of future United States ocean-borne imports, exports, and intercoastal trade was made according to the past relationships between national income and these three factors. Next, the past relationships of Panama Canal commercial traffic to these factors were determined. The relationships established from these estimates were used to derive estimates of future intercoastal, United States import, and United States export traffic through the canal. A fourth factor of Panama Canal commercial traffic, "foreign-to-foreign" shipments, was determined on a percentage basis from past records. The summation of the four components of Panama Canal commercial traffic, intercoastal, United States import, United States export, and "foreign-to-foreign" shipments, resulted in the following net-vessel tonnage forecast of predicted growth of ocean-going traffic (see Fig. 1):

Year	Net vessel tons
1950	36,185,000
1960	44,929,000
1970	54,639,000
1980	65,207,000
1990	75,502,000
2000	86,312,000

This prediction is considered to represent the probable maximum load that will be put on the canal in the future.

For purposes of comparison, earlier predictions of Panama Canal traffic, made by Emory R. Johnson in 1912 and Grover G. Huebner in 1936—both of the University of Pennsylvania, are shown in Fig. 1. As a matter of interest, the traffic history of the Suez canal is also shown.

Conversion of Ocean-Going Commercial Tonnage to Transits.—The number of ocean-going commercial transits expected annually is obtained by dividing Professor Kramer's estimate of yearly tonnage by the estimated average net vessel tonnage per ocean-going commercial transit. The estimate of the average tonnage of ocean-going commercial vessels is based on past experience at Panama and on long-term trends developed at the Suez Canal.

The average net vessel tonnage of oil tankers transiting the Panama Canal has been about 30% higher than that of vessels carrying other types of cargo (general cargo vessels). Also, the relative volume of tanker traffic in the past is very large when compared to its future relative volume as predicted by Professor Kramer. For these reasons general cargo traffic at Panama was analyzed separately from tanker traffic.

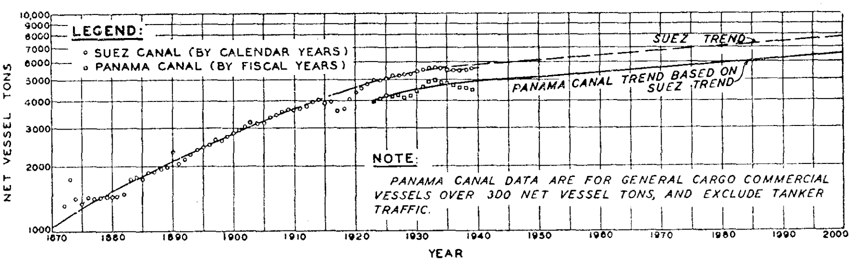

FIG. 12.—AVERAGE NET VESSEL TONNAGES OF GENERAL CARGO SHIPS

The average tonnage of ocean-going general cargo vessels at Panama was determined for 17 "normal" years (from 1923 to 1939) during which traffic was relatively independent of influences arising from World War I and World War II. Because the Panama Canal statistics covered so few years, no well-defined trend could be detected in a plot of these data. Accordingly, the plot was compared with the average tonnage of vessels transiting the Suez Canal in the years from 1870 to 1939 (Fig. 12). The trend of Suez data is well defined and may be extrapolated to the year 2000. The average tonnage of vessels transiting at Suez was thus assumed to increase from 5,600 tons in 1939 to 7,800 tons in the year 2000. As the Panama Canal data are roughly parallel to those at Suez, the Panama Canal trend was assumed parallel to that of Suez and the average tonnage of general cargo vessels transiting at Panama was assumed to increase from about 4,500 tons in 1939 to 6,600 tons in the year 2000. Because of the predicted relative insignificance of tanker traffic, the trend in the average tonnage of general cargo vessels is assumed to be applicable to all ocean-going commercial traffic. The number of ocean-going commercial transits for any year up to the year 2000 is then obtained by dividing the estimated tonnage for that year (Fig. 1) by the estimated net vessel tonnage per transit for the corresponding year (Fig. 12). The predicted increase in the number of ocean-going commercial transits is shown in Fig. 3 and in Col. 2, Table 6.

TABLE 6.—PREDICTED GROWTH OF VESSEL TRANSITS THROUGH THE PANAMA CANAL

Year (1)	Ocean-going commercial vessels (2)	Small commercial vessels (3)	Tolls-free vessels (4)	Total transits (5)	Average daily transits (6)
1950	6,959	567	1,339	8,865	25
1960	8,199	669	1,577	10,445	29
1970	9,486	773	1,825	12,084	34
1980	10,796	880	2,077	13,753	38
1990	11,947	974	2,298	15,219	42
2000	13,078	1,066	2,516	16,660	46

In the absence of any reason for change, it is expected that in peacetime ocean-going commercial vessels will comprise the same percentage of canal traffic in the future that they have in the past—namely, 78.5%.

TRAFFIC OF SMALL COMMERCIAL VESSELS

Tolls-paying vessels of less than 300 tons—small commercial vessels—are considered separately from ocean-going commercial vessels. Although these vessels make up about 6.4% of all transits, they carry only 0.1% of the commercial cargo passing through the canal. However, they are a tax on the capacity of the canal in that they are vessels that must be handled and therefore they cannot be disregarded. It is expected that the traffic of these vessels will increase at a rate corresponding to that of ocean-going commercial traffic and thus will continue to comprise 6.4% of total Panama Canal transits in peacetime. The predicted growth in the traffic of small commercial vessels is shown in Col. 3, Table 6.

TOLLS-FREE TRAFFIC

International treaties and federal legislation permit several classes of vessels to transit the canal without charge. They consist of United States Navy combat and auxiliary vessels, United States Army vessels, cargo-carrying or other vessels operated in the service of the United States Government, warships of the Republic of Colombia, Panamanian Government vessels, vessels transiting solely for the purpose of repair at Canal zone shops, and Panama Canal operational and maintenance equipment. In peacetime, these tolls-free vessels account for 15.1% of all transits. There is no indication that the relative volume of this traffic is changing and therefore the 15.1% factor is used to determine the future peacetime volume of all tolls-free traffic (Col. 4, Table 6).

FUTURE TRAFFIC TOTALS

Total Panama Canal traffic in the future will consist of all commercial traffic and all tolls-free traffic whose predicted values are shown in Fig. 2 and in

Cols. 5 and 6, Table 6. These totals are based on percentage factors derived for peacetime years. The volume of military combat and cargo traffic in future wars is indeterminate, but experience during World War II indicates that commercial traffic volume would drop and tend to offset the increase in military traffic. Because there is no assurance that the drop in commercial traffic will completely offset the rise in military traffic, a canal should have some excess capacity to meet the unknown demands that may be put on it during wars.

Traffic Characteristics Affecting Capacity

Variations in Daily Traffic.—The traffic characteristic which controls the capacity requirement of the Panama Canal is the daily variation in the demand for transits. Annual and seasonal variations in traffic volume are so much less than the daily variation that they are of no significance in establishing the capacity requirement of the Panama Canal. Daily variations in traffic volume have been extreme; immediately before World War II, however, the number of transits on the peak day of the year had decreased and appears to be leveling off at 160% of the number of transits on the average day.

The frequencies with which peak traffic days are expected to occur are shown in Table 7. It is evident that very few delays to shipping would occur

TABLE 7.—Predicted Frequency of Peak Days, Panama Canal

Peak-Day Conditions		Days per year in which peak-day traffic would be exceeded
Percentage of average daily traffic	Transits in the year 2000	
130	60	45
140	64	24
150	69	9
160	73	0

if the capacity requirement were established so as to provide for traffic peaks 50% in excess of the daily average. With this daily capacity, traffic would be held over to the next day on an average of only 9 days a year, and, in the year 2000, delays would be of such an order that about 20 vessels would be held up a total of about 400 ship-hours with 24-hour operation of the canal in effect. This service is compatible with the high standards expected of the Panama Canal. Therefore, days in which the traffic is 50% in excess of the average are used to establish the capacity requirement of the Panama Canal and are called "design peak days." In the year 2000, this requirement is the capacity to transit 69 vessels of all types in 24 hours; this requirement and the requirements for other years are shown in Fig. 1.

Direction of Traffic.—In the past, 52% of the yearly ocean-going commercial transits passed through the canal from the Atlantic to the Pacific and 48% transited from the Pacific to the Atlantic. From this, it is assumed that, in effect, future traffic will be evenly divided with respect to direction. The assumption applies only to annual traffic; on individual days traffic may be considerably unbalanced and traffic in any one direction may be nearly three times as great as traffic in the opposite direction. Under certain conditions, the expected maximum unbalance of traffic on peak days would reduce the maximum capacity of a lock canal slightly, but would have no significant effect on the capacity of a sea-level canal. The effect of unbalanced traffic on the capacities of lock canals is described subsequently in this paper.

Vessel Length.—The number of vessels which can be placed in a given lock chamber at one time is controlled by their lengths. This has an important effect on the capacities of the lock structures in a lock canal or in any tidal locks used in a sea-level canal. Using the records of the Panama Canal (from 1921 to 1940), traffic has been grouped into the vessel-length categories shown in Table 8.

TABLE 8.—COMPOSITION OF ALL TRAFFIC THROUGH THE PANAMA CANAL FROM 1921 TO 1940 BY LENGTH OF VESSEL

Length (ft)	Percentage	Transits
> 500	6.2	8,004
400 to 499	47.3	60,870
300 to 399	20.0	25,806
200 to 299	7.3	9,395
< 200	19.2	24,535
Total	100.0	128,610

The records show that the composition of traffic by vessel lengths has varied little from year to year. However, studies made of the relations of net vessel tonnage, beam, length, and tonnage per foot of length, indicated that the predicted increase in average net vessel tonnage would be accompanied by a slight increase in vessel length. Furthermore, it was indicated that a 10-ft increase in the average length of all vessels would be sufficiently conservative for estimating the capacities of lock structures in the year 2000. It was assumed that this increase would be uniform from the present to the year 2000. Table 9 shows the percentages of vessels that may be expected in each length category for the years 1960 and 2000.

TABLE 9.—PREDICTED COMPOSITION OF ALL TRAFFIC THROUGH THE PANAMA CANAL BY LENGTH OF VESSEL

Vessel length (ft)	PERCENTAGE OF TRANSITS		Transits, year 2000
	1960	2000	
> 500	8	11	1,832
400 to 499	46	45	7,498
300 to 399	20	19	3,165
200 to 299	7	7	1,166
100 to 199	8	8	1,333
< 100	11	10	1,666
Total	100	100	16,660

Experience at Panama indicates that the composition of traffic by length of vessel on peak days is very similar to the average annual composition. This fact is of particular significance in establishing the capacity requirements of lock structures which should be able to accommodate shipping on design peak days.

DIMENSIONAL LIMITATIONS OF PANAMA LOCK AND SEA-LEVEL CANALS

The maximum size of vessel that could transit a lock canal or that could pass through the tidal lock of a sea-level canal would be limited by the dimensions of the lock chambers. At present only two commercial vessels, the *Queen Mary* and the *Queen Elizabeth,* are too large to be received in the existing 110-ft by 1,000-ft lock chambers. These vessels are used in the North

Atlantic service and would not be expected to operate through the canal. Until the approach of World War II, the lock chamber dimensions of the Panama Canal were accepted as a limitation in the design of Navy vessels. However, several warships constructed or modified during the war exceed the dimensions of the present locks and are therefore unable to utilize the canal. The proposed lock chamber dimensions of 200 ft by 1,500 ft for either a high-level lock of a lock canal or the tidal lock of a sea-level canal would permit the transit of the largest commercial vessels expected up to the year 2000 and would allow a considerable increase in the size of naval vessels without prohibiting their transit. The 750-ft-wide navigable pass of the sea-level canal would permit the transit of any type of vessel of any size likely to be built in the twentieth century.

Capacity of the Present Panama Canal

The capacity of a lock canal is established by the capacity of its locks as reduced by any conditions which affect the availability of the locks for the transit of vessels or which prevent vessels from entering the locks. Examples of such conditions are the occurrence of fog, restrictions fixed by channel dimensions, or lay-up of the locks for overhaul.

The length of the lock chamber, as related to lengths of future vessels, determines the average number of vessels per lockage. Using the length composition shown in Table 9, it was found that, under ideal conditions, an average of 1.8 vessels could be transited simultaneously in the existing locks (110 ft wide by 1,000 ft long). For conservatism, the estimated number of the vessels per lockage was reduced to 1.5.

The time interval between successive lockages in the same direction has been determined from time and motion studies of operations on the existing locks, and amounts to 45 min with double-culvert filling and emptying of the lock chambers. Thus, operating 24 hours daily under ideal conditions, the present locks could accommodate a total of 96 vessels.

An unbalanced flow of traffic in either direction would require some changes in the direction of lockages. The time interval between successive multiple-lift lockages in opposite directions in the same lane is greater than that for successive lockages in the same direction because, in the first instance, the incoming vessel must wait until the outgoing vessel has cleared the lock structure before entering. The longer interval results in a loss of capacity; in the present canal, however, the physical limitations of Gaillard Cut which necessitate one-way traffic at night would give ample opportunity to balance traffic and capacity losses from this cause would not occur.

The frequency with which fogs occur in the Panama Canal is such that the dependable daily capacity of either a lock or a sea-level canal must be based on the assumption that fog is a daily occurrence. Fogs in Gaillard Cut are most frequent during the rainy season and occur generally between midnight and 8 a.m., with an average duration of 4 hours. At present, it is hazardous for a ship to enter Gaillard Cut whenever there is a possibility of fog. This possibility exists during the wet season and, thus, for 8 months a year Gaillard Cut is closed 8 hours a day. The effect of this closure is a loss in capacity of 32 vessels daily.

Traffic through Gaillard Cut is limited to one direction in the case of unwieldy vessels or of vessels with more than an 80-ft beam, for vessels carrying explosive cargoes, and for all vessels at night. This restriction results in a loss of capacity of 6 vessels daily.

The locks are overhauled biennially at Panama, alternating between the Pacific and Atlantic locks. Thus each lock structure is overhauled once every 4 years. The necessary repairs and replacements are completed in about 4 months, and during that time only one lock lane is available. For most of the overhaul period, only a single culvert is available for filling and emptying the lock chambers, and, therefore, the time interval between successive lockages in the same direction is lengthened to 56 min. The daily lock capacity during overhaul is then 39 vessels.

Since only one lock lane is available during overhaul, the direction of lockages must be changed to permit transits in both directions. Two such changes are assumed to occur daily and this results in a daily loss in capacity of 3 vessels during periods of overhaul.

Overhaul is performed in the dry season when fogs are rare and therefore there are no losses in capacity because of fogs during overhaul periods. Since traffic during the overhaul period is essentially single lane traffic, there are no losses in capacity at such times resulting from traffic restrictions in Gaillard Cut.

Dependable Daily Capacity of Present Canal.—The estimated capacity of the present canal, based on the factors previously described, is developed in

TABLE 10.—DAILY CAPACITY OF THE PRESENT PANAMA CANAL (NUMBER OF VESSELS)

Line	Description	Outside of overhaul periods[a]		During overhaul periods[b]	
1	Theoretical capacity		96		39
	Capacity Losses Resulting from—				
2	Changes in direction of transit	0		3	
3	Possibility of fog in Gaillard Cut	32		0	
4	Restricted channel in Gaillard Cut	6	−38	0	−3
5	Net capacity		58		36

[a] Operation outside of overhaul periods; maximum theoretical capacity of two lock lanes, with double-culvert filling and 1.5 vessels per lockage. [b] Operation during overhaul periods; four consecutive months every two years; maximum theoretical capacity of one lock lane, with single-culvert filling and 1.5 vessels per lockage.

Table 10. Table 10 shows that the daily capacity of the present canal is a minimum during periods of lock overhaul. Each lock overhaul lasts 4 months, and it is neither reasonable nor practical to delay vessels over such an extended period. Therefore, the dependable daily capacity of the existing canal is taken at its minimum daily capacity during overhaul—or 36 vessels.

CAPACITY OF THE PRESENT CANAL WITH MINOR IMPROVEMENTS

Table 10 indicates that the greatest improvement for increasing the capacity of the present canal would be to eliminate the necessity of taking one lock lane out of service during overhaul periods. Such an improvement is entirely feasible and could be accomplished by modification of the lock gates, gate settings, and culverts. Lock filling and emptying would then be done with a single culvert for each lock lane while the valves in the third culvert were being serviced. This operation would take place in the fog-free dry season to permit 24-hour operation of the locks. Under these conditions, the net capacity of the canal would be the capacity of both lock lanes, each in single-culvert operation, less losses resulting from channel restrictions imposed by Gaillard Cut—or 70 vessels daily. This is in excess of the 58-vessel daily capacity of the present canal outside of overhaul periods, and the latter capacity would become the controlling capacity of the canal.

To increase the dependable daily capacity of the canal to more than 58 vessels, additional improvements are necessary. The most economical of such improvements is the provision of tie-up stations in Gaillard Cut. These would permit transits to continue until the actual occurrence of fog and it is estimated that they would increase the dependable daily capacity of the canal to 65 vessels.

The foregoing modifications constitute the minimum plan of improvement of the existing canal and would provide adequate capacity to handle traffic until close to the end of the twentieth century.

If a further increase in capacity is desired, it can be obtained by providing the navigation aids that would permit one-way navigation in fog. With navigation in fog the daily capacity of the canal outside of overhaul periods would be 96 vessels less losses caused by channel restrictions imposed by Gaillard Cut—or 86 vessels. However, the daily capacity of the canal during the modified overhaul period would be 70 vessels. Since this is the smaller capacity, it is controlling, and represents the increase in capacity achieved by use of navigation aids. This capacity slightly exceeds the expected design peak-day demand for transits in the year 2000.

Capacity of a Completely Modernized Panama Lock Canal

Complete modernization of the present Panama lock canal would provide for the construction of two new locks (200 ft by 1,500 ft) at each end of the canal, abandonment of the existing locks, widening of Gaillard Cut permitting two-way navigation at night, raising of Miraflores Lake to the level of Gatun Lake to obtain a Pacific summit-level anchorage, and the construction of tie-up stations throughout Gaillard Cut. Using the length composition of vessels for the year 2000 shown in Table 9, an average of 2.9 vessels, under ideal conditions, could be locked through the proposed locks in tandem. For conservatism, however, it was assumed that the locks could accommodate an average of 2.4 vessels per lockage. With the time interval between successive lockages in the same direction estimated to be 57 min, the theoretical maximum capacity of the locks would be 120 vessels daily. Losses caused by an assumed single change in direction to provide for unbalanced traffic would reduce this to 118 vessels.

Without tie-up stations and summit-level anchorages, the loss in capacity in the 8-hour period during which Gaillard Cut would be closed because of the occurrence or possibility of fog would amount to 40 vessels daily.

The provision of a Pacific summit-level anchorage to match that at Gatun would permit vessels to lock-up to the summit level when Gaillard Cut was closed because of fog. With this operating procedure, the loss in capacity due to fogs would be reduced from 40 vessels daily to 25 vessels daily.

With tie-up stations, vessels could proceed through Gaillard Cut until the actual occurrence of fog. The loss in capacity due to fogs would then be further reduced and would amount to 15 vessels daily. The dependable daily capacity of a completely modernized lock canal would therefore be 103 vessels (118 minus 15)—well in excess of the requirement for the year 2000.

The time required to transit, if two new and larger locks were constructed at each end of the canal, would be 7.25 hours, as compared with 8 hours in the present canal under normal operations and 8.5 hours during overhaul periods.

Capacity of the Panama Sea-Level Canal

The capacity of the Panama sea-level canal is established by the conditions under which its tidal-regulating structures are assumed to operate, by the assumed vessel speed and spacing, and by the duration of fogs along the channel.

The tidal-regulating structures of a sea-level canal would consist fundamentally of a tidal lock and a navigable pass. Shipping would utilize the tidal lock when the tidal head between the canal and the Pacific Ocean was large, and the navigable pass when tides were at or near their mean.

The amount of tidal regulation necessary to limit the channel currents to a selected maximum value depends upon the range of the Pacific tides. (This subject is treated subsequently in the fourth Symposium paper.) The capacity of the tidal-regulating structures increases with increased use of the navigable pass—that is, inversely with the tidal range. The dependable capacity of the sea-level canal using tidal regulation has been taken as its capacity on days when 20-ft tides would prevail. Since the navigable pass would be open for longer periods 98% of the time, the assumption is extremely conservative.

The tidal-regulating structures would be designed to control the currents to any value between the maximum velocity resulting from the Atlantic tides alone (about 0.5 knot) and the maximum velocity of about 4.5 knots produced in an uncontrolled waterway by the combination of both Pacific and Atlantic tides. A maximum allowable current of 2 knots has been selected in these studies as the basis upon which to establish the dependable capacity of the sea-level canal.

Vessel Speed and Spacing.—The capacity of a channel depends on vessel speeds and vessel spacings; in these studies an average water speed of 10 knots has been assumed. The factors that bear on the safe distance between vessels traveling in the same direction are their speed, size, power, loading, controllability, operating dependability, channel dimensions, alinement, visibility, weather, and currents. In consideration of these factors, the spacings shown in Table 11 for vessels more than 300 ft long have been selected for the purpose of estimating capacity. Vessels less than 300 ft long can be interspersed with larger vessels spaced in accordance with Table 11 without impairing navigational safety. If vessels were to travel at higher speeds, greater spacing would be required for safety and the net effect would be a reduction in capacity. (Spacings in Table 11 are for vessels longer than 300 ft, traveling at an average water speed of 10 knots.)

Capacity of Tidal Lock.—The daily capacity of the tidal lock depends on the hours of operation, the time interval between lockages, and the number of vessels that would be assembled in the lock chamber at one time. The tidal locks would be the same size as the locks of a modernized lock canal and 2.4 vessels could be handled during each lockage. For 24-hour operation and an average time interval between lockages of 40 min, the daily capacity of the tidal lock would then be 86 vessels.

TABLE 11.—VESSEL SPACING

Channel current (knots)	Spacing (nautical miles)
0	1.0
2	1.5
4 to 5	2.0

Capacity of Navigable Pass.—When open, the navigable pass would give full freedom to the flow of traffic and its capacity would be the same as that for the sea-level channel. The daily capacity of the pass thus depends on the length of time it would be open for navigation and on the adopted vessel spacing (Table 11). If the tidal-regulating structures were operated to limit channel currents to 2 knots, the pass would be open about 4.9 hours during each day having 20-ft tides. The daily capacity of the navigable pass would then be 88 vessels. An increase in the maximum allowable current would permit the pass to remain open for longer periods, and the capacity of the pass would thereby be increased. The daily capacity of the navigable pass for various tides and allowable current is shown in Table 12.

TABLE 12.—DAILY CAPACITY OF NAVIGABLE PASS

Maximum allowable current (knots)	Daily Capacity in Vessels for a Tidal Range of:		
	20 ft	16 ft	13 ft
1	53	66	80
2	88	110	130
3	140	180	240
4	260	320	320

TABLE 13.—CAPACITY OF SEA-LEVEL CANAL WITH TIDAL REGULATION (16-HOUR OPERATING DAY)

Maximum allowable current (knots)	Number of Vessels Passing through:			Notes
	Lock	Pass	Lock and pass	
1	56	35	91	[a] For currents of 4 knots or more, the combined capacity of pass and lock exceeds channel capacity, so the latter controls.
2	57	59	116	
3	59	93	152	
4	61	174	214[a]	

Capacity of Sea-Level Canal with Tidal Regulation.—The 24-hour daily capacity of the sea-level canal with tidal regulation as assumed in these studies would be the combined capacities of the pass and lock—or 174 vessels of all sizes (86 vessels through the tidal lock and 88 vessels through the navigable pass).

Because there are no anchorage areas above the tidal lock or navigable pass, they would not be used when the channel is fogbound, and the capacity of the canal would be reduced accordingly. Observations made prior to the formation of Gatun Lake indicate that fogs in a sea-level canal would have an average duration of 6.6 hours and would reduce the operating day from 24 hours to 17.4 hours and the daily capacity of a controlled sea-level canal from 174 vessels to 126 vessels. In a 16-hour operating day, with operations suspended during the hours in which fog usually occurs, the daily capacity would be as shown in Table 13. It can be noted that, even with the most conservative control of tidal currents, the capacity of a sea-level canal is in excess of the 69-vessel daily capacity required for a Panama Canal in the year 2000.

The average transit time in a controlled sea-level canal would be either 4 hours or 4.75 hours, depending on whether the pass or the lock was used.

Capacity of Sea-Level Canal Without Tidal Regulation.—If operating experience should eventually indicate that tidal regulation is unnecessary or if, during wartime, the regulating structures were not utilized or were rendered inoperative, the canal would have no structures on which the transits of vessels would be dependent. Its capacity would therefore be the capacity of the channel, which is dependent solely on vessel speed and spacing and on the length of the operating day as established by the occurrence of fog.

Using an average water speed of 10 knots and a vessel spacing of 2 miles established for the 4.5-knot current resulting from a 20-ft tide (Table 11), the daily capacity of the canal would be 240 vessels longer than 300 ft. Vessels longer than 300 ft are expected to comprise only 75% of all vessels transiting the canal (Table 9), and therefore 80 additional vessels shorter than 300 ft could be interspersed with the larger vessels; the daily capacity would then be 320 vessels.

Fog of average duration would reduce the operating day from 24 hours to 17.4 hours and the daily capacity from 320 vessels to 232 vessels. If the canal, because of fog or for other reasons, is operated 16 hours a day, the daily capacity would be 214 vessels—far in excess of the requirements for the year 2000.

At an average water speed of 10 knots, the time required to transit the sea-level canal without tidal regulation would vary from 3.3 hours to 5.0 hours.

Summary

The present Panama Canal does not have sufficient capacity to prevent undesirable delays to traffic on design peak days beginning about 1960. With modification of the existing locks to eliminate extensive outages during overhaul periods and with tie-up stations provided in Gaillard Cut to reduce the delays created by fogs, the present lock canal would be adequate until close to the end of the century. In addition to these improvements, if navigation aids were provided which would permit navigation in fogs, the adequacy of the present canal would be further extended. This minimum plan of improvement, however, would not provide for the transit of vessels larger than those accommodated in the present locks.

The Panama sea-level canal or a completely modernized Panama lock canal with new and larger locks would have capacities greatly in excess of requirements for the year 2000. The modernized lock canal and the tidal lock of a sea-level canal would be able to transit any commercial vessel expected to be operating during the twentieth century. They would also permit the transit of much larger naval vessels than now exist. The navigable pass of the sea-level canal would permit the transit of any type of vessel of any size likely to be built prior to the year 2000.

The capacity of a lock canal is fixed by the physical dimensions and characteristics of its locks. In comparison, a sea-level canal has no such limitations, provides faster transit than does a lock canal, and has a larger reserve of capacity which can accept the short-time traffic peaks that may occur during wars.

FLOOD CONTROL

By F. S. Brown,[13] Assoc. M. ASCE

Synopsis

The development of a Panama sea-level canal would require that shipping be protected by adequate control of floods on the major rivers and streams tributary to the canal. If large uncontrolled flows entered the canal, the maximum velocities would be appreciably increased over those normally caused by tides, and dangerous crosscurrents would be generated at each point where large flood flows entered the canal. This paper defines the requirements for flood control in the proposed sea-level canal, describes the hydrological characteristics of the drainage basin, formulates the preliminary basis for hydraulic design of the flood-control projects, and develops the plan of control that has been adopted as an essential feature of the sea-level canal, wherein 87% of the tributary area would be diverted directly to the Atlantic Ocean and an additional 4% would be controlled by retarding reservoirs.

Requirements for Flood Control

Floods have rarely interfered with navigation in the present lock canal because all major tributaries of the watershed, except the upper Chagres River, enter Gatun Lake at remote distances from the navigation lane. Gatun Lake is impounded by a large earth dam 8 miles above the mouth of the Chagres River and has an area of 164 sq miles. The upper Chagres River enters the canal at Gamboa where the lake is narrow. Prior to the construction of Madden Dam in 1934, floods entering the canal at Gamboa twice required the temporary suspension of traffic. The control afforded by Madden Lake has largely ended the likelihood of futher major flood disturbances in the present canal.

Gatun Lake would be drained upon final conversion to a sea-level canal, and the rivers of the watershed, if uncontrolled, would enter the sea-level canal directly. The total area of the watershed (Fig. 13) would be 1,358 sq miles, 94% of which would lie on the Atlantic side of the Continental Divide. The largest tributary would be the Chagres River above Gamboa, with a drainage area of 520 sq miles, of which 393 sq miles are controlled by Madden Dam. Other important tributaries would be the Gatun River, draining 150 sq miles on the east side of the canal, and the Caño Quebrado and Trinidad rivers on the west side, with drainage areas of 120 sq miles and 313 sq miles, respectively. The watershed on the Pacific side of the Continental Divide is only 79 sq miles.

Combined Flood and Tide Flow in a Sea-Level Canal.—Tidal ranges of 20 ft in the Pacific and 2 ft in the Atlantic, which closely approximate maximum tide conditions, would produce a velocity of 4.5 knots near the Atlantic end of

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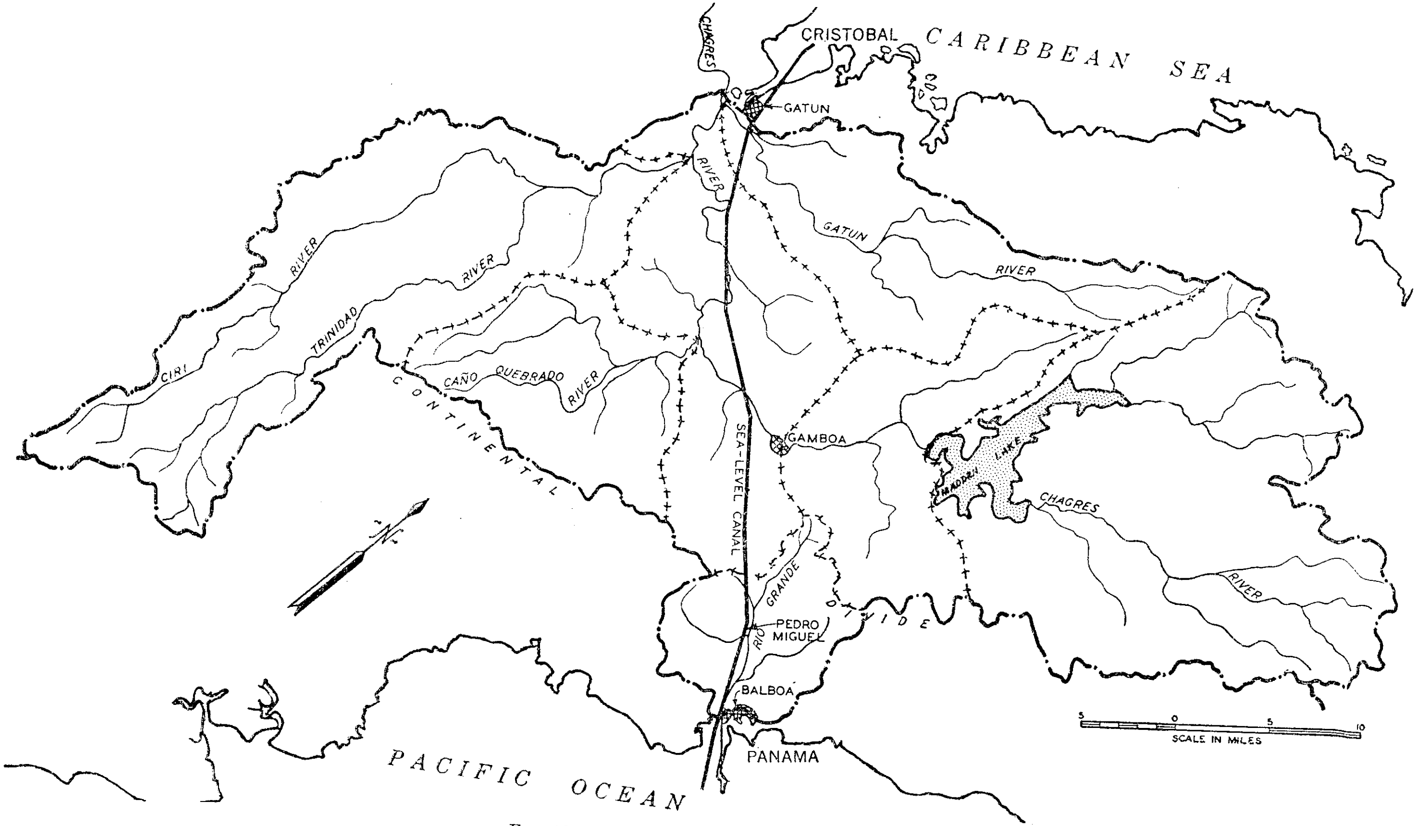

Fig. 13.—Watershed, Sea-Level Canal

an uncontrolled canal. Ships of adequate power and good controllability could safely transit the canal in currents of 4.5 knots, but any appreciable increase in velocity would be undesirable. Tidal-regulating structures would be installed at the Pacific end of the sea-level canal for the reduction of tidal currents to limits probably as low as 2 knots in the initial period of canal operation. Flood control would be needed regardless of the extent of tidal regulation, although disturbances to shipping from floods would be more pronounced in an unregulated canal, a condition that may be imposed during wartime if the tidal-regulating structures were destroyed.

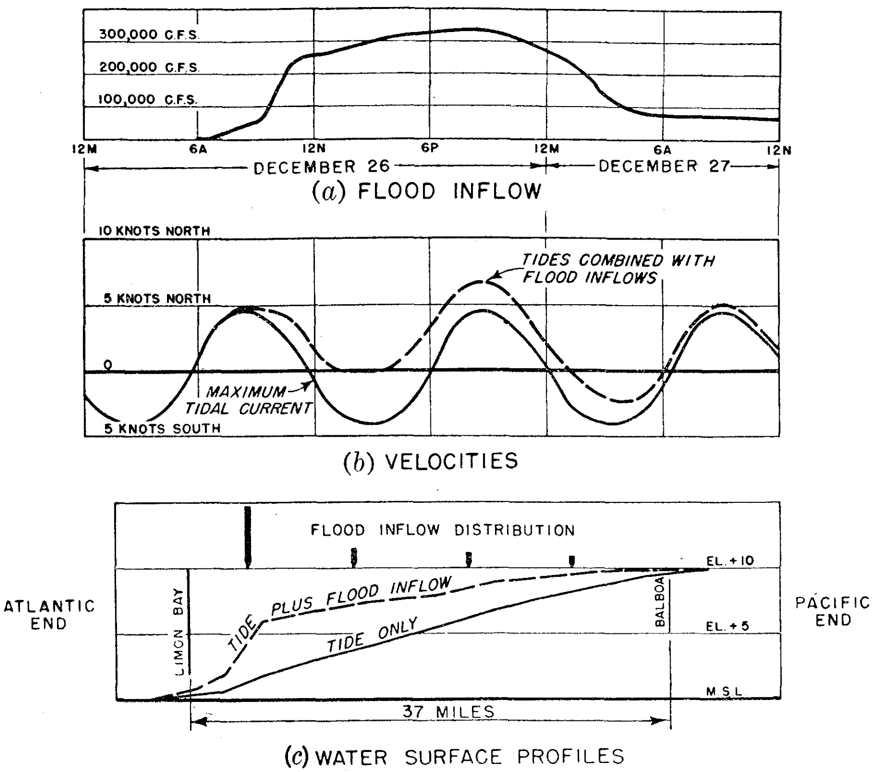

FIG. 14.—VELOCITIES OBSERVED IN MODEL TESTS TO DETERMINE EFFECTS OF 1909 FLOOD

The effect of large floods on velocities in an uncontrolled canal has been demonstrated in the hydraulic model of the sea-level canal by superimposing the flood of December 26–30, 1909, upon the conditions producing maximum tidal flow. This flood, largest of record on the Chagres River at Alhajuela, was extended with suitable adjustments to cover the entire watershed. The net rates of flood inflow into the canal were determined for two degrees of control: Case I, with only Madden Lake in operation (393 sq miles controlled), and case II, with a comprehensive flood-control system in operation (1,240 sq miles controlled). Fig. 14 summarizes the test results for case I and includes a comparison with tide flow only. Flows were introduced in the canal at four locations corresponding to the main tributaries, those entering the canal at Gamboa being modified to reflect the control afforded by Madden Dam. The tests were run without tidal regulation. The profiles of Fig. 14 demonstrate that the gradient of flow is materially flattened upstream from the point of largest inflow. This flattening tends to retard the entrance of tidal flow into the canal and, as a consequence, the combination of tide and flood flow would be less than the arithmetic sum of the separate flows. Maximum combined velocities observed during these tests (open canal; 20-ft tide) were as follows:

Condition	Velocities (knots)
Case I, flood control	7.0
Case II, flood control	5.0
No flood inflow	4.5

The high velocity of combined flow given by case I and the virtual restoration of the combined velocity in case II to the value for natural tidal flow are indicative of the requirement and effect of a large measure of flood control.

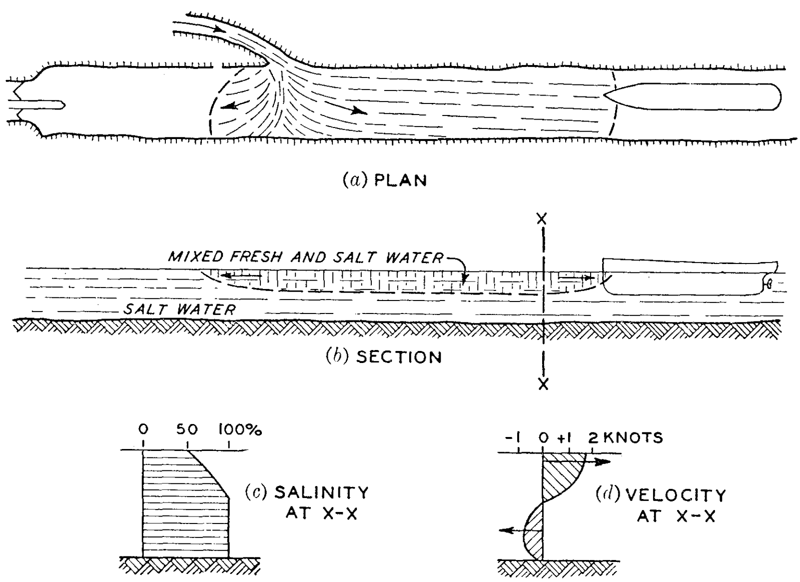

Fig. 15.—Inflow Tests, Existing Panama Canal

Density Currents in the Sea-Level Canal.—The foregoing tests were performed using fresh water for both flood and tidal flow and hence do not fully represent the conditions that would arise in nature. From observations in tidal estuaries, it is known that fresh-water and salt-water flows tend to retain separate identity, the lighter fresh water, of course, remaining on top of the salt water. Recent full-scale measurements have been made of fresh water from a tributary stream flowing into the salt water of the sea-level approach channel to the Miraflores Locks. The results of these field tests are summarized in Fig. 15.

Fig. 15(*a*) indicates the pattern of currents near the surface, and the movement of the fresh-water front as it spreads out over the salt water, The successive locations of this front were observed with electrical salinity meters.

The water near the surface of the canal was not wholly fresh water but a mixture of fresh and salt, starting with about 50% salinity at the top and increasing to 100% salinity at about half the channel depth. The water moved in one direction near the surface and in the opposite direction near the bottom. For a tributary inflow of 24,000 cu ft per sec, the velocities in the canal averaged 1.8 knots in the upper zone—or approximately three times the velocities that would have occurred had the discharge over the full cross section been uniform.

Summary of Requirements.—It is difficult to define specific criteria for flood control, since the necessity for control is measured by the safety and efficiency of navigation in the canal, the standards for which are largely qualitative As a result of the tests and studies performed on flood currents in the canal, however, little doubt exists that the plan of control should be comprehensive, approaching virtual elimination of flood disturbances.

The Drainage Basin

Topography.—The principal subdivisions of the Panama Canal drainage area are illustrated in Fig. 13. The watershed is characterized by conical hills of relatively low relief in the vicinity of the canal, changing to irregular knife-like ridges in the upper regions. Peak elevations of from 500 ft to 1,000 ft above sea level near the canal increase to 2,000 ft in the Gatun River basin, 2,500 ft in the upper watershed of the Chagres River, and 3,500 ft in the headwaters of the Trinidad River. The general steepness of the watersheds produces a rapid concentration of runoff.

Surface Cover.—In the thickly-forested areas of the upper regions, dense foilage prevents sunlight from reaching the ground, and consequently the ground cover is sparse and offers little deterrent to rapid runoff. In regions of lower elevation, the forest cover is less dense, the undergrowth is thick, and open areas are covered with a rank growth of grass, causing rainfall losses and the retardation of runoff to be more pronounced.

Climate.—The Canal Zone region has a distinct seasonal variation of rainfall, a uniform air temperature, and a high relative humidity. The dry season generally extends from January through April, and the wet season spans the remaining 8-month period from May through December. The mean monthly air temperature in the Canal Zone varies only from 2° F to to 3° F for the entire year, and the range between extreme temperature is less than 40° F. The highest recorded temperature is 98° F and the lowest is 59° F, both occurring at Madden Dam.

Rainfall.—The mean monthly rainfall at the Atlantic and the Pacific ends of the canal is shown in Fig. 16. Approximately 92% of the mean annual rainfall occurs during the wet season, and 16% of the annual rainfall occurs in November. All major floods in the Canal Zone region have occurred in the last 3 months of the year.

Fig. 17 is an isohyetal map of the mean annual rainfall on the canal drainage area. Note the gradation of rainfall across the Isthmus, ranging from depths

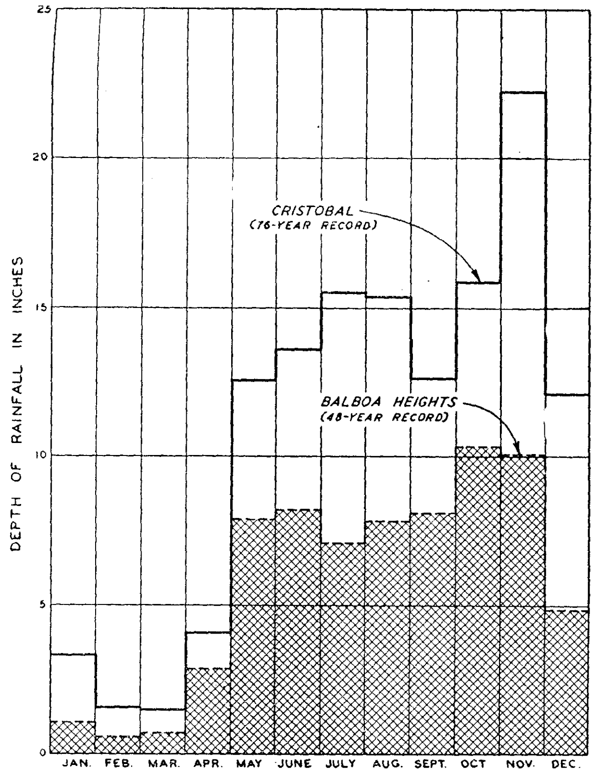

FIG. 16.—MEAN MONTHLY RAINFALL

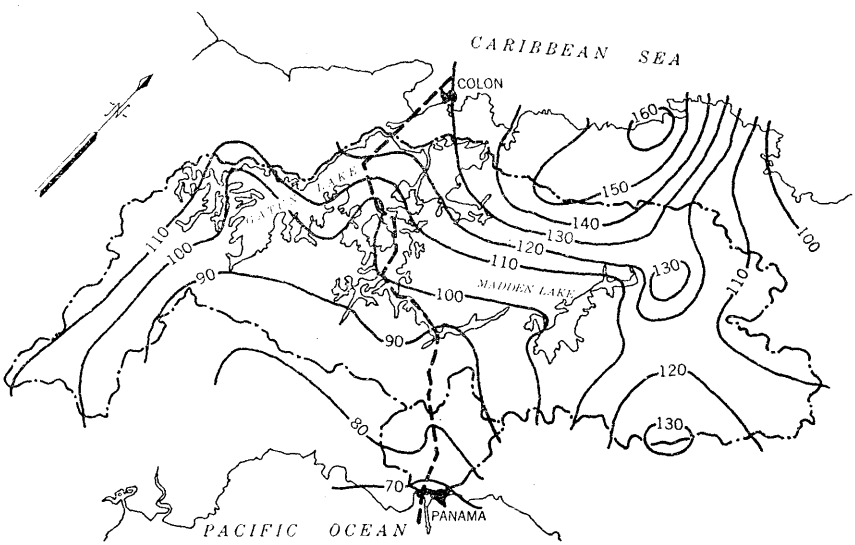

FIG. 17.—ISOHYETAL MAP OF MEAN ANNUAL RAINFALL

of 160 in. on the Atlantic coast to 70 in. on the Pacific coast. This gradation is also characteristic of monthly rainfall and occurs during many of the major flood-producing storms.

A large part of the rainfall results from convective activity in which the areal distribution is limited, the duration is a few hours, and the intensity is relatively high. Large flood-producing storms result from frontal activity and generally extend over the entire watershed of the canal. Rainfall caused by frontal activity usually has a lower intensity but may continue intermittently for a week or longer in storm periods having a succession of frontal passages. Maximum point rainfall in the Canal Zone region and at several locations in the United States is shown in Table 14.

TABLE 14.—MAXIMUM POINT RAINFALL; CANAL ZONE REGION AND UNITED STATES

(Rainfall Depth in Inches)

Duration	Canal Zone and vicinity	First Order Weather Bureau Stations					
		Los Angeles, Calif.	New Orleans, La.	Key West, Fla.	St. Louis, Mo.	Washington, D. C.
5 min.	0.90	0.44	0.77	0.65	0.60	0.80
10 min.	1.76	0.66	1.20	1.03	1.04	1.21
1 hour	5.68	1.51	3.66	4.30	3.47	3.42
24 hours	13.62	7.36	14.01	13.54	8.78	7.31
Annual	237.3	40.3	85.7	58.5	68.8	61.3

Rainfall and Discharge Records.—Heterogeneous rainfall and stream-flow records are available to the hydrologic investigator, some dating back to the 1860's. The primary objective of past observations has been the collection of data for definition of the dependable supply of water for lockages rather than for the development of flood-control projects. Although the number and distribution of rainfall stations in operation in recent years, particularly since 1941, have been sufficient to approximate a pattern of the mean monthly and mean annual rainfalls on the watershed, the records are insufficient for accurate study of storm and flood relations in any tributary area. The best records of discharge are those of the upper Chagres River where observations were started at Alhajuela in 1899 and have been continued to the present time, although the construction of Madden Dam in 1943, immediately upstream, prevented further recording of natural flood discharges at this station. Five rainfall stations have been in operation above Alhajuela since the early 1930's and three more were added in 1941. The general inadequacy of the records has made it necessary to develop data for the hydraulic design of the various projects largely from the Chagres River records.

Comparative Distribution of Rainfall.—Table 15 shows the comparative distribution of rainfall on the major drainage areas of the watershed. The comparison was made for four periods—the annual; the combined flood season months of October, November, and December; the month of November alone; and the average of twenty-two storms. For easy comparison, the values for the Chagres River above Madden Dam have been equated to 100% in each column. This study was undertaken to develop adjustment factors for transfer of storm and flood data from the Chagres River above Madden Dam to other areas. The values for the mean November rainfall were selected because most large floods have occurred during that month and because the November ratios would produce a larger design flood on the other areas.

TABLE 15.—Areal Distribution of Rainfall
(Rainfall Depth in Inches)

Drainage area	Mean Annual Rainfall		Mean Flood Season Rainfall (October–November–December)		Mean November Rainfall		Average Rainfall of 22 Storms (1941–1946)	
	Average depth	%	Average depth	%	Average depth	%	Average depth	%
Chagres River above Madden Dam......	116.0	100	45.0	100	15.9	100	4.2	100
Chagres River, Madden Dam to Gamboa..	100.9	87	36.7	82	14.8	93	3.4	81
Gatun River..........	131.0	113	51.5	114	22.4	141	4.9	117
Caño Quebrado River..........	90.4	78	34.0	76	14.5	91	3.2	76
Trinidad River..........	103.7	89	41.4	92	14.6	92	3.1	74
Entire Watershed..........	109.0	94	42.3	94	16.0	101	3.8	90

Major Storms and Floods on the Upper Charges River.—Since the beginning of authentic records in 1899, the greatest peak discharge of the Chagres River at Alhajuela was 140,000 cu ft per sec (356 cu ft per sec per sq mile), recorded during the storm of December 26-30, 1909. The second largest peak discharge was 129,000 cu ft per sec (328 cu ft per sec per sq mile), recorded at Alhajuela on December 3, 1906.

The largest volume of runoff in a single flood period was produced during the storm of November 12–23, 1935, and averaged 36.8 in. above Alhajuela. Summary data of the floods described are given in Table 16.

TABLE 16.—Major Floods of Chagres River at Alhajuela
(393 Sq Miles)

Date of flood	Rainfall		Peak Discharge		Meyer[a] rating	24-Hour Volume	
	Period (days)	Average depth	Cu ft per sec	Cu ft per sec per sq mile		Day-sec-ft	In.
December 26–30, 1909..	5	14.8[b]	140,000	356	71	94,600	9.0
December 3, 1906.....	1	5.2[b]	129,000	328	65	77,800	7.4
November 12–23, 1935.	12	36.8	....[c]	....[c]	....[c]	52,200	5.0

[a]Expressed in percentages, in which 100% equals 10,000 times the square root of the drainage area. [b] Rainfall observed at Alhajuela. [c] Construction of Madden Dam in 1934 prevented further recording of peak discharge.

Flood Frequency.—The occurrence of Chagres River floods since the beginning of records in 1899 is plotted with respect to the magnitude of peak discharges and 24-hour discharges in Fig. 18. Computations of occurrences have been made by arranging the floods in decreasing order of magnitude and dividing the rank of the flood into the years of record. This assigns a frequency to the maximum value equal to the period of record.

Rainfall-Runoff Relations.—In examining past records, it was noted that the recorded runoff was frequently much more, and occasionally less, than the

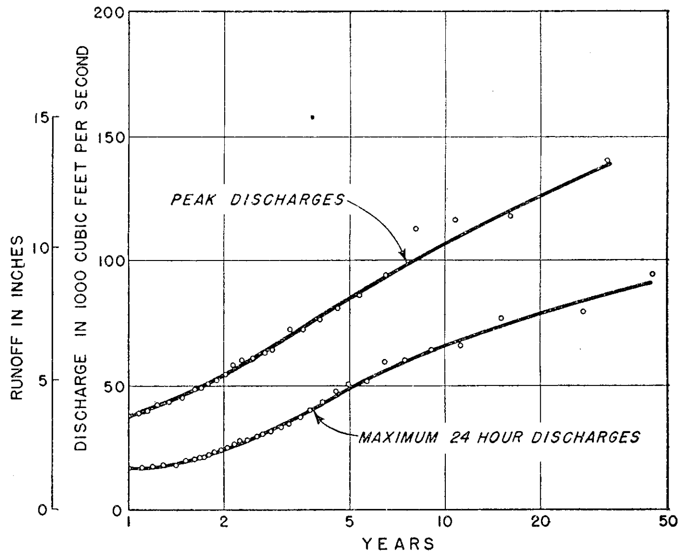

FIG. 18.—FLOOD OCCURRENCES, CHAGRES RIVER AT ALHAJUELA

corresponding recorded rainfall, presumably the result of an insufficient number of stations. As a consequence, rainfall losses were not directly determinable from observed data, and large adjustments of observed rainfall were frequently required for a satisfactory reproduction of recorded flood hydrographs in unit hydrograph verification studies.

PROJECT DESIGN FLOOD

Hydrographs of two extended flood periods yielding the largest volumes of runoff on the Chagres River at Alhajuela are shown in Fig. 19. These two floods were studied for the purpose of developing a project design flood that could be used for selecting the storage capacities of reservoirs and the discharge capacities of outlets and diversion channels. The total period of the plotting is 50 days. The 1909 period has three distinct peaks, the last corresponding to the maximum of record cited in Table 16. The 1935 period has two distinct peaks, the first peak corresponding to the 12-day flood of Table 16. The accumulative runoff for the two flood periods is of the same order of magnitude, the highest being 75 in. Because of the relatively short period of record and the importance of positive elimination of flood interference in the canal, the volume of each of these floods was increased 25%. Routing of these floods through the Madden Lake reservoir demonstrated that the maximum storage requirements would result from the augmented 1935 flood. This flood therefore was adopted as the project design flood for the existing Madden Lake and as the general basis for design of projects on other large areas. The flood from the Madden Lake area to other areas was transposed by applying two ratios: (1) The direct ratio of the drainage areas; and (2) the ratio of the mean November rainfall of Table 15.

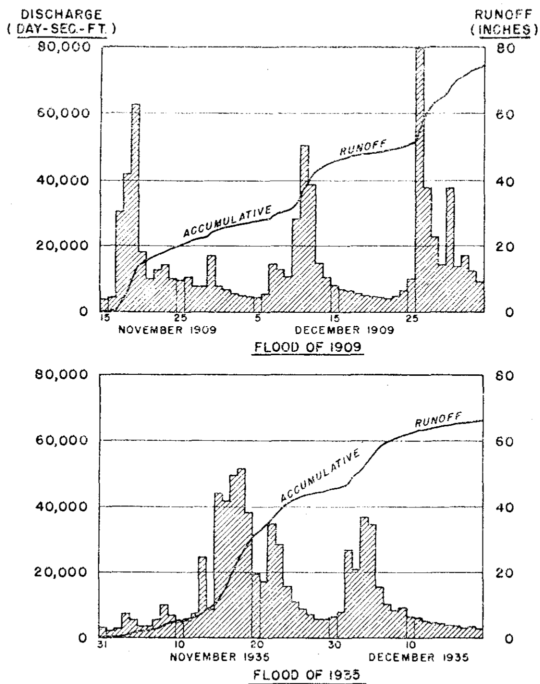

FIG. 19.—FLOOD HYDROGRAPHS, CHAGRES RIVER AT ALHAJUELA

Project Design Flood for Small Drainage Areas.—The proposed flood-control system includes the regulation of runoff on several small tributary areas of from 10 sq miles to 30 sq miles. To assure dependable control on such areas, the project design flood was selected to be 75% of the spillway design flood—a flood of shorter duration but higher discharge than the transposed flood of November, 1935.

SPILLWAY DESIGN FLOOD

The spillway design flood determines the size of spillways and the maximum water level in the reservoir and is derived from the estimated maximum possible storm for the watershed. This storm was established for the Madden Dam areas by the United States Weather Bureau in 1942 following an investigation of all the measurable meteorologic characteristics of the past storms that have produced heavy rainfall over or near the Canal Zone region. Depth-duration data of the storm for the Madden Dam area, extrapolated to apply to areas ranging from 10 sq miles to 600 sq miles, are shown in Fig. 20.

Development of the spillway design flood on the various watersheds required (1) selection of applicable rainfall data from Fig. 20, (2) adjustment of selected rainfall using the ratio of mean November rainfall between the various large areas and the Madden Lake drainage area to permit transposition of data, and (3) conversion of adjusted rainfall into flood runoff using inflow unit hydrographs. On the small drainage areas of 30 sq miles or less, the rainfall values of Fig. 20 were used without adjustment since these values could result from an intense convective storm of short duration which would be of like magnitude in any region.

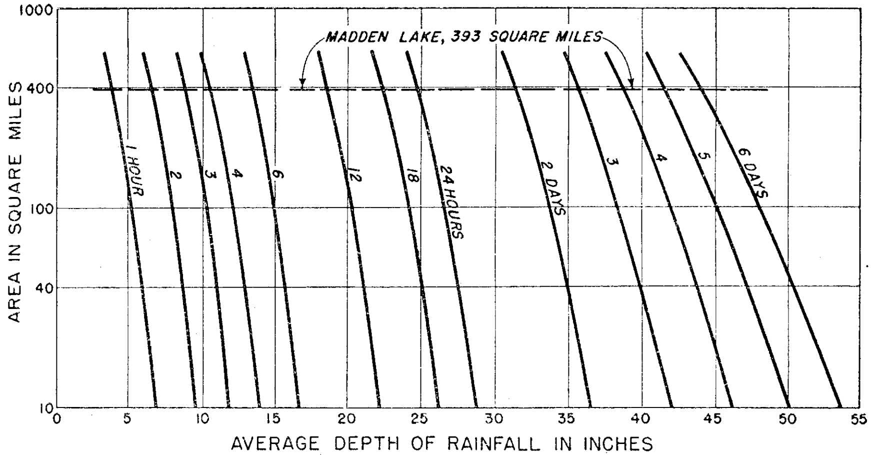

FIG. 20.—DURATION-DEPTH-AREA CURVES OF MAXIMUM POSSIBLE STORM, MADDEN LAKE

Inflow Unit Hydrographs for Reservoir Watersheds.—Unit hydrographs for converting storm rainfall into spillway design flood inflow for each flood-control project were constructed synthetically by the Snyder method.[14] General guidance for appropriate values of the empirical constants C_t and C_p, which

TABLE 17.—SNYDER'S CONSTANTS FOR ADOPTED UNIT HYDROGRAPHS, UPPER CHAGRES RIVER BASIN

Station	Stream	Drainage area (sq miles)	Unit time of rain (hours)	C_t	$640\,C_p$	Peak discharge, q_p (cu ft per sec per sq mile)
Alhajuela	Chagres	393	2	1.00	400	55
Chico	Chagres	160	1	0.40	500	192
Candelaria	Pequeni	52	1	0.45	600	290
Peluca	Boqueron	35	1	0.50	400	200
For general use	....	10–200	1	0.50	500	....

reflect the lag time and peaking characteristics of the watershed, respectively, was obtained by constructing unit hydrographs for four gaged areas above Alhajuela and verifying them by the reconstruction of twelve floods. The constants derived from the study are given in Table 17, and a typical example

[14] "Synthetic Unit Graphs," by Franklin F. Snyder, *Transactions*, Am. Geophysical Union, Vol. 19, Pt. I, 1938, p. 447.

of verification is shown in Fig. 21. The final inflow unit hydrograph constructed for Madden Lake, using constants summarized in Table 17, was verified by the reconstruction of a number of floods which have occurred on the watershed since the construction of the dam in 1934. An initial loss of

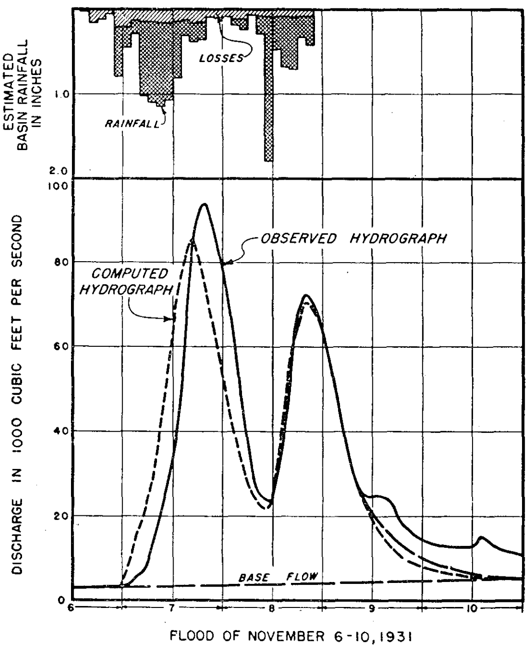

FIG. 21.—UNIT HYDROGRAPH VERIFICATION, CHAGRES RIVER AT ALHAJUELA, 1909 FLOOD

0.5 in. and infiltration at the rate of 0.05 in. per hr were deducted from rainfall in these studies, and a base flow of 10 cu ft per sec per sq mile was added to the computed inflow to complete the spillway design flood hydrograph.

THE FLOOD-CONTROL PLAN

Early French and American Plans.—The importance of controlling floods on the tributaries to a Panama sea-level canal was appreciated by both the French and American proponents of a sea-level canal. The French envisioned a dam at Gamboa for regulation of inflow into the canal from the Chagres River and diversion of the Gatun and Trinidad rivers in separate channels leading to the Caribbean Sea. In 1906, the majority group of the American Board of Consulting Engineers recommended the construction of a sea-level canal and proposed the following flood-control plan, similar to that of the early French planners:

> "To control the Chagres River, a dam * * * is proposed at Gamboa * * * forming a reservoir called Gamboa Lake, of which the maximum flow line is to be at elevation 170 * * *.

> "This dam is to be fitted with controlling sluices by which a maximum discharge of 15,000 cubic feet per second is to be admitted to the canal prism * * *. Of the tributaries entering the Chagres below Gamboa, the most important are diverted entirely from the canal and conducted by separate channels to the sea * * *.
>
> * * *
>
> "The Caño [Quebrado] and the Gigante [now included as part of the Caño Quebrado], * * * are to be cut off by dams. The Trinidad will occupy the old channel of the Chagres River and the Chagres diversion. The Gatun will be cut off from the canal by the partly finished Gatun diversion, * * *."

Development of Present Plan.—The physical possibilities for developing flood control for a Panama sea-level canal are abundant and variable. Control could be established by a system of reservoirs located in the lower central part of the main drainage basins, similar to the location of Madden Lake on the Chagres River Valley above Gamboa. Inflow into the canal from such a system would be composed of regulated flood releases from the reservoirs and unregulated runoff from areas below the reservoirs. A study of stream-flow records on the Upper Chagres River indicated that this system of control would permit a peak discharge of 45,000 cu ft per sec to enter the sea-level canal at Gamboa on the average of once in 5 years. This rate of inflow is considered to be from two to three times greater than the maximum that could be tolerated without seriously disturbing navigation. It was concluded that an effective plan of control by reservoirs would require location of dams close to the sea-level canal to prevent runoff from areas of even relatively small size from entering the canal directly.

Diversion of runoff from tributary areas, particularly for the larger streams, would obviously afford the most satisfactory method of preventing floods from interfering with navigation. The large drainage areas lie on the north or Atlantic side of the Continental Divide and, if uncontrolled, would discharge into the canal at distributed locations from Gamboa to Gatun (Fig. 13).

The two main tributaries on the west side, the Caño Quebrado and the Trinidad rivers, could be diverted by excavating a relatively shallow channel in the broad valley of the old Chagres basin, generally paralleling the sea-level canal and connecting with the former channel of the Chagres River below Gatun Dam. Another west side diversion plan would employ material available from sea-level canal excavation for the formation of a series of low dams between west side islands in Gatun Lake to create a diversion reservoir of moderate depth in which a permanent pool could be maintained for the general benefit of better sanitation and access in the region. The material from canal excavation would be available for embankment construction at little or no extra cost because large amounts would be barged to the lake for economical disposal regardless of flood-control requirements.

Control of the two principal tributaries on the east side, the Chagres and Gatun rivers, could be effected generally by either of two plans: (1) By diverting both tributaries, or (2) by diverting Gatun River to the sea and controlling the Chagres River by construction of a regulating reservoir at Gamboa as proposed in the French and American plans. Complete diversion would in-

TABLE 18.—PERTINENT D

Line	Reservoir	River	Drainage area (sq miles)	Elevations, Pacific Level Datum (Ft): Normal water surface	Spillway crest	Top of dam	Design Flood: Project	Spillway	Peak outflo (cu f per se
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)
1	Madden (existing) . .	Chagres	393	210	250[b]	273	239.8	262.9	30,00
2	Gamboa	Chagres	127	95	140	174	129.0	158.8	38,20
3	Monte Lirio	Gatun	180	55	55	92	61.5	82.2	72,00
4	Trinidad	Trinidad and Caño Quebrado	488	55	55	82	58.4	71.3	58,00
5	Madinga	Mandinga	10.5	150	208	235	207.2	223.7	2,97
6	Cocoli	Cocoli	11.5	70	107	135	106.6	122.4	3,13
7	Miraflores	Five small rivers	30.0	70	90	115	89.8	102.3	5,89

[a] Peak outflow, project design flood, in cubic feet per sec

clude a reservoir at Gamboa, a 13.7-mile diversion channel from Gamboa northward to the Gatun River Valley, and a second diversion channel 4.7 miles long from the north rim of the Gatun Valley to the sea. This plan is similar to the one proposed by John G. Claybourn,[7] M. ASCE. Under plan (2), the 13.7-mile diversion channel would not be constructed and the capacity of the reservoir on the Chagres River at Gamboa would be increased to reduce further the reservoir outflow, which in this case would enter the sea-level canal. In either system, Madden Lake would be operated largely for flood control of the upper Chagres River. The plan of complete diversion is considered decidely superior because it would avoid a prolonged inflow of regulated releases at Gamboa during the wettest months. It would be difficult to reduce these releases sufficiently to prevent possible disturbances from crosscurrents or eddies formed by the interflowing of masses of salt water and fresh water.

The Adopted Flood-Control Plan.—The adopted plan of flood control embodies (*a*) complete diversion of all runoff from the major tributaries, and (*b*) control by regulating reservoirs on several small streams entering the canal south of Gamboa. The plan is comprehensive, since less than 9% of the entire watershed would remain uncontrolled. The largest uncontrolled area would not exceed 4.5 sq miles. The location of the flood-control projects and a summary of the areas controlled are shown in Fig. 11. The diversion of major streams lying on the Atlantic side of the Continental Divide and draining 87% of the watershed is accomplished by two distinct projects designated the East Diversion and the West Diversion. General descriptions of the projects and their operation are outlined in the paragraphs following, and statistical data pertaining to the reservoirs and dams are given in Table 18.

East Diversion

Madden Lake.—The storage in Madden Lake[15] now used principally for power generation and lock water supply would be reallocated, approximately

[15] "More Water for the Panama Canal," by E. S. Randolph, *Civil Engineering*, May, 1932, pp. 283–288.

—FLOOD-CONTROL RESERVOIRS

Available Storage Below Spillway Crest		Embankment Dimensions (Ft)			Uncontrolled length of spillway (ft)	Gated Outlets		Ungated Outlets		Line
Acre-ft	In.	Crest length	Crest width	Crest height		No.	Size (ft)	No.	Size (ft)	
(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	
382,000[b]	18.2[b]	5,700	22	183	(4–100)[c]	6	5.67 by 10	2	7-ft needle valves	1
357,000	52.7	3,900	100	129	1,000	1	Diameter, 38 ft	1	Diameter, 45 ft	2
..	..	23,000	100	82	600	5	20 by 20	..	......	3
..	..	47,000	50	72	500	3	20 by 20	..	......	4
6,000	10.7	1,200	50	117	100	1	Diameter, 10 ft	1	Diameter, 9 ft	5
6,820	11.1	1,000	50	95	100	1	Diameter, 11 ft	1	Diameter, 10 ft	6
18,500	11.6	6,500	50	85	200	2	Diameter, 11 ft	2	Diameter, 10 ft	7

[b] Top of 18-ft crest gates. [c] Four, 100-ft crest gates.

60% being assigned to flood control and 40% to power. The flood-control storage would be equivalent to a runoff depth of 18.1 in. on the 393 sq miles of tributary watershed. Regulated discharges up to a maximum of 30,000 cu ft per sec would be released from Madden Dam. The project design flood would utilize two thirds of the assigned flood-control storage, the remainder constituting a reserve for such contingencies as silting and possible changes in operating procedure.

Gamboa Reservoir.—This reservoir would receive discharges from Madden Lake and runoff from the 127 sq miles of drainage area below Madden Dam. The normal level of the reservoir would be El. 95, 10 ft above the present Gamboa arm of Gatun Lake. The project design flood would raise the water level in Gamboa reservoir to El. 129, leaving approximately one third of the storage below El. 140.0, the crest of the spillway, available as a reserve for operating contingencies. Outflow from this reservoir would be diverted northward via the Chagres River diversion channel to the Monte Lirio reservoir, third and last reservoir in the East Diversion. Any discharge that might occur over the Gamboa spillway would enter the abandoned channel of the existing canal and would flow into the sea-level canal. The top of the Gamboa Dam, an earth and rock embankment (Fig. 22) would be 125 ft above the river bed and the crest length would be 1,800 ft. A 38-ft-diameter tunnel would be constructed through the south abutment for diversion of the Chagres River during construction of the dam and would be retained for emergency release of flood discharges into the sea-level canal under remote contingencies such as closure of the Chagres diversion channel by slides.

Chagres Diversion Channel.—Diverted flows would leave the Gamboa reservoir through a 45-ft-diameter tunnel, 2,700 ft long, and thence through an open channel, 13.7 miles long, varying in width from 100 ft in the sections of deeper cut to 500 ft in the low areas (Fig. 23(*b*)). As shown in Fig. 23(*c*), two weirs would be constructed in the diversion channel—the upper, with crest at El. 95, would be located 6 miles below the tunnel outlet and would control the normal level of the Gamboa reservoir; and the lower, with crest at El. 70, would be located 6 miles farther downstream. These weirs would keep the bottom of the channel submerged in the dry season and thus prevent inception of natural growth. Discharge from the Gamboa reservoir would be controlled

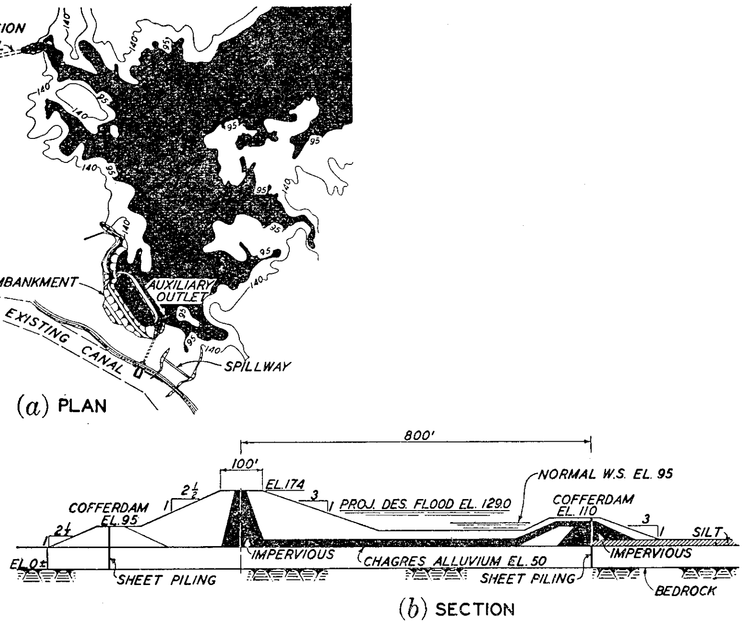

FIG. 22.—PLAN AND SECTION, GAMBOA DAM

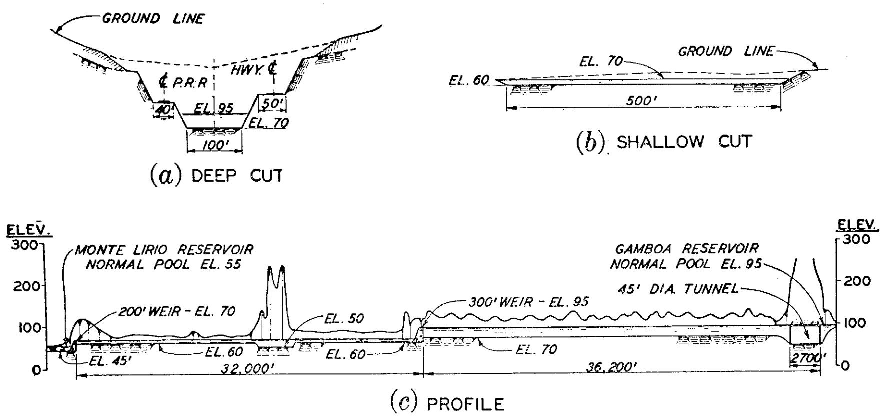

FIG. 23.—SECTIONS AND PROFILES, CHAGRES DIVERSION CHANNEL

by the natural capacity of the diversion channel and upper weir at low reservoir stages and by the capacity of the tunnel at higher stages. Valleys of existing streams crossed by the diversion channel would require nominal diking on the sea-level canal side.

Monte Lirio Reservoir.—Monte Lirio reservoir would occupy the lower valley of the Gatun River now inundated by the eastern arm of Gatun Lake and would receive flows diverted from the Gamboa reservoir, runoff from streams intercepted by the Chagres diversion channel, and flow from the Gatun River. The normal level of the reservoir would be maintained at El. 55, 30 ft below the present level of Gatun Lake, a level which is suitable for domestic water supply, area sanitation, and hydroelectric power development. The Monte Lirio Dam (Fig. 24(*a*)) would be formed by strengthening the existing Panama Railroad embankment which extends between several islands across the eastern part of Gatun Lake for a total distance of 4.4 miles.

All flows received in the Monte Lirio reservoir would be diverted to Las Minas Bay on the Caribbean coast through a channel 4.7 miles long (Fig. 24(*b*)). The diversion channel would have a width increasing from 200 ft at

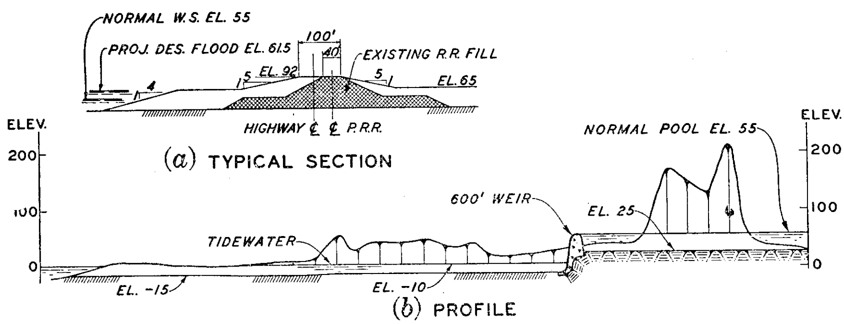

FIG. 24.—MONTE LIRIO DAM AND DIVERSION CHANNEL

the reservoir to 1,000 feet near Las Minas Bay. A 600-ft spillway weir with crest at El. 55 and an outlet structure containing five gates would be constructed in the channel 1 mile from the reservoir.

The outlet gates would be operated to maintain the water surface at El. 55 in the Monte Lirio reservoir and, during the passsing of a flood, would be fully open when the reservoir level rose above El. 55. The project design flood would raise the water surface of Monte Lirio reservoir to El. 61.5.

WEST DIVERSION

Two interconnected reservoirs, the Caño Quebrado reservoir and the larger Trinidad reservoir, with normal water surfaces at El. 55, would comprise the West Diversion (Fig. 11). Discharge from the Caño Quebrado reservoir would flow through an uncontrolled channel circling the south and west sides of Barro Colorado Island, and would join the Trinidad reservoir near the existing Gatun Dam. A 500-ft spillway with crest at El. 55 and an outlet structure containing three gates constructed in the west abutment of the existing Gatun Dam (Fig. 25(*a*)) would serve both reservoirs and would be regulated in the same manner as the Monte Lirio spillway and outlet structure. Discharges would flow to the sea through the old channel of the Chagres River. The project design flood would raise the water surface in the reservoirs to El. 58.4.

The dams for the West Diversion (Fig. 25(b)) would consist of low embankments placed on broad spoil areas of sea-level canal excavation. The surface of the spoil would be at El. 65 and the crests of the embankments would be at El. 82. The dams would extend from Gatun Dam to a point 4 miles north of Gamboa and would connect numerous islands in the present lake.

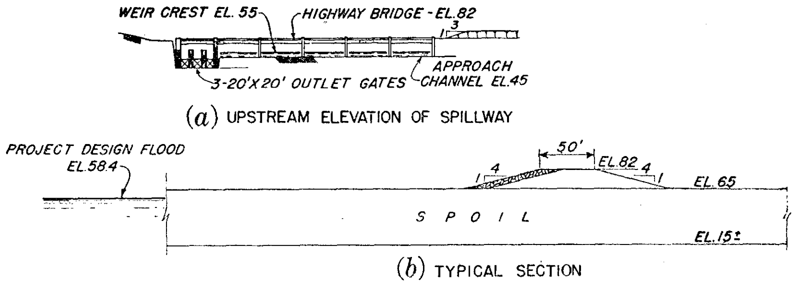

FIG. 25.—TRINIDAD AND CAÑO QUEBRADO DAMS

CONTROL OF SMALL TRIBUTARIES

Three retarding reservoirs on small tributary areas between Gamboa and Balboa Harbor (Fig. 11) are included in the flood-control plan. These are designated as the Mandinga, the Cocoli, and the Miraflores reservoirs, and control areas of 10.5 sq miles, 11.5 sq miles, and 30.0 sq miles, respectively. The last consists of five interconnected small basins with two outlet channels. Each reservoir would have a spillway and both uncontrolled and gated outlets. The maximum discharge into the canal at any one point would approximate 3,000 cu ft per sec.

FOUNDATIONS AND CONSTRUCTION MATERIALS

Preliminary explorations have revealed that sound rock is available at suitable depths in all foundation areas for spillway and outlet structures and in areas of tunnel excavation. Likewise, no major problems are anticipated in excavation of diversion channels. Most of the 13.7-mile diversion channel from Gamboa to Monte Lirio lies in regions of hard rocks of volcanic origin and medium-hard siltstones and limestones. The channel from the Monte Lirio reservoir to the sea would be located largely in sound siltstone and sandstone of the Gatun formation. The river section of the Gamboa Dam would be founded on alluvium, 50 ft thick, composed largely of sand and gravel. Measures for watertightness and safe relief of seepage would be incorporated in the design of the dam, as illustrated in Fig. 22(b).

Parts of the embankments for the Monte Lirio, Trinidad, and Caño Quebrado dams would be founded on a relatively soft alluvial deposit of organic clays, silts, and sands that is termed Atlantic muck. Fortunately, the great abundance of spoil material from the sea-level canal excavation would permit Caño Quebrado and Trinidad dams to be founded entirely upon a broad terrace of sound excavation spoil extending not less than 1,500 ft on both sides of the center line—with a top at El. 65, only 17 ft below the crest of the dams.

The Monte Lirio Dam, which would be an enlargement of the railroad causeway completed in 1912 across the eastern arm of Gatun Lake, would be similarly reinforced with broad berms for distribution of load.

Suitable and extensive deposits of gravel are available near the Gamboa dam site, and impervious, random, and rock-fill materials for the Gamboa Dam would be obtained from spillway excavation. Fill for embankments in the lake area would come mainly from canal excavation transported by barge.

Construction

Flood-control construction would be scheduled to utilize the material from canal excavation as it becomes available and to avoid unnecessary peaks in personnel. None of the structures would be required to function during the canal construction period, but Madden Lake would be assigned largely to flood control during the construction of the Gamboa Dam, the only project requiring major cofferdam construction and river diversion. The earth-fill cofferdams for the Gamboa Dam would form a part of the permanent structure.

Operation of Flood–Control Projects During Final Lowering of Lakes

All flood-control projects would be placed in operation prior to the rapid lowering of Gatun and Miraflores lakes upon conversion of the canal to sea level. More than 68% of the normal volume of Gatun Lake would be cut off by flood-control dams and thus the time of emptying the central region of the lake would be greatly accelerated. The total interruption to traffic at this time would be less than 7 days, including the time required for removing the lake-retaining barriers in the channel for the sea-level canal.

Hydroelectric Power

Hydroelectric power is generated in the Canal Zone at Gatun and Madden dams, which have an installed capacity of 46,000 kw. The Gatun station would be abandoned on conversion of the canal to sea level and, as noted previously, the storage assigned to power in Madden Lake would be reduced to one third of the total storage, sufficient to produce 13,000 kw at a 40% load factor. Opportunities would be available at the Monte Lirio and Trinidad spillway structures for small hydroelectric installations. The plant at Monte Lirio, receiving regulated flow from Madden Dam and unregulated runoff from the remaining watershed of 307 sq miles, would have a firm capacity of 13,500 kw at a 25% load factor. The Monte Lirio reservoir would not be regulated for power because drawdown of the normal water surface would affect the quality of water for domestic supply adversely. The Trinidad and Caño Quebrado reservoirs would be regulated between El. 55 and El. 45, permitting the generation of 7,500 kw of primary power at a load factor of 50%.

Safety of Structures During Wartime

The dams in the lake area would actually be low levees ranging from 17 ft to 27 feet in height constructed on broad spoil terraces at El. 65. These teraced areas would be virtually unbreachable by bombing of any type and a breach in the levees would be of no consequence since the occurrence of a flood even as large as the project design flood would not raise the water level in either the East Diversion or the West Diversion above El. 65. If the spoil terraces were ever breached, the outlet conduits in the Trinidad and Monte Lirio spillway structures could be left open for continued diversion of flow to the sea with only minor repair of the breach. A breach in Madden Dam would not result in overtopping the Gamboa Dam even if both reservoirs were filled to spillway levels at the time—an extremely remote possibility. The Gamboa Dam would be difficult to breach by any conceivable conventional bomb, as it would be 350 ft wide at the level of the project design flood and 650 ft wide at normal water surface.

Summary

The control of floods on the tributaries of a Panama sea-level canal, in accordance with the adopted system, would completely eliminate hazards to shipping from flood inflows. The flood-control system would have sufficient capacity to control flows well in excess of the largest flood that has occurred in 47 years of record. Runoff from 87% of the area tributary to the sea-level canal would be diverted directly to the Atlantic Ocean and 4% would be controlled by retarding reservoirs. It would be difficult to breach the flood-control dams adjacent to the canal in wartime. If breached, only minor remedial work would be needed to restore the system to effective operation.

TIDAL CURRENTS

By J. S. Meyers,[16] and E. A. Schultz,[17] Assoc. Members, ASCE

Synopsis

A current of about 4.5 knots, at the Atlantic end of the Panama Canal, is estimated both by computations and by hydraulic model tests as the maximum that would be caused in an open sea-level canal at Panama by a tidal range of 20 ft in the Pacific Ocean.

Channel roughness would affect the velocity of the reversing flow in this tidal canal in the same way as in any one-way channel since, at the time of maximum current, the situation is nearly that of ordinary steady flow between the extreme tide levels. Results of measurement of roughness in the existing canal and a summary of roughness data for other large channels are presented. A Manning n of 0.024 was selected for the investigations.

The hydraulic model of the sea-level canal, at 1:100 undistorted scale, is described briefly. Close agreement throughout the tidal cycle was obtained between the velocities measured in the model and those computed by the method developed by General Pillsbury.

An example of tidal flow as controlled by tidal-regulating structures is presented and discussed briefly.

Early Estimates of Tidal Currents at Panama

The currents that would be produced by the tides in a sea-level canal at Panama have been the subject of speculation and concern since the earliest canal proposals. The first careful study known to have been made was that reported on May 31, 1887, by a committee appointed by the French Academy of Sciences[18,19] at the request of Count Ferdinand de Lesseps, chief engineer of the French Isthmian Canal Company, who was undertaking to build a sea-level canal at Panama as he had already done so successfully at Suez. The French analysis was based on the use of the Chézy formula for steady flow, applied to a number of short reaches, with heads adjusted to take account of wave celerity. The committee reached the conclusions that the Atlantic tidal range was so small in comparison with the Pacific range that it could be disregarded. The estimated maximum tidal current for a canal 45 miles long was 2.5 knots.[20]

The Board of Consulting Engineers, Isthmian Canal Commission, reported in 1906: "It is probable that in the absence of a tidal lock the tidal currents during extreme oscillations would reach five miles per hour."[21] This is the

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[18] "The Cape Cod Canal," by William Barclay Parsons, *Transactions*, ASCE, Vol. LXXXII, 1918, p. 138.

[19] *Comptes Rendus*, French Academy of Sciences, Vol. 104, May 31, 1887, p. 1484.

[20] *Transactions*, ASCE, Vol. LVI, 1906, p. 211.

[21] "Report of the Board of Consulting Engineers for the Panama Canal, 1906," U. S. Govt. Printing Office, Washington, D. C., 1906, p. 56.

equivalent of 4.3 knots. The channel to which this estimate applied was about 40 miles long and 40 ft deep, with bottom width at 150 ft in earth and 200 ft in rock. The method used to obtain this velocity was not stated.

An estimate of 2.6 knots for the mean cross-sectional velocity in a sea-level canal was made by the United States Coast and Geodetic Survey (in a letter to the Panama Canal, dated February 3, 1924), for a channel 40 miles long, 1,000 ft wide, and 50 ft deep. The Eytelwein formula was used to compute the steady-flow velocity that would be produced by an 11-ft difference in head. It was stated that the velocities computed by this formula agreed reasonably well with observed values in the Cape Cod Canal in Massachusetts, which is about 7 miles long, but that progressive wave motions might be created in the greater length of a Panama sea-level canal which would tend to increase the computed value.

Tides at Panama

The tides at the Atlantic and Pacific termini of the Panama Canal are very different both in magnitude and general character. The Pacific tide at Balboa is remarkably regular, with two highs and two lows of almost equal magnitude occurring in every lunar day of 24 hours and 50 min. Extreme tides have reached levels 10.8 ft above and 11.9 ft below mean sea level. The maximum range between consecutive tides, however, has only occasionally exceeded 20 ft, as indicated by the following percentages of Balboa tides that reach different ranges:

Percentage of total tides	Tidal range (ft)
2	20
20	16
50	13
80	10
99	6

The Atlantic tide at Cristobal is irregular and much smaller than the Pacific tide, with no simple cyclic pattern. Extreme tides have reached 1.8 ft above and 1.25 ft below mean sea level. The mean tidal range is 0.9 ft and the minimum practically zero. Atlantic high tides precede Pacific high tides from zero hours to 6 hours, averaging about 3 hours. Mean sea level on the Pacific side of the Isthmus averages 0.77 ft higher than on the Atlantic side, but the Pacific mean level on individual days has ranged from 2.04 ft above to 0.68 ft below the Atlantic mean level.

Most of the analytical and model studies described in this paper were based on assumed maximum semidiurnal tides of 20-ft range at Balboa and of 2-ft range at Cristobal, with the Cristobal tide preceding the Balboa tide by 3 lunar hours, as shown in Fig. 26. (One lunar day is the time required for one revolution of the moon around the earth. It corresponds with 24 hours 50 min of ordinary solar time.) Selection of the essential features of a sea-level canal generally required consideration of the conditions produced by these maximum tides, but investigations were also made of the effects of average and low tides for some comparisons.

Because the Pacific mean sea level is 0.77 ft higher than the mean level on the Atlantic side of the Isthmus, a greater head and a steeper slope in a sea-level canal would normally be expected when the Pacific tide was at its highest point than when it was lowest. The maximum head in 20 years of record, however, was 11.6 ft from the Atlantic level down to a low Pacific tide level. The maximum head in the opposite direction, from a high Pacific level down to the Atlantic level was 11.3 ft. Since the tidal heads in the two directions are so nearly the same for maximum conditions, the convenient assumption of a common Atlantic and Pacific mean tide level was made for study purposes.

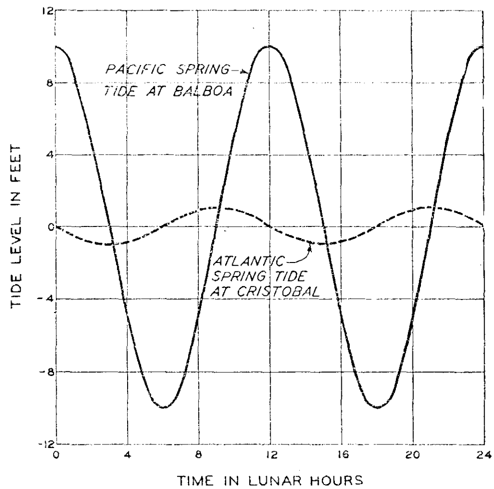


FIG. 26.—MAXIMUM TIDES USED FOR VELOCITY STUDIES

ANALYSES OF TIDAL CURRENTS FOR 1947 INVESTIGATION

Estimates of velocity for the proposed 60-ft by 600-ft sea-level canal have been made, using data and methods more complete than those available to earlier investigators. Most of the computations were made by the method developed by General Pillsbury for analysis of tidal flow.[8] By this method the channel is subdivided into a number of reaches, and flow in each reach is computed from an equation of motion in which the frictional resistance is expressed by the Manning, the Kutter, the Bazin, or any desired flow formula. Corrections are made for changes in storage, slopes, and other factors caused by the changing tides. The precision of the results depends on the number of successive adjustments. This method indicates that a maximum current velocity of 4.5 knots would be caused at the Atlantic end of the canal by the tides of Fig. 26 acting on an open sea-level canal with the Manning roughness assumed as 0.024. This velocity, and all others given in this paper, whether from model tests or computations, are mean velocities over the entire channel cross section.

A computation procedure suggested by Professor Bakhmeteff was also used for comparison. The basic assumption was that of steady flow between the maximum differences of water level shown in Fig. 26, with corrections for changes in acceleration, velocity head, and channel storages that would prevail in a tidal cycle. The estimated maximum steady-flow velocity was 4.6 knots, again using a Manning roughness of 0.024.

Tests on the 1:100 scale hydraulic model of the proposed sea-level canal channel, which is described subsequently in this paper, indicated a maximum current of 4.4 knots, for the same 20-ft and 2-ft tides. The roughness of the model channel corresponded to that expressed by a 0.024 Manning coefficient for the full-size channel.

During the preliminary discussions of these analyses and their implications, Professor Bakhmeteff stated that velocities for tidal flow cannot exceed those for steady flow. For the situation at Panama, he reasoned that the steady-flow velocities would be only slightly greater than the tidal velocities, since "preliminary calculations show that at periods of extreme flow about 90 percent of the actuating head is absorbed by friction resistance." The results of the tests on the sea-level model closely support that statement, indicating that channel roughness is a controlling factor in any estimate of tidal velocities in a canal as long as that at Panama.

Hydraulic Roughness of Large Channels

Observations made on open channels of moderate and small size have provided most of the data upon which the current understanding of flow is based. The flow formulas in common use were developed primarily from measurements made on small test channels by Bazin and other early experimenters. In some cases, they were developed with a view to their use for

TABLE 19.—Values of Manning's Roughness Coefficient for Large Channels at Most Efficient Depths

Line	Watercourse	Coefficient n	Notes on channel
1	Emory River	0.029	Earth and ledge rock.[a]
2	Tennessee River	0.026	Natural earth channel of lower Tennessee River.[b]
3	Colorado River	0.028	In newly excavated rock.[c]
4	Mississippi River	0.024 to 0.031	In alluvial valley.[d]
5	Atchafalaya River	0.024	In alluvial valley.[e]
6	Yazoo River	0.026 to 0.031	In alluvial valley.[f]
	Columbia River—		
7	McNary dam site near Umatilla, Ore.	0.030	Stable bottom of sand, gravel, boulders, and solid rock.
8	Central part of State of Washington	0.036	Little bank vegetation.
9	Cape Cod Canal	0.031 to 0.036	Subject to tidal currents.
10	Houston Ship Canal	0.025	Coefficient measured roughly.[g]
11	Panama Canal (Gaillard Cut)	0.026	In rock.[h]

[a] In backwater from Watts Bar reservoir on the Tennessee River below Knoxville, Tenn. [b] In backwater from Kentucky Dam near Gilbertsville, Ky. [c] Downstream from powerhouse at Parker Dam near Parker, Ariz. [d] This range of values was applicable to 1928–1930 measurements, omitting widely divergent values. [e] This range of values was applicable to 1929, omitting widely divergent values. [f] After clearing and snagging. Climate is conducive to vegetation. [g] This coefficient, although roughly measured, is considered applicable to design. [h] Observations in 1946, after channel had been submerged for 33 years.

closed pipes as well as open channels. Although the desirability of having the formula fit data for large as well as for small channels was recognized, notably by Ganguillet and Kutter, the information then available on large channels was meager and in some cases unreliable. Because of the importance of the roughness value in this Panama Canal study, special effort was made to obtain and compare information on large channels only, and to weigh the applicability of the different formulas to those conditions.

Roughness Data on Large Channels from Other Sources.—In response to requests made in 1946 by The Panama Canal to a number of agencies in the United States, information on hydraulic roughness of large channels was received from the Bureau of Reclamation, the Tennessee Valley Authority, and several offices of the Corps of Engineers. The data were for long reaches of natural and artificial channels with a considerable variety of slopes, discharges, depths, and bed material. The roughness coefficients have been averaged, and a summary for channels having hydraulic radii exceeding 20 ft and flowing nearly bankfull is given in Table 19.

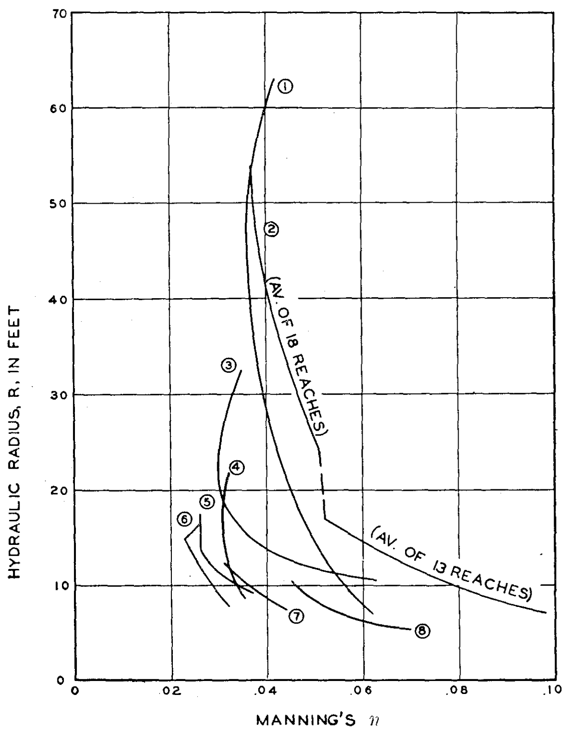
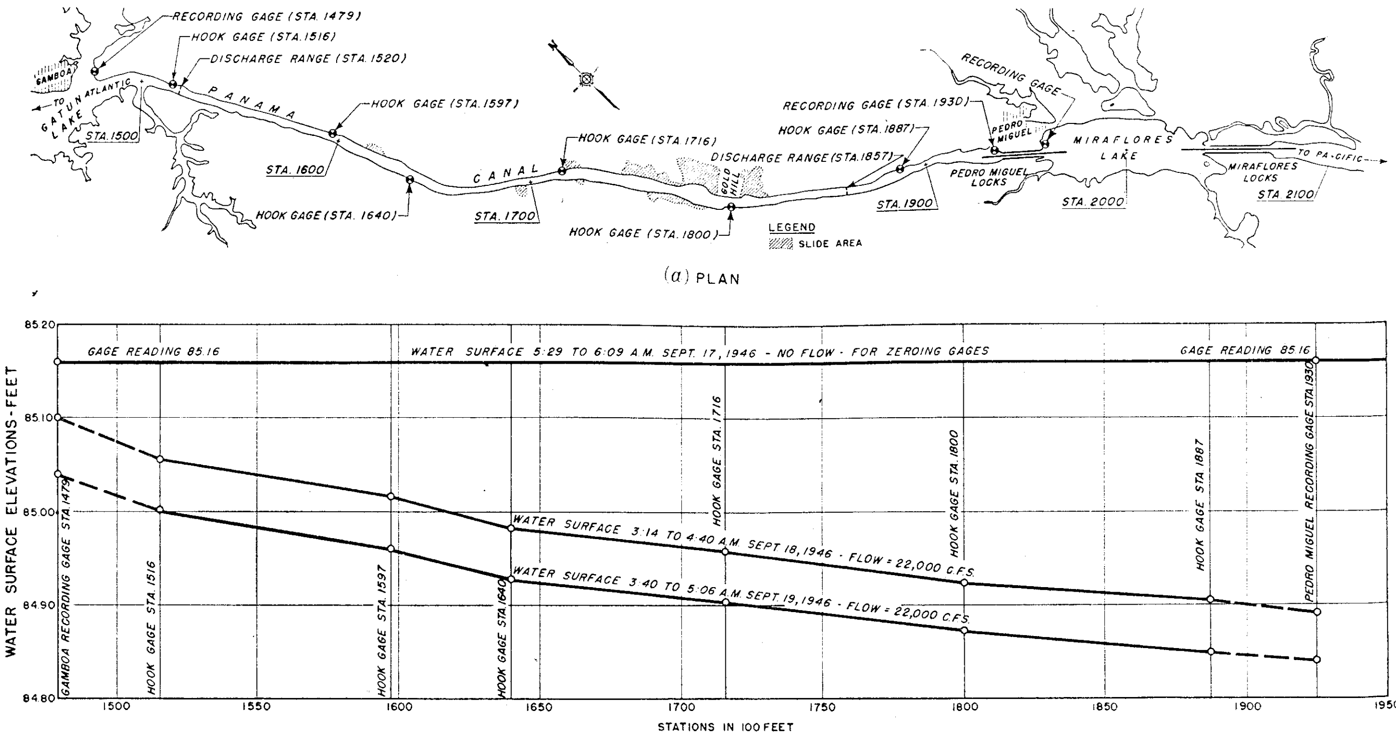


FIG. 27.—VARIATION OF MANNING'S n, WITH HYDRAULIC RADIUS

The two most important conclusions reached from a study of these data were:

a. The Manning coefficient for a river channel is least when the stage is at or somewhat above normal bankfull stage, and tends to increase for both higher and lower stages. This fact is indicated both by the Yazoo River in Mississippi, flowing in alluvium in an area where vegetation is rank, and by the Columbia River in Washington, with a rocky bed and practically no vegetation on its banks. The effect is shown in Fig. 27, the several curves being identified by numbers as follows:



(b) WATER SURFACE PROFILES

FIG. 28.—ROUGHNESS MEASUREMENTS IN GAILLARD CUT