TRANSACTIONS

OF THE

AMERICAN SOCIETY

OF

CIVIL ENGINEERS.

(INSTITUTED 1852.)

VOL. LVIII.

JUNE, 1907.

Edited by the Secretary, under the direction of the Committee on Publications.

NEW YORK:
PUBLISHED BY THE SOCIETY.

1907.

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Page 377, last line: For "hauls" read "banks."

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1038.

THE ATCHAFALAYA RIVER: SOME OF ITS PECULIAR PHYSICAL CHARACTERISTICS.*

By J. A. Ockerson, M. Am. Soc. C. E.

With Discussion by Messrs. Henry B. Richardson, T. G. Dabney, Frank M. Kerr, L. J. Le Conte and J. A. Ockerson.

As the Atchafalaya River, in Louisiana, differs in so many respects from the usual conception of streams, the writer believes that a general description of it will be of more than ordinary interest to the hydraulic engineer, as well as to the layman. It is a stream which is largest at its source and is deepest in places of excessive width.

It is an alluvial stream flowing between banks of its own formation, and having its source at an old bend of the Mississippi River at about latitude 31 degrees. At the present time it is an important outlet of the Mississippi. Opinions differ as to its origin. It is probable that, at one time, it was the bed of the Red River which flowed directly to the Gulf of Mexico, but which was made a tributary of the Mississippi River by the elongation of the bend of the latter which cut into the smaller stream and made the mouth of the Red River near where it is now located. Even be-

* Presented at the meeting of December 5th, 1906.

fore this occurred the Atchafalaya portion served as an outlet for some of the overflow waters of the Mississippi River.

At the head of the Atchafalaya, the entire volume of water is derived from the Mississippi and Red Rivers. Below the source, the drainage from the westward finds its way into the stream through numerous bayous which contribute a considerable amount to its volume. A very large percentage of the flow, however, is contributed by the two streams first named, which in a sense are, therefore, tributaries of the Atchafalaya.

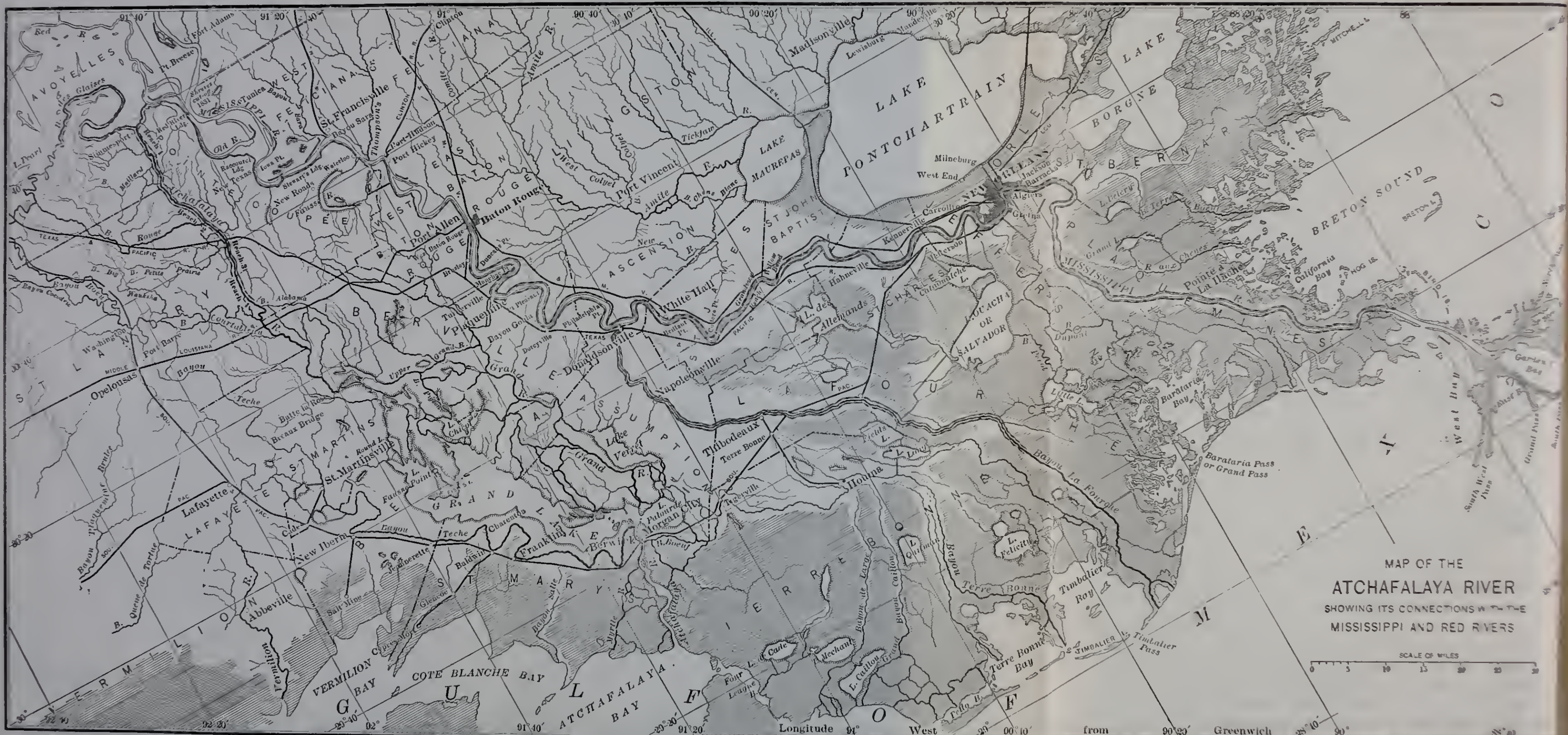
A maximum flood in the Mississippi River delivers to the Atchafalaya River more than 400 000 cu. ft. per sec. At low water little or nothing is derived from this source, and, in fact, the flow is often reversed, a portion of the waters of the Red River passing by the head of the Atchafalaya and flowing into the Mississippi. The low-water discharge of the Atchafalaya is sometimes less than 1 000 cu. ft. per sec., while, at high water, it serves as an outlet for about one-fourth of the flood volume of the Mississippi.

Measured along the channel of the Mississippi, the distance to the head of the Atchafalaya from the Gulf of Mexico is some 315 miles. Following down the latter stream, it is found that it reaches the Gulf in a distance of about 156 miles, at a point 125 miles west of the mouth of the Mississippi. It practically reaches tide water at Grand Lake, a distance of only about 100 miles from its source. (See Plate I.)

At the head of the stream the banks are 50 ft. above mean Gulf level, or 48 ft. above extreme low water, and rapidly decrease in height as one goes down stream. At Butte la Rose, 67 miles from the source, the banks are only 12 ft. above the same datum, giving a difference of 38 ft., or a ground slope of 0.57 ft. per mile.

The water surface slope for the same reach, at a stage near bank-full at the lower end, is 14.6 ft., or an average water slope at that stage of 0.22 ft. per mile.

The excess in the height of the banks near the source is readily accounted for by the fact that they are built up by the sediment carried in the water, which not only rises much higher at the upper end, but a greater proportion of the sediment is dropped in its first escape from the banks of the streams which are always submerged at high water.


MAP OF THE
ATCHAFALAYA RIVER
SHOWING ITS CONNECTIONS WITH THE
MISSISSIPPI AND RED RIVERS
SCALE OF MILES
LAKE PONTCHARTRAIN
LAKE MAUREPAS
LAKE BORGNE
BRETON SOUND
GULF OF MEXICO
ATCHAFALAYA BAY
COTE BLANCHE BAY
VERMILION BAY
Terre Bonne Bay
Timbalier Bay
Barataria Bay
New Orleans
Baton Rouge
Port Allen
Plaquemine
Donaldsonville
Thibodeaux
Houma
Morgan City
Berwick
Franklin
New Iberia
Abbeville
Lafayette
Opelousas
St. Martinsville
Napoleonville
White Hall
St. Francisville
Simmesport
Longitude West from Greenwich

At the source of the stream there is a maximum oscillation of stage, between high and low water, of 52.3 ft. A mile below the source, the stream is 900 ft. wide between the tops of the banks, with a maximum depth of 85 ft. at bank-full stage. At Butte la Rose the width is 350 ft., with a maximum depth of 58 ft. At the head of Alabama Bayou, which depletes the stream to some extent, the width is 650 ft., and the maximum depth 65 ft. The depletion by Alabama Bayou is very largely recovered by the return flow of the same bayou, and by the inflow of several bayous from the westward.

When the Mississippi River flowed bodily around the bend, past the mouth of the Red River (Fig. 1) and the head of the Atchafalaya, vast accumulations of drift were carried down the latter stream, which finally choked it up, forming a compact mass of floating trees and logs known as "raft."

This raft began about 20 miles below the source and continued with intervals of open water for about 20 miles down stream. It was so dense in places, that, although rising and falling with the variations of stage, willow trees grew upon it, and in some places it served the purpose of a bridge which was readily crossed by cattle. "Shreves Cut-off" was made in 1831 (Fig. 2), and the main body of the Mississippi, with its burden of drift, no longer flowed by the head of the Atchafalaya, and the growth of the raft practically ceased.

The banks of the Atchafalaya, down to Butte la Rose, were occupied by flourishing settlements, and the demands of navigation called for the removal of the raft so as to permit the passage of water craft laden with the products of the fertile soil. The work was begun by the State of Louisiana prior to 1860, and continued for several years until a channel was finally opened through it. The immediate result of the removal of these obstructions was a great increase in the velocity and a consequent tendency toward enlargement, through the erosion of both bed and banks. It also increased the volume of the floods, and levees to protect the lands from overflow became necessary. The levee on the right bank extends down stream a distance of about 37 miles from its source. This levee begins at high ground some miles up the Bayou de Glaise.

The levee on the left bank connects with the Mississippi River levee system, and continues along the Atchafalaya a distance of some 43 miles. Below the ends of the levee the flood waters are scattered through the bayous, swamps and lakes on their way to the sea.

Definite figures as to the extent of the enlargement, from the days of the raft to 1880, cannot be given, owing to the lack of accurate surveys covering the early period. If the testimony of the "oldest inhabitant" is accepted as correct, the stream, at a point 5 miles below its source, has enlarged within the memory of man from one which could be crossed on a foot-log 40 ft. long, to the present stream 650 ft. wide and 40 ft. deep at low water. There are, however, good reasons for doubting the accuracy of the early testimony as to the width, but, at the same time, a great enlargement of the channel is apparent.

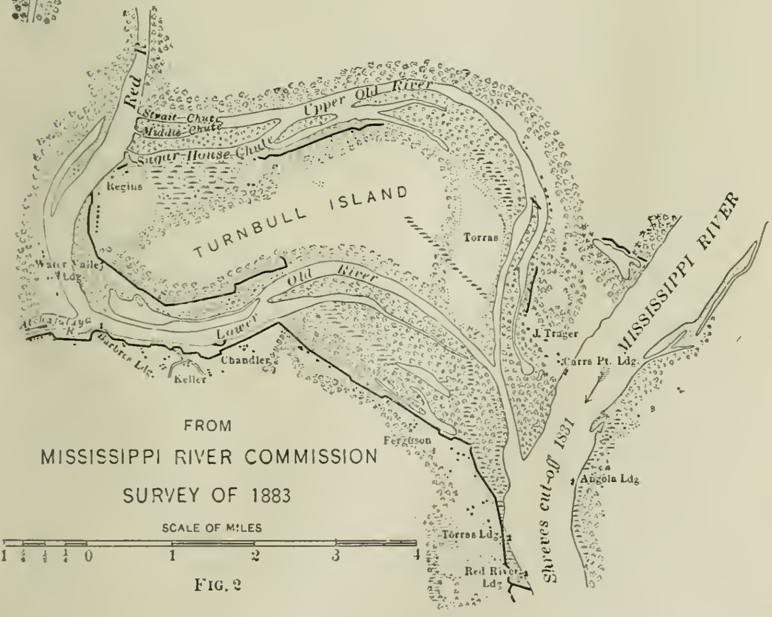
The masses of drift still found embedded in the banks of the stream suggest that at some remote time the channel was even larger than it is now. The enormous quantity of raft also indicates a very great outflow of water which brought the material down the waterway.

On account of the shorter distance to the Gulf by way of the Atchafalaya, and the consequent increase in slope, there have long been grave fears that the main river would abandon its own bed and take this route to the sea, leaving New Orleans, Baton Rouge and other cities inland. Such a radical change would also disastrously disturb the regimen of the river for many miles above the present junction of the two streams. The Yellow River, in China, furnishes a forcible precedent for anticipating such a change. This river abandoned its original bed and took the shorter route to the sea, the present mouth being some 200 miles from the former mouth.

To prevent such a catastrophe, two mattress sills were laid across the stream by the Mississippi River Commission in 1887-89. They are located at Simmesport, about 4 miles below the head of the Atchafalaya, and consist of willow mattresses, 3 ft. thick and 300 ft. wide, heavily ballasted with stone. They extend entirely across the stream and up to the high-water banks. The two sills

REDUCED FROM MAP
MADE IN 1805
BY
B$^{\underline{m}}$ LAFON, C.E.

FIG. 1



FROM
MISSISSIPPI RIVER COMMISSION
SURVEY OF 1883

SCALE OF MILES
1 ¾ ½ ¼ 0 1 2 3 4

FIG. 2

are about 1 800 ft. apart, and have effectually prevented further enlargement of the section of the river at this point.

In the meantime a decided tendency toward the closure of the lower old river, near the present position of the Mississippi, by deposits of sediment, threatened to divorce both the Red and Atchafalaya Rivers from the main river (Figs. 1 and 2). This was a serious menace to the navigation interests of these streams, and, in order to meet the requirements of these interests, it became necessary to resort to dredging during each low-water season. The channel is still maintained in this way.

On the completion of the locks at Plaquemine, on the Mississippi River, now under construction, access to the two streams above named will be by the Bayou Plaquemine and the Upper Grand River, as well as by the present route.

The deposits from the sediment-laden waters of the Mississippi are gradually building a barrier of great proportions between its banks and the head of the Atchafalaya, which will have the effect of depleting more and more the volume contributed by the Mississippi during floods. Were the dredging suspended, this depletion would doubtless be greater from year to year, and, as it goes on, it necessarily means an increase in the flood volume of the main stream below the juncture and a decrease in the volume of the Atchafalaya, the greatest load of which would finally be the Red River floods.

Whether the banks and bed of the Mississippi River will accommodate themselves to the increased volume without undue strain, as the barrier develops, is an important question, which has received a great deal of careful study, and is still under consideration.

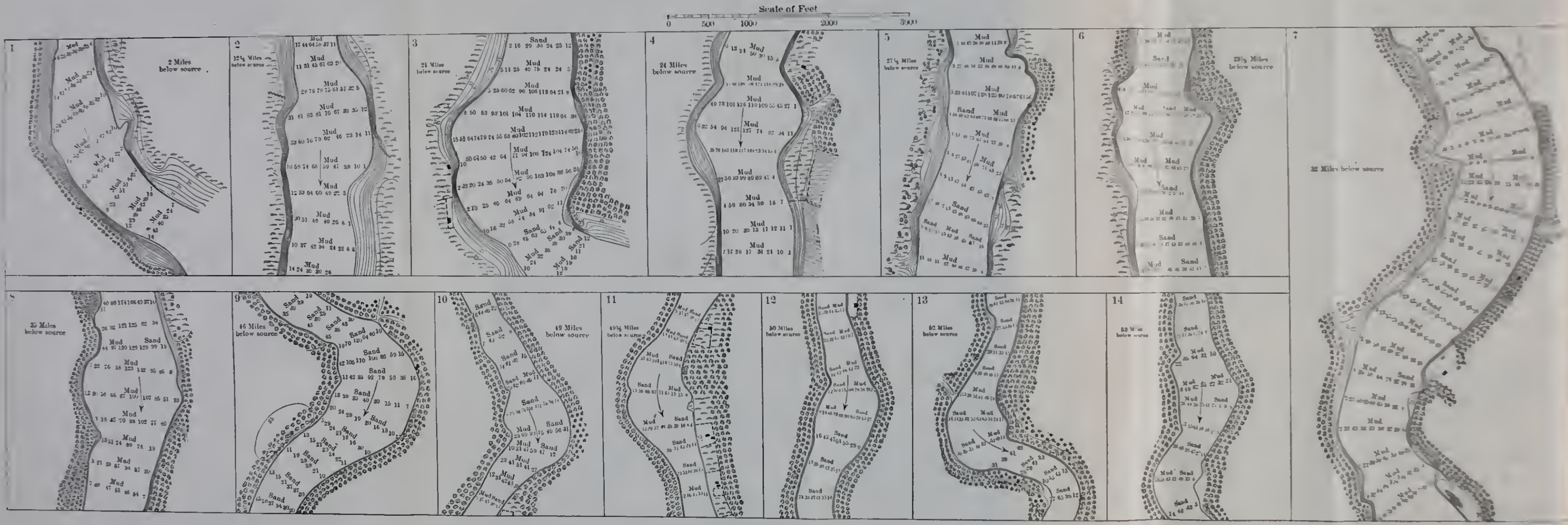
A survey of the Atchafalaya, made about a year ago by the Mississippi River Commission, brings to light some of the peculiarities of the stream, and also furnishes data from which can be determined the changes that have occurred during the 25 years which have elapsed since the previous survey was made.

The doctrine of contraction as a means of increasing the depth of streams is almost universally accepted as correct. In this stream there are repeated and radical violations of this principle under natural conditions, as a glance at Figs. 1 to 14, Plate II, will

PLATE

SHORT REACHES OF THE ATCHAFALAYA RIVER.

SHOWING MARKED VARIATIONS IN DEPTH AND WIDTH, THE GREATEST DEPTH BEING COINCIDENT WITH THE GREATEST WIDTH. SURVEY OF 1905.



show. The extremes, in contiguous narrow and wide stretches, are shown in Table 1.

TABLE 1.—WIDTHS AND DEPTHS ON CONTIGUOUS WIDE AND NARROW SECTIONS OF THE ATCHAFALAYA RIVER.

Section.	Distance from source, in miles.	Narrow Section.		Wide Section.	
		Width, in feet.	Depth, in feet.	Width, in feet.	Depth, in feet.
1	2	500	39	825	69
2	12½	575	64	1 100	83
3	21	800	30	1 620	124
4	24	750	34	1 300	127
5	27½	700	49	1 075	128
6	28½	650	55	1 150	122
7	32	750	48	1 200	139
8	35	700	54	1 200	132
9	45	500	39	1 050	120
10	49	350	41	800	114
11	49½	400	45	800	116
12	50	450	45	720	95
13	52	300	49	730	65
14	53	470	41	720	67

In the case of Section 3, where the width is more than double that of the narrow section, the depth is more than four times as great. All the cases cited are striking, in this respect.

Plate II shows fourteen reaches of the river as derived from the survey of 1905. The figures in the stream indicate the depths in feet below the stage when sounded. This stage did not in any case reach the top of the banks, but ranged from low to medium stage. The contour intervals along the banks are 5 ft., and the figures indicate the elevations of the banks above mean Gulf level. The character of the bottom, as revealed by the sounding lead, is noted at frequent intervals.

Plate II amplifies the results in Table 1, and shows graphically, in a very forcible manner, that in this river a narrow channel means less depth, and that excessive width is invariably accompanied by great increase in depth.

No satisfactory explanation of the cause of these great holes has been found. It has been suggested, since they occur mainly in the reach formerly occupied by the "raft," that the washing out of the drift accounts for the holes; but, where the bed and banks have been rip-rapped with logs and trees embedded in the soil, it would seem to be more difficult to erode them than where no such obstructions existed. Furthermore, a given volume of water flowing along a stream of uniform slope should have the greatest velocity in the smallest section, neglecting the item of friction. Double the area of the section, and the velocity required to carry the volume should be reduced one-half and its eroding power be correspondingly reduced.

Another suggestion is that levee building along the river has progressed by stages, and that the flood waters have been confined until they reached the ends of the levee, where they were set free with an enormous increase in slope at that point, and consequently with greatly increased velocity, which expended itself in whirls and eddies which attacked both bottom and sides of the stream. In that case the succession of holes should coincide with the successive prolongations of the levee system. The holes are limited in length, and are adjacent to much narrower and shallower sections which must also have been within the sphere of influence of the exaggerated slope, if such slope actually existed.

The closing of outlet bayous, which, when open, carried off considerable of the volume, doubtless had much to do with the enlargement of section in the lower reaches.

In streams like the Mississippi and Missouri, the maximum erosion is invariably in the bends or concave shores, which continue to elongate until a cut-off occurs, and the length is restored to normal conditions. In the Atchafalaya, however, the erosion has been largely confined to the points, or convex shores, the direct effect being a material reduction in its length.

It is evident, from the conditions already described, that in case an increase in the navigable depth of the shallow reaches were required, the engineer would probably find it necessary to abandon the usual practice of contraction in such cases, and resort to other means to secure the desired results.

The survey recently made provides the data necessary for a study of the changes, and throws considerable light on the causes thereof. This study has consisted mainly of a comparison of the area, depth and width of some 278 cross-sections, at low, medium, and bank-full stages, which are practically common to the surveys of 1880-81 and 1904-05. In order to deduce mean elements for certain lengths of the stream, it was first divided arbitrarily into

Fig. 1.—Head of Atchafalaya River. November 16th, 1900.

Fig. 2.—On the Atchafalaya River. November 17th, 1900.

Fig. 3.—Head of Bayou Alabama, November 26th, 1905.

reaches 5 miles in length. This, while indicating certain tendencies, failed to satisfy all the requirements, and a division into reaches conforming to the several epochs of levee building was finally resorted to as the most rational. A division of the river on this basis gives four well-defined reaches, and the comparison of the mean elements of the two surveys on this basis leads to certain conclusions which afford satisfactory explanation of some of the phenomena observed.

The mean results are set forth in Table 2.

The location of the reaches is shown on Plate I. Reach 1 begins at the head of the river and extends down stream 13.7 miles. The levees covered both banks at the time of the first survey, and the flood control, therefore, was more or less complete during the entire interval between the two surveys; the only change in the conditions was the enlargement and strengthening of the levees, which added chiefly to their safety, rather than to their utility in the matter of flood control. It will be seen that in this reach the depths and area of the mean low-water section have decreased, and the enlargement of the high-water section has been due to a very moderate erosion of the banks. In short, the bed of the stream had long since adapted itself to the volume which it was required to carry, and practically stable conditions were the result.

In Reach 2, which covers the next stretch of 16.1 miles, there was a levee on the left bank for its entire length at the time of the first survey. This levee was more or less effective during the interval elapsing between the two surveys, but there was no levee on the right bank. The right bank levee was first effective for flood control some 12 years after the first survey was made, and this for about half the length of the reach. The lower half was not effective until some 4 years later. In this reach it is found that when the first survey was made, there was already considerable enlargement due to the influence of one levee for the entire length, and the enlargement continued when the flood control was completed by the construction of the right bank levee. There are also indications here, showing that the very deep pools have filled up in the upper portion where the levee conditions have been practically the same during several floods.

Reach 3 covers a distance of 7.1 miles. At the time of the first

TABLE 2.—MEANS OF CROSS-SECTION ELEMENTS BY REACHES, AND CHANGES DEVELOPED IN THE INTERVAL BETWEEN THE SURVEYS OF 1880–81 AND 1904–05.

Reach.	Stage.	Widths, in Feet.		Changes.		Areas, in Square Feet.		Changes.		Mean Depths, in Feet.		Changes, in Feet.	Maximum Depth, in Feet.		Changes, in Feer.	Bank Elevations, in Feet.		Changes, in Feet.
		1880–81.	1904–05.	Feet.	Per Cent.	1880–81.	1904–05.	Square Feet.	Per Cent.	1880–81.	1904–05.		1880–81.	1904–05.		1880–81.	1904–05.	
1.	Low water.	639	712	+ 73	+11.4	17 902	15 697	— 2 205	—12.3	27.1	22.1	—5.0	43.3	34.5	— 8.8	....	....	.....
13.7 miles..	Bank-full..	991	1 147	+153	+15.4	53 171	55 572	+ 2 401	+ 4.5	53.4	48.8	—4.6	....	....	......	46.4	47.4	+1.0
2.	Low water.	594	768	+174	+29.3	14 862	23 159	+ 8 297	+55.8	23.4	28.9	+5.5	39.2	47.7	+ 8.5	....	....	.....
16.1 miles..	Bank-full..	830	1 059	+229	+27.6	40 668	55 925	+15 257	+37.5	47.7	52.0	+4.3	....	....	......	38.9	42.0	+3.1
3.	Low water	546	741	+195	+35.7	14 203	26 138	+11 935	+84.0	25.5	34.7	+9.2	41.4	56.6	+15.2	....	....	.....
7.1 miles..	Bank-full..	713	983	+270	+37.9	32 241	50 702	+18 461	+57.3	44.7	51.3	+6.6	....	....	......	30.8	33.8	+3.0
4.	Low water.	399	555	+156	+39.1	10 327	16 233	+ 5 906	+57.2	25.9	29.3	+3.4	39.5	44.8	+ 5.3	....	....	.....
6.2 miles..	Bank-full..	535	717	+182	+34.0	22 109	32 250	+10 141	+45.9	41.5	45.1	+3.6	....	....	......	27.5	29.3	+1.8

survey there were no levees on either side of the river, and this condition existed for 22 years, when effective levees were completed on both sides of the stream. These levees were effective during the great flood of 1903 and the succeeding years, with the exception of somewhat less than 1 mile, which was partially open on one side at the lower end. Here, the enlargement shown reaches the maximum, and the process is evidently not yet completed. Individual sections show changes amounting to double in width, eight times in area, and an increase of 80 ft. in depth.

In Reach 4, which covers 6.2 miles of river, there were no levees at the time of the first survey, and only the left bank levee was effective during the floods of 1903 and subsequent years. This one levee, effective during a great flood, began the process of enlargement, but it is still much smaller than the reaches above it.

The results seem to indicate clearly that the levees are responsible for the enlargement of the stream by confining to one channel the waters that were once dissipated by flowing over the banks and escaping through outlet bayous. Here is a direct and positive measure of the influence of levees in enlarging the bed of a stream of no mean capacity.

It seems to be clear, also, from the results in the upper reach, that the volume to be carried needs a bed having a bank-full cross-section of about 56 000 sq. ft., and when the enlarging bed reaches such proportions further enlargement is confined mainly to a comparatively slight abrasion of the banks. The deep irregular pools fill up, and the tendency is toward a stream more uniform in section and practically stable in its regimen.

As the levee system along the left bank of the Mississippi River is carried down stream toward the present connection, through "Old River," with the Atchafalaya River, and as the deposits at the entrance of "Old River" continue to increase year by year, the contribution to the Atchafalaya from the floods of the Mississippi will naturally diminish. This, together with the stable condition of Reach 1 for so many years, suggests that there is very little danger of the great river taking the Atchafalaya route to the Gulf of Mexico.

The writer is indebted to Frank M. Kerr, M. Am. Soc. C. E., Chief State Engineer of Louisiana, for statistics as to levee construction along the Atchafalaya River.

DISCUSSION.

Mr. Richardson.

HENRY B. RICHARDSON, M. AM. SOC. C. E. (by letter).—This paper, apart from its professional interest, merits special attention from those who remember the many diverse views and earnest discussion of what has been commonly referred to, in the Lower Mississippi Valley for the past quarter century or more, as "The Atchafalaya Problem." It presents, in condensed form, the results of recent surveys, and brings to view features and characteristics of the stream that have heretofore received little notice.

This "problem" has, perhaps, never been very clearly stated, and seldom twice alike. The proposed solutions have been naturally even more unlike than the statements of the problem. All, however, have sought in some way to improve one or all of the three streams involved—that is, the Red, the Atchafalaya and the Mississippi—either for navigation or for the prevention of destructive overflow. For instance, it has been proposed (1) to divert the Red from the Atchafalaya, either by turning it into the Mississippi some 20 miles above where it now enters, or at some one of several nearer places; (2) to close the Atchafalaya at or near its head, thus cutting off its water supply from the Mississippi and the Red; and (3) to dam up the mouth of Old River (Fig. 2), thus divorcing both the Red and the Atchafalaya from all connection with the Mississippi, except for navigation through a lock at the head of Bayou Plaquemine and thence by the way of that bayou and Upper Grand River.

These general projects, with numerous variations, have each been strenuously advocated in the past, so that the subject-matter has well earned the popular title of a problem.

The last-named plan—that of divorcing the Red and Atchafalaya River System from that of the Mississippi—gives promise of most desirable results in the matter of protection from overflow; and now that there is good prospect of an early opening of the lock at Plaqueminé, through which navigation interests should be better served than at present, it may be expected that it will meet with increasing public favor.

The most serious objection ever urged against this plan has been prompted by the apprehension that the waters of the Mississippi cannot be successfully carried to the Gulf, between levees, below the mouth of Old River (or the head of the Atchafalaya) in such volume as they arrive there during great floods; and that for the security of the country below, along the Mississippi, the Atchafalaya must be left open as a safety-valve, or relieving outlet, for a part of the Mississippi flood waters. "Whether the banks and bed of the Mississippi River will accommodate themselves to the increased volume without undue strain," is, as the author remarks, "an important question." Mr. Richardson

While by no means settling this question, the fact that, at the times of the highest recorded flood stages in the Mississippi, at the mouth of Old River, the water surface has been at a still higher elevation at the head of the Atchafalaya, should throw some light on what would have been the situation if the two streams had then been separated. With the open channel, as it is and has been, there must have been at the times of highest flood referred to, a flow through Old River into the Mississippi, so that its flood volume, at the most critical moment, was increased instead of depleted. Of course, had Old River then been closed, this could not have happened; the water that was then tributary to the Mississippi could not have entered it, but would have found its way down the Atchafalaya to the Gulf of Mexico.

It is true that during the highest flood stages above mentioned the larger part, if not all, of the water that came through Old River to swell the floods of the Mississippi had escaped from it through breaks in the levee above, and was simply rejoining the stream it had earlier deserted. But the fact remains that in the times of highest flood at the mouth of Old River, the Atchafalaya could not have acted as an outlet of the Mississippi, but quite the contrary.

The building of a dam or embankment across the mouth of Old River, with a short extension connecting it with the adjacent levee lines, would close the only gap in the existing levee system on the right bank of the Mississippi below Amos Bayou ridge, in Desha County, Arkansas, and make it an unbroken line, extending practically from the Arkansas River to the Gulf of Mexico. With this gap closed, all overflow of the right bank of the Mississippi should be prevented. With it open—as at present—the waters "back up" through it in every flood, and annually inundate a wide area of land, sometimes several hundred square miles in extent, and often remaining for two or three months. The lands thus deluged are as fertile, and—except for their lack of protection against overflow—as desirable for cultivation and habitation as any in the alluvial valley.

Another objection to this plan has been, that, in case large crevasses should occur in the levees, permitting the Mississippi floods to invade the basin above the mouth of Old River, there might be such an impounding of water there as to cause deeper overflow than could take place with Old River left open. In other words, the Atchafalaya might not have sufficient capacity to discharge the water received from such crevasses fast enough to prevent its accumulation in so great a volume as to produce more extended inundation than would result from the "back water" alone with Old River open.

Mr. Richardson.

But the levees on the Mississippi have been, and are being, so constantly improved in height and strength that there should be less and less danger each year that there will be such crevasses; and the recent great enlargement of the Atchafalaya, described in the paper, should give some assurance that it will be able to dispose of all the crevasse water now likely to reach it. During the flood of 1890 its discharge was gauged near its head, and at one time found to be about 480 000 cu. ft. per sec. This is more than two and a half times the greatest measured discharge of the Red at Alexandria—that is, of Red River proper—and from this it appears that the Atchafalaya, after receiving a maximum flood from the Red, still has room to take care of more than as much again from other sources.

The correctness of the author's conclusion that the confinement of the waters of the Atchafalaya, which formerly escaped over its banks, to a single channel by means of levees, is the chief cause of the recent rapid enlargement of its bed cannot be doubted. It seems probable, however, that the removal of the obstruction caused by the "raft" of 20 miles of floating trees and driftwood—referred to in the paper—must have considerably increased the volume and erosive power of the water within its natural banks. But it appears still more probable that the strikingly rapid and irregular enlargement described must be principally due to the progressive confinement of its flood waters to one stream only, and the consequent concentration, upon its own bed and banks, of their scouring energies.

The physical characteristic of the Atchafalaya presented by the fact that it is deepest in places of excessive width is possibly peculiar to waterways which have undergone, or are undergoing, rapid enlargement, rather than to this particular stream. The instances illustrated in Plate II, and Table 1, are very interesting, and are remarkable for the magnitude of the variations shown in the areas of adjoining cross-section. Similar irregularities in section, on a very much smaller scale, of course, may often be seen in ditches cut through rather stiff, but erosible, soil, when enlarged by the action of rapid currents; and it is not always easy to account for the numerous expansions, contractions and "pot-holes" found in them. Where the ditch is largest the scouring would have been greatest, and there the current velocities, in order to do the work, should have been highest. And doubtless they were so, though mostly indirect—whirling around in eddies—and entirely independent of the area of cross-section. Such eddies, both horizontal and vertical, are evident in most streams, and, when rapid and violent, seem sufficient to account for the formation of the holes and abruptly enlarged places, not only in small ditches, but also in the largest rivers.

Two instances shown on the charts of the Lower Mississippi River may be cited where extremes of depth and width are coincident: one opposite Ormond Plantation, about 28 miles above New Orleans, and another just below the Old Quarantine Station, 5 miles above the Forts. Mr. Richardson.

TABLE 3.

Section.	Width, in feet.	Depth, in feet.	Approximate area of cross-section, in square feet.
	Opposite Ormond.		
Wide section	3 300	221	435 900
Narrow section, 3 000 ft. above	2 050	126	159 500
Narrow section, 4 600 ft. below	2 300	110	156 400
	Near Old Quarantine Station.		
Wide section	3 000	248	372 900
Narrow section, ½ mile above	2 200	80	138 800
Narrow sec ion, ½ mile below	2 450	137	189 600
Narrow sec ion, 2¼ miles below	2 200	100	160 200

The widths, scaled from the charts, the extreme depths, as shown by soundings, and the approximate areas of cross-section, at and near these places are shown in Table 3.

While the irregularities in form and area of section at these points are not so striking as those noted in the channel of the Atchafalaya, they show that extreme width and extreme depth may occur and persist together, even in a stream not undergoing rapid enlargement.

T G. Dabney, M. Am. Soc. C. E. (by letter).—It has been related that a student, when required to write a dissertation on the crab, described the creature as being "a red-colored fish that swims backward"; to which his professor replied that the definition was correct, except as to three particulars; "the crab is not red, is not a fish, and does not swim backward." Mr. Dabney.

Mr. Ockerson defines the "Atchafalaya River" as "an alluvial stream flowing between banks of its own formation"; to which it may be replied that it is neither a "river" nor an "alluvial stream" as defined above.

As to the latter characterization, the weight of geological authority has pronounced that the bulk of the material composing the so-called "alluvial valley" of the Mississippi River is not an alluvial formation, and was not transported and lodged where it now rests through the agency of the Mississippi River and its subsidiary

Mr. Dabney channels, except as to superficial ridges or "natural levees" along the banks, thinning out to a mere "veneering" a short distance away.

The so-called "Port Hudson" formation, which mainly occupies the basin, is pronounced to be a diluvial or estuarian deposit, brought from more northern regions by large volumes of glacial water, liberated by the melting and recession of the great "ice cap" in pleistocene time; antedating the present drainage system of the Mississippi Valley, which has incised its channels down into and through some of the older deposits.

This is not especially significant in the present discussion, however, as the "Port Hudson" deposits possess no greater resisting powers to current erosion, perhaps, than the more modern alluvium.

The writer attaches much greater potency to the first-named distinction, as bearing upon the subject under discussion. The term "river," as commonly understood, applies to a channel which receives all its water from the "run-off" of its own proper water-shed or drainage area. Its form and dimensions are the result of practically constant agencies which have operated for a very long time, by which practically stable conditions and regimen have been established.

The Bayou Atchafalaya is not a stream of that character, and is entitled to be called a river, if at all, only in a much modified degree, receiving the local "run-off" from a limited area on the western side for a portion of its length.

The bulk of its flood water comes from a wholly extraneous source, being delivered at its head by the Mississippi and Red Rivers, where its burden of 400 000 cu. ft. per sec. thus received must be the dominating influence in determining its character. It is a "bayou," a distributary, and not a river in any proper sense.

A marked characteristic of the Atchafalaya, which is a reversal of the rule of rivers, is that it probably carries a larger flood volume at its extreme head, than in its lower course where the low unleveed banks permit part of the flood to escape from the channel.

The burden thus imposed on the Bayou Atchafalaya was, as shown by Mr. Ockerson, rather suddenly and greatly increased by the removal of the obstructing rafts and the leveeing of its upper portion.

To those who are conversant with overflow conditions in the Mississippi bottoms, the phenomenon is the familiar one of scouring action induced by obstacles in the path of the flowing water, in consequence of vertical and lateral currents which are set up by obstructions, in a formation that yields readily to such destructive agencies.

When a crevasse occurs in a river levee at flood stage, the escaping water speedily scours out a deep hole or "crater" under the base of the levee and for some distance beyond the levee line. Mr. Dabney.

These "crevasse holes" are familiar objects along the Mississippi River levee lines, are generally of large dimensions, and are locally called "blue holes" on account of their great depth, which gives a bluish, or rather greenish, tinge to the water, when in a state of repose. The occurrence of similar holes on a smaller scale, in situations remote from the river, is not unusual, as a result of obstacles encountered by the flood water, which set up scouring action. The writer has seen such a hole, "big enough to bury a church in," caused by a large stranded log, which, being over-topped by the flowing water, acted as a tumbling dam.

The sectional area of one of these great crevasse craters, when the water is flowing violently into and out of it, is much larger than that immediately above and below.

The whole content of this larger section, of course, is not moving in the general direction of the flow, with the same velocity as in the lesser entering and departing sections; nevertheless, the water moves with great energy in multitudinous diverse directions, vertical and lateral, which energy is expended on the bottom and sides of the crater.

It seems probable that in the case of the Atchafalaya a somewhat analogous action was set up in those reaches formerly occupied by the rafts, the bottom and sides of which are described by Mr. Ockerson as being "rip-rapped" by logs, remnants of the former rafts.

These logs probably projected all sorts of ways from the bottom and banks of the channel, offering obstruction to the direct flow of the flood water, and affording ideal conditions for the generation of violent vertical and lateral currents, with inevitable scouring as the result, and a consequent deepening and widening of those parts of the channel.

The material thus dislodged was carried along down the channel, and probably deposited in the narrower intervening reaches, by which their depth was further reduced.

Here, as in the case of the crevasse crater, while great energy of movement was developed, it was not in the general direction of the flow of the stream, but expended itself on the bed and the banks of the channel. In the smaller sections all the energy of movement is utilized in passing the volume of water down stream, while a total of much greater energy of movement in the pools is largely dissipated in vertical and cross-currents.

To Mr. Ockerson's observation that in this case there appears to be a reversal of the law of rivers, by which narrowing the waterway increases the depth of channel, it may be suggested that the

Mr. Dabney. conditions described as existing in the Atchafalaya did not result from narrowing the waterway anywhere, but by the abnormal widening and deepening of the channel in certain localities through the operation of particular agencies present there and not elsewhere.

The deduction to be drawn from the foregoing is that the readiest way to enlarge a channel in this so-called "alluvial" formation, should be by placing partial obstructions in the path of the flowing water; and this conclusion is verified by observation; but whether such agencies can be applied and controlled so as to realize desired results is a more doubtful proposition.

The somewhat dogmatic tone of this "brief" is assumed for the sake of brevity. The writer, nevertheless, is modest enough to invite correction if any faulty positions have been taken.

Mr. Kerr. Frank M. Kerr, M. Am. Soc. C. E. (by letter).—The past individuality, and present conduct, general characteristics and peculiarities, together with its relations toward the Mississippi and the Red Rivers, so interestingly described and ably discussed in this paper, no doubt single out the Atchafalaya River as, in many respects, distinctive from most other streams, in most other parts of the country, but hardly notably so in Louisiana.

Mr. Ockerson says:

"Opinions differ as to its origin. It is probable that, at one time, it was the bed of the Red River which flowed directly to the Gulf of Mexico, but which was made a tributary of the Mississippi River by the elongation of the bend of the latter which cut into the smaller stream and made the mouth of the Red River near where it is now located. Even before this occurred the Atchafalaya portion served as an outlet for some of the overflow waters of the Mississippi River."

There is little or no question about the popularity of this opinion. However, there have been several other theories advanced, two of which it may not be uninteresting to recall.

The first of these may probably be more properly classed as a legend. But, however that may be, it would appear to possess sufficient ingenuity to merit resurrection in this discussion.

This theory, or legend, set forth that, in the earlier history of the valley, the Atchafalaya River had no existence at all, but, during a great overflow in the Mississippi River, it was given birth in the form of a breach occurring in the west bank of the Mississippi, at or near the point in the bend now known as Old River, through which the waters of the river rushed madly across country, by the shortest route to the sea, traversing the great network of small streams occupying that territory and lying in easterly and westerly directions, and cutting out the channel from the Mississippi to the Gulf of Mexico, later known as "Bayou Atchafalaya." In support of this it is shown, and is a matter of fact, that for every natural opening, or bayou, found on the right bank of the river, there will also be found practically throughout its length, almost directly opposite, on the left bank, a similar natural opening or bayou, as though the continuity of a succession of streams had been interrupted by some such "hop, skip and a jump" process, herein attempted to be described. Mr. Kerr

Next, there was the theory of Mr. Charles Ellet, advanced in a report made by him, under the auspices of the United States, in 1853, on the Mississippi and Ohio Rivers. Similar deductions will be found in the Report on the Mississippi River, prepared and submitted in 1861, by Messrs. Humphreys and Abbot, and in the report of the Louisiana Commission of Engineers (Caleb G. Forshey and M. Jeff Thompson), in 1873.

Mr. Ellet, in this report, in part, said:

"The Atchafalaya is by far the largest of the existing or former outlets of the Mississippi; and it has been often proposed to resort to its channel as the best and most efficient drain for the floods which now threaten the country below its source.

"In concurring with this popular idea, so far as to advise a commencement of the gradual and progressive enlargement of this great stream, it is not intended to represent the work as easy to accomplish, or in itself an effectual remedy for the floods of lower Louisiana.

"It will, in fact, be found to be an exceedingly difficult and costly undertaking, and one which will need to be conducted cautiously and not too rapidly, if it is to be effected without serious injury to the region through which the waters are to be conveyed.

"The Atchafalaya leaves the old channel of the Mississippi about two miles below the mouth of Red River, and 310 miles, by the windings of the channel, above the Gulf of Mexico. It flows nearly in a southwardly direction; and when the Mississippi is swollen by floods, it serves as a natural vent for a portion of the present excess of waters, of which it bears off a large volume to the sea.

"At its source its average surface width in extreme high water is about 600 feet; its depth fifty-five feet; its slope six inches per mile; and its discharge not less than 85 000 cu. ft. per sec. It is about equal, measured by the area of its channel and the volume of water which it conveys, to one-twelfth part of the capacity of the Mississippi above New Orleans.

"It has long been supposed that the Atchafalaya was the ancient bed of Red River, when that stream had no connection with the Mississippi, but found its way to the Gulf of Mexico by an independent channel. The union of the two streams—the Mississippi and Red River—is accounted for, in this theory, by the supposition that, at the point where their waters now mingle, their channel then exhibited opposing flexures, and the current gradually cutting away the intervening soil, brought the streams together and made their waters common.

"This has become of late years a very popular theory, and is sup-

Mr. Kerr. ported by several plausible arguments. The position of the mouth of Red River on the one hand, and that of the source of the Atchafalaya on the other; the direction by which Red River enters, and that by which the Atchafalaya leaves the old channel of the Mississippi correspond perfectly with the assumption that the curves of the two adjacent streams gradually approached, until they finally cut into each other. Besides, the color of the soil composing the west bank of the Atchafalaya at its source indicates clearly a Red River origin. But notwithstanding the plausibility and force of these facts, they are not at all conclusive, but apply with equal directness to another view that will be here suggested.

"In fact, the hypothesis which attributes the original formation of the Atchafalaya to the discharge of Red River is found, on a careful examination, to be wholly untenable. It results from actual measurements of the channel of these two rivers; that while the Atchafalaya at its source has a prevailing depth at high water, in mid-channel way, of only about fifty-five feet, Red River, at its mouth, only three miles distant, exhibits a depth of more than one hundred feet; that, while the Atchafalaya is confined within a channel less than six hundred feet wide at its surface, in high water, the width of Red River, between banks, a mile above its mouth, is more than 1 100 feet; and that, while the descent of the Atchafalaya at or near its mouth is six inches per mile, that of Red River, where it enters Old River, is at low water less than one inch per mile.

"The hypothesis of a former continuous channel, common to these two streams, so different in all their features, must, therefore, concede a sudden and remarkable change in the character of the supposed ancient Red River, at the precise point of the present junction of that stream with the Mississippi. Such a change, and the exact coincidence of that change with the point of accidental contact of the two shifting channels is, indeed, not impossible, but it is, at least, quite improbable. A less violent and much more satisfactory theory, to account for the existence of the Atchafalaya—one of the most remarkable features of the Mississippi—may be suggested, though the writer has not had full opportunities to submit it to a very rigid inspection.

"Black River, the proper continuation of the Ouachita, corresponds much more closely in the dimensions of its channel with those of the Atchafalaya than Red River. The general direction of the Ouachita is from north to south, corresponding well with the present general course of the Atchafalaya.

"The idea has impressed itself upon the mind of the writer, that in the original condition of the delta, the Ouachita, as well as Red River, descended by an independent channel to the Gulf, which then, perhaps, set up through a bay as far as the head of Lake Chicot.

"The Mississippi at that period pursued its present general direction. Red River had also its own independent channel to the Gulf, in the present Valley of the Teche, where it has left abundant traces of its course, in the composition of the soil and in the width and form of the immediate valley of the present stream from above the rapids at Alexandria down to Berwick's Bay. The Ouachita was thus an independent stream, descending to the sea between Red River and the Mississippi.

"According to this hypothesis, the Ouachita and the Mississippi, by the gradual approach of opposite bends, ultimately united their waters, and the Ouachita was, so to speak, cut in two—the northern part afterwards serving as a feeder to the Mississippi, and the southern end acting as an outlet for its surplus water in times of flood. Mr. Kerr.

"The Ouachita having been a stream of smaller class than Red River, may be adduced as a reason why the present channel of the Atchafalaya, which was formed to accommodate the volume which that river, and not Red River, brought down, is insufficient for the discharge of the present volume of Red River. The Ouachita flowing directly down the plane of the delta, which it has been shown, descends at the rate of eight inches per mile, accounts for the greater fall of the Atchafalaya, which takes the same direction parallel with the dip of the same plane.

"Subsequently to the junction of the Ouachita and the Mississippi, Red River—then continuing on below Alexandria in the same southeasterly direction which it still pursues above that point—flowed over its natural levee, and, taking an easterly direction through the swamps, united its waters with those of the Ouachita at the present mouth of the Black River. Under this hypothesis, the increased volume below the confluence of these streams, in the course of thousands of years, may have produced the larger channel known as Red River, which even now is scarcely sufficient to accommodate their collected waters.

"According to this view, which is suggested as the most plausible explanation of the existence of the Atchafalaya, that stream was the ancient channel of the Ouachita, or its prolongation, Black River; and the present channel of Red River, below the mouth of Black River, was subsequently enlarged by the union of the waters of the Red and Black."

In the foregoing, without going into other reasonings, the contrasts shown to have existed formerly between the section, slope and discharge of the Red and Atchafalaya Rivers, and the then practical agreement in all three factors between the Ouachita and the Atchafalaya, would appear to have given much color to the theory. The theory, however, is interesting not only in this respect, but, accepting the data, and comparing them with the facts furnished by Mr. Ockerson, not forgetting the shorter route to the sea, also in overcoming surprise at the irregularities and peculiarities of the area of section and slope in the Atchafalaya River, with its alternating wide and deep, and narrower and shallower reaches, and comparatively abnormal holes, here and there, found at this time.

Nor does the alarm then sounded about the possibility of the Mississippi River abandoning its own bed and taking such a route to the sea, leaving New Orleans, Baton Rouge and other cities inland, in the light of Mr. Ellet's statements and deductions, appear to have been altogether unwarranted.

Think of a stream in any soil, and particularly in a soil made up of detritus, from many sources at many periods in the history of the

Mr. Kerr. Valley of the Mississippi, with an unusually steep slope, whose high-water discharge, between banks, was in 1853, 85 000 cu. ft. per sec. (Ellet) accommodating itself in 1905 to a discharge, also between banks, or levees, of 400 000 cu. ft. per sec. (Ockerson).

That the "holes" demonstrated, by the recent survey under the direction of the Mississippi River Commission, to exist, are largely the result of levee building by progressive stages, confining the flood waters until they reached the ends of the levee, where, set free, with enormous local increase in slope, greatly increased velocities were created, and rapid whirls, eddies, counter and cross-currents were developed, attacking both sides and bottom, resulting in extraordinary conditions, is unquestionably true. Without the benefit of extended surveys, the writer had long ago accepted the existence of such conditions, from observations made on trips taken down the river in an ordinary skiff, during periods of high water. Traveling swiftly through the narrower sections of the river, the skiff would enter the wider sections with a pitch and a lurch, and meet such a combination of counter and cross-currents and eddies as would often produce the sensation of shooting the rapids. Imagine the effect, upon any irregular section of any stream, of a current velocity which would carry a craft propelled by one ordinary oarsman, a distance of 26 miles in 3 hours. On the crest of the high-water wave of 1897, the writer made such a trip—from Simmsport to Melville—and the appearance of many of the eddies, at points in the river where the comparatively narrow reaches debouched upon the wider, was simply appalling; so that, though not fortified with the valuable statistical information now presented by Mr. Ockerson, the anomalies as to widths and depths of stream were, from observation alone, generally known and accepted long before his interesting study was taken up.

The effect of the struggle of the Atchafalaya River to get around and dislodge the extensive rafts known to have existed here and there within its bed, many of which the writer has seen and closely examined, must not be lost sight of, either, in accounting for the wider parts of the river and the consequent disturbance of regimen, tending to produce deep water in the wider reaches and shallower water in the narrower. Any stream undergoing the rapid development marking the history of the Atchafalaya must necessarily develop the apparent peculiarities, if not contradictions, pointed out by Mr. Ockerson, when compared with other streams of more orderly and deliberate action. The writer can also verify the author's deduction of the apparent relation between the volume of the flood to be carried and the cross-section of such a stream. This has been noticed by the writer, not only on the Atchafalaya River, but also on the Red River. Just as soon as the cross-section of a stream in process of development, such as in the Atchafalaya and Red Rivers, attains a certain uniformity of section, throughout a reach, caving often materially diminishes, if it does not actually cease. Mr. Kerr.

The enormous increase in slope, at flood stages, near the ends of the levee lines confining the waters of the Atchafalaya, to which Mr. Ockerson calls attention, is notably true. So great is this increase at such times, that while ordinary steamboats can make fair progress up stream on that part of the river above the influence of the ends of the levee, at times barely any progress is possible for many miles below.

In 1897 the actually recorded high-water slope of the Atchafalaya River, from Simmsport to Scotland Plantation, 23½ miles below its head and within about 7 miles of the lower end of continuous confinement by levees on both banks, at that time, was about 0.54 ft. per mile. From Scotland Point to Melville, 7 miles, it was 1.1 ft. per mile.

It is not surprising, therefore, to find the deep, wide section given as No. 4 immediately below Scotland Point; and this is but one instance of the many that preceded and continued to follow the extension of the levee lines.

With each successive stage of extension of the levee system, flood levels near the lower end are, primarily, materially raised, and an enormous increase of slope is created for some distance above the ends of the levees, precipitating the water into the unleveed portion below like a cataract, creating whirls, vertical, counter, and cross-currents and eddies, which could not but result in wider and deeper holes below than could, except under previous similar conditions in the progress of levee extension, prevail on the more regulated reaches of the river above.

Nor is this peculiar in the Atchafalaya River. Similar, though possibly not quite as marked instances are innumerable on Red River, a stream which has also undergone and is still undergoing rapid development. Here, too, the influence of levees in enlarging and deepening the bed of the stream is probably more striking than in almost any instance in the world.

It is an undoubted and incontrovertible fact that on that part of Red River in Louisiana where, for some 15 years past, levees have been maintained, and the waters of the river have been regularly confined to the main stream, the width of the stream has almost uniformly more than doubled itself, and the increase in depth has been generally not less than 25 ft., and in places much more. When first confined, the plane of high water was in many places materially raised, but it is now decidedly lowering, there being apparent, in the last two or three high waters, a lowering of flood levels, of as much as 2.0 ft. along those parts of the river confined by levees.

Mr. Kerr. The past relations of the Atchafalaya, and the Mississippi and the Red Rivers, are matters of history, and it is not necessary to lengthen this discussion further by reviewing them, but, as to its present relations, and their ultimate adjustment, much remains to be considered and said. There can be no serious division of opinion, however, as to the treatment necessary to bring about this ultimate adjustment, and the writer does not hesitate in placing himself on record as advocating that the Atchafalaya and the Red Rivers, in the proper course of time and events, should be divorced from the Mississippi. This would relieve and bring about equitable conditions in both those streams, where the influence of their relations is felt, and eventually result in improved conditions in the Mississippi, at and below what is called the mouth of Red River, as well.

Anyone who knows anything at all about high-water conditions on the Mississippi in the past knows that the highest stages of water at Red River Landing (on the Mississippi River, just below the mouth of Old River, more commonly, but erroneously, called the mouth of Red River) were brought about by a congestion of high waters at Barbres Landing, at the head of the Atchafalaya River, from the Red and the Mississippi Rivers, joined by crevasse waters from breaches in the line of public levee, on the Mississippi River, above the mouth of Red River, resulting in an elevation of surface at the point of conjunction, which brought about a reversal of slope and consequent tributary effect into the Mississippi.

If the line of public levee above the mouth of Red River be perfected and maintained, the divorcement of the Red and the Atchafalaya Rivers would, as a matter of course, result in a lowering of flood levels on the Lower Red River, and the Upper Atchafalaya, and, in the writer's opinion, if not at first, at any rate eventually, in the Mississippi River as well, in the vicinity of the mouth of Red River, with no material raising of flood levels below, that could not be readily taken care of.

The Atchafalaya, as an outlet to the Mississippi, is nothing more than a crevasse, tending constantly to deteriorate the body of the main stream below, as well as the connecting link between it and the Mississippi. The usual adjustment would undoubtedly follow the closure of the outlet or crevasse—rehabilitation of section, slope and velocity—and the much mooted and long discussed "Atchafalaya River Problem" relegated to the every-day treatment and control governing public work of similar character in the Mississippi Valley.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—This paper is extremely interesting, and the facts presented are unusually valuable, in that they tend to controvert many old absurd ideas about the hardness of the bottom of this stream based largely on wild theories. This brings to mind the old adage: "An ounce of good experience is worth a ton of vague theory." The words "vague theory" should be emphasized for the reason that the writer does not intend to cast a slur upon good reliable theory which has good reliable facts as a foundation. Mr. Le Conte.

After reading the category of interesting facts submitted by the author, one cannot help being impressed with his conclusion that the progressive building of the levees is responsible for the prodigious scouring out of the bottom at and near the free ends of the levees which existed at that particular time. That is to say, the first effect of building a single levee or twin levees results in scouring out the bottom at and near the outfall, or free, ends of the levees. As the twin levees are extended, the scoured channel gradually shoals up until the section becomes adapted to the normal discharge.

It seems to the writer, after due consideration, that the scouring of the river-bed in this case is undoubtedly due, not only to the local increase in slope, at and near the free ends of the levees, but also, to a much greater degree, to the entire alteration in the character of the vertical velocity curve, as shown in the vertical section, commonly known as the prime vertical. The bottom velocities he thinks are always enormously increased just as soon as the free ends of the levees are approached; that is to say, at full flood stage of the river. The true character of the channel discharge may be likened to that of a chute or sluice-way in every particular—the bottom velocities, as is well known, being enormously accelerated. As soon as the twin levees are extended sufficiently, of course the vertical velocity curve takes its normal shape again, the bottom velocity diminishes, and the bottom naturally shoals up to a normal section. This is a perfectly rational sequence, now that it is known, and it might have been anticipated if we had known how.

The writer was an eye witness to many interesting cases of "crevasse," or large breaks, in a line of new levee, during flood stages. The facts observed are worthy of record. Both ends of the break generally retreated rapidly until the width of the opening was some 3 000 ft. or more. One of these crevasses occurred on the Sacramento River not more than 20 miles above its mouth. Nothing particular was noticed about the break until some time afterward, when the river went down within its banks, and then it was surprising to note the quantity of material which had been deposited on the landward side. The new deposit was composed of an extensive field of barren sand, the surface being covered with gravel and pebbles, some of them as large as pigeon's eggs. The writer's first thought was that this material came from the river bank when cut away, but, as the river went down, it was found that the bank was composed of hard clay, and absolutely intact. To the

Mr. Le Conte. writer it always has been more or less of a mystery how that large quantity of heavy material was deposited on top of the high bank, 15 ft. above low water. There certainly must have been some enormous uplifting force, developed by the break, which caused such quantities of heavy material to be raised and landed on top of the high bank of the river. Whatever this uplifting force is, it certainly always exists and manifests itself at and near the free end of a levee during flood stage. The direct effect is an enormous scouring out of the bottom for some distance above the free ends.

The foregoing case is not a special one, as the same facts have been noted invariably along both banks of the river, wherever a crevasse has taken place. Another interesting case, going to show somewhat the same general phenomenon, is to be noted, at and near the foot or lower end of every new cut-off. The writer has watched closely many of these cut-offs in the river channel, and studied their progressive changes. The particular point to be brought out is the broad general and universal fact that a new bar is always formed across the foot of the old channel, and that the crest of this new bar is nearly as high as the general level of the river banks, say, 15 to 16 ft. above low water. Also—and this is the point to which the writer would particularly call attention—this new bar is composed of finer material on the bottom and much coarser material on top. In the upper river, where the currents are strong, the cobble stones, which occupy the entire crest of these bars—fully 16 ft. above the low-water plane—are as large as a man's head. By what means was this coarse heavy material lifted up and stranded so high above the low-water plane?

The only answer the writer can give, worthy of the name, is: Because such material always lies at the foot of a chute. This feature is always true of a new cut-off, before it has cut out a new channel to the normal section.

The writer is greatly interested in the practical effect exhibited by the one-bank levee. There is a great deal to be said in its favor, far more than river engineers, generally, are willing to admit. In India the single-levee system has been much more largely practised than in the United States. Such levees are far safer against extreme floods than twin levees, and, of course, cost only half as much, and sometimes less.

Moreover, the flooding of land, if not continued too long, is not an unmixed evil by any means. Ground squirrels, gophers, moles, grubs of every kind, and an endless number of fruit parasites, are done away with at each flooding. In fact, the farmers along the banks agree that an occasional flooding is highly desirable, if the water does not stay too long. Fruit and vegetables are always best and command the highest prices immediately after a general flooding.

The author's remarks about the fact that the erosion has been largely confined to the points, or convex shores, calls to mind an interesting article on this subject by a noted French engineer, M. Pasqueau, in 1889.* The article is too long to be quoted, even in brief, but the facts presented are very interesting. The same features have also been observed in tidal estuaries on the Pacific Coast, notably at Oakland Harbor and at Humboldt Bay. Where currents are strong, and the bends are not too pronounced, flowing water always tries to take the shortest course, that is, tries to follow a straight line as near as it possibly can; hence the deep-water pools are generally found off the points and not in the bends. Mr. Le Conte.

J. A. Ockerson, M. Am. Soc. C. E. (by letter).—The writer feels highly complimented by the discussion brought out on the Atchafalaya paper by those who have long been close observers of the phenomena described. Mr. Ockerson.

Mr. Richardson cites certain minor irregularities in contiguous sections of the Mississippi River, and no doubt similar conditions might be found in other streams; but the persistent repetition of wide and deep sections and narrow, shallow sections of the Atchafalaya is such as to mark this stream as unique in this respect.

It is particularly gratifying to secure the very valuable contribution to the discussion presented by Mr. Kerr, who has had a long personal acquaintance with all the streams of Louisiana. One of his most important statements verifies the observations of the writer as to the river under discussion, and as it may be more general in its application, it is well to call special attention to it as follows:

"Just as soon as the cross-section of a stream in process of development, such as in the Atchafalaya and Red Rivers, attains a certain uniformity of section, throughout a reach, caving often materially diminishes, if it does not actually cease."

Another important suggestion of the writer, confirmed by the discussion, relates to the enlargement and deepening of the bed of the river and the ultimate lowering of the flood line through the influence of levees.

It is interesting to note that both the above-named participants agree in the opinion that the Red and Atchafalaya should be divorced from the Mississippi River, and that the added volume would not result in a serious elevation of the flood level below the juncture of these streams. Valuable information bearing on this matter will be developed by the discharge measurements of the

* Exposition Universelle de 1889—Congrès International des Travaux Maritimes, "Le Port de Bordeaux et ses Passes," par M. Pasqueau, Ingénieur en Chef des Ponts et Chaussées.

Mr. Ockerson. present flood on the Lower Mississippi River which are now being made.

Mr. Le Conte cites cases where gravel and sand have been deposited on banks 15 ft. above low water. On the Mississippi River such deposits are sometimes carried to elevations of 35 ft. or more. In sharp bends, with the high water flowing across narrow necks of land to bends below, sand deposits sometimes accumulate to such an extent, even, as to kill the standing timber. In case of crevasses through levees, the sand deposits often effectually ruin extensive plantations.

If we were willing to abandon one-half of a river valley to the ravages of the floods it would doubtless simplify the matter of maintaining a "one-bank levee." The Mississippi River is not adapted to such treatment, neither would floods be regarded as blessings, as they are generally of too long duration and too great in depth. Without the protection of levees, but very small portions of the great basins of the Mississippi Valley would be habitable by man.

While "flowing water always tries to take the shortest course," the erosion of the banks of the Mississippi is invariably found to be in the bends and close along the concave shore. The main current is likewise along the same shore. If the particles of water in a section taken normal to the stream reach their ultimate destination at the same time, then it follows that the particles at the point end of the section must move very slowly while the other end of the section sweeps around the concave bend. Hence, instead of erosion at the point there should be a deposit, and this is found to be the case in the Mississippi River; but it is not true of the Atchafalaya.

AMERICAN SOCIETY OF CIVIL ENGINEERS
INSTITUTED 1852

TRANSACTIONS.

Paper No. 1039.

TWENTY YEARS' RUN-OFF, AT HOLYOKE, MASS., OF THE CONNECTICUT RIVER.

By Clemens Herschel, M. Am. Soc. C. E.

With Discussion by Messrs. H. V. Hinckley, John C. Hoyt and Clemens Herschel.

While Hydraulic Engineer of the Holyoke Water Power Company, 1879-1889, the writer instituted a system of keeping a daily record of the discharge of the Connecticut River at Holyoke, which was continued, after his departure, by his old-time assistant and successor, A. F. Sickman, Assoc. M. Am. Soc. C. E., until a full 20 years' record had been obtained, 1880 to 1899, inclusive. The record could not be continued because in 1900 the new stone dam of the company* flowed the water back on the old dam, and, by causing irregularity of flow over both dams, made further weir gaugings at these dams impossible. It seems desirable that the record of these 20 years' gaugings should be published in permanent form, so as to become, and remain, available to engineers, and the Holyoke Water Power Company having kindly furnished them, they are herewith given.

The figures in Table 1 represent cubic feet per second. They give the average for that 24 hours, and include water drawn through tur-

* *Transactions*, Am. Soc. C. E., Vol. XV, p. 543.

bine wheels; wasted over the dam; estimated leak of the dam; leak of the canals; and water used for purposes other than power.

It would serve no useful purpose to enter into a disquisition as to how the figures were arrived at. The hydraulic work at Holyoke has been carefully done, and the results may be considered more accurate than river gaugings as ordinarily found by the use of current meters and the like.

The drainage area, as determined from the best maps available in 1880, is 8 144 sq. miles.

Nor will the subject of rainfall be more than mentioned in passing. The object of this paper is the practical one of serving engineers in forming a judgment of what the discharge of the Connecticut River, or of a similar stream, similarly situated, may be expected to be during a series of years. For this purpose, a knowledge of rainfall is of no importance. What we wish to arrive at is river-flow, and this is reached better by gauging the river than by measuring rainfall, and then theorizing about and computing the river-flow from a computed average rainfall on the drainage area.

It is also well known that, with the same computed rainfall, the discharge of a river will vary greatly in different years; that a larger rainfall often produces a smaller run-off, depending on the distribution of the rainfall during the year and other such causes; and, generally, it is high time that rainfalls cease to have the attention given them as in the illogical reports on stream-flow of the past; and that considerations of stream-flow of rivers direct, no matter whence or how generated or originating, take their place.

The years when the gaugings here given were made need also not be given great weight in the discussion of these gaugings. Twenty years is a long enough period to present the presumable average or type of a like period in the future. As will presently appear, this consideration had influence in determining the manner of representing the 20 years' records in Table 1.

In the original, these 20 years' records were plotted in the form of twenty curves, one for each year, and formed, by plotting in the order of their magnitude and irrespective of the dates when they obtained, the gauged discharges of the river on the 312-314 week days of each of the 20 years in question. On Sundays, no gaugings were made, and, in the original plot, the Sundays present blank spaces.

TABLE 1.—TWENTY YEARS' FLOW OF THE CONNECTICUT RIVER.

Figures Represent Cubic Feet per Second in 24 Hours.

1880	1881	1882	1883	1884	1885	1886	1887	1888	1889	1890	1891	1892	1893	1894	1895	1896	1897	1898	1899
44 100	49 050	43 650	68 300	71 900	63 950	80 150	85 500	99 750	37 800	46 750	67 300	63 100	94 350	43 300	115 000	112 050	75 350	76 150	82 500
28 900	45 100	30 850	36 550	49 500	44 350	42 700	63 550	65 150	32 300	39 650	54 450	42 950	42 550	33 400	50 800	61 950	53 650	50 750	67 250
22 900	32 850	25 900	25 200	37 950	36 650	36 600	50 150	49 850	29 050	36 550	43 000	31 700	36 950	26 550	33 550	37 300	44 200	41 550	45 900
19 800	25 200	23 500	19 950	30 750	30 700	29 250	37 000	40 600	26 450	33 500	32 750	27 200	32 000	18 550	24 700	26 150	37 000	31 500	23 200
17 550	20 100	22 150	16 550	27 700	25 950	25 450	27 600	34 600	24 800	30 050	28 800	23 500	28 200	16 050	21 950	23 150	32 950	25 850	17 700
15 250	17 800	20 800	15 200	26 200	22 550	19 750	22 900	31 700	23 200	28 200	24 350	20 800	23 450	15 100	18 850	20 100	28 300	22 800	15 400
14 250	16 600	18 900	13 000	24 250	19 200	17 600	20 550	30 000	21 350	26 250	22 250	18 050	17 700	13 950	16 200	18 300	26 350	21 000	14 000
11 750	15 100	16 850	10 450	22 000	16 950	11 700	18 650	26 350	20 500	24 600	20 250	17 150	15 100	13 000	13 550	17 150	24 500	19 200	12 300
10 900	14 300	15 300	9 000	18 950	16 200	14 050	17 000	24 150	19 400	23 150	19 050	15 600	12 700	11 750	10 900	15 050	22 950	17 850	11 400
10 050	12 900	14 500	8 250	15 550	14 150	12 500	15 250	21 850	18 500	21 700	17 850	14 550	11 250	10 000	9 600	13 900	20 350	16 250	10 450
8 900	11 750	13 500	7 600	13 150	13 250	12 100	14 200	20 200	17 450	19 650	16 600	13 800	9 950	8 800	9 050	12 800	19 200	15 150	9 950
8 500	11 000	12 500	7 000	12 300	12 400	11 500	12 950	19 250	16 650	18 450	14 500	13 250	9 100	8 200	8 400	11 650	18 050	14 400	9 000
•8 300	10 050	11 600	6 550	11 100	11 550	10 900	12 300	18 250	16 100	17 750	13 700	12 600	8 500	7 800	7 750	10 700	15 800	13 750	8 150
7 950	9 400	10 500	5 950	10 300	11 050	10 250	11 450	17 600	15 550	17 100	12 500	12 200	7 800	7 400	7 250	9 800	16 100	12 950	7 750
7 500	8 850	9 250	5 150	9 250	10 550	9 800	10 750	16 700	14 650	16 050	11 350	11 600	7 300	6 950	6 300	9 300	15 300	12 450	7 200
7 300	7 900	8 350	4 850	8 750	9 200	9 600	10 150	15 600	14 250	14 950	10 500	11 100	6 850	6 550	6 050	8 550	13 950	12 000	6 600
6 900	7 150	7 500	4 650	8 150	8 850	9 200	9 850	14 300	13 550	14 400	9 600	10 250	6 450	6 250	5 800	8 050	13 200	11 550	6 050
6 350	6 650	6 950	4 400	7 550	8 300	8 850	9 500	13 500	12 800	13 700	8 800	9 500	6 050	5 950	5 200	7 700	12 650	10 700	5 650
5 350	6 350	6 400	4 100	7 200	8 050	8 500	9 150	12 400	12 150	13 400	7 600	9 100	5 900	5 700	4 900	7 150	11 800	10 150	5 400
4 600	5 500	5 750	3 900	6 700	7 800	8 100	8 850	11 400	11 650	12 800	7 200	8 600	5 650	5 450	4 750	6 850	10 550	9 700	5 350
4 000	5 200	5 050	3 750	5 950	7 650	7 800	8 500	10 600	11 250	12 050	6 650	8 150	5 200	5 200	4 550	6 500	10 200	9 200	5 250
3 650	4 750	4 850	3 650	5 700	7 250	7 400	8 250	9 200	10 850	11 400	6 100	7 850	4 900	5 050	4 350	6 250	9 700	8 750	5 150
3 500	4 200	4 400	3 600	5 600	6 800	7 050	7 900	8 500	10 500	10 500	6 000	7 500	4 750	4 950	4 100	6 050	9 150	8 450	5 000
3 350	3 800	4 150	3 500	5 350	6 600	6 600	7 650	8 000	9 850	9 900	5 550	7 000	4 600	4 800	3 950	5 800	8 550	8 050	4 650
3 200	3 700	3 800	3 450	5 100	6 250	6 000	7 550	7 650	9 500	9 600	5 250	6 600	4 450	4 750	3 800	5 250	8 000	7 700	4 500
3 000	3 600	3 700	3 400	4 900	5 900	4 900	7 350	7 300	9 150	9 000	5 100	6 400	4 350	4 650	3 600	4 900	6 500	7 550	4 150
2 850	3 400	3 650	3 350	4 600	5 550	3 900	7 150	7 100	8 650	7 950	4 850	6 150	4 250	4 500	3 500	4 650	5 250	7 200	3 950
2 350	3 100	3 550	3 250	4 350	5 450	3 450	6 600	7 000	8 000	7 250	4 600	5 900	4 050	4 250	3 400	4 500	5 000	6 700	3 550
2 350	3 000	3 500	3 100	4 150	5 250	3 300	6 300	6 800	7 550	6 850	4 400	5 550	3 900	4 000	3 300	4 400	4 900	6 150	3 300
2 150	2 950	3 450	2 950	4 000	5 000	3 200	6 150	6 500	7 250	6 300	4 150	5 150	3 850	3 600	3 150	4 300	4 700	5 850	3 200
2 150	2 950	3 250	2 900	3 700	4 850	3 150	5 750	6 300	7 000	5 950	4 050	4 950	3 750	3 350	3 150	4 150	4 450	5 050	3 050
2 050	2 950	3 150	2 850	3 600	4 600	3 000	5 400	5 800	6 600	5 300	3 600	4 700	3 600	3 000	3 050	3 900	4 350	4 750	2 800
2 050	2 950	3 000	2 750	3 500	4 300	2 800	4 600	5 350	6 250	4 800	3 500	4 500	3 350	2 750	3 000	3 750	3 900	4 400	2 700
2 000	2 950	2 900	2 500	3 100	4 150	2 700	4 300	4 800	5 900	4 300	3 150	4 400	3 200	2 450	2 950	3 450	3 700	4 000	2 500
1 950	2 950	2 850	2 300	2 950	3 850	2 500	4 150	3 700	4 650	3 700	2 950	4 300	3 050	2 350	2 900	2 850	3 550	3 600	2 400
1 750	2 850	2 500	2 150	2 700	3 550	2 350	3 750	3 450	3 650	3 550	2 900	4 150	2 800	1 900	2 850	2 700	3 350	3 450	2 350
1 650	2 700	2 400	1 950	2 650	3 050	2 100	3 650	3 000	3 100	2 750	2 750	3 000	2 550	1 600	2 700	2 100	3 000	3 200	2 250

To have reproduced these twenty curves would, it was thought, not be so convenient in use as to present their data in the form of a table. And to attempt to plot all twenty of the curves to the same co-ordinates would require color printing to prevent confusion.

Abstaining now from presenting a record of the precise years when the gaugings were made, and aiming only at presenting a 20 years' table for future use, it seems allowable to make Table 1 by giving, in the head line, the maximum day of flow of each of the 20 years, and thereafter of every tenth day following, arranged in order of size, with the lowest day's run in the year as a last entry in each column. When such tenth day happened to be Sunday, the average of the Saturday and Monday, preceding and following, was given as the flow for that tenth day.

On the other hand, should it ever be required to reproduce the record of any one certain year from Table 1, it will only be necessary to replot that year from the indications just given.

For water-power purposes, the most instructive part of Table 1 will be the lower half of it—below a selected row of the figures between the two heavy horizontal lines.

It will be found that this selected horizontal row represents the stream-flow which, by a process of "survival of the fittest," has been found on streams in general to be the measure of quantity for which it is the part of a prudent mill-owner or water-power company to supply wheel capacity, when setting the wheels to be driven on that river, at that point. A greater wheel capacity is not profitable because the wheels stand idle too long. A less wheel capacity causes too much water to run by unused.

To furnish the necessary power for making all the stream-flows below the selected horizontal line of figures supply a uniform amount of power, is the office of the auxiliary steam plant, which is always requisite (on the outflow of the Great Lakes alone excepted) to cause the investment in dams, canals, power-houses, etc., to yield its greatest possible return.

Even so, and in spite of the ordering of 20 years' data, represented by Table 1, we find the upper of these horizontal lines to vary from 15 600 to 4 850; and the lower one, from 12 050 to 3 750.

The lowest run for the whole year varied from 3 650 to 1 600; showing that double, three-fold, and even more times, the lowest run of the river is allowable, as representing the proper turbine-wheel capacity to be set, on rivers of this class.

Other computations may be made and lessons drawn from Table 1; such as the greatest auxiliary steam power that will be needed on any one day to bring the power furnished by the river up to the desired average or "peak-load" power; the average auxiliary steam power needed during an average year to bring the output of water-power during such a year up to the desired average, or "peak-load," annual output of power; and many more such questions as they arise.

It is much to be desired that the United States Geological Survey will publish its records of stream-flow in some such form as that here exemplified.

DISCUSSION.

Mr. Hinckley. H. V. Hinckley, M. Am. Soc. C. E. (by letter).—One of the elements entering into the availability of a stream for power purposes is illustrated by comparing the Connecticut and Kansas Rivers. The record of the Kansas River at Topeka is about as follows:

Maximum daily record..........	221 000	cu.	ft.	per	sec.
Maximum ten days' record......	173 450	"	"	"	"
Average monthly record.........	4 000	"	"	"	"
Two months' minimum record..	550	"	"	"	"
Daily minimum record..........	376	"	"	"	"

The minimum ten days' flow is but little if any less than the two months' minimum. The writer assumes them to be equal. On the Connecticut, in 1896, the year of greatest variation, by Table 1, the 10 days' maximum rate of flow was 53 times the 10 days' minimum, while at Topeka the maximum 10 days' flow is 316 times the minimum 10 days' flow, and the maximum daily is 588 times the minimum daily.

Mr. Hoyt. John C. Hoyt, Assoc. M. Am. Soc. C. E. (by letter.)—The two points brought out in Mr. Herschel's paper, namely, the estimation of run-off from rainfall, and methods of publishing stream-gauging data, are ones in which the writer is especially interested, and to which he has given considerable study.

In regard to estimating rainfall from run-off: The truth of the statements made by Mr. Herschel is so evident to one who has given the matter any consideration that it is difficult to understand how engineers can continue to estimate run-off from rainfall data. The writer has just compiled, and presented for publication, a comparison of the rainfall and run-off over the following drainage areas in the northeastern portion of the United States:

TABLE 2.

River.	Drainage above.	Length of records. in years.
Connecticut	Orford. N. H...........	5
Housatonic	Gaylordsville. Conn..................	5
Susquehanna	Harrisburg. Pa	14
Ohio............................	Wheeling. W. Va....................	21
Potomac.........................	Point of Rocks. Md.............	10
James	Cartersville, Va.......................	7
Roanoke............	Roanoke. Va...............	9

Mr. Hoyt.

Table 3 gives the minimum, maximum, and mean run-off as percentages of the rainfall for 1894 to 1905, inclusive, for the Ohio River above Wheeling, W. Va. This is typical of the data for the other basins, and shows the great variations to which the relation of these two phenomena is subject:

TABLE 3.—Mean, Maximum, and Minimum Run-Off, as Percentages of the Rainfall, for the Ohio River above Wheeling, W. Va., for 1894-1905, inclusive.

(*Drainage area, 23 820 sq. miles.*)

Month.	Minimum.	Maximum.	Mean.
January	46	129	86
February	54	151	98
March	82	191	120
April	58	156	98
May	18	82	48
June	10	67	30
July	9	53	23
August	5	56	20
September	6	39	17
October	6	72	27
November	12	114	39
December	27	99	64
The Year	44	63	54

In preparing stream-flow data for publication, the Geological Survey has endeavored, as far as funds for publication are available, to present the data collected in such a form as will be most useful to engineers. Those engaged in this work fully appreciate the desirability of additional features in these reports, such as the one mentioned by Mr. Herschel, and have added them in all cases when funds have been available for their preparation and publication.

In the Water-Supply Papers which will contain the results of stream measurements for 1906, tables giving the duration of flow for the principal stations will be published. The method adopted, as shown by Table 4, is somewhat different from that suggested by Mr. Herschel; it is believed that this form will be of more practical value to the engineer and general reader.

In addition to these tables of duration, the 1906 report will contain statements in regard to the accuracy of the computed monthly flow.

The Survey welcomes suggestions and criticisms, such as have been given in Mr. Herschel's paper, and will be glad to receive the ideas of other engineers.

TABLE 4.—DURATION OF DISCHARGE, AND CORRESPONDING HORSE-POWER (80% EFFICIENCY) PER FOOT FALL, FOR THE POTOMAC RIVER AT POINT OF ROCKS, MD.

Horse-power.	Discharge, in cubic feet per second.	Days Deficiency.*											
		1895†	1896‡	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906
90	990	......	......	......	......	......	......	......	......	......	6	......	......
100	1 100	5	......	......	......	......	3	......	......	......	23	......	......
120	1 320	40	14	......	......	......	39	......	12	......	58	......	......
140	1 540	79	28	3	......	......	78	......	25	......	86	......	......
160	1 760	107	43	9	6	23	101	......	51	......	107	14	......
180	1 980	124	60	29	8	50	110	2	51	......	107	14	......
200	2 200	128	72	56	29	82	115	5	83	22	126	32	......
250	2 750	150	101	98	59	133	140	49	109	59	140	46	9
300	3 300	162	130	107	71	152	152	60	142	74	161	90	28
350	3 850	182	162	137	95	172	173	88	162	103	196	130	59
400	4 400	190	187	162	106	176	193	101	175	128	207	149	106
450	4 950	201	222	186	128	188	223	126	185	139	210	162	128
500	5 500	208	238	195	136	197	233	133	195	161	233	195	151
More than 500	More than 5 500	110	128	170	229	168	132	232	170	204	143	170	214

* Gives the number of days when the horse-power, and corresponding discharge, is less than that given in the first and second columns.
† February 17th to December 31st.
‡ Missing days estimated from Millville records.

Mr. Herschel

CLEMENS HERSCHEL, M. AM. SOC. C. E. (by letter).—Mr. Hoyt's Table 3, giving, among other such data, 191% of the rainfall as a run-off in a single month, is a good example of the absurd computations he and the writer have wished to illustrate.

Whether the form of Mr. Hoyt's Table 4, or that of the writer's Table 1 is to be preferred, each engineer will have to decide for himself. It has seemed to the writer that, for popular illustration, the form of Table 4 may have value; but that for engineers' use in their work, the form given in the original paper was the better.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1040.

THE NECAXA PLANT OF THE MEXICAN LIGHT AND POWER COMPANY.*

By F. S. Pearson and F. O. Blackwell, Members, Am. Soc. C. E.

With Discussion by Messrs. Edwin H. Warner, H. F. Labelle, F. G. Baum, J. D. Galloway, and F. O. Blackwell.

The Mexican Light and Power Company was formed to develop the water power of the Necaxa, Tenango and neighboring rivers and transmit it to the Cities of Mexico and Puebla, and to the mining camps of Pachuca and El Oro. About 90 miles northeast of the City of Mexico these streams have cut through the mountains surrounding the elevated central plateau of Mexico, and fall abruptly to the low country bordering the Gulf. The rivers come together a few miles below, and eventually discharge into the Gulf of Mexico some 70 miles farther east. There is a series of picturesque falls like those in Fig. 1, Plate XI, with vertical drops of from 300 to 800 ft., and a total fall of 4 500 ft. in 10 miles.

The drainage area of the Necaxa and Tenango is approximately 200 sq. miles, which will be largely increased by diverting neighboring streams into the Necaxa Basin. There is more or less rain throughout the entire year, the heaviest precipitation occurring

*Presented at the meeting of January 2d, 1907.

during the months of June, July, August, September and October. The driest month is May.

At Necaxa the rainfall for the 6 years during which records have been kept has varied from 85 to 135 in., with an average of 105 in., while on the upper drainage area there is only half as much precipitation. The combined flow of the two principal streams has been

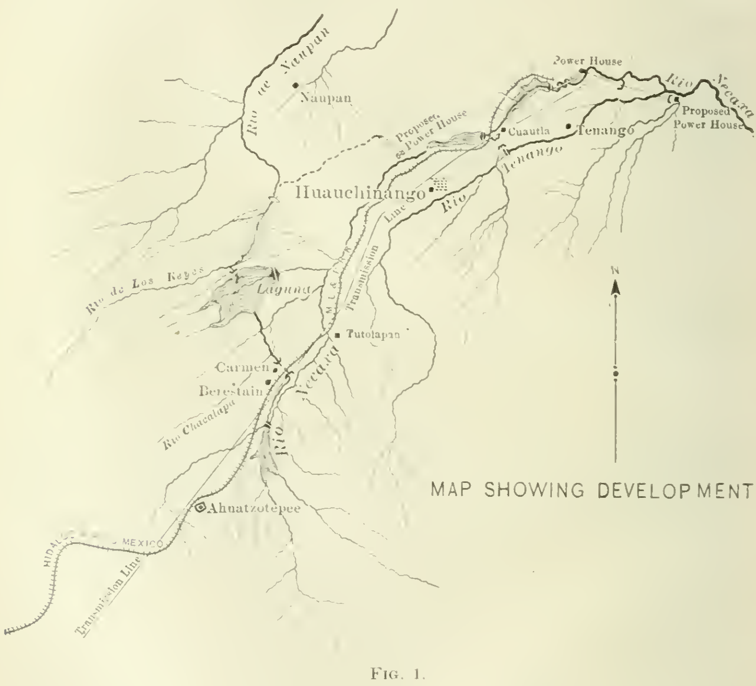
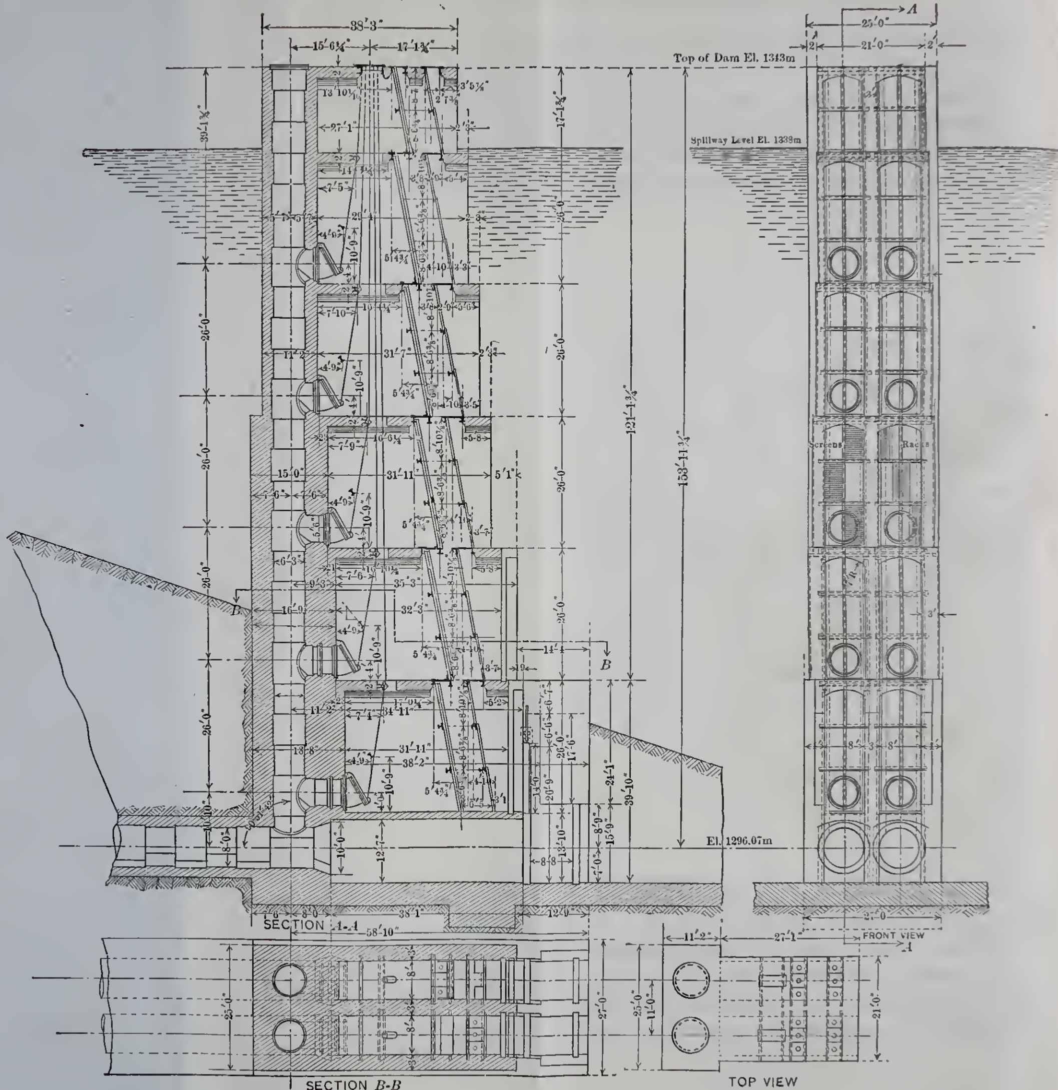


Fig. 1.

as low as 90 cu. ft. per sec., and the maximum observed flood was 3 000 cu. ft. per sec., which will probably be exceeded at times. The average run-off during the year of minimum rainfall was 245 cu. ft. per sec., and the average for 7 years was 350 cu. ft. per sec.

Fig. 1 is a map which indicates the general scheme of development. The water of the Tenango is diverted into the Necaxa Valley

GENERAL. VIEW OF INTAKE FOR DAM No. 2 NECAXA.



by a 40-ft. dam and a tunnel 12 ft. wide, 9 ft. high, and 3 000 ft. long. This tunnel is lined with concrete wherever there is not solid rock, and is constructed with a slope of 0.004. This will give a capacity of 875 cu. ft. per sec., which is sufficient to carry all but the extreme flood waters of the Tenango.

Storage Reservoirs.—No good storage basins exist on the Tenango water-shed, but there are a number on the Necaxa. Of these, the farthest down stream is the Necaxa, which is below the Tenango Tunnel, and above are the Texcapa and Laguna Reservoirs. The storage capacities will be 1 585 000 000, 642 000 000 and 700 000 000 cu. ft., respectively. The Laguna Reservoir, however, will be increased eventually to a capacity of 2 450 000 000 cu. ft. by raising the dam after its drainage area has been increased by the diversion of the Los Rayos and other neighboring streams into it. This storage, amounting to 4 600 000 000 cu. ft., will be sufficient, not only to equalize the entire run-off of the streams during any year, but to enable the average flow of a succession of years to be depended upon.

Dams.—The rock at the sites of the various dams is of volcanic origin and not sufficiently reliable to be used as a foundation for masonry. For this reason it was decided to build earth dams by the hydraulic-fill method, utilizing water at heads up to 400 ft.

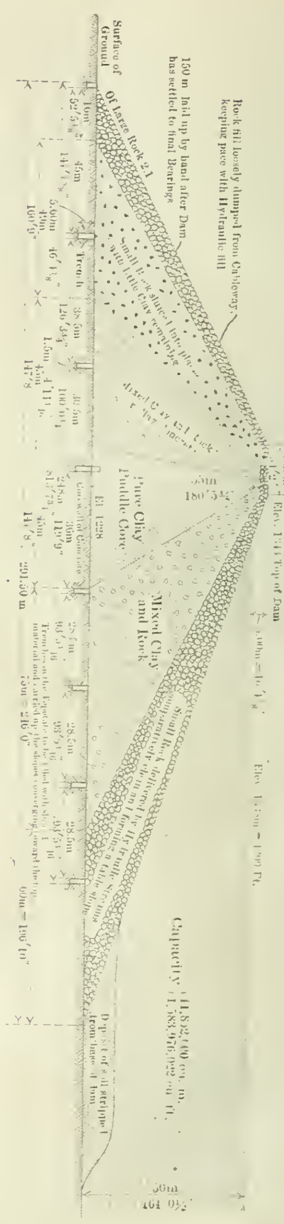
The Necaxa Dam will be 180 ft. high, 1 276 ft. long at the crest, with a thickness of 950 ft. at the base, and of 54 ft. at the top. The slopes will be 3 to 1 on the up-stream and 2 to 1 on the down-stream faces. About 2 000 000 cu. yds. of material will be required in its construction, and this will be obtained from the neighboring hills and sluiced into place.

Fig. 2 is a cross-section of this dam. It will be noted that the faces will be covered with broken stone 8 ft. thick at the top and 60 ft. thick at the bottom. The method of construction is similar to that shown in Fig. 2, Plate XI, which is a photograph taken during the construction of the small Tenango Dam. The ground is first cleared and stripped, a trestle to support the flume is erected, and low earth dikes are made at the up-stream and down-stream limits of the fill, to hold the mud and water. The material is then sluiced in, the pipes discharging near the embankments so that the boulders and gravel are deposited on the faces, and the fine mud in the center, of the dam. The dikes are raised as the dam fills, and the water spills over the up-stream face into the pond. During construction the water of the river is passed through the discharge gates, which are made large enough for the purpose. A spillway is provided for the Necaxa Reservoir over a neck of rock to the north of the dam, and the overflow returns to the river below the power-house. The outlet is through a rock tunnel, 1 550 ft. long, near the southern end of the dam and 153 ft. below the crest. The water is admitted to the pipe lines through the gates shown in Plate IV. In order to avoid the operation of gates and racks under such high pressures, two vertical stand-pipes, each with five flood-gates, have been constructed, either of which will permit all the water required for power purposes to pass. By this arrangement it will not be necessary to operate the gates or racks under a head of more than 26 ft.

The Texcapa Dam will have a height of 174 ft., a thickness of 905 ft. at the base, and a length of 1 191 ft. at the crest.

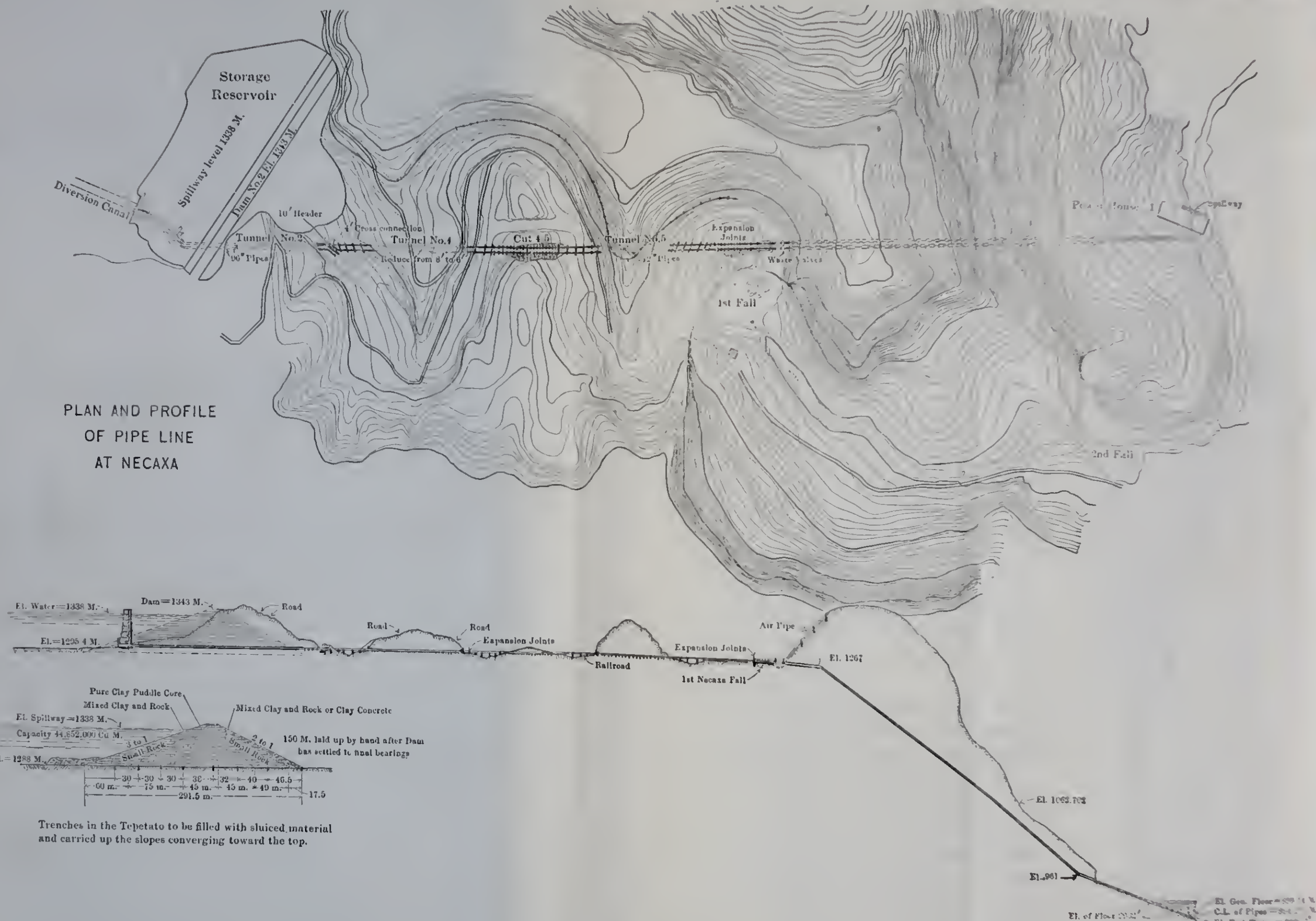
The small Laguna Dam now being constructed will have a height of 40 ft., a length of 400 ft., and will form the up-stream toe of the higher dam.

Pipe Lines.—Two riveted steel



TYPICAL SECTION OF DAM NO. 2, NECAXA RESERVOIR.

FIG. 2.



PLAN AND PROFILE OF PIPE LINE AT NECAXA

penstocks, 8 ft. in diameter and ⅜ in. thick, pass through the Necaxa Tunnel, with sluice-gates discharging into the river. Below this point they are reduced to a diameter of 6 ft. and continue down stream a distance of 2 200 ft., 800 ft. of which is through tunnels. Near the upper Necaxa Fall these pipes are joined by two receivers, each 22 ft. long and 7 ft. in diameter. From the receivers six smaller pipes pass down to the power-house through two inclined tunnels. On each of the 30-in. pipes there are relief valves, 30 in. in diameter, pipes from which lead up the hillside 310 ft. to an elevation above the crest of the dam. All pipes are connected with the receivers through valves, and a central gate separates the two halves of the system, so that either half can be shut down without interfering with the other, as shown in Fig. 1, Plate XII.

Two expansion joints, of the diaphragm type, are provided in each of the 6-ft. pipe lines, and there are packed slip-joints in each of the 30-in. pipes.

The velocity of the water in the upper and lower pipe lines, when the plant is running at full load, is 7.5 and 15 ft. per sec. Under extreme overload conditions, the velocity is at times as high as 8.7 and 18 ft. per sec.

All pipes are supported on concrete piers throughout their length. Plate V shows a plan and profile of the pipe line. The six pipes from the receiver are carried down to the power-house, a distance of 2 460 ft., the upper 1 900 ft. of which is through two parallel tunnels, 13 ft. wide and 10 ft. high, constructed at an angle of 41° from the horizontal. A track and cableway will be constructed on top of the pipes in one tunnel, through which supplies for the power-house will be transported. The pipes with flanges are forged complete from one piece of sheet steel. Before the flanges are hammered out, two cast-steel clamping rings are slipped on each 30-ft. section of pipe. The outside diameter of the tubes is 30⅝ in. throughout the entire length of the line. The internal diameter is less at the lower than at the upper end, on account of the greater thickness of the metal, which varies from 0.4 to 0.95 in., the minimum diameter being 29 in.

The details of the pipe joints are shown in Fig. 3, and the lower end of the pipe line, as seen from the roof of the power-house, is shown by Fig. 2, Plate XII.

The static head at the water-wheel, with the reservoir full, is

1 452 ft., which in service is reduced to between 1 200 and 1 300 ft. by the reservoir surface being lowered and by frictional and other losses.

Each section of tube, before being shipped from the works of the manufacturer, The Actien Gesellschaft Ferrum, of Kottowitz, Germany, was subjected to a hydrostatic test of one and three-quarters

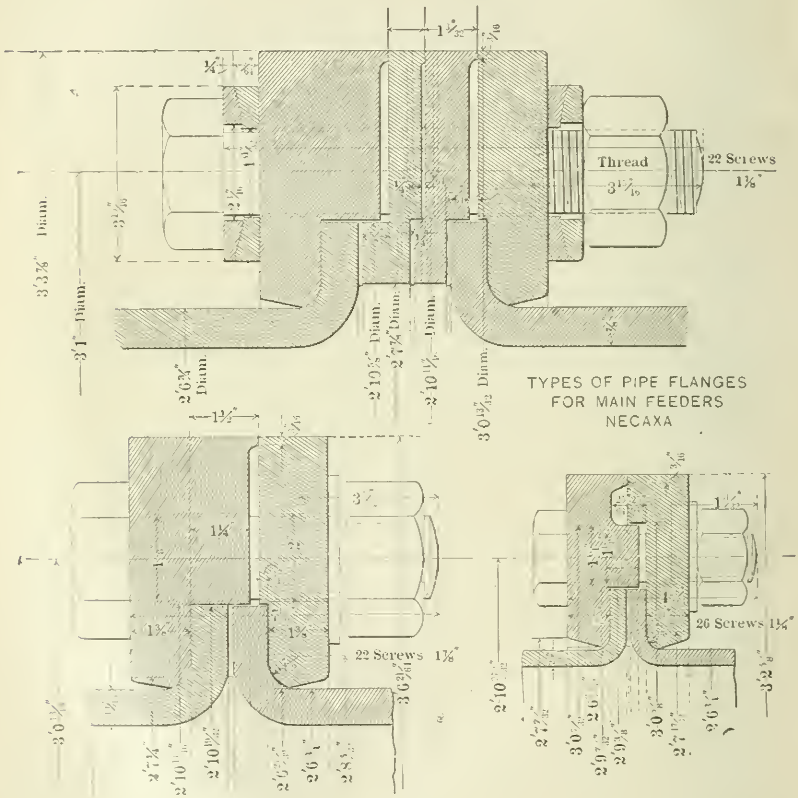
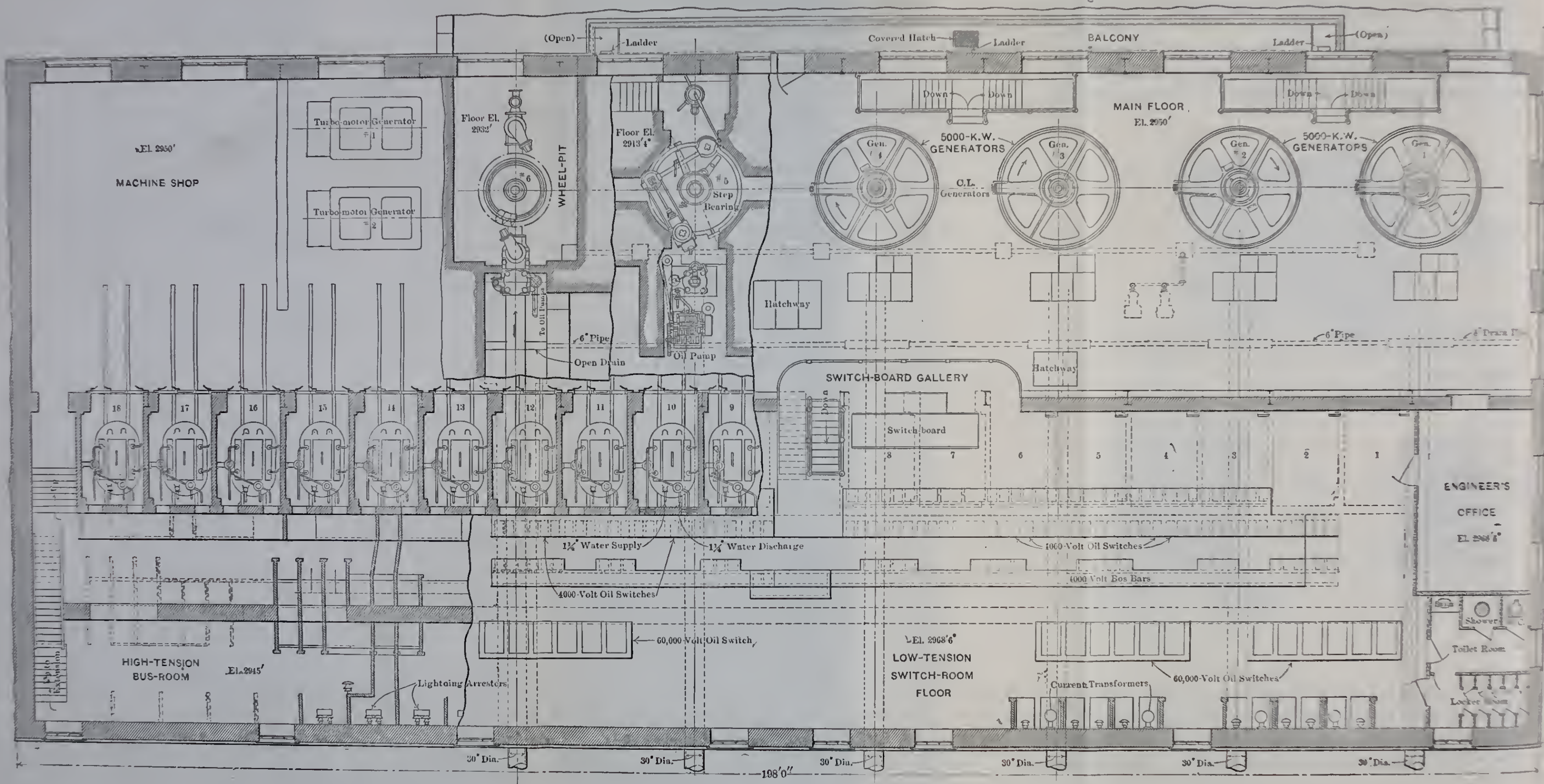


Fig. 3.

times the maximum pressure which it is under in service. The form of pipe is stronger and more reliable than one built up of plates riveted together, and the smooth interior will materially reduce the loss of head due to friction, especially at the higher velocities occurring at times of overload.

GENERAL PLAN OF POWER-HOUSE NO. 1. SHOWING DIFFERENT FLOOR LEVELS. NECAXA.



Power-House.—The power-house is located in the cañon below the lower Necaxa waterfall, with a vertical drop of 740 ft., as illustrated in Fig. 1, Plate XIII.

The building stands on a massive concrete base which forms the foundation for the hydraulic and electric machinery. The walls are of concrete, with steel columns in the front and center to support the crane girders and roof trusses.

The roof is of ferro-inclave covered with cement plaster, the window frames are of steel, the floors of cement, and no combustible materials are used in any part of the structure.

The dimensions and general arrangement of the power-house are shown on the plan and elevation, Plates VI and VII, and in the photograph, Fig. 2, Plate XIII, taken while the building was under construction. The front half, over which travels a 40-ton electric crane, contains the water-wheels and generators. The back part is divided into two stories, the lower being occupied by the transformers and the high-potential electrical connections, and the upper by the switches, switch-board and low-potential wiring.

The building is 235 ft. long, 88 ft. wide, and 37 ft. 6 in. from the main floor to the roof trusses. At one end is a machine shop and store-room for supplies and spare parts.

Hydraulic Apparatus.—The pipe lines pass into the basement of the building and terminate in 28-in. valves operated by hydraulic power. Each of the six impulse-wheels is rated at 7 000 h. p., with a maximum capacity of 9 000 h. p., making it possible to supply a peak load of 50 000 h. p. from the station. The wheels are of 100 in. pitch diameter, run at 300 rev. per min., and have solid cast-steel center discs to which are clamped 24 steel buckets, as shown in Fig. 1, Plate XIV. Each has two 4½-in. square, regulating nozzles, fixed on opposite sides of the wheel, but joined together, so that they are opened and closed simultaneously.

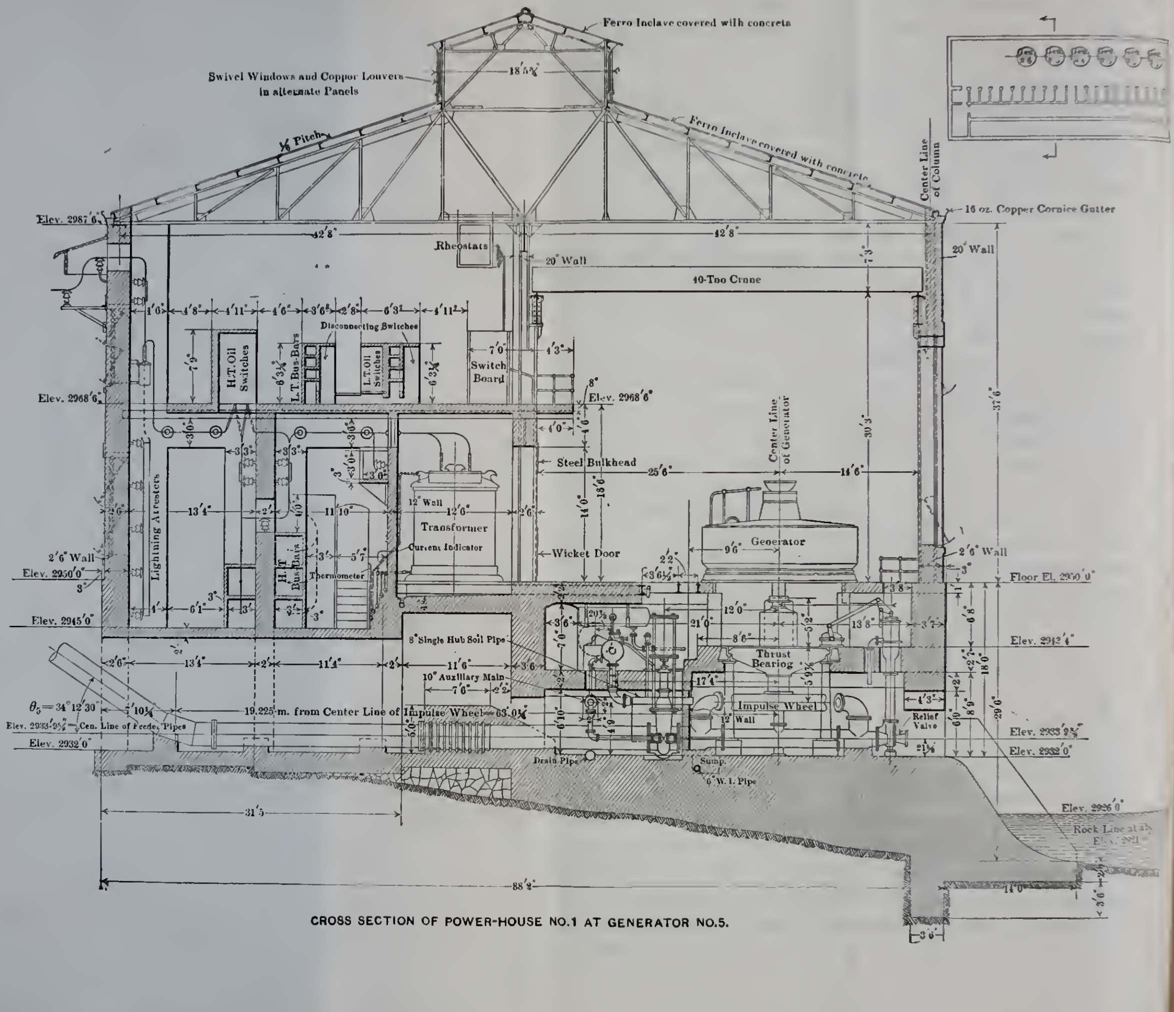
A by-pass valve is provided at the end of the pipe line which feeds each unit, and is mechanically connected to the nozzles, so that when one opens the other closes, keeping the flow of water in the pipe constant. This entirely avoids the possibility of rams, which might prove disastrous with such a long penstock and high velocity.

A vertical shaft, 14 in. in diameter, and flanged at the bottom to take the water-wheel disc, carries the revolving parts of both the generator and water-wheel. The weight is supported by a thrust-bearing, 30 in. in diameter, under which oil is forced, at a pressure of 150 lbs. per sq. in., by a pump driven either by a small water-wheel or by a belt from each main wheel. This construction has many marked advantages for large units over impulse water-wheels heretofore built with horizontal shafts and single deflecting nozzles. The double nozzle reduces the size of both the jet and the bucket, and permits the use of a water-wheel of smaller diameter and higher speed of rotation without sacrifice of efficiency. The mechanism is more easily operated than a deflecting nozzle, and the two nozzles being set in opposition to each other there is no thrust on the steady bearings, and all the parts can be made much smaller than would have been required for a horizontal shaft. The diameter and length of the shaft itself can also be much reduced. The oil thrust-bearing is a very simple and reliable device which operates with a minimum of attention. The maximum quantity of water supplied to each wheel can be adjusted at the governor, so that water will not be wasted unnecessarily, and the by-pass is also arranged so that it can be adjusted to close slowly after opening—a feature which is desirable with fairly steady loads.

The governor controls the valves which admit either oil from the pump (which supplies the thrust-bearing), or water from the penstock, to the pistons which operate the nozzles and by-pass. Under actual running conditions the speed regulation is extremely close, and the by-pass wastes practically no water except when the circuit breakers throw the full load off the plant instantly.

All the hydraulic apparatus was supplied by Escher-Wyss and Company, of Zurich, Switzerland.

Electric Apparatus.—Each water-wheel drives a 5 000-kw. alternator, of the revolving-field type, which generates three-phase current at a periodicity of 50 cycles, and a potential of 4 000 volts. Two of the alternators are equipped with a 60-kw. direct-connected, 125-volt exciter, but in ordinary operation exciting current will be obtained from two 250-kw., 125-volt generators driven by induction motors wound for 4 000 volts. Exciting sets taking power from the main units are used because with small wheels the nozzles are constantly getting stopped up by materials floating in the water. The generator is shown by Fig. 2, Plate XIV.



CROSS SECTION OF POWER-HOUSE NO.1 AT GENERATOR NO.5.

The low-potential current from the generators is raised to a pressure suitable for long-distance transmission by five banks of oil transformers, each bank consisting of three 2 000-kw. units, which can deliver power, at 40 000, 50 000, or 60 000 volts, to the transmission circuits. The transformers are shown in Fig. 1, Plate XV. They are placed in a closed fire-proof compartment separated from the generator-room by steel bulkheads on account of the large quantity of oil used for insulation. The heat generated in the transformers, due to the losses in the iron and copper, is carried away by copper cooling coils in the cases, through which water is constantly circulated by motor-driven pumps. Plate VIII is a diagram of the electrical connections, which are made as simple and direct as possible.

The wiring consists either of insulated cables in conduits or of bare wires in brick and concrete compartments, so that the damage from an arc is limited, and cannot injure adjoining circuits. This is particularly important in laying out the 60 000-volt conductors, and a substantial concrete wall isolates every wire, switch or lightning arrester. The main switches are all of the oil type, controlled electrically from a distance, and any switch will open instantaneously a short-circuit of the entire power system. They are also equipped with disconnecting air switches so that the oil switches and other devices can be cut off from the system for repair and inspection.

The oil switches are on the gallery, with the controlling switchboard in the center, where the operator can obtain an unobstructed view of the machinery in all parts of the building. The small controlling switches for the large oil switches, with the miniature lamps which indicate their position, are mounted on a bench-board with dummy bus-bars to form a wiring diagram, so that the attendant can always see how the apparatus is connected, and thus avoid mistakes.

On vertical panels above the bench-board are the indicating instruments which show at any instant the speed, potential and power at which the plant is operating. These instruments are also placed in such a manner, with relation to the dummy busses and operating switches, that the operator cannot become confused. At the back of the board are the registering instruments which make a record of the pressure and output of the plant. The switch-board forms a room surrounded by ornamental ironwork, with the instruments and controlling devices on the outside, and doors at the ends to give access to the inside connections, as shown in Fig. 2, Plate XV. The transformers and switch-board were furnished by the General Electric Company, of Schenectady, N. Y. The generators and exciters were supplied by the Siemens-Schuckert-Werke, of Berlin.

Construction Plant.—The Necaxa development being located in one of the most inaccessible parts of the State of Puebla, it was first necessary to establish communications, by constructing some 30 miles of wagon roads. At the start, traction engines were used, but proved unsatisfactory, on account of the constant rains during the first year of active operation, and eventually the Hidalgo Railroad was extended to Beristain, from which point the Mexican Light and Power Company built its own narrow-gauge line 21 miles to Necaxa. This road is remarkable from the fact that only 10% of it is tangent, many of the curves are of 60 ft. radius, and the grade for a considerable portion of the line is 6 per cent.

The company now operates some 27 miles of track, 3 Shay locomotives and 20 flat cars, in addition to the locomotives and cars used in excavation work.

The machinery and materials for the power-house were lowered at the two falls by cableways capable of handling 15 tons.

A temporary plant of 1 000 h. p. at the upper falls supplied 500-volt continuous current for lighting and power, and compressed air for construction purposes.

An engineering camp for the management was built on the mesa, 1 700 ft. above the power-house, and three new towns, to house the 4 000 Mexicans employed on the work, and to replace the native villages flooded out by the reservoirs, were built.

Extensions of Power Plant.—The future plans of the Mexican Light and Power Company contemplate the construction of another power-house, immediately below the existing one, to utilize the same water under a head of 700 ft. and developing 20 000 h. p. There is also a fall of 2 100 ft. between the Laguna and Texcapa Dams, which will be developed when required, and can furnish 30 000 h. p. additional.

Transmission Lines.—Four 60 000-volt power transmission circuits and two tower lines have been built from the power-house to

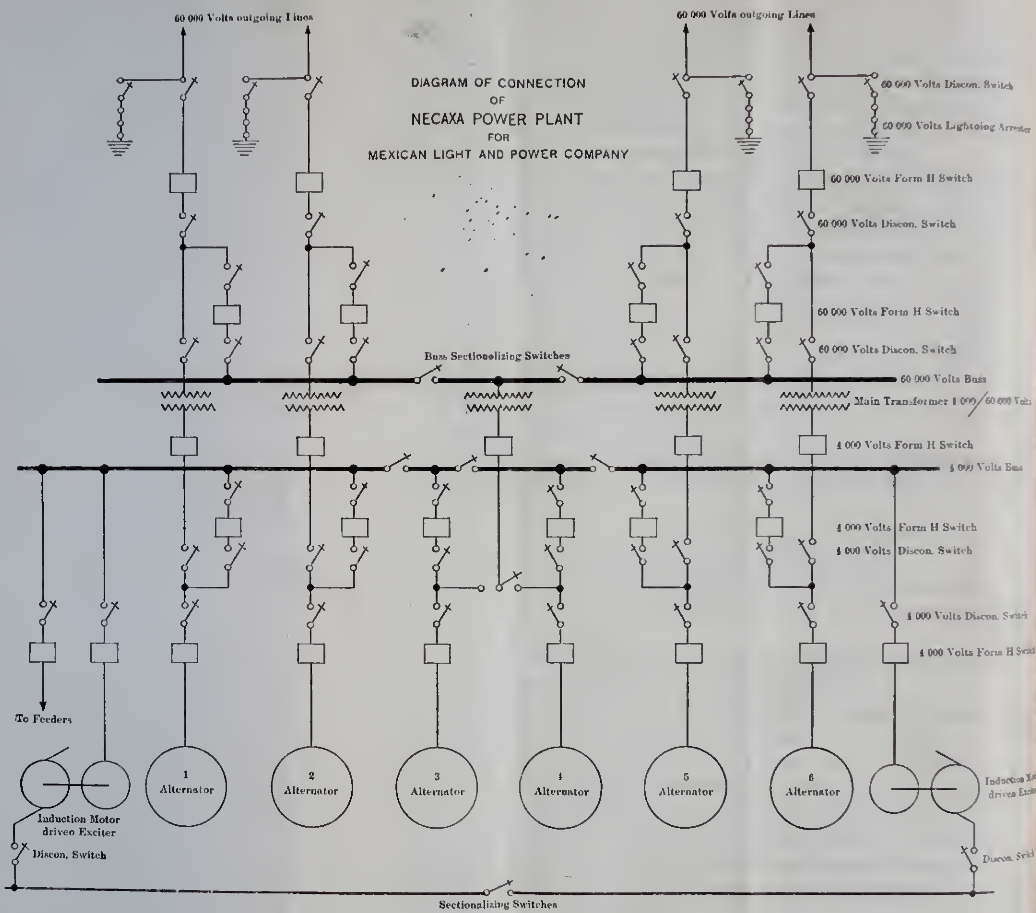


DIAGRAM OF CONNECTION
OF
NECAXA POWER PLANT
FOR
MEXICAN LIGHT AND POWER COMPANY
60 000 Volts outgoing Lines
60 000 Volts outgoing Lines
60 000 Volts Discon. Switch
60 000 Volts Lightning Arrester
60 000 Volts Form H Switch
60 000 Volts Discon. Switch
60 000 Volts Form H Switch
60 000 Volts Discon. Switch
Buss Sectionalizing Switches
60 000 Volts Buss
Main Transformer 4 000/60 000 Volts
4 000 Volts Form H Switch
4 000 Volts Buss
4 000 Volts Form H Switch
4 000 Volts Discon. Switch
4 000 Volts Discon. Switch
4 000 Volts Form H Switch
To Feeders
1 Alternator
2 Alternator
3 Alternator
4 Alternator
5 Alternator
6 Alternator
Induction Motor driven Exciter
Discon. Switch
Sectionalizing Switches

the City of Mexico, and two circuits from the City to El Oro, as shown on the map, Fig. 4. They are carried by steel towers, on each of which are two circuits of 168 000 cir. mils, copper cable, equivalent in cross-section to No. 000 B. & S. gauge wire, supported on porcelain insulators 14 in. in diameter. The towers for the transmission line are built up of steel angles, all parts being heavily galvanized after all machine work had been completed. Plate IX shows the dimensions of the standard tower, and the photographs, Figs. 1 and 2, Plate XVI, illustrate the line construction. The towers are 58 ft. high, over all, the feet being set 6 ft. in the ground. The cables are supported 40 and 46 ft. above the ground, the conductors forming two equilateral triangles with 6-ft. sides. A 14-ft. extension in the center of the tower carries a galvanized-steel cable with lightning arrester, and protects the electrical apparatus from damage. The towers will stand a horizontal side strain of 1 650 lb. at each insulator, or 10 000 lb. altogether. The standard distance between supports is 500 ft., but spans up to 1 500 ft. are made with higher towers having greater strength.

In tropical countries the use of wooden poles is objectionable on account of their short life, and in Mexico the cost is also excessive as they must be imported from abroad. The substitution of steel not only reduces the first cost, but also gives a strong, permanent and fire-proof construction, making the line much less likely to be interrupted. Towers can be handled more conveniently than poles in a mountainous country without roads, as they can be knocked down and packed in light bundles for mule-back transportation and then readily assembled at the points where they are to be erected.

With long spans the smaller number of insulators is also an advantage, in that it permits larger and safer insulators and the use of higher potentials than is possible with short spans. The insulators are made in three parts, which are shipped separately and cemented together on the ground in Mexico. Each part is tested before shipment by subjecting it while wet to a test potential of 60 000 volts. After being assembled they will withstand a potential of 120 000 volts. The insulator pins are 15 in. long, and are made of 2-in. steel pipe set in drop-forged sockets, which in turn are mounted on a 4-in. pipe cross-arm for the lower conductors, and on a 3-in. pipe extension for the upper ones. The pins are set in the insulators with Portland cement.

The conductors are six-strand copper cables, ½ in. in diameter, with hemp centers, and have a strength of 60 000 lb. and an elastic limit of 40 000 lb. per sq. in. The cable is shipped in lengths of 3 000 ft., and the joints are made with an 18-in. twisted copper sleeve. The stress on the cables and supporting structures was calculated by assuming a wind velocity of 100 miles an hour at right angles to the line, and allowing a stress in the materials of one-half the elastic limit. The cables are attached to the insulators by bolted clamps, no tie wires being used. Duplicate telephone circuits of No. 10 copper or steel wire are erected on each of the tower lines, and are placed 10 ft. below the high-potential cables on porcelain insulators.

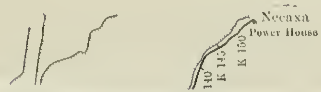
The distance from Necaxa to Mexico is 94 miles, and from Mexico to El Oro 75 miles. The total length, therefore, will be 169 miles, making it the longest power transmission in regular operation. The loss in the transmission circuits between Necaxa and Mexico, with 100% power factor at 60 000 volts, will be 8% at full load, so that the entire power can be delivered over two of the four circuits, or one-half the transmission system, with only 16% loss, should the other half be disabled. The loss in transmission from Mexico to El Oro is only 5 per cent.

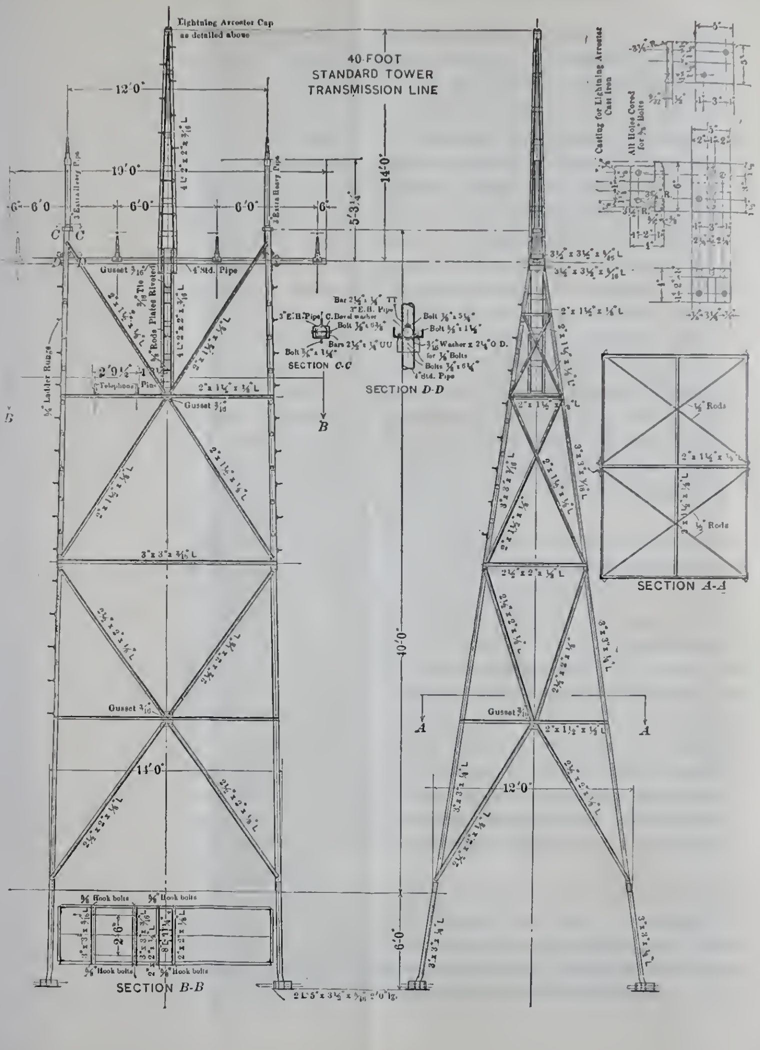
The circuit breakers at Necaxa, Mexico, and El Oro are arranged to cut out automatically any transmission circuit which becomes damaged without interruption to the power service.

Sub-Stations.—The sub-station buildings in the Cities of Mexico and El Oro are shown in Figs. 1 and 2, Plate XVII, and follow in general the design adopted for the power-house. In Mexico there are sixteen 1 800-kw. oil transformers in separate fire-proof compartments covered by a traveling crane. The switches and wiring are in separate cells, and every precaution has been taken to isolate all parts from each other. In addition to the high-tension apparatus, an extensive low-potential feeder system for the city distribution is provided for in the switch-board. The step-down transformers are arranged to furnish current at either 1 500, 3 000, or 6 000 volts, as may be required.

The building is 203 ft. long and 65 ft. wide, and is near one of the company's steam plants, originally built by the Siemens-Halske Company. In addition to this there are three other steam central stations in Mexico, aggregating 20 000 h. p., which are now entirely or partially shut down by the cheaper power from Necaxa.

MAP SHOWING TRANSMISSION LINE
BETWEEN NECAXA, MEXICO CITY AND EL ORO




40-FOOT
STANDARD TOWER
TRANSMISSION LINE
Lightning Arrester Cap as detailed above
SECTION C-C
SECTION D-D
SECTION A-A
SECTION B-B
Castings for Lightning Arrester Cast Iron
All Holes Cored for ½" Bolts

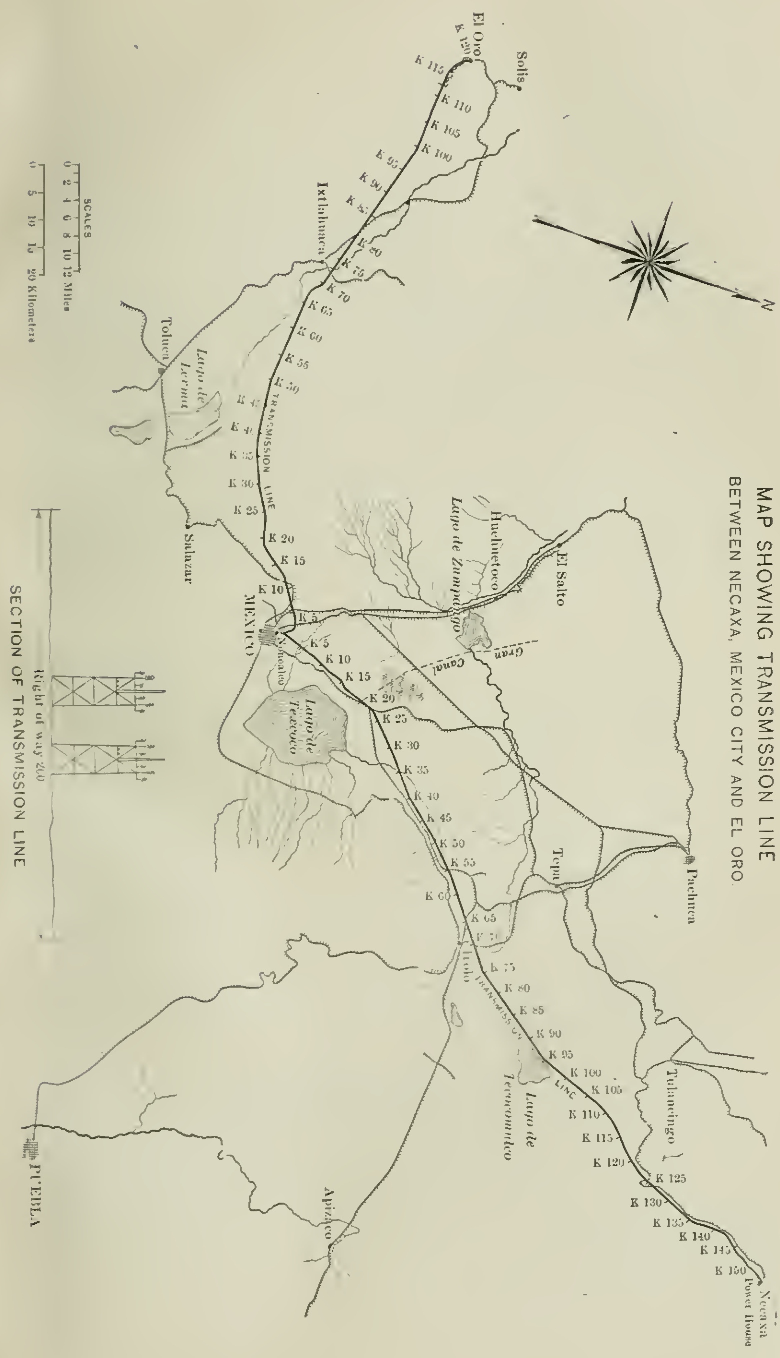

MAP SHOWING TRANSMISSION LINE
BETWEEN NECAXA, MEXICO CITY AND EL ORO.
TRANSMISSION LINE
El Oro
Solis
Ixtlahuaca
Toluca
Lago de Lerma
Salazar
MEXICO
Huehuetoco
Lago de Zumpango
El Salto
Gran Canal
Lago de Texcoco
Tepa
Pachuca
Otumba
Lago de Tecocomulco
Tulancingo
Necaxa Power House
Apizaco
PUEBLA
SCALES
0 2 4 6 8 10 12 Miles
0 5 10 15 20 Kilometers
Right of way 200
SECTION OF TRANSMISSION LINE

FIG. 4.

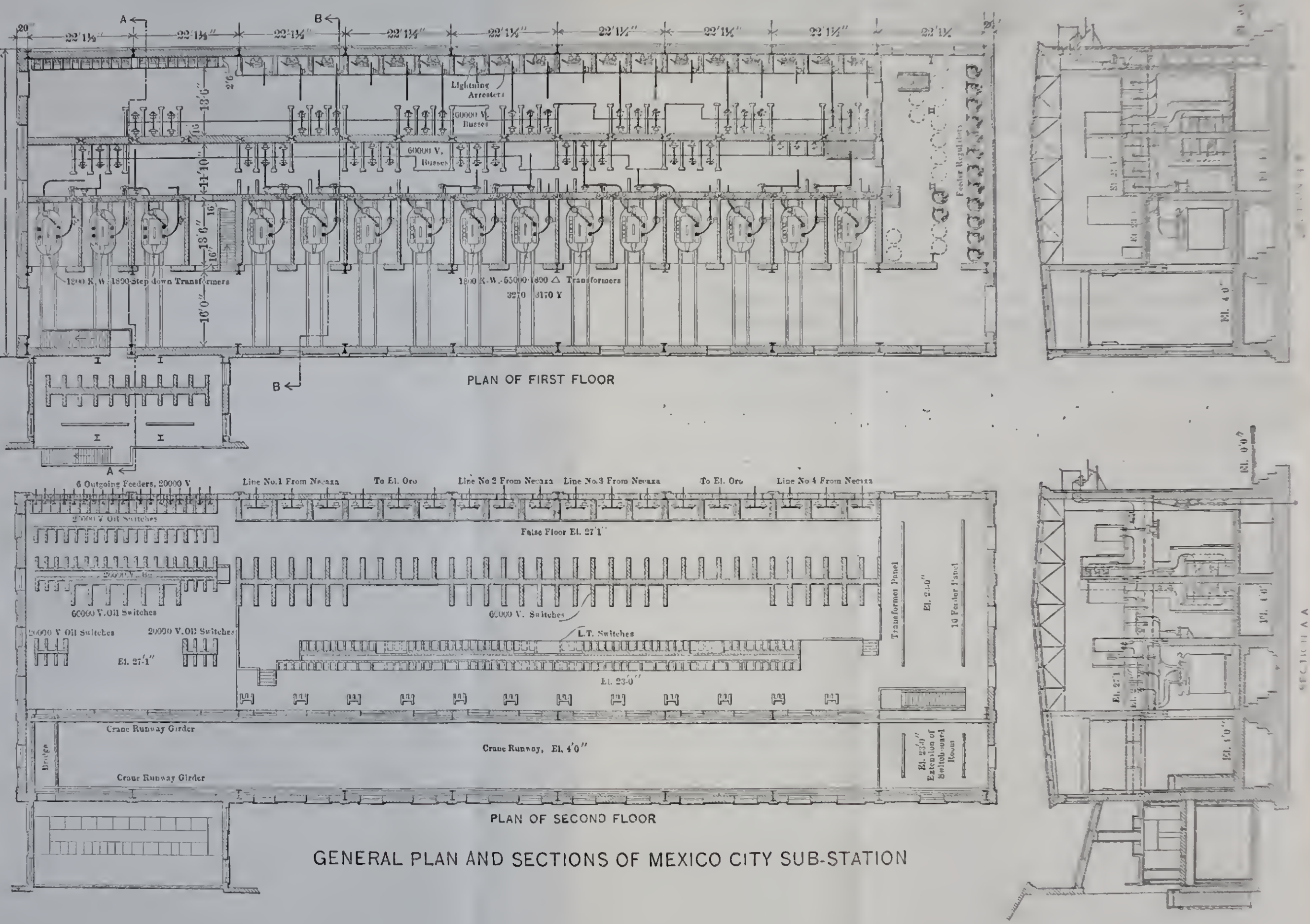
At El Oro, a sub-station, 115 ft. long, and 59 ft. wide, has been erected. This is similar to the one in Mexico, and the nine 1 800-kw. step-down transformers installed there are duplicates of those in the latter city.

The distribution to the gold and silver mines of the El Oro and Tlalpujuhua districts is at 3 000 and 6 000 volts.

An extension of the transmission system to Pachuca is now under way.

The original hydraulic plans of the Necaxa development were made by H. L. Cooper, M. Am. Soc. C. E., who also had charge of the construction in Mexico up to 1905. The work is now being conducted under the direction of Albert Carr, M. Am. Soc. C. E., as Manager of Construction, with Mr. Walter Diem as Resident Engineer and Mr. F. S. Hyde as Hydraulic Engineer. J. D. Schuyler, M. Am. Soc. C. E., is engaged as Consulting Engineer in connection with the construction of the hydraulic-fill dams. Mr. R. F. Hayward has charge of the operation of the plant and the completion of the electric installation.

PLATE X
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVIII, No. 1040.
PEARSON AND BLACKWELL ON
THE NECAXA LIGHT AND POWER PLANT



GENERAL PLAN AND SECTIONS OF MEXICO CITY SUB-STATION

DISCUSSION.

EDWIN H. WARNER, M. AM. SOC. C. E. (by letter).—Approximately ideal conditions exist in the plant designed and described by the authors—a high head, moderate length of penstock, cheap storage, and a market well within the limits of economical transmission. Mr. Warner.

The ideal of a practically uniform annual flow, doing away with seasonal storage, exists in but one plant on this continent,* except in cases of very moderate heads.

Table 1 shows the need of storage, and, since this need exists, it has cheapness to recommend it.

The authors' prediction of excessive floods has been confirmed during the present rainy season; a maximum flow of 5 280 cu. ft. per sec. for ½ hour occurred on August 13th; a flow of 3 720 cu. ft. per sec. for a similar length of time was observed on October 11th; the flow for 24 hours on each of these days approached very nearly the authors' maximum of 3 000 cu. ft. per sec. The conditions making possible these extreme floods were the complete saturation of the water-shed by 4 days' rain, and a fall of 6½ in. during the 24 hours prior to the dates mentioned.

TABLE 1.—AVERAGE PRECIPITATION AT NECAXA, 1901 TO 1906, BOTH INCLUSIVE; AT CÁRMEN FOR 1906: AND TOTAL PRECIPITATION AT NECAXA FOR SIX YEARS.

Month.	Necaxa.	Cármen.	Month.	Necaxa.	Cármen.	Year.	Necaxa.
January..	2.66 in.	0.24 in.	July	21.81 in.	4.06 in.	1901	117 in.
February.	1.87 "	2.66 "	August	15.20 "	13.54 "	1902	83 "
March	2.32 "	0.78 "	September	9.21 "	2.52 "	1903	101 "
April	4.28 "	3.39 "	October ...	11.78 "	12.40 "	1904	132 "
May	3.18 "	3.50 "	November.	4.71 "†	4.40 "	1905	92 "
June......	15.71 "	0.96 "	December.	3.28 "	4.40 "‡	1906	92 "

Total precipitation .... 617 in.
Average precipitation, per annum ...... 103 in

† November and December, 1906, averaged from preceding five years.
‡ Average of preceding ten months.

The diversion of many small streams into the Nexaca Valley at low cost gives, with the large storage areas, a perennial supply equal to all possible demands for many years to come. Already the demand is approaching the rated output of the first installation now nearing completion.

The extension of the plant by a second installation, with a fall of 720 m. (2 341.6 ft.) between the Tezcapa and Los Reyes Dams

* "The Hydraulic Plant of the Puget Sound Power Company," by Edwin H. Warner, Am. M. Soc. C. E., *Transactions*, Vol. LV, p. 228.

Mr. Warner. (Fig. 1) will involve an interesting development of large unite at high speed, with the related problem of so nice an adjustment of sizes of intake, penstock and nozzles as to secure continuous operation. The pipe flanges (Fig. 3) deserve careful consideration. The writer confesses to a prior preference for riveted pipe in large sizes; but, having laid, full-bolted and tightened, 1 870 ft. of this pipe in 6½ days (three shifts of 24 hours each), it has his heartiest recommendation. The German shop work is admirably standardized, as must be the case where a misplaced gasket prevents the bolting rings taking the correct position, and localizing any leakage in the bolting. The pipes have generally a length of 9 m. (29.5 ft), and weigh, in the larger thicknesses, 4 tons each. To handle this pipe on 36 and 41° slopes (Plate V) and lay it in the confined space of a 13-ft. tunnel does not favor rapid work.

Whether a rubber gasket, under heavy pressure from side and edge, will rot or harden with time, is an open question. The writer is inclined to the latter belief. The one criticism to be offered with respect to this pipe is on the variation in the bolts; there are seven sizes of what may be called standard bolts, and three special forms, and as many sizes. These latter are necessary. The number of sizes of standard bolts, from practical considerations, could be materially reduced.

The arrangement of gates in the vicinity of the upper end of the 30-in. pipes (Fig. 1, Plate XII) is very elaborate, and while their number—eleven in all within a space of 20 ft.—is larger than usual, all possible contingencies are provided for. Motor-driven valves on the 30-in. pipes, operated from the power-house, would insure quicker closing than is now to be had, but the instantaneous action of the relief valves gives ample time for hand operation, except in the very remote contingency of a serious break in the pipe.

Mr. Labelle. H. F. Labelle, M. Am. Soc. C. E. (by letter).—The plant described in this paper has a great deal of interest for the hydraulic engineer, inasmuch as it contains many novel features of design. The large hydraulic-fill dam, the intake tower, the use of large lap-welded pipes in the lower parts of the penstocks, the introduction of square nozzles and, above all, the application of the impulse turbine to the operation of generators on a vertical shaft, are all points worthy of special attention.

The managers of hydraulic plants try to have the system of racks connected with their stations under their thumbs, so to speak. Racks are a fruitful source of trouble to superintendents of hydraulic stations, and the fight, at times, assumes important proportions. Various means are used to keep racks clean and prevent fluctuation of head; there are baffling walls, heating coils, steam,

FIG. 1.—TENANGO FALLS, MEXICO.

FIG. 2.—DAM NO. 1 ON THE TENANGO RIVER. LOOKING NORTH.

mechanical rakes operated by electric motors, zigzag racks, herring-bones in combination with sluices and, of course, the ubiquitous man-rake. In the Necaxa plant the intake, fortunately, is placed in a large body of water, and, in proportion to the discharge of water, very little drift will come to it. Nevertheless, the designer has placed abundant protection, at the intake, against the entrance of floating matter into the penstocks. It is hard to see, however, how the inside screens can be cleaned in case they require it. Are they made in sections, and has special provision been made for lifting them? The writer would like to be informed regarding the operation of these screens and racks under various conditions. Mr. Labelle.

It is believed that the branching of the penstocks, near the power-house, each branch supplying one or two generating sets, has originated in American practice; on this Continent it is not a new thing. In Europe, the turbines of two of the best-equipped high-pressure plants, one at Tivoli, near Rome, the other at Vallorbes, in the Swiss Jura (which, by the way, is said to have the most perfect and up-to-date electrical installation in Europe), are fed by a single line of pipe, and only recently the Kubel plant, near St. Gall, Switzerland, has duplicated its original penstock. The plant recently completed at Obermatt (supplying Lucerne) has also a double penstock in operation, with two additional pipes for future extensions.

The use of lap-welded, flanged pipe of large diameter for penstocks is a comparatively recent innovation, both in Europe and America. This kind of pipe was used at Vallorbes, but with more commonplace flanges than those described in this paper. In case of accident to this kind of pipe, it is obvious that it is far easier to make repairs than with a riveted pipe. At the head of each of the 30-in. lines and just below the stand-pipes, the writer would have liked to see the installation of a butterfly, or other kind of valve, operated by a motor from the station switchboard, and designed to shut off the water in case of accident to, or bursting of, its appurtenant 30-in. line, thus saving the other lines from possible destruction, and preventing serious damage to the power-house.

Heretofore, the high-pressure or impulse turbine (in America commonly called the "Pelton" wheel) has been designed with a horizontal shaft, with very few exceptions, and this for turbines of the smaller sizes. The designs have carried one or more nozzles, the multiple nozzle being seldom used in Europe. In a few of the plants designed by Escher-Wyss and Company, the double parallel nozzle, operating two sets of buckets, is used, but in the majority of even the most recent European high-pressure plants, the old design is adhered to. The generating units of the Necaxa plant are radical departures in this respect, inasmuch as they show the com-

Mr. Labelle. bination of the impulse turbine and the vertical dynamo in new roles, the former on a vertical shaft, and the latter, which heretofore has been confined to low heads, operating a plant under 1 300-ft. head.

It is pertinent to state here that the two patterns of turbines used almost exclusively in Europe to-day—the Francis turbine and the Pelton machine—are of American origin. A few years ago European designers of turbines, and among them the firm of Escher-Wyss and Company, which represents the best practice in Europe, attempted to force the Francis reaction turbine out of its legitimate field, by applying it to high-pressure plants; the Pelton machine, however, soon showed its superiority in that field over all reaction outfits, and it is seen now in all the latest European developments running under a high head. Exception is to be made of the Tivoli plant, which has Girard turbines; improvements in that line, however, have been so rapid that the hydraulic plant at Tivoli is now almost old-fashioned. In 1903, of 108 turbines built during the year by Escher-Wyss and Company, only 3 were of the Girard type.

In the 18 or 20% which separates present efficiency in turbines from perfection, there is some 5 to 8% of frictional resistance between the moving and non-moving parts; this, in a large machine, constitutes a considerable loss of power. This resistance in vertical direct-connected sets is reduced considerably by the introduction of oil pressure to support the moving parts, which are under about the same conditions as the rotating screen of a lighthouse, which floats and revolves on a body of mercury. In that way, friction is reduced to such a point that when the oil pumps are in operation, the rotor of a 1 000-kw. machine can be moved by one man. This has been done repeatedly in the Chèvres station, which supplies Geneva, Switzerland. For this and other reasons, given in the paper, this new turbine design should have a very high efficiency, and it is to be hoped that some record of the performance of the turbines described will be given by the authors in their review of the discussion.

The 4 000 voltage of the Necaxa generators is somewhat above the time-honored and quasi-classical 2 000 to 2 500 voltage commonly used in America, but it is still far below the voltages which have been used successfully in Europe for some years. Tivoli has been one of the pioneers in that line, the generators at that plant running at 10 000 volts, and this is also the line voltage. Other plants generate at 11 000 and as high as 20 000 volts. In America these high voltages have been accepted, at last; the new plant near Salisbury, S. C., on the Yadkin, will generate at 11 000 volts. The Anglo-Roman Electric Power Company, which operates the Tivoli plant, is so well pleased with this high voltage of generation that it

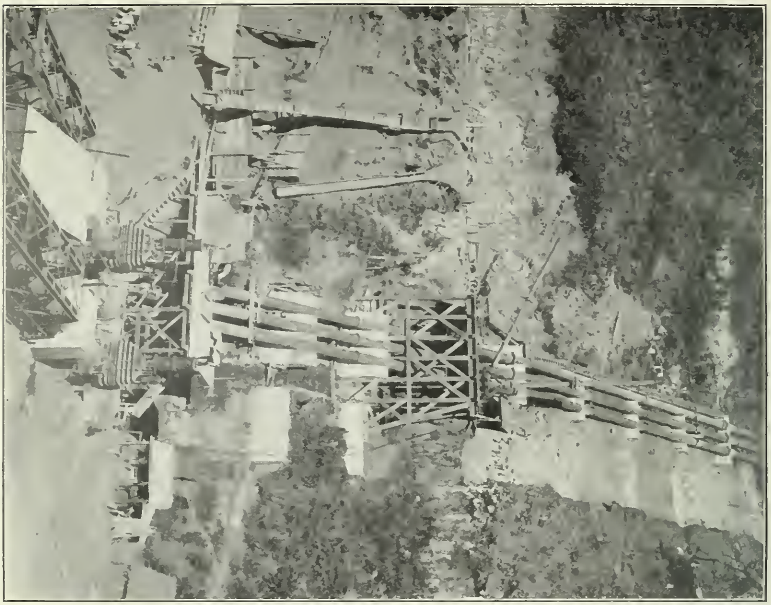


Fig. 1.—Receiver and Valves at Tunnel No. 3.

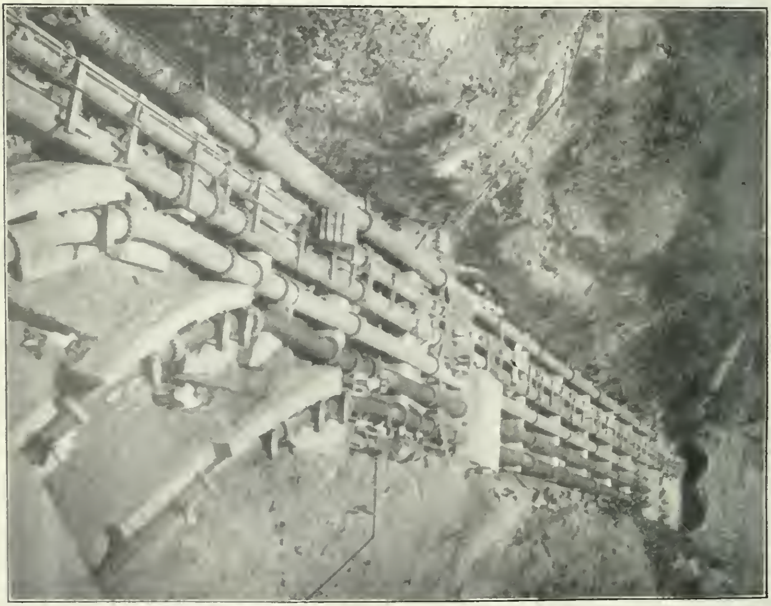


Fig. 2.—Pipe Line from Power-House.

is preparing to develop 5 000 h.p. above Tivoli, and is installing generators which will run at 30 000 volts; the current will be transmitted to Rome at that pressure, and then stepped down to 10 000 volts, so as to connect it with the old system. Mr. Labelle.

Some years ago, hydro-electric developments were carried out without much concern about storage possibilities in the water-shed. If a stream had a fairly even discharge throughout the year, it was considered a good proposition; if not, it was regarded as of doubtful value, and no attempt was made to increase this value by storage. In sections where the rainfall is distributed evenly over the whole year, and where streams have more constancy of flow, storage, although always helpful, is not as important as in places where the rainfall is confined to one-half the year, as in certain parts of the tropics and in semi-arid countries, where, at the end of the dry season, streams have dwindled to nothing. Even if the yearly rainfall is abundant, it occurs mostly during one part of the year, and, therefore, the streams fluctuate too much in quantity to be of any use for power purposes. Storage, when practicable, and where the cost of reservoirs does not bring the enterprise outside of proper limits financially, can improve a power proposition wonderfully, and change an otherwise valueless stream into a valuable property.

Although this statement may appear self-evident to many, still there are people who wish to be considered wise and well-informed on these matters, who refuse to see any advantage in storage. A few years ago, it is true, storage reservoirs in connection with hydro-electric plants were not a fashionable combination. Works were designed on the basis of minimum or normal flow, and at some plants as much as 75% of the yearly run-off went down the river without being utilized. Lately, however, at least one section of the country is making for storage. Having outgrown the capacity of their plants, or having tired of paying exorbitant prices for coal used in their steam auxiliaries, the electric power companies of California have begun to investigate storage; as their country is topographically well adapted to reservoir building, it is to be expected that this phase of power development will soon come into prominence on the Pacific Coast. It is to be feared, however, that the reservoir sites will cost a great deal more now than if they had been purchased at the start, and if storage considerations had been included in the original designs.

The map (Fig. 1) shows the importance which the designer has given the question of storage. The proper conditions, being found, have been quickly taken advantage of. Without storage the combined minimum flows of the two rivers would have produced about 12 000 h. p.; with reservoirs (and in dry years there is more than

Mr. Labelle. 15 000 000 000 cu. ft. of available rainfall to fill them), the average flow can be used, producing about 50 000 h.p., at probably no greater cost per kilowatt of output delivered than with the smaller development. Further, the plant has the advantage of obtaining its storage from several reservoirs, which can be built successively as the output requires it; besides, it has at its command auxiliary streams which may be utilized when necessary.

What the writer has just stated about auxiliaries to hydro-electric plants must not be construed as a wholesale condemnation of them. This question of auxiliaries is a complicated affair which has not been completely sifted out in America, and, until lately, it has been completely neglected in Europe. Whether auxiliaries should be adopted at the start, their adoption postponed to some future time, or neglected altogether, are questions of much moment for designers and promoters of works dealing with the transmission of hydraulic power.

Dr. Perrine* truthfully describes the present state of this matter in America, when he says:

"After all has been done that is possible in obtaining the best possible storage and in selecting the most satisfactory customers, it is often the case that there remains a residuum in the problem which may be economically handled only by steam or gas auxiliaries. But a short time ago the presence in any hydro-electric system of steam or gas auxiliaries was considered a confession of weakness in the hydraulic system. Fortunately, this false idea is fast losing ground, and it is recognized that the best of engineering is shown by their use, and in consequence fine hydro-electric opportunities are being utilized which were previously neglected."

Among Europeans, the clear-headed Swiss were the first to recognize the value of auxiliaries. After the plant at Chèvres, some four miles below Geneva, on the Rhône, had been for some time in operation, it became evident that the works would never be able to give a constant output single-handed, too many things causing the head on the turbines to fluctuate. The station had been built in the river near and parallel to the right shore; the barrage was composed of piers and balanced or Stoney gates of 10-m. spans, extending from the upper end of the power-house to the left shore. The forebay, in consequence, received all the detritus brought down by the river, not only from Geneva, but from the numerous towns located on Lake Leman and from the mountain streams emptying into it. In winter large bodies of frazil had to be handled, these occasionally completely stopping the works in spite of heating coils and the generous use of steam around the racks. Moreover, at certain seasons, the discharge of the river was considerably reduced. Brief-

*"The Value and Design of Water Power Plants as Influenced by Load Factor." *Journal* of the Franklin Institute, Oct., 1906.

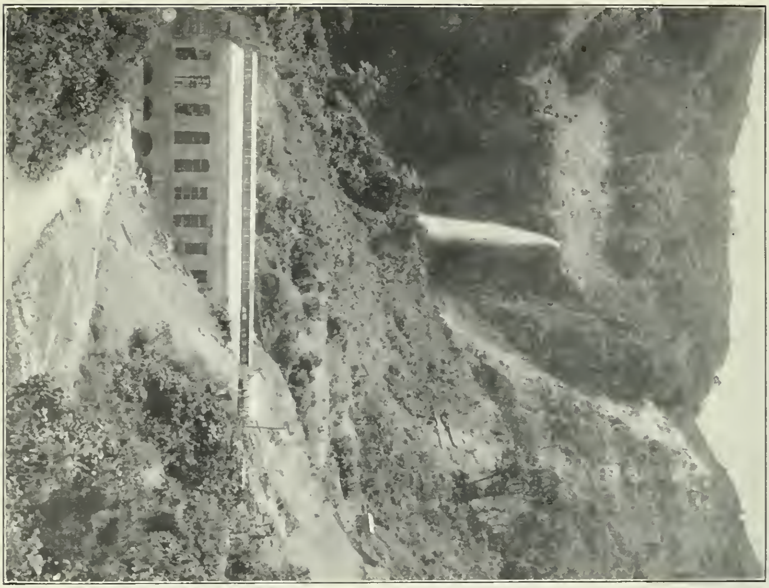


FIG. 1.—VIEW OF POWER-HOUSE FROM FRENCH TRAIL.

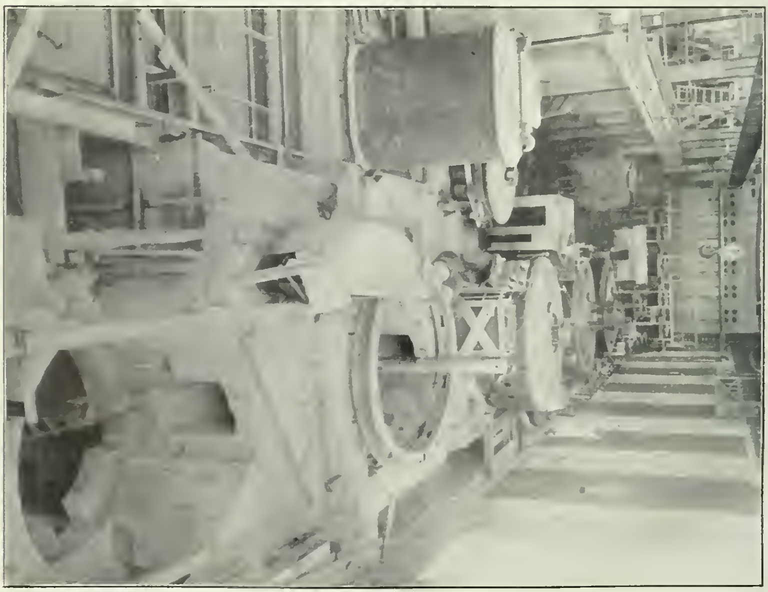


FIG. 2.—INTERIOR OF POWER-HOUSE No. 1.

ly, the power output was very irregular, and as if these troubles were not sufficient, the river has begun to pile up sand and gravel in the upper end of the fore-bay. Mr. Labelle.

The engineers of the plant, however, believe that they have found a remedy. They have constructed a combination trestle and racks extending from the upper end of the power-house, about 500 ft. up stream, to a point on the right bank, thereby enclosing the forebay. The structure is placed at such a small angle with the thread of the current that it is expected to divert the drift from the forebay and lead it over the wasteway provided in the barrage.

The Geneva Council, however, did not stop at the above device, but, having once recognized the utility of an auxiliary plant, it was not slow in erecting in the city what the writer considers a model electric power station throughout. The building is spacious and modern, and can accommodate machinery to generate more than 5 000 h.p. The generators are directly driven by Curtis turbines, and the steam is furnished by a battery of Babcock and Wilcox boilers, with all coal and ash conveyors, mechanical stokers, economizers, etc. It now seems that the problem of constancy of power in Geneva has been solved satisfactorily, and that the full normal output of the Chèvres station, with the help of its steam auxiliary, will be available at all times.

In the rapid advance of hydraulic-power transmission, both America and Europe have contributed largely, and each has taken advantage of the other's improvements in practice. It is true that Europe has the lead in turbine design and high voltage of generation, but America leads in high voltage of transmission and the development of auxiliaries. Besides, America is awakening to the importance of rational turbine designing, and at least one firm (the I. P. Morris Company, of Philadelphia) has cut loose from boiler-plate outfits, and has entered the field heretofore occupied exclusively by Swiss and German designers. It is to be hoped that other firms of turbine manufacturers in America will follow this example, and avail themselves of this surest way of increasing the efficiency of what is now the least efficient machine in all hydro-electric transformations.

F. G. Baum, Assoc. M. Am. Soc. C. E. (by letter).—The statement made by the authors, that a lap-welded pipe is stronger and more reliable than one built up of plates riveted together, is open to question. Out of an installation of four pipes of this kind, two failed at the weld, in one case after the pipe had been in service about seven years. The rupture showed the pipe to be blistered nearly the entire length at the weld, and, for a distance of about 9 in., there was a flaw running two-thirds through the metal. At the time of the break very cold water was being drawn through the Mr. Baum.

Mr. Baum. pipe, and it is supposed that the contraction due to the cold (the pipe was flattened at the joint) caused an extra strain at the weld, and it yielded. Numerous joints of tested pipe of this kind have had to be discarded. A test of a few minutes is not sufficient.

The fact that pipes break occasionally makes it advisable to have as few cross-connections as possible, and these connections, where possible, should be normally closed. It is also advisable, where possible, to take a nozzle for two or more different units from the same pipe.

The vertical units cannot be justified on account of a saving of cost of power-house or ground space, as horizontal units could have been installed in the same space. The right-angled connections of the nozzles to the main pipes demanded by vertical units are open to objection on account of the loss of power, and the pipe connections are more costly and complicated than with horizontal units. There is also objection to the vertical unit on account of the fact that the water discharged from the buckets does not clear the wheel as well as in a horizontal unit; and certainly a horizontal bearing is preferable to a vertical bearing, other conditions being the same.

To obtain the same result with a horizontal unit would have required two disks and two sets of buckets as against one disk and one set of buckets for the vertical unit, assuming the buckets and nozzles to be of the same size. The extra cost, however, would be offset by reduced losses in the nozzle connections and probably by increased wheel efficiency.

In laying out high-tension electrical connections, it should be borne in mind that a bare wire will give less trouble than one to which there are all kinds of connections, and flexibility should be sacrificed for simplicity. Series transformers, etc., should be eliminated, where possible, from the high-tension leads, and there should be no more switches than are absolutely necessary. As far as possible the switches, transformers, etc., should be within view by the operator, so that he may at once locate any trouble in the station, and not repeatedly throw the power on a piece of disabled apparatus which may be in a cell, out of view, not 20 ft. away. The plans show that the transformers, switches, etc., occupy more than one-half the building, which certainly is a very large proportion.

The use of steel towers on this line adds another experience to what will ultimately be the regular practice, for it takes no prophet to predict that some time in the future all high-tension, long-distance power lines will be on towers.

Mr. Galloway. J. D. Galloway, M. Am. Soc. C. E. (by letter).—Some of the authors' statements seem to be open to criticism, although the plant as a whole represents an example of high-class construction.

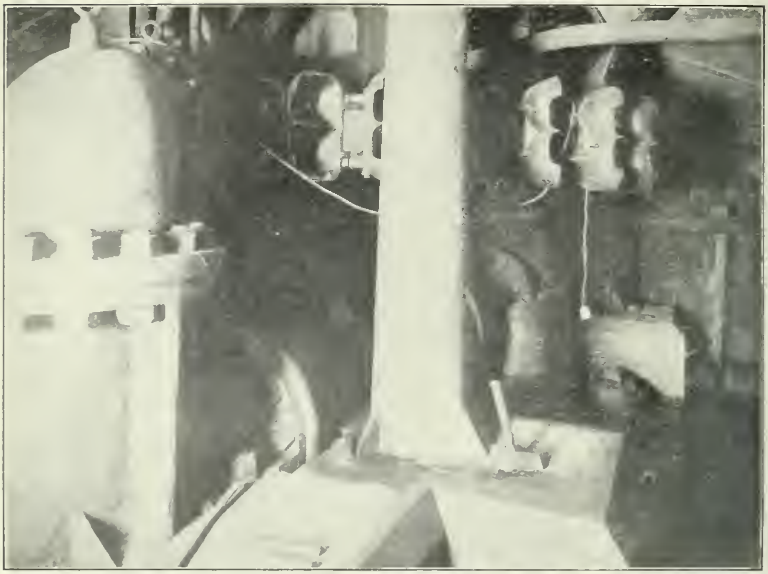


FIG. 1.—WATER-WHEEL.

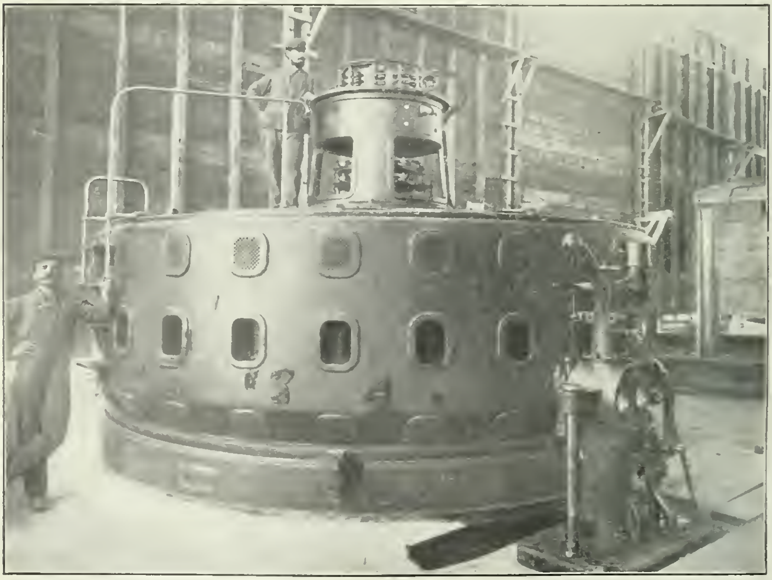


FIG. 2.—GENERATOR.

Pipe Lines.—The pipe lines consist of about 2 400 ft. of 8-ft. riveted pipe, 2 300 ft. of 6-ft. riveted pipe and 2 460 ft. of 30-in., outer diameter, lap-welded pipe. The details of the construction of the large riveted pipe are lacking. It is generally a serious question as to how to preserve the shape of such pipe when built of relatively thin material. Bands of angle iron are sometimes used. The method of support is not stated, except that the pipe is on concrete piers. It would be interesting to know how close the piers are placed. The weight of water in the pipe is more than 3 000 lb. per lin. ft., which would seem to render the method of support quite a problem. Mr. Galloway.

There is no mention of air-valves in this section of the pipe; the only means for air to enter is at the stand-pipes, below the receiver, they being connected to the 30-in. pipes. The distance from the reservoir to the receiver is about 4 500 ft., and, if there are no air-valves in that length, a break in the pipes would inevitably cause the collapse of the portion of the pipe above. Stiffening rings might prevent this, but it is doubtful. The collapse of pipe has occurred in a number of plants designed in a similar way, and the resulting damage was enormous.

The writer doubts very much the statement that a lap-welded 30-in. pipe is "stronger and more reliable than one built up of plates riveted together," and the following is offered as evidence:

At a plant which will be designated as "A" there are two 30-in. lap-welded pipes. Shortly after installation, one of these opened at the weld, causing much damage. After being in use seven years, another section opened at the weld. Inspection showed a very imperfect weld and also a shearing of the outer portion of the steel from the inner along the neutral axis. The steel seemed to be in layers. At plant "B" there are two lines of pipe, partly of 30-in., outer diameter, lap-welded. A number of sections (five, if memory serves correctly), were rejected in the field, owing to defective welds. Examination by drilling showed that in cases less than one-half the metal was in contact.

Another instance occurred at the Bishop plant of the Nevada Power, Mining and Milling Company, of the construction of which the writer had charge. A 24-in., outer diameter, riveted pipe, from the same factory that furnished the Necaxa pipes, was used, the maximum head being 1 068 ft. This pipe was subjected to a shop test similar to that of the Necaxa pipe, and was inspected by a prominent firm of inspectors. On arrival at the plant, one section was found to have opened along the weld a length of more than 5 ft. After being in place, with the water turned on for two weeks, another section opened at the weld, causing a shut-down and a troublesome patch.

Mr. Galloway. At the Necaxa plant there are more than $2\frac{1}{2}$ miles of weld, and engineers can appreciate the chance of such defects as mentioned. Personally, the writer believes such pipe to be entirely unsatisfactory for power plants. The resulting damage from a break would be enormous, both in cost of repairs and in prestige.

The use of riveted pipe is general throughout the West, and gives entire satisfaction. The writer calculates for a stress of 12 000 lb. per sq. in., and uses butt-strap joints, triple riveted. Joints of this type can be made with an efficiency of 80% and more. The riveting forms a positive connection, the strength of which has been established by numbers of tests. All parts can be inspected, and the chance of defects is reduced to a minimum. At Electra a riveted pipe supplying two 5 000-kw. generators, under 1 460 ft. head, designed as above, has been in use nearly two years, and no defect has shown.

A comparison of cost, made by C. D. Marx, M. Am. Soc. C. E., and the writer, was afforded by the design of pipes at Stanislaus, Cal. Bids were taken on the two pipes, 36 in. in diameter at the lower end, under 1 495 ft. head. Riveted and lap-welded joints were each taken as at 80% efficiency, the stress in the steel being 12 000 lb. per sq. in. Allowance was made in friction factors, due to less friction in the lap-welded pipe, the riveted pipe being of larger diameter. Including installation, the riveted pipe was 10% higher in cost than the lap-welded, while the time was about 40% longer for the lap-welded pipe. The numerous failures of lap-welded pipe, mentioned above, together with the time required, led to the adoption of the riveted pipe at a higher cost.

The design at Necaxa, which has one 30-in. pipe leading to each generator, seems to be not as economical nor as effective as one pipe serving two generators. A single 42-in. pipe has the same area as two 30-in. pipes, and, theoretically, the same quantity of metal in the shell. There is, however, a greater carrying capacity, due to decreased friction in the larger pipe. In laying, there is only one-half the number of pieces to be handled, and only one-half the number of longitudinal and transverse joints. In this, the cost is largely concerned with the number of parts handled, and not so much with their size.

At Necaxa the cost of the tunnel enters largely into the cost of these pipes. Judging from the photograph, Fig. 2, Plate XII, there seems to be a space of about 2 ft. between the pipes, which is necessary to make the joints. Each is about 14 ft. wide. As three 42-in. pipes would occupy only 2 ft. more in width, one 16-ft. tunnel would have taken the place of two 14-ft. tunnels. As the tunnels are some 1 900 ft. long, and are excavated on an incline of 41°, and in a remote place, it would seem that a saving of at least \$30 000 could

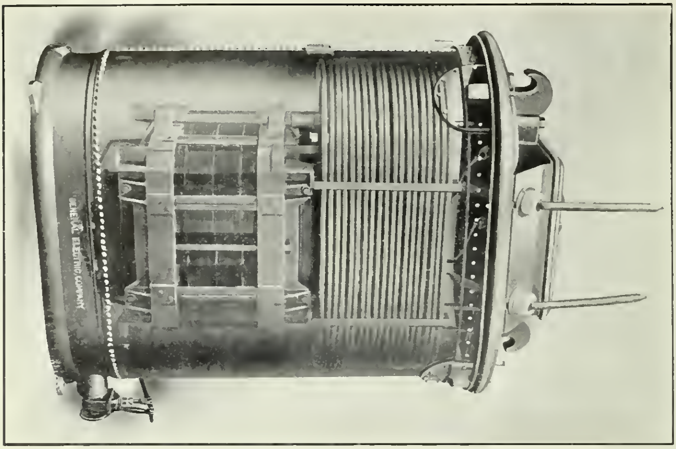


FIG. 1.—TRANSFORMER.

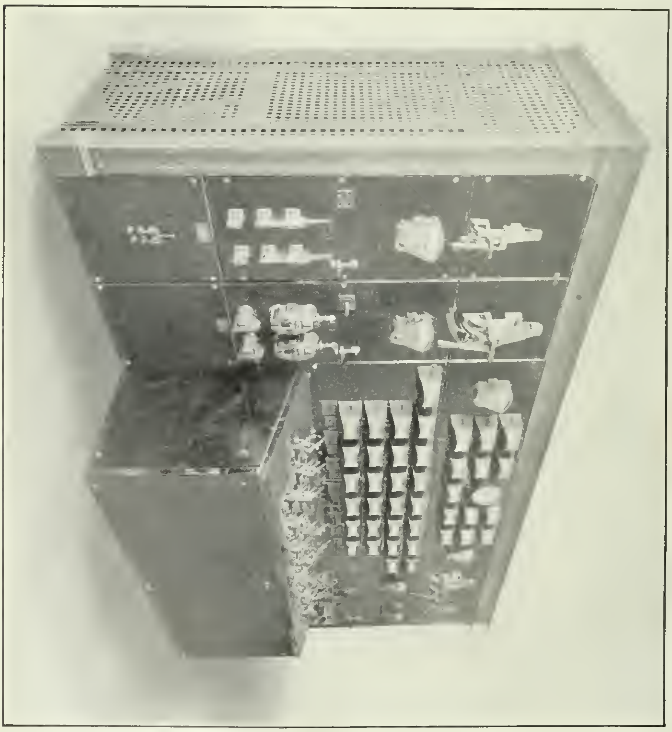


FIG. 2.—SWITCH-BOARD.

have been effected by using larger pipes in this item alone. In the manufacture, there is but one joint instead of two, and the labor is reduced correspondingly. Heavier and larger pipes always come at a lower price per pound. Again, with two generators coupled to one pipe, the chances of a stoppage of flow of water and corresponding water hammer are much reduced.. The writer regards the placing of two generators upon one pipe as the most effective insurance against damage to the pipe. Mr. Galloway.

The Hydraulic Machinery.—In connection with the water-wheel, as designed with a vertical shaft, the authors state:

"This construction has many marked advantages for large units over impulse water-wheels heretofore built with horizontal shafts and single deflecting nozzles. The double nozzle reduces the size of both the jet and the bucket, and permits the use of a water-wheel of smaller diameter and higher speed of rotation without sacrifice of efficiency."

To one familiar with what has been done in California, this seems to be an absurd claim, in many respects, as no statement of the wheel efficiencies is given. In the design as made, the water entering the nozzles must make two right-angled bends, or 180°, before reaching the wheel. To compel water moving at from 15 to 20 ft. per sec., under some 1 400 ft. head, to turn an abrupt right angle would not seem as effective as to cause it to turn two angles of 30°, as in the Pelton Water-Wheel Company's designs. No detail is given of the nozzles or the regulating device, so that comment cannot be made. To the writer, it does not seem possible to devise a simpler method of regulation than the deflecting nozzle, with the needle adjustment of the size of the jet. With a horizontal shaft, it is possible, and probably advisable, to place a wheel at each end, on generators of more than 3 000 kw. capacity. Little space has been saved, as the generators are some 24 ft. apart on centers. Single-wheel generators of 5 000 kw. capacity, on horizontal shafts, can be placed in 30 ft. Generally, the floor length depends upon the space required for transformers and switches and not on that required for the generators. The position of the nozzles, forming a couple, is theoretically correct, but no difficulty has been found in running generators of equal size under the same head, with a single nozzle. Little, if anything, is saved in head by the arrangement of placing the wheel near high water, as the part of the total head is a fraction of 1 per cent. The authors comment upon the higher speed possible by using the vertical shaft. The speed is 300 rev. per min. The writer knows of three 5 000-kw. generators, with single wheels on horizontal shafts, running constantly at 400 rev. per min., with a corresponding reduction in cost in the generators and wheel parts, both at the shop and in transportation. One

Mr. Galloway. of these, at de Sabla, was started nearly three years ago, and descriptions of it appeared in the technical press. Several more of the same size are now being installed. In fact, the speed of 400 rev. per min. for large generators seems to have become a standard. In the light of these facts, the claim for higher speed with the vertical shaft does not seem to be substantiated, as the examples cited were in operation before the Necaxa plant was built.

A point of interest, not noted by the authors, would be the action of the wheels under the variable head, due to the lowering of the storage reservoir. With the reservoir drawn down to a level 10 ft. over the lowest intake valve, there would be a decrease in head of about 8 per cent. The efficiency of pipe and water-wheels would suffer in this case, and information as to what the effect is would be interesting.

The writer is not familiar with the design of electrical apparatus, and makes no comment upon those parts.

The towers of the line are of excellent design throughout, and undoubtedly represent what will ultimately become a standard form of construction. Cheap poles and right-of-way questions have prohibited tower construction in the West.

Some points of the design claimed to be of much merit by the authors, and differing radically from construction with which the writer is familiar, have led to the foregoing comments, which, however, must not be taken as criticism. It is by the trial of new devices that progress is made.

Mr. Blackwell. F. O. Blackwell, M. Am. Soc. C. E. (by letter).—Referring to the discussion by Mr. Edwin H. Warner, in which he writes of the desirability of motor-driven valves at the head of the 30-in. pipes, the writer would say that hydraulic apparatus for operating these is now being installed, and they will be electrically controlled from the power-house.

In reference to Mr. H. F. Labelle's inquiry regarding the cleaning of the inside screens, it is the intention to remove the screens above water when the water level of the reservoir is lowered. Should the screens become clogged on any level but the lowest, it is possible to open the gate below. A crane is to be provided at the top of the head-gate for handling the valves and rocks. Attention is also called to the duplication of the intakes, either one of which can supply the entire plant with water.

In connection with the discussion by Mr. F. G. Baum, the writer is informed that the de Sabla 5 000-kw. units are 26 ft. long and 23 ft. wide, while the Necaxa units are but 16 ft. in diameter. Reduced to the same speed, the de Sabla units take up about three times as much floor space as those at Necaxa. The actual size of the generator-rooms cannot be compared, as the Necaxa plant

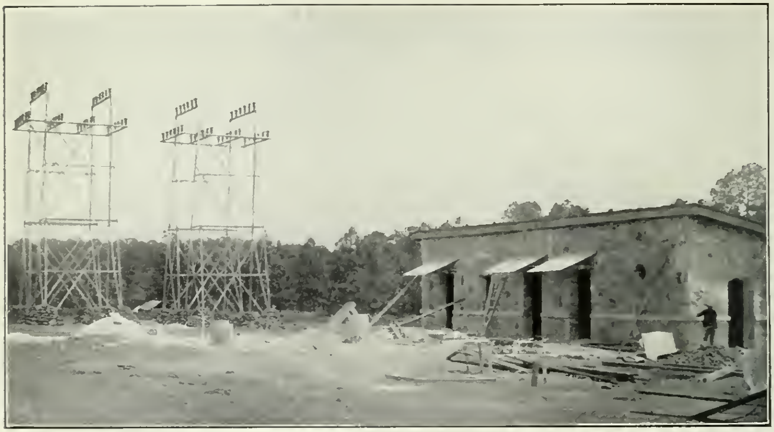


FIG. 1.—PATROL HOUSE AT CARMEN.

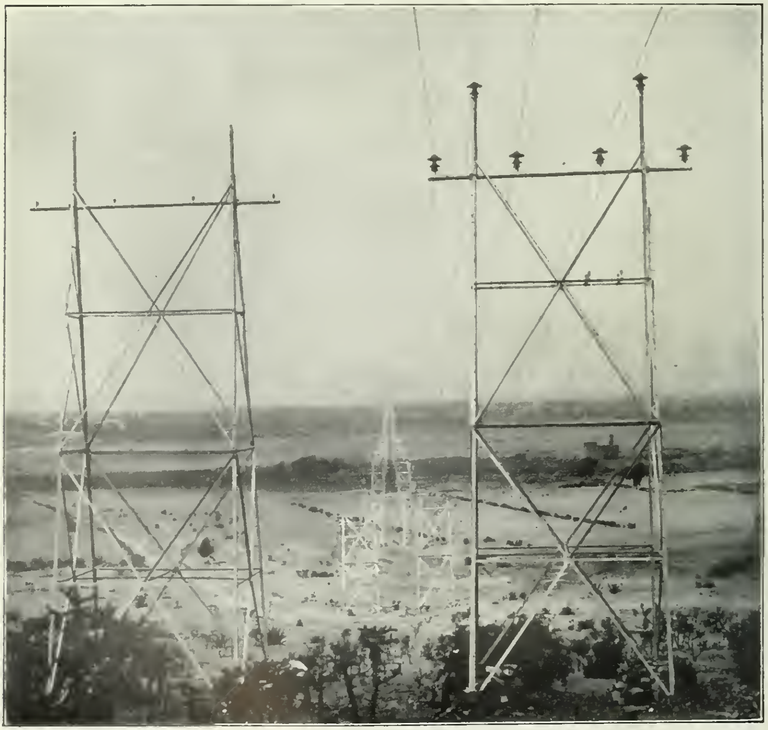


FIG. 2.—NECAXA LINES TOWARD MEXICO.

has been laid out with ample gangways and open spaces to get at all the machinery, whereas, in the California plants, the different pieces of apparatus almost touch each other. Mr. Blackwell.

The space allowed for switches and wiring in the Necaxa plant permits protective devices at every point, so that any portion of the station may be cut out automatically, should an accident occur, without interrupting the service. In the California plants, the station is a unit, and an injury to any part shuts down the entire system completely.

Regarding the water-wheel, all tests thus far made indicate a higher efficiency for the vertical than for the horizontal shaft type, especially at fractional loads. Practically, no water is wasted, and the jet always hits the center of the bucket of the wheel. In operation, the water does not fall back on the wheels, and there is an almost entire absence of spray. When the plant is completed, careful tests of efficiency are to be made.

The oil thrust-bearings have been particularly satisfactory, and the station operators say they require much less attention than large, high-speed, horizontal bearings. There is almost no friction, and the whole unit can be turned by hand with ease. The steady bearings of the unit are but one-half the size of those at de Sabla, notwithstanding the fact that the latter runs at one-third higher speed and uses a special, hollow, nickel-steel, oil-tempered shaft.

The speed of 300 rev. per min. was selected in order to obtain the higher efficiency possible with a large bucket diameter relative to the nozzle. The same results could only be obtained with a single nozzle at a speed of 200 rev. per min., which would greatly increase the size and cost of the water-wheel.

Referring to the discussion by Mr. J. D. Galloway: The 6-ft. riveted pipe is supported on concrete piers, which are carried up level with the center line of the pipe. The piers are spaced 7 m. apart, and are shown in the photograph, Plate XVIII.

There is no necessity for air valves in the pipe lines, and none was supplied, as the pipe is stiff enough at all points to stand a vacuum inside. This is particularly true of the 30-in. pipes from the stand-pipe to the power-house, which are also reinforced by the flanges where the sections come together.

Comparing lap-welded with riveted pipe, the writer thinks that the evidence is in favor of the former being more reliable. Neither kind of pipe is infallible, and many instances of the failure of riveted pipe might be cited. The Necaxa lap-welded pipe has been particularly successful, and repeated tests and a year's use have developed no weaknesses, although several sections were subjected to five times the working pressure.

The success of the lap-welded pipe depends largely on the material used. In order to get satisfactory welds, it is necessary to

Mr. Blackwell. use a very homogeneous, soft steel, and this in itself is a safeguard against failure in case of water-hammer. They should also be thoroughly annealed; tests made of the Necaxa pipe by pounding the sides together developed no fracture at the weld.

Riveted pipe under such a high head would have to be made with double butt straps and triple rivets. More than 35% of the total surface would be covered with butt straps and rivet heads. The writer does not know of any reliable data on the friction coefficients of pipe under these conditions, but he believes it will be found to be much higher than is generally supposed.

The lap-welded pipe permits of a large reduction in both the diameter of the pipe and in its thickness, other things being equal. This effects a saving in the quantity of material used, so that, notwithstanding the better grades of steel required, the cost of such pipe should be less than that of riveted pipe. The largest pipe manufacturers in the United States are putting in apparatus to make welded pipe in large sizes, and the writer believes that it is only a question of time before riveted will be displaced by welded pipe in large sizes, as it has been already in small sizes.

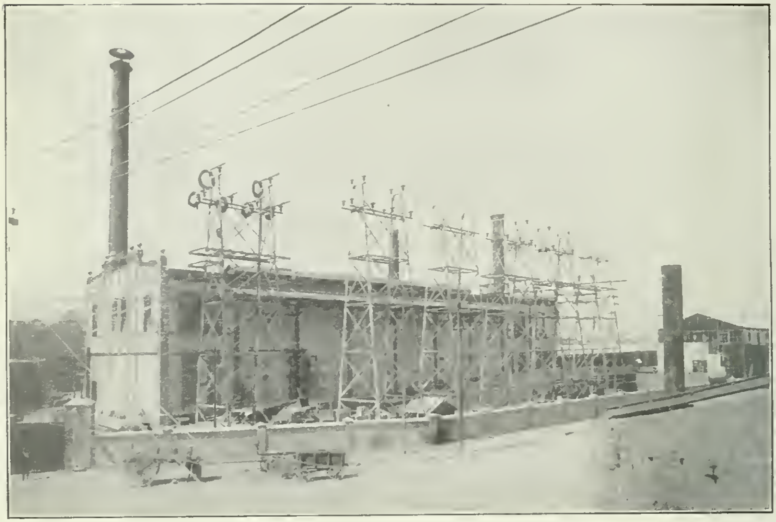


FIG. 1.—SUB-STATION AT MEXICO.

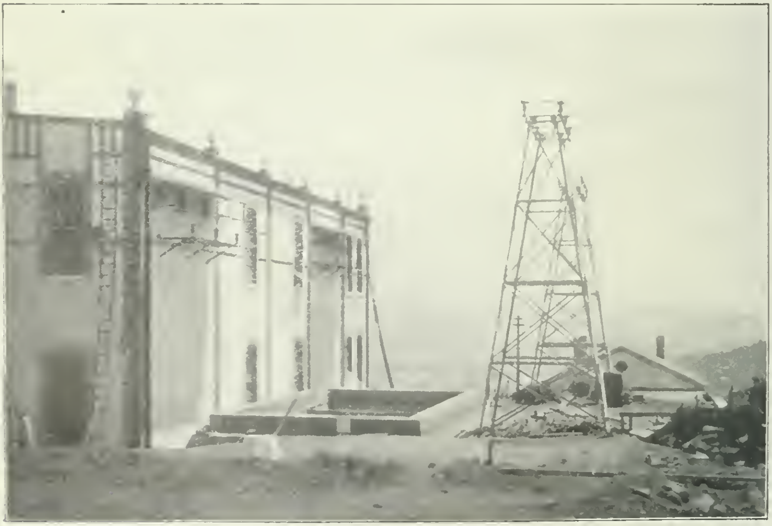


FIG. 2.—SUB-STATION AT EL ORO.

Pipe Lines from Tunnel No. 5.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1041.

ADDITIONAL INFORMATION ON THE DURABILITY OF WOODEN STAVE PIPE.*

By Arthur L. Adams, M. Am. Soc. C. E.

With Discussion by Messrs. Shirley Baker, Clemens Herschel, D. C. Henny, G. P. Hawley, Andrew Swickard, A. M. Hunt, L. J. Le Conte, T. Chalkley Hatton, J. C. Ralston, C. D. Marx, D. Farrand Henry, Edwin Duryea, Jr., and Arthur L. Adams.

Ten years ago the writer made a special study of wooden stave pipe with reference to its suitability for conveying water under pressure, and the principles governing its economic design. The result of those studies he has previously laid before this Society.† A description of the use of 7½ miles of this class of pipe at Astoria, Ore., has also appeared in the *Transactions*.

That this pipe at Astoria has proven deficient in durability, in defiance of all preconceived ideas as to what may be expected of wood when subject to constant water pressure and buried in the earth, is certainly somewhat startling, but nevertheless true, and the writer deems it a duty to the profession that the facts be made known. In order that what is stated herein may be readily followed, a profile (Fig. 1) of the pipe line in question is submitted, and for full information as to the extreme care with which this line was designed and built, reference is made to the writer's paper, entitled "The Astoria City Water-Works."‡

* Presented at the meeting of October 17th, 1906.

† "Stave Pipe—Its Economic Design and the Economy of Its Use." By Arthur L. Adams. *Transactions*, Am. Soc. C. E., Vol. XLI, p. 27.

‡ *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 1.

The staves of the pipe in question had deteriorated so much after ten years' use, that, during 1905, very extensive renewals and repairs became imperative, involving an expenditure of more than one-third of the original cost of the pipe. These repairs were made under the immediate direction of Mr. Lars Bergsvik who served as an Assistant Engineer on the original construction of the Astoria Water-Works. The information herein contained has been derived from him, from personal interviews with several members of the Water Commission who have served on the Board continuously from the beginning, and from personal inspection of many of the staves removed during the process of repair.

The salient and important facts brought out by this experience are as follows:

(1).—Staves, which are constantly subject to water pressure from within and are buried in the ground, may be very short-lived.

(2).—The magnitude of the water pressure, beyond a moderate head, has had little or no influence in preserving the timber.

(3).—The pipe laid above ground has not deteriorated to any considerable extent, nor has the pipe laid in the tunnels leading from the distributing reservoir.

(4).—Where buried, its durability has depended upon the soil conditions and the depth of backfill.

(5).—When the depth of backfill has exceeded 2 ft. above the pipe, and the material has been free from vegetable matter, and has been of a fine and impervious character, much less deterioration has taken place.

(6).—Wherever the staves have been in contact with loamy earth or earth containing vegetable matter, or wherever they have been covered with porous material, or to a depth of less than 2 ft., rapid decay has resulted.

(7).—Decayed staves have been found all around the pipe.

(8).—Sound staves have been frequently found contiguous to badly decayed staves.

(9).—The character of the grain, whether slash or grain edge, has not influenced the durability.

(10).—The bruising of the staves during the process of erection seems to have been one of the chief agencies in hastening decay.

(11).—Decay has been confined to the outside of the pipe.

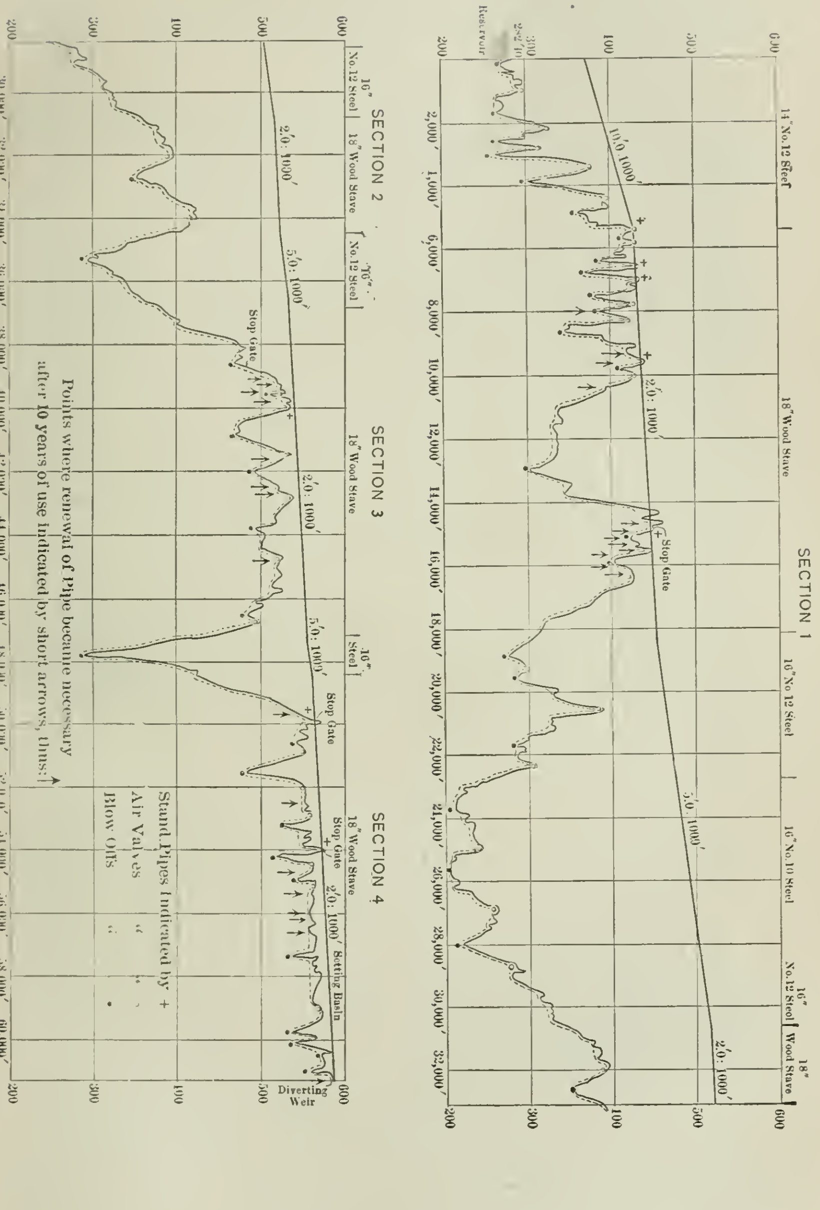

SECTION 1
14" No. 12 Steel
18" Wood Stave
16" No 12 Steel
16" No. 10 Steel
16" No. 12 Steel
18" Wood Stave
10.0: 1000'
2.0: 1000'
Stop Gate
5.0: 1000'
2.0: 1000'
Reservoir
SECTION 2
16" No. 12 Steel
18" Wood Stave
16" No. 12 Steel
2.0: 1000'
5.0: 1000'
Stop Gate
SECTION 3
18" Wood Stave
16" Steel
2.0: 1000'
5.0: 1000'
Stop Gate
SECTION 4
18" Wood Stave
Stop Gate
2.0: 1000'
Setting Basin
Diverting Weir
Stand Pipes Indicated by +
Air Valves " "
Blow Offs " " •
Points where renewal of Pipe became necessary
after 10 years of use indicated by short arrows, thus:

Fig. 1.

(12).—The pipe has not usually shown leakage as long as sound wood has remained in excess of $\frac{1}{4}$ in. in thickness.

(13).—The malleable cast band fastenings have been found to be in good condition.

(14).—The bands, $\frac{7}{16}$ in. in size, have been considerably corroded save where secured by the nut, but all have been used again by placing the nut in its original position.

As to the future of the pipe, Mr. Bergsvik gives the following opinion:

"The pipe in the Second and Third Sections, $2\frac{1}{2}$ miles (Fig. 1), is nearly all buried in fine-grained sand, and will last perhaps 10 years more by giving it a general repairing, say 5 years hence; but the greater part of the First and Fourth Sections will have to be replaced, I think, in about 4 years."

Table 1 shows the extent of the repair and replacement work performed. In explanation, it may be said that, where actual reconstruction was unnecessary, the staves on the bottom of the pipe were not generally disturbed, it being thought better economy to renew them only as leakage rendered it necessary.

Staves having a depth of decay of less than $\frac{1}{2}$ in. were not disturbed. When no repairs were required for a continuous length of 75 ft. or more, the pipe was included in the third section of Table 1, as "in fairly good condition." Shorter lengths were included in the first section, as "requiring partial substitution of new staves."

TABLE 1.

	Section 1.		Section 2.		Section 3.		Section 4.	
	Linear feet.	Percentage of whole.	Linear feet.	Percentage of whole.	Linear feet.	Percentage of whole.	Linear feet.	Percentage of whole.
1.—Pipe requiring partial substitution of new staves	6 850	54	......	......	2 200	21	6 350	49
2.—Pipe requiring rebuilding	1 450	11	......	......	360	$3\frac{1}{2}$	600	5
3.—Pipe uncovered and found in fairly good condition	1 450	......	600	......	6 940	......	4 800	......
4.—Pipe not uncovered, but thought to be in fairly good condition	3 050	......	3 000	......	900	......	1 200	......
Total	12 800	......	3 600	......	10 400	......	12 950	......
New staves required, equivalent to feet of pipe	1 960	......	7	......	800	......	1 500	......

At the stations noted in Table 2 (indicated on the profile, Fig. 1) were located the most important of the sections requiring rebuilding.

TABLE 2.

Station.	Length of pipe. in feet.	Station.	Length of pipe, in feet.	Station.	Length of pipe, in feet.
No. 80	40	Nos. 157 and 158.	128	No. 447	110
" 95	47	" 160 and 162.	108	" 496	20
" 97	40	" 164 and 165.	54	" 526	97
Nos. 103 and 104.	153	No. 391	25	" 548	30
" 147 and 149.	206	" 393	40	Nos. 553 and 555.	175
" 150 and 152.	140	" 395	36	No. 560	93
" 152 and 153.	108	" 396	45	" 561	84
" 154 and 155.	122	" 416	46	" 566	43
No. 156	30	" 425	34	" 611	36
....	....	" 427	24	" 612	22

Table 3, taken from the book record of the Astoria Water Commission, shows the total cost of repairs on the entire 12 miles of conduit, including both the stave and steel pipe:

TABLE 3.—COST OF REPAIRS ON 12 MILES OF CONDUIT.

Year.	Cost.	Year.	Cost.	Year.	Cost.	Year.	Cost	Year.	Cost.
1895...	$108.58	1897...	$63.67	1899...	$46.10	1901...	$243.18	1903...	$350.18
1896...	15.90	1898...	65.50	1900...	71.59	1902...	314.03	1904...	896.10

The cost of making good the damage resulting from two quite extensive land slides is included in these figures. Of the remainder, by far the greater part has been expended on the stave pipe.

The steel pipe is considered to be in fairly good condition. Perforations, however, have occurred in a few places, the total number, for the years indicated, being as follows:

1902	7	perforations.
1903	3	"
1904	9	"
1905	8	"

The total cost for repairs has been $297 for 27 perforations.

The writer hopes that the recounting of this experience with stave pipe may lead to the accumulation, through discussion by others, of much additional information on this important subject. He wishes, however, to caution any one against hastily attributing the lack of durability in the case of the Astoria pipe to the use of fir for staves instead of redwood, as he is prepared to cite an important redwood pipe line in Southern California, built at about the same time as the line under consideration, which has not shown much, if any, better results. Lack of full and exact data makes further reference to it inexpedient at the present time.

DISCUSSION.

Shirley Baker, Assoc. M. Am. Soc. C. E.—The speaker believes that the conclusions drawn in this paper are entirely too general, and in several instances are not supported by the facts as set forth. The practical indestructibility of wood thoroughly saturated with water has become an axiom, so that when the wood of a structure which is designed to act under the condition of saturation decays, it would lead to the supposition that complete saturation was not always maintained, rather than that the fundamental principle, so well proved by time, was at fault. Mr. Baker.

In his Conclusion 2 the author states, "The magnitude of the water pressure, beyond a moderate head, has had little or no influence in preserving the timber," and yet, from his profile, Fig. 1, the points of renewal, as shown by arrows, are found to be entirely on the lighter pressures.

In Conclusions 4, 5 and 6, the author states in effect, that, where buried, the durability depends on the depth of the backfill and the character of the soil, but in Conclusion 8 he states, "Sound staves have been frequently found contiguous to badly decayed staves," and, in Conclusion 10, "The bruising of the staves during the process of erection seems to have been one of the chief agencies in hastening decay."

These conclusions appear to the speaker to be somewhat contradictory. The fact that sound and decayed staves were contiguous would seem to indicate reasons for decay other than the depth of the backfill and the character of the soil.

The author states that the renewal and repair work in 1905 involved an expenditure of more than one-third of the original cost of the pipe. The speaker believes that this statement should have explanation, otherwise the cost of the repair work, compared with the length of the line, becomes greatly magnified. The Astoria pipe line was built at a time of great financial depression, consequently the cost of construction materials was very low. All engineers and contractors know of the great advance in prices during the last ten years. This advance aided in increasing the cost of this repair work; but, further, the speaker is advised that California redwood was used to a great extent in these repairs, and that redwood is much more expensive than the fir which was originally used.

Table 1 is interesting in a general way, but, since the object of the paper is to place the conditions and amount of repair work fairly before the Society, it would seem that the author has not gone far enough in computing the percentage of repair work to the length of the line. From an economic standpoint, this becomes a matter of cost, and, therefore, should include only the material actually used.

Mr. Baker. The quantity of new material used is clearly shown in the last line of the table, under the caption, "New staves required, equivalent to feet of pipe," where the actual quantity of timber in new staves is reduced to the equivalent length of pipe. The summation of these lengths, or 3 267 ft., compared with the summation of the total length of line, or 39 750 ft., shows that the actual quantity of new pipe built is only about 8% of the line. From the fact that in Table 3 is included the cost of repairing "the damage resulting from two quite extensive land slides," one is led to ask if the replacement required on account of these slides is included in Table 1, and, if so, what length of line was carried away by the slides.

Mr. Bergsvik's opinion that part of the line will need replacing in about 4 years, but that about 2½ miles will last perhaps 10 years more, places the ultimate life of the sections in question at 15 years and 21 years, respectively. Although this is a shorter life than is usually ascribed to wooden pipe, the fact that its use effected a total saving of nearly 50% over the cost of a No. 12 gauge steel pipe of equal capacity,* goes a great way toward justifying its installation.

The durability of wooden stave pipe is, indeed, an important question, as many miles are now in use, and engineers, both in the East and in the West, are continually specifying it in connection with municipal water supply, power plants, outfall sewers and irrigation. The speaker is in possession of several interesting reports on the present condition of wooden pipe lines, and begs to quote from a letter, written under date of January 16th, 1906, by Eugene Carroll, M. Am. Soc. C. E., Superintendent of the Butte Water Company, Butte, Montana, where a length of about 30 miles of 24-in. and 26-in. continuous stave pipe is in operation. Mr. Carroll's letter is, in part, as follows:

"The banded redwood pipe laid at this place in 1892, 1899, and 1900 has a permanent covering of 2 ft. above the top of the pipe. The pipe has been uncovered in places at various times for repairs. The bands look first-rate thus far; they are considerably rusted on the older pipe around the threads, and the nuts are so thoroughly rusted to the band that when we have to remove a band it is necessary to replace it with a new one. These old bands which are removed we use again by welding on a new threading. As far as we know, none of the bands have failed on account of rust. Where bands have been removed or replaced, for various reasons in repairing the pipe, we have had no trouble in replacing them, and do so without shutting off the flow, though I would consider it a good practice in case it was necessary to reband any considerable length of pipe, that the flow be shut off and the pressure removed in order to get an even bearing in recinching the bands.

"In the first pipe we built, in 1892, for a short distance through a rocky cut, we were not particular about backfilling, and consider-

* Adams, on Astoria Water-Works. *Transactions*, Am. Soc. C. E., Vol. XXXVI, p. 1.

able of the covering was broken stone. In a short time after using Mr. Baker. it, we discovered that the staves were beginning to show indications of decay along this portion of the line; believing it was caused from the rock filling, allowing the air to get to the outside of the pipe, thereby causing decay, I had the filling entirely removed and replaced with good dirt on top of the pipe. Since then we have had no trouble, and have never found a stave rotted sufficiently to require its removal. Where the pipe is properly filled with good dirt all around the pipe, the seepage from the pipe keeps the earth damp and practically water-logs the wood, so that I see no reason why we should ever have trouble with rot. The expense of maintaining this pipe has been practically nothing.

"The pipe connects our reservoirs, one 13 miles and the other 22 miles out, with our reservoirs in town. The watchmen, which we have to keep at each reservoir, make a trip over the pipe line once a week. Occasionally, in making these trips, it is necessary to dig out the pipe for small leaks, such as wormholes or butt joints, but, with two exceptions, we have never had to use more than two men in repairing leaks, and have never had to shut off the water. Our two exceptions are: first, during the winter of 1893, ice formed inside of our pipe line, being caused from the fact that our reservoir was not completed, and a jam was caused inside the pipe, bursting it, requiring the shutting off of the water, and about 12 hours to repair it. Last spring, on our new pipe line, a leak developed near one of our valve chambers, and, before it was discovered and the water shut off, a bad wash-out took place, which washed the support away from the pipe line for about 1 000 ft., necessitating the rebuilding of the line, taking about four days to do it. This section of pipe last referred to was built by an incompetent foreman at the time, and should have been rebuilt during the original construction, but, on account of the good nature of the writer, was allowed to stand.

"Referring to the pipe line built in '92, will say that it is apparently in as good shape to-day as the day it was built, and it is impossible to estimate as to what its future life will be. I can only say that I am so pleased with my experience with wooden pipe for supply lines that I would use it wherever possible in future construction work. It seems to me that, from the fact that the pipe is thoroughly water-soaked, the life of the wooden part should be very great. The nuts seem to be thoroughly rusted on the threads, and, when they are not disturbed, it looks as if the pipe will not need rebanding for many years to come."

Clemens Herschel, M. Am. Soc. C. E.—It was about twenty Mr. Herschel. years ago that engineers in the East first heard of stave pipe. The speaker was then living in Holyoke, Mass., and, speaking for the class of engineers who had something to do with water-power in New England, he would say that to them it seemed like a very old institution. It rather amused them to think of people out West having made what they were pleased to term a great invention—this, of wooden stave pipe—when it had been used, in the shape of wooden

Mr. Herschel. penstocks, in New England water-powers, very nearly ever since the country was settled. New England engineers thought that it was a great advance to be rid of wooden stave pipe and, instead, to use riveted—in those days—wrought-iron pipe, and, subsequently, steel pipe.

There is just one difference that the speaker can see between wooden stave pipe and wooden penstocks. In the case of water-power and wooden penstocks, it is customary to draw the pipe—empty it—on holidays and Sundays, or at least frequently during the year, whereas, the wooden stave pipe would remain full of water all the time; and he had thought that, perhaps, that might save the wooden stave pipe, where it had not saved the wooden penstocks.

The speaker has told the story a great many times about wooden penstocks, illustrating how they were viewed in New England, and he might as well tell it again. There was one of these old wooden penstocks to take out, and it was done on a Sunday, and the next day he met the mechanic who had had charge of taking it out, and said, "Joe, did you have any trouble about taking that penstock to pieces and getting it out?" "Oh, no," said Joe, "as soon as we got it cleared off, we shoveled it out—no trouble at all." That illustrates the end, the usual end, of a wooden penstock pipe.

It is stated in this paper that the pipe rots from the outside in, until a very thin portion of the wood alone remains sound. That feature, also, is nothing new. In the paper read before the Society in 1886,* on the preservation of the Holyoke Dam, by the speaker, it is stated that in repairing that wooden dam it had been found that the 6-in. timbers which constituted the covering of the dam, constantly under 20 ft. of water, were rotten from the up-stream side down stream, until there was no more sound wood left than would form a veneer. It was really curious to see how closely those 6-in. timbers came to being rotten clear through and yet have that little veneer on the water side preserved, sometimes not more than ⅛ in. thick. That is a parallel case to stave water pipe rotting from the outside in.

Mr. Henny. D. C. Henny, M. Am. Soc. C. E. (by letter).—The writer has read with interest the account of the experience with the stave pipe portion of the gravity supply pipe line of Astoria, Ore., completed in December, 1895. The information contained therein is of the greater interest to him as he was at that time manager of the contracting company, and, for a portion of the time, in personal charge of construction. He is familiar, therefore, with the character of the material which entered into its construction and with the general conditions affecting its life.

* "On the Preservation of the Dam at Holyoke, Mass., in 1885, and on Some Studies for a New Stone Dam for the Same Place," *Transactions*, Am. Soc. C. E., Vol. XV, p. 543.

Reliable information, regarding the actual condition of structures which are buried, is, as a rule, very difficult to obtain, and, except under the most favorable conditions, can hardly ever be acquired at first hand. Such information as can be secured should therefore be closely sifted from error and exaggeration, and this process of sifting has constituted one of the greatest difficulties in the writer's search after facts of this kind. Even when standing in the relation of consulting engineer for a company operating steel and wooden pipe lines buried underground with which trouble had been experienced, due respectively to corrosion and decay, he has never been enabled to make personal examination, except by the merest accident. Thus the information gathered by him has been fragmentary and unsatisfactory in character. It has been sufficient, however, to lead him to a realization that exceptional conditions may place a limit on opinions previously entertained regarding the life of wooden stave pipe. Mr. Henny.

It has been generally held that sound wood in a condition of complete saturation will prove to be immune from decay, and that the shell of a wooden pipe, even if under very moderate water pressure, will be fully saturated. A great mass of evidence, constantly increasing, tends to confirm this conclusion under ordinary conditions. Among the large numbers of pipe lines built by others, as well as by the writer, there are in all six individual cases in regard to which he has had knowledge of decay having resulted during a use of 6 years or more. In three of these cases the decay was confined to the immediate vicinity of summits close to or touching the hydraulic grade line, or to places where it was known that the pipe had been running only partially full for a considerable portion of the time. A fourth case was that of a pipe line left unburied, and exposed to the trying atmosphere of Southern California. These cases, therefore, are in confirmation of early views on this subject. It is only in the remaining two cases where the facts seem to contradict the broad conclusion above mentioned, and where decay appears to have set in at points where, owing to the depth below grade line and below summits, the pipe must have been full practically all the time. These are the two cases quoted by the author; and it is fortunate that for one of these cases, at least, some information is now at hand. This information has been obtained through the assistant engineer on the original construction, who has been in direct charge of the repairs, and, therefore, it may be looked upon as of unusual reliability. It is to be regretted that neither the writer, for reasons previously stated, nor the author, appears to be in a position to furnish reliable data regarding the second case referred to, and it is hoped that such data may yet be at hand and be presented before the author closes his discussion.

With the experience gained, it now behooves engineers to study

Mr. Henny. it closely, and determine whether it furnishes sufficient basis to justify a radical modification of previous views.

During the 10 to 11 years' service of the pipe it appears that 11% of all staves have been renewed, and $5\frac{1}{2}\%$ of the line has been reconstructed. On the profile, Fig. 1, the locations of the reconstructed portions of the line are indicated by arrows. It will be seen at a glance that all these arrows are in close proximity to summits and within 50 ft. of the nominal hydraulic grade line. No reconstruction has been required on the long sections of pipe where the pressure has exceeded 50 ft. To that extent, at least, the pressure has had an effect upon the preservation of the wood; which is a salient and important fact, not in complete harmony with the series of facts or rather conclusions presented by the author. Whether it warrants the conclusion that the protection afforded by pressures exceeding 50 ft. has been sufficient, not only to obviate reconstruction, but to prevent decay, depends upon the present condition of the staves under such circumstances. This is a crucial point, in regard to which no satisfactory information has been furnished.

In view of the generally favorable experience with this class of pipe, under conditions of constant pressure and protection from evaporation, direct and conclusive evidence is required to upset previous views to the extent now advocated by the author; and, until such evidence is presented, the writer cannot agree to the sweeping character of the author's conclusions, and takes exception notably to Conclusion 1, unless qualified as to the degree of pressure, and to Conclusions 2, 4 and 6, because they are not fully supported by the information submitted.

Why light pressures have generally been a satisfactory protection to the shell of a wooden pipe, and why, in the case of the Astoria pipe, a different experience has been had, is an interesting phase of the question, into which the author has not entered.

Decay, being a growth of fungus, is clearly communicable, and it may well be considered that the soil in a primeval forest, such as is traversed by the Astoria pipe line, containing decaying woody material, the accumulation of ages, may have produced this exceptional result.

With the experience gained, it may be concluded that, under such conditions and in a moist climate, evaporation from the surface is less to be feared than contact with soil, and that better results would have been obtained if at least the light-pressure portions of the pipe had been constructed above ground.

Increased knowledge of this character should lead to a determination of the conditions under which certain classes of construction may be considered safe and lasting, and under which they can be used with confidence; and this confidence will be the greater because of a more perfect understanding of the proper limitation.

G. P. Hawley, Assoc. M. Am. Soc. C. E. (by letter).—In 1905 the water-works system of DePere, Wis., was enlarged and improved, under the writer's supervision. Some mains were replaced by larger ones, and in many other places the pipes were uncovered to make connections and repairs, so that there was abundant opportunity for examining the old pipe, laid during the period from 1886 to 1888. The old system was under the natural pressure of Artesian wells. This was said to be about 40 lb. per sq. in. when installed, but the pressure has gradually fallen to less than 20 lb. at the present time, and therefore pumping has been resorted to. Mr. Hawley.

The old mains consisted of pine logs, turned and bored from one piece, in various lengths up to about 13 ft. They were wrapped with continuous iron bands, and covered with a tar mixture to protect the bands from rust. They were then rolled in saw-dust to prevent sticking. The three sizes used had internal diameters of 2, 4 and 6 in., and external diameters of 4, 7¼ and 10 in., respectively.

From observations made in many different sections of the system, the wood was found to be in a perfect state of preservation in every case. Where there has been any trouble, it has been due to the bands rusting and allowing the pipe to split under the pressure, and can be traced to local causes. Usually, the coating had been knocked off, in making connections, digging sewer trenches, etc., thus exposing the bands to rust; otherwise, on removing the tar coating, the iron was found as bright as new.

The pipes lie about 4 ft. deep, mostly in a heavy red clay. One stretch of about 1 000 ft. through sand and sandy clay was replaced by larger pipe; here the pipe coating had many blisters, under some of them water had penetrated, and the bands had commenced to rust; this was particularly noticeable in the 2-in. size; the 4 and 6-in. pipes, having a heavier coating, were very little affected.

Since completing the new system, a pressure of from 60 to 70 lb. per sq. in. has been maintained on these old pipes (more than a year), and, except for some local defect, as above noted, they have given no trouble.

Andrew Swickard, Assoc. M. Am. Soc. C. E. (by letter).—If this paper brings out a thorough discussion of the durability of wooden pipe, as affected by general and local conditions, the profession will have been benefited. It is to be hoped that the discussions will be more than mere statements of the condition of the materials of which any particular pipe is constructed. Such statements are well enough as far as they go, but the writer believes them to be comparatively without value unless the circumstances and conditions, both general and local, which have brought about or influenced the deterioration of the pipe, are considered in connection with them. This is true of any kind of pipe—cast-iron, Mr. Swickard.

Mr. Swickard. riveted steel, or wood. It is well known that iron pipe gives good service under some local conditions and exceedingly poor service under others. That this is also true of wooden pipe is shown by the fact that the durability of the many pipe lines which have been installed is not dependent upon the number of years they have been in use. Different lines built of the same kind of wood, but in widely separated localities, vary greatly in length of life, as do different parts of the same pipe line. There are always reasons for these differences, and a knowledge of such reasons is what is most desired by the profession. The fact that the condition of the Astoria wooden pipe line does not compare at all favorably with that of many others, which are much older, suggests that there must be local causes which have brought about this comparatively rapid deterioration.

The author has made no effort whatever to account for the rapid deterioration of the parts of this pipe which have had to be renewed, notwithstanding the fact that he must know there are many wooden pipe lines, which are much older, and are in excellent condition; and that the reasons for the rapid deterioration of this pipe are what the profession most desires. The mere fact that any engineering structure has failed is of no value to an engineer unless he knows why it failed. No one is in a better position than the author to give the facts in this case.

There are but two conditions under which wood is absolutely decay proof: Absolute dryness, and constant submersion in water. The staves of a wooden pipe full of water are never in the former condition, and can only approximate the latter. As regards the degree of saturation, the staves in a pipe, therefore, are at some stage between the two extremes, and, as a consequence, are more or less subject to decay, depending upon the amount of moisture in the outer portion, which in turn is dependent upon the pressure.

If the moisture condition of the staves, alone, controlled the ravages of decay, then stave pipe built of wood of any particular kind or quality would last as long in one locality as in another, the pressure being the same. This is not true, and leads to the inevitable conclusion that there are other causes entering into the determination of the life of wooden pipe. These causes are almost entirely local, and generally cannot be controlled, but could probably be avoided or rendered less effective if known and understood. If local causes cannot be avoided, decay can be arrested somewhat by rendering the moisture condition of the staves less favorable by increasing the pressure. For example, had the Astoria pipe been located so that the portions which have had to be renewed (which are under a very low head) had been under a pressure equal at

Mr. Swickard.

least to the "moderate" pressure referred to by the author, then the life of this entire pipe line would have been as long as that of Sections 2 and 3, or about 20 years. It is to be regretted that the author has not mentioned definitely what this "moderate" head is, above which the water pressure had little or no influence in preserving the wood. The only condition affecting the ravages of decay, that can be controlled at all directly, is the moisture condition, and, if this has little or no effect above a certain "moderate" head, it is desirable to know what this head is.

Decay is a vegetable growth, the branching threads of which gradually spread themselves in all directions in the wood, converting the cell walls and the contents of the cells into food for their consumption. In order to thrive, this vegetable growth must have, among other things, some moisture. Air-dried wood does not contain sufficient moisture, and wood constantly submerged in water contains too much. Somewhere between these two the most favorable conditions will be found. Heat, to some degree (from 60 to 100° fahr. is most preferable), is necessary for a prolific growth of decay; too great a degree of heat will kill it, but intense cold will only arrest it. If the moisture and heat conditions are favorable, and there is an abundance of decaying wood scattered about to furnish the decay spores or seeds, new wood can be expected to begin to decay very soon and very rapidly.*

The Astoria pipe line is located for the most part in a dense forest of Oregon fir and hemlock. This region has an enormous rainfall which is distributed over a great part of the year. Throughout the forests of the Northwest there is much fallen timber, which is found in all stages of decay. The rainfall furnishes a favorable moisture condition, and, taken in connection with a moderate and even temperature, the fir logs decay rapidly, and, as a consequence, decaying wood is found all about. New wood in such a locality will show evidences of the deposit of decay spores within a very few days, if sought in the proper way. It is not to be understood that the new wood will show evidences of decay, but that the decay seeds have been deposited in the wood and are ready to grow. Favorable conditions, then, are all that is necessary, and the decay fungus will take root and thrive.

The arrowheads on the author's profile (Fig. 1) indicate that the repairs were confined to those portions of the pipe which were subjected to low pressure, and where the moisture condition for decay growth was most favorable. These parts are where it would be expected that decay would be most rapid. That, at these points, the pipe lasted for 10 years indicates that the low pressures prevailing produced moisture conditions which were not the most

*Bulletin No. 10, U. S. Department of Agriculture, Division of Forestry.

Mr. Swickard. favorable for decay. Fir wood decays very rapidly, and pieces of timber similar in form to the staves used would decay beyond any possible use in 2 or 3 years, if left subject to the natural conditions that prevail in that region.

The writer is not willing to accept the author's statement that "water pressure, beyond a moderate head, has had little or no influence in preserving the timber." This is contrary to reason, and very probably the facts in this case, if carefully ascertained, would not bear out the statement. The quotation of Mr. Bergsvik, as to the future of the pipe, is relevant. Sections 2 and 3 will last 10 years longer, and it is very probable that the repairs mentioned as necessary 3 or 4 years hence will be confined mostly to Section 3, which is under a lower pressure than Section 2. It is to be regretted that the author has not gone into more detail in these matters, which are really the most important.

There are other local conditions which very probably have exerted an influence in the deterioration of this pipe line. All the different kinds of wood available for pipe work have not the same characteristics that affect their durability. The wood from trees of the same species always varies in durability with localities and consequent changes in climatic conditions; even wood from different parts of the same tree varies. The most favorable conditions as to soil, moisture, light and heat, produce, in hard woods, the greatest degree of durability, but have the opposite effect in coniferous woods. The Astoria pipe line was built of Douglas spruce or Oregon fir, which are coniferous woods growing in the region about the Columbia River under most favorable circumstances. As a consequence, the fir lumber from this region would be expected to be less durable than that grown farther north, where the climate is more rigorous.

The author makes no mention of the fact that at least some of the most badly decayed portions of this pipe were at points where roots had grown down and entwined themselves about the surface of the staves. Brush or shrubbery, which grows rapidly and requires an abundance of water, will cause much annoyance along a wooden pipe line. About 3½ years ago about ½ mile of 24-in. wooden pipe was laid in Long Beach, in Southern California, a portion being in ground overgrown by willows (willow grows very rapidly and requires much water). Roots from these willows have covered the surface of the pipe, and the staves are deteriorating rapidly. The other portion of the pipe is in perfect condition. This mile of pipe was laid as an extension to about 11 000 ft. of pipe built in 1900. In making the connection between the old and the new portions, it was necessary to take apart about 200 ft. of the old pipe in order to lower the grade, and the staves of the older portion were so perfect that they were all rebuilt into the pipe. The decay of a portion of this line is caused by a purely local condition. To cite this case as a criterion of the life of wooden pipe, without some explanation of the prevailing conditions, would be unfair. The same remark will apply to the Astoria pipe. **Mr. Swickard.**

The redwood pipe which the author refers to as being in no better condition than the Astoria pipe is very probably the 30-in. pipe built for the West Los Angeles Water Company in 1896 and 1897—the plant of the West Los Angeles Company has been transferred to the City of Los Angeles. One of the main troubles with a considerable portion of this pipe line is that it passes through ground overgrown with willow brush; and where the roots entwined themselves about the pipe there was a rapid deterioration of the staves. The roots seem to reduce the wood to a condition different from that produced by ordinary decay. Causes of deterioration of this character are local, and, usually, can be avoided. Since 1898 this pipe has been full of water with the exception of 8 or 10 days each year during the past 3 years. Repairs were made during these 8 or 10-day periods, and certainly the time indicates that they were not extensive.

The importance of keeping a wooden pipe full of water at all times cannot be over-estimated. If a pipe is but partly full of water, even for a few months, decay, which otherwise might not have started, will begin. Wood once attacked by decay is predisposed to further deterioration, even if measures are adopted to render the conditions less favorable. The winter of 1897-98 in California was exceedingly dry, and the West Los Angeles Water Company did not have sufficient water to keep the 30-in. pipe, before referred to, filled. During the summer of 1898, it was found that decay had started at some points, and the company was put to considerable expense trying to keep the pipe full of water. The decay was of a kind that destroyed the staves very rapidly. In a comparatively short time in a few places the staves were rotted nearly through. This rapid decay was the more remarkable because redwood has greater durability than any other wood available for pipe work on the Pacific Coast.

The Southern California Mountain Water Company, at San Diego, had a similar experience with a line of 36-in. pipe, due to the pipe, or portions of it, being only partly full of water during an exceedingly dry and hot summer. The writer saw samples of staves, taken from this pipe, which had the appearance of wood subjected to high heat in the presence of a limited quantity of oxygen. The company recognized the fact that its troubles were due to avoidable causes, and has repaired this pipe. About 10 miles of 36 and 24-in. pipe have since been built as an extension

Mr Swickard. of this line, and the company is planning to make another large installation of redwood pipe.

In 1890 the City of Cheyenne, Wyo., built about 9 000 ft. of pipe, using Oregon fir staves. The upper part of this pipe was never full of water, and, as a result, the top staves decayed rapidly, and a portion of the line—about 4 000 ft.—was replaced with sewer pipe. The remaining portion of the wooden pipe is not kept full of water all the time, but is still in operation; it is in an unsatisfactory condition. This is an instance where wooden pipe should not have been used.

A decaying fence post will show most evidence of rot at or just below the surface of the ground, with a gradual decrease downward, while it will be absolutely sound above the ground. This may also be expected to be true in the case of wooden pipe, where it is built with portions above and under the ground.

All forms of construction which remove a portion of a stave from immediate contact with the water should be avoided. In making junctions between wooden and iron pipe, the method of putting the iron inside of the wood should be avoided. The 10 or 12 in. of stave-lap on the outside of the iron pipe is almost sure to give trouble in a comparatively short time. It is best to make a special iron hub to receive the end of the wooden pipe, and caulk the joint with oakum and lead in a manner similar to that used in making joints in cast-iron pipe.

When, for any reason, decay attacks a pipe which is subjected to considerable pressure, there does not seem to be any reason why staves at the bottom of the pipe should not decay, as well as those at the top. If the head on the center of the pipe is so slight that the difference of pressure between the top and bottom is comparatively large, one would naturally expect decay to take place in the upper staves first and gradually extend to the lower ones. That sound staves are found contiguous to decayed ones is due to the difference in the durability of the wood of different trees of the same species, and also to the difference in the durability of staves cut from the same tree. Staves taken from the butt cut of a coniferous tree are always more durable than those taken from the upper cuts. It is not practicable to obtain lumber cut from the butts of trees only, except in limited quantities and at considerable extra expense.

If the Astoria pipe line was the only wooden pipe line that had ever been built, or the oldest, it would naturally be taken as a criterion of the life of this class of water conduit. There are many pipe lines, which were built a good many years before the Astoria line, which are in good condition to-day. The first wooden pipe built strictly in accordance with modern construction was installed by the Denver Water Company. Some of its oldest pipe was installed about 25 years ago. This company is undoubtedly satisfied with its experience, for, at the present time, it has under consideration the installation of about 12 miles of large-sized wooden pipe. This will probably be built in 1907. It is fair to presume that the numerous installations made in recent years at points not far distant from Denver, and also the projects now under consideration in that region, which include the extensive use of wood pipe, are due directly to the fact that the early installations have proven highly satisfactory. To select one of the oldest wooden pipe lines, which has proven satisfactory, as an example of what might be expected from such pipe in any locality, without reference to the conditions that made a long life possible, would be manifestly unfair; to go to the other extreme, would deserve the same criticism. Mr. Swickard.

The author neglected to mention that the Astoria Water Commission installed 500 ft. of redwood stave pipe in connection with the water system. This was built in 1900. Where the old pipe has been rebuilt, redwood staves have been used. It would be interesting to know whether the present condition of the 500 ft. of redwood pipe, compared with that of the fir pipe when it was of the same age, influenced the Commission to use the redwood staves. Redwood railroad ties last more than twice as long as fir ties, as far as decay is concerned, and it is probable that this same ratio of durability would exist in all cases of similar conditions and circumstances. It seems fair to presume that, had redwood been used instead of fir, the life of those portions of the pipe which have been repaired would have been 20 years instead of 10, and that the life of the remainder, especially those portions under a moderately high pressure, would have been 35 or 40 years.

Since preparing the foregoing, the writer has received information, from a most reliable source, to the effect that the air-valves referred to and indicated on the profile of the pipe line (Fig. 1), are merely check-valves. These check-valves do not allow the entrapped air to escape, and are only intended to let air into the pipe—if for any reason (by design or accident) the water is suddenly allowed to escape at a lower point—and to allow the air to pass out when the pipe is being filled.

As the air-valves represented are not such in the sense that they will allow the air to escape while the pipe is under pressure, it is certain that air accumulates at the high points on this pipe line. This accumulation of air, without doubt, has had the effect of producing a moisture condition in the staves at these points, which is more favorable for decay growth than would otherwise have existed.

Mr. Swickard. It will be noted that, while most of the repairs are indicated as having been made at the high points of the pipe, none (with perhaps one exception) are indicated as having been made at points where stand-pipes are located. This is a significant fact, and seems to indicate very clearly that the entrapping of air at the high points has been more than an ordinary factor in hastening the decay that has taken place.

Mr. Hunt. A. M. Hunt, M. Am. Soc. C. E. (by letter).—Within the past year the writer has had occasion to inspect three short lines of wooden stave pipe installed in connection with power plants on the Truckee River, in California.

One is a 9-ft. pipe, about 1 800 ft. long, supplying water to operate a paper mill at Floriston, Cal. It is of the usual construction, having redwood staves about $3\frac{5}{8}$ in. thick. The pipe is mainly above the surface of the ground, the lower half of its circumference being bedded with earth and stone spalls. It is under a pressure varying from about 16 to 35 ft. It was built in 1898 and has been in continuous use ever since, being kept full of water. It shows no evidence of decay of staves or corrosion of bands.

The writer had occasion to bore through the pipe, to make velocity measurements, more than a year ago, and, although the outside of the wood was dry, on getting below the surface, it was found to be moist.

There are two other lines of 6-ft. pipe just below this line, supplying water to the wheels of the Truckee River General Electric Company. These lines were built in 1899, and the staves, the writer thinks, are of Oregon pine. Up to the present time, the pipe has shown no evidence of decay. It is supported in the same manner as the pipe first mentioned, the upper half being exposed. The pressure varies from 6 to 65 ft.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The profession is greatly indebted to the author for boldly giving the results of his experience. Such cases bring out most important facts, which ought to be known, and the sooner the better.

The writer has not had much direct experience with pipes of this class, but has always held the opinion that they are good, cheap and durable, and entirely suitable for light pressures. It now seems that experience along the entire Pacific Coast goes to show that the life of wooden stave pipe, when buried in the ground, is frequently only from 10 to 11 years. A closer observation leads to the conclusion that the chief danger, as far as known, lies in loamy soil, or where the back-filling contains vegetable mould. Apparently, it makes no difference whether the ditch is deep or shallow, the pipe under heavy or light pressure, or whether the staves are of pine or redwood—the results seem to be about the same.

Mr. Le Conte.

Nevertheless, there are certain places along the same pipe line where for some reason the pipe is apparently all right. The exact nature of these conditions, in any case, and, of course, their full extent, are as yet largely unknown. It remains for future experience to bring out and develop the facts clearly. The writer learns, from good authority, that the supply-main feeding a certain section of Los Angeles and the Soldiers' Home is of redwood stave pipe some 30 in. in diameter. This pipe line is now about 11 years old, and is reported to have practically gone to pieces. It is entirely proper, however, that the facts in this interesting case should be submitted by the City Engineer of Los Angeles, which no doubt he will do.

The Oakland Water Company has some wooden stave pipe leading from the flowing wells at Alvarado to the sump at the pumping station. This piece of pipe is laid in wet, soggy ground, and one would naturally suppose it would be all right in such soil. It has been lying there for about 11 years, and examination shows that it, also, has all but gone to pieces.

Wherever this pipe is laid underground, therefore, there will always be a certain doubt as to its probable durability. It is likely to rot out, in places, in 11 years, depending upon the character of the soil in which it is laid. As far as known, vegetable mould is one of the active agents which seem to hasten decay.

In view of the many uncertainties connected with the probable life of such pipe, when laid underground, it would seem to be advisable, wherever practicable, to lay it on top of the ground, where it is known to have good lasting qualities. In a great many cases this can be done without objection.

Mr. Hatton.

T. Chalkley Hatton, M. Am. Soc. C. E. (by letter).—This is a very important subject for the several reasons that the prices of steel and cast-iron pipe have advanced to such a point that many small cities and towns are prevented from carrying out much needed improvements to their present water supplies; and also from the fact that the demand for cast-iron pipe at present is so great that, even though the municipality is possessed of ample funds, such pipe cannot be secured in any great quantity in less than 6 or 9 months. The costs of lead and labor have also advanced very much during the last year, and thus it becomes almost mandatory to seek some material to take the place of iron and steel.

For these reasons alone, the writer, during the past three years, has investigated in considerable detail the merits of wood pipe. This investigation has covered much of the irrigation work in the West where continuous wooden stave pipe has been used for many years, and several of the municipal and industrial plants where

Mr. Hatton. both continuous and machine-made wooden stave pipe have been in use for at least forty years. It may be of some interest to present briefly the results of these investigations.

Continuous wooden stave pipes, where bruised, either in the handling or by cinching the bands too tightly, were found in most instances to have suffered some decay where the pressure was moderate—less than 50 lb. This was also found to be the case where the longitudinal joints were wide at the outer surface and the contact not good, or at the end joints, which were not in close contact, or where a dry crack existed in the end of the stave before it was built into the pipe. The cause for this is very clear. Where the wood fibers do not have natural and direct contact throughout, and recesses are left, the water pressure from within is not continuous, and perfect saturation does not result. Thus lodgment is found by the organisms which destroy the wood, and its decay is inevitable. To avoid this, the writer believes that the outer surface of the wood should be thoroughly coated with some protective material which will prevent the entrance of these organisms, and that this coating should be applied as soon as the wood comes from the planer, thus preventing the organisms from finding a home. If it should be applied after the wood has been exposed to the atmosphere, these organisms will work just as well under the coating. This fact is apparent to anyone who has closely observed the life of timber under various conditions.

These investigations did not disclose any facts such as Mr. Adams describes in his Conclusions 5 and 6, as far as contact with vegetable matter was concerned. In many places where the pipes were uncovered the material in contact therewith was at least 50% vegetable matter, and no evidence of decay was found, although the pipes had been in constant use from 8 to 14 years. It was found, however, that where the top of the pipe was only partly covered, or had but a few inches of dry earth over it, a dry rot had extended to a depth of from $\frac{1}{8}$ to $\frac{1}{4}$ in., but as soon as the point of constant saturation was reached, the wood was sound.

The greatest defects which the writer found in the continuous wooden stave pipe were in the end joints between the staves, due to the saw kerf of the two ends not being exactly in position, so that, when the metal tongue was inserted, the inner or outer edge of one stave was a little higher or lower than that of its neighbor. Another marked defect was in the durability of the steel bands. These bands are from $\frac{1}{4}$ to $\frac{3}{4}$ in. in diameter, are bent to a circular form, coated with a preservative of asphaltum pitch or material of similar character, and are supposed to be free from oxide when placed on the pipe. As a matter of fact, however, the writer saw delivered upon the work hundreds of these bands, which had been exposed to the weather for sufficient time to give them a thick coat of rust. In this condition they were dipped into a bath of the protecting material, thus covering the rust but not stopping its deteriorating effects by any means. In many instances the threads at one end of these bands were badly rusted, not having been coated; and in most instances it would have been dangerous to move the nut, as stripping would have doubtless occurred. Mr. Hatton.

The writer's investigation of continuous wooden stave pipe led him to believe that there were too many uncertainties in its manufacture; that, while the wood would undoubtedly outlive steel, and perhaps cast iron in many instances, yet its construction must be necessarily left to irresponsible, partly unskilled workmen, who in many instances had to prosecute their work under the most unfavorable conditions. For these reasons he turned his attention to a study of machine-made wood stave pipe.

Here he found a wooden pipe being made of the same kind of materials as the continuous wood pipe, all made and banded together by automatic machines run by capable workmen, and every stave and band, and the coating and all workmanship entering into the construction, open to inspection under the most favorable conditions. The inner and outer surfaces of the pipe are cut to fit a circle; the steel bands, of either continuous band steel of any width required by the pressure to which it was to be subjected and of about No. 16 gauge, or of steel or copper wire of any gauge demanded, while bright and free from rust, are run through a bath of warm asphaltum pitch and then wound spirally upon the pipe, carrying with it a heavy coating of the pitch, thus preventing the inner surface of the band from coming in direct contact with the outer surface of the staves. Each end of the band is well secured by a double band around each end of the 7 or 8-ft. section, the length in which the pipe is usually made. As soon as the banding is completed, the section of pipe is run over two rolls moving through a warm bath of the asphaltum pitch, thus coating the outside surface of the wood and bands with a full protective coating of from $\frac{1}{4}$ to $\frac{3}{8}$ in. in thickness. To prevent this coating from running, the pipe is immediately rolled in saw-dust, which adheres to the coating and prevents it from being knocked off or abraded in handling. An extra coating can be readily applied to each section by repeating the operation, and, in all instances occurring under the writer's notice, this covered the bands and pipe to a thickness of nearly $\frac{1}{2}$ in. This coating, as soon as cool, becomes very tough, and can only be removed by a chisel. It is not injured in shipping and handling, and the pipes come upon the work in as good condition as when they leave the factory.

These machine-made pipes, as made in the West by the Washing-

Mr. Hatton. ton Pipe and Foundry Company, are provided with a wood stave coupling about 8 in. long, which fits over the spigot end, said spigot end being driven into it, making a strong and water-tight joint. The machine-made pipe, as made by the Wyckoff Companies, of Elmira, N. Y., differs from the above in having a tenon, 4 in. in length, on one end and a mortise of the same depth on the other end. The latter is in the form of a truncated cone, so that when driving the pipe home the joint becomes tighter the further it is driven. The inner and outer surfaces of the pipe, when formed by this method, are continuous throughout, that is, there are no projections, and the swelling of the wood when saturated by the water makes the joints as good as the body of the pipe. These pipes can be made of any diameter from 6 to 60 in., to withstand any pressure up to 300 lb. per sq. in., according to the banding, and can be laid by unskilled workmen in either wet or dry trenches.

The writer is now laying 4 miles of 24-in. Wyckoff pipe in Southern New Jersey to carry water from a reservoir to the Carney's Point plant of the E. I. DuPont Powder Company; and 800 lin. ft. of this pipe have been laid in a day, much of it in from 6 to 8 in. of water in the bottom of the trench. The jointing of the pipe is such a simple matter that the inspector, standing on the surface above the trench, can tell at a glance whether it has been properly done, as any change in alignment or gradient can be detected at once at the joint, because the two shoulders of the adjoining pipes must abut all around when properly laid.

When this work is completed, the writer hopes to present to the Society the results of experiments as to flow, costs, etc., which he is going to make. It may be stated now that the contracts let for the 4 miles of supply main show a saving of $53 130 over cast-iron or steel pipe for which preliminary bids were received.

The description of the manufacture of machine-made wooden stave pipe has been given in some detail because the writer knows of many members of this Society who have never had it brought to their attention and know nothing of its merits, and he hopes his investigations may help them to revise their future estimates, as he was glad to be able to do.

In investigating the durability of this machine-made pipe, the writer visited a number of places where it had been used for 30 years under a varying pressure of from 50 to 200 lb. per sq. in. Some of this pipe had been used for 8 years under a direct pumping pressure of 200 lb. per sq. in. Some lengths of pipe were shown which had been in service for 30 years and had been taken up to be replaced by the same make of pipe of a larger size. The wood was in an excellent state of preservation; the bands, while showing slight corrosion, were in good condition to reinforce the pipe; the asphaltum coating showed some deterioration on the outside, but that on the under side was in a good state of preservation. The pipe had been laid with about 4½ ft. of covering, the soil being gravelly clay with some quicksand. **Mr. Hatton.**

The writer has received letters from the officers of twenty-five municipalities, water companies and industrial establishments where this pipe has been in use from 5 to 46 years, and in every case these officials testify to the durability of the pipe. In no case was an answer to his inquiries unfavorable, or indicating that the pipe was not perfectly satisfactory in every way.

Several years ago, while building a sewerage system in Wilmington, Del., the writer dug up some 300 ft. of wooden water pipe, buried some 8 ft. underground. This pipe was made of bored logs of white pine, was full of water not under pressure, and was sound throughout, although it had been laid prior to 1837—how long before that was not known. The joints were made by a tenon and spigot, and were in excellent condition. The logs had cracked in every case in two concentric rings about equidistant, and also at right angles to the axis at several points, no crack extending through the body of the pipe. Every pipe was in this condition.

This season, while making some improvements to the streets of Salisbury, Md., some old corduroy road was dug up on the main street. The logs were of white pine, from 6 to 8 in. in diameter and were buried approximately 12 in. below the surface of the street, which was surfaced with oyster shells. These logs, of which there were a great many, were carefully examined, and were found to be in perfectly sound condition throughout.

The foregoing illustrations lead the writer to believe that wooden stave pipe when made of the proper material, namely, good soft white pine, Douglas fir, cypress or even red cedar, well selected, and free from dry or black knots, either air- or kiln- dried, free from cracks, well jointed and secured with steel bands protected from corrosion, will last, under average conditions, as long as cast-iron pipe; and the lasting qualities of steel are not in the same class.

The writer further believes that the life of the wood can be greatly increased by giving it a further protective coating of asphaltum pitch, and that the steel, iron or copper bands should be thoroughly covered with the same material, and therefore they should be of flat band-steel or wire. He also believes that machine-made pipes are more durable, as they can be inspected more carefully during manufacture, built by more skilled workmen, and can better be covered with protecting material. "The proof of the pudding is in the eating;" proof of the durability of wooden stave pipe is obtainable in all parts of the United States, and may be secured by any engineer for the asking.

Mr. Ralston. J. C. RALSTON, M. AM. SOC. C. E. (by letter).—This paper seems to be timely and interesting. The writer desires to emphasize several of the points raised by Messrs. Baker and Henny in their discussions.

It should be borne in mind that all this literature relates specifically to the so-called "continuous stave pipe" used for conduit purposes, and ranging in diameter from 18 in. upward. A great deal of machine-banded, or wrapped, pipe, ranging in diameter from 4 to 20 in., is now manufactured, principally on the Pacific Coast, and is used extensively in municipal and private water-works. The adverse findings on the Astoria line, even in their broadest generalizations, can hardly be applied to the latter pipe.

Mr. Adams, in his Conclusion 1, says, "Staves which are constantly subject to water pressure from within and are buried in the ground, may be very short-lived." It is the writer's experience that had Mr. Adams qualified his conclusion by interpolating "subject to light water pressure" a truer expression would have been recorded. The proof of this is readily found by an examination of the published profile. Mr. Baker also refers to this item. It may be noted that within Section 1, running approximately from Stations 105 to 143, a distance of 4 000 ft., the average pressure is perhaps more than 40 lb. Again, between Stations 163 and 181 (1 800 ft.) the pressure ranges roughly from 15 to 55 lb. Section 2 has a run of 3 400 ft. of wooden stave pipe between Stations 308 and 342, approximately, all of which is under heavy pressure, say, about 56 lb. On Section 3, between Stations 370 and 390, pressures ranging from 17 to 60 lb. obtain. Also between Stations 400 and 415, a maximum pressure of about 30 lb. prevails, while, between Stations 428 and 470, a distance of 4 200 ft., with the exception of 110 ft. at Station 447, the pressure is substantial, running up to as high as 60 lb. And Section 4, a run of 1 400 ft., between Stations 482 and 496, carries a pressure from about 13 to 60 lb. Finally, on the same Section, between Stations 497 and 525, the head is between, say, 10 and 35 lb.

All the points at which repairs were necessary, according to the profile, fell within the limits of low-pressure conduit pipe; that is, wherever the pipe hugged the hydraulic grade line, where the pressure was nominal, there and there only did rot occur in the staves. Where a substantial pressure existed, no repairs are reported. The inference, therefore, seems conclusive that where the pipe departs from the function of a mere conduit and becomes a pressure pipe, so to speak, then the old well-recognized law of preservation due to saturation becomes operative. This is a noteworthy distinction, and one which the writer has long recognized; in fact, he believes that, when a conduit line is laid out, the engineer should fix a minimum distance below the hydraulic grade line and, as far as possible, lay the pipe at or always below such a contour. The evidence in Mr. Adams' paper would seem to indicate that for an 18-in. line, had he been able to keep down, say, 40 or 50 ft. below the grade line, there would never have been any rotting of staves. Mr. Ralston.

Mr. Adams' Conclusions 4 and 5 justify the well-entrenched belief that a deep backfilling, of clean, pure earth, containing no vegetable mould or other unstable material, is as necessary for length of life to the wooden stave pipe as to the steel or iron pipe. It is of equal importance that the backfilling for this class of pipe be as thoroughly tamped as on any high-class pressure line. Vegetable matter contained in the backfilling results in one of two things: first, as it decays, it reduces the bulk of the filling, leaving numerous voids and channels for the entry of air and water; or, second, it causes a measurable subsidence of the covering, often reducing it to a very thin insufficient coat, thus, in either event, opening the pipe to the malign influences of air and surface water.

Conclusion 7 states that "decayed staves have been found all around the pipe." This would seem to indicate that, notwithstanding all the refinements of inspection and selection, it is impossible to secure staves of absolutely uniform character. To obtain such a result would involve so much inspection and so much culling that the cost would be prohibitive. In the Astoria line all sap edges were culled; nevertheless, it is clear that a slash-sawn stave taken from that part of a swamp-grown log immediately contiguous to the sap ring would not have the same life under light pressure as an adjoining stave cut from a hill-grown log and sawn from within a few inches of the heart ring. Or, any two staves taken from either of the above classes—or, for that matter, even from the sap-ring—and then dried, one in a kiln and one in the wind and sun, would have vastly different lengths of life, assuming always that they were under a very light pressure. If all the preceding classes of staves are put under a heavier pressure, then their lives will be found to be much more uniform, and certainly much longer than reported from Astoria. Moreover, under such conditions, the same care in their selection and time of drying is not imperative, nor is refined culling a necessity.

Conclusion 10 points out a noteworthy fact, which should be borne generally in mind. It states that staves bruised during the process of erection rot much sooner than unbruised staves. The principle here involved is in a manner much the same as a bruise on a creosoted pile, wherein an opening is thus furnished for the attacks of the teredo.

Conclusion 11, stating that decay was "confined to the outside

Mr. Ralston. of the pipe," reaffirms the theory of preservation due to saturation, and emphasizes the necessity for sufficient pressure to maintain saturation.

Mr. Herschel's comments have certain historical and literary merit; but they hardly reflect anything more. The conditions of pioneer engineering in the West are so forbidding to some New Englanders that they may well be allowed to felicitate themselves on being able to get away from wooden penstocks. Their determination, however, does not militate against wooden pipe, made from such material as redwood, Douglas fir or long-leaf yellow pine. This class of pipe has a distinct economic position, well recognized in the hydraulic fields of the West, and certainly in some parts of the East also.

Mr. Marx. C. D. Marx, M. Am. Soc. C. E. (by letter).—Through the courtesy of Mr. William T. Cannon, Superintendent of Power Stations for the Utah Light and Railroad Company, the writer is enabled to present some information relating to the upper end of the 6-ft. wooden stave pipe of that company at Ogden, Utah, in the Ogden Cañon.

In one short section of this pipe there has been some trouble due to dry rot, although the reports in relation to this have been greatly magnified.

In the latter part of November, 1906, upon shutting off the water at the head, in order to drain the pipe part way to repair a break lower down, it was found that the pipe had collapsed, during this operation, at two places just below the intake valve. Upon examination, it was found that the wood was almost rotted away at these points, there being some places only about $\frac{3}{8}$ or $\frac{1}{2}$ in. of sound wood left on the inner side of the original 2-in. staves. As the company was in great need of the power, only temporary repairs were made then, the question of replacing the defective parts being deferred until a more favorable time.

Immediately thereafter, the pipe was uncovered at some forty places along the line, but, excepting in the part extending about 700 ft. down from the intake, there was practically no deterioration. Below that point the wood was thoroughly water-soaked, and appeared to be as sound as when first put down. In the upper 700 ft., although some staves are still sound, many of them are rotted from $\frac{5}{8}$ to $\frac{1}{2}$ in. from the outside.

The trouble appears to be due to the fact that the low pressure does not force the water into the fibers of the wood, though this does not seem to account for the abrupt cessation of the rotting 700 ft. from the intake. The character of the soil may have been the cause, for where the trouble occurs the pipe tranverses a flat, while at the end of this bad section its starts along the hillside.

D. FARRAND HENRY, M. AM. SOC. C. E. (by letter).—About 1826 wooden logs for carrying water were first used in Detroit. Between that time and 1852, when the Water Board was formed by Act of Legislature, many miles were laid. Mr. Henry.

The pipes were of tamarack, 8 ft. long and about 8 in. in diameter, with 2¼-in. holes bored by hand. For ten years, under the direction of the Engineer, their use was discontinued, and iron pipes were put in. Then an inventor, with a machine for boring pump logs, cheapened the boring so much that the Board returned to their use.

From 1873 to 1878, the writer was Engineer, and persuaded the Board again to discontinue laying log pipes, there being then about 200 miles in use.

The subsoil in Detroit is generally a stiff blue clay with occasional pockets of sand and gravel. Where the wooden pipes were embedded in clay, the constant seepage under pressure kept them continually wet, and it seemed as though they would last indefinitely. The writer has taken them up after 45 years of use, and found them in perfect preservation; but, where they were laid in sand, in 5 years there was left only a shell of perhaps ½ in. around the bore, the remainder could be scraped off with a shovel. Tree roots also caused decay, by absorbing the moisture around the pipe. In one instance a linden had sent a root half way across the street to embrace the pipe. The writer has seen a section of a log, about 4 ft. long, entirely filled with a mass of fine roots which entered a wormhole and stopped the water flow. In fact, tree roots seemed to cause decay almost as rapidly as sand.

EDWIN DURYEA, JR., M. AM. SOC. C. E. (by letter).—The sewerage system of Palo Alto, Cal., was constructed during the spring and summer of 1899 under the supervision of C. E. Moore, M. Am. Soc. C. E. The outfall sewer, 12 in. in diameter, is proportioned to carry household sewage only, and, for about 1½ miles before reaching San Francisco Bay, is buried in saturated marsh lands, almost at bay level. Continuous wood stave pipe was used for this sewer in order to present more strength than earthenware pipe against unequal settlements. The sewer outfall is about 2 ft. below mean high tide, and at every high tide the pipe is covered, and the sewage is caused to back up and flow under some pressure. At extreme high tide the entire length of wood pipe is under some pressure, the greatest head at any point being about 6 ft. During low tide the sewer near its end is only partly filled with sewage. An inspection of pieces removed from the pipe shows that eight years of service as a sewer, with daily fluctuations from air to water, have caused not the slightest decay in the staves. The bands, however, are very badly corroded, and some of them Mr. Duryea.

Mr. Duryea. are actually rusted through so as to be entirely broken. The marsh land below Palo Alto is a soggy, adobe soil, heavily impregnated with salt, and presents the conditions most destructive to steel bands. The loose stave shown in the photograph, Plate XIX, is covered with more than 1 in. of barnacles, showing well the sea-water conditions to which it was subjected. The low head, the alternate exposures to air and water, and perhaps, also, the presence of sewage, would appear to be conditions very destructive to wood pipe. The wood, however, is entirely unaffected, both internally and externally, being clean and smooth; and the interior shows no signs of erosion.

The two sections of the pipe, the piece of stave and the bands, shown in Plate XIX, were removed by the City Engineer of Palo Alto, Mr. J. F. Byxbee, Jr. In a report to the writer (one of the City Trustees), Mr. Byxbee states:

"In one place, about 600 ft. from the outfall, there was evidence of sewage on the surface above the pipe, so I uncovered the pipe for a distance of 15 ft. in that location, and found a $\frac{1}{4}$-in. slit between two wood staves through which the leakage occurred. The only remains that I could find of iron bands at this point was a small piece about 6 in. long. In my opinion, the pipe is entirely without support of bands, or at best only feebly supported, throughout its whole length, and is held together merely by the pressure of the mire around it. * * *

"Of the two sections of wood pipe removed, one was taken from the covered portion of the pipe near its outfall and the other from an exposed point close to the end of the pipe. At high tide the pipe was covered and at low tide it was exposed to the air; air exposure seems to have had no particularly bad effect on the life of the wood during the years that it has been installed, probably due to its water-soaked condition. About half the bands on the piece of pipe from which the sections were taken had disappeared, the remaining hoops were very weak, and the pipe required careful handling to keep it from collapsing. The bands of the exposed portion of the pipe were stronger and less corroded than those where the pipe was buried."

In the spring of 1905 the writer removed another piece of wood stave pipe from its original position to serve as an exhibit. This piece of pipe, 3 or 4 ft. long, was sawed from a 30-in. continuous wood stave pipe, and had been in use for 15 years in a small inverted siphon forming part of an irrigating system within the city limits of Los Angeles, Cal. The pipe line from which the piece was cut was examined by the writer in several places which were uncovered, and this examination, and the more careful examination of the piece removed, showed that the destructive effect on the staves in its 15 years of use would indicate that it still had 30 years of useful life remaining. The bands were also af-

Sections of Wooden Stave Pipe, Bands, etc., from Outfall Sewer at Palo Alto, Cal.

Mr. Duryea.

fected, but had many years of life remaining. The conditions under which the pipe was used were not favorable to long life, the siphon being used only for a few months of each year. The head of water on it at the time it was sawed out was only 3 or 4 in. above the top of the pipe. The piece was removed from the lowest point of the siphon, the soil being damp, black, vegetable mould and the pipe covered by about 18 in. of soil.

The two pipes referred to, and one other, are all the examples of wood stave pipe which the writer has had personal opportunity to inspect. The third pipe is an outfall sewer of Stanford University, located near the Palo Alto outfall sewer, and of the same dimensions and age. Careful external examination in a stream-bed crossing shows it to be in good condition there.

These three instances, and some data which the writer has collected from original users of wood stave pipe, convince him that, while there may be conditions under which wood stave pipe will fail before it should be expected to, still, under many conditions, it is a very efficient form of pipe, and will fully justify its use. This is especially true if its relatively small cost is taken into consideration, its cost being approximately only one-half that of riveted pipe of fair thickness. This smaller cost and the consequent saving in fixed charges will go a long way toward paying for the replacement of wood stave pipe, even should its life be less than that of riveted pipe.

In reporting on improvements for the water supply of Bakersfield, Cal., about 18 months ago, after careful consideration, the writer had no hesitation in recommending spiral-wound, wood stave pipe for the largest (12-in.) mains; and he expects to repair the Palo Alto outfall sewer during the coming summer by rebanding the wood stave pipe, as he believes that under such conditions wood stave pipe, even though the bands have failed in eight years, is still a better form of construction than any other of practicable cost. In the rebanding an effort will be made to secure a longer life for the bands by a thicker coating, bands of greater diameter, or by galvanizing the bands before coating them with asphalt—or by a combination of all these methods.

Mr. Adams.

ARTHUR L. ADAMS, M. AM. SOC. C. E. (by letter).—Since submitting the paper the writer has had occasion to go over the entire conduit line at Astoria, of which the stave pipe in question constitutes a part. The results of his observations are in substance confirmatory of the representations made in the paper.

The writer welcomes such assurance as is given by Messrs. Baker, Henny, Hawley and Hatton that many of the important wooden stave pipe lines built in different parts of the country, from ten to twenty years ago, are still in excellent condition. Un-

Mr. Adams. favorable results in a single case, or in several cases, do not warrant indiscriminate condemnation. The important deduction to be made is that wooden stave pipe in common with other classes, under not unusual conditions, may prove short-lived; and that compliance with the usually accepted requirements, of being kept full, and under pressure, and buried in the earth, are not always sufficient.

Of course, there is much room for difference of opinion as to the relative degree of importance attaching to the different causes which seem to have contributed to the decay of this pipe. Some of this difference of opinion shown in the discussion may be reconciled by a little further explanation and statement of conditions. Some cannot be until a greater fund of knowledge concerning unfavorable experience with the pipe is available.

As shown by the profile, very few, if any, of these points of failure are located at summits. The decay was from the outside. For both of these reasons, air accumulations, if any, have not contributed to the result.

The profile shows that, save for three places near Station 390, the pipe at every point of failure was of necessity always full and under pressures varying from 20 to 50 ft., while, at the places designated, the attention of the pipe walker in partially closing a gate was necessary when the pipe was carrying less than full capacity.

Those parts of Section 1 and all of Section 2, which are under comparatively heavy pressure, and have as yet required no renewal, are laid in a soil consisting of fine sand and clay, clean and relatively free from vegetable matter. The writer has attributed the better condition of the pipe at these places to this fact rather than to the heavier pressure. A pressure of 50 ft., and even very much less, ought to insure saturation when the pipe is well buried. The writer thought that in projecting the original location in such a manner as to insure, save for an occasional sharp summit, a minimum pressure of about 20 ft., his course was conservative.

That the decayed portion of a great many staves was confined to 2 or 3 ft., the remainder being sound, answers the suggestion that the character of the grains, or the part of the tree from which the stave was cut, exercised controlling influence in the result.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1042.

THE NAVAL FLOATING DOCK—ITS ADVANTAGES, DESIGN AND CONSTRUCTION.*

By Leonard M. Cox, M. Am. Soc. C. E.

With Discussion by Messrs. George B. Rennie, J. R. Baterden, Cecil H. Peabody, C. Colson, A. C. Cunningham, Lyonel Clark, Edward Box, B. C. Laws, L. J. Le Conte, W. H. Pretty and Leonard M. Cox.

A nation's foreign commerce is a measure of its wealth. Foreign commerce is carried on by means of ships; and ships require harbor and docking facilities. Other conditions being equal, that port offering the best facilities for cleaning and repair work, in addition to deep water and protected berthing, will reap the largest measure of prosperity. The speed of a ship, for a given coal consumption, is a factor of its earning capacity, and marine growths foul a ship's bottom, affecting thereby its speed. Iron and steel corrode in salt water, and must be given protective coverings; these coverings last but a short time in active service, and require periodical renewals. Ships are likely to be damaged by grounding or by collisions, and access to their bottoms must be provided, in order to make the necessary repairs. In fact, from the very beginning of sea commerce, the question of repair docks has been of vital interest to merchants and ship owners as well as to governments, and, in view of the great movement toward commercial expansion, which has marked the last

* Presented at the meeting of December 5th, 1906.

decade, it is safe to assume that this interest will at least continue, and that a discussion of the subject will not fail to be of interest to an engineering society.

Historically, three epochs in ship-repairing appliances are marked: By the use of tide flats, when the Phœnicians careened their vessels on the shores of the Mediterranean; by the crude mud docks of the Greeks; and, by the (legendary) use by the North Country captain, of the hulk *Camel* as the forerunner of the floating dock. From these crude prototypes, the evolution of our modern structures followed. Increased demand and more exacting requirements have brought about improvements of design and of methods of operating; but, as regards general type and working principles, very little that is really new can be claimed.

Repair docks may be considered under four heads: The Graving Dock; the Floating Dock; the Marine Railway; and the Lift Dock.

The Graving Dock or Dry Dock.—The graving dock is an excavation in the foreshore of a harbor, cut off from the basin by portable gates or caissons. It may be lined with timber, stone, concrete, or combinations of these materials. After the ship is floated into the dock and centered over the blocking, the gates are closed, and the water is removed by pumps.

The Floating Dock.—The floating dock is a hollow structure, of wood, iron or steel, capable of being submerged by the simple admission of water to its interior, and of being raised to its lightest flotation again by means of pumps. When the dock is submerged, the ship is floated into position, the pumps are started, and the dock, with the ship on its deck, is raised until the ship's bottom is out of the water.

The Marine Railway.—The marine railway consists of inclined ways extending for some distance under the water. The ship is received on a cradle and hauled up on the shore.

Lift Docks.—Lift docks include those devices which consist essentially of a platform, capable of being raised and lowered by hydraulic or other power.

The limits of this paper confine consideration to the floating dock proper, but no discussion of one particular type of dock would be complete without some attempt to set forth the considerations governing the choice of types.

One of the results of many investigations prosecuted by intending builders is the apparent division of engineers and naval architects into dry dock and floating dock adherents. It is evident, however, that each type has its own particular field of usefulness which the other cannot with advantage fill, and that, for a given set of conditions, a careful study of both types, as applied to the special requirements of the case, must govern the choice.

For commercial purposes, the advantages of the stationary or basin dry dock consist in the fact that it can be used in shallow harbors, and that the ship, when seated, is safe. Its disadvantages lie in its greater first cost (except in the case of timber docks in favorable foundations); the greater quantity of water to be pumped; the lack of good ventilation and proper light under the ship's bottom; and the area of land required. The floating dock has the advantage of smaller first cost; no land space required; the ability to maintain the ship in virtually the same shape as when waterborne; and finally, its mobility. Its disadvantages are the possibility of accident (the chances of which, however remote, can never be entirely eliminated); the necessary occupation of useful water front; and the greater depth of water required for operation.

From the standpoint of naval requirements, the case is different. Sir William H. White, Hon. M. Am. Soc. C. E., in the discussion on dry docks,* at the International Engineering Congress, at St. Louis, in 1905, says:

"A modern ship, especially a modern warship, obviously requires careful handling in docking. An armored ship with hundreds of tons of armor on her sides, and of great width, is built with a bottom that is comparatively an egg-shell, and, unless properly supported, there are enormous risks in docking such a structure. It is quite easy to crush up the bottom of such a ship unless great care is taken. The case of the U. S. Cruiser, *Columbia,* when docked at Southampton, is well known. The spacing of the blocks was just such as would have been practiced with ordinary merchant ships, but it was not suitable to the light structure of the warship and the results were anything but satisfactory. The speaker has been in the double bottom of a large ironclad docked in a Government dockyard in England, where, owing to want of care, certain portions of the structure were pushed up in relation to others, and the light framework at the bottom was squeezed in a very uncomfortable manner."

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* *Transactions*, Am. Soc. C. E., Vol. LIV, Part F.

The modern warship has a delicate framework, of small local resistance, and, as it represents an outlay of several millions of dollars, the question of subjecting it to any sort of risk is one which calls for serious consideration. For docking ships of this character, the consensus of engineering opinion seems to favor the basin dock, because of its safety, its rigidity, and the smaller liability of error from the personal equations of operators. One of the objects of this paper is to show just how far the latest practice in floating dock construction has gone toward minimizing, or even eliminating, these objections, while developing the peculiar advantages of the type.

First Cost of Docks.—Ten years ago, or more, the timber dry dock was much favored in America on account of its small first cost and the short time required for construction. As far as naval purposes are concerned, however, the question of material for docks appears to have been settled in favor of masonry, since experience with timber docks established the fact that they were temporary structures at best, and that the small first cost was more than offset by the large charges against maintenance. As compared with the timber dry dock, the first cost of the floating dock is greater; but, as compared with the cost of the modern masonry dock capable of docking the same ships—and the policy of the Navy Department for the last ten years has been in favor of masonry docks—the floating dock costs about the same and in some instances less.

The first cost of masonry dry docks depends to a great extent upon the nature of the foundation encountered, and the necessary uncertainty of this factor often renders it impossible to prepare accurate preliminary estimates. The cost of a dry dock varies according to different localities, labor conditions, and prices of materials; and, in consequence, it is impossible to make a general comparison of cost between the floating and the stationary structures. Whenever such comparison is attempted, however, docks of essentially the same capacity should be selected. To arrive at an equitable comparative rating, regard must be had for the lifting capacity, the clear width of entrance, the available length, and the maximum draft of water over the blocks. A dry dock will support the heaviest ship that can be placed in its basin, while a floating dock will lift only the load for which it is designed. A floating dock has an available depth of water equal to the height of the side walls, less the height of the keel blocks, plus the required freeboard, while the depth of a dry dock is fixed by the depth of water over the sill. A floating dock can lift a ship whose keel length does not exceed the length of the blocking, whereas the capacity of a dry dock is limited by its own length. As regards clear breadth between walls, that dimension is usually the same throughout the length of the floating dock, while in dry docks the width at entrance must govern. In making comparisons, the lifting capacity of the floating dock may be disregarded, as its dimensions usually govern a ship's weight, and a dock of a certain lifting capacity would rarely be designed with such dimensions as would admit a ship of excessive weight.

The most important particular, however, is the matter of breadth. At present, the largest and heaviest battleships are of the new 16 000-ton type, 450 ft. in length, 76 ft. 10 in. in breadth and having a mean draft of 24 ft. 6 in. That the limit in size has not yet been reached is evident from the fact that even now tentative plans for 18 000-ton ships are being proposed, while even 20 000-ton ships have been advocated. It is reasonable, perhaps, to suppose that such increase in size will be accompanied by some increase of beam, and it is interesting to note that of the American naval dry docks now constructed, only four could take ships of these dimensions, and, of these four, those at Portsmouth and Boston have only recently been completed, while Dock No. 3 at the New York Navy Yard and the Puget Sound Navy Yard dock afford such narrow margin between the entrance walls and the ship that it is doubtful if constructors would care to undertake the risk involved in docking. When those now being constructed are completed, five docks, amply large for such ships and with a fair margin for future expansion, will be added to this list. On the whole, then, it is here proposed that, in comparisons of the cost of the two types, the floating dock be regarded as equivalent to a dry dock having a width at entrance the same as the clear width between its side walls, having a length exceeding its own by at least 100 ft., and having a maximum depth of water over the keel blocks equal to the maximum depth attainable in the floating dock. On this basis, the dock, *Dewey,* for instance, could be compared with Dock No. 3 at the Norfolk Yard, Dock No. 1 at Charleston (both under construction), and with the new Boston and Portsmouth docks. Table 1 gives the first cost of the naval docks of the United States.

TABLE 1.—First Cost of United States Naval Dry Docks.

Basin Docks.

Location.	Material.	Date of Completion.	Original cost.	Remarks.
Boston, No. 1	Masonry.	1833	$972 717.29	
Boston, No. 2	Masonry.	1905	1 105 665.27	
New York, No. 1	Masonry.	1851	2 003 498.05	
New York, No. 2	Timber.	1890	505 019.24	Rebuilt in concrete, 1900.
New York, No. 3	Timber.	1897	554 707.08	
New York, No. 4	Masonry and Concrete.	Under construction.	757 800.00	Body only.
League Island, No. 1	Timber.	1891	548 700.00	
League Island, No. 2	Concrete and Masonry.	Under construction.	1 301 111.76	
Norfolk, No. 1	Masonry.	1834	943 676.00	
Norfolk, No. 2	Timber.	1889	504 980.00	
Norfolk, No. 3	Concrete and Masonry.	Under construction.	876 776.00	Body only.
Port Royal, No. 1	Timber.	1895	449 437.09	
Mare Island, No. 1	Masonry.	1891	2 772 322.08	
Mare Island, No. 2	Concrete and Masonry.	Under construction.	1 385 000.00	
Puget Sound, No. 1	Timber.	1896	632 636.33	
Charleston, No. 1	Concrete and Masonry.	Under construction.	906 351.86	Body only.

Floating Docks.

Algiers	Steel.	1902	$809 713.52	
Dewey	Steel.	1905	1 143 959.68	

Note.—In the case of the Norfolk, No. 3, Charleston, No. 1, and New York, No. 4, docks, the bodies were let independently, the caissons, machinery, and pump wells to follow. Bids on the Charleston caisson were from $105 000 to $85 000; it was estimated, however, that the work could be done at a lower figure by a Navy Yard department, and therefore no contract was awarded.

The cost of the Mare Island and League Island new docks will very probably be increased before being placed in commission.

An important matter to be considered in the first cost of a floating dock is the preparation of the site to receive it. Expensive dredging may be required; and more or less permanent moorings and expensive shore connections must be provided. This work should properly be charged to first cost, in any comparison with the cost of stationary docks.

Time Required for Construction.—As regards the time required to construct, it would seem that the floating dock has a decided advantage. The granite dock at Portsmouth, N. H., required 6 years to complete; the granite dock at Boston, 6 years; and the docks at League Island and Mare Island, contracted for in 1899, are yet unfinished. The small granite dock at the Boston Navy Yard was commenced in 1827 and finished in 1833, and the small dock in the New York Navy Yard was under construction from 1841 to 1851. Of the Naval timber docks, No. 3, at the New York Navy Yard, required 5 years; the Port Royal dock, 4 years; and the Puget Sound dock, 4 years.

In a most excellent paper, read before the Institution of Civil Engineers, in August, 1905, by Mr. L. E. Clark, of the firm of Clark and Standfield, the time of construction of a number of floating docks designed by that firm is given; and this information has been used in Table 2:

TABLE 2.—DATA RELATING TO FLOATING DOCKS.

Dock.	Lifting capacity, in tons.	Weight of hull, in tons.	Date of completion.	Time required to build.
At Havana......	10 000	4 260	1897	11 months.
" Pola..........	15 000	5 200	1904	23 "
" Stettin	11 000	4 002	1897	7½ "
" Port Said....	3 000	1 514	1904	3 "

To which may be added:

Algiers...........	18 000	5 850	1902	27 months.
Dewey	20 000	9 200	1905	23 "

Of course, the time element, as well as every other factor which enters into the question, is largely dependent on local conditions; hence no true generalization can be made.

Maintenance Charges.—The granite or concrete dock should require virtually no maintenance, except that due to the ordinary wear and tear on the machinery, caisson, and auxiliary appliances. The floating dock, on the other hand, requires constant attention, including frequent self-docking, painting and cleaning. To the ordinary repairs must be added those due to accidents to the hull—an item which cannot be estimated in advance.

The expense of self-docking operations varies with the design and with the ability of the operator, but experience would indicate that it is not necessary to overhaul the bottom of steel docks at smaller intervals than from 5 to 8 years. The old idea that steel is

TABLE 3.—Data Relating to Repairs of Dry Docks.*

Location.	Period.	Dock.	Accessories.	Total.
Boston	1889–99	$4 158.25	$55 050.09	$59 208.34
New York, No. 1	1889–99	1 815.80	11 698.01	13 513.81
New York, No. 2	1890–99	In progress, $300 000 expenditure.		
New York, No. 3	1897–99	171 360.76	2 233.63	173 594.39
League Island	1891–99	70 721.38	7 874.39	78 595.77
Norfolk, No. 1	1889–99	260.20	29 912.73	30 172.93
Norfolk, No. 2	1889–99	54 010.77	39,432.79	93 443.56
Port Royal	1895–99	23 766.72	1 664.00	25 430.72
Mare Island	1890–99	4 363.62	19 912.63	24 276.25
Puget Sound	1897–99	328.12	11 025.00	11 353.12

* Annual Report of Chief of Bureau of Yards and Docks, 1899.

more liable to injury by corrosion in sea water than iron, seems to be losing ground; and authorities are not wanting for the statement that there is little, if any, difference in the deterioration of the two metals, if the steel has first been thoroughly cleansed of mill-scale. Indeed, no less an authority than Sir William H. White has given his experience as follows:

"If the manufacturer's scale (black oxide) is entirely removed, and equal care taken in protecting the surfaces with paint or composition, iron and steel have about the same rate of corrosion, the steel wearing somewhat more uniformly than the iron."

As a floating dock is not intended to show speed qualities, the bottom is ordinarily only cleaned for the purpose of protecting the plating. A heavy sea-growth, however, seems to afford the best protective covering that can be had, and there is a question as to the advisability of removing it until the increased weight becomes of appreciable importance. The experience of Assistant Naval Constructor W. G. DuBose, U. S. N., in charge of the self-docking of the Pensacola dock (Havana dock), is of great interest. In his official report of the operation, he says:

"The under-water plating of these pontoons was found to be completely covered to a depth of 8 in. or more with a growth of large oysters, barnacles and various shell growths. By weighing the foreign matter from various places on known areas, it was found that the average weight of shell growth on the bottom plating was 9 lb., and on the side plating 4 lb. per sq. ft. The total weight of shell growth removed from these three pontoons was 99 tons. In addition, it is estimated that about 30 tons of mud, scale and other dirt was removed from the inside of the pontoons. This growth formed an almost perfect protection for the plating, which was found to be in excellent condition throughout, with practically no corrosion. It is not believed that self-docking of the dock will again be necessary for at least five or six years."

Based on the experience of the contractors in the acceptance tests, the probable maintenance charges for the *Dewey* could be estimated as follows, assuming that it is thoroughly overhauled every fifth year:

Expense of self-docking	$10 000
Three coats of paint complete	27 000
Cleaning bottom	1 000
Cleaning interior	2 000
Maintenance of fixed equipment	5 000
Total	$45 000

For one year, $9 000, or 0.72% of the first cost.

The average cost of maintenance of nine docks, cited by Mr. Clark in the paper previously referred to, is 1.12% of the first cost. As Mr. Clark has done more to advance floating-dock design than any other one person, and, as he has had ample opportunity for observing a large number of docks built according to the designs of his firm, his experience must be regarded as of great value.

Operating Expenses.—The attendance required in docking a cruiser of the *Colorado* class in a stationary dock is slightly less than that for docking the same ship in a floating dock. On a floating dock, an engineer and fireman are required for each pumping element, besides one or two valve men, whereas the pumping-plant of a dry dock would require only one man—with the proper proportion of the expense of the central power plant. The quantity of water to be removed from a dry dock depends upon the size of the ship docked, and is greater for the small ship than for the large one; while, in the case of the floating dock, the pumping required is directly proportional to the weight lifted. On this account, the first cost of a pumping plant required to dock a maximum ship and a minimum ship in a specified time would be greater for the dry dock, and the coal consumption should also be greater. As a matter of fact, however, the coal consumption depends so much upon the operator that little benefit can be derived from comparisons.

As the most economical condition for pumping, in the case of a dry dock, is when docking the largest ship it is capable of holding, the following comparison of the quantity of water to be removed in both types is based on the U. S. S. *Colorado,* as docked by the ***Dewey,*** and the case of the same ship, if docked in Dry Dock No. 4 (now under construction) at the New York Navy Yard. As the *Colorado* is 504 ft. between perpendiculars, and as the dry dock in question is only 550 ft. on the floor, it will be seen that the dock prism is about as nearly filled as practicable.

Dry Dock No. 4, New York Navy Yard:

Total water contained in prism of dock.	58 700	tons.
Displacement of *Colorado* in docking trim.	13 500	"
Water to be removed in docking	45 200	"
Water to be removed to arrange blocks after docking	58 700	"
Total water to be removed	103 900	"

Floating Dry Dock Dewey:

Total water pumped from floating dock *Dewey* from 30 ft. over deck, raising *Colorado* to 2 ft. freeboard	28 700	tons.
Total weight of water removed to bring dock to 2 ft. freeboard after docking.	15 200	"
Total water pumped	43 900	"
Difference in favor of floating dock	60 000	"

Or, in other words, the comparison, under the most favorable circumstances to the dry dock, would indicate that the water pumped in the case of the floating structure would be about 42% of that from the stationary dock.

Location.—The cost of a dry dock depends upon the nature of the foundation. Good solid rock foundation is not obtainable in all localities; and when, as is the case at the New York Navy Yard, a soft bottom underlaid by a water-bearing stratum of fine sand is encountered, or, as at Bermuda, the problem of founding on coral is presented, the building of a suitable dry dock involves great expense and much uncertainty. On the other hand, the floating dock requires for its operation a depth of water considerably greater than the dry dock, and, if the proposed site has not this depth of water naturally, expensive dredging operations must be allowed for in the estimates. If the water is silt-depositing, the cost of maintaining the desired depth must also enter the final decision as a factor.

It has been pointed out that there is an advantage in warping a vessel into a floating dock lying parallel to the shore in a strong current. While this is very probably a real advantage, it is, in some cases, more than offset by the saving in valuable frontage by the use of a dry dock. Undoubtedly, there are times, however, when, for one reason or another, it is desirable to change the location of a yard, and, under such circumstances, the mobility of the floating dock might save the cost of the entire plant.

Safety.—As regards the safety of the ship docked, the dry dock has an advantage which at first glance appears to be of great moment, but which on closer study of service conditions would seem to be reduced to a minimum in the latest types of floating docks. It is true that when a ship is safely seated in a dry dock, there is small chance of accident from the structure itself. Indeed, if the dock is well founded, this risk may be regarded as limited to caisson accidents alone. A vessel on a floating dock, however, may be jeopardized in a number of ways, such, for instance, as inherent weakness of the structure itself, injury to the dock by collision, careless handling of valves, faulty moorings, etc. All these dangers have been the subject of careful study, and, in the latest designs, simplicity of operation and uniform strength with a large factor of safety have been sought, and, judging by the recent tests of the floating dock at Algiers and the floating dock, *Dewey,* have been attained to a very satisfying degree.

Commercial ship owners seem to prefer the floating dock, on account of its greater flexibility, which enables it to lift a ship in a condition approaching that in which it rests in the water. If a ship with a sag in the keel is placed on a rigid floor, as in a dry dock, it is deformed by the amount of the sag, and unusual strains are induced; while the flexibility of the floating dock would permit it to assume the shape of the keel, and, by proper manipulation of the valves, the ship can be given either a hog or a sag. The warship, on the other hand, because of its delicate structure, demands that a perfectly rigid base be provided in docking. As an indication of the approach to this requirement attained in practice, it may be stated that the U. S. S. *Colorado,* 14 500 tons, 504 ft. long, caused a maximum deflection, in the length of the *Dewey,* of $1\frac{1}{16}$ in., while the shorter battleship, *Iowa,* 11 400 tons, caused a maximum deflection in the same dock of 4 in.

Besides those mentioned above, other considerations often enter into the choice of types, among which may be mentioned the strategic value of mobility, possibilities of protection from the attacks of an enemy, possibilities of expansion for future demand, and facilities for affording light and air. The strategic value of mobility has been ably discussed by Civil Engineer A. C. Cunningham, U. S. N., in an article entitled "The Movable Base,"* from which the following is quoted:

"By placing a thoroughly developed military floating dock at each of our naval bases, we may in time of threatened danger double our available bases on the coast by towing the docks to various points of vantage, or in the last extremity if forced to retreat up our rivers and bays, we may take our floating docks with us and establish movable bases that will the sooner enable us to again reach our coasts."

As the dry dock is usually built in the foreshore of a protected harbor, and as the structure proper is below the level of the ground, it is better protected from the fire of an enemy than the floating dock with high towers forming conspicuous targets. A crippled ship would be safer in a dry dock in such a case than in a floating dock, since the latter would not be able to submerge without endangering its charge. The floating dock, however, could be towed farther up the harbor, and anchored in a place of greater safety, and this might more than offset the advantage of the stationary dock.

From a commercial standpoint, much might be said for the floating dock, because of the possibility of starting with a short dock designed so as to be fit for use as a section of a longer dock. If expansion of trade warrants, other sections may be added from time

* *Proceedings* of the Naval Institute, Vol. XXX, No. 1.

to time, and the capital invested in the plant can be kept at a figure more nearly proportional to the earning capacity.

In working under a ship, light is a matter of necessity, and in masonry docks, especially those with small side-wall batter, this object is not always obtained in a satisfactory manner. Paint will not dry quickly or thoroughly in poorly ventilated places, nor are workmen thus situated satisfied or efficient. In both these respects, the floating dock has a great advantage. The dry dock, however, has an equal advantage when it comes to handling material, heavy pieces of machinery, guns, etc. It is practically impossible to install traveling cranes of any considerable capacity on the narrow walls of a floating dock without interfering with the handling of the lines to such an extent as to render them undesirable. All material must come to a floating dock by bridge, wharf or float, and in neither case can the work be done as conveniently nor as economically as in the case of the dry dock, where cranes of from 40 to 100 tons capacity may encircle the excavation, and yard railways communicate with nearby shops.

There is, however, one particular in which the floating dock has a most important advantage over stationary docks, from a naval point of view. A dry dock can never dock a ship with a draft exceeding the greatest depth of water over the blocks, whereas a floating dock can be submerged to within a short distance of its side-wall decks. This advantage may be realized in comparing the *Dewey*, which can safely be given a depth of 37 ft. over 4-ft. keel blocks, while the side walls retain a freeboard of 4 ft., with the deepest naval dry dock (Norfolk dock No. 3, under construction), which will have a maximum depth of 32 ft. over the blocks. By docking on 3-ft. keel blocks, the depth of water on the *Dewey* could be increased to 38 ft. In an emergency such as the accidental disabling of a ship, so as to bring her down considerably by the bow or stern, this advantage of the floating dock might be the means of saving an investment of millions of dollars from total loss.

In passing from this part of the discussion, it may be worth while to draw attention to the fact that sewage can be easily and safely carried from a ship in a floating dock by pipes suspended beneath the ship's scuppers and led overboard through the side walls. This arrangement enables the ship's crew to be kept on board and under the supervision of officers, and adds greatly to favorable health conditions in tropical stations.

Types of Steel Floating Docks.—Very complete reviews of floating dock construction have recently appeared in the technical press and the *Transactions* of various engineering societies, among the most comprehensive of which may be mentioned that contained in "The Cavite Floating Dock"* by Civil Engineer A. C. Cunningham, U. S. N., and Mr. Clark's paper on floating docks previously referred to. It could hardly add to the knowledge of the subject to attempt such a review in this paper, but it may help to a freer discussion if a brief description of the designs marking successive steps in the evolution of the modern floating dock be given.

The earliest design consisted of a vessel of rounded form approximating the shape of a ship. This vessel was permanently closed at one end and fitted with a movable gate at the other, and was operated like an ordinary stationary dock. The gate-dock gave way to the round-bottom dock, depending for its lifting power on the buoyancy of its interior compartments alone, and self-docked by careening. The old Bermuda dock is of this type, and is still in use. The solid-trough dock next appeared, and is still much favored for wooden structures of small capacity. Steel docks of this type are sometimes built for fresh-water harbors, but when located in salt water there is usually included in the plant a shore basin for self-docking purposes. This type has many advantages in the way of strength, rigidity and low first cost, and, beyond a doubt, would be the ideal floating dock to-day if the lack of self-docking facilities could be remedied. Many devices have been proposed for getting at the under-water body of the solid dock, but, thus far, none has given sufficient promise of success to warrant adoption. A caisson arrangement for attachment to the bottom has been in use in Holland, but it is not known with what measure of success. The old Carthagena dock is an example of the solid-trough dock.

Sectional Docks.—In order to permit of self-docking, the ordinary sectional dock was evolved. It consists of a number of solid-trough sections, of such length that one could dock another, as shown in outline in Fig. 1. The different sections were loosely connected by timber toggles, and the operation of docking required skill

* *Journal*, Am. Soc. Naval Engineers, Vol. XV, No. 2.

and careful attention, in order that each section might take its own share of the load, and all lift in unison. Examples of this dock in timber are very common.

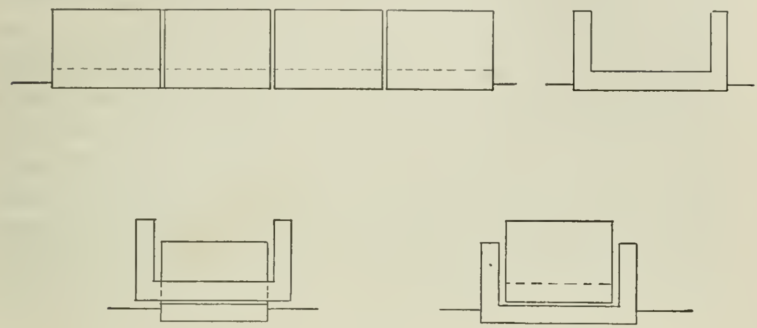


FIG. 1.

Rennie Type.—The type designed by Mr. G. B. Rennie, one of the pioneers in floating dock work, consisted of side walls in one piece, forming continuous girders for longitudinal strength, resting on a number of pontoons. This dock could be self-docked by disconnecting a pontoon, turning it so that its breadth lay parallel to the

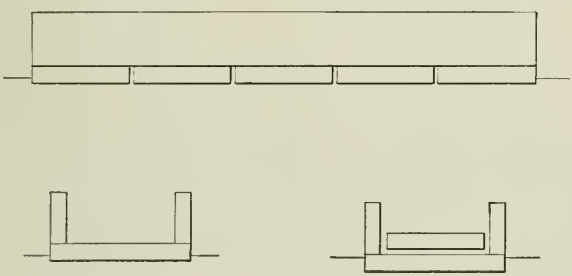


FIG. 2.

axis of the dock, and hauling it over the pontoons remaining in position, as shown in Fig. 2. The 17 000-ton Bloom and Voss dock, at Hamburg, is an example of this type. Its chief objection consists in the fact that it is impracticable to place the pumps in the lowest position on account of the break between the side walls and the pontoons.

Clark and Standfield Type (Havana).—In this dock, Fig. 3, the side walls are in one piece, but, instead of resting on the pontoons, they extend to within a few feet of the full depth of the pontoons, and the latter lie between the former, and are connected thereto by joints made up of fish-plates and bolts. The number of pontoons is unlimited, and the operation of self-docking consists in disconnecting one pontoon at light draft, submerging the dock, bolting the loose pontoon to a second connection diaphragm higher up on the side wall than the regular connection, and pumping to the desired elevation in the regular manner. The side walls are docked by careening. The Havana dock—now Dock No. 2 at the Pensacola Navy Yard—and the new dock at Algiers, La., built by the Maryland Steel Company, are examples of this type. The self-docking operation is entirely practicable, but tedious and rather complicated.

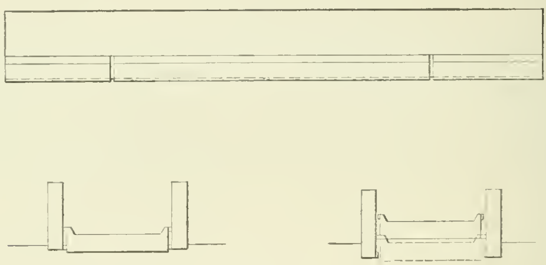


Fig. 3.

The greatest drawback to this type of dock is the difficulty of making it sufficiently rigid to meet naval requirements, the expense of construction, and the inconvenience of self-docking. During the acceptance tests of the Algiers dock, the self-docking was successfully accomplished in 40 days, the pontoons being lifted 4 ft., and the side walls 21 in. above the water.

Clark and Standfield Pola Type.—This dock, Fig. 4, is sectional, and each section is a solid trough with side walls and pontoons in one piece. The connections between the sections are made by bolts through flange angles which run around the edges of the transverse faces. To make the joint water-tight, rubber gaskets are placed between the flanges. The method of disconnecting, as described by the inventor, consists in removing the upper bolts while the lower bolts remain in place; the outer ends of the pontoons are then water-ballasted, causing the sections to press together at the bottom, and thus enabling workmen to enter and remove the remaining bolts in the dry. For self-docking purposes, the side walls are cut away for a short distance on the bow and stern sections, and these portions of the pontoon deck are drawn under the third section, as shown in the sketch. It is claimed by the inventor that, as each section is built solid, and they are connected all around their contact edges, it is the strongest self-docking dock—a claim which probably held good at the time it was invented, though, in the matter of strength, it could hardly be compared with the *Dewey*, of later date. Whether or not the Pola dock, the only one thus far constructed on this plan, has been self-docked is not now known, but it would certainly appear to be a difficult matter to keep the connection chambers free from water; and the removal of bolts might be attended with some danger.

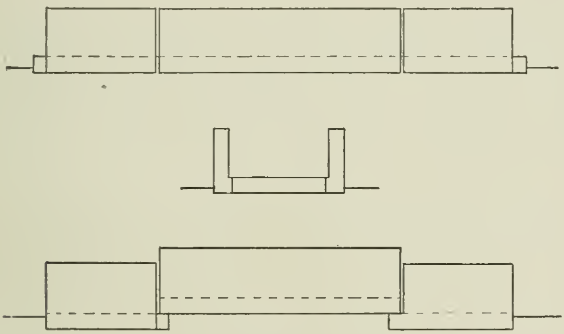


Fig. 4.

Holding the docked pontoon on the narrow ledge of deck available, would be out of the question in anything but perfectly still water, and, under the most favorable conditions, self-docking would require very skilful handling.

The Cunningham Sectional Dock.—This dock, Fig. 5, developed and patented by Civil Engineer A. C. Cunningham, U. S. Navy, consists of solid-trough sections joined together by fish-plates and bolts. The self-docking is effected by connecting two of the sections to the third by diaphragms, placed for the purpose at a higher elevation than those ordinarily used. This dock claims the advantage of having exactly similar elements, thus permitting expansion at any time by the addition of sections. The connections can be made of sufficient strength for naval requirements, and the self-docking operation is a very simple matter. This type should, and doubtless will, prove especially attractive for commercial purposes.

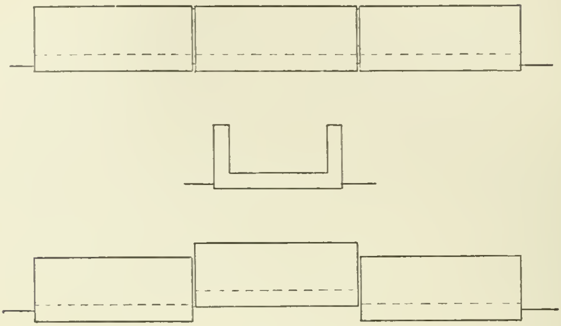


FIG. 5.

Maryland Steel Company Type.—This type of dock, originated and patented by Mr. Henrik F. Hansson, consists of a main pontoon of solid-trough section, with side walls extending beyond its length and bearing directly on the decks of the two shorter end pontoons.

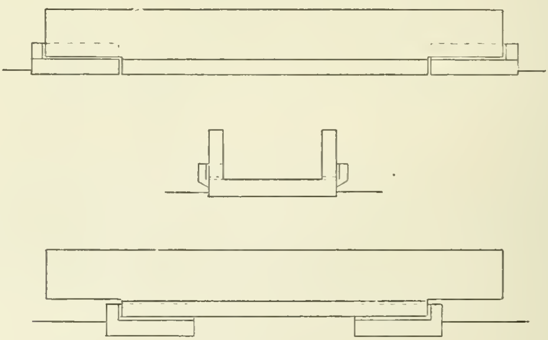


FIG. 6.

The end pontoons have low independent side walls, to afford stability in self-docking operations, and contain independent pumping plants. In self-docking, the center pontoon takes both end pontoons on the blocks in the regular manner, and, to dock the center pontoon, the bow and stern pontoons are drawn under its ends, as shown in Fig. 6. This arrangement, with its long solid main section, gives nearly the strength and rigidity of the solid-trough dock, and is the simplest of all types in its self-docking operations.

DESIGN OF A FLOATING DOCK.

Instead of discussing in general the principles governing the design of a floating dock, it is believed that the example of a dock actually constructed will prove of greater value in forming the basis for discussion of the subject, and, with this in view, a rather detailed account of the inception, design and construction of the U. S. floating dock *Dewey*, intended for the Philippine service, will be given.

Choice of Dock.—In 1902, the date of the act authorizing the construction of the dock, the Philippines had been in the possession of the United States for five years, and the necessity of maintaining a fleet in Philippine waters had been recognized for some time. The old Spanish naval station at Cavite was badly located, and Congress hesitated to authorize further expenditure at this place until the question of a site for a permanent station had been settled. The need for docking facilities in the Philippines was urgent, as without them a voyage to Hongkong, Kobe or Yokohama was a periodical necessity for every ship of the squadron. If a large investment were to be tied up in docking facilities, it was desirable to obtain a structure which could be moved, should future developments render such a step advisable. Again, the Department's experience with masonry dry docks established the fact that from 5 to 7 years are required for construction here at home, where labor conditions, prices of material, etc., are known, and the 18 000-ton floating dock at Algiers was constructed in 27 months from the date of awarding the contract. These considerations alone would have been sufficient to have caused the adoption of the floating dock, but other features of the type were so happily adapted to the requirements of the case that no other decision could have been wisely reached. Not least among them was the fact that a floating dock could be equipped with a small machine shop, and in a location where there are no shops, contain within itself an outfit for making all minor repairs.

The Specification.—The Bureau of Yards and Docks, in whose province lay the carrying out of the terms of the act, had just built the floating dock at Algiers, La., a structure which represented the highest progress then attained in dock building, and, with the idea of obtaining the very best results, decided to leave the design to open competition under a general specification. This specification was drawn up by Civil Engineer A. C. Cunningham, U. S. Navy, whose experience as officer in charge of the Algiers dock gave him a peculiar fitness for the task. The result was a specification, acknowledged to be the most complete ever written for a structure of this kind. Paragraphs setting forth requirements, the knowledge of which is necessary to a complete understanding of the detailed design, are given herewith.

Extracts from the Specification for the Cavite Self-Docking Steel Floating Dry Dock.

"1. *Intention.*—It is the declared and acknowledged intention and meaning to provide and secure a complete and substantial self-docking floating steel dry dock of American manufacture suitable for docking all of the present and projected ships of the United States Navy, for which appropriations have been made, located and installed in complete and perfect working order, together with all moorings, wharves, approaches, accessories, and appurtenances necessary for its perfect, complete, and convenient operation and maintenance, to the entire satisfaction of the Chief of Bureau of Yards and Docks.

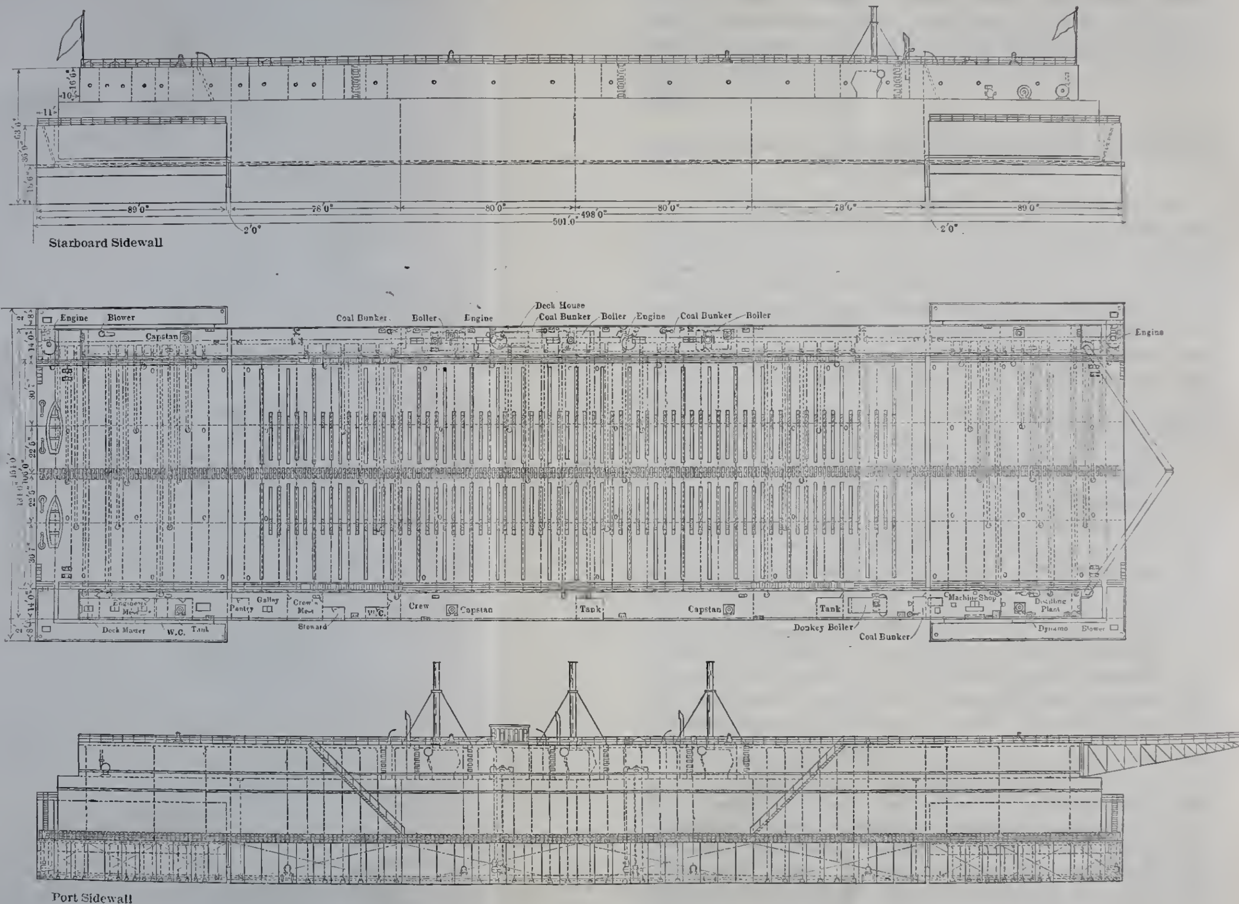
"9. *Time of completion.*—The dock proper shall be entirely completed and ready for test in every respect and particular within twenty-seven calendar months from the date of the contract.

"23. *Plans and specification.*—General plans in sufficient detail to give the Bureau a perfect understanding of the design of the dock, its method of operation, manipulation, and construction, and of the character and distribution of all material, machinery, and appliances shall be furnished by bidders. The general plans shall be accompanied by a specification, stress diagrams, and calculations in further amplification of the design and its capabilities, with full explanations of all operations and manipulations. The character and make of all machinery and appliances shall be described in the specification, and all other details necessary to enable the Bureau to arrive at a correct and perfect understanding of what is proposed. The contractor shall furnish the Bureau with tracings of all general plans.

"24. *Detail plans.*—The contractor shall prepare detail plans in amplification of general plans showing all parts of the dock and its appliances. Triplicate blue prints of these plans shall be submitted to the officer in charge for examination and approval before any work is performed. Tracings of approved plans shall be furnished to the Bureau. Approval of detail plans shall be of a general nature and shall not relieve the contractor from errors, discrepancies, or omissions that may occur therein which shall be remedied or supplied whenever discovered or required.

PROPOSED SELF-DOCKING STEEL FLOATING DOCK OF 16,000 TONS CAPACITY

FOR U. S. NAVAL STATION, CAVITE, P. I.



GENERAL ARRANGEMENT

"*31.*—The location of the dock is at the Naval Station, Cavite, P. I., at a site to be selected, but the Government reserves the right to reasonably vary the location as may be to its best interests before the final acceptance of the dock. The contractor shall provide moorings, approaches, and other necessary accessories and appurtenances suitable for the location finally selected by the Government.

"*32. General description.*—The dock in general shall be an open-hearth steel structure, so designed and arranged as to be readily self-docking without the aid of divers or auxiliary constructions. It shall be self-contained as to operating machinery, and capable of being towed from place to place safely without auxiliary bracing. It shall be of the general type composed of water-tight side walls and body, or pontoons, with a general ⊔-shaped cross section, and divided into sufficient water-tight compartments to give great stability, there being not less than 6 transversely. Simplicity and certainty of operation and freedom from possible disablement in all operations should be given careful consideration by designers.

"*33. Length.*—The dock shall be not less than 500 feet long over all, none of which length shall consist of bracketed platforms without lifting power.

"*34. Width.*—The dock shall have a clear width between fenders of not less than 100 feet.

"*35. Height and draft.*—The decks of side walls shall have not less than 8 feet of clear height above the water, with 30 feet draft over 4-foot keel blocks.

"*36. Lifting capacity.*—The dock shall have a lifting capacity of not less than 16 000 gross tons, uniformly distributed over its entire length, with the main deck not less than 2 feet above the water and with not less than 1 foot of water in the compartments.

"*37. Unit stress.*—No portion of the dock, or its connections, shall have a stress of more than 10 000 pounds per square inch under the specified loads, or of 15 000 pounds per square inch in self-docking, with a wind pressure of 30 pounds per square foot of exposed surface.

"*38. Shiploads.*—The dock shall be designed to dock all classes of vessels of the United States Navy, either centrally or with the center line of the keel 1 foot off the center line of the dock, with a free board of 2 feet, and shall provide for bearing over the full length of dock. Diagrams of weights, as far as available, will be furnished on application.

"*39. Distribution of load.*—The dock shall be so designed that the entire weight of a battle ship may be safely carried by the main keel blocks, or one-half the weight on each line of docking keel blocks, in whichever position the ship may be docked. The side walls shall be designed to take shoring at any point that may be necessary.

"*40. Working deck.*—The working deck of the dock shall be flush-plated and so strengthened that docking keel blocks may be placed in any position.

"*41. Uniform pumping.*—The dock shall be so designed that the specified unit stress shall not be exceeded when the dock is pumped uniformly from all compartments to a free board of 2 feet with any specified shipload docked centrally.

"*42. Allowable deflection.*—With any specified shipload docked centrally and all compartments pumped uniformly until the dock has a free board of 2 feet, the longitudinal and lateral deflection over the entire working deck of the dock shall not exceed 1 in 2 000. Within the limits of allowed deflection the shipload shall be assumed to be perfectly flexible.

"*43. Keel blocks.*—All keel blocks shall be of clear heart oak, of a uniform length of 5 feet, a width of 16 inches, and planed to a uniform thickness of 12 inches so as to be interchangeable.

"*44. Spacing of blocks.*—Main keel blocks shall be spaced 2 feet on centers and docking keel blocks 4 feet on centers.

"*45. Block sills.*—Docking keel blocks shall rest on sills of clear heart long-leaf yellow pine, 16 inches wide and planed to a uniform thickness of 12 inches so as to be interchangeable. The sills shall be of sufficient length to accommodate all the docking keels in the Navy.

"*46. Sliding blocks.*—Every third sill shall be fitted with a sliding block, and shall extend to within 2 feet of the main blocking and sufficiently outboard so that the blocks can not be hauled off the sill.

"*47. Drainage.*—Athwart-ship and fore-and-aft drainage shall be provided on the working deck of the dock.

"*48. Side wall decks.*—The decks of side walls shall have a clear passage fore and aft of not less than 5 feet in width. They shall have a hand rail on the outboard side and a 12 by 16-inch clear heart yellow pine timber on the inside fitted with fair leads and cleats.

"*49. Passage.*—Passage from one side wall to the other shall be provided.

"*50. Communication.*—Telephone or speaking tube communication shall be provided from one side wall of the dock to the other, along the side walls, and to the engine rooms.

"*51. Headline.*—Provision shall be made for a central headline and for hauling the same from side walls.

"*52. Capstans, winches, and bitts.*—There shall be not less than four capstans on each side wall and the necessary capstans or winches for handling moorings. There shall be not less than eight bitts on each side wall.

"53. *Moorings.*—Two sets of moorings shall be provided at each corner of the dock.

"54. *Runways and shoring stages.*—Two lines of runway and shoring stages not less than 36 inches wide shall be provided on the inner side of each side wall and a runway about 2 feet above the main deck.

"55. *Ladders and steps.*—Access shall be had to the runways and shoring stages by suitable ladders and steps from the top of side walls and main deck.

"56. *Fenders.*—All parts of the dock liable to be fouled by a ship in docking shall be fitted with heavy rubbing timbers and fenders, so arranged as to not injure the dock if carried away and to be readily replaceable. The exterior of the dock shall be fitted with fenders and rubbing timbers as a protection from drift and fouling.

"57. *Fire service.*—A fire service and washing-down system shall be provided the entire length of each side wall at or near the top, and with not less than four hose connections on each side.

"58. *Indicator system.*—The dock shall be fitted with a reliable pneumatic or hydraulic indicator system to show the depth of water in all compartments at all times.

"59. *Levels and gauges.*—The dock shall be fitted with levels and gauge boards to indicate the trim.

"60. *Height of self-docking.*—When self-docked, all under-water portions shall be raised to a clear height of not less than 5 feet, and shall be safely and readily accessible for inspection, painting, and repairs.

"61. *Self-docking connections.*—With the dock at light-draft line, all self-docking and strain-transmission connections shall be above water.

"62. *Power.*—The dock shall be operated by steam power, and shall be fitted with all the necessary boilers, engines, pumps, feed-water heaters, steam separators, and other accessories desirable to make a first-class self-contained plant.

"63. *Boilers and engines.*—There shall be not less than 600 nominal horsepower of boilers and engines suitably distributed to give the best results. Simplicity and certainty of action and freedom from possible breakdown in operation are to be given the first consideration. Engines of a type and style which will produce the least vibration in the side walls are desired.

"64. *Main pumps.*—If of the centrifugal variety, the main pumps shall have a discharge of not less than 16 inches and an equivalent discharge for other varieties.

"65. *Piping.*—All piping shall be of ample size to supply the pumps at maximum speed, and so installed as to be readily accessible for repairs or renewal.

"66. *Valves.*—The piping and flow of water shall be completely controlled by a system of simple and durable bronze-mounted valves, of the wedge variety, of easy and certain operation. All valves shall be fitted with indicators.

"67. *Fuel and water.*—Storage shall be provided for fuel and fresh water sufficient for two complete successive dockings, of the maximum load.

"68. *Connections to ship.*—Provisions shall be made for supplying a ship in dock with water and for carrying off her waste water and sewage.

"69. *Machine shop.*—A small machine shop, suitable for light repairs to the dock, shall be installed in one side wall.

"70. *Storerooms and quarters.*—Such portions of the side walls above the engine decks as are not occupied by machinery shall be fitted as storerooms, and as quarters for the dock's officers and crew, with suitable mess arrangements.

"71. *Hatches, skylights, deadlights, and ladders.*—The side walls shall be fitted with all the necessary hatches, skylights, deadlights, ladders, and other conveniences necessary or desirable.

"72. *Lighting plant.*—An electric-light plant shall be installed on the dock for lighting all interior working and storage compartments and with connections for portable lights on the dock.

"73. *Ventilating system.*—A blower system shall be installed for ventilation of all working and storage spaces and quarters in the dock.

"74. *Time of operation.*—The dock shall be designed to lift a load of 16 000 gross tons, with a draft of 30 feet clear of the water, in 4 hours. Lighter loads of less draft shall be lifted in a correspondingly shorter time, and the pumps shall readily operate under a head of 35 feet. The time of operation will be reckoned from when the ship has taken the blocks and shores and pumping is commenced until the keel is out of water.

"75. *Place of tests.*—All docking and self-docking tests shall be made at a suitable and convenient place at or near the works of the contractor.

"76. *Preliminary tests.*—Preliminary tests in sinking and raising the dock and in operating all machinery shall be made by the contractor to satisfy the officer in charge that the dock is in perfect working order.

"77. *Cruiser test.*—The dock shall be tested in docking a cruiser furnished by the Government, centrally or off line, as specified.

"78. *Battle-ship test.*—The dock shall be tested in docking a battle ship furnished by the Government, centrally or off line, as specified.

"79. *Deflections.*—Observations shall be made for deflection and permanent set during the dockings, with specially designed instruments furnished by the contractor, which will become part of the dock's outfit. In determining the final deflection, allowance shall be made for permanent set and temperature deflection, and the blocking shall be straight.

"80. *Self-docking tests.*—The dock shall be completely self-docked upon the completion of the docking tests.

"81. *Board of tests.*—The tests shall be conducted by a board of naval officers, appointed by the Secretary of the Navy, one of whom shall be a line officer expert in steam engineering, one a naval constructor, and one a civil engineer. The dock will be carefully examined by the board and tested for all specified requirements.

"82. *Conduct of tests.*—In docking naval vessels the ships shall be maneuvered, entered, and placed in position in the dock by the commanding officer and the naval constructor of the board, according to naval practice, with tugs and labor furnished by the contractor. All preparations and manipulations of the dock in testing shall be conducted by the contractor to the satisfaction of the board. A mutual understanding and agreement shall be had by the board and contractor preceding docking tests, to prevent accidents to the ship or dock.

"83. *Duration of battle-ship test.*—On the last test the battle ship shall be carried centrally on the dock for 48 hours without the dock showing any undue signs of strain or fatigue.

"84. *Condition on delivery.*—If the dock is delivered by the contractor at Cavite, all machinery, valves, strain-transmission, and self-docking connections shall be put in perfect working order before the dock is turned over to the Government.

"85. *Docking material.*—All the necessary material and appliances needed in testing and operating the dock shall be supplied by the contractor and become part of the dock's outfit.

"86. *Dock equipment.*—The dock shall be provided with all conveniences for operation, manipulation, and self-docking.

"87. *Boats*—Two 20-foot metallic lifeboats with complete equipment shall be provided with the dock.

Materials.

"88. *Quality.*—All materials and workmanship shall be of the best quality of their respective kinds when the grade is not specifically mentioned, and the acceptance of same is understood and agreed to be subject to the approval of the officer in charge.

Structural Steel and Iron.

"*89.*—The steel for this work shall be made by the open-hearth process.

"*90. Structural steel.*—Structural steel shall have a maximum tensile strength of 55 000 to 65 000 pounds per square inch, an elastic limit of not less than one-half the maximum tensile strength, an elongation of not less than 23 per cent in 8 inches. It shall bend when cold 180 degrees around its thickness and when at or above a red heat 180 degrees flat, all without rupture on outside of bent portion.

"*91. Rivet steel.*—Rivet steel shall have a maximum tensile strength of 47 000 to 55 000 pounds per square inch, an elastic limit of not less than one-half the maximum tensile strength, an elongation of not less than 25 per cent in 8 inches. It shall bend when hot, cold, or quenched in water of 70° F. 180 degrees flat, all without rupture on outside of bent portion.

"*92. Steel castings.*—Steel castings, after annealing, shall have a maximum tensile strength of not less than 60 000 pounds per square inch, an elongation of not less than 15 per cent in 2 inches. They shall bend when cold 90 degrees around three times their thickness and when at or above a red heat 180 degrees flat, all without rupture on outside of bent portion.

"*93. Phosphorus and sulphur limits for steel.*—Acid open-hearth steel shall not contain more than eight one-hundredths of 1 per cent of phosphorus, and basic open-hearth not more than five-hundredths of 1 per cent. No steel shall contain more than five one-hundredths of 1 per cent of sulphur.

"*94. Identification of steel.*—All steel shall be stamped with its cast number.

"*95. Wrought iron.*—Wrought iron shall be tough, ductile, fibrous, and free from steel scrap. It shall have a maximum tensile strength of not less than 48 000 pounds per square inch, an elastic limit of not less than one-half the maximum tensile strength, an elongation of not less than 12 per cent in 8 inches. It shall bend when cold 90 degrees around three times its thickness and when at or above a red heat 180 degrees flat, all without rupture on outside of bent portion.

"*96. Iron castings.*—Iron castings shall be made of the best quality of tough gray foundry iron and shall have a maximum tensile strength of not less than 18 000 pounds per square inch.

"*97. Tests of iron and steel.*—All tests shall be selected by a Government inspector. For steel there shall be not less than one of each kind for determining the physical and chemical qualities of each cast, and for iron a sufficient number to determine the quality of the lot under inspection.

"*98. Variations.*—Rolled material shall not vary more than $2\frac{1}{2}$ per cent from section or weight ordered, except in the case of wide sheared plates, where standard variations will be allowed; castings shall be true to drawings.

"*99. Defects.*—All rolled and cast material shall be free from defects and imperfections and true to section.

"*100. Inspection of steel and iron.*—All steel and iron shall be tested and inspected by a Government inspector at the place of manufacture before shipment. Orders for steel and iron shall be marked "Inspection by Bureau of Yards and Docks, Navy Department," and triplicate copies shall be forwarded to the Bureau from the shop where the material is ordered in order that the inspection may be arranged, and one copy to the officer in charge for his information. Lists of material offered for inspection (with cast numbers in the case of steel) shall be furnished the inspector by the mill.

Workmanship.

"*101. General character.*—All members and parts shall be manufactured and finished in a neat and workmanlike manner in accordance with best American ship practice, and to the satisfaction of the officer in charge.

"*102. Details and connections.*—All details and connections shall be designed to develop the full strength of main members and for the greatest possible stiffness. The least number of parts consistent with the best results shall be employed in all details and connections.

"*103. Accessibility.*—All details and connections shall be readily accessible for inspection, painting, and repairs.

"*104. Calking.*—All water-tight work shall be carefully machine-calked as the work progresses and tested in detail as far as possible. The work shall be so designed that all calking edges are accessible in the finished dock.

"*105. Water-tightness.*—The completed dock shall be made perfectly water-tight before acceptance so that it may be held in any position, light or submerged, without settlement.

"*106. Access to interior.*—Easy access to all interior portions of the dock shall be provided.

"*107. Least thickness.*—No material of less than $\frac{3}{8}$ inch in thickness shall be used in any portion of the dock which is subject to stress.

"*108. Pitch of rivets.*—In water-tight work the pitch of rivets shall be such as to insure perfect water-tightness, but in no case shall it be less than three diameters, and in general not to exceed 6 inches.

"*109. Rivet holes.*—Rivet holes shall be smoothly punched $\frac{1}{16}$ inch larger than the rivet to be used. When work is assembled the rivet holes shall so closely coincide that the hot rivet shall enter without drifting. Slight mismatching of holes shall be corrected by reaming, provided a perfect hole can be obtained and a correspondingly larger rivet is used.

"*110. Rivets.*—Wherever practicable all rivets shall have button heads on both ends of an approved form, and a sufficient length of shank to completely fill the holes and form perfect concentric heads. Rivets in decks and wherever else necessary shall be countersunk.

"*111. Driving rivets.*—Rivets shall be carefully heated and machine-driven wherever possible, in such manner as to completely fill the holes and form perfect heads.

"*112. Defective rivets.*—Loose, burnt, or imperfect rivets shall be cut out and replaced with satisfactory ones.

"*113. Inspection of workmanship.*—All workmanship, machinery, and appliances shall be inspected at the place of manufacture by a Government inspector.

Painting.

"*114. Cleaning metal.*—Before painting, all metal shall be carefully cleaned of all loose scale, rust, grease, dirt, chips, and other foreign matter by hammering, scraping, and brushing with wire brushes.

"*115. Paint.*—The paint in general shall consist of red lead, white zinc, japan dryer, and raw linseed oil, mixed in proportions as directed, and applied in such manner and at such times as directed by the officer in charge.

"*116. Contact surfaces.*—Contact surfaces shall each receive one coat of paint before assembling.

"*117. Painting.*—The entire dock shall receive three coats of paint before launching, and shall be touched up wherever necessary on the completion of the tests. In addition to the general painting, the engine rooms, boiler rooms, and quarters shall be finished as directed in oil paint, ground cork, and cement floors.

"*118. Paint materials.*—All paint materials shall be of approved quality and delivered in original packages.

Proposals.

"*119. Certified check and bond.*—Each proposal must be accompanied by a certified check, payable to the Chief of the Bureau of Yards and Docks, for the sum of $25 000, as a guaranty that the bidder will execute the required contract within ten days after its delivery to him for that purpose, and give a bond (preferably that of a first-class security company) in a penal sum equal to 20 per cent of the contract price, conditioned upon the faithful performance of the contract. Checks of unsuccessful bidders will be returned immediately after the contract is awarded, and of the successful bidder upon the execution of the contract.

"*120. Form of proposals.*—Proposals and all exhibits, alternate plans, letters of explanation, circulars, and all other papers (except the certified check) which it is desired to have considered in connection therewith must be made in duplicate. Proposals shall be made upon the prescribed blanks furnished bidders, as follows:

> "*Item 1.*—Price for dock and appurtenances delivered and moored at Cavite ready for operation, and the time required for such delivery.
>
> "*Item 2.*—Price for dock, moorings, and appurtenances delivered at the works of the contractor ready for towing, and insured by the contractor for delivery at Cavite.
>
> "*Item 3.*—Price for the dock, moorings, and appurtenances delivered at the works of the contractor ready for towing, and without insurance.
>
> "*Item 4.*—Price for towing the dock from the works of the contractor and delivering and mooring at Cavite.
>
> "*Item 5.*—In case the bids on any of the above items exceed the appropriation available, an opportunity is given under this item to submit alternate proposals for completing the work within the appropriation.

"*121. Acceptance and rejection of proposals.*—In giving great latitude in the design of the dock, both as to its general features and details, it is the desire of the Government to secure a structure that will fully meet the present and future needs of the naval service in a dock for the safe and convenient docking of all naval vessels, and for safe and convenient preservation and repair of itself; and the Government reserves the right to determine the relative merits of all the designs presented, irrespective of the price bid, and to accept the design which most fully meets its requirements, and to award the contract upon any of the above items, to accept any bid, to waive any defects and informalities in the proposals, and to reject any or all bids.

"*122. Bidder's ability.*—Before he is awarded the contract any bidder may be required to show that he has the necessary facilities, experience, and ability to perform the work in a satisfactory manner.

"*123. Amount of appropriation.*—The appropriation now available is $1 225 000.

"*124. Information.*—For any further information needed by intending bidders, application should be made to the Chief of the Bureau of Yards and Docks.

"NAVY DEPARTMENT,

"BUREAU OF YARDS AND DOCKS, *December, 1902.*"

A careful study of the designs submitted resulted in the award of the contract to the Maryland Steel Company, of Sparrow's Point, Md. The dock proposed has already been described under the name "Maryland Steel Company Type," patented by Henrik F. Hansson, and shown in outline sketch in Fig. 6.

General Dimensions.—Paragraphs 33 to 36 of the general specification govern the length, clearance between side walls, minimum draft of water over keel blocks, minimum freeboard, and the lifting capacity. The type of dock adopted required no altar at the junction of the side wall and deck (which was necessary in the Algiers dock for the pontoon connections) and valuable space was gained by omitting that objectionable feature. A side-wall width of 14 ft. was assumed for strength, space and stability considerations. The width of the painting stages was taken at 3 ft., to the outside of the fender, so that the over-all breadth of the dock would be 134 ft.

The skeleton structure of the dock is the result of the assumption that the local ship load is to be transmitted by the transverse members to the side walls, which, extending the full length of the dock, act as longitudinal girders. The ship rests directly upon keel blocks spaced at 2-ft. centers, and sills for bilge blocks are spaced at 4-ft. centers. As it is desirable to have the load placed symmetrically as regards the transverse girders, the position of the bilge sills limits the advantageous girder spacing to 8 ft., since a 4-ft. spacing would require too many, and, for a 12-ft. spacing, the necessary strength could not be obtained.

For preliminary calculations, the load curve of the new 16 000-ton ships was assumed to be made up of a uniform load of 30 tons per lin. ft. over 50 ft. of the keel length at each end, and 50 tons per lin. ft. for the remainder of its length. On this assumption, the loads on the transverse girders, spaced at 8-ft. centers, due to the ship, amount to 400 tons. This load may be considered as applied equally at three points, two points, or one point, according as the ship is assumed to be supported by keel and bilge blocks equally, the two rows of bilge blocks, or by keel blocks only. The negative loading due to the buoyancy amounts to:

$$\frac{16\,000}{500} \times 8 \times \frac{106}{134} = 202 \text{ tons, uniformly distributed.}$$

The transverse girders are fixed at the ends to the side walls; but, on account of the deformity of the structure as a whole, under stress, the span is taken from the centers of the side-walls, or at 120 ft., and the girder is considered as simply supported; therefore:

The maximum bending moment, ship resting on three blocks, is .... 5 682 ft-tons.
The maximum bending moment, ship resting on two blocks, is .... 4 216 " "
The maximum bending moment, ship resting on one block, is .... 8 616 " "

from which, using the specified unit stress of 4.46 tons per sq. in., the moment of resistance required is 15 290 inch-thirds for the first

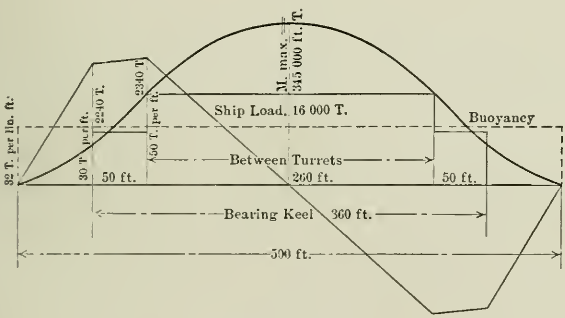


FIG. 7.

case, 11 340 for the second and 23 184 for the third. As the case of a large ship supported by the keel blocks alone would be impossible in actual practice, the required resistance moment in the first case above caused the tentative adoption of 18 ft. as the depth of the pontoons. This, however, was increased to 18 ft. 6 in. as a result of later and more detailed calculations.

Assuming the weight of the dock and machinery to be 11 000 tons, the weight of the dock and the ship is 27 000 tons, to which must be added the weight of 12 in. of contained water, as provided in the specification, making a total of 28 900 tons when the specified ship is raised to 2 ft. freeboard. The displacement of the pontoons per foot may be taken at 1 900 tons. The depth of pontoons, less the 2 ft. specified freeboard, is 16.5 ft., which at 1 900 tons per ft. gives a buoyancy of 31 350 tons, an ample margin above requirements.

To obtain the required moment of inertia of the dock section as a whole, the bending moment at the center is obtained (Fig. 7) as 345 000 ft-tons; from which the moment of resistance is found to be 928 250 inch-thirds. As it was found impracticable to obtain this resistance with the limiting side-wall requirements set forth in Paragraph 35 of the specifications, and for the further purpose of insuring rigidity, the side-wall freeboard was increased 3 ft. 6 in., making the total depth of the side walls 36 ft. 6 in.

The width of the independent side walls was taken at 8 ft., and increased to 10 ft. on the ends to provide working room for operating the independent pumping plant.

Hull Details.

Skin Plating.—The displacement per foot, based on the above general dimensions, is 1 900 tons for the pontoons and side walls to the level of the main deck. The displacement per foot of both side walls is 380 tons. When the dock is at its maximum submergence (34 ft. of water over deck) the displacement would be 48 480 tons, assuming that the independent side walls, except engine rooms, admit of a free flow of water. Taking the weight of the dock at 11 400 tons, and assuming that 12 in. of air remain in the pontoons, the water in the side walls will rise to 9 ft. above the deck, or 25 ft. below the water line. If no air is assumed to be left in the pontoon, the water will rise to 5.1 ft. in the side walls leaving 28.9 ft. head on the side-wall plating—a maximum head for side-wall plating. The maximum head on the bottom plating occurs when 12 in. of air is considered as remaining under the deck, and amounts to 35 ft.

All attempts to derive a rational formula for the stress in flat plates have resulted in complicated expressions too cumbersome for practical use, and the paucity of experimental data on plates of the size commonly used and under conditions approaching actual service has, so far, prevented the discovery of a satisfactory empirical formula. About all that is definitely known on the subject, at present, is that rectangular plates attached to frame angles by good riveting are subjected to a bending stress up to a certain point, after which the character of the stress changes to something like that which obtains in a loaded chain.

Experience in the use of plates under water loading would indicate that there is little danger of actual failure under a static loading due to the range of heads with which the designer has to do, and that the methods used in current practice afford an ample margin to cover all dynamic loading. It is here suggested that if ship-builders would adopt a maximum permissible deflection, expressed as a percentage of the shortest dimension of the unsupported rectangle of the skin plating, a satisfactory empirical formula might be derived from experiments on full-sized units, to which an impact factor could be added to provide for wave effect. Such a formula, if generally adopted, would not change present practice further than to provide uniformity of method.

To determine the thickness of the skin plating on the *Dewey*, an elementary beam, 1 in. in width, was considered as fixed at the ends and supported at the edges of frame angles. If K represents the allowed intensity of stress, H, the head of water, in feet, and l, the span, the expression for the thickness may be written:

$$t^2 = 0.0001 \frac{H l^2}{K}.$$

Using the dimensions shown in Fig. 8 for a beam 1 in. wide, having a thickness, t; and a span of 21 in., under a 35-ft. head of water:

$$\text{Then, } t = 0.586 \text{ in.} = \tfrac{19}{32} \text{ in.}$$

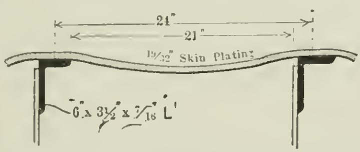
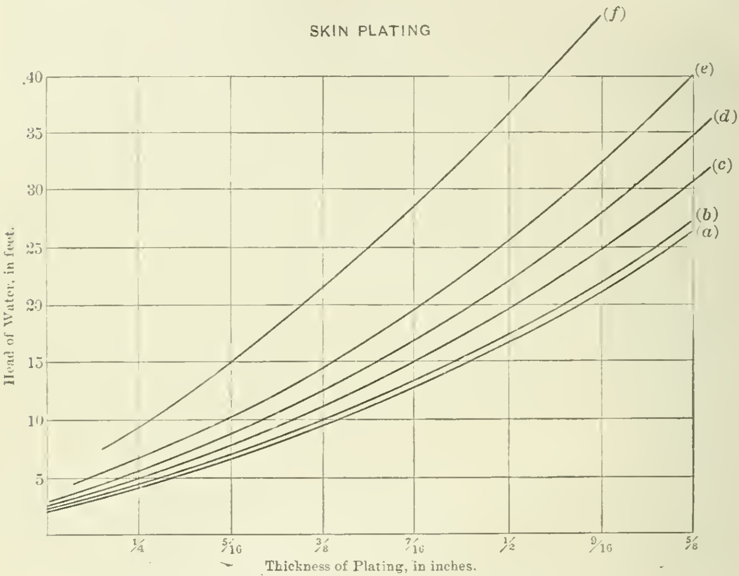


Fig. 8.

This method does not take into account the metal lost by punching, nor does it seem to be rational to take the span as the distance between the edges of the supports, because any deformation of the flanges will destroy the usefulness of the flexure formulas.

Experiments on full-sized plates, which were conducted during the construction of the dock (the results of which may be submitted to the profession at a later date), would indicate that the actual deflection is from one-third to one-fifth of that obtained by the fixed-beam formula, and that a considerable excess of strength is obtained by its use. It should be stated that these experiments were on plates supported only at their edges and not continuous over the supports. Therefore, they do not show the effect of continuity, nor do they indicate the effect of the ship's distortion as a whole.



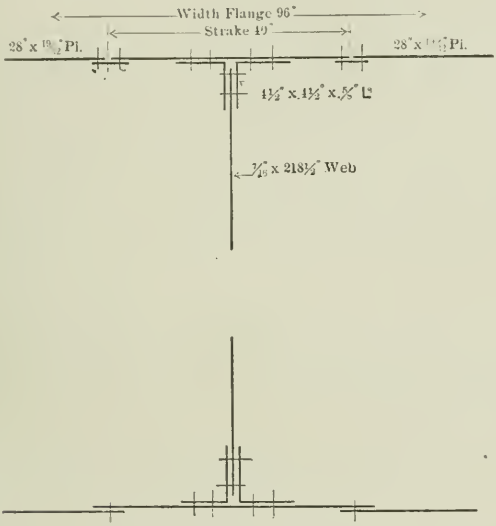
(*a*) Fixed beam formula, Span 21", ¾" rivets, 6" Spacing deducted (⅞" gross hole)
(*b*) " " " " " " " " " " (net rivet area)
(*c*) Rectangular plate formula. $K = wa^4b^2 \div 2\,t^2\,(a^4 + b^4)$
(*d*) " " " Bach, $t^2 = 0.468\,wa^2b^2 \div K\,(a^2 + b^2)$
(*e*) Fixed-beam formula, Span 21", no rivets deducted.
(*f*) Experimental,- deflection = 1/360 Span

Fig. 9.

Fig. 9 shows the curve of thickness required for different heads, as obtained by the fixed-beam formula, Bach's formula for rectangular plates, and other well-known formulas. As a matter of interest, a curve is added showing the thickness obtained by limiting the deflection to $\frac{1}{360}$ of the span, as determined by the experiments referred to above.

If the lap is considered as a beam supporting two edges of a rectangular section of plating, a slightly less degree of thickness is required by the use of Bach's formula. Taking the lap at $2\frac{1}{2}$ in. in width, and deducting rivet holes, it is found to be amply strong, with $\frac{9}{16}$-in. metal, to support its proportion of the load, but as the lap must suffer some distortion it would be hard to say just what dependence could be placed upon it.

Transverse Bulkheads.—The maximum bending moment to which the transverse girders are subjected is 5 690 ft-tons, as obtained above. The plating required for the static head is $\frac{19}{32}$ in., and the bulkheads are spaced at 8-ft. centers. As the longitudinal frames



TRANSVERSE BULKHEAD

FIG. 10.

keep the deck plating rigidly connected as a whole, the entire 96-in. section of skin plating may be considered as forming part of the girder flanges. This width will include part of three transverse strakes, and the weakest section will occur at the butt of one strake. The butts are assumed as double-riveted in the center with rivets of 3-in. pitch, and one strake may be taken at about 40 in., or as including about 22 rivets. Assuming the girder section shown in Fig. 10, then, allowing only the rivet equivalent for the area of the middle strake, and making a proper reduction for rivet holes in the angles.

gives a moment of resistance of 15 220 inch-fourths, as against 15 290 required with the allowed unit stress of 10 000 lb. per sq. in. At 11 ft. on each side of the center, it is desired to cut a central 15 by 30-in. limber hole; and, at a distance of 17 ft., two such holes are needed. The reduced moments of resistance at those points are found to be amply large for the corresponding load moments. The transverse bulkheads are continuous between the side-wall bulkheads, and they are stiffened by the longitudinal frames besides the center and intermediate longitudinal bulkheads.

To determine the deflection of the transverse bulkheads, the general equations for deflection are used.

Moment between O and A, $M\ (O - A) = R\,x$,

Moment between A and B, $M\ (A - B) = R\,x + \frac{1}{2}\,p\,(x - a)^2$

Moment between B and C, $M\ (B - C) = R\,x + \frac{1}{2}\,p\,(x - a)^2 - W\,(x - b)$.

Equating each of these expressions to $E\,I\,\frac{d^2\,w}{d\,x^2}$, integrating, and noting the conditions when a, b and d are substituted for x, the values of the constants are determined, and an expression for the maximum deflection is obtained. Substituting the proper load and span values in the deflection expression, the theoretical maximum deflection of the transverse bulkheads under the maximum loading is found to be 0.56 in., which is well inside the specified deflection of 1 in 2 000 (0.636 in.).

Longitudinal Frames.—A frame spacing of 24 in. was adopted on account of the low unit stresses allowed and the extreme rigidity required. On the assumption that the ship rests on the bilge blocks alone, the direct load on each block placed 4 ft. apart would be 100 tons. As the bilge blocks were specified to be 5 ft. in length and were to rest on 12 by 16-in. yellow pine sills, the loading may be considered as distributed between four longitudinal frames, making a direct concentrated load of 25 tons to each frame. Using the fixed-

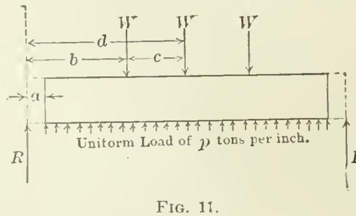


FIG. 11.

beam formula for concentrated loading, and taking the span as the distance between the centers of the rivet groups connecting the top member to the gussets, gives a required moment of resistance of

$$\frac{I}{d} = \frac{25 \times 6 \times 12}{8 \times 4.46} = 50.4.$$

Using a 15-in. by 33-lb. channel for the top chord of the frames, and regarding the effective section of the plating above it as a part of the beam, gives a section composed of the channel, and a 24 by $\frac{19}{32}$-in. cover-plate the area value of which is found by dividing the value of seven $\frac{3}{4}$-in. rivets for single shear by the allowed intensity of stress. This gives a moment of resistance of 48.7. To make up for the uncertainty as to the ends, and to insure rigidity, two knee-braces, composed of one $4\frac{1}{2}$ by 3 by $\frac{7}{16}$-in. angle, are added.

The bottoms of the frames sustain the maximum water pressure only, and, by the use of knee-braces, the span may be broken up into three sections, the longest of which may be taken at 28 in. Regarding each span as a series of fixed-end beams, non-continuous over the braces, gives for the required resistance moment with a 35-ft. head of water:

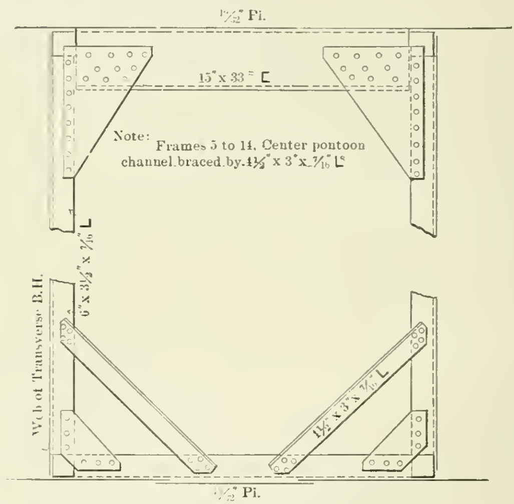
$$\frac{I}{d} = \left[\frac{H}{35} \times \frac{2}{12} \times \frac{2}{28}\right] \div 12 \times 4.46 = 4.$$

Using a 6 by $3\frac{1}{2}$ by $\frac{7}{16}$-in. angle, and considering the rivet equivalent of a 24-in. strip of plating, gives a section having a moment of resistance of 4.2.

The knee-braces carry the reactions from two 28-in. and one 18-in. span, or, $23 \times \frac{H}{35} \times \frac{2}{12} = 3.8$ tons. The braces are set at 45°, and are about 48 in. long, therefore the resultant stress is 5.4 tons. Using a $4\frac{1}{2}$ by 3 by $\frac{7}{16}$-in. angle, with an $\frac{l}{r}$ of 12, and a factor of safety of 4, the safe strength would be 3.5 tons per sq. in.—a total of 11 tons. The end spans of the bottom member would be subjected to a thrust, in addition to the bending, but as in that case the gross sectional area could be depended upon, its capacity is ample. The side angles of the frames are required to transmit the total upward thrust of the water to the transverse bulkheads, and a 6 by $3\frac{1}{2}$ by $\frac{7}{16}$-in. angle, having a capacity for 50 tons, is used, and is attached to the bulkhead by $\frac{3}{4}$-in. rivets spaced 6 in. apart.

The end frames of the pontoons are provided with vertical plate stiffeners, calculated in the usual way to withstand the greatest unbalanced water pressure to which they are exposed. The top chords of the last five frames next the side walls are reduced in size to 12-in. by 25-lb. channels, as they lie beyond the direct ship loads.

Side-Wall Frames.—As the side walls form the main longitudinal girders, the framing serves mainly as spreaders, and are placed athwartships, instead of fore and aft. At intervals of 8 ft., and abutting the pontoon transverse bulkheads, is a braced frame for the



ORDINARY FRAMES.

Fig. 12.

purpose of transmitting the pontoon loads. Between the braced frames, and spaced 2 ft. apart, are the ordinary box-frames stiffened by knee-braces and struts. The ordinary side-wall frames are calculated in the same manner as the pontoon frames, with the exception that their members are subjected to an upward thrust as well as to direct bending.

The condition of loading for the braced frames is shown in Fig. 13. The buoyancy of the dock beyond the ship load is the supporting force, and must be transmitted through the side walls, acting as girders, to the ends of transverse girders. The braces carry this supporting force of 100 tons from the side-wall skin to the connections.

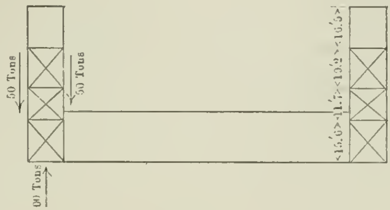


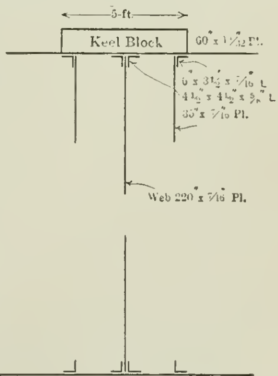
FIG. 13.

Assuming that 100 tons is divided between the verticals, and that each of the six braces takes equal proportions, the vertical component of the brace stress is 8.3 tons, and the resultant brace stress is:

For the top pair..........................	10.5 tons.
For the middle pair....................	13 "
For the lower pair......................	11.3 "

A 5 by 5 by ½-in. angle is adopted, having a safe capacity for 18 tons. The connection between the transverse girder and the side walls is calculated to withstand the shear and bending moment at that point.

Longitudinal Girders.—The central longitudinal bulkhead is intercostal between the transverse girders, and, together with Frame No. 1 on each side, forms a compound girder receiving the direct loading from the keel blocks. The span is 8 ft., and the girder is strengthened by an intermediate breathing-plate extending from the center to Frame No. 2 on each side. The keel blocks are 5 ft. long, and the ship load is regarded as distributed over that space. Considering the ship as supported on keel blocks alone, the load per span of 8 ft. is 400 tons, to be carried between transverse girders. The section of girder adopted and sketch plan showing the area of rivets counted is shown in Fig. 14.



CENTER LONGITUDINAL BULKHEAD

FIG. 14.

The resistance moment of the section is ample, of course, for the bending. To transmit the direct load into the girder, the rivets are distributed as follows:

	Double shear.	Equivalent single shear.
Center bulkhead..............	29	46
Transverse bulkhead...........	16	26
Breathing-plate	10	16
Two brackets.............................		20
Four curtain-plates.........................		52
Total single shear......................		160

$400 \div 160 = 2.5$ tons per rivet in single shear, or 12 800 lb. per sq. in.

Rivets counted in area between dotted lines.

FIG. 15.

This result is deemed satisfactory when it is considered that the assumption that the whole ship rests on the keel blocks alone is one which can never materialize.

The side and intermediate longitudinal bulkheads are made up, like the center, of $\frac{7}{16}$-in. plate and $4\frac{1}{2}$ by $4\frac{1}{2}$ by $\frac{1}{2}$-in. angles. The intermediates are strengthened by a channel rib placed horizontally at about one-third the depth to provide for unequal water pressure when listing the dock.

Other Hull Details.—All non-watertight bulkheads have 12 by 30-in. openings, to allow the passage of water, and are provided with small limber holes near the top and bottom flanges for air drainage.

All connection details are computed in the regular manner, care being taken to avoid direct pull on the rivet heads.

The bending moment at the joint between the pontoons, for both ship and dock loads, may be taken at 120 000 ft-tons. A section resistance moment of 323 000 is therefore required. Using the connections shown in Fig. 16, consisting of seven elements each comprising forty-four 2-in. bolts, and considering the overhang with bolts alone, a resistance moment of 730 000 inch-thirds is obtained. Using the overhang section and the section of the ¾-in. diaphragm plate, a moment of 613 400 inch-thirds gives nearly twice the required strength. In addition to the vertical connections, a row of 1½-in. bolts connects the end pontoon decks with the four bottom edges of the overhangs, and there are 96 bolts in each row.

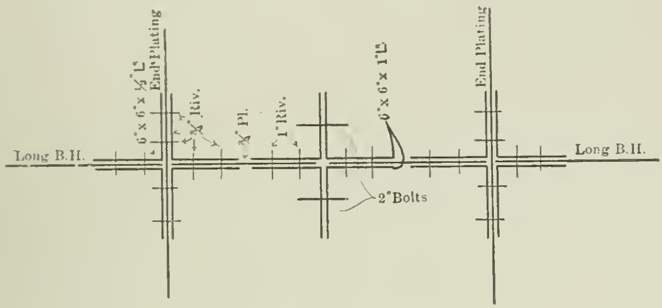


FIG. 16.

The strength of the overhang section when unsupported by the end pontoon may be determined by the following assumed weights and their location:

Weight of overhang............	320 tons,	arm 40 ft.
" " quarters.............	11 "	" 44 "
" " bridge...............	8 "	" 80 "
" " live load...........	20 "	" 40 "

Moment = 14 724 ft-tons, which, divided by the resistance, 102 346, gives 1.16 tons per sq. in. stress.

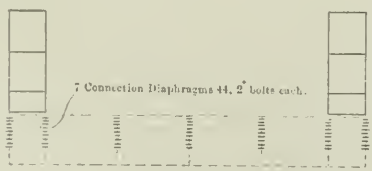


FIG. 17.

Strength of the Dock as a Whole.—The moment of inertia of the dock, according to the above preliminary design, was taken at different sections as follows:

Center	384 681 832	inch-fourths.
(*a*) 31.5 ft. from center	379 382 072	" "
(*b*) 39.6 " " "	371 764 400	" "
(*c*) 49.5 " " "	370 011 050	" "
(*d*) 56.25 " " "	368 218 500	" "
(*e*) 70. " " "	348 510 714	" "
(*f*) 110. " " "	321 140 084	" "
(*g*) 131.17 " " "	313 661 487	" "

(The center of gravity of the center section is 371.2 in. from the center of the bottom plating.)

A second bending-moment diagram was then drawn, using the actual load curve of a 16 000-ton battleship. A comparison of the required resistance with that provided above led to the doubling of certain of the tower deck plates, the addition of a channel stringer and a slight increase in the thickness of the upper side-wall plating. The final moments of inertia corresponding to the above sections are as follows:

At center	384 681 832
Section *a*	379 382 072
" *b*	375 872 940
" *c*	374 188 880
" *d*	372 456 168
" *e*	353 061 010
" *f*	326 317 300
" *g*	322 667 487

A curve was then drawn, with ordinates obtained by dividing the bending moment at various sections by the corresponding moment of inertia. Treating this curve, called the "$\frac{M}{I}$" curve, as a load curve, the bending moment obtained from it indicates the deflection of the dock. The calculation was made graphically, and resulted in a maximum estimated deflection of 2.9 in. at the center, slightly less than 3 in. allowed by the specification.

FIG. 1.—U. S. S. *Colorado* ENTERING DOCK.

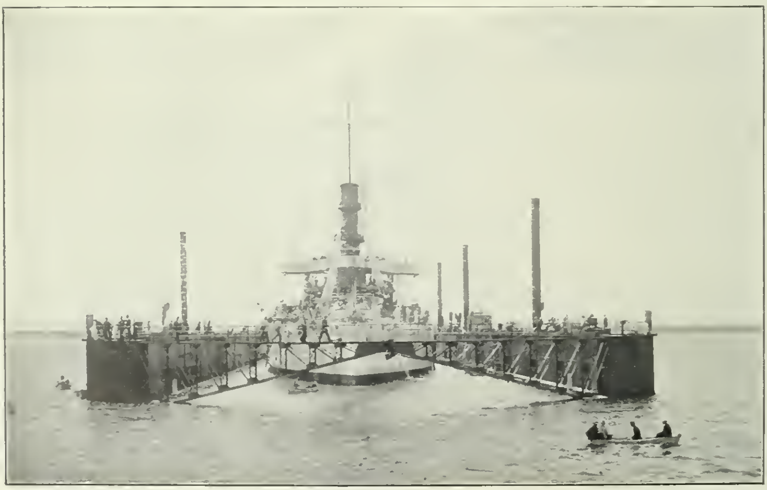


FIG. 2.—U. S. S. *Iowa* CENTERED IN DOCK: READY TO BEGIN PUMPING.

In investigations of the dock's strength on the crest or in the hollow of a wave of height equal to one-twentieth of the span, no bending moment can be obtained exceeding 200 000 ft-tons, which is small in comparison with that given by the ship. In self-docking the center pontoon, each end rests on an 80-ft. deck length of end pontoon, and considering the dock as a beam of span equal to the distance between the center of bearings, a bending moment of 231 000 ft-tons is obtained. This, however, is on the assumption that the weight of the center pontoon, 7 750 tons, is distributed over the span, a more severe condition than that which actually occurs as the overhangs tend to reduce the moment.

STABILITY.

Referring to Fig. 18:

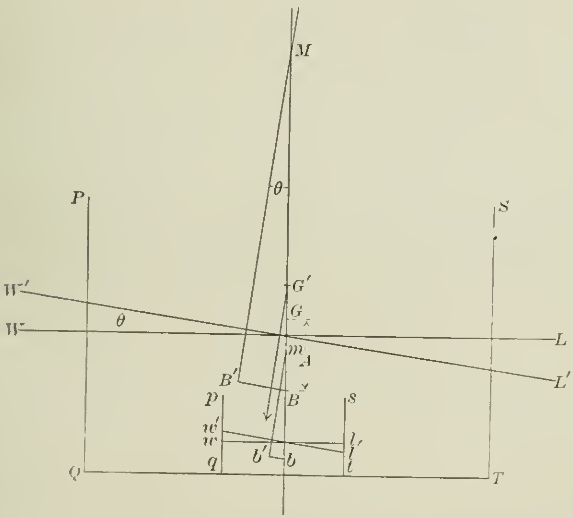


FIG. 18.

$PQST$ is a floating vessel;

$pqst$ is an interior compartment, partly filled with water;

WL and $W'L'$ are water lines, before and after inclination;

wl and $w'l'$ are interior water surfaces, before and after inclination;

B and B' are centers of buoyancy, before and after inclination;

b and b' are centers of gravity of interior water carenes, before and after inclination;

D is the displacement of the dock, in tons;

d is the weight of the contained water, in tons;

V is the value of the displacement of the dock, in cubic feet $= 35\ D$;

v is the value of the contained water, in cubic feet $= 35\ d$;

I is the transverse moment of inertia of the water planes;

i is the transverse moment of inertia of the contained water surface.

The metacentric radius, called R, is the distance, $B\ M$; and the metacentric height, called H, is the distance, $G\ M = R - B\ G$, or $R - A$. By a simple process, it is easily proven that $R = \frac{I}{V}$ when I is the transverse moment of inertia of the water plane and V is the volume of displacement. The same expression can be used for the water in a contained compartment of a floating body, in which case the center of buoyancy and the center of gravity are identical.

When a vessel rolls, the contained water moves from side to side and acts in all respects as if it were suspended from a point which corresponds with the metacenter, m, of an empty vessel. The effect of the contained water, therefore, may be said to raise the center of gravity a distance, $b\ m$, in Fig. 18, which would raise the center of gravity of the whole vessel a distance

$$G\ G' = \frac{d \times b\ m}{D}.$$

in which d = the weight of the contained water, in tons; and D = the displacement of the dock, in tons. Assuming 35 cu. ft. of sea water = 1 ton.

$$D = \frac{V}{35}\text{, and } d = \frac{v}{35}.$$

therefore,

$$G\ G' = \frac{v \times b\ m}{V} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

But, considering the interior compartment as an exterior carene,

$$b\ m = \frac{i}{v}.$$

therefore,

$$G\ G' = \frac{v}{V} \times \frac{i}{v} = \frac{i}{V} \ldots\ldots\ldots\ldots\ldots (2)$$

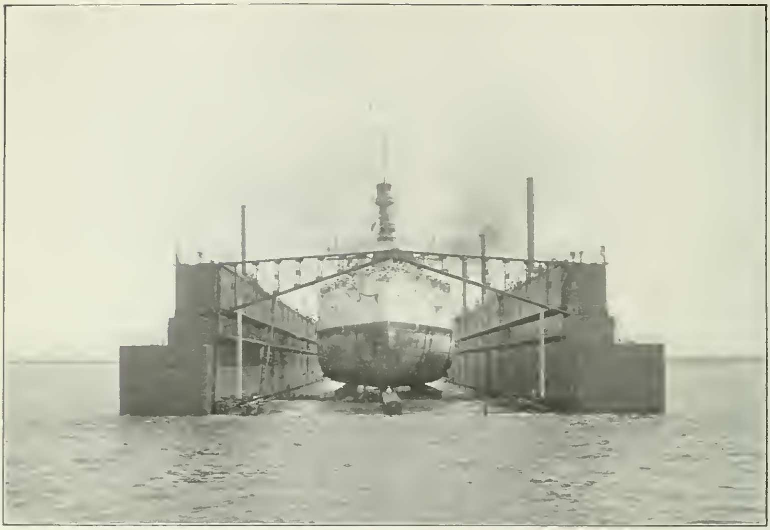


FIG. 1.—U. S. S. *Iowa* IN DOCK, WITH PONTOON DECK OF DOCK AWASH.

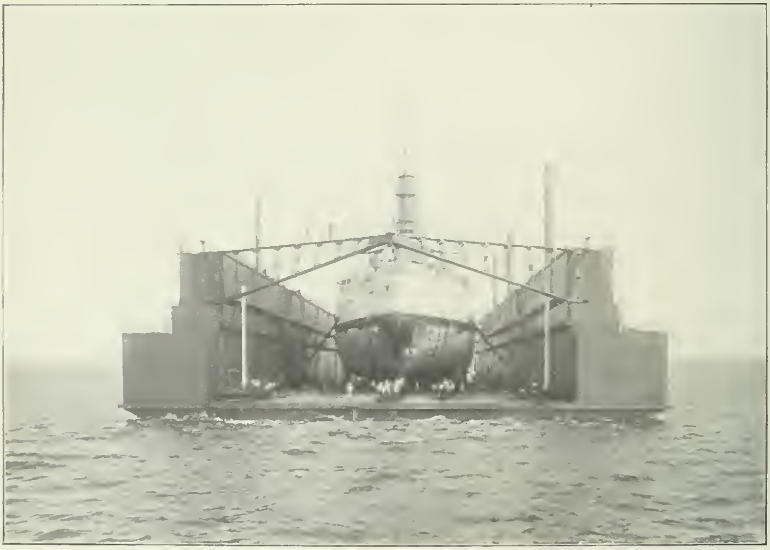


FIG. 2.—U. S. S. *Iowa* IN DOCK. DOCK AT $4\frac{1}{2}$ FT. FREEBOARD.

The moment of stability, therefore, is,

$$D \,.\, G' M \sin.\, \theta = D\,(G M - G G') \sin.\, \theta$$

$$= D \left(G M - \frac{i}{V}\right) \sin.\, \theta,$$

$$D \,.\, G' M \sin.\, \theta = \text{new stability moment} = M'$$

$$= D \left(H - \frac{i}{V}\right) \sin.\, \theta \ldots\ldots\ldots\ldots (3)$$

Therefore the effect of the contained water is to reduce the metacentric height by $\frac{i}{V}$; or $\Sigma \frac{i}{V}$, should there be more than one compartment.

$$B M = R = \frac{I}{V}$$

$$H = R - A = \frac{I}{V} - A$$

$$H - \frac{i}{V} = H' = \frac{I}{V} - A - \frac{i}{V}$$

$$H' = \frac{I - A\,V - i}{V} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

Or, when there are more than one of such interior compartments,

$$H' = \frac{I - A\,V - \Sigma\, i}{V}.$$

From Equation 3.

$$M' = D \,.\, H' \sin.\, \theta = \frac{V}{35} \left(\frac{I - A\,V - \Sigma\, i}{V}\right) \sin.\, \theta.$$

$$M' = \frac{\sin.\, \theta}{35}\,(I - A\,V - \Sigma\, i) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

For $\theta = 1°$,

$$M' = \frac{I - A\,V - \Sigma\, i}{35} \times 0.0175 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

The stability curves shown in Fig. 19 are constructed on Equations 4, 5, and 6. When the ship is on the dock its own stability must be taken into consideration, and the I of the above expressions becomes $I + I_1$.

It will be seen from an inspection of Fig. 19 that there are critical points at the pontoon-deck level and at the top of the blocking before the water plane cuts the ship.

In determining the heeling effect of wind pressure, the moment of the wind, taken at 30 lb. per sq. ft. over one side wall into the distance between the line of its action and the intersection of new and old water lines, is equated to the righting moment and solved for the angle of inclination.

Calculations for stability were made for the following positions: When the dock has a freeboard of 2 ft. 6 in.; when the deck is flush with the surface of the water; and at points where the drafts are 4 ft., 17 ft. and 30 ft. 6 in. above the deck. The curves shown on Fig. 19 are based on these calculations, and give the righting properties of the dock, with the lifted ship, at all positions. Similar curves were also drawn for the dock as a whole and for each section alone, with and without ship loading, and for self-docking operations.

Machinery.

The specifications require the dock to lift a 16 000-ton battleship to a freeboard of 2 ft. in 4 hours. The quantity of water to be removed is 34 700 tons in 4 hours. Three main pumping elements were decided upon, all of which were to be placed on the port side. The pumps were to be of the vertical-shaft centrifugal type, and to rest directly upon a main drain running throughout the length of the side wall, with branches to each pumping compartment. Each pump, therefore, was required to lift 1 686 cu. ft. of water per min. against a 35-ft. head. Assuming a velocity of 10 ft. per sec., the area of the main drain required would be 2.8 sq. ft.; therefore a 24-in. pipe was used, giving a velocity of 9 ft. per sec. The dimensions of the pumps for the preliminary design, with a maximum head of 40 ft., were determined as follows:

Outside diameter of wheel............	4 ft. 10 in.
Inside " " "	2 ft. 3 in.
Revolutions per minute..............	225
Diameter of suction and discharge....	24 in.

Engines.—The theoretic power required with a head of 40 ft. would be 130 h. p., and, assuming a hydraulic efficiency of 58%, the actual engine power to be provided for each of the three pumping elements would be 225 h. p. Horizontal, compound, non-condensing engines, 14½ by 25 by 14-in., with cylinders set at an angle of 135°, were adopted. The angle between the cylinders gives a slightly unbalanced turning moment on the shaft, but the disadvantage is of small

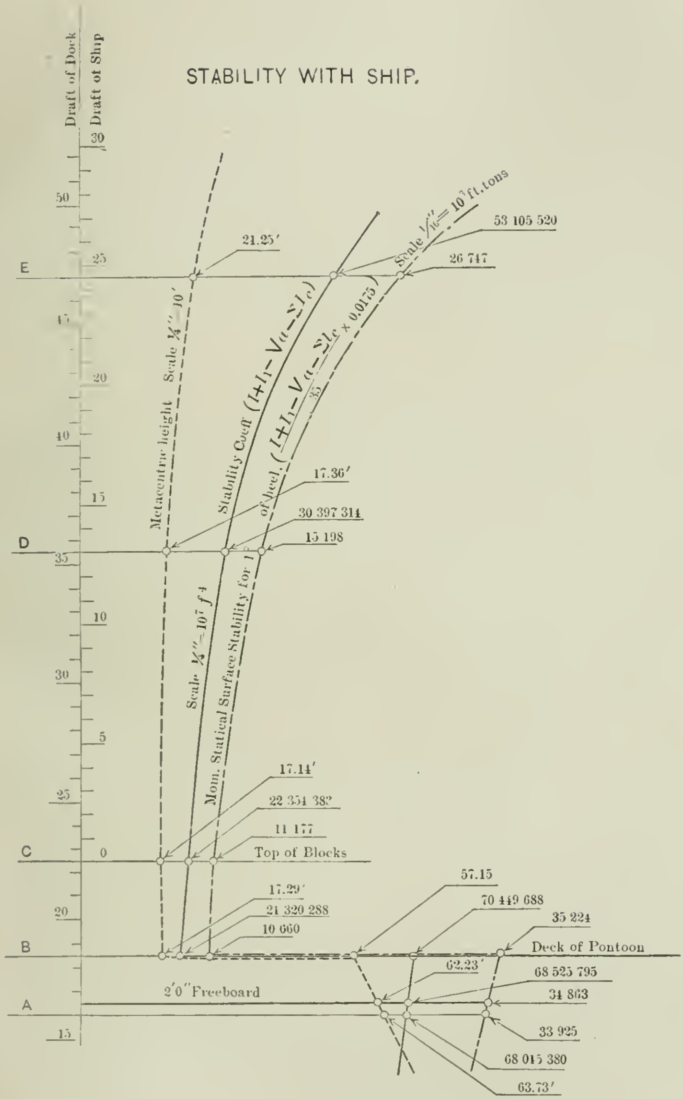

STABILITY WITH SHIP.
Draft of Dock
Draft of Ship
E
D
C
B
A
21.25′
53 105 520
26 747
Scale 1/16″ = 10³ ft. tons
17.36′
30 397 314
15 198
17.14′
22 354 382
11 177
Top of Blocks
17.29′
21 320 288
10 660
57.15
70 449 688
35 224
Deck of Pontoon
62.23′
68 525 795
34 863
2′0″ Freeboard
33 925
68 015 380
63.73′
Metacentric height Scale 1/4″ = 10′
Stability Coeff $(I+I_1-V_a-\Sigma I_c)$
Scale 1/4″ = 10⁷ f⁴
Mom. Statical Surface Stability for 1° of heel. $\left(\frac{I+I_1-V_a-\Sigma I_c}{35} \times 0.0175\right)$

FIG. 19.

account, and, by placing the engines as nearly as possible in the direction of the length of the dock, vibration was reduced to a minimum. The pumps and engines were designed and manufactured by the Morris Machine Works, of Baldwinsville, N. Y.

Boilers.—Babcock and Wilcox, marine type, water-tube boilers were adopted, each of 1 750 sq. ft. heating surface, 46 sq. ft. grate surface, and were designed to work at 150 lb. steam pressure.

Auxiliary Machinery.—An air compressor having a capacity of 527 cu. ft. per min. was installed on the port side. On the starboard side was installed a small machine shop, with lathe, drill, shaper, etc., an electric generating set for lighting the dock, and an evaporator, fire pump, etc., together with a donkey boiler for generating the necessary steam.

Construction.

The dock was constructed by the Maryland Steel Company, at its works at Sparrow's Point, Md. A basin was excavated in the foreshore and closed by a coffer-dam. A sufficient number of spruce piles were driven to support the estimated weight of the dock, allowing 6 tons to the pile. A 6-in. centrifugal pump, electrically driven, was operated continually for drainage. The structural material was rolled at the works of the Pennsylvania Steel Company, at Steelton, Pa., and of the Central Iron and Steel Company, at Harrisburg, Pa., and was milled at Sparrow's Point. As the dock was designed to avoid all curved work, and as the type of structure renders possible the use of similar members, the multiple punch was available in machining all the plating and bulkheads, while all frames were assembled and riveted by yoke machines in the yard.

Erection was carried on in a simple and rapid manner. A trestle was built out into the basin at the same level as the pontoon decks. Two 10-ton traveling cranes, with arm height and reach sufficient to handle weights at any part of the completed structure, were erected on these trestles. As the first bay of the end pontoon was erected, the trestle stringers with crane tracks were extended over the transverse bulkheads. As the work progressed, the track was extended throughout the length of the dock.

All detailed plans were prepared in the course of construction, and, as the supervising engineer was permanently stationed at the

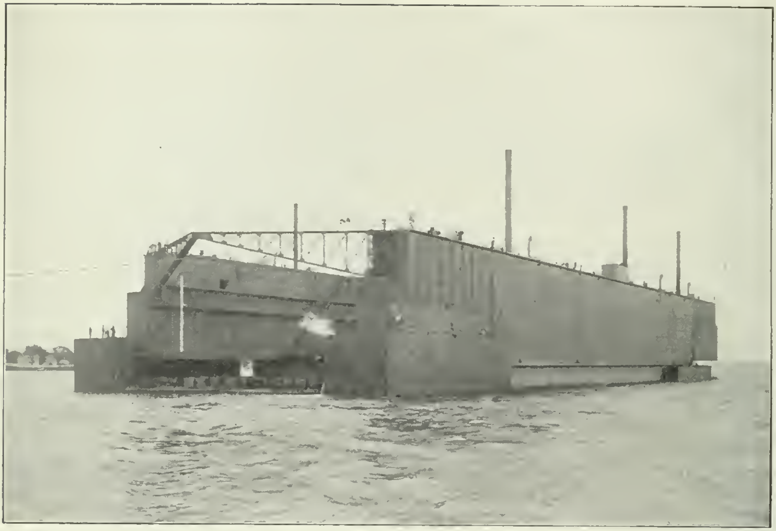


FIG. 1.—SELF-DOCKING: STERN PONTOON SUBMERGED AND DRAWN UNDER CENTER PONTOON; BOW PONTOON IN POSITION FOR SIMILAR OPERATION.

FIG. 2.—SELF-DOCKING: SIDE VIEW, SHOWING CENTER PONTOON DOCKED.

works, all plans were checked and approved with the least possible delay.

The selection of a protective coating for the dock was made the subject of careful study. Samples of a number of the best known paints on the market, exclusive of the oxide paints, were applied to test-plates and subjected to different conditions. Three plates were coated with each sample, one was exposed to the weather at the company's works, a second was suspended half in air and half in water, and a third was submerged in the water of Chesapeake Bay. The tests extended over a period of 2 years, during the construction of the dock, and resulted in the choice of a mixture of red lead, white zinc and linseed oil in the following proportions: 100 lb. of red lead, 15 lb. of zinc ground in oil, and 5 gal. of linseed oil. It is only fair to state that in these tests a graphite paint manufactured in Detroit showed as good results as the red lead, but was not used because of the lack of available data bearing on its behavior in salt water.

As the pontoons have more or less water in their bottoms at all times, except when self-docked, it was very necessary to provide adequate protection for their floors. Experiments with Bitumastic Enamel led to its application to the whole of the interior floors and to all vertical bulkheads, braces, etc., to a height of 12 in. The process of applying this mixture consists in a careful cleaning and drying of the metal, one coating of a solution the function of which is to provide a surface to which the enamel will adhere, and a final heavy coating of the enamel, from $\frac{1}{16}$ to $\frac{1}{8}$ in. in thickness, applied hot.

Great care was taken to rid the hull plating of mill scale. The specification provided that all loose scale should be removed by hammering, scraping, and brushing with wire brushes, and to make its removal easier and surer, no paint was applied until a short time before launching. Nearly all the material, therefore, was exposed to the weather in the yard for periods ranging from 12 to 24 months. The existence of mill scale on the completed structure has such an important bearing on the subject of corrosion that the additional expense entailed by pickling would be a good investment in the way of insurance, and it is recommended that specifications for future docks include this requirement. It was also noticed that, in the process of weathering, the mill scale, where covered with paint, adhered closely and could not be removed without the use of hammer and chisel. This scale, of course, will come off in time, and for this reason the requirement, that all contact surfaces be given one coat of paint before assembling, should be limited, so as to apply to rolled shapes only.

The contract for the dock was awarded on April 20th, 1903, and the first shipment of plates was received at Sparrow's Point on July 17th. The first bottom plate was laid on the blocking on September 23d, 1903, and the dock was launched on June 10th, 1905. The official ship docking and self-docking tests were held in the mouth of the Patuxent River, where the dock was towed immediately after the launching. A very complete description of the official tests by Civil Engineer A. C. Cunningham, U. S. Navy, Member of the Testing Board, was published in the *Journal* of the American Society of Naval Engineers,* and as the information contained therein may be desirable for purposes of discussion, it is deemed expedient to quote directly from that excellent paper.

"The U. S. S. *Colorado* was docked on June 23, 1905, having a displacement of 13 300 tons at that time. The main and docking keel blocks were all set at the same height. In this preliminary test no effort was made to secure speed, and one-half hour was used in making flushing and fire connections. The elapsed time from when the ship landed on the blocks until the keel came out of water was two hours and sixteen minutes. Pumping was continued until the dock had a uniform freeboard of two and one-half feet, only enough excess of water being retained in the side walls and end compartments to give the necessary trim. The *Colorado* was carried on the dock about twenty-four hours without changing the water ballast. When the dock had reached a freeboard of two and one-half feet with the *Colorado*, the deflection on the main keel line in the five hundred feet of length of the dock was about one-quarter of an inch; after about twenty-four hours the deflection in five hundred feet increased to about one and one-sixteenth inches. After undocking the *Colorado* the dock was found to have practically straightened without retaining any set.

"After the undocking of the *Colorado,* deflection observations were continued for three days, and variations in deflections with the dock unloaded, due to temperature changes, of seven-eighths of an inch were noted.

"The battleship *Iowa* was docked on June 27, 1905, for a record test, having a displacement of 11 600 tons at the time, and was car-

* Vol. XVII, No. 3.

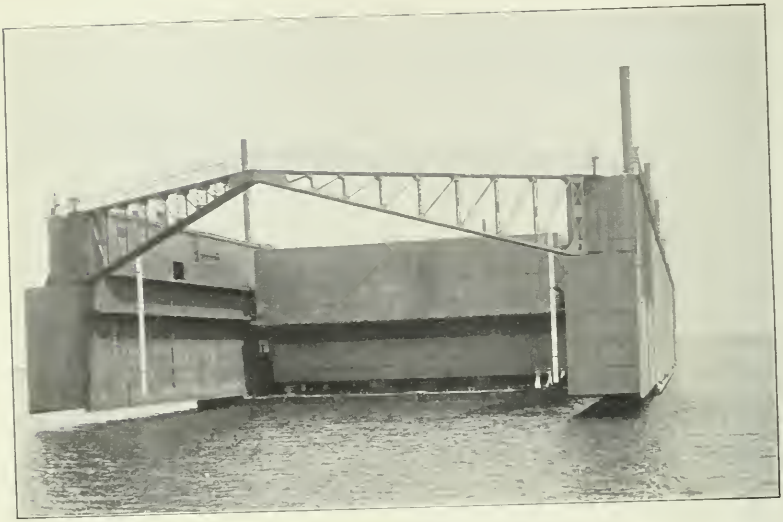


FIG. 1.—SELF-DOCKING: END PONTOONS DOCKED.

FIG. 2.—VIEW OF DOCK, MARCH 16TH, 1904, SHOWING BOTTOM PLATING IN PLACE ON BLOCKING IN BUILDING BASIN, MAIN PONTOON FRAMING, ETC.

ried on the dock for forty-eight hours. The specifications required that a 16 000-ton ship should be raised in four hours from the time the ship took the blocks until the keel was out of water. For the equivalent of a 16 000-ton ship the dock was pumped to a freeboard of four and one-half feet. From the time the *Iowa* took the blocks until the keel was out of water was one hour and thirty-seven minutes; to the time the dock had a freeboard of four and one-half feet, two hours and forty-two minutes. During the docking of the *Iowa* one of the three pumping engines was out of commission for forty-two minutes with a slipped eccentric, so that the actual time of operation of the dock is about half that allowed by the specification.

"The *Iowa* was docked by uniform pumping, as in the case of the *Colorado*, and carried for forty-eight hours without change of water ballast in the dock. The specification required that when a ship had been docked by uniform pumping until the dock had a freeboard of two feet the deflection in the entire 500 feet of length of the dock should not exceed three inches. When the dock reached a freeboard of four and a half feet with the *Iowa*, the deflection was about two inches. During the first twenty-four hours, the dock remaining uniformly pumped, the deflection increased to four inches in the 500 feet, and during the second twenty-four hours showed a recovery to three and three-eighths inches.

"Immediately following the undocking of the *Iowa* the dock was pumped up to the same depth of water in the compartments as when the ship was docked, which gave a freeboard of nine feet and six inches, and it was found that the dock had a hog of one inch. During the night this hog disappeared, and early the next morning was a half inch sag. The greatest deflection in the bearing length of the *Iowa* while carried on the dock was about one and three-quarter inches. The deflection observations indicate that there was no permanent set caused by the docking, and that temperature variations may cause considerable hog or sag.

"After the undocking of the *Colorado* the main and docking keel blocks were found to be uniformly indented about one-sixteenth of an inch with no crushing. No change was made in the blocks for the *Iowa*, and after undocking she was found to have rested even more easily than the *Colorado*."

During July the pontoons were successfully self-docked, and full particulars of this operation, by the writer, have also appeared in the *Journal* of the American Society of Naval Engineers.* The total time consumed in the self-docking tests, exclusive of time spent on the blocks, was about 15 days, and it would seem probable that the time required for future self-dockings could be reduced to from 10 to 11 working days.

* Vol. XVII, No. 3.

The self-docking was in every way a success, but it may be of interest to call attention to one point which, while not affecting the safety of the structure, may yet be considered a defect of design. Preliminary to docking the center pontoon, it was necessary to disconnect the end pontoons and sink them to sufficient draft to allow them to be drawn under the center section. During the process of sinking the bow section, it was observed that after the pontoon deck was below the surface of the water it was very difficult to maintain longitudinal trim. First the forward end would sink more rapidly and when this was checked by the valves the after end would forge

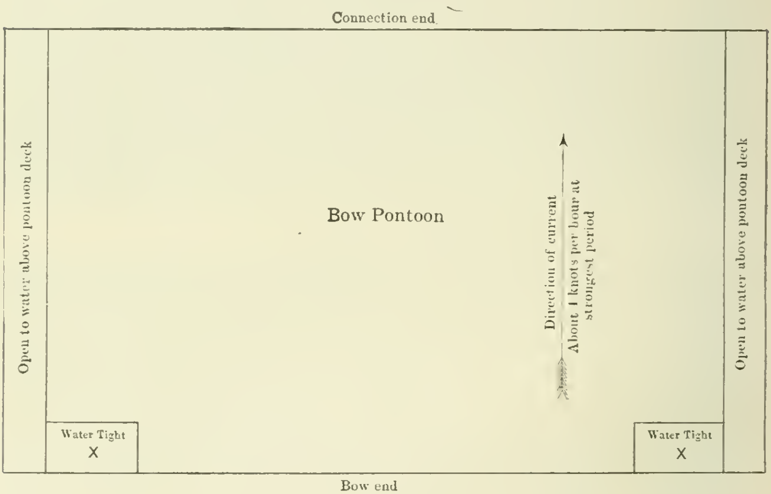


Fig. 20.

ahead. This surging was not alarming, and would bring up within 3 or 4 ft. either way. The natural explanation of this action would seem to lie in the lack of symmetry of the horizontal section. When the deck is once covered with water, the flotation of the pontoon having been destroyed, the excess flotation at X (Fig. 20), where the side walls come around the end, hold that end up while the other tends to go down. The opposite result was probably due to the momentum acquired by the bow when the forward valves were closed. It should be observed, however, that the stern pontoon did not surge, so that the explanation offered may not be the correct one.

Fig 1.—View of Dock, May 1st, 1904; Bottom Plating and Bow Pontoon in Position.

Fig. 2.—View of Dock, August 1st, 1904, Showing Skin Plating of Pontoons and Side Walls.

It is possible that the surging experienced in submerging the Algiers dock may be explained by its pointed end pontoons and short side walls, and if this is true it would indicate that rectangular pontoons and full-length side walls are the best for stability during submergence.

The *Dewey* has proven a complete success in every way. It has performed everything that was required of it, both as regards the docking of ships and the raising of its own pontoons; and it should be said here that the highly satisfactory results attained were largely due to the very evident desire on the part of the contractors to reach the highest mark in dry-dock construction, and the structure must stand as a monument to the executive ability and skill of their engineers.

Any prediction as to the lines along which future development in floating dock construction will take place can have only the value of a personal opinion, but it would seem, from the tendency of recent designs and the demand for a rigid structure for naval purposes, that the solid-trough dock will again come into favor. It is, without doubt, the ideal dock, and it would be generally adopted for naval use if it could be self-docked. It is possible that a designer may invent a plan or device for self-docking the solid dock, and, when this is an accomplished fact, it would seem that future advance must be confined to matters affecting the lifting capacity, structural strength, and general convenience.

The choice of power for operating the machinery is governed by special circumstances, but the desirability of furnishing light and power to ships in dock may influence the more general use of electricity in the future. When located at repair yards the current could be supplied from power-houses ashore, and the dock could be provided with a generating plant for independent operation, as in the case of the Pensacola (formerly Havana) dock.

To facilitate towing, the pontoons might be given a scow-shaped bow and stern, and, though involving considerable additional expense, the corners might be slightly rounded without interfering to any great extent with the stability. Self-propelled docks have been proposed, and such a dock is practical, but it is doubtful if there is yet a sufficient demand to warrant their serious consideration.

It should be pointed out, in explanation of the incompleteness of this paper, that it is intended merely as a basis for a helpful discussion of a subject which must be of interest to all who are in any way connected with modern ships; and, if it should succeed in eliciting such discussion, its preparation will have been in a measure justified, despite its numerous and obvious shortcomings.

DISCUSSION.

George B. Rennie, Esq.* (by letter).—The writer's attention was first called to the subject of floating docks when Mr. Gilbert, of the United States, undertook the construction of a wooden dock for the Imperial Austrian Government, at the Austrian Naval Arsenal, at Venice, which, when completed, was towed to Pola. Having taken great interest in the construction of dry docks, his grandfather, John Rennie, having done much in that line, and his uncle, Sir John Rennie, having written a book on docks and harbors, it seemed to the writer that the wooden floating dock, as constructed by Mr. Gilbert, had many advantages over the docks formerly used. Mr. Rennie.

On his return to England in 1852, he heard that General Quesada, Chief Engineer of the Spanish Navy, who had been to the United States and had seen the wooden floating docks in use at some of the naval arsenals there, had asked his firm whether such docks could be built of iron. As the writer had studied this question, he proposed an iron-plated floating dock for the naval arsenal at Cartagena, Spain. There was an excellent harbor at this place, but the shore was of soft rock with many crevices, etc., and, with much height and pressure of water, it was difficult to keep a dry dock tight. For this reason a floating dock was well suited for this harbor. It was then proposed to excavate a shallow basin in which to place the dock in order to clean and repair it, etc. This plan was carried out, and the writer understands that the dock has been there ever since—more than 40 years—and when he last heard of it, it was in as good condition as when new. It was, or is, 325 ft. long, 105 ft. broad, and takes a ship of 27 ft. draft on 3-ft. keel blocks; and lifts more than 4 500 tons. The Cartagena Dock was soon followed by one for Ferrol, Spain.

In designing these floating docks, many new things had to be considered and proposed; the iron structure had to be made to float, and the ends of the dock left out. The wooden docks were built with pontoons at each end; this arrangement allowed of docking a ship longer than the dock itself. The dock being in one piece, was very strong. Tests were made at Cartagena by filling any compartment and pumping it out, independent of any other compartment, and the dock was made to heel over sideways, as well as at the ends, and righted again without danger or difficulty. A more detailed and constructive account of this dock will be found in the *Minutes of Proceedings*† of the Institution of Civil Engineers, and also in the *Transactions*‡ of the Institute of Naval Architects.

The iron dock for Ferrol was sent there, but was never completed, a revolution having broken out in Spain. The pieces of

* Member, Institution of Civil Engineers.

† Vol. XXXI, p. 295.

‡ Vol. X, p. 17.

Mr. Rennie. plating, etc., were used for other purposes. It was 50 ft. longer than the dock at Cartagena, and lifted nearly 1 000 tons more.

It may be mentioned that the bases of these docks were in single pieces riveted up, the means for cleaning and repairing being otherwise provided for. It was afterward arranged to have the bases divided into separate pontoons, so that the docks might be more easily launched, but where the base can be made in one piece it is better, as it is very difficult to connect the pontoons again after they have been in the water some years.

It is a satisfaction to the writer, after having constructed these iron floating docks for 50 or 60 years, to learn that so much interest is being taken in the subject, and that the docks have increased so much in number and size. This short account of what has been done may not be of much interest, but the writer is glad to learn that the subject is being taken up before the American Society of Civil Engineers.

Mr. Baterden. J. R. BATERDEN,* Esq. (by letter).—The writer cannot admit that the cost of a graving dock is greater than the cost of a floating dock of equal capacity and in a similar situation. A comparison of the costs, when completed, of No. 3 Dock, Norfolk Yard, and the Charleston No. 1 Dock, leads one to the conclusion that the *Dewey* will have cost rather more than either of these.

The writer's experience in the construction of a large number of graving docks is that in every case floating docks of equal capacity on the same sites would have been more costly. Moreover, in the confined situations to which docks and repairing yards have to be adapted in an industrial district where land is valuable, the floating dock, owing to the large amount of land taken up by slopes, in cases where dredging is required, often renders its adoption an impossibility from a paying point of view.

In one location where the writer had charge of the construction of graving docks, and where the water frontage was short and the work had to be carried on in a confined space, two graving docks were already in use, and another was being constructed, but, owing to high ground on three sides, and the soft nature of the foundation, it would have been impossible, without interfering with adjoining property, to have constructed more than one floating dock of a capacity equal to one of the graving docks.

In another place, however—a long narrow strip of land along the banks of the River Tyne—where two floating docks are in position, it would have been impossible to construct one graving dock of a capacity equal to the larger of the floating docks, without cutting up the yard in such a way as to detract seriously from its value.

These examples show that, as a general rule, each location has to

* Associate Member, Institution of Civil Engineers.

Mr. Baterden.

be considered on its merits, one suiting a graving dock, another a floating dock. There are few situations, however, in which a graving dock could be built in fairly good ground, where the writer would recommend a floating dock.

As the author states, the expensive dredging which may be required, also moorings and shore connections, should be added to the cost of a floating dock for comparison with a graving dock, but in many cases this is not done. The dredging, which corresponds to the excavation required for a graving dock and much exceeds it in quantity, is in many cases a considerable item.

Judging by the photographs, one is led to assume that the *Dewey* is moored in deep water, and possibly no dredging was required, but, if dredging were done, it would be interesting to know the cost.

In the case of the two floating docks on the River Tyne, above referred to, the cost of dredging the site was about 14% of the cost of the docks, and was a long and tedious process, owing to the hard nature of the ground, in addition to which, the surrounding quays cost about 11 per cent.

The ideal site for a floating dock is where deep water is available close to the shore. If the dock has to be placed some distance out, in order to ensure deep water without dredging, then connecting jetties may have to be provided; otherwise, most of the material required for repair will have to be transhipped to lighters, taken to the dock, and lifted on board, thus being handled twice; whereas, in the case of a graving dock, it would probably be brought to the side by rail.

It is true that, except in rock, there is always an element of uncertainty about the foundations of a graving dock, which does not, to the same extent, enter into the calculations for a floating dock.

The element of risk to vessels in the latest improved floating docks is not a serious one, and it is known that accidents have occurred to vessels in well-equipped graving docks.

A floating dock can certainly be constructed in less time than a graving dock, but, if there is much dredging to be done, as in the instances above mentioned, and as in the case of the Bermuda dock, the time required might be much greater than the time occupied in building the dock, and even as long as in building a graving dock.

The depth of water required for floating docks of the capacity of the *Dewey* with a 30-ft. draft over 4-ft. keel blocks, would be more than 50 ft. Thus, it would be beyond the working depth of dredges of any power and capacity, and the dredging would have to be done by special means, which is always costly.

As the author says, if there be plenty of water under the bottom of a floating dock, one is not limited to the draft of ship taken on, as by the sill of a graving dock; but if dredging, especially of a

Mr. Baterden. difficult character, has to be done to obtain the necessary depth, it is unlikely that more than is absolutely necessary will be taken out, and the writer knows instances where the bottoms of floating docks rest upon the ground when sunk to receive a ship.

Unless it be confined by quays, a floating dock does not, like a dry dock, restrict the length of ship lifted, which is often a great advantage, but the advantage of adding sections to lengthen a floating dock when trade warrants, to which the author refers, is equally available by putting a temporary wooden end to a graving dock—not a costly item—and extending the dock when required.

The maintenance of floating docks is an item which, like their first cost, depends greatly upon circumstance, and there is not at present sufficient information to compare it accurately with the maintenance of graving docks, but it will be generally admitted that the latter is less. The dredging required from time to time, if the dock is in a location likely to be silted up, might be the larger proportion of the cost of maintenance (which may be anything up to 1½% per annum on the first cost, or more), and has the disadvantage of being more uncertain than the cost of painting and docking. If, as in the case of the *Dewey*, where no re-dredging enters into the cost of maintenance, it amounts to 0.72% per annum on the first cost, it will be seen that if, as in places the writer knows, floating docks are in locations where silting goes on at the rate of from 12 to 18 in. per annum, the cost of maintenance might easily be doubled, because, with such excessive silting, dredging may be required more frequently than cleaning or painting.

Again, in the comparison of maintenance between a graving dock and a floating dock, it must be noted that ordinary repairs could be executed and re-dredging done outside while vessels were in the graving dock undergoing repairs, whereas, in the case of the floating dock, for any dredging required, the dock has to be removed from the site, and in this case, as in docking and painting, the whole of the work is stopped.

As regards pumping, as far as the British Isles are concerned, with the high range of tides available, unless the graving dock be entered from a basin, or the ship be of the maximum capacity of the dock, the draft of ship regulates the quantity of water to be pumped, as ships can be admitted at different stages of the tide, some requiring more water than others; whereas, on the southern and eastern seaboard of the United States, for 1 500 miles or more, there is no appreciable tide, and on the remainder of the coast, east and west, the highest tides are only about equal to the lowest of those on the British coasts, and pumping would be much the same for all ships. Hence, in seas where the tidal rise is very small, the advantage of pumping is largely in favor of the floating dock.

The writer knows of no place on the British coasts where deep water, such as required by the *Dewey*, could be available except in a situation so exposed as to render it impracticable. Mr. Baterden.

The moving of a large floating dock from site to site has so many objections that, from a commercial point of view, it could not be entertained. The loss of the floating dock while being towed from the Tyne to Durban, the grave fears which were entertained as to the safe arrival of the *Dewey* in the Philippines, the fact that these craft, with side area approaching the sail of a full-rigged ship, with the disadvantage of not being able to reef it, exposed to the action of wind and sea when being towed along a coast line or taken up rivers, the great cost of towing and insurance, however necessary it might become from a strategic point of view in time of war, must necessarily be very costly, and would entail the presence of a small fleet for protection.

It will be seen, then, that graving docks and floating docks each have their advantages; there is scope for both, and, in coming to a decision as to the selection of either, many things have to be taken into consideration.

CECIL H. PEABODY, ESQ.* (by letter).—This paper has the double interest that comes from a full description of the *Dewey*—the most important floating dock—and from a detailed statement of the computations for strength and stability. With such a wealth of computation, compressed as it must be for presentation in a paper of fifty pages, one can grasp at the first reading only the methods and general results, with perhaps a casual question or two. For example, on page 127, the question arises: What was the distance from the keel to the bilge blocks? To be sure, a little puzzling over the figures leads to the conclusion that the distance must have been 22 ft. Again, the specifications require the dock to be able to dock all classes of ships, either centrally or with the keel 1 ft. off center, but no mention appears to be made of this in the calculation of the stability, nor is there any statement of the probable inclination of the dock under the assumed wind pressure of 30 lb. per sq. ft. As for eccentric loading, it is easy to see from Fig. 19 that a 16 000-ton ship, 1 ft. off center, will give an inclination of only $1\frac{1}{2}°$ in the worst condition. Mr. Peabody.

One of the most interesting features of the paper is the discussion of the proper thickness of the outer shell plating. The author concludes that "all attempts to derive a rational formula for them in flat plates have resulted in complicated expressions too cumbersome for practical use," and, if he had it not in mind, would doubtless include in this category a paper by Ivan G. Boobnoff, I. R. N., "On the Stresses in a Ship's Bottom Plating Due to Water Pressure."† Truly, the computations required by his method, even with

* Professor of Naval Architecture and Marine Engineering, Massachusetts Institute of Technology.

† *Transactions*, Inst. of Naval Architects. Vol. XLIV, p. 15.

Mr. Peabody. aid of the special tables which he provides to save labor, would ordinarily be considered to be complicated, though it appears to the writer that too high a price can hardly be paid for computations which lead to reliable results.

It may be interesting to know that observations were made on the deflection of the rectangular panels of the inner plating of the double bottom of a certain warship, when tested under water pressure, by an assistant naval constructor who, at the time, was studying at the Massachusetts Institute of Technology. The results showed a fair conformity with the theoretical calculations made by Boobnoff's method. This work, which was under the direct supervision of Captain Hovgaard, Professor of Naval Design, it is hoped will be carried further and will be used as the basis of a logical determination of the proper thickness of shell plating, or the proper spacing of frames. Meanwhile the best practical guide is unquestionably the common practice in ship building, as given in tables of scantlings furnished by marine insurance companies and associations, or, as may be inferred, from practice in warship design.

The results of the experiments, referred to by the author on page 130, will be awaited with much interest, though it is to be regretted that the plates were supported only at their edges and were not continuous over the supports.

Mr. Colson. C. Colson, Esq.* (by letter).—A cardinal point presented for consideration by the author is whether there are advantages attaching to the floating dock which are of such a character and economic value as to outweigh those offered by the sunk or dry dock, both for commercial and naval purposes, but with special reference to the latter.

No question arises as to the possibility of designing a floating dock of dimensions, strength and stability capable of lifting the largest and heaviest ship constructed or contemplated. Recent examples of floating-dock construction conclusively prove this point.

There are places where the construction of a dry or sunk masonry dock would be, practically, an impossibility, or where it could be accomplished only at an enormous outlay; in such a case the only alternative would, probably, be the floating dock.

Among the many points to be taken into consideration with special reference to the floating dock are: Water area available in a convenient and sheltered position for berthing such a dock without obstructing navigation or interfering with wharfage; depth of water on any available area at low tide; draft of water required over the sill or keel blocks; character of the ground to be removed in order to obtain the depth of berth required; the class of vessel to be dealt with, and the use for which a dock is required; and, whether the depth of water over the keel blocks or sill should be for light or full-load draft?

* Member, Institution of Civil Engineers.

Even for commercial purposes, it appears desirable that the power of docking at full-load draft at low water of spring tides should be available, although such a facility may not be often in demand. With regard to naval ships, not only is it essential that the power of docking at full-load draft at low water be provided, but there must be a margin of depth to admit of a seriously damaged ship being docked at low water. These requirements will increase the depth, and, consequently, the first cost and subsequent maintenance of the berth, and augment the ancillary works. Mr. Colson.

Where docking is required for cleaning and coating, or for the execution of minor repairs, the floating dock is eminently fitted, but, in cases where extensive repairs are required, involving the manipulation of heavy weights and the employment of a large number of men, the conditions become more complicated, and, in the writer's opinion, indicate the adoption of a dry dock, from the point of view that heavy weights can be handled with greater facility, resulting in the saving of time and labor; further, a ship in a dry dock is under more complete observation from the wharf, and in case of fire is much more accessible.

With regard to the comparative cost of a steel floating dock and a masonry dry dock of equal docking capacity, there are so few data available as to the cost of the different items making up the complete installation, that any general statement must be fallacious; a reliable comparative estimate is only possible after careful consideration of the conditions obtaining in each case, and the preparation of alternative designs in detail. The total cost of installation should include the acquisition of the site of the berth and any other areas required in connection with the dock; the cost of the dock, complete; the conveyance from the place of construction to the permanent site; the preparation of the berth, moorings, shore facilities, including wharf walls or settees, shops, stores, etc.; and, means of communication between the dock and the shore. In this connection any information that the author can give as to the installation of the floating dock, *Dewey,* will be most valuable.

The time required to install a floating dock, complete in every particular, is not always determinable by the time required to construct the dock itself; it must depend greatly on the character of, and time required to complete, the ancillary works.

It is suggested that no land space is required in connection with a floating dock. This view will hardly hold good in all cases, or in the majority of cases, where a floating dock is required for cleaning and coating a ship and for minor repairs. It is conceivable that it might be moored in a position in the open, but even then it might be necessary to acquire the area of the harbor bottom for the berth. Where a dock is required in connection with an existing shore es-

Mr. Colson. tablishment, with water-frontage rights and wharfage, of course, a very considerable outlay will be avoided; but, failing any existing water and land facilities, a very substantial expenditure in this connection will be necessary.

The greater the width required on the floor, the greater will be the depth of the bottom member of the dock, and, consequently, the greater will be the total depth of the berth; while the area to be dealt with will be increased by the slopes necessary to ensure the stability of the sides, 59 to 60 ft. would not be an excessive depth below low water for a dock with a draft of 35 ft. over the keel blocks. This would probably mean the maintenance of a hole 24 or 25 ft. deep, below the approach channel; or, the bottom of the harbor, if the berth is to be located in deep water. The preparation of such a berth might be a long and costly operation, unless under very favorable circumstances. A site might be found with an ample depth of water at low tide, but such a case would be exceedingly rare, and then only at a very inconvenient distance from the shore.

With regard to the author's remarks as to the comparative power of floating and dry docks, he must refer to dry docks constructed to take in ships of maximum draft at high tide only, or, at any rate, not at low water. In dry docks of modern construction the sills and floors, as a rule, are designed so that battleships of the greatest draft can be docked at low spring tide. Therefore, the advantage may be taken as being with the dry dock, inasmuch as the rising tide will increase the available depth of water over the sill or blocks. The exceptions to this are docks opening into a practically tideless harbor, or into closed basins where, however, the water, as a rule, can be raised 1 or 2 ft. by pumping.

A floating dock, when raised with a ship upon it, would be a very prominent target, and would be far more susceptible of damage from shell fire than a sunk dock. A suggestion is made that in an emergency the floating dock would have the advantage, inasmuch as it could be towed to a place, or places in succession, of greater safety. This, of course, would be possible, but does not the suggestion presuppose the provision and maintenance of emergency sites and moorings—adjuncts which cannot be prepared at very short notice even from a temporary point of view?

In the case of a badly damaged war vessel with a heavy list and all her weights on board, it is the writer's opinion that she would be placed with greater confidence in a dry dock than on a floating dock, because, in the former, the facilities for the removal or adjustment of weights, with a view to getting the vessel upright, would be greater than in the latter. It has been suggested, however, that the floating dock could be trimmed (*i. e.*, listed) so as to accommodate it to a listed ship, and the vertical position restored with the rising of the dock. This view is probably correct to a limited extent with a sectional dock, when dealing with vessels floating light—especially commercial vessels—and, to a greater extent, with the solid type of floating dock, but it is a doubtful point as to how far a heavy, deep-draft battleship with a bad list could be thus dealt with on a sectional floating dock, bearing in mind the character of the ship's bottom, the distribution of weights, and consequently the risks involved by insufficient or irregular support. In this connection it would be interesting if recorded details of any such operations could be quoted, and the opinion of the author and other experts would be of great value. Mr. Colson.

Although no doubt exists as to the possibility of lifting the largest and heaviest war vessel, under favorable conditions, there are risks and difficulties attending the docking operations on, and subsequent use of, a floating dock, which do not obtain in the docking operations, and subsequent use of, a dry dock. While the disabilities, by design, care in manipulation, and forethought, may be reduced to a minimum, they cannot be entirely eliminated. The balance of argument, therefore, remains in favor of the dry dock, especially for naval purposes. At the same time the writer fully concurs with the author's remark, "that each type has its own particular field of usefulness which the other cannot with advantage fill;" therefore, no thoughtful engineer will fail to recognize the value of the floating dock, or hesitate to recommend its adoption where the conditions undoubtedly indicate its adaptability in preference to a dry dock, although his bias may be in favor of the latter.

A. C. Cunningham, M. Am. Soc. C. E. (by letter).—Mr. Cox has treated the subject of floating dry docks so thoroughly and so ably, that there remains but little to say in the way of discussion which will not be a repetition of his ideas in a different form, or an enlargement upon the same. Mr. Cunningham.

The short and certain time in which a floating dry dock can be constructed may be an important factor, both from a commercial and a military point of view. From the actual records of the time of construction of various docks, and the observation of what might be the possible rate of progress of construction, it is safe to say, that, in emergency, a floating dock of the first magnitude could be built in a year. If such a dock was also of the sectional type, each part complete in itself, the time of construction might be still further reduced.

The possibility of manufacture in one or more places and of final assemblage and erection at the destination are also of great importance from the commercial and military points of view, especially from that of the latter. Since the trip of the *Dewey,* it is an

Mr. Cunningham. established fact that a floating dry dock can be towed anywhere with certainty and safety. Such towing, however, is a slow and tedious operation, and in time of war would be attended with several apparent risks, all of which can be eliminated by the erection of the dock at its destination.

The British and Japanese Governments have just demonstrated that 20 000-ton battleships can be built in a few months. This is a demonstration that radical departures and advances in ship building may occur at any time in the future. When such departures and advances cannot be cared for with existing masonry docks, they are still rendered possible by floating docks. No matter what length, beam and draft may be given a ship, a floating dock that will accommodate it can be built and put in operation in less time than the ship can be constructed.

Increased and careful attention is being given to the docking of ships which may have abnormal draft from accident. To provide for this contingency with a masonry dock means not only greatly increased difficulties and cost of construction, but a continuously increased cost of maintenance, and more especially operation.

The floating dock is an ideal structure for dealing with ships at abnormal draft. Its increased cost for this condition is a trifle; there are no increased difficulties of construction; the increased cost of maintenance is hardly perceptible; and, the cost of operation for ordinary conditions is not increased at all.

While the desirability of a floating dock for dealing with ships at abnormal draft is becoming recognized, general attention has not yet been called to its possibilities as an auxiliary to masonry docks for this condition. The masonry docks of the future must accommodate the length, beam, and normal draft of the ships which they are to dock, but if they are to accommodate also the possible abnormal draft which may occur, it means a large and constant outlay of money for which there will seldom be a return. From a military point of view, abnormal drafts will occur in groups at unknown intervals, and may be with our own or captured ships.

With a suitable distribution of floating dry docks, the provision for abnormal draft in masonry docks may be dispensed with, and the increased first cost of constructing and the continual excess cost of pumping avoided. With such a combination the ship would be lifted first by the floating dock, such temporary repairs would be made as would restore normal draft, and the ship would then be placed in the masonry dock for complete and final repairs. At first thought it would seem that one deep-draft masonry dock in a suitable group would fully meet the requirements, but the mobility of the floating dock again introduces special advantages. Abnormal draft, in any event, means a dangerous condition, and may mean

Mr. Cunningham.

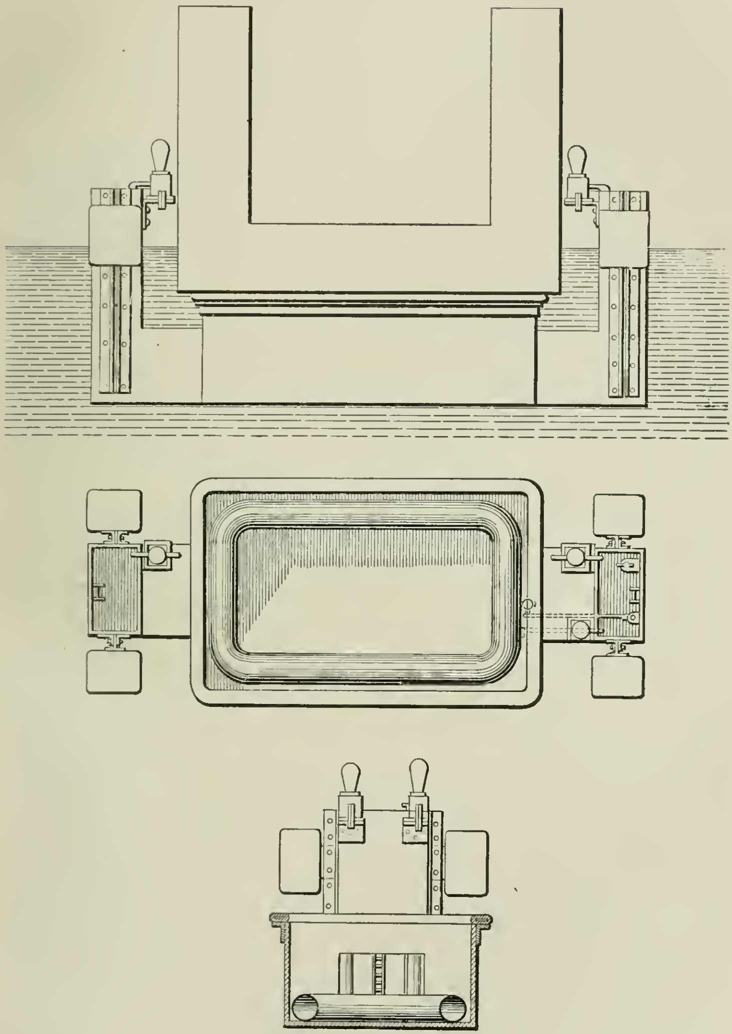


FIG. 21.

Mr. Cunningham. that a ship cannot pass through a harbor or regular channels in order to reach a masonry dock. In such an event the floating dock can go to a place where the ship can be lifted, and, as the draft of the combination will be small, the combination of dock and ship can be towed to a place of shelter and safety. One important requisite in docking a ship in a floating dock is that there shall be no independent and uncontrolled vertical motion in either. In extreme cases, however, as the saving of a battleship, even some vertical motion might be safely risked by suitably padding the deck of the dock with timber.

Floating docks located at the entrance of New York Harbor, the mouth of the Delaware River, the entrance to Chesapeake Bay, and in the Harbors of San Francisco and Puget Sound, might, in saving one or at the most two battleships, more than return the cost of all the docks together.

A floating dock in connection with a floating repair plant constitutes a repair station that can be taken anywhere. The repair plant can also be in the form of barges which can be placed on the deck of the dock when a move is desirable.

As to convenience and accessibility in use: If a floating dock is moved into a slip after lifting a ship, the bottom is more accessible to workmen than in the case of a masonry dock, and, with a suitable gantry crane, everything movable on the ship can be handled more conveniently and safely than with the jib cranes used around a masonry dock. With a suitable depth of water in the slip this same gantry crane would be far more desirable for handling the heavy weights on ships that are not docked than the shears and jib cranes now generally used.

In the matter of possible accident the floating dock, theoretically, is more exposed than the masonry dock. In this matter, however, every dock, whether masonry or floating, must be judged by its special conditions. A masonry dock, however safe and perfect in other respects, may always have its caisson rammed, and, with a ship in dock, such an accident would be very serious. A floating dock in an entirely exposed position may be rammed from all sides, but, if empty, such an accident would not result in the loss of the dock, on account of the very numerous water-tight compartments, and, for the same reason, if a ship were in the dock, the damage to it is not likely to be as great as in the case of the rammed caisson of a masonry dock. If a floating dock is placed in a slip with a level bottom not much below the bottom of the dock, ramming from any very heavy ship is impossible, and any serious sinking from failure of valves is guarded against. Floating docks are entirely free from failures of foundations due to quicksands, springs, and the hydrostatic pressure to which all masonry docks are more or less exposed. The caissons of masonry docks may fail from other causes than ramming, and, in the case of an isolated dock having no spare caisson, such a failure might put the dock out of commission for several months. Mr. Cunningham.

For certainty of preservation and repair, it has been necessary so far to make steel, floating dry docks self-docking. This feature not only complicates the construction and increases the cost, but is a source of more or less weakness, according to the number, location, and complication of the joints which connect the various parts of the dock. To eliminate the self-docking feature, the writer proposes an independent caisson for the repair and preservation of the bottoms of solid docks, which, if it can be made effective, will simplify the floating dock problem.

The proposed caisson is illustrated in Fig. 21, which is diagramatic, and for purposes of explanation only.

The action of the caisson is as follows: Caisson floating light with main and supplemental chambers empty; main chamber opened, and water freely admitted; complete submergence is prevented by the supplemental chamber. Water is next admitted to the supplemental chamber until buoyancy is just destroyed; the caisson then sinks until arrested by lines or supplemental floats. In this condition the caisson is moved under the bottom of the dock. Expelling the water from the supplemental chamber causes the caisson to rise and seal against the bottom of the dock; the water is next expelled from the main chamber, causing a still firmer seal against the bottom and providing a space in which work may be performed. The edges of the dock may be worked on by canting out of the water, or the caisson may be sealed against the bottom with the supplemental chamber and with one end projecting; after the edge of the dock is canted out of the water, the main chamber may be emptied; a working platform is thus provided. It is evident that entrance shafts and supplemental floats are not essential, as the caisson could be controlled by lines, and access had through a manhole in the bottom of the dock after the caisson was sealed.

If the self-docking feature of floating dry docks can be eliminated, a valuable gain will be secured. Not only do self-docking features increase the cost and complication of a dock and cause weakness on account of the necessary joints, but the self-docking of itself causes a loss of valuable time which, at least, may amount to from one to three months.

If a successful caisson method can be devised for working on the bottom of a dock, no time whatever need be lost unless it is in working on the edges of the dock, and, furthermore, it will be possible to carry on work while a ship is in the dock.

Lyonel Clark,* Esq. (by letter).—The writer has read this paper with much interest, especially that portion which deals with Mr. Clark.

* Member, Institution of Naval Architects; Member, Institution of Civil Engineers.

Mr. Clark. the calculations of the strength of the parts of a dock and of a dock as a whole. Many years ago, when the writer's firm* first took up dock designing, calculations were conspicuous by their absence; but his firm has gradually built up a collection of rules which are found very useful in the rapid determination of the parts of any given dock. It is also interesting to note that the justifications arrived at are practically the same as those in Mr. Cox's paper, so that the conditions under which the Algiers Dock was designed are practically the same as those which governed the construction of the *Dewey*.

Calculations for a dock, however, are somewhat like those of a ship: they are of great value if used as a guide for what is required rather than as a rule; for, as in the case of ships, there are many portions of a dock in which experience alone can determine what extra strengthening is required, or what portion may be lightened. A notable case in point is the determination of the required thickness of the skin plating to withstand a certain head of water. In the Cavite Dock, the permissible working stress was extraordinarily low. Mr. Cox appears to recognize this, and gives a supplementary curve (f) which he bases on experimental deflection. It is of interest to note that this curve follows closely that which would be given were the plating considered as a continuous beam uniformly loaded with a permissible stress on the material of 7 tons per sq. in. The writer's experience, however, is that much higher stresses than this can be safely supported.

Fortunately, the thickness of the skin plating of a dock is very frequently subservient to other conditions than mere stress considerations, but, where this is not so, in general practice, the writer's firm assumes the plating as a beam working at a stress of about 8 tons, and has, indeed, gone to higher figures than this, the highest, as a matter of fact, amounting to about 12 tons per sq. in. This was in a small dock built some 20 years ago, which, however, is still in use, and has certainly shown no signs of weakness.

In the case of the Rotterdam Town Dock, also, which has a lifting power of 15 000 tons, the stress on the bottom plating, according to the published plans, would work out, if considered as a beam, at not less than 14½ tons per sq. in., and it is believed that this dock, which has now been in service for the last two or three years, was built entirely according to the specification. It is evident, therefore, that skin plating does not behave solely as a beam, but, to a very great extent, as a suspended or stretched cord; and, although it is difficult to justify by calculation to what extent this action takes place, it is abundantly demonstrated by practice that, when considered as a simple girder or beam, skin plating may be

* Messrs. Clark and Standfield, of Westminster, England, and Washington, D. C.

stressed safely to considerably more than what would be allowed in other portions of the dock. Mr. Clark.

The other calculations, as already mentioned, follow pretty closely the practice of the writer's firm, and as his firm prepared a complete scheme for the Cavite Dock (which scheme, however, owing to the late arrival of necessary information, could not be completed in time for submission to the authorities), he is enabled to compare somewhat closely the scantlings proposed and those actually used in the *Dewey*. The difference is very slight, and although the type of dock worked out about 500 tons lighter than the *Dewey*, this was doubtless due to the fact that the design was of the "Bolted Sectional," or "Pola" type, as Mr. Cox calls it. This type, of course, requires fewer transverse bulkheads than the *Dewey* type, and the extra side-walls on the end pontoons are also unnecessary.

While mentioning this Pola type of dock, the writer desires to criticize some of the author's statements as to the advantages or otherwise of this type, and must still hold to his claim that the "Bolted Sectional" is the strongest type, even when compared with the *Dewey*. As regards the walls of this dock, and that portion of the pontoon above the light water line, these bolted sectional docks can be, and sometimes are, completely riveted up, so that the walls, the pontoon deck, and some of the side plating of the pontoons, are practically as continuous a dock as the *Dewey*, and only the extreme bottom edge of the pontoon has to be joined by bolts. The writer cannot help thinking that anyone must readily admit that a continuous line of close-pitched bolts running all around the bottom edge of the pontoon of a dock must be a considerably stronger form of joint than the vertical lugs shown in the *Dewey*, which—if he may be permitted to say so—somewhat recall that old engineering paradox of how the flange of a girder may be weakened by adding fresh material in the form of an extra web on top of it. However, be that as it may, there was no difficulty in designing and justifying a bolted sectional dock, under the Cavite conditions, that gave the required strength everywhere, and this, indeed, without having to deepen the side girders.

Mr. Cox also seems to be under a slight misapprehension as to the self-docking of this type of dock. The bolted under-water joint is nothing new, but has been used by the writer's firm for years past; indeed, it is now thirty years since he himself made and worked his first joint of this type, which is still in satisfactory condition, and since then many other docks have been joined by the same system, and, thus far, none of the difficulties foreseen by Mr. Cox have presented themselves.

As regards the self-docking itself, although in the sketches the

Mr. Clark. lands upon which the lifted portion rest appear to be very small, in practice they are of considerable length, and, generally speaking, a still further length of the pointed ends of the pontoon can also be utilized to give bearing surface. In the case of a large bolted sectional dock which Messrs. Clark and Standfield are now designing for Germany (which has a lifting power of 36 000 tons), the docking lands are more than 12 ft. long, and in addition to this, the points project some 50 ft. below the remainder of the dock, and can be utilized as supports if required.

That still water is necessary for self-docking this type of dock is self-evident; and, indeed, still water is necessary for any other self-docking system with which the writer is acquainted. If anything, the bolted sectional dock should be in a more favorable position, because it is evident that the larger and heavier the separate sections to be handled, the less movement they will have in a given seaway.

With regard to the author's general conclusions as to the line on which future developments in floating dock construction will take place, the writer is certainly in agreement with them, but with some modifications. The "Box" or "Solid Trough" dock is doubtless an ideal design, but there are great difficulties in its construction if it is to be of any size. There are but few sites where a dock could be built in a basin and floated out as a whole, and the difficulties of launching a dock as large as the *Dewey,* in one piece, from ordinary launching ways, would be very serious indeed. It becomes necessary, therefore, if the dock is to be built by ordinary ship builders, to construct it in sections, and, if it has to be built in sections, it is certainly advantageous to arrange them so that they can self-dock the remainder of the structure, the more so, as the writer most strongly holds, that the joints between such sections can be made as strong as the remainder of the dock. This could be done equally well whether the *Dewey* or the Pola type were adopted, and the writer would here like to express his appreciation of the ingenuity of the former design, which, if the dock has to be square-ended—as was specified in the case of the Cavite Dock—and especially if the end pontoons were bolted to the main structure after the manner of the Pola type—which would be perfectly feasible—he would consider as complete a solution of the problem as one could possibly devise.

This question, however, as to whether a dock should be square-ended, raises many considerations. The specification for the *Dewey* calls for a long dock of uniform buoyancy, and then bristles with all sorts of conditions as to how it is to be built in order to carry a short and heavy ship resting only over a portion of its length—in other words, to act as a short dock—these conditions being carried to such an extent that the ship is assumed to be a flexible mass, which, of course, is an impossibility. It is a mistake to carry the full lifting power of the dock over its whole length, and the writer is strongly of the opinion that the ends of a dock should be in the form of points having much less buoyancy than the remainder of the dock, and, indeed, at the extreme ends only working platforms are necessary. Mr. Clark.

It is interesting to note here, that, in spite of the low theoretical working stress allowed in the *Dewey*, when the *Iowa* was lifted, the deflection of that vessel was about 1¾ in. The Bermuda Dock, which has but little more than half the weight of the *Dewey*, and was built under no special conditions as to longitudinal strength, when lifting the battleship *Dominion*, which had a displacement of 16 380 tons, at the time of docking, produced a breakage on this vessel of only ⅞ in., although, since then, under certain conditions of temperature, it has deflected as much as 1⅛ in. This is solely due to the fact that the lifting power of this dock was mainly concentrated under the bearing length of the ship, the pointed ends giving but feeble buoyancy in comparison with the remainder of the structure.

The writer is very strongly of the opinion that, where the depth of water will permit, instead of putting metal into the walls of the dock in order to make them stiff enough to bear a concentrated load over the middle of the dock, this metal should be put into the pontoons so as to increase the lifting power, and make this practically equal to the weight of the ship bearing on the dock and concentrated under the same. This point has been emphasized most strongly in his mind when designing the large bolted sectional dock previously alluded to. This dock, which has a length of 721 ft. 6 in. over all, has a mean lifting power of about 55 tons per foot run over the square portion. This means that it can oppose a lifting power per foot run considerably greater than the weight per foot run of any ironclad built or building, and, indeed, such a weight is only just attained by the huge quadruple-turbine Cunard steamers now nearing completion, and for the lifting of which, or of vessels of similar size, this dock was designed. That this increase in lifting power is not attained at great expense may be shown by the fact that this dock has only 10 000 tons of steel in its hull, that is to say, but very little more than the *Dewey*, although it is of twice its lifting power.

In conclusion, therefore, the writer's opinion is that the dock of the future should be self-docking, and approaching as nearly as possible to the "Trough" or "Box" dock in strength, but with a lifting power per foot run in its pontoons superior to the unit weight of the biggest ships with which it has to deal. The end

Mr. Clark. sections, coming under the overhanging bow and stern of the ship, should be of much lighter construction and of small buoyancy, if any. With a dock of this sort, all deflection of the ship would be absolutely eliminated; indeed, if necessary, a contrary deflection, that is, a "hogging" effect, might be placed on the ship, and, in short, the docking berth on a dock of this type would approach as closely as possible to the support afforded by the bottom of an ordinary excavated masonry dock, which, for heavy ironclad and armored cruisers, it must be admitted, is as nearly as possible the ideal solution of the problem of supporting suitably a vessel, which, as Sir William White says, has a bottom which is comparatively an egg-shell.

Mr. Box. EDWARD BOX,* ESQ. (by letter).—The writer has been much interested in this most excellent paper.

In Great Britain, floating docks, in most cases, have been adopted only where the site has rendered it impracticable to build a graving dock.

The facilities offered by the surroundings of graving docks, recessed as they are into the land, apart from the actual method of docking, have doubtless favored the adoption of this type; but the fact remains that a great many ship owners give the preference to floating docks, when the two methods are equally obtainable. A good example of this is seen at the Smith's Dock Company's works, at the mouth of the Tyne.

The company has eight dry docks and a ship-building yard. Two of the docks are floating docks of the off-shore type. A fair idea of the business done by this company can be obtained from the fact that, for purposes of painting and repairing, nearly 800 vessels pass through its hands every year.

Regarding the floating dock from a dock owner's point of view, the periodical self-docking is tiresome, and, in locations where silting is rapid, the steady but persistent loss in depth of water is most troublesome. Yet the many advantages possessed by the one-sided floating dock, not the least being its rapidity of action, are likely to keep it in favor as a docking machine for commercial purposes.

The author, not unnaturally, has referred to Mr. Lyonel Clark's paper before the Institution of Civil Engineers, and as the writer, in his contribution to the discussion on that paper, criticised the several known types described by Mr. Clark, he does not think it necessary to repeat his remarks here, as undoubtedly those interested in the subject will have read Mr. Clark's paper. It was in Mr. Cunningham's discussion of that paper that the type of dock with which the author deals first came to the writer's notice.

There can be little doubt that the nearer we keep to the simple

* Associate Member, Institution of Civil Engineers.

box form of floating dock, the stronger it is possible to design. The type of dock which the author describes would appear to solve more nearly the problem of a two-sided or self-contained floating dock than most designs which the writer has seen; but, until auxiliary constructional works are entirely dispensed with, the problem cannot be considered to have been entirely solved. Mr. Box.

The difference in the quantities of water pumped in the two systems of dry docks, as given by the author, is interesting. In a similar calculation made some time ago, the writer took the total number of vessels actually docked in a graving dock and in a floating dock of nearly the same dimensions for a period of one year, and found the quantities of water to be as follows: Graving dock, 8 200 tons; off-shore dock, 3 500 tons. As no account is taken of emptying for blocking purposes, the difference at times is much greater.

This difference may not be of very great moment in the case of Government docks, at least in places where coal is reasonable in price, but it certainly is of great importance in that of commercial docks, more especially large docks where small vessels often have to be accommodated.

Based upon experience, the writer is strongly of the opinion that long pointed ends are a mistake, and, for purposes of control in working, the sides should be carried not necessarily the full length of the dock, but nearly so.

It is interesting to note that the *Dewey* was constructed in a basin. The writer would not like to be too confident as to who first suggested the shallow basin for floating-dock construction, but the method was strongly advocated by the late John Standfield, and adopted by his firm at Gray's many years ago. The Cardiff off-shore dock, the first of that type, was constructed in a shallow basin from which the water was excluded for the purpose by a bank forming a dam, which was removed for launching purposes.

No doubt the method has much to commend it, and it would be interesting to know from the author whether the Maryland Steel Company contemplates building ships upon the same berth, or whether the basin is intended for floating-dock construction only?

It would also be interesting to know the cost of the dock when leaving the builders; the cost of preparing the basin in which it was constructed, including removing and replacing the dam; and how the cost of the latter was apportioned.

As far as can be gathered from the small-scale drawing accompanying the paper, the author appears to have adopted the same arrangement of pipes as that used in the Clark and Standfield docks. Although this arrangement appears to have become a standard, it seems questionable whether pipe arrangements could not be

Mr. Box. considerably modified in the case of two-sided docks, and it would be worth while to consider this seriously in future designs.

As regards the surging experienced when sinking one of the pontoons for self-docking purposes, this is probably due to want of sufficient end-controlling power to cope with the natural tendency to dip whichever way it receives encouragement. It is just possible that at the time of submerging the dock, the structure was riding upon air compressed under the deck due to the mouth of the air-pipe being shut off by the rising water inside the tanks. Why one section should behave differently from the other, it is difficult to say, but it might be explained by the position of the mooring chains, coupled with the tide being in an opposite direction, making it difficult to control them.

As one who received his early training in the works and offices of Messrs. Clark and Standfield, under the late Mr. Latimer Clark and Mr. John Standfield, founders of the firm which bears their names, and, moreover, having taken a deep interest in the development of floating docks, the writer is able, from personal observation, to endorse all that Mr. Cox has said respecting Mr. Lyonel Clark. There can be no doubt that, showing considerable enterprise, Mr. Clark has done much to develop this particular branch of the profession.

Mr. Laws. B. C. Laws,* Esq. (by letter).—The writer would like to add his tribute of praise for the very able and exhaustive manner in which the author has treated the subject, not only from the general standpoint, but more especially from that of design; and it is mainly with regard to the latter that a few remarks will be made.

The writer can hardly understand why the commercial shipowner—as the author states—would prefer the floating dock on account of its flexibility. Vessels are and should be built without either hog or sag, that is, with a straight keel, and, under this condition, the best dock would be one in which that straightness would be preserved. But, if a vessel with a sagged keel were to be placed on the blocks of a dock designed for a certain maximum deflection, it would not be difficult to conceive that the deflection due to the weight of the vessel might be so augmented by the natural sag of the keel that the maximum (safe) deflection allowed might be exceeded, so that the dock would become permanently strained. Of course, the dockmaster, if he were cognizant of this peculiarity in the vessel, would guard against such a contingency by ballasting the dock properly, but this information is not generally obtainable.

This—perhaps not very important—objection to the floating dock is minimized by the assumption of flexibility in the vessel itself, so that—more or less—it will accommodate itself to the flexibility of the dock. However, it is just the uncertainty as to the

* Associate Member, Institution of Civil Engineers.

extent to which a vessel is capable of bending that renders the problem of dock designing so difficult; and the general assumption that both the vessel and dock are flexible and bend together is perhaps the only reasonable one to make. Mr. Laws.

The one great advantage of the floating over the ordinary graving dock is unquestionably the facility with which the operation of painting and repairs can be carried out, due mainly to the supply of light, and the ease with which material can be put on the deck. In this respect, probably the L or single-wall dock has an advantage even over the double-wall type.

With regard to the design, generally, it would have been advantageous in checking some of the data in the paper had the author given even a skeleton idea of the distribution of the weight of the dock.

In the absence of this information, such a distribution might be assumed, as follows:

Weight of wall and equipment $\times$ 2 = 1 750 $\times$ 2 =	3 500	tons.
Weight of dock bottom or pontoon.............. =	7 900	"
Total weight of dock...................... =	11 400	"

These weights may be assumed to be distributed in the usual way, both transversely and longitudinally, in the sides and pontoon, respectively. Taking the length of the walls as 476 ft., and assuming that the various transverse girders carry the weight of the walls, then, by considering any one girder, the passive forces due to the weight of the structure alone are:

$$(a)\ldots\ldots\ldots\ldots\ldots P = \frac{3\,500}{2} \times \frac{8}{476} = 29 \text{ tons,}$$

which may be assumed to act at a distance of 7 ft. from the outer faces of the walls.

$$(b)\ldots\ldots\ldots\ldots\ldots Q = \frac{7\,900}{1} \times \frac{8}{500} = 126 \text{ tons,}$$

uniformly distributed along each girder.

Together with the foregoing, there are also the active forces due to the load and buoyancy, which, on the assumption of distribution stated by the author, give:

(c).....A total concentrated load on each girder = 400 tons.

(d).....A uniformly distributed upward force on each girder due to buoyancy = $\frac{31\,350}{500} \times 8 = 500$ tons.

To this must be added the balance of 3 950 tons of water (about 2 ft. deep) for the assumed freeboard of 2 ft. Of this, the weight borne by each girder = 63.2 (say 63) tons, uniformly distributed.

The whole of the forces are indicated in Fig. 22. Summarized, these forces are:

Mr. Laws. Downward forces:

Due to weight of dock walls $= 2 \times 29$	$=$ 58	tons.
" " " " bottom.........	$=$ 126	"
" " " " contained water......	$=$ 63	"
" " " " vessel...............	$=$ 400	"
Total	$=$ 647	"

Upward forces:

Due to buoyancy of water.............	$=$ 500	"
Balance (downward)	$=$ 147	"

This means that, if the assumption that the weight of the vessel distributed uniformly over the 360 ft. of keel blocks be correct, then there will be an upward supporting force of 147 tons, to account for

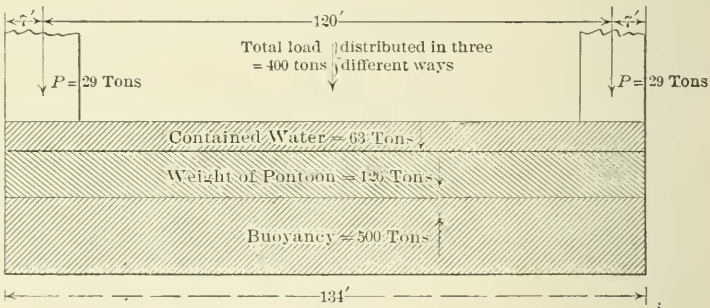


FIG. 22.

which, it may reasonably be assumed to be supplied by the walls, so that an upward force of 73.5 tons acting at 7 ft. from the outer wall faces has to be included in the calculation for strength of the transverse girders.

All these forces, collectively, give a maximum bending moment (at the center of the girder) as follows:

(*1*)—When the vessel rests on the keel and bilge blocks $= + 3\,387$ ft-tons.

(*2*)—When the vessel rests on the bilge blocks only $= + 1\,162$ ft-tons.

(*3*)—When the vessel rests on the keel blocks only $= + 7\,887$ ft-tons.

Using the positive sign with the ordinary meaning, and assuming that the bilge blocks are situated so that they support the vessel at 33.5 ft. from the center line—a reasonable figure for a warship of the displacement given—the foregoing values for the bending moment may be read from the diagram, Fig. 23, in which the bending moment curve, *A*, for the resultant load due to buoyancy and the other uniformly distributed loads, is superposed on the bending moment diagram for the three systems of concentrated loading in- Mr. Laws.

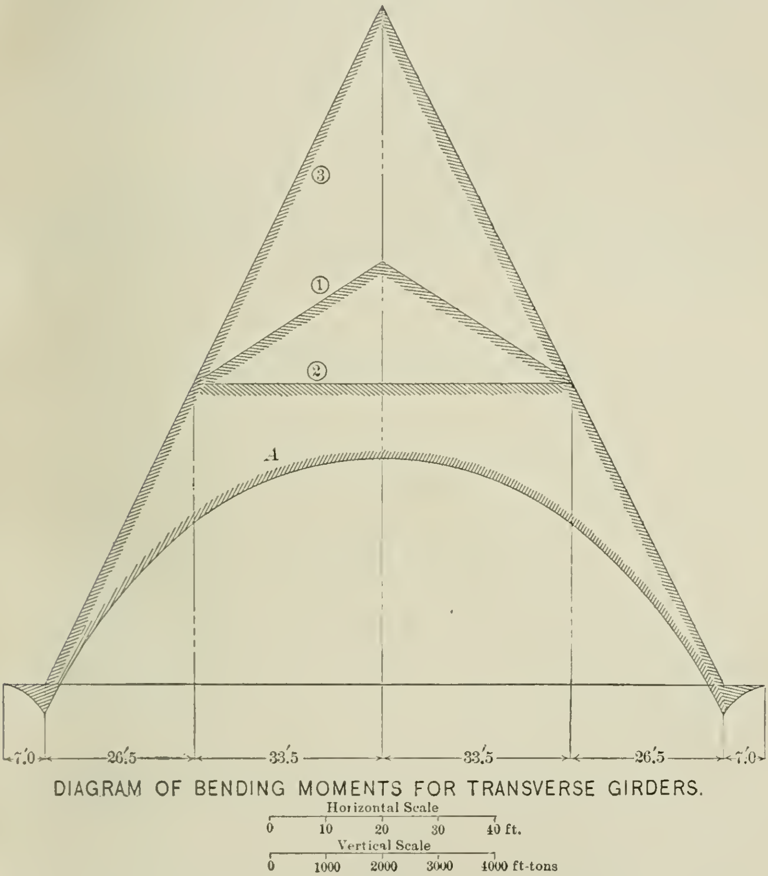
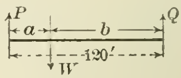


Fig. 23.

dicated above; and the resultant bending moments for the entire loading are read off above the curve, *A*.

The values thus determined differ from those given by the author, inasmuch as the latter were obtained without reference to the weight of the dock itself, or of the contained water; probably, Mr. Laws. however, a calculation was made including these items, and, if the author could state the results obtained, it would add to the value of the paper.

Upon the same assumption of loading, the deflections of the transverse girders have also been determined, the concentrated (downward) load being 400 tons, disposed in the three different ways already cited, and a resultant uniformly distributed (upward) load of 311 tons.

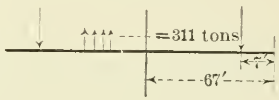


The formulas used were:

(a_1).....For a concentrated load, W; taking the origin at that point on the deflection curve in the line of action of W:

$$\text{Deflection} = T = \frac{Q}{E\,I}\left(\frac{b\,x^2}{2} - \frac{x^3}{6}\right) + \frac{P\,a^3 - Q\,b^3}{3\,E\,I\,(a+b)}\,x.$$

(b_1)....For a uniformly distributed load of intensity $= w$ tons per foot run, taking the origin on the deflection curve on the center line of the girder.



$$\text{Deflection} = \frac{w}{2\,E\,I}\left(L\,l\,x^2 + \frac{x^4}{12} - \frac{L^2\,x^2}{2}\right):$$

that is,

$$y'' = \frac{311 \times x^2 \times 12^3}{12 \times 268 \times E\,I}\,(x^2 - 21\,306).$$

where $L = 67$ ft.; $l = 7$ ft. and $w = \dfrac{311}{134}$ tons.

The deflection curves are shown in Fig. 24, from which the resultant maximum deflection—at the center of the girder—may be obtained as follows:

Where $E = 15\,000$, and $I = 1\,045\,000$.

(*1*).....Vessel resting on the keel and bilge blocks $= + 0.51$ in.
(*2*).....Vessel resting on the bilge blocks only.... $= + 0.3$ in.
(*3*).....Vessel resting on the keel blocks only.... $= + 0.89$ in.

These deflections should be reckoned for a length of 120 ft. The amount specified, *viz.*, 1 in 2 000, gives 0.72 in. for this length, a value less by a small amount than the maximum obtained by calculation according to the third condition above. It is not likely, however, that such a heavy vessel as the one chosen would, in practice, ever be supported on the keel blocks alone, but would be sustained at the bilges, more or less, a case approaching the first condition.

Mr. Laws.

DIAGRAM OF DEFLECTIONS FOR TRANSVERSE GIRDERS.

A Deflections due to vessel supported on keel and Bilge blocks, only.
B " " " " " " " " Bilge blocks, "
C " " " " " " keel blocks, only.
D " " " Buoyancy and other uniformly distributed loads, only.
E Resultant Deflections due to all loads, collectively.

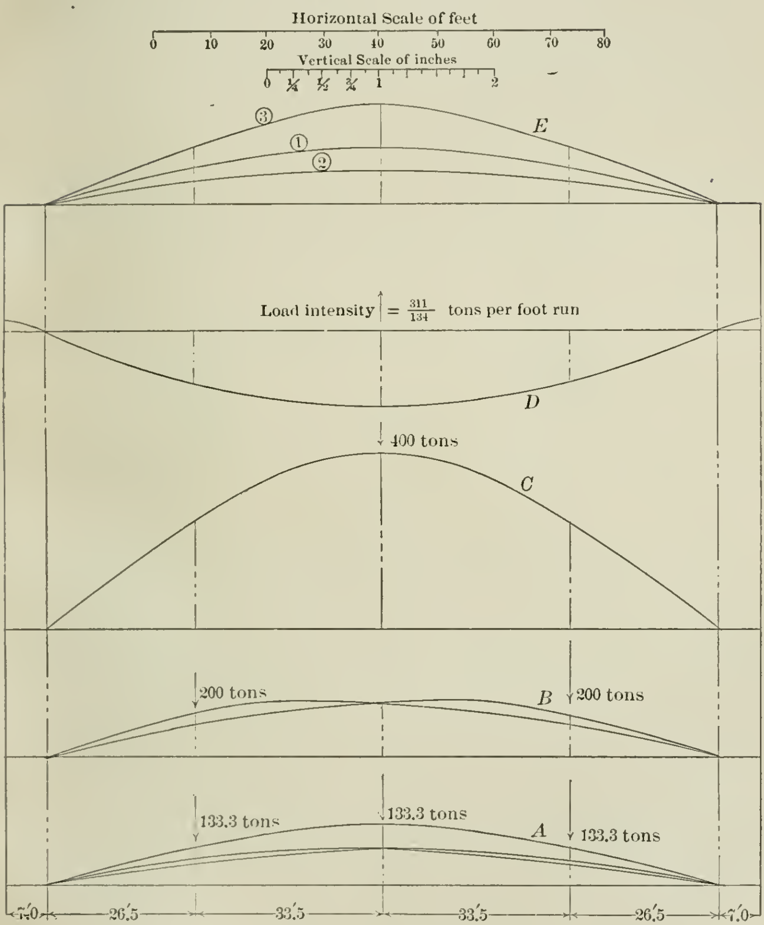


FIG. 24.

Mr. Laws. With reference to the skin plating, it would seem reasonable to treat it—with some modification—as a continuous beam bearing on broad-surfaced supports and fixed at regular intervals of 24 in. by riveting at the frames.

If a 1-in. strip of the plating between any two consecutive frames is treated as an elementary beam of length equal to the frame spacing, *i. e.*, from center of flange to center of flange, and if it be assumed that the plating between the line of rivets and the edges of the frame flange has no curvature, then it may be considered that the pressure over the supports varies uniformly from zero at a or b—the ends of the beam—to a maximum at the bearing edges, c or d, Fig. 25, and the resultant pressure or reaction will act through a point distant one-third of $a\,c$ or $b\,d$, from c or d, respectively, and the distance between these points may be taken as the virtual length of the beam to be used in the calculation, *viz.*, $22\frac{1}{6}$ in. for a $3\frac{1}{2}$-in. flange. This would mean that there is no outward pull on the rivets due to the bending of the plate over the supports, a condition which, probably, is only realized approximately in practice.

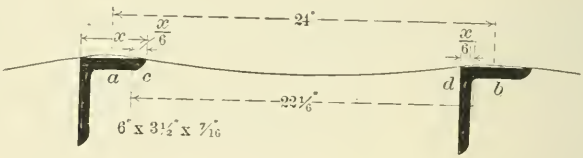


Fig. 25.

The law governing the distribution of pressure over the bearing surface, as $d\,b$, is generally unknown, and may be represented by a curve such as $b\,f$, on Fig 26, the ordinates of which at any point represent the pressure at that point. The assumption of uniform varying pressure, however, is probably not far from the truth, when the curve would become the straight line, $b\,f_1$, whereof the area, $b\,d\,f$, is equal to the area, $b\,d\,f_1$, and the resultant pressure which will always act through the center of gravity of the area of the pressure curve cuts $d\,b$, at (or about) a distance of $\frac{d\,b}{3}$ from d.

If, however, the rivets experience an outward pull, the curve of pressure will be of the nature of $e\,f$, Fig. 27, where $b\,e$ denotes that pull, and the pressure on the support will only extend over the distance, $d\,g$.

The hydrostatic pressure on the plating will be resisted partly by the stresses due to bending and partly by those due to the stretching of the plate; and if p denotes the intensity of stress due to bending only, and q denotes the intensity of stress due to tension only, in consequence of the beam when bent being longer than when straight, then both p and q should strictly be used in determining the thickness of the plating. Mr. Laws.

Theoretically, however, the formulas connecting p and q are very complicated and too unwieldy for practical use; they are built up on assumptions which are not altogether trustworthy, and, even when used in the calculation, the results obtained differ by only a small margin from those obtained by using formulas derived in a more simple way. The best assumption to make is that the ends remain undeflected when the beam bends under the load put upon it, when the latter may be regarded as equivalent to one span in a continuous beam of an indefinite number of equal spans; the "theorem of three moments" may then be applied, and the reasoning is

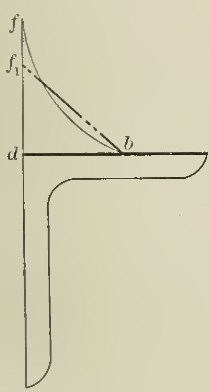


FIG. 26.

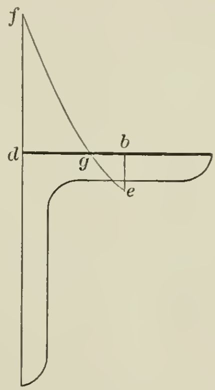


FIG. 27.

parallel with that relating to the encastre beam, in which case the distance between the edges of the supports would be 21 in. This, however, would give a result for the value of the stress less than what is probably realized, whereas the length of 24 in. would give too great a value; hence the reason for taking $22\frac{1}{6}$ in. as the length for purposes of calculation, as explained above.

In the encastre beam, the maximum value of the bending moment occurs at the edge of the support (and is twice that at the center of span), and the thickness of the plating will be given by the formula:

$$t = \sqrt{\frac{w\,L^2}{2\,k}},$$

where w is the uniform load intensity, and k is the stress intensity in the material.

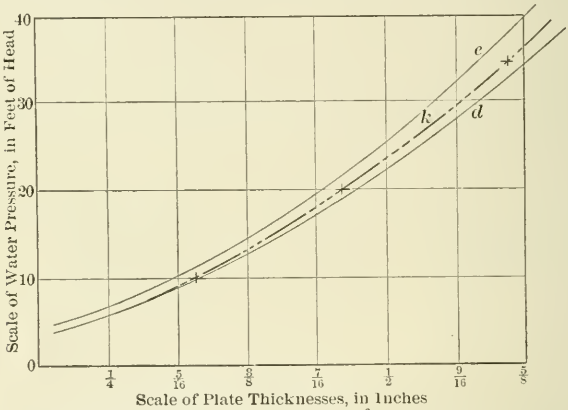
Mr. Laws. Converting w into terms of H, the head of water.

$$t = \sqrt{\frac{H L^2}{10\,322\,k}}$$

which, making $L = 22\frac{1}{6}$ in., $H = 35$ ft. and $k = 4.46$ tons, gives a value of $t = 0.611$ in. $= \frac{39}{64}$ in.; slightly greater than that obtained by the author.

Values obtained by this formula produce a curve which agrees fairly well with that obtained with Bach's formula, but it falls below the fixed-beam formula, Curve e of Fig. 28.

Space will hardly permit of a survey of the design of other portions of the structure, notably the walls of the dock, which form



(e) Fixed-beam formula: $t^2 = 0.0001\ \frac{H L^2}{k}$

(d) Rectangular plate formula (Bach): $t^2 = 0.468\ w\ a^2\ b^2 \div k\ (a^2 + b^2)$

(k) Formula: $t^2 = H\ L^2 \div 10322\ k$, where L is as defined above.

Fig. 28.

practically in themselves the longitudinal strength of the structure. The author, apparently, calculates this latter without any reference whatever to the vessel docked; some designers, however—on the assumption that the dock and vessel bend together in concert—allow a margin of strength on account of the aid rendered by the vessel to the longitudinal strength of the combined structures. The author's method is perhaps the best one to adopt, and, if it errs at all, it does so on the side of safety.

The stability of the dock forms one of the most interesting features of the design, and is indeed of the utmost importance, since upon it depends the safety of the vessel when docked. Any tendency of the dock to move away from the upright position, due to wind pressure or other causes, would at once bring unknown forces to bear on the supports, and, through them, on the structure of the dock itself, to the possible detriment of the latter, and disaster to the vessel. Mr. Laws.

The three important cases to consider are:

1.—When the vessel takes the blocks;

2.—When the keel just becomes emerged; and

3.—From the time of emergence of the keel until the deck of the dock becomes awash.

These points have been set out clearly in the diagram given by the author.

It is rather surprising, however, to find the curve of metacentric height showing so large a value at the deep drafts such as the *E* water line as compared with that indicated at the smaller drafts at *C* or *B*, but, in the absence of sufficient data, it is not possible to check the diagram.

The critical points referred to by the author are presumably those at the *C* (top of blocks) water line and the inner three points at the *B* (deck of pontoon) water line. Why the designation "critical" should be given to these points or to the period of time between *C* and *B* is not quite clear.

It is true that, with the dock lifting, the system loses the influence of the vessel as regards its stability, but, down to *C*, this influence has diminished gradually, and there is no sudden break in the curve at *C*, so that, if the stability is good for points above the level of *C*, it will be satisfactory also for points below *C*, until, when the deck emerges, there is a sudden and large increase in the value of the stability.

This is made more clear by the consideration of the fact that, with the pumping out of the water, the center of gravity of the system is continually increasing its distance above (say) the bottom of the dock, while the height of the center of buoyancy is diminishing; but the increase in distance between these two points is seldom greater than the increase in the value of the bending moment.

L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The long and eventful voyage of the floating dock *Dewey* to Manila Harbor, covering a total distance of some 16 000 miles, will always be remembered with admiration by harbor engineers the world over. It was a great feat, well performed. It is not necessary to state that repair docks, of whatever type selected, constitute an important part of the necessary equipment of every seaport. The cleaning of barnacles, sea-grass and other marine growth from a ship's bottom, after a long voyage, is always necessary if it is cared to maintain her speed. In the case of the smaller vessels, this may possibly be done on suitable tide flats, or on marine railways; but, in the case Mr. Le Conte.

Mr. Le Conte. of all large vessels, and even for extensive repairs on smaller vessels, dry docks, floating docks, or lift docks become an absolute necessity. At least one dock in every port should be sufficiently long, of ample width, and deep enough, to accommodate the largest vessel coming into that port. For convenience, it would probably be well enough to have in such a dock an intermediate gate seat to accommodate smaller vessels.

A good sheltered harbor, properly developed and well supplied with internal railway communication, together with ample facilities for cheap, rapid and reliable repairs to vessels, also with facilities for the rapid dispatch of cargoes, both in loading and discharging, will certainly benefit, not only the natural trade of the port, but also, what is most important, tend to make the harbor a port of call for a widely extended trade. Of course, the lower the harbor dues, the more inducements there will be for vessels to use the port, and sometimes the authorities in charge will find it to their advantage to make the port entirely free and charge dues only for services actually rendered, such as pilotage and cranage, with possibly some very small amount for the maintenance of the docks.

A few years ago timber dry docks were in great favor on account of their cheapness and the rapidity with which they could be built, say, in four years. Long and varied experience, however, has developed the necessity of inordinate expenses to meet annual repairs and maintenance. This, of course, takes away greatly from the usefulness of such structures, and relegates them to the class where they properly belong, namely, to temporary structures.

Now comes the steel floating dock, of 20 000 tons capacity, which can be built complete and ready for active service in 23 months. The cost is somewhere between that of the timber dock and that of the old style of granite dock. This puts the whole question in a new light, and adds much to its economic value.

Seafaring men, as a class, are very conservative, too much so for their own good, and they do not care to try anything that has not already gone through the test of widespread experience. This trait is highly commendable, if not carried so far as to destroy an officer's usefulness in the service. He should always be on the alert to pick up quickly any important improvement, and be able to analyze the true scope of its bearings on other features. Very few men have this faculty.

Every new project, no matter how well conceived, developed and carefully studied out in all its details, is always met by an endless chain of objections. Practically speaking, objections really have little weight unless it can be proved that they have some financial value. Hence the size of any objection, after all, is generally measured in the financial scale. When this test is favorable, the remain-

Mr. Le Conte.

ing objections soon disappear. Therefore, from a financial point of view, everything seems to be in favor of the steel floating dock. From a strategic point of view, it has great advantages in its complete mobility. Again, it possesses the invaluable feature that, in cases of emergency, the depth of water available over the keel blocks can be increased, at will, to 37 ft. or more, to accommodate vessels in distress, which naturally draw more water than usual when on an even keel. It seems to the writer that everything points strongly to the steel floating dock as the coming style for future harbor works, as it has so many important intrinsic advantages. Sectional docks are certainly a great improvement on the old style, as they permit of self-docking, a most important requisite. The Maryland Steel Company's type really leaves very little more to be desired.

All new docks should have an available depth of at least 40 ft. on the sill, at high water of spring tides, and a total length of from 900 to 1 000 ft. This is not too large, taking into consideration the rapid growth in the size and draft of vessels.

Mr. Pretty.

W. H. Pretty,* Esq. (by letter).—The writer has examined this paper with much interest. The author does well to emphasize the need of more and suitable repair docks, of the graving or floating type, to meet the requirements of the world's shipping. Their value, in time of peace or war, cannot be overestimated in the development and defence of the commerce of any country. The mobility of the floating dock, and the fact that it can be constructed at a convenient economic base and towed to its destination, ready for immediate use, are conditions greatly favoring this type of dock; and when, combined with this, one considers the ease with which it can be put out of action, temporarily or finally, to prevent it from falling into the hands of a powerful enemy, it is not without strategical importance.

In the historic portion of the paper, Mr. Cox has not mentioned a certain useful type of graving dock, in which the natural fall of a river, a rapid, or a tidal difference of level is utilized, the available supply of water being used to dock ship, and the natural fall to a lower level to drain the dock, no pumping machinery being used. A few small docks of this class, and apparently of ancient date, existed in Cardiff, some years ago. It may not be without interest to mention here that in the Village of Willington, Bedfordshire, England, on the River Ouse, about 4 miles from Bedford, there exists, in a good state of preservation, the remains of a Danish camp and dockyard, presumably a repairing yard.

It does not follow that there is always more water to pump from a "graving" dock, for, in the case of a ship absolutely fitting the

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Mr. Pretty. dock, there is no water to pump out—this is the analogue of Lewis Carroll's "bath" in a minimum quantity of water. In a floating dock of the same lifting capacity, under similar conditions, the whole of the water corresponding to the displacement of the dock and ship, or the "maximum" load, will have to be dealt with by the pump.

The elastic deck of a floating dock is unquestionably a valuable feature, from a shipowner's or underwriter's point of view, and might be copied, in principle, in the construction of the floors of graving docks, with advantage to the vessels docked therein.

It is interesting and satisfactory to notice that the value of docking keels, or virtually "three lines of keel blocks" was recognized in the construction of the *Dewey* dock, and it is to be hoped that all vessels, in the near future, will be designed to meet this method of docking; it seems almost barbarous, even to think, that vessels should be now designed for docking on any other plan, and to see a large warship going through the old-fashioned process of shoring on a floating dock gives food for much thought, while the time thus lost is of the utmost importance, to say nothing of the risk to the 800 or more souls on board. On the other hand, one is tempted to ask for a few more data as to the distribution of pressures, and the means taken to allow the vessel to bed herself without excessive stress in any place. Something is known of the distribution of stresses when on a single line of keel blocks.

It should not be forgotten that a timber dry dock can be destroyed more readily than a masonry structure in time of war, should such a course become necessary.

There is no question that all floating docks should be self-docking; the best means of doing this, however, is still a debatable question. The plan adopted for the *Dewey* is excellent, but the design of the end pontoons does not appear to the writer to be by any means ideal, although the general scheme is good. The water-tight freeboard should always be sufficient, and distributed so as to guarantee absolute control in sinking or raising the section under manipulation; but, if the writer understands Fig. 20 correctly, this does not seem to have been done.

In painting the surface of steel structures generally, sufficient importance is not attached to freedom from moisture in the paint used, or the absence of water or moisture blisters between the clean metal and the outer surface or film of paint, often introduced by unskilful laborers, it being a general impression that anyone can use a paint brush.

The author's comparison of graving and floating docks, on page 105, is open to criticism, in the particular case in question. When docking a small ship in a graving dock, she can be run over the sill at a comparatively low draft on a rising tide, and only the water thus admitted need be dealt with, whereas, in the case of the floating dock, the minimum weight to be dealt with is that of the dock plus the ship. Mr. Pretty.

The only fair way of comparing the work done in pumping during docking ship, in graving and floating docks, is by electrically-driven plant, when the whole record of kilowatt-hours would tell the story, the real question at issue with respect to the pumping plant in docking ship is the total energy in foot-pounds, foot-tons, or kilowatt-hours (say) consumed from the moment the ship touches the blocks, to the deck or floor awash in floating and graving dock, respectively. There is no excuse for the non-introduction of electrically-driven pumps, either in graving or floating docks, while this form of energy can be utilized for all purposes; and a central station on a floating dock would place all the pumps under one controlling center, not even excluding the independent docking sections, which could be fed by special cables.

It would add greatly to the value of the paper if a plan and section of the No. 4 dry dock (mentioned on page 106) were given, to enable a calculation to be made for comparison with the figures shown. The writer would ask Mr. Cox to confirm these figures, particularly the 58 700 tons of water which, it is understood, are to be removed below the level of the top of the blocks. From a rough estimate, the writer would expect this graving dock to be capable of dealing with a 25 000 to 30 000-ton ship.

In reference to the remarks on page 109, a disabled or leaky ship very frequently has a list, and it is probable that sectional docks, in which one section can be detached, floated under the ship and then have its buoyancy added to assist the disabled portion, would do more good for temporary assistance, or say, "first aid." It would be interesting to have some records of ships with a list docked in floating docks. On the other hand, the value of a floating dock for deep-draft vessels is unquestionable.

The method of self-docking adopted in the *Dewey* is ingenious and simple, and possesses considerable merit. Plate XXIII conveys the impression that the side walls are undercut outside below the dock level, while the letter press appears to indicate that the side walls are continued vertically downward outside to the level of the keel plates of the dock. The writer would like Mr. Cox to explain this a little further. The solid-trough dock, although stronger, might be less valuable in time of war than a sectional dock, particularly in dealing with torpedo-boat destroyers, or submarines.

The specification of steel on page 122 should state a definite ratio of measured length to diameter in measuring the elongation, Mr. Pretty. and in the case of other forms of section, an equivalent ratio of length to sectional area should be stated, otherwise the firm possessing the most powerful testing machine will score. Flat steel plates should be bent, cold, 180° in and also at right angles to the direction of rolling, or, "with the grain," and "against the grain," as it is sometimes expressed.

The provision of 12 in. of water in the bottom of a pontoon dock when the ship is docked is wise, as it gives some little control over the pumping operation toward the end of the pumping. In the writer's opinion, the depth should be increased for docks used in choppy seas, as any surging tends to uncover the suction pipes, and the nearest pump may lose its water, and race away. Under ideal conditions this trouble does not arise, but in the absence of suitable gauges for indicating the depth of water in the various pontoons, a depth of water sufficient to guarantee the suction pipes against drawing air, with reasonable attention at the pumps, should be allowed for. There does not seem to be any description of a water-level indicator, or of an air inlet, but presumably these were used.

Mr. Cox. Leonard M. Cox, M. Am. Soc. C. E. (by letter).—All who are interested in the subject must highly regard opinions based upon the experience of Mr. George B. Rennie, the dean of floating dock designers. His discussion has historic value, and it is especially interesting to note that, notwithstanding the numerous devices proposed for self-docking floating docks, the apparent tendency to-day is to return to the solid one-piece type represented by Mr. Rennie's Cartagena dock.

Mr. Baterden's fair discussion of the relative merits of the two types of docks under varying conditions, seems to call for some explanation on the part of the writer. While confining itself generally to the subject of floating docks, with special reference to the design and construction of a particular structure, the paper has failed to attain the object for which it was written if it stands as an argument in favor of floating docks, as opposed to graving docks, for any and all situations and requirements. On the contrary, a frank discussion was attempted on the idea that "each type has its own particular field of usefulness which the other cannot with advantage fill, and that, for a given set of conditions, a careful study of both types, as applied to the special requirements of the case, must govern the choice." This, it would appear, is Mr. Baterden's position, while admitting a bias in favor of the dry dock in the majority of instances.

It was not the intention to state as a general law that the cost of a graving dock is greater than the cost of a floating dock, but that, for equal capacity and in situations equally favorable or unfavorable to each type, the floating dock "costs about the same, and in some instances less." Table 1 is unsatisfactory in that it fails to give the cost, completed, of three of the new docks. These data were not available at the time the paper was written, but it has since been learned that the total costs of the new Charleston and Norfolk docks are $1 085 273.00 and $1 201 347.82, respectively, the former less and the latter more than the cost of the *Dewey*. The new docks at League Island and Mare Island yards exceed the *Dewey* in first cost, while no estimate of the completed cost of No. 4 Dock, New York, can be made. The new dock at Boston cost $1 105 000.00, or $19 000.00 less than the *Dewey*. The *Dewey* at the site and on the date of the official trials cost $1 143 959.68, of which amount all above the contract price, $1 124 000.00, was expended for very desirable though not absolutely necessary improvements, such as compressed air plant, bitumastic enamel paint for pontoon bottoms, etc. Mr. Cox.

While on the subject of comparative costs, it may not be amiss to point out the fact that the 20 000-ton battleships, so the writer has been informed, will have a beam of about 85 ft. 2⅝ in., and a length of 518 ft. over all. The *Dewey* can easily dock these ships with 7 ft. clearance between the sides of the ship and the painting stages. Table 4 gives the docking capacity of the only United States docks capable of admitting ships of this size, and it will be noted that only two of the docks listed are now completed.

TABLE 4.—Docking Capacity of Largest Naval Docks.

Dock.	Draft over sill, M. H. W.	Maximum Ship.		Remarks.
		Length, over all.	Beam, in feet.	
Portsmouth No. 2	30 ft. 0 in.	728 ft. 9 in.	86	Completed 1906
Boston No. 2	30 " 7½ "	727 " 0 "	86	" "
New York No. 4	31 " 0 "	518 " 9 "	88	Under constr.
League Island No. 2	30 " 0 "	729 " 10 "	86	Nearly complete.
Norfolk No. 3	34 " 0 "	529 " 8 "	96	Under constr.
Charleston No. 1	34 " 0 "	554 " 0 "	96	" "
Mare Island No. 2	30 " 0 "	726 " 4 "	86	Nearly complete.
Puget Sound No. 2	38 " 0 "	655 " 0 "	98	Construction not yet begun.

Note.—A clearance of 2½ ft. has been allowed forward to head of dock and 2½ ft. aft to overhang of caisson. Maximum length of ships has been taken on line 15 ft. above bottom of keel. At the entrance, net clearance 4 ft. each side at M. H. W., ship drawing 29 ft. New ships (20 000-ton) to be approximately 518 ft. over all, have 85 ft. 2½ in. draft; 27 ft. 0 in. normal and 29 ft. 3 in. maximum.

The writer agrees with Mr. Baterden in the opinion that the floating dock is not suitable for locations requiring extensive dredging with periodical re-dredging, unless other considerations warrant the necessary additional expense both in first cost and in maintenance.

Mr. Cox. The high range of tides existing on the coast of Europe and the British Isles does not obtain in America, and was not considered in the comparison of pumping required for each type of dock. Such a condition would undoubtedly affect the quantity of water to be pumped, in the case of the graving dock, yet, even under the most favorable circumstances, considering the variation in the size of the ships docked, the average pumping expense for a definite period of time would, in the writer's opinion, be appreciably less for the floating structure.

For naval purposes, the advantage of the quality of mobility would be valued more in times of war than in times of peace. At such times the matter of towing expense or the cost of ancillary structures, should such be necessary, might be ignored in view of the strategic importance of a required move. For commercial purposes, it is not proposed that floating docks be moved at all, except for weighty reasons; such, for instance, as an entire change of plant location, impending danger from water-front conflagrations, or other equally important considerations. The necessity for moving the dock would evidently occur infrequently, but, if it occurred once during its lifetime, its mobility might be the means of increasing the possible earnings, or even saving entirely a large investment.

Mr. Peabody very justly criticises the unfortunate obscurity of certain parts of the calculations. It should have been stated on page 127 that the position of the bilge blocks was assumed from the preliminary design of the 16 000-ton class battleships. As Mr. Peabody infers, they were spaced 22 ft. from the keel blocks. The limits of a paper of this kind make it necessary to omit much that might be of interest, and tempt the writer to general statements of methods rather than detailed computations. This may account for the omission of the calculations for stability with ship 1 ft. off center, as well as the stability under the specified wind. As Mr. Peabody has remedied the defect as regards the former, and as the general method for obtaining the latter is indicated in the paper, they will be omitted from this discussion.

The value of such a paper as the one referred to by Mr. Peabody (Boobnoff's "Stresses in a Ship's Bottom Plating Due to Water Pressure") is undeniably great, and it may yet be the means of assuring to the profession an easily handled tool, but it is not too much to say that the computations contained therein will hardly come into general practical use in their present form. The results of the experimental work of Captain Hovgaard, of the Massachusetts Institute of Technology, will doubtless prove of great value, judging by the thesis of Assistant Naval Constructor Ferguson, U. S. N., presumably referred to by Mr. Peabody. The ordinary procedure, of consulting Lloyd's or other Marine Under-writers' tables of scantlings, is the simplest and possibly the best way at present of determining detailed dimensions, but the great stumbling block to their use, from the standpoint of the engineer, is that he is not always able to discover just how their results were obtained. Few of us can use formulas or tabulated results without having a clear idea of their derivation or source. Mr. Cox.

If, as Mr. Colson points out, "it appears desirable that the power of docking at full-load draft at low water of spring tides should be available," would not the requirement greatly increase the cost of a graving dock in localities where, as in the British Isles, the range of tide is great? It is admitted that the same objection would hold for floating docks if deep water near shore is not available, but, given the depth of water, the floating dock is the obvious solution of the problem.

The writer agrees with Mr. Colson that one of the weakest points of the floating structure is the difficulty of handling heavy weights from the side walls. This problem will doubtless be solved, but, up to the present time, no satisfactory arrangement for heavy deck cranes has been evolved, owing to the interference with lines, stacks, guys and general deck gear. As regards fire, it is conceivable that a ship in dry dock might be seriously threatened by burning shop buildings, whereas in a floating dock on the waterfront it would be out of danger, or if in danger, could be saved either by sinking the dock and moving the ship, or moving the dock and ship together.

The cost of preparing berth, shore connections and permanent moorings of the Algiers floating dock was included in the contract price. The cost of the same work for the *Dewey* cannot now be given, as permanent plans will depend upon the final arrangement of the naval station at Olongapo or Cavite.

While actual cases of a heavy battleship being docked with a list, by trimming the pontoon to accommodate the list, are not known, the Algiers dock has successfully performed the operation with commercial vessels, and, in a similar manner, a careful dockmaster should be able to dock a warship by bringing his dock back to an even keel shortly after the ship takes the blocks and before it has lost much water support. Of course, there would be a limit to the amount of list that could be taken out in this way, but, even thus limited, circumstances can be imagined which would make this a valuable quality in a dock.

Mr. Cunningham points out a new use for the floating dock, which, from a naval standpoint, would seem to be a most important one. The possibilities of the floating dock as a dry dock auxiliary should appeal strongly to legislators who have to do with appropriations for public works. If the Government must provide docks

Mr. Cox. for the heaviest ships at abnormal drafts, all the naval docks of the United States would eventually have to be enlarged or replaced at enormous cost, while, since the heaviest ships are few in number and the condition of abnormal draft rare, every purpose would be answered by few large floating docks distributed at suitable points.

The independent caisson proposed by Mr. Cunningham appears to meet the requirements of the case so fully that experiments with a large or full-sized model will be awaited with much interest. The supplemental chamber for the control of the buoyancy is novel. To avoid surging in submergence, he would doubtless arrange the chamber or chambers so that the admission of water could be controlled for trim—an object easily accomplished. Could not this caisson be fitted with a movable deck for a working float, and on its tower have cranes installed with reach sufficient to handle heavy weights over the dock's side wall when alongside? Also, could not such a caisson be designed for easy towing, that is, fitted with bow and stern in the direction of its length? If so, one such would answer all requirements for docks proposed for the Atlantic Coast, one for the Gulf and one for the Pacific. If, by the use of this device or some other, the objections to the solid dock can be overcome, the result will be a stronger and stiffer dock, and, what is equally important, increased confidence on the part of the dock-master.

Mr. Clark's discussion of the question of skin plating brings out with greater clearness the point to which the writer desired to call attention in his paper, and he has endeavored to illustrate this by a comparison of actual examples of ship practice, in his reply to Mr. Laws. There is no doubt that the use of beam formulas for skin plating gives stresses largely exceeding any to which the material is ever actually subjected. As stated in the paper, the fact that beam formulas gave a "considerable excess of strength" was known and duly considered by the designers, but the extra thickness obtained by their use was considered desirable for other reasons. The formation of a protecting marine growth before the thickness of metal is reduced to that absolutely necessary for the stress may be the means of effecting a saving in self-docking expense, besides insuring against neglect. Docks are not intended to show speed qualities, therefore marine growths are of no particular detriment to steel bottoms except by reason of the added weight.

There is also a general tendency in the United States toward the liberal use of material and large safety factors in anticipation of remote contingencies, which, in older and more heavily taxed countries, probably seems like extravagance, and in the future this may seem to be so here. As an indication of the wisdom of this policy, the case of the Havana dock may be cited. There were two years of the life of this dock during which it was badly neglected. The Spanish Government did not expect to retain it, and the United States had not determined upon its purchase. When the matter of its purchase finally came up, a board of officers was ordered to examine and report upon its condition; it was found that in the foul waters of Havana Bay the plating had scaled to a depth of from $\frac{1}{8}$ to $\frac{3}{16}$ in., all the under-water rivet heads had swelled and opened like chestnut burrs, and the whole under-water body had deteriorated. It was the opinion of the board that the lifting capacity of the dock had been reduced from 10 000 to 6 000 tons. Now, with an allowed unit stress of 13 tons per sq. in., the formula for skin plating used in the dock calculations gives $\frac{3}{8}$ in. instead of the $\frac{19}{32}$ in. thickness actually used. This leaves $\frac{7}{32}$ in. of extra metal, which exceeds by a very little the $\frac{3}{16}$ in. of corrosion found in the Havana dock. Mr. Cox.

As regards the "bolted sectional" dock of Messrs. Clark and Stanfield, the writer readily admits the possibility of making a joint of a structure stronger than the body of the structure itself, but, considered merely as types, it would seem that the one-piece side walls with overhanging ends bearing on short end pontoons would be considered stronger than a dock composed of square-cut sections joined to each other at their edges. Mr. Clark's statement, that these sections are sometimes riveted, would lead to the conclusion that frequent self-dockings are not contemplated.

The inspection of the Havana dock, prior to its purchase, led to the recommendation that all connection bolts be placed above the normal water line, as under-water bolts caused much annoyance in self-docking, and could rarely be withdrawn in a usable condition. While it is true that the *Dewey* at the time of her tests was new and in good condition, it may be worth while to note that, in the self-docking operations, every bolt was withdrawn and reinserted without the aid of mauls, and that not a bolt was lost. Besides the objection to them in self-docking, under-water bolts must be a constant source of anxiety, since their condition after long immersion could never be ascertained with certainty.

Though a docking deck only 12 ft. in length would seem to be inadequate for the seating of a long and heavy section, the fact that the pointed ends project some 50 ft. farther under the lifted section removes doubt as to the possibility of safely self-docking the bolted sectional dock. It is understood that the existence of a basin dock at Pola capable of docking the sections will probably prevent a practical test of the self-docking features of the Pola bolted sectional dock. Experience with the Detroit dock, of the same type, and with the new docks mentioned by Mr. Clark, will

Mr. Cox. be awaited with interest, and will doubtless remove many of the objections suggested by a mere consideration of the scheme on paper.

Paragraph 42 of the specifications provides that, "Within the limits of allowed deflection the shipload shall be assumed to be perfectly flexible." As limited, the assumption is substantially correct. Mr. Clark seems to interpret the term "flexible" as meaning flexible without elasticity, and to overlook entirely the limits of the assumption set forth in the specification. This assumption is made for bending and not for shear. As a crude illustration, it would take the slightest transverse pressure to deflect a taut string 1 in., but a very appreciable force to deflect it 16 in.

The uniform pumping clause in the specifications was inserted to insure against danger from unskillful operation. While the official tests required uniform pumping, it was not contemplated for ordinary docking operations, and an intelligent dockmaster would as surely regulate the buoyancy according to the loading as in any other type of dock. The square ends and uniform buoyancy permit the docking of a ship having a keel length equal to the length of the dock, and, as far as was learned from the voyage of the Algiers dock, the square bow detracts nothing from the towing qualities.

Mr. Clark's comparison of the deflections caused in H. M. S. *Dominion* and the U. S. S. *Iowa,* when docked in the Bermuda dock, and the *Dewey,* respectively, would have been of more value had the length of those ships been given. The Naval Pocket Book of 1904 gives the length of the *Dominion* as 425 ft., and that of the *Iowa* as 360 ft. In comparing the performances of the two docks, it should be borne in mind that, in the test of the *Dewey,* the dock was raised by uniform pumping, and, furthermore, in order to allow for the difference in weight between the *Iowa* and the specified 16 000-ton ship, the dock was raised to a freeboard of 4½ ft. instead of 2 ft., as specified. During the docking, the exceedingly high temperature which obtained was broken by a sudden rain of short duration; these conditions gave quite a variation of temperature, and the deflection ranged from 1.8 in. to a maximum of 4 in. There is no doubt that temperature strains are induced under such conditions which might well warrant the low unit stresses adopted. The whole of the deflection could easily have been eliminated had it been permissible, under the regulations of the official tests, to use discretion in pumping. With the *Dewey,* as with the docks cited by Mr. Clark, it is quite easy to hog a slightly sagged ship by manipulation of the pumps and valves.

Mr. Clark's objection to the trough or solid dock, namely, that there are locations where basins could not be built capable of holding the entire structure, would hardly hold good in many parts of the United States, though it might prove a serious obstacle abroad. Nevertheless, it would seem that broadside launching could be accomplished without extraordinary risk, and that in a country with a great range of tides, as in the British Isles, there would be few locations which would not offer some solution of the problem. At all events, it might be worth the trouble and extra expense to attempt the solution. Mr. Cox.

The writer desires, in passing, to acknowledge his obligation to Mr. Clark for his frank discussion of the paper, and to express the high estimation in which he holds any opinion given by that eminent engineer.

Mr. Laws' discussion of the question of transverse strength and deflection is pertinent, and his novel treatment of stresses in skin plating forms a valuable addition to the literature of the subject.

As regards the method of determining bending moments and deflection of transverse girders given in the paper, it should have been there stated that the weight of the girder was disregarded in the desire to attain maximum stiffness. This concession on the part of the contractor was offset by a similar one on the part of the Bureau of Docks in allowing the use of the unit stress specified for self-docking operations for all calculations involving the assumption that the weight of the ship is carried on keel blocks alone. These provisions, together with others affecting methods of conducting tests, etc., were incorporated in the contract as "Contractor's Supplemental Specifications." This will also explain the fact that in the paper only the calculated deflection for ship on keel and bilge blocks was given. It should be stated, perhaps, that computations, to all intents and purposes identical with those suggested by Mr. Laws, were made, and as the net result of allowing for the weight of the pontoon and side walls is the reduction of the end upward forces by an amount equal to the weight of the sides, both the bending and deflection under ship loads are reduced. Inasmuch as experience with the Algiers dock led to some doubts of the practical value of theoretical methods of determining deflections, owing to the extraordinary effects of temperature changes, it was thought best to err on the side of safety and make for increased stiffness.

Mr. Laws' method of determining the thickness of the skin plating is ingenious, and his assumptions appear to be reasonable, yet, as the results obtained by its use are even greater than those obtained by the fixed-beam formula with span measured from edge to edge of frame flanges, it would seem that efforts to reconcile formulas based upon the theory of beams with current ship practice are far from being crowned with success. In order to

Mr. Cox. illustrate the difference which exists in the methods for determining the thickness of plates under water pressure, Table 5 is made up from ships actually in service or under construction, and represents the best shipbuilding practice of the day. The age and service of the majority of these ships would seem to be a fair guaranty of safety.

The results shown in Table 5 would indicate that the stresses in at least one instance equal the elastic limit of the materials, a condition of affairs which not only does not obtain, but which, in all probability, is not even approached.

Mr. Laws' criticism of the use of the term "critical points" in connection with the stability of the dock with ship is deserved. It was not the intention to convey the idea that at any normal position of the dock, either with or without ship, its stability is, in the ordinary sense, "critical," but that the stability is least at or between the points indicated.

The writer agrees with Mr. Le Conte in the opinion that there should be adequate berthing and docking facilities at every accessible port, and there is little doubt that, with harbor fees reduced or remitted entirely, the effect of such facilities would soon be felt, not only locally, but nationally.

Mr. Le Conte's recommendation as to the dimensions of new docks appears to be extreme. For commercial purposes, graving docks of this size would require expensive pumping on account of the small number of maximum-size ships which would be docked. The depth of 40 ft. on the sill at high water would be an advantage, but it is doubtful if, at the present time, the additional expense is warranted. For naval purposes, this depth is certainly desirable, but the length should be cut down to about 650 ft. and improvement should be made in obtaining additional width of entrance.

The writer agrees so nearly with Mr. Box that he finds it difficult to reply to his discussion. The omission of the off-shore dock as an important, if not the most important, type of dock for commercial purposes, is a defect in the paper. This dock, on account of its economy and rapidity of operation, has held a high place in the esteem of the dock owners abroad, and will undoubtedly grow in favor in America.

The comparison of quantities of water pumped from graving and floating docks drawn by Mr. Box is interesting in that his operations cover a large number of actual dockings, and necessarily include vessels varying in size.

The building basin was originally constructed for the Algiers dock, which was completed in 1902. This was followed by the *Dewey*, begun in July, 1903, and launched in June, 1905. Since the completion of the *Dewey* the basin has been out of commission, Mr. Cox.

TABLE 5.

Vessel.	Head at maximum draft.	Displacement: maximum draft, in tons.	Greatest unsupported rectangle $a \times b$ – square inches.	Average thickness of skin plating at rectangle, in inches.	Unit stress formula: $t^2 = 0.0001 \frac{H\ L^2}{K}$ ($l = b - 3$ in.).	Unit stress: Bach's formula for rectangular plates.
Dock *Dewey*.....	35 ft 0 in.	....	96 × 24 = 2 301	19/32	4.1 tons.	5.33 tons
Ship *A*..........	22 " 9 "	13 000	132 × 48 = 6 336	19/32	13.07 "	13.00 "
" *B*..........	20 " 3 "	13165	102 × 42 = 4 284	19/32	8.5 "	8.51 "
" *C*..........	22 " 8 "	12 500	204 × 42 = 8 568	5/8	13.54 "	12.74 "
" *D*..........	25 " 7 "	12 965	90 × 48 = 4 320	5/8	13.26 "	11.63 "
" *E*..........	26 " 3 "	11 650	84 × 48 = 4 032	5/8	13.61 "	11.55 "
" *F*..........	26 " 1½ "	11 700	108 × 48 = 5 184	5/8	13.54 "	12.74 "
" *G*..........	18 " 2 "	14 680	168 × 48 = 8 064	19/32	10.43 "	10.86 "
" *H*..........	26 " 9 "	15 900	72 × 48 = 3 456	5/8	13.87 "	10.81 "
" *I*..........	26 " 9 "	17 900	69 × 48 = 3 312	5/8	13.87 "	10.52 "
" *J*..........	15 " 2½ "	5 700	192 × 36 = 6 912	1/2	6.62 "	7.85 "

Mr. Cox. but it is understood that its use is contemplated for the construction of barges or other light-draft craft.

Mr. Box evidently had the off-shore dock in mind when stating that the Clark and Standfield arrangement of pipes was adopted for the *Dewey*. In former two-sided docks it was customery to place pumping elements in each side wall, with pipes leading inboard to compartments, thus dividing control. In docking large ships on the Algiers dock it was found that, with the ship between, it was only possible for the dockmaster to be in sight of both valve houses when he was stationed on the forward bridge, while the noise and confusion of the operation rendered telephones and annunciators of little use. Therefore, it was recommended by the testing board that control of all valves and pumping gear be concentrated at one point. The success of this plan was very evident at the docking tests of the *Dewey*.

Mr. Pretty's addition to the history of docks is of interest. While localities in America where purely tidal docks could be utilized are rare, they are occasionally met with abroad, and very probably represent the earliest form of the graving dock known, following as the first step in the evolution from tide flats.

While the analogue of the bath in a minumum quantity of water illustrates Mr. Pretty's point, it also confirms the statement made in the paper, which was to the effect that the quantity of water to be pumped in the case of the floating dock varies with the size of the ship, while in the case of the graving dock the variation is in inverse order.

While the surging of the end pontoons during the self-docking was not sufficient to cause the slightest alarm, yet, as stated in the paper, the lack of symmetry of the horizontal water plane cut when the pontoon is submerged is in the nature of a defect in design which may possibly be remedied in future docks.

The importance of a dry paint and a dry surface on which to apply it cannot be too strongly urged, and is second only to the importance of removing scale, oil or grease. Skillful application often causes an inferior paint to show better results than a high-grade article poorly applied.

The 58 700 tons, in the calculation on page 106 represents the total weight of water contained in the dock prism from floor of dock to mean high-water level. The main dimensions of this dock are as follows: Length on coping, head to sill, 542 ft.; length on floor, head to sill, 516 ft.: length on floor, head to abutment, 501 ft.; width of entrance at coping, 103 ft. 10½ in.; width of entrance at bottom, 73 ft. on flat, with corners rounded on a 10-ft. radius; width at coping, in body of dock, 130 ft.; width on floor, in body of dock, 78 ft.; depth on sill, at mean high-water, 31 ft.; distance from coping to mean high water, 5 ft. 8 in.; greatest depth from coping to floor, in body of dock, 39 ft. 8 in. From these data the writer's results may be roughly checked. Mr. Cox.

The photographs of the self-docking operation reproduced in Plate XXIII convey a wrong impression. The side walls are vertical, but the discoloration of the under-water body with the fender emphasizing the water line makes the walls appear to be undercut.

The air inlet pipes are located on the inner faces of the side walls, and are therefore not shown in photographs. Mr. Pretty makes a pertinent inquiry in asking for details of the indicator system required in Paragraph 58 of the specifications. The only reply possible is that all efforts to perfect a practical scheme resulted in failure. A number of devices were tried on an experimental scale, and some few gave promise of success. A system was finally installed on the dock, but, though it gave reliable results in slow tests, it proved absolutely untrustworthy in rapid docking operations.

In closing the writer desires to express his gratification on account of the interest shown in the subject as evidenced by the number and character of the discussions submitted.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS

Paper No. 1043.

RECENT PRACTICE IN HYDRAULIC-FILL DAM CONSTRUCTION.*

By James D. Schuyler, M. Am. Soc. C. E.

With Discussion by Messrs. Clemens Herschel, William L. Butcher, T. G. Dabney, Frederic P. Stearns, and James D. Schuyler.

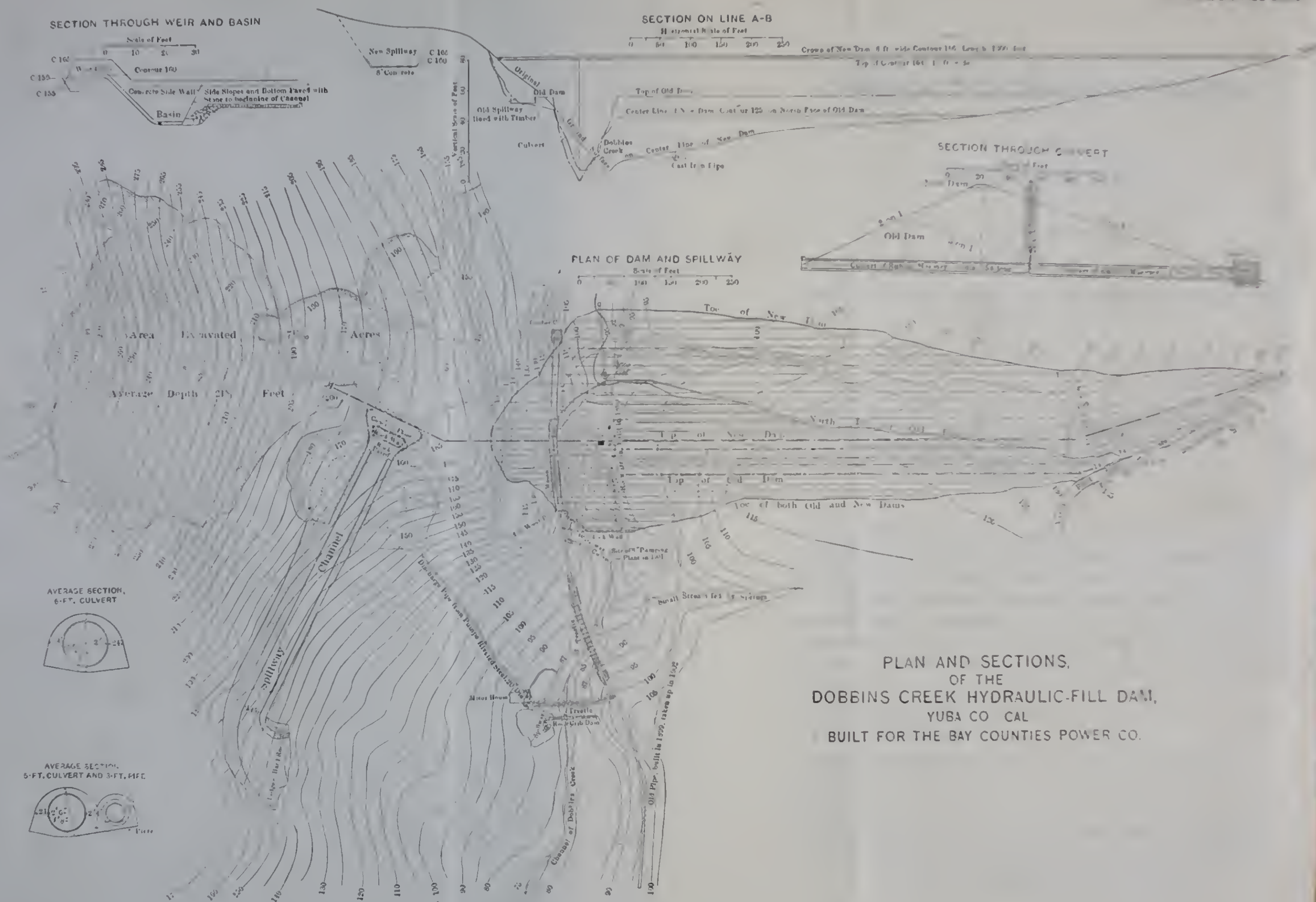
The widespread interest which is being taken by engineers throughout the world in the methods used in Western America in the construction of dams for water storage, by the agency of water for loosening, conveying, assorting, distributing, depositing and consolidating the materials, has prompted the writer to prepare an account of his experience in this novel class of construction, and to gather such data as were obtainable from the experience of others.

The principles primarily involved in the design of any earth dam are the following:

First.—It must be founded on an impermeable foundation, and form a water-tight connection with the rock or clay bed on which it rests.

Second.—It must be practically impervious to water, in the whole, or at least in a goodly portion, of its entire cross-section.

* Presented at the meeting of December 19th, 1906.



PLAN AND SECTIONS,
OF THE
DOBBINS CREEK HYDRAULIC-FILL DAM,
YUBA CO. CAL.
BUILT FOR THE BAY COUNTIES POWER CO.

Third.—It must have slopes sufficiently flat to be stable under all conditions of saturation from the water in the reservoir, or from soaking rains.

Fourth.—The elevation of its crest above the highest water line must be sufficient to insure against the possibility of the dam ever being overtopped by extraordinary freshets, due to cloud-bursts, or to waves driven up its inner slope by gales of wind, or to a combination of these contingencies.

Fifth.—It must not settle, crack, or show any sign of change or movement after final completion, and when put into service.

These requirements may be fulfilled in building earth dams by the usual methods of moistening, rolling and tamping, by proper precaution in the selection of materials, and by the exercise of great vigilance and care in construction; and, where the dams are of moderate height, not exceeding 30 or 40 ft., they are generally successful, with rational precautions in the important matter of size of spillway, even if the materials are not placed with such extreme care. Where the height exceeds 100 ft., however, these methods, in all ordinary situations, lead to excessive cost, and a form of construction is requisite which will permit of the proper gradation of materials and their perfect consolidation in a manner which will insure water-tightness, avoid future settlement and, at the same time, accomplish the handling of large masses of material rapidly and at moderate expense.

It is believed that this cheaper and more efficient method has been found and demonstrated in a definite and positive degree by the perfected system of construction known as the "hydraulic-fill," which had its origin in the hydraulic mining regions of California. In this system the entire work is done by the agency of water, which is brought in large volume to the dam site in ditches and pipes, and delivered through the nozzle of a flexible and easily controlled device called a "hydraulic giant," or "monitor," with which hydraulic mining is performed. The stream, issuing from the nozzle of the monitor with a velocity of from 100 to 200 ft. per sec., is directed against the face of a hill containing the materials desired for use, and, by undermining, cuts and loosens everything in its path except the solid rock, and even this, if seamy or stratified, cannot resist its force. The mass, thus disintegrated, is rapidly washed with the water down the steep sluiceway provided, and is conveyed through flumes and pipes to the several points on the dam selected for the deposit of the different grades of material, their final position depending upon their respective degrees of fineness.

Having had occasion to use this system in several important dams, the writer has been urged to give the profession the benefit of his experience in this somewhat novel type of construction. This he cheerfully does in the following paper, prepared in the few and widely separated leisure moments of a busy life, mostly on railroad trains, or trolley cars.

The Lake Frances Dam.

As an example of unusual difficulties and adverse conditions successfully overcome where other methods have failed, the repair and enlargement of the Lake Frances Dam by the hydraulic method is, perhaps, the most conspicuous which could be selected.

This dam is on Dobbins Creek, Yuba County, California, about 35 miles from Marysville and 2 miles from the Colgate power-house of the Bay Counties Power Company (subsequently merged with the California Gas and Electric Corporation), and was built originally by that company, in 1899, to form a small reservoir for emergency service. It is on a little tributary stream, the water-shed of which is only 6.5 sq. miles above the dam, from which the run-off yields a discharge fluctuating from practically nothing to an extreme flood flow of about 1 000 cu. ft. per sec. The dam is about 400 ft. in elevation above the penstock of the power-house, which receives its water supply from the North Fork of the Yuba River, through a wooden flume, 9 miles long, built in the rocky cañon of that stream. The flume is subject to occasional interruption from falling rocks, and, in such an emergency, the small reserve storage of the Lake Frances Reservoir is drawn upon for a few hours at a time in order to maintain the load. The original dam of 1899 was built with a maximum height of 50 ft., a top length of 992 ft. and side slopes of 2 on 1, outside. and 3 on 1 on the reservoir side. The crest width was 16 ft., at a height of 4 ft. above the spillway level. The latter was excavated to a width of 40 ft., in earth, and was lined with a wooden flume of 2-in. plank.

FIG. 1.—BREAK IN ORIGINAL LAKE FRANCES DAM. LOOKING UP STREAM.

FIG. 2.—NEAR VIEW OF RIGHT SIDE OF BREAK IN EMBANKMENT OF LAKE FRANCES DAM, SHOWING ROOTS EXPOSED BY THE BREAK.

The reservoir formed by this dam covered an area of 42.67 acres, and had a capacity of 30 545 000 cu. ft.

The site for the dam, as well as a large part of the reservoir basin, was heavily timbered with pine forest when construction began, and had to be cleared and grubbed to get at the material in the borrow-pits.

The embankment, for two-thirds of its length from the south end, was built with slip and wheel scrapers, the earth being spread in 6 and 8-in. layers, and moistened and rolled in the customary manner. The remainder was built late in the season, when the water supply had practically failed. No attempt was then made to place the material in layers; in fact, toward the end there was such haste to finish that the earth was dumped in the most convenient way, as in an ordinary railway embankment. Much of the surface of the steep hill slope on the north end was not even cleared of stumps and roots, as discovered subsequently. As the dam had been designed by a reputable firm of engineers, of San Francisco, and built under their supervision, it was considered to be as secure and perfect in construction as any dam of that character could be, when made by such methods and with the kind of earth available. This earth consisted of red clay and gray sandy soil, resulting from the disintegration of syenite devoid of mica. In the valley proper it contained a slight admixture of gravel and enough vegetable mould to constitute a dark loam. On the hill slopes, where the mass had not been disturbed since the processes of Nature had converted it from soft rock to clay and sand, the material was composed chiefly of red clay, containing angular fragments of rock, with occasional streaks of gray sand. The clay was unstable when saturated, but very firm when the surplus water had been drained from it. The sand would stand vertically in excavations.

Rupture of the Dam.—A few days after the dam had been completed, and following a rainfall of 9 in. in 36 hours, the reservoir filled very rapidly, and when the water had risen to within 6 ft. of the spillway, or 10 ft. below the crest, at 11 A. M., on October 21st, 1899, it was suddenly ruptured by a great crack at the north end, due to settlement, which allowed the water to escape with rapidly accelerating force, creating a gap, 98 ft. in width, measured along the crest of the dam, and from 30 to 40 ft. wide at the bottom.

The first alarming symptom noted by the engineer in charge was a considerable leakage along the outlet pipes at the toe of the dam. A few minutes later, a large stream appeared near the steep bank, more than 100 ft. west of the original creek channel, and about 20 ft. above the base of the dam. This enlarged with great rapidity, extending to the top, and, accompanied by a great roar of rushing water, scoured a new channel deeper than the old bed of the stream. The reservoir was emptied within 1 hour to within 23 ft. of the 16-in. scour pipe at the bottom.

Strangely enough, a triangular section of the embankment resisted the erosion of the escaping waters, and held back a pond covering more than 10 acres and containing 2 500 000 cu. ft. This natural weir in clay soil had a sharp crest at the upper slope line, over which the water dropped almost vertically into a hole 28 ft. lower, which had been excavated by the flood some 7 ft. deeper than the original creek channel. By building a temporary flume, 12 ft. wide and 4 ft. deep, at an angle of 45°, from the lip of this earthen weir into the hole below, the weir was preserved, and subsequent floods of the seasons of 1899-1900 and 1900-01 were carried past the dam in that way, with the aid of a plank fence on top of the weir.

The volume of the original dam was 80 265 cu. yd., of which 16 160 cu. yd., or 20%, were washed away by the break and had to be replaced.

Repair and Enlargement of the Dam.—During the summer of 1900 the writer was engaged to report on the repair of the broken dam, and, after an examination of the site and the materials available, he recommended that the hydraulic sluicing process be used to loosen, convey and solidify the earth, provided that further experiments should demonstrate the soil to be suitable. This latter qualification was due to the fact that, in its dry state, the clay, when placed in water, exhibited no plasticity, but dissolved and floated off in a muddy cloud, although when moist it was solid and firm. It should be explained that in the long, dry summer, without rain for months, the clay soils of California lose their moisture entirely, and crack to great depths, unless protected from the sun. When tested in this condition, by immersion, the clay seemed to be devoid of all cohesion, and was apparently about as promising a material for dam building as so much pea soup. When settled and drained,

Fig. 1.—Lake Frances Dam. Appearance of the Old Dam and Empty Reservoir. August 31st. 1901.

Fig. 2.—Masonry Outlet Culvert Built in Lake Frances Dam.

however, the clay, thus treated, resumes its tough, plastic and impervious characteristics.

The date of this report was so late in the season that it was evidently inadvisable to assemble a plant and begin the repairs that fall, the rainy season being too near at hand. In the following spring, the subject was again taken up, and the writer was asked to take charge of the construction. He associated with himself J. M. Howells, M. Am. Soc. C. E., who made an independent investigation and report in the latter part of March, 1901, corroborating the writer's recommendation to use the hydraulic method of handling the work. This report was accompanied by such a conclusive series of tests of material taken from test-pits as to set at rest any fear of unfavorable results from the use of water in doing the work. In this examination Mr. Howells was assisted by F. S. Hyde, Hydraulic Engineer, who had been Engineer-in-Charge of the construction of La Mesa Dam, California, and who, as Resident Engineer, subsequently conducted the work on the Lake Frances Dam to completion. Mr. Hyde is now carrying on similar construction for the Mexican Light and Power Company, Limited, at Necaxa, Mexico, where the two highest earth dams in the world are being built by this process under the supervision of the writer.

Mr. Howells, who had had previous experience in the building of two important dams by this process, at Tyler, Tex., and La Mesa, Cal., regarded it as unwise to repair the dam by simply filling the gap, as it would subject the remainder of the embankment, which had already failed at one point, to the possibility of a repetition of the same disaster, as well as subject the old and proposed new parts of the dam to unequal settlement. Therefore, he recommended that a heavy layer of earth be placed against the upper slope, of sufficient thickness to give an impervious core of selected fine clay between porous zones of coarse, stable material—one of which, next to the dam, should be of sufficient thickness to drain properly the clay core down along the slope of the old embankment to a stone drain, planned to be carried along its upper toe, and connected with a drainage pipe which had been provided originally for carrying off some little springs in the base. The outer permeable layer of stone and sand was required to give stability to the 3 on 1 slope of the outer face. These three layers, in his judgment, required an aggregate thickness of 125 ft., measured horizontally. This plan, if carried out simply as a repair of the old embankment, would have left the dam with a crest width of 141 ft. With this broad base it was evidently safe to add 27 ft. to the height of the dam. To make such an addition involved the handling of only an additional 81 371 cu. yd. with the same plant that would have to be installed for placing the 133 782 cu. yd. in the break and north face. It was shown that, by this addition to the height, the storage would be increased from 30 540 000 to 101 951 000 cu. ft., extending the area of the lake to 92.33 acres. The extension of the height proposed increased the top length of the dam to 1 300 ft.

The super-elevation of the crest over the spillway was to be 6 instead of 4 ft., as originally built, and it was recommended that the new spillway be increased to a width of 80 ft. in addition to a 5-ft. culvert through the dam near the bottom, to be opened on a rising flood before the reservoir was entirely full. The spillway will be described later in more detail. These recommendations were concurred in by the writer, and the plans being accepted by the company, the work was ordered to be commenced at once.

Hydraulic Sluicing with a Pump.—One of the obstacles encountered in carrying out the hydraulic method of construction was the lack of a gravity water supply at sufficient elevation to do the work of sluicing. For the most rapid and efficient service, water should be used under a pressure of from 100 to 150 lb. per sq. in. at the nozzle of the monitor, and in a volume of from 10 to 15 cu. ft. per sec. In this case the only water available was Dobbins Creek, the flow of which is reduced for months during the dry season to about 1 cu. ft. per sec., and at times runs as low as 0.06 cu. ft. per sec.—a mere trickle. To meet this emergency, a small crib dam was built some 300 ft. below the main dam, by which 150 000 gal. were accumulated before sluicing began, and a two-stage tandem centrifugal pump, which had been nearly completed on a previous order from the company for use near Colgate, was ordered finished for this service. This pump had a capacity of 6 cu. ft. per sec., under a pressure of 120 lb. per sq. in. at the pump, delivering through a line of 20-in. pipe, varying from 300 to 700 ft. in length.

As a considerable length of time would be required to secure the delivery of the pump and motor. it was determined to commence

FIG. 1.—LAKE FRANCES DAM. SLUICING PLANT USED WITH SIX-INCH CENTRIFUGAL PUMP FOR FILLING THE BREAK.

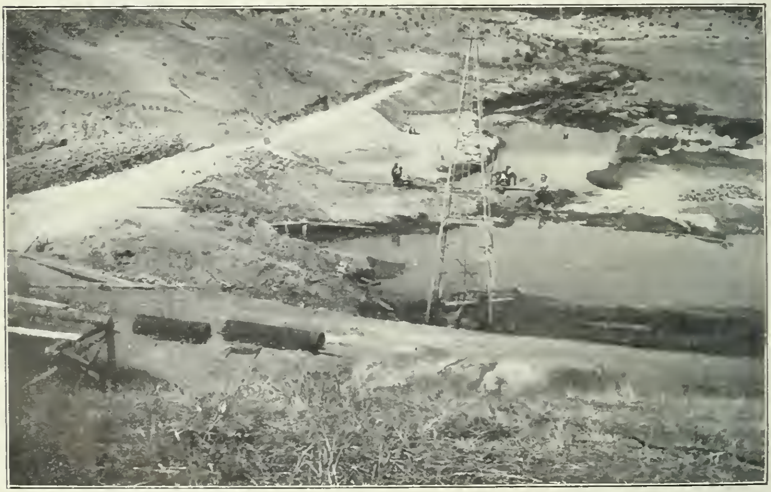


FIG. 2.—LAKE FRANCES DAM. LOOKING UP INTO THE RESERVOIR FROM THE TOP OF THE ORIGINAL DAM. AUGUST 31ST. 1901.

sluicing into the break with a smaller pump and motor, which could be more readily secured and quickly installed. A 6-in., single-stage centrifugal pump was obtained, was direct-connected to a 30-h.p. motor, and supplied with electric current from the Colgate power-house, 2 miles distant. With this primitive apparatus the work of sluicing began on May 10th, 1901. This pump delivered 1.76 cu. ft. per sec., under a head of 100 ft., and did very good service for more than a month. The muddy water draining from the dam, and overflowing from within the levees thrown up on each slope in the break, after depositing its load of earth, was caught in the little reservoir below, where it was pumped over and over again. At times, the flow of the stream was little more than sufficient to equal the loss by soakage into the earth, and maintain the supply. Several times the water became so muddy as to add measurably to the load upon the motor, and to require momentary stoppage in order to permit the mud to settle somewhat. This shortage of water occurred after the larger pump had been installed. There was an abundance of water during the time the smaller pump was in service—May 11th to June 15th, 1901.

In this way 4 090 cu. yd. of filling in the repair of the break were deposited between May 10th and June 15th, at a cost of 18.27 cents per cu. yd. This was obtained by excavating the hill slope immediately adjoining, and was sluiced into place with 11-in. pipes. This brought the work up to the level of the new masonry culvert, and sluicing was suspended, awaiting the completion of the culvert and the erection of the permanent and larger pumping plant. During this work, done under most disadvantageous conditions, the average ratio of solids deposited to water pumped was about 13 per cent. After August 30th, when the larger pump was installed, the material was delivered through a 22-in. riveted steel pipe, laid on a grade of 3%, and the repair of the break was completed on October 17th. The overflow from this work went to the north face and deposited 5 220 cu. yd.

New Outlets to the Reservoir.—The outlets to the original dam consisted of two 36-in. cast-iron pipes, laid in trenches, 15 ft. apart from center to center, about 100 ft. east of the creek bed, and a 16-in. riveted steel "scour pipe," laid at a lower elevation in the creek channel. The two larger pipes were provided with sluice gates, 4 ft. square, placed at the upper toe of the dam, and were to be operated from a light steel tower standing in the water. This tower was erected and the gates in position at the time of the break, but no attempt was made to raise the gates at the beginning of the flood, as the danger was not realized. When the reservoir had partially filled and the pressure increased, it was found that the device for raising the gates was so insufficient that they could not be opened, and the dam would probably have been overtopped had the break not occurred.

The 16-in. scour pipe was provided with a steel tower similar to that controlling the main gates, which was subsequently buried in the embankment, as shown by some of the accompanying photographs.

After the failure of the dam it was found that the cast-iron pipes were badly broken by the pressure of the earth and the softening of the foundation under them. The location of maximum settlement was under the crest of the embankment, and resulted in destroying four joints of pipe in one line and eight or ten in the other. The uninjured pipes, for a length of four joints at each end, were subsequently dug out, and most of them were relaid as a permanent outlet, reaching from the gate chamber to the outer toe of the dam, a distance of 145 ft. The remaining sections, each 180 ft. long, were plugged with clay and left in place.

Beginning at the outer toe of the dam, the new outlet culvert of masonry, before referred to, was built through the embankment at an elevation of 19 ft. above datum (which is 2 ft. above the lowest contour of the reservoir) on a bench cut in the side of the steep bank of the creek in partly decomposed and partly solid rock. It is circular and 6 ft. in diameter as far as the gate chamber, a distance of 135 ft. from the crib or grillage of timber at its upper end. The gate chamber was placed 82 ft. north of the center of the enlarged dam, its position being determined by the location of a hard bit of bed-rock on which it was placed. From the gate chamber outward, a distance of 150 ft., the waste culvert was reduced to a diameter of 5 ft., and consisted of rough rubble, laid in Portland cement, the exterior being made as irregular and rough as possible, so as to form a bond with the puddle of the dam, while the interior was well plastered (Fig. 2, Plate XXVIII. and Fig. 1,

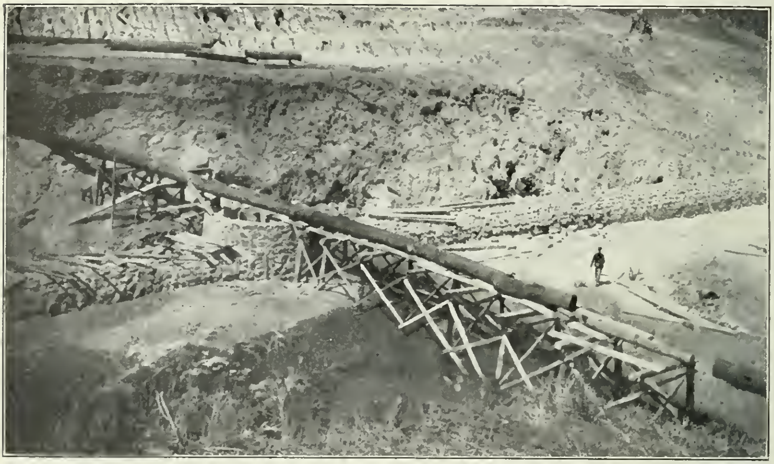


FIG. 1.—LAKE FRANCES DAM. SLUICE PIPE ALONG INNER TOE OF ORIGINAL DAM, FOR DEPOSITING COARSE, POROUS MATERIAL.

FIG. 2.—LAKE FRANCES DAM. SLUICE PIPE USED WITH SIX-INCH PUMP, SHOWING GRIZZLY AT END.

Plate XXIX). The 36-in. outlet pipe laid adjoining the culvert, between it and the high bank, was surrounded with masonry, like an arch, and was separated from the pipe by a small space of 1 in. at the bells.

The 4-ft. gates, removed from their original position, were placed side by side in the gate chamber, one of them connecting with the 5-ft. waste culvert, which discharges freely at the outer toe of the dam, and the other with the 36-in. cast-iron pipe, which, in turn, was subsequently coupled up with a line of wood-stave pipe, 24 in. in diameter and 2 miles long, which conveys the water to a chute leading down to the penstock of the power plant.

The gates were covered with a hemispherical dome of masonry, through which the gate stems were carried to the top of the dam in 6-in. pipes supported by a trestle tower. The culvert, pipe arch and gate-chamber required 522 cu. yd. of masonry, in which 480 bbl. of cement were used. The entire cost of this outlet was $8 540.50, including the reclamation of pipe, gates and towers, the building of the inlet grating, 16 by 16 by 12 ft., the resetting of gates, and the relaying of pipe, but not including the cost of the cement, which increased the total to about $11 000.

There were special reasons justifying this expense, one of which was the desire on the part of the management to be able to waste water in considerable volume at any stage of water in the reservoir, so as to anticipate a great flood, such as produced the previous disaster. Another reason was the treacherous character of the material through which the spillway had to be constructed. It was manifest that had the water ever reached the original spillway, and gone over it for a few hours, it is probable that it would have washed out a hole similar to the one made in the dam, as the flimsy wooden lining would have been easily undermined by leakage through, under and around it, and thus have been rapidly destroyed. Fortunately, it was never put to the test, and the lumber reclaimed from it proved to be useful in subsequent work.

The Main Sluicing Operation.—The outlet culvert having been partially completed, and the pump and motor finally delivered and erected (after long delay due to a general strike of machinists in San Francisco), sluicing operations were resumed on a larger scale on August 30th, 1901, by which date it had been hoped to have nearly completed the dam. The work had been in progress for 5 months and it had been expected that it would be finished before October, consequently the delays were exasperating and costly. The pumps were belt-connected with a 350-h.p. synchronous motor, which required an exciter and a smaller motor to start it. Electric current was supplied at 2 400 volts, by a 2.5-mile transmission line, built direct from the Colgate power-house. The pump suction was connected with the adjoining sump in dry weather, but later, when water had accumulated in the restored reservoir, increased head was given by connecting with the wooden discharge pipe leading to Colgate. When the new pumps were erected, with sufficient power to give the head needed for more efficient execution, the monitors were moved up to a new position at a higher level than could be reached by the small pump, and the excavation was directed so that the new spillway channel would be opened out for future use. To this end, one line of pipe, with a monitor attached, was laid so as to make a face in the south end of the spillway next the pump-house, and another, branching from the main, attacked the north end. These were not put into action simultaneously, but, by having two lines of pipe to be used alternately, the necessary change of position could be made without other interruption than that of moving from one line to the other the one monitor provided.

The first work of the new plant was to complete the filling of the gap in the original dam, and 9 620 cu. yd. were then deposited from August 30th to October 17th. This work was much hampered by the contracted area to which it was reduced, requiring periodical suspension of work in order to allow for proper settlement and drainage. Levees, 1 or 2 ft. in height, were maintained along the up-stream and down-stream slopes, and were composed of the most stable material brought down by the water, while the fine clay mud was deposited in the pond of water confined between these levees. The excess of water was drained from this pond by flumes at one end of the work, or by a pipe siphon, or both, and returned to a pond at the north side of the dam, in the zone called the "North Face." By this means 5 220 cu. yd. of material were collected from the overflow.

Sometimes, however, the mass became so soft as to cause slips and sloughing down the slopes, due to the lack of sufficient coarse

FIG. 1.—LAKE FRANCES DAM. LOWER FACE OF DAM AT BREAK. RIP-RAPPED WITH ROCK.

FIG. 2.—LAKE FRANCES DAM. SLUICING ON THE BREAK APPROACHING THE TOP.

gravel, rock and sand to afford the desired friction and requisite drainage. This generally occurred during the temporary absence of the engineer, and was caused by the use of material on the face of the dam which was not sufficiently porous. During all this filling of the break, as well as during all subsequent work, the water contained in the fine mud which composed the bulk of the interior of the dam, was constantly being pressed out, as the settlement of the mass continued, and appeared as a continuous ooze over the entire face of the slopes.

The theory upon which hydraulic-fill dams are generally planned and attempted to be carried out is about as follows: That the inner third of the dam should be composed of impervious material, or material which, by drainage and natural settlement, should consolidate into a mass which will become impervious to water, and remain in a moist, semi-plastic condition; that the outer half of each of the other thirds should be coarse, porous, open material, through which water, draining from the interior, would pass freely; while the inner halves of the outer zones should be a mixture of the coarse and fine, or a semi-porous material, in condition to act as a filter so as to prevent the escape of any of the fine particles from the inner third, but at the same time allow of the slow percolation of water through it.

In this case, however, the great length of the dam, and the necessity of securing all the material from one side, compelled the use of such low gradients that but little of the rock encountered in the borrow-pits was available. As clay was the preponderating material, there was a constant lack of sufficient rock and gravel to keep the slopes from slipping and sloughing, and finally it was found necessary to resort to the use of brush to maintain slope stability while the embankment was settling and draining.

Team Work.—During the filling of the break, a low levee was thrown up, with teams and scrapers, at the toe of the north face, 125 ft. from the toe of the original dam, to form the pond to retain the overflow previously referred to, and the use of teams was continued for the maintenance of the outer slope levee until November 22d, 1901, when brush was adopted for slope building, its use being continued until the dam was finished.

Pine and cedar boughs, and young trees, some 6 ft. long, were laid with the butts in, and covered with earth shoveled from the pond. There was no trouble with sinking slopes subsequently, and, although it made a somewhat unsightly and rough exterior appearance, the brush performed the service of knitting the mass sufficiently to prevent sliding. Its use was also a measure of economy, as well as necessity, for heavy rains had made the borrow-pits, from which the teams had been getting gravel and sand, quite unapproachable.

During the entire progress of the work, a pond of water and thin mud, from 1 to 5 ft. deep, was maintained on top of the rising dam, and the reservoir was allowed to fill behind it and remain at a height of from 8 to 11 ft. below the top. Thus the stability of the structure was constantly undergoing a crucial test. At one time, when the water had reached a height of 25 ft. above datum, and was suddenly lowered 5 ft., a few cracks appeared on the inner slope, from 40 to 100 ft. long and from 3 to 24 in. wide, caused by the supersaturation of the surface material on the water slope which had not reached its final settlement. These were filled with small stones, and gave no further trouble. It was this manifestation which confirmed the necessity for the use of brush, for which authority had previously been given by the supervising engineers. Had there been from 20 to 30% of gravel, or even less, in the mass deposited on the slopes, this sloughing or slipping on a 3 to 1 slope would not have occurred; or, if it had been possible to keep up dry levees at the edges of the bank with scraper teams, the stability of the slopes would have been secured; but heavy rains and deep mud made it impossible for teams to go out on the embankment for this purpose, and hence brush was the only substitute.

The general plan of building the north slope was to deliver the sluiced materials, as heretofore stated, through a 22-in. pipe, placed on a high trestle, running parallel with the axis of the dam, and far enough inside the slope lines to reach the slope with lateral flumes of moderate length. The posts of these trestles were necessarily left buried in the embankment which rose around them. The trestles were of varying length, according to the height of the dam, and had an average height of 25 ft. The first trestle, 215 ft. long, was completed on October 25th. The second was finished on November 16th. The third, 6 ft. higher and 150 ft. longer than the

Fig. 1.—Toe Levee of North Face, Lake Frances Dam, and Inlet Crib at Head of Six-Foot Outlet Culvert.

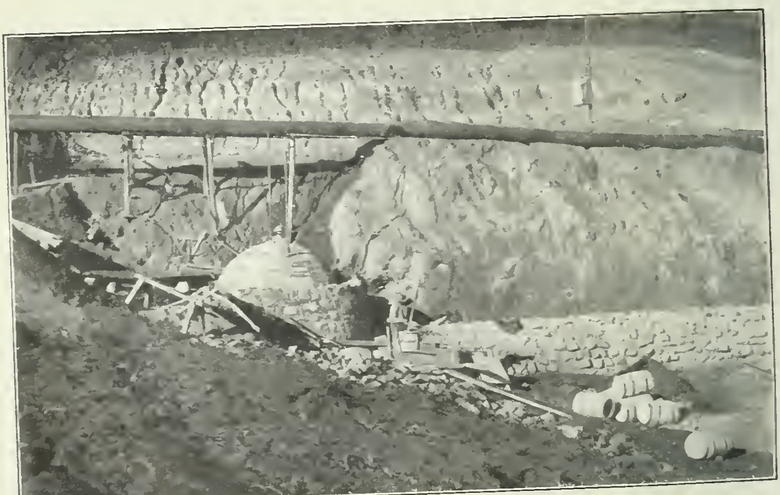


Fig. 2.—Dome on Gate Chamber, Lake Frances Dam: Also Twenty-two-Inch Sluice Pipe.

second, was completed on November 30th. The fourth, 850 ft. in length, built on a grade of 1.7%, was finished on December 16th, and a week later was raised to give the sluice pipe a grade of 2.2%, on account of clogging. This pipe was 42 ft. in elevation above the base of the dam at its delivery end. To complete the north face, a fifth trestle was erected, with a length of 1 150 ft., on a grade of 2.2%, discharging at an elevation of 59 ft. above datum. This work occupied two weeks, and was finished on January 21st, 1902. The final trestle, with which the additional height of 27 ft. was built, was begun on April 17th and completed on May 2d. Its entire length was 1 560 ft., besides thirteen branch trestles and flumes, reaching to the slopes on either side, as shown in Fig. 1, Plate XXXVII. The highest bent in this trestle was 40 ft. above the dam, and there were 100 bents in the main and 33 in the lateral trestles.

The pipes laid on the trestle were joined by key bands of sheet iron about 4 in. wide, made so as to be quickly unjointed by driving out a key and slipping off the band. Thus delivery of a part or the whole discharge of the pipe could be made at any point desired. Bent or curved plates placed under the joint served to deflect the stream of earth-laden water to the steep lateral flumes reaching to the slope levee.

Work continued throughout a winter of very inclement weather, during which the rainfall amounted to 52 in. Much of the time a night shift worked by electric light, except on very stormy nights. A storm on March 1st was accompanied by high winds, which blew all the pipes off the trestles, one of them being buried and lost in the deep mud. After that the pipes were wired down.

The North Face filling was completed to the level of the top of the original dam on February 24th, 1902, and the entire work of extending the dam to a height of 77 ft. above datum was finished on June 30th, 1902.

The volume in the completed dam, including the original dam, is 280 700 cu. yd., as measured several months after completion, when settlement had ceased; of which amount, 182 937 cu. yd. were deposited in the period of 253 days, between October 19th, 1901, and June 28th, 1902.

The aggregate of the weekly measurements taken during this period was 195 293 cu. yd. of material deposited, or 12 356 cu. yd. in excess of subsequent measurement in the bank.

A record of the actual time of sluicing shows an aggregate during this period of 1 581 hours, or a little more than 25% of the total time. Omitting Sundays and half the nights, the sluicing, in shifts of 10 hours per day, was carried on in about one-half the maximum available time. The other half was lost by reason of stoppages required for building trestles, and also from lack or shortage of power, as various accidents at the power-house compelled a suspension of pumping for periods of several hours and even days.

The volume of water used was carefully measured, and found to vary from about 4.5 to 7.0 cu. ft. per sec. The total quantity pumped in the period stated was estimated at 30 740 000 cu. ft., while the volume carried by the water and deposited in the dam amounted to about 4 940 000 cu. ft. of solids, or 16.6% of the water. This was in addition to about 2 or 3%, carried in suspension, which finally drained off in the tail-race and back into the reservoir.

The weekly percentages of solids carried by the water varied greatly with the conditions encountered. When the power was not restricted, and the water pumped was not overloaded with mud, when the bank against which the monitor was cutting exceeded 25 to 30 ft. in height so that the jet would do rapid execution, and when stones and roots were not too abundant in the pit, and the delivery pipe did not clog on account of too light a grade, the material poured in rapidly, and the ratio was maintained at from 20 to 30%, and ran from 32 to 38% for several weeks. The highest of all showed 47.7 per cent. The minimum week's run gave 6.1% average solids. The best week's work was 22 350 cu. yd., deposited in 94.5 hours, or 236 cu. yd. per hour. During that week 6.5 cu. ft. of water per sec. were used, and the ratio of solids to water was 27.2 per cent. The average work performed was 127.3 h. p., while the recorded electric energy consumed was 176 kw., or 236 h. p., showing a combined efficiency in pump and motor of 53.9% (motor 90%, pumps, 59.9%).

The maximum quantity of material moved as a weekly average was 443 cu. yd. per hour; the minimum, 47 cu. yd. per hour, and the mean, 123 cu. yd. per hour.

Fig. 1.—Lake Frances Dam, Showing Delivery of Water and Earth from Pipe on North Face.

Fig. 2.—Lake Frances Dam, November 6th, 1901. From South Borrow-Pit, Looking Along Axis of Original Dam.

The maximum pressure shown by the water gauge at the pump was 118 lb. per sq. in.; the minimum, 98 lb.

The maximum pressure recorded at the nozzle of the monitor was 75 lb. per sq. in.; the minimum, 38 lb.

The maximum power, measured by electric ammeter at the pump, was 324 h. p., and the mean, 236 h. p.

The total power used during the period covering these statistics, embracing almost the entire work, was estimated at 410 800 h.p-hr. At a cost of ½ cent per h.p-hr., this would amount to a total of $2 054, or about 1 cent per cu. yd. of material moved.

The minimum cost for labor on any one week's work averaged 3.8 cents per cu. yd., sluiced and deposited in the dam. The average labor cost was about 15 cents per cu. yd., and the total cost was probably less than 20 cents per cu. yd., including all power, materials and plant. Under more favorable conditions, such as could be secured in many other localities, the work could have been performed for a fraction of this cost. These improved conditions, easily attainable in many localities, may be briefly noted as follows:

(1).—Constant, uninterrupted, electric current, in sufficient volume to do the work required, if the water be supplied by pumping, but preferably a gravity supply of 10 to 15 cu. ft. per sec., or more than double that which was available in this case, giving greater carrying power and doing more than double the work with the same force of attendants;

(2).—Shorter top length of dam, permitting the use of steeper gradients in the sluice-boxes and delivery pipes, and consequently giving higher velocities, and ability to move much of the rock which in this case had to be left in the pit;

(3).—A larger proportion of sand, gravel or broken spawls of stone less than 10 or 12 in. in greatest dimensions, which would have rendered the slopes stable without the use of brush, and would have avoided the expense incident to the cutting and placing of brush, or the building of levees with teams.

The fact that a considerable portion of the filling was made at a cost of less than 5 cents per cu. yd. is an indication of what ought to be achieved on a larger scale, with all materials favorable, a gravity water supply of adequate volume and uninterrupted delivery, and a plant planned for and perfectly adapted to the work.

In the excavation of 10 540 cu. yd. from the spillway, the actual cost was only 1.4 cents per cu. yd., while the channel below the spillway, excavated with the hydraulic monitor to the extent of 9 150 cu. yd., cost 3½ cents per cu. yd. Under ideal conditions, this low cost might be maintained as the average of an entire embankment.

Settlement.—The Lake Frances Dam was constantly undergoing settlement as the water was being squeezed out of it. This was observed by the distortion of the flumes and pipe trestles, and the slope boards on the slopes. The greatest amount of settlement, measured on the posts of trestle No. 5, was 3.43 ft. From July 8th to July 28th—20 days—the maximum settlement observed was 0.4 ft. The original dam, after being re-saturated with water from the wet embankment built against and over it, settled very greatly, and reached a maximum subsidence of 2.5 ft., although it had been exposed to the rains of two previous winters, and was supposed to have reached its limit of settlement before the reconstruction work was commenced.

A measure of the compactness of the earth put in by this method, compared with the same material in its original condition in the bank, was afforded by exact measurements of the main borrow-pit and the embankment made from it. Contours of the original surface were carefully taken before excavation and again afterward, and the areas and volume ascertained by planimeter. The results showed that the material in the dam was only 82.6% of the volume taken out of the borrow-pit. A portion of this difference of 15.4% was material carried away in suspension and deposited in the reservoir. Some tests of this overflow waste indicated that it might have averaged 4.4 per cent. The remaining 11% of shrinkage by the solidifying action of water is suggestive of the practical efficiency of this method of handling earth, although there is some uncertainty as to the estimate of average volume of overflow. Under unfavorable conditions it may reach as much as 35% of the total solids carried, as shown by the fact that, while 9 620 cu. yd. were being deposited in the repair of the break above the level of the

FIG. 1.—LAKE FRANCES DAM. JANUARY 29TH, 1902. TRESTLE NO. 5. 1 150 FT. LONG; DISCHARGE END AT ELEVATION 59 FT. ABOVE DATUM.

FIG. 2.—LAKE FRANCES DAM CONSTRUCTION. PUMPING STATION. SHOWING SUCTION PIPE CONNECTED TO TANDEM CENTRIFUGAL PUMP. CRIB DAM IN FOREGROUND BUILT TO STORE WATER FOR EARLIER OPERATIONS.

culvert, 5 220 cu. yd. were deposited from the overflow on the North Face. From the general appearance and condition of hydraulic-fills generally, it seems doubtful if it be practicable to secure such density and compactness of earth by any other process, at many times the cost.

To build the dam, a borrow-pit, covering 7.12 acres, was excavated to a mean depth of 21.5 ft.

Precautions Required.—The plan of dumping all the sluiced material at the edges of the slopes resulted in building them up rapidly, while the gradients toward the center were flattened as more and more of the coarser particles were dropped, and, with a greatly lessened grade, the water, after depositing all but the finest mud, moved slowly toward the farther end, where it was returned to the reservoir. The 2 or 3% of the mud which it still carried into the pond was rather beneficial than otherwise. At times there was a tendency toward the formation of strata of almost clear sand extending from the sides to or beyond the center. This was the result of carrying the levees too high, or depositing for too long a time in any one place, thus increasing the gradients toward the center core. Where such local strata were discovered, the remedy was to distribute the deposit of tailings more evenly, and also to break up the strata by thrusting planks down into the soft mud, as deeply as they could be driven by a man's weight, and then withdrawing them. This work was followed in several regular, continuous lines, parallel with the longitudinal axis, from a raft or barge, floating in the pond. It appeared to be effective and satisfactory, permitting silt to enter where the plank had been withdrawn, and thus establishing breaks in the continuity of the sand strata. This tendency is one of the numerous details of construction requiring the exercise of due precaution, watchfulness and skill. It is not an automatic process by any means. The formation of sand streaks is more particularly noticeable as the embankment becomes narrower near the top. In the writer's opinion, it is a matter of sufficient importance to justify a material increase in the thickness, and superelevation, of the dam above the spillway level.

Novel Construction Features of the Spillway.—The lack of anything like bed-rock, of a solid and reliable character, in the hills on either side of the dam, which would serve as a foundation for the spillway, necessitated a resort to a novel plan for its construction. About 500 ft. below the dam, on the west side of the creek, an outcrop of hard porphyritic bed-rock appeared for a short distance, protruding through the surrounding soil like an isolated chimney of ore. The top of this rock was about 15 ft. below the level fixed for the crest of the spillway, and a channel was excavated from the spillway to the rock on a grade of 1 per cent. A water-cushion basin was arranged, into which the water overflowing the spillway could drop, the bottom of which was level with the top of the rock 500 ft. below. Thus the possible deepening of the canal by erosion between the spillway and the rock was limited.

The spillway was formed by a slab of concrete, 8 in. thick, laid over the surface of the earth, which had been trimmed to an ogee form, the crest being 80 ft. long, with sloping ends; the extreme height of the drop into the water cushion was about 16 ft. The slab was reinforced by sheets of expanded metal overlapping one another at the edges and ends, and placed so as to resist the erosion of the overflow. This metal was held in position by loops of $\frac{1}{8}$-in. wire, 18 in. long, embedded in the concrete every 18 in., the ends being allowed to protrude about 1 ft. When the sheets were laid over the hardened first layer of 6.5 in. of concrete they were effectively wired down to its face by twisting each of these pairs of wires over a sub-division of the metal. Concrete grout was then flushed into all the interstices of the expanded metal, and a plaster coat of cement and sand, in equal parts, 1.5 in. thick, was spread over the surface and given a smooth finish. The plan and sections of the spillway are shown on Plate XXVI, and a general idea of its construction is given by Fig. 2, Plate XXXVIII.

Results Achieved.—For the first two winters after the completion of the dam it was not permitted to be filled higher than the top of the original structure, because an economical streak in the management inspired an order suspending work before the spillway channel was finally completed to the rock of refuge below, and the superintendent did not like to take chances on its use. Therefore, the 5-ft. waste culvert was kept open a sufficient length of time during these two rainy seasons to prevent the reservoir from filling higher than the crest of the original dam. In the summer of 1904 the wasteway

FIG. 1.—WEST END OF LAKE FRANCES DAM, SHOWING THE BREAK RESTORED, THE HIGHER DAM NEARLY COMPLETED, THE HYDRAULIC GIANT AT WORK, AND THE MAIN PIPE LINE SUPPLYING WATER TO THE PUMP.

FIG. 2.—HYDRAULIC GIANT IN ACTION UNDERCUTTING THE BANK.

channel was finally completed, and in the winter of 1904-05 the reservoir was permitted to fill. After one of the heavy storms of that season the lake filled to overflowing, with the 5-ft. waste culvert discharging its full capacity; and water ran over the spillway to the depth of 22 in. This severe test established confidence in the dam so fully that the reservoir has been doing maximum service ever since. Opportunity was offered, by an excavation made in 1905 into the body of the dam, to disclose the condition of the material forming the mass. It was reported to be moist, solid, and "cutting like cheese"—evidently very satisfactory. The consulting engineer of the present company has even recommended that the dam be raised again to obtain additional storage. The photographs on Plate XXXV show the pits in detail, with the jet from the monitor in operation, although in these pictures the apparatus appears to have been arranged rather more for spectacular effect than for the performance of certain work. In practice, the jet is not thrown so high in the air, but is rather directed to the under-cutting of the face of the bank, from a position as close as safety to pipeman and monitor will admit. It is also played upon the material that has fallen from the face, cutting up the landslides and masses of earth which the under-cut has caused to topple over into the pit with majestic movement and crushing power. When thus jarred and shaken up, most materials yield quickly to the terrific velocity of the torrent directed upon them, and pass off in suspension in the water.

The work must be managed in the pit so that its floor will be constantly "cleaned" up by moving out and washing down all desirable material with the least loss of gradient, water supply and time, keeping the water as nearly as possible fully loaded with sediment, and at the same time excluding perishable trash, and rocks having a flat or angular form, which unfits them for carriage through the sluice boxes or pipes without blocking.

Hydraulic-Fill Dam in Crane Valley, California.

Almost simultaneously with the construction of the Lake Frances Dam, a similar structure was being erected in Madera County, in Central California, by the hydraulic process, but under conditions so distinct as to give the work a peculiar interest. The plans were prepared by Mr. J. M. Howells, in the capacity of Consulting Engineer, but the work of construction was under the direct supervision of Mr. J. J. Seymour, President of the San Joaquin Electric Company, for whom the work was done. Mr. Seymour has kindly contributed the following account of the construction features:

"The San Joaquin Electric Company had a long-distance electric plant carrying power from a power-house on the North Fork of the San Joaquin River, in Madera County, to Fresno City and neighboring towns in the San Joaquin Valley. The management had found that during three or four months in the late summer and early fall of each year, the natural flow of the stream was insufficient to run the plant to its full capacity.

"The Crane Valley Dam was built to impound some of the excess waters of the abundant winter and spring flow of the streams, to be liberated later in the season as the company's needs demanded.

"The site selected was in the lower end of Crane Valley at a point where the valley contracts into a narrow cañon through which the North Fork River flows. The stream at this point makes a horse-shoe bend around a rocky point, the location of the dam being at the toe of the shoe.

"The general dimensions proposed were as follows:

Maximum height	100 ft.
Length on top	720 "
Width on top	20 "
Slope on water side	2 : 1
Width of cañon at base	50 ft.
Width 60 ft. higher	300 "

"The bottom and sides of the cañon were composed of hard granite formation. Near the stream the rock was extremely hard, but it became less hard as the sides receded from the stream, and near the top on the west side considerable difficulty was experienced in obtaining a sufficiently firm base on which to lay the center core of the dam.

"The hills on either side of the stream have an underlying base of granite which has disintegrated very irregularly, leaving hard exposures at various points, while in places the depth to the solid rock is great. On opening up the borrow-pits the granite itself was found to vary greatly in hardness and cohesion within a few feet, seemingly without change in its chemical formation. The pits would open out in places, showing a vertical face of 30 or 40 ft., the material being unquestionably original granite formation but yielding to water erosion almost like a sand bank. On being under-cut by a jet of water, it would fracture vertically, exactly

FIG. 1.—LAKE FRANCES DAM FROM THE EAST END. SHOWING FINAL TRESTLE AND SLUICE PIPE, AND GENERAL VIEW OF BORROW-PIT.

FIG. 2.—LOOKING EAST ALONG LAKE FRANCES DAM. SHOWING OPERATION OF BUILDING UP SLOPES WITH BRUSH.

like a sand bank, and would fall in great masses which would pulverize in falling, and the continued play of the jet of water would immediately sweep it into the flumes.

"A few feet to one side, and its cohesive strength would suddenly increase to such an extent that water would have no effect on it. Or, on being cut with extreme difficulty, immense granite boulders would fall down, and these were so hard that powder would be required to break them up, and sluicing in that direction would be brought to a sudden close. Hence the hydraulicing in this granite formation was restricted to certain zones.

"In other places were borrow-pits which had been filled in by the action of water. The material here showed a large percentage of red clay mixed with sand. When the material from these latter pits was being sluiced on the dam, the workmen had great difficulty in keeping the outer edges of the dam up to the steep slopes at which they were designed, as they showed a tendency all the time to keep sloughing out, due to the lubrication of the contained clay.

"The material from the granite formation, however, seemed to be ideal for hydraulic dam building. Samples of it were taken, thoroughly dried, weighed and washed in a gold-pan, the water being changed until there was no discoloration. The coarse material left behind, consisting of sand with grains of varying size, was dried and weighed again. It was found that the percentage of sand left varied from 70 to 80% of the total material.

"The center line of the dam was excavated to bed-rock, and all loose material, boulders and sand, resting on bed-rock was removed for a distance of about 20 ft. from the center line on the up-stream side. Then a concrete foundation wall, 2 ft. thick, firmly fastened to bed-rock, was laid along the center line. This wall was carried up about 2 ft. in height. In the center of the stream bed it was 5 ft. high. About 9 in. from the up-stream side of this wall a center wooden core-wall of doubled 1-in. sheeting was fastened, by firmly embedding it with concrete of a high grade.

"This sheeting was carried up some 30 ft. above the bottom of the dam. Its object was mainly to prevent the stratification of the material, on being deposited, from extending across the center of the dam. It was also thought that it might act as a safeguard against the percolation of water through the core of the dam after construction.

"It was originally planned to carry this sheeting up to the water line of the reservoir. As the dam building progressed, however, it was found that possible stratification could be effectually prevented by a system of cutting into the plastic material in the central part of the dam, by the pushing down of broad paddles made of 1-in. boards. This process was followed in a systematic manner from day to day when sluicing was being carried on, care being taken that the boards or paddles were manipulated so as to make a continuous line of cleavage in the center plastic material from end to end of the dam. These lines of cleavage were repeated at intervals of 2 ft., measured from the center line of the dam, on the up-stream side, and were continued until a distance of 20 ft. from the center line was reached, making a total of eight or ten lines of cleavage each time the process was gone through with. As the workmen were able to shove the paddles into the mushy middle mass some 10 ft., it followed that the cleavage was repeated over and over again. Of course, after each paddle was withdrawn, the oozy material about would immediately fill in behind it.

"The result of this kneading process was so satisfactory that it was not thought necessary, as stated above, to carry the wooden sheeting higher than 30 ft. from the bottom. Toward the sides of the cañon, as the dam building progressed, this height was gradually decreased, until, at an elevation of 60 ft. from the bottom, the sheeting was only extended 6 ft. above the concrete foundation wall. This height was proposed to be maintained to the top of the dam.

"On top of the center concrete wall, against the wooden sheeting on the down-stream side, there was built, in place, a porous cement conduit 3 in. in diameter. This was built along the entire length of the wall. At the lowest point in the stream bed, the two ends were made to empty into a 6-in. porous cement conduit which was laid down the thread of the stream until it extended beyond the toe of the lower slope. The object of this drainage system was to draw off any water that might possibly reach below the center line of the dam, after construction. But, as will be explained later, through a series of adverse happenings, it became a source of menace to the safety of the dam.

"It had been planned originally to run a tunnel through the rocky point, around which the stream runs at the dam site, to carry the flood waters during construction, and this tunnel was begun. But, after running some distance in, the rock was found to be broken and seamy, with clayey fissures running through it. It showed a tendency to cave in, and was so unstable and uncertain that provision was made for carrying the water of the stream through the dam while in process of construction, by arching over a cut, blasted out of the solid rock by an old mining company for a ditch grade, at a level of 14 ft. above the stream bed. Here the rock was compact and free from fissures. This cut was arched over with masonry for the entire width of the dam. Its size was 6 ft. in width by 7 ft. high. Gates were set in this culvert on the center line of the dam. The culvert was to be bulk-headed just above the

FIG. 1.—HYDRAULIC-FILL DAM BUILDING, SHOWING OUTER LEVEES MAINTAINED WITH BRUSH. DEFLECTING BOARDS FOR DISTRIBUTING SLUICINGS ALONG SLOPE. GENTLE SLOPES TOWARD CENTER OF DAM, AND GRADUAL MOVEMENT OF DRAINAGE TOWARD EXTREME END.

FIG. 2.—LAKE FRANCES DAM COMPLETED. SHOWING THE TOP OF THE BURIED GATE TOWER ON THE SIXTEEN-INCH OUTLET.

gates after the completion of the dam, after which the gates could be closed and water storage begin. Directly above the gates extended a circular shaft, 22 in. in diameter, reaching to the top of the dam. This shaft was made of successive rings of cement pipe 12 in. in height, which were added one at a time as the dam rose. During construction this shaft served to draw off the water used in sluicing, which formed a pool in the center of the dam after it had dropped its load.

"It had been planned to bring the water for sluicing purposes a distance of some 5 miles, by flumes and ditches, from a branch stream having a higher elevation than the top of the dam. Unfortunately, a part of the ditch line was on a Government Forest Reserve, and there was delay in getting a permit to use the ditch over this reserve. It was seen that to wait the slow action of official proceedings would throw the construction of the dam too late in the season to raise it high enough to be beyond possible danger from winter floods, and if construction was postponed another year the company would be again crippled for lack of water. The management, therefore, decided to begin operations by pumping water for sluicing purposes, to be carried on until the ditch right-of-way permit came. This was in the spring of 1901. A hasty change of plans was made, and by the time the snow waters had subsided sufficiently to enable the stream to be forced through the culvert, the company was ready to begin active operations in sluicing material by means of the pump.

"The pump was a Worthington compound duplex, superseded and thrown out of commission by the Fresno Water Company on account of insufficiency of size. It was capable of pumping at the rate of about 60 000 gal. per hour, equal to $2\frac{1}{4}$ cu. ft. per sec. It was placed on the bank of the stream above the dam site. The water it pumped was carried through an 11-in. riveted steel pipe to the borrow-pits, where it was forced through a "Little Giant" monitor, having a nozzle $2\frac{1}{2}$ in. in diameter. The material to be sluiced was from 75 to 110 ft. above the stream. At this elevation the pump threw a stream with sufficient force to do the work of undercutting required of it effectually.

"Flumes were laid on a grade of 6 per cent. Attempts were made to lessen this grade, but they had to be abandoned because of the clogging of the flumes. Had there been more water, the grade could have been lessened. The flumes were made of 1-in. pine lumber 12 in. wide. This gave them a size of 10 by 12 in., thus being 2 in. more in height than width. They were enclosed on all four sides until the dam was reached, when the top board was left off. Attempts were made to use the V-shaped flume, but the results were not as satisfactory as with the rectangular shape.

"Two sets of flumes were constructed on the dam, one on either side. They were used in alternation. Material was sluiced through the flume on one side until that side was raised to a height from which the heavier particles, making their own grade toward the center of the dam, approached it as near as was thought expedient. Sluicing was then carried on through the other flume, and that side of the dam was raised to correspond with the other. At the bottom of the dam the coarser sand was not allowed nearer the center line than about 40 ft., but as the dam rose and its top width decreased, this distance was decreased correspondingly.

"After both sides were raised, as described above, the water level of the pool, always remaining in the middle of the dam, was raised by adding one of the cement rings to the circular shaft described above, through which the surplus water was allowed to escape. This raised the water level of the central pool about 13 in. The process of sluicing was then repeated. As the dam approached the 60-ft. level, this rise of 13 in. was found to throw the water too near the edge of the dam, and rings 6 in. in depth were added thereafter. The flumes were placed so that when the dam had reached an elevation equal to the lower end of the flume, which, of course, was on the farther side of the dam from which the excavation was taking place, the flume was on the outer edge of the dam. The flume was then raised, moving it toward the center of the dam sufficiently to allow the process to be repeated on the higher level.

"The trestles were made of 2 by 4-in. plank, and generally the 2 by 4-in. pieces could be pried out of the sand and used over again. A hand-spike with a sharp iron spur bolted to its end would generally start the stick from its bedding in sand. The flumes were raised about 10 ft. in elevation each time.

"At short distances along the flume, slotted openings were made in the bottom, through which a driblet of water was allowed to run. This driblet, being from the bottom of the flume, carried with it a large percentage of the coarser particles of sand coming down the flume until they were shoveled out by the workmen to carry up the outer slopes of the dam.

"The great bulk of the water was turned on, as desired, at different points along the flume by sawing the inner side of the flume in such a way that a section of it could be thrown across the flume diagonally, turning all the water out of it at that point. This could then be closed and another opening made as the occasion demanded.

"The sand carried in the flumes, on reaching the dam, was distributed on a light grade until the pool in the central part of the dam was reached. Here almost the entire quantity of sand remaining was deposited at once, forming a bluff bank under the water at

FIG. 1.—LAKE FRANCES DAM. DISCHARGE END OF WASTEWAY OUTLET CULVERT, USED AS AUXILIARY TO MAIN SPILLWAY FOR EMERGENCY RELIEF.

FIG. 2.—SPILLWAY OF LAKE FRANCES DAM, 80 FT. LONG. THE WATER-CUSHION IS 5 FT. DEEP.

its edge, while the finer particles were distributed throughout the inner section of silt.

"Attempts were made to carry small boulders and rocks through the flume while the process of sluicing was going on, but it was found to be impracticable with the quantity of water and the grades used. It would seem that this method of carrying boulders is only practicable where the rock is mixed with the material to be used, and the grades much steeper or the quantity of water much greater. It was found not to work in the construction of this dam at any stage. We have no data on hand from which to give definite figures as to cost of the movement of material. Like all other dams of this type, when conditions were favorable, and material plentiful, it was moved at an extraordinarily cheap rate. While working in the granite borrow-pits described above, there were days when the material carried in the water exceeded 25% of the quantity pumped.

"About two-thirds of the total quantity of material moved into the dam was carried by the water pumped. The permit to use the ditch came at last, and the final third was ground-sluiced in, the methods of handling the flume being similar to those used in pumping.

"The dam was built to a height of about 70 ft. At this height a secure temporary spillway was afforded, through the horseshoe-shaped hill described above, around which the stream ran. Here it was stopped, as the reservoir capacity at that level was ample for the needs of the company at that time.

"The two faces of the dam were rip-rapped with broken stone gathered from the adjoining hillsides and latterly from the temporary spillway.

"After the sluicing was practically suspended and the force was engaged in finishing the rip-rapping with broken stone blasted out in the process of widening the spillway, there occurred a break in the dam, to which allusion was made in describing the small conduits under the dam.

"Before describing the break, reference will be made to a mishap which occurred some time previously, which was primarily responsible for the damage done.

"In preparing the culvert for turning water through it, the bases of the gates, to be used afterward for letting water out, were firmly embedded in the bottom of the culvert with concrete. This concrete extended several feet up stream from the gate, and was intended to be the base of the bulkhead by which the culvert was to be blocked after the completion of the dam. A wooden door, consisting of a double thickness of 2-in. pine boards, was hinged to the upper side of this bed of concrete, and was accurately fitted to the culvert, so that when the time came for bulkheading the culvert it could be flapped shut and held in place by the pressure of the water while the concrete bulkhead was being placed. This wooden door was securely fastened down, as was thought, but, through some mishap, when the dam was about 40 ft. high, and during a heavy freshet, the fastening worked loose, the gate flapped shut, and water began to rise rapidly in the reservoir, threatening to go over the top of the dam, the outlet gates not being sufficiently large to carry off the volume of water then coming down stream. In this emergency it was decided that the wooden flap-gate would have to be blown out with powder to prevent an impending catastrophe. After experimenting on two or three bulkhead doors of similar dimensions, which were backed up with sand and blown to pieces on the top of the dam, in order to ascertain the quantity of powder necessary to do this without subjecting the culvert to more shock than was necessary, the door was blown out, the flood passed off through the culvert, and the danger of overtopping the dam was averted. After the water had subsided sufficiently the tunnel was thoroughly examined on the inside and no indications of damage were visible. Unfortunately, as ascertained later, the small, 3-in. cement conduit, extending along the center line of the dam, was shattered, by the upward thrust of the explosion, at the point where it passed over the top of the culvert. The center wall had been omitted here, and the conduit and the cement footing of the wooden sheeting had been fastened directly to the top of the culvert. The cement footing of the sheeting was also badly shattered. Of course, there were no means of knowing this at the time.

"The break began by showing an increase and discoloration in the small quantity of water which had been running through the 6-in. pipe extending below the toe of the lower slope in the middle of the stream. This was watched with dismay, and it slowly increased until it ran about 3 or 4 miners' in. of highly discolored water carrying much sand. It did not increase beyond this quantity, but continued running for several days before its effect was shown on the dam. It was decided that it must be coming from a break somewhere in the 3-in. drain pipe, because of the small quantity of water being discharged, but at that time it occurred to no one to locate the trouble at the point where the 3-in. drain had been shattered, as this was not known. At this time the reservoir was full, almost to the spillway, with the culvert under the dam discharging its full capacity of water.

"The first visible break occurred in the top surface of the dam about 40 ft. from the center on the up-stream side, and a little to one side, vertically, of the culvert under the dam. It began by a

Fig. 1.—Crane Valley Hydraulic-Fill Dam. Showing Method of Loosening the Materials Sluiced Into the Dam.

Fig. 2.—Borrow-Pits From Which Material Was Sluiced to the Crane Valley Dam.

subsidence, conical in shape, like a crater, and continued until a depression, some 15 or 20 ft. deep, was made in the dam. Around this crater were several concentric rings of fracture in the top of the dam. The outermost of these were nearly 60 ft. from the center of disturbance. When the subsidence began, great quantities of gravel boulders of varying sizes, and bags of sand, were thrown into the center of it. These disappeared, and when, after several days, the leak stopped, it was supposed that they were the means of checking it. More than 1 000 cu. yd. of material went out of the dam during the leak.

"To remedy the break, a shaft was sunk around the 22-in. circular cement pipe through which the water had been drained while sluicing. The movement of the top of the dam toward the depression had fractured this pipe, and there were other signs which indicated that the seat of trouble might be at its foot.

"On reaching the culvert top with the excavation the break was found. It was also discovered that it had been mended by being plugged with roots and leaves which had been washed into the dam in the process of sluicing.

"The small conduit was plugged with cement, the circular drainage shaft was removed, its opening into the tunnel was closed, and the large exploration shaft was filled up again. The gates were afterward removed to the lower outlet of the culvert under the dam.

"A rational explanation of the causes of the break may be stated as follows:

"*First.*—The small conduit and the footing of the wooden sheeting, where they passed over the culvert, were shattered at the time of the blowing out of the wooden gate, as explained above.

"*Second.*—The continued explosion of blasts, in excavating the spillway about 300 ft. distant, gave a jarring motion to the culvert, and permitted the gradual working of a film of water, probably, along the outside of the large culvert, until it had reached the center of the dam and found its way under the shattered sheeting into the 3-in. drain pipe.

"*Third.*—The center puddled material of the dam was compact enough to admit of a small fissure or water-worn conduit being formed beneath it, but when the point toward the outer edge of the dam to which sand had been allowed to come in sluicing was reached, this had not sufficient cohesion to stand, but poured into and followed the crevice or water-worn conduit to the center of the dam where it reached the 3-in. drain pipe through which it finally discharged.

"At this date (1906), it is announced through the newspapers that the dam is to be raised again to afford more storage capacity.

"It has been in service continually since its completion."

The foregoing graphic account of the construction of the dam and the remarkable break by which it started to flow away after completion is a most interesting illustration of the varying action of the different grades of material which may be used successfully in hydraulic-fill dam building. Evidently, the quicksand which composes the center of the dam is the poorest material of which a dam can be constructed, and yet, when held in place by coarse, stable sand and gravel on the slopes, it is water-tight because of its extremely fine texture, and can be made to constitute one of the important parts of a stable, permanent, safe and useful dam. By no other process, however, could an equally tight and stable dam have been built, except by covering it with an impervious facing, or building a heavy core-wall in its center, either of which would have added to the cost to a prohibitive degree.

Hydraulic-Fill and Rock-Fill Dams on Snake River, Idaho.

The ideal type of earth dam is one where stability of the mass is secured by a heavy embankment of loose rock on the down-stream side, against which the earth fill can be placed by the sluicing method, with an up-stream slope of at least 4 on 1; or, in other words, a combination of rock-fill and hydraulic-fill. The rock-fill affords stability against sliding or overturning, and, at the same time, gives perfect drainage to any seepage which may find its way through the earth. Thus the saturation and sloughing of the outer slopes are prevented, and one source of weakness and cause of failure in earth dams is rendered impossible.

Three dams of this type were built in 1904-05 by the Twin Falls Land and Water Company for the diversion of Snake River into irrigation canals on either side of that stream, in Cassia County, Idaho, under plans designed by W. G. Filer, Manager, P. S. A. Bickel, Chief Engineer, and the writer acting as Consulting Engineer. The construction was under the direct supervision of Mr. M. M. Murtaugh, as Assistant Manager. These dams have some special features which put them in a class by themselves as the first of a type which may be adapted to many other localities to good advantage. At the site selected for headworks, the river was divided into three channels, by two islands of basaltic rock, both of which were utilized for waste ways. The North Channel was the

FIG. 1.—CRANE VALLEY DAM, SHOWING WOODEN FENCE, OR CENTER DIAPHRAGM, WHICH WAS CARRIED UP TO A HEIGHT OF 30 FT. ABOVE BED-ROCK.

FIG. 2.—CRANE VALLEY DAM, SHOWING DISCHARGE OF SLUICED EARTH AT END OF CONVEYING FLUME.

permanent bed of the stream, the other two being dry at ordinary low stages of the river. The Middle Channel carried a considerable volume at medium high water, while the South Channel was never occupied except in extraordinary floods.

To reach the level of the canals and fill them to the depth of 8 ft. above the floor of their headgates, it was necessary to lift the river bodily a height of 49 ft. above normal low water. For this purpose, the dams required to be of the following dimensions:

Main Channel Dam: top length, 340 ft.; height above lower toe, 86 ft.; upper toe, 80 ft.; volume of rock-fill, 39 650 cu. yd.; earth-fill, 58 000 cu. yd.

Middle Dam: top length, 335 ft.; height above upper toe, 56 ft.; lower toe, 81 ft.; volume of rock-fill, 42 800 cu. yd.; earth-fill, 62 850 cu. yd.

South Dam: top length, 560 ft.; height above upper toe, 56 ft.; lower toe, 66 ft.; volume of rock-fill, 34 700 cu. yd.; earth-fill, 48 000 cu. yd.

The total length of the three dams and the intervening spillways and regulating gates, considered as one structure, is about 2 100 ft. Plate XLIII.

Construction.—The rock-fill portion of the dams was made with slopes of 1½ on 1 on the down-stream side, ¾ on 1 on the up-stream side, and 10 ft. wide at the crest. The base of each dam was stripped of surface soil and loose material, and in the center line of the rock-fill a trench was excavated into the bed-rock, throughout the longitudinal axis and extending up to the top on the sides. In this trench a continuous core-wall or barrier of double 2-in. plank was built from the bottom to within 6 ft. of the top, the plank being laid horizontally, breaking joints, and being spiked to 3 by 6-in. uprights placed 2 ft. apart from center to center. The base of this wooden partition was embedded in concrete, which filled the trench to above the line of the bed-rock, and formed a tight bond with the rock. The trench was from 5 to 6 ft. wide, and the concrete was well rammed on either side as far up as it extended, which was usually from 4 to 6 ft. The loose rock embankment was built on each side of the fence, which was carried up considerably in advance of the rock. For several feet on either side of the fence the rock was laid by hand with sufficient care to avoid crushing the wood by settlement, and give a continuous bearing of rock against every square foot of the fence. Outside of this the rock was loosely dumped into the embankment from a cableway. The rock was hauled to the site from the south side canal, the excavation of which was in solid basalt for the first 2 miles. In this section the canal was 100 ft. wide on the bottom, and was designed to carry water 8 ft. deep. By the terms of the contract, all the rock in these 2 miles, estimated at 96 000 cu. yd., measured *in situ,* was required to be delivered at the dams if needed. As the total bank measurement of rock in the three dams was only 114 250 cu. yd., and as the 96 000 cu. yd. of solid rock yielded enough loose rock to measure more than 150 000 cu. yd., there was much to spare. This rock generally broke in large masses, was quite hard, and made a very solid embankment.

The Hydraulic-Fills.—The function of the wooden core-wall in the rock-fill was to permit the earth embankment built against the rock-fill to be sluiced in place without flowing out through the voids in the rock. The earth available for this work was of one class only, and consisted of the fine, white or grayish soil which covers the entire region to a depth varying from 2 to 20 ft. or more. It is exceedingly fine in texture, an almost impalpable powder, free from grit, and wonderfully uniform. A test made by the writer showed that nearly 90% would pass through a sieve of 10 000 meshes per sq. in. It is classed by geologists as loess or æolian (wind-borne) soil, and probably was deposited originally in the water by the wind. It absorbs water very slowly, like flour, but when once wet packs very solidly and becomes as stable, impervious, and solid as clay, with this advantage that it does not shrink and crack when it dries out. In building the south and middle dams the plan used was to throw up a levee at the upper toe of the embankment with dry earth, hauled in by wheel scrapers or wagons. Between this levee and the rock-fill the bulk of the filling was subsequently sluiced in place by water delivered by centrifugal pump from the river to the earth dump at one end of the dam. The earth was hauled by cars and electric locomotive from borrow-pits a mile or more distant on the south side of the river, and dumped at such an elevation at the nearest end of either dam that the

Fig. 1.—Crane Valley Dam, at a More Advanced Stage. The Sluiced Earth After a Few Days' Settlement, is Able to Bear Up a Team on the Outer Thirds, while the Center is Still Liquid.

Fig. 2.—Crane Valley Dam, Nearing the 70-Ft. Level, where Sluicing Operations Ceased.

water would carry it on a grade to the other end. The grade naturally taken by the earth thus sluiced was from 2 to 4 per cent. The liquid mud freely entered the voids of the rock-fill, and filled them solidly as far as the wooden core-wall. As it rose in height, some slight leakage occurred in the wood partition, but the joints quickly swelled, filled with mud, and became entirely tight. The earth was always 20 ft. or more lower than the top of the rock-fill, and the work progressed at such a moderate rate that the embankments had ample time to settle and solidify. The earth packed so readily that 4 days after sluicing was suspended a team could be driven over the sluiced material without sinking in, although during the time sluicing was in progress a pole could be easily pushed down into the mud 10 ft. or more, particularly at the extreme end, where the water stood longest in the pool. Very little surface drainage was required to get rid of the surplus water. It seemed to sink out of sight and become absorbed in a mysterious way, without showing any sign of reappearance, either at the outer side of the rock-fill or inside the earth-fill. The volume of water used was about 1.5 cu. ft. per sec. It was sprayed upon the dusty earth coming from the cars, and saturated it to the softest of mud almost immediately. About 80% of the entire mass of earth in these two dams was thus deposited as fine mud, and in a few weeks became as solid as the hardest roadway after being traveled over by teams, while about 20% was put in dry at the outer slope. This dry portion constantly absorbed much of the surplus moisture from the adjacent mass of mud, and thus became equally hard and solid.

After their completion, and when the water rose to full height against these dams, they showed no sign of leakage, although a slight settlement occurred at one end of the middle dam, requiring the addition of a small quantity of fresh earth, and the relaying of a limited area of rip-rap thus disturbed.

At the North Dam the conditions were somewhat more difficult, and required different methods of operation. This dam was built in the bed of the main stream, where the discharge was from 5 000 to 10 000 cu. ft. per sec. during all the time work was in progress. To take care of the water, a large tunnel was driven at the north end of the South Island, next to the middle dam, and when it was completed the river was forced to go through the tunnel. When work started on the dam the water was 20 ft. deep in the North Channel. Two parallel embankments of rock, constituting outer toe walls of the rock-fill, were formed across the channel, far enough apart to leave room for sinking a line of timber cribs, 24 ft. wide, placed so that their upper edges were in line with the center core-wall subsequently built. These embankments were formed of large stones dropped from a cableway. Before water was finally admitted into the tunnel (by blowing up the earth coffer-dam in front of it), the embankments referred to had obstructed the flow sufficiently to create a difference of level of 13 ft. above and below the site of the dam. After opening the tunnel this head was reduced several feet, as about 4 000 cu. ft. per sec. passed through the tunnel, while about 1 000 cu. ft. per sec. found their way through the voids in the ridges of rock built across the river channel. When this was accomplished, the velocity in the river bed was so reduced that it was possible for divers to clean off the bed-rock where the timber cribs were to be located, and adjust these cribs to their correct position with reasonable accuracy.

The cribs being loaded with stone, sheet-piling, consisting of 2-in. plank laid double and breaking joints, was placed vertically against the upper face of the cribs and spiked thereto, the bottom of the piling being fitted into a shallow trench excavated by blasting the bed-rock under water. Concrete in bags was carefully placed at the toe of the piling, and was built up against it, about 6 or 8 ft. wide at the base, and 4 ft. or more in height. This constituted the jointing of the piling to the bed-rock below the water line. This work was exceedingly tedious and difficult, and occupied about two months, with two divers, working from September 12th to November 15th, 1904.

The principal part of the hydraulic-filling for the North Dam was delivered from the north side of the river through the flume, in the upper end of which a receiving box was placed where the earth was dumped from wagons into a trap. Water pumped from the river washed it down to the dam. The earth was loaded in the wagons by a modern excavator with belt-conveyors delivering a continuous stream of earth, loosened by plows, to wagons traveling alongside until they received their load. This is an economical device, where the borrow-pit is large and need not be confined to

Crane Valley Dam. General View from Lower Side.

narrow limits. The water used was about 1 cu. ft. per sec., delivered by a No. 4 centrifugal pump. The lower end of the flume discharged along the upper side of the wooden core-wall, first filling the voids in the rock-fill and then extending up stream in the water, assuming a very flat slope of 6 or 7 on 1 under the water line. Great difficulty was experienced for some time in stopping a few leaks through the wooden partition, and considerable earth filling was carried through the dam and lost. This may have been due to settlement of the cribs under the weight of rock, or to imperfect joining with the bed-rock. The necessity for doing much of the work in freezing weather was one of the causes of the serious difficulty encountered in making the hydraulic-fill. Layers of frozen earth were formed in the embankment, and these subsequently thawed out when the water was allowed to rise against the dam, creating alarming settlement in the earth next to the rock-fill. This alarm was due to the extent of the disappearance of the earth-fill below the water line along almost the entire length of the dam, and the volume of leakage through the dam, when the water reached its normal height. This leakage was not definitely measured, but it was estimated at one time at more than 6 cu. ft. per sec. In an ordinary earth dam such leakage would necessarily be fatal. In this case it was never a source of actual danger, and only resulted in the loss of 2 000 or 3 000 cu. yd. of earth filling (possibly less), before the leaks were finally closed with fine gravel brought in a barge from a few miles above.

The irrigation system was inaugurated and the dams formally put in service on March 1st, 1905, by the act of closing down the eight huge gates at the tunnel entrance, but it was two or three months later before the North Dam was made secure from leakage, and the earth-fill completed. Since that time the water has stood constantly against the dams, and they have shown no sign of leakage or further settlement, and are in a perfectly satisfactory condition, in which they are likely to remain.

The contract prices for this work were:

Embankment, dry earth	27.5 cents per cu. yd.
Earth embankment, placed by sluicing.......	37.5 " " " "

These prices were necessarily high, on account of the remoteness of the locality, the high cost of fuel, the scarcity of earth in close proximity, the high price of labor, etc. The prices, however, should have given a handsome profit to the contractors.

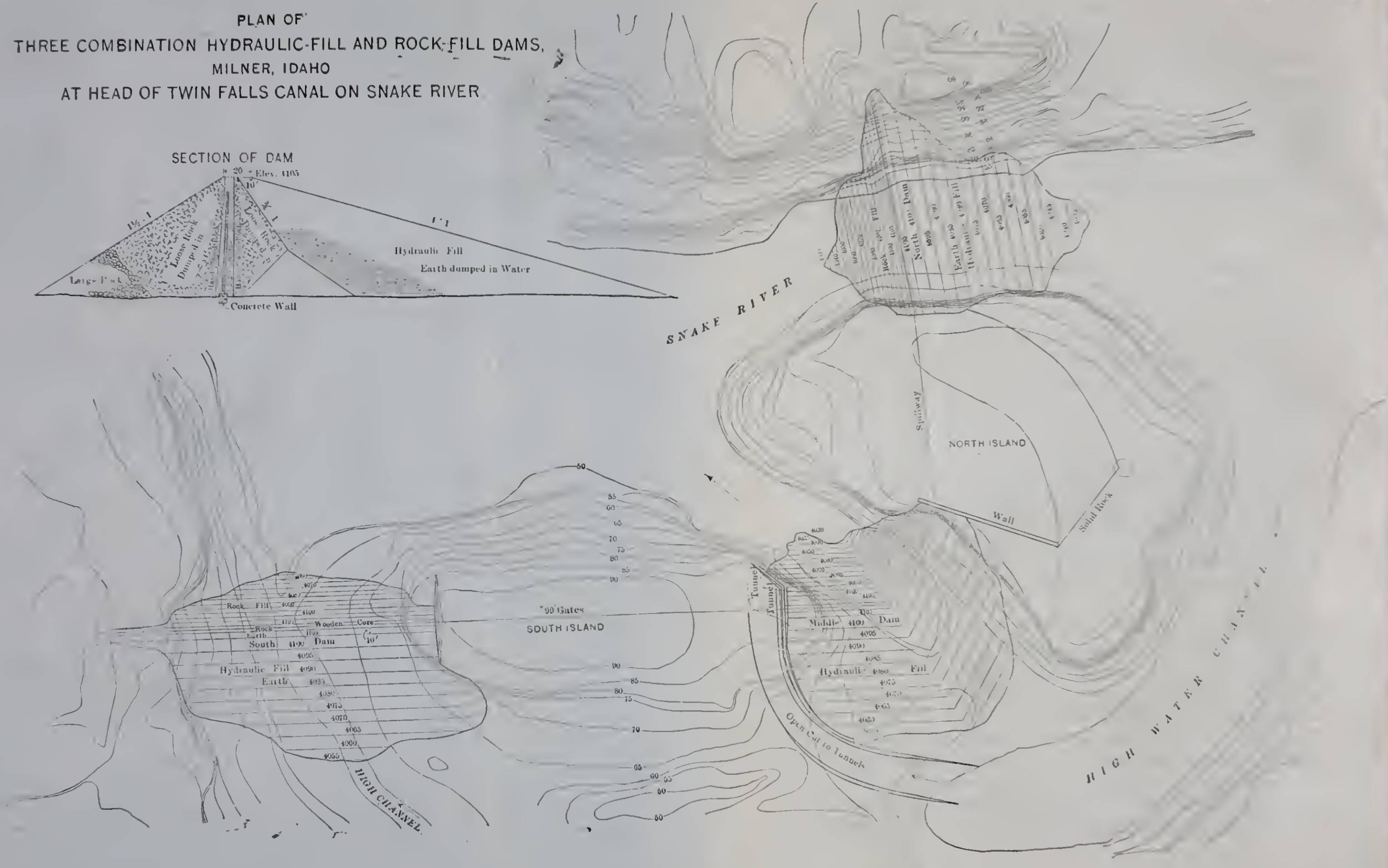
The combination rock and earth-fill of the type just described, with a core of wood to serve the temporary function of stopping the flow of liquid earth until it has become solidified, is one which can be recommended unqualifiedly as worthy of imitation and confidence. It is seldom that any earth dam has to be built in a river, or in water of such great depth as 30 ft., and the embarrassing conditions and difficulties encountered must be considered as marking the extreme to which any dam of that height would be subjected.

An Hawaiian Combination Dam, Recently Finished.

In 1903 the writer was engaged to report on the construction of a dam for the storage of water for the irrigation of sugar cane, on the Island of Oahu, 22 miles from Honolulu. The site is on one of the torrential streams of the island, called the Kaukonahua Gulch, which heads in the Koolau Mountains at an elevation of 2 360 ft. The stream drains from the windward side of the island where the maximum annual rainfall is about 180 in., well distributed through the year. The run-off fluctuates spasmodically between wide extremes, and storage is needed to utilize the stream to any advantage, although its distribution is such that a reservoir of moderate capacity, which will be filled several times each year, will be as serviceable as a much larger reservoir filled less frequently. In this case the average run-off was estimated at about 50 000 acre-ft. per annum, sufficient to fill the reservoir more than six times each year.

The Waialua Sugar Plantation is at the mouth of this stream, and extends up the mountain slopes from sea level to an elevation of about 700 ft. or more. Sugar cane requires an enormous volume of water per acre for irrigation, estimated at a depth of nearly 10 ft. per annum, and this plantation has installed four great pumping stations, by which the greater portion of the water used is supplied from wells near sea level. The aggregate capacity of these pumps is 72 000 000 gal. per 24 hours; their average lift is from 231 to 540 ft., the extreme lift being 650 ft. The cost of pumping runs into high figures. The fuel bill for irrigation pumping alone was

PLAN OF
THREE COMBINATION HYDRAULIC-FILL AND ROCK-FILL DAMS,
MILNER, IDAHO
AT HEAD OF TWIN FALLS CANAL ON SNAKE RIVER



$180 000 in 1902, and the average cost of water was $63.36 per acre for that year. These figures are mentioned in order to illustrate the importance of the dam and storage reservoir, which will supply water to the high levels of the plantation, and reduce the cost of irrigation very materially.

After examining the site, the writer recommended the construction of a combination rock-fill and earth-fill dam, 98 ft. in extreme height, with a crest width of 25 ft., 10 ft. above the spillway level, with a wooden core-wall in the rock-filled portion, embedded at the bottom in a concrete wall, to be carried down in a deep trench far enough to intercept certain strata of porous material encountered in the test-pits. The earth-fill was to be placed on a slope of 4 on 1, and sluiced into position against the rock-fill.

Mr. H. Clay Kellogg, of Santa Ana, Cal., who had made the original surveys and test-pits of the dam and reservoir site, was engaged to construct the dam, and carried out the work in a very efficient manner practically on the plans recommended by the writer.

The reservoir occupies two forks of the stream, which join immediately above the dam. They are generally parallel for some 5 or 6 miles, and are cut down through a sloping plain like miniature cañons, to a depth of 150 or 200 ft. The formation thus exposed in section is all of volcanic origin, and consists of the following general characteristics at the dam site as well as throughout the length of the reservoir: Reddish brown soil for 2 or 3 ft., then bright red tufa for from 20 to 50 ft., underlaid by yellow tufa for from 50 to 100 ft. more, with various intermediate shades of color, from purple to red. This tufa is generally quite free from any tendency to slide, and will often stand vertically in trenches or tunnels, without timbering, for an indefinite time. It also resists the erosive action of water in a remarkable manner.

The trenching for the concrete core-wall was carried to a maximum depth of 38 ft. below the surface, while it was extended laterally into the hill on the sides from 14 to 28 ft. The trench was made about 5 ft. wide, and was entirely filled with concrete. This wall was carried up to, or slightly above, the original surface, as shown by the section, Fig. 1.

The dam is 460 ft. long and 25 ft. wide on top, with a base of

580 ft. up and down stream, Fig. 2. The rock-fill has a base of 80 ft., and is 11.5 ft. thick at the top, containing 26 000 cu. yd., a considerable portion of which is laid up as a dry wall by hand, the outer slope being ¾ to 1, and the inner side vertical. The wooden core-wall is 2 ft. below the upper face of the rock-fill (down stream) and consists of double 2-in. redwood plank, laid horizontally and spiked to 3 by 6-in. uprights, placed 2 ft. apart, from center to center, with a double layer of burlap dipped in hot asphaltum between the two layers of plank.

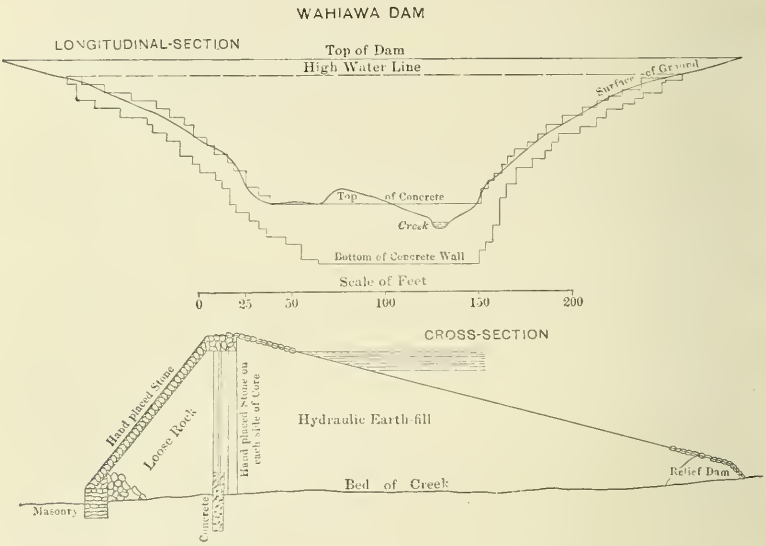


FIG. 1.

The rock was brought to the site by cars from a distance ranging from 1 to 6 miles, and dumped from a high trestle built along the longitudinal axis of the dam. It is basalt, found in the stream beds in boulders of all sizes. Many of these boulders required to be blasted for handling.

Water for sluicing was delivered by pipes from an upper ditch to a point about 2 000 ft. distant, and 50 ft. higher than the dam. Between this point of delivery and the dam was a body of earth of great depth which was considered suitable for the embankment. The ground-sluicing method was used, as the head available was insufficient to give the necessary velocity for mining and disinte-

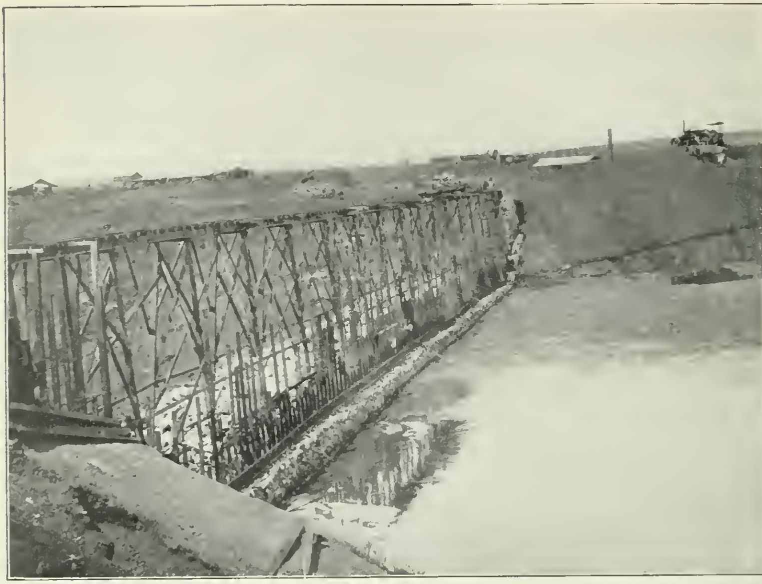


Fig. 1.—Waialua Dam. Showing Hydraulic Fill Being Sluiced in Against the Rock-Fill.

Fig. 2.—Waialua Dam Site. Looking Down Stream. Showing the Gorge Filled by the Dam.

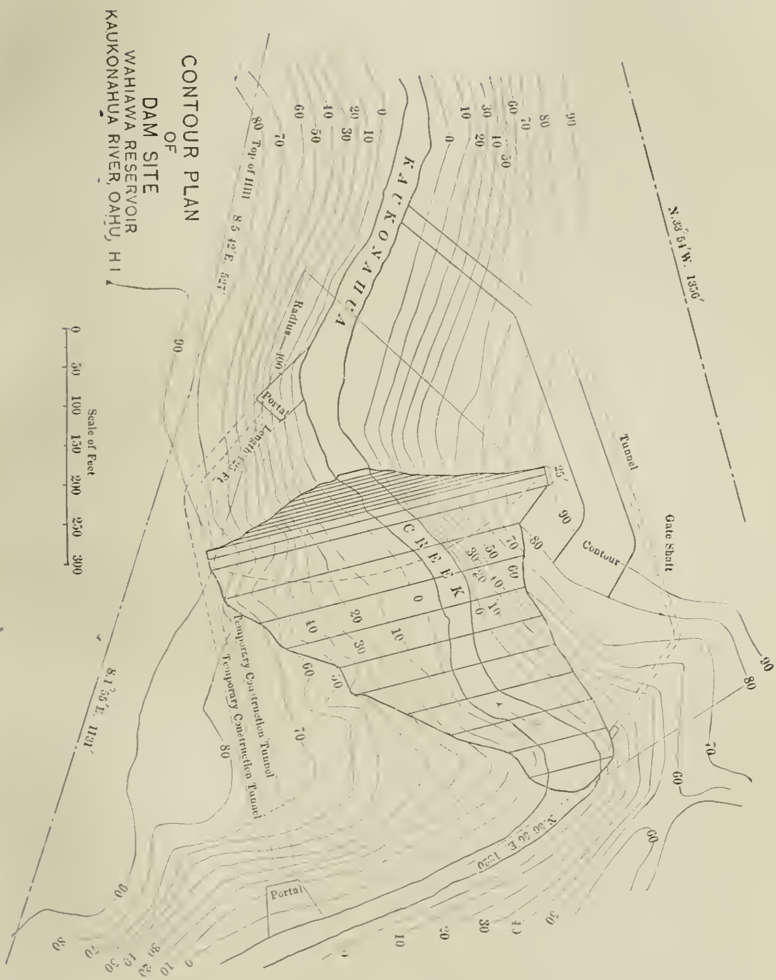

CONTOUR PLAN
OF
DAM SITE
WAHIAWA RESERVOIR
KAUKONAHUA RIVER, OAHU, H I
Scale of Feet
0 50 100 150 200 250 300
KAUKONAHUA
CREEK
Tunnel
Gate Shaft
Contour
Portal
Portal
Radius
Top of Hill
Temporary Construction Tunnel
Temporary Construction Tunnel

FIG. 2.

grating the material by the hydraulic monitor. Ordinary soils can be washed very readily by flowing water over it if sufficient velocity is given, and usually very moderate grades have to be given to irrigation ditches to prevent them from cutting their beds and forming deep miniature cañons. This volcanic tufa of Hawaii, however, contains a large amount of iron oxide, and sufficient structural cohesion to make it exceedingly difficult to sluice, even when plowed, chiefly because it is totally lacking in sand and grit, or any cutting substance to furnish the tools for erosion. Strange to say, the tufa, while resembling the clay, and having an unctuous feeling to the touch, settled in the pond very rapidly, and the water flowing away at the drainage outlet was generally clear enough to drink. This is a most interesting and remarkable fact, which is without precedent or parallel in the experience of the writer.

The method finally adopted to move the earth through the sluices to the dam was to dig a ditch about 4 ft. deep at the upper end, and from 12 to 16 ft. deep at the lower end next to the dam, through which the water, to the amount of 8 cu. ft. per sec., was turned. This ditch was 1 300 ft. long, and at each end a steam plow was placed, plowing the soil to a width of 12 ft. on either side. After plowing the ground the traction engines of the plow were used to drag a V-shaped scraper, or "crowder" as it was called, along the plowed surface, thus crowding the loosened earth into the running water of the ditch alongside, the velocity of the water being sufficient to carry the load thus received and deliver it to the dam. This was the only effective means found for handling the earth, as the water had no effect whatever when turned upon the plowed ground, and merely ran over it clear, without washing it away. This process was continued until the ditch grade was reached, when a new strip was plowed, and the ditch shifted over to the bluff bank on either side of its original position. This work of loosening and delivering the soil to the ditch was done by contract for 8 cents per cu. yd. The cost of distributing it properly averaged 3 cents per cu. yd., a total of 11 cents per cu. yd. The total volume of the earth-fill was 141 000 cu. yd., of which 100 000 cu. yd. were put in by aid of the four steam plows used for plowing and crowding; the remainder was loosened by hand work, blasting, and picking and shoveling into the ditches. The total cost of the

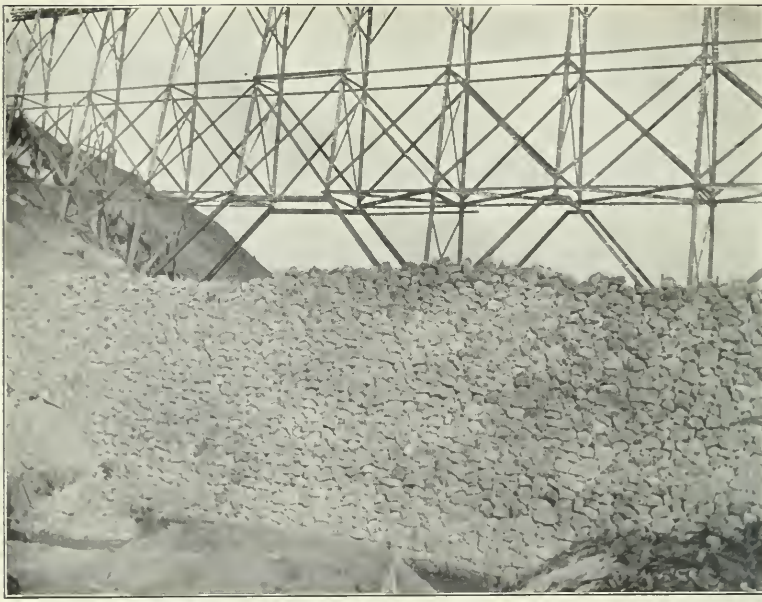


Fig. 1.—Rock-Fill on Down-Stream Side of Waialua Dam.

Fig. 2.—Rip-rap on Face of Hydraulic-Fill, Waialua Dam.

dam was $281 000, including a diverting or relief dam, 29 ft. high, four long tunnels around the dam, the regulating-gate tower, 48-in. outlet pipe, etc. The hydraulic-fill is reported to be very hard, solid and entirely water-tight, and the dam is a satisfactory one in every way, which is gratifying to all concerned, as the situation presented grave difficulties.

The elevation of the spillway of the dam is 844 ft. above sea level. It commands all lands below 700 ft. elevation in the Waialua Plantation, and will be used to irrigate lands between the 400 and 700-ft. levels. This reservoir is expected to effect the irrigation of 3 500 acres, at a saving of not less than $50 per acre per annum. If this expectation is realized, the cost of the dam will be covered in two years. The reservoir has a capacity of 2 500 000 000 gal., or 7 800 acre-ft.

The construction of the dam is illustrated by the photographs on Plates XLIV and XLV.

A Combination Dam in New Mexico.

The United States Indian Bureau is engaged in building a dam for the storage of water on the Zuñi River for the purpose of irrigating the lands of the Zuñi Indian Reservation, in McKinley County, New Mexico, about 40 miles south of Gallup. J. B. Harper, M. Am. Soc. C. E., whose title is Superintendent of Irrigation, is the Engineer-in-Charge, and the writer is Consulting Engineer. Mr. W. H. Code, Inspecting Engineer of the Indian Bureau, has also collaborated on the plans, with valuable suggestions.

The dam when completed will be 720 ft. long on top, 70 ft. in height, and will consist of a loose-rock embankment, backed by an earth-fill, Fig. 3 and Plate XLVII. The rock-fill is being carefully hand-laid on the top slopes, with loose rock well chinked. The down-stream slope of the rock is 1¼ on 1, stepped in regular benches, while the up-stream slope is ½ on 1, the cubic contents being 32 400 cu. yd. The base is thus about 125 ft. Plate XLVI shows the upper and lower faces of the rock-fill as they appeared in January, 1906.

The hydraulic-fill is to have a slope of 3 on 1. The crest width will be 20 ft., with vertical parapet walls, 5 ft. in height on each side below the crest. Its cubic contents will be 51 300 cu. yd. No core-wall is to be built in the dam, but the liquid earth to be sluiced in position will be held in place between levees of dry earth hauled with teams. Against the face of the rock wall a dry bank about 10 ft. wide will be built up a few feet in advance of the rising pond of mud to keep the liquid earth from penetrating the rock-fill and escaping through the voids, while on the up-stream slope a similar bank will be built. The stream will be carried through the outlet tunnel while sluicing is in progress.

A steam pump with a capacity of about 3 cu. ft. per sec. has been provided, and water will be delivered, through an 8-in. pipe

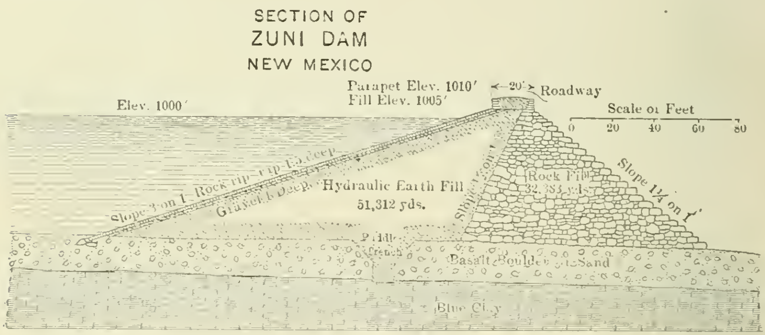


FIG. 3.

to a hydraulic giant, with ample pressure to give a cutting stream, sufficient to loosen the soil and convey it through the sluice-boxes from the borrow-pits on the north side of the cañon to the dam.

A mechanical analysis of the soil available for the hydraulic-fill, made by the writer, showed that it contains 73% of sand and 27% of clay. The sand was found to have an exceedingly fine texture, 99% passing through a 30-mesh sieve, having 900 holes per sq. in. These materials are considered to be extremely favorable for the purpose, and will not only be readily disintegrated with a hydraulic stream of moderate force, but will flow readily and without clogging in the flume on a grade of 3%, and, after settlement in the bank, will form a dense embankment, of great solidity,

FIG. 1.- ZUNI DAM, LOOKING NORTH ALONG UP-STREAM FACE OF ROCK-FILL, AGAINST WHICH THE HYDRAULIC-FILL IS TO BE BUILT.

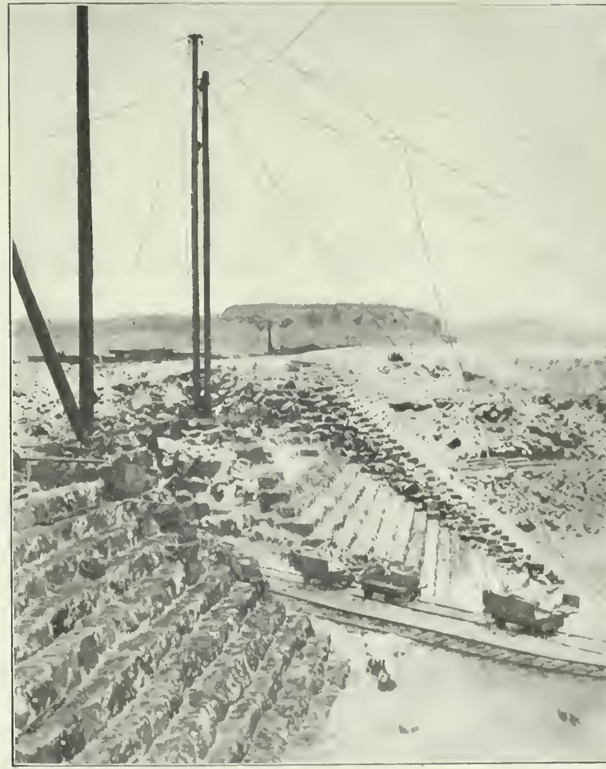


FIG. 2.—ZUNI DAM, LOOKING SOUTH ALONG DOWN-STREAM SLOPE, SHOWING HAND-LAID DRY WALL.

the voids in which will be entirely filled by the contained clay. It will thus be impervious to water and constitute a safe dam. The extreme elevation of the borrow-pit will be 35 ft. higher than the top of the dam. Assuming a pressure of 100 lb. per sq. in. at the nozzle of the giant, the total lift, including friction in the pipe, will be equal to a static head of 330 ft. The plant will require about 100 h. p., and the cost for power is estimated at from 2 to 4 cents per cu. yd. The fuel to be used is obtained from a coal mine opened by Mr. Harper on the Reservation a few miles above the dam.

The reservoir will have a capacity of 16 000 acre-ft., covering an area of 623 acres, nearly circular in shape. It intercepts the drainage from 650 sq. miles, and is expected to fill ordinarily once each year, from the usual rainfall and run-off. It is hoped to accomplish the irrigation of from 5 000 to 10 000 acres, and bring prosperity to an interesting tribe of Pueblo Indians.

The work is being done chiefly with Indian labor, but under many difficulties and with many interruptions, due to the numerous religious festivals which these curious people observe, during which they suspend all work. It is expected that the dam will be completed during 1906. An appropriation of $85 000 is being expended on the construction.

Terrace Dam, Alamosa River, Colorado.

The highest hydraulic-fill dam yet projected in the United States is under construction in Southwestern Colorado, for the storage of water for irrigation. The location is at the head of the lower cañon of the Alamoso River, a few miles above the point where the river enters the San Luis Valley. The situation is quite unique, inasmuch as the lower 70 ft. of the dam is in a narrow slit in the bed-rock, from 20 to 60 ft. wide, where the river has cut its way down through a ledge of hard trachyte, forming a cañon with vertical side walls. Above this ledge the section across the cañon has a bowl-shaped form like the end of an ellipse. The dam has been projected to the ultimate height of 180 ft. above the river bed in the bottom of the cañon, or about 110 ft. above the valley proper. The length of the dam at the height of 180 ft. will be about 500 ft. The crest width, 10 ft. above the water line, or spillway

TERRACE RESERVOIR DAM
ALAMOSA RIVER, CONEJOS CO., COLORADO

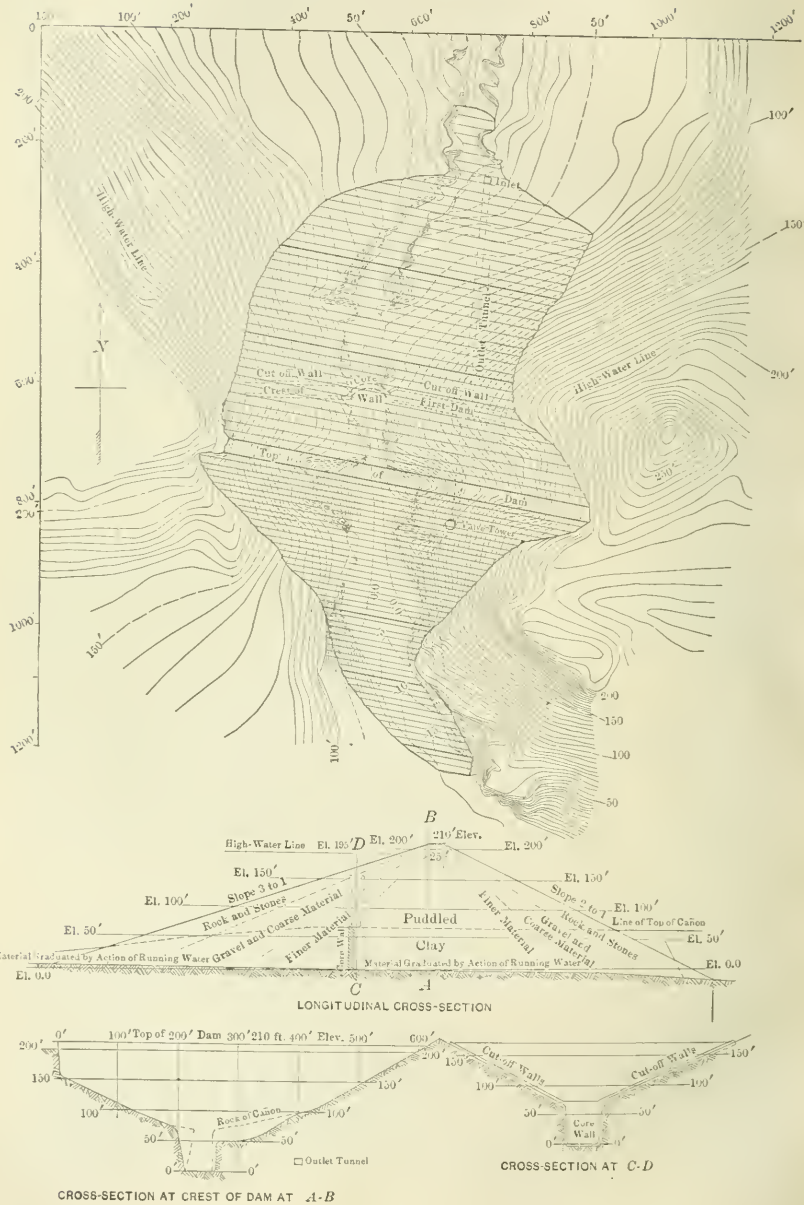


Fig. 4.

PLATE [illegible]
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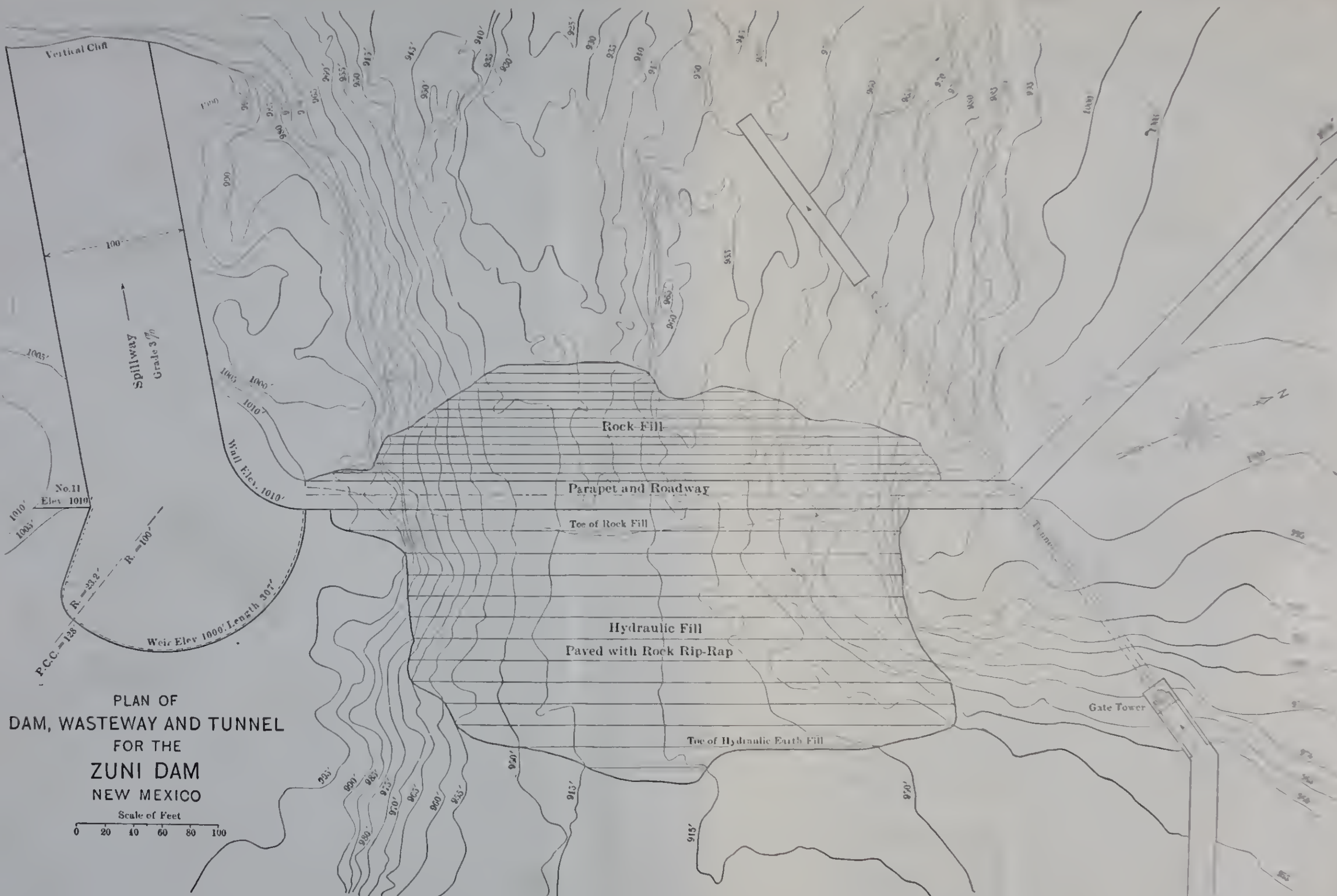
SCHUYLER ON
HYDRAULIC-FILL DAMS.



PLAN OF
DAM, WASTEWAY AND TUNNEL
FOR THE
ZUNI DAM
NEW MEXICO

level, will be 20 ft.; the up-stream slope being 3 on 1, and the down-stream slope 2 on 1. The plan and sections are shown in Fig. 4. The capacity of the reservoir at the 170-ft. level is about 18 000 acre-ft., of which only 6% is in the lower 70 ft., and 10% is in the upper 5-ft. layer.

There are ample evidences that the valley to be occupied by the reservoir was once a lake, which was carved out of the mountains by glacial action. A gap in the rim of the reservoir, to the north of the dam, exhibits a great mass of glacial drift deposited as a lateral moraine, and consisting of sand, gravel and boulders, with an intermingling of clay. This is sufficiently convenient to be used for sluicing into the dam. On the south side the trachyte has decomposed and disintegrated to considerable depth, yielding excellent material for hydraulic filling, as the portion that is loose enough to be moved by the hydraulic giant is composed of spawls and splinters of hard rock, intermingled with clay and loam.

A ditch and flume, Fig. 1, Plate XLVIII, 5 miles in length, has been built from the Alamosa River, taken out at a point above the reservoir at sufficient elevation to deliver water about 200 ft. higher than the top of the dam. This ditch was completed too late in the fall of 1905 to begin sluicing, but as soon as spring opens in 1906 it is expected that the work will begin by ground-sluicing and hydraulic mining at both ends of the dam. The dam is being built by contract for 18 cents per cu. yd. A tunnel, 725 ft. long, 7 ft. high, and 10 ft. wide, has been excavated for carrying the stream during construction and as an outlet to the reservoir. The outlet gates will be operated from a shaft, 75 ft. deep, sunk through the solid rock to the tunnel, at a point about 200 ft. down stream from the crest of the dam. The engineer in charge of the work is Mr. E. W. Case, of Colorado Springs. Fig. 2, Plate XLVIII, and Figs. 1 and 2, Plate XLIX, illustrate the situation at this remarkable dam site.

The method of building the dam by hydraulic sluicing was first suggested by T. W. Jaycox, M. Am. Soc. C. E., State Engineer of Colorado, by whose recommendation the writer was engaged to report upon the plans and make such modification as, in his judgment, seemed desirable.

In the lower 70 ft. of cañon section, on the center line of the dam, a concrete wall, 15 ft. thick, has been built, curved on a short radius of 50 ft., to serve as a core-wall to the height of 70 ft. Connecting with this wall, and extending up the slopes to the high-water line, are two parallel concrete walls, 25 ft. apart. These walls are built in trenches cut in the bed-rock, and extend 2 or 3 ft. above the original surface. They are designed to intercept seepage along the surface of the bed-rock, and are to be enveloped in the body of sluiced earth or clay composing the center core.

The completion of this dam in the summer of 1906 is looked forward to with great interest.

Hydraulic-Fill Dams of the Mexican Light and Power Company.

The Mexican Light and Power Company, Limited, a corporation supplying electric light and power to the City of Mexico and to surrounding towns, is constructing four dams by the hydraulic process, on the Necaxa and Tenango Rivers, about 100 miles north-east of the National Capital, two of which are to be 175 and 190 ft. in height, respectively, and of unusually large volume. All these works are under the general direction of F. S. Pearson, M. Am. Soc. C. E., who is Vice-President and Consulting Engineer of the Company. Albert Carr, M. Am. Soc. C. E., is Manager of Construction, and Mr. F. S. Hyde is Resident Hydraulic Engineer. The responsibility for recommending this type of dam for these huge embankments rests with the writer, who was called upon in January, 1905, to report on the subject, and has since been retained as Consulting Engineer to supervise their construction. His reasons for recommending the hydraulic-fill were, briefly, that in his judgment the foundations as exposed by test-pits over the dam site and as shown in the outlet tunnel of the reservoir at Dam No. 2, which had pierced the hill against which one end of the dam is to rest, are unsuitable for supporting a high masonry dam such as had been projected, although sufficiently stable and entirely satisfactory as a base for an earthen dam. Earth dams of the ordinary type, if made of great height, involve the handling of such enormous masses of earth as to become prohibitive in cost, and therefore, up to the present date, the limit of height of earth dams has been about 130 ft. Where earth filling costs from 50 to 80 cents,

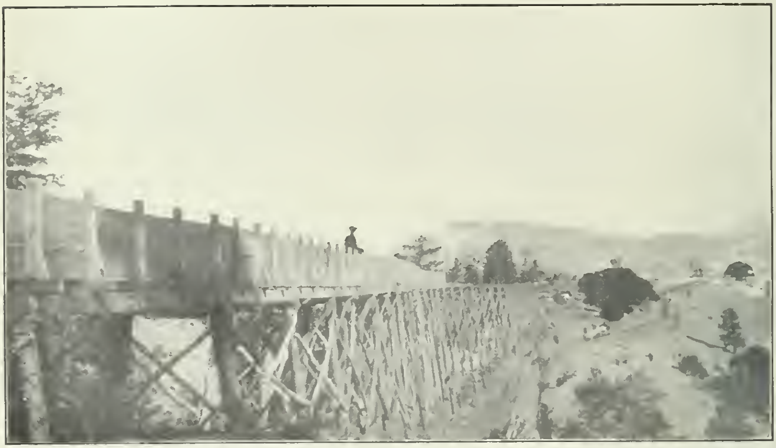


FIG. 1.—FLUME UNDER CONSTRUCTION FOR CARRYING WATER FOR SLUICING OPERATIONS ON TERRACE DAM.

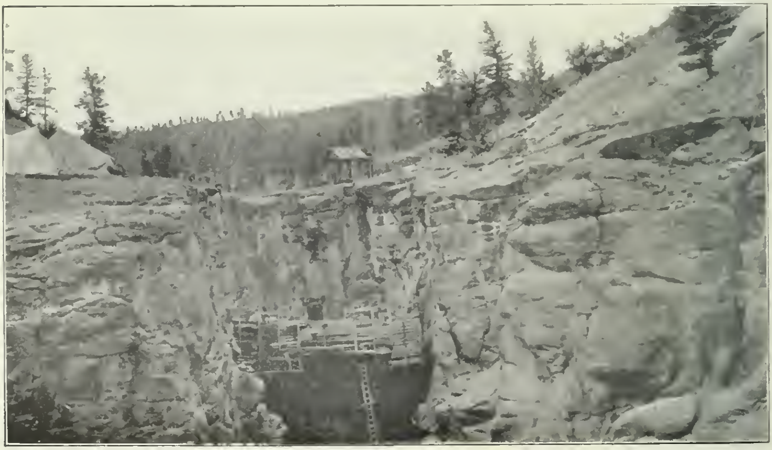


FIG. 2.—TERRACE DAM. CONCRETE CORE-WALL UNDER CONSTRUCTION.

and clay puddle from $1 to $2 per cu. yd., when selected, spread, sprinkled and rolled with the extreme care needed in high earth dams, it is evident that the capital invested becomes too great, in ordinary situations, to warrant the risk of building earth dams by the ordinary methods much beyond the limit of 120 or 130 ft. In reality, there is no more reason why an earth dam, if it can be built at sufficiently moderate cost to justify the expense, should not be 250 to 300 ft. in height, provided the materials used are suitable, are placed with sufficient care, and it is of such massive proportions as to afford an ample factor of safety.

In the case of Dam No. 2, at Necaxa, a high dam was required in order to afford the desired storage for equalizing the stream, and to waste as little water as possible. As between an earth dam of the ordinary type and a hydraulic-fill for this situation, there was no comparison, either in cost, length of time needed for construction, or in stability and security when finished, every condition being in favor of the latter. Fortunately, there is right at hand, at convenient distance and proper elevation, a sufficient mass of broken limestone, intermingled with pure yellow clay, to build the dam of any height desired, with the use of streams of water under high pressure brought to the site by a ditch. The proportions of rock and clay are about equal, and the separation of the two materials by water, depositing the rock on the outer side and the clay in the center, will manifestly make an embankment more stable and secure than could have been made with the earth which was available, and at far less cost and in shorter time.

The volume in Dam No. 2, when finished, will be 1 525 000 cu. m. (1 993 700 cu. yd.), and the dimensions as follows:

Length on top	1 220 ft.
Height above lower toe	190 "
Height above upper toe	178 "
Down-stream slope	2 on 1
Up-stream slope	3 on 1
Width of crest ....16.5 m. =	54 ft.
Base, measured along stream bed	975 "

The ditch for water supply for sluicing is 17.5 km. long, and delivers water for building Dam No. 3 as well as Dam No. 2, the former being on the same stream and about 6 miles above No. 2. The ditch is of sufficient capacity to deliver 70 cu. ft. per sec. to Dam No. 3, and 53 cu. ft. per sec. thence to Dam No. 2. The height of the ditch above the base of Dam No. 2 at its point of delivery is 728 ft. This will give an effective head for mining the material, at the successive levels opened out for delivery, from the base to the crest of dam. Plate LIII shows the general plan of the dam, and the photographs on Plate LII illustrate the location of the dam, and the excavation for the core-wall, etc. The material will all come from one side at both dams.

The plan of operation proposed at each of these dams is to build up the base to a height of 50 ft. by delivering material through pipes supported on trestles. These will be built parallel to the center lines on each side, and at a convenient distance back from the toe of the slope, in order to permit of delivery to the slopes with the shortest possible side flumes. The lower 25 or 30 ft. will require delivery from one side of the flume, until the slope reaches a point beneath the trestle, when the side flumes will be reversed, delivering toward the center of the dam until the limit of height of the first trestle is reached.

Thus the trestles will project out of the slopes on either side. From about the 50-ft. level to the top it is planned to deliver sluiced materials through flexible-jointed pipes supported on pontoons floating in the pond maintained on top of the dam. The stream of tailings from the pipe will be distributed evenly along or near the line of the slope, first on one side and then on the other, thus building up 2 or 3 ft. in height in a continuous "windrow" or levee. The larger stones contained in the material will be raked out and laid by hand on the slopes, forming faces of dry rock wall. At Dam No. 2 it is expected that the proportion of rock in the borrow-pit will be so large that very little additional rock will have to be provided by mechanical means to maintain the slopes properly. Fig. 5 shows a cross-section of Dam No. 2.

The spillway excavation will be made with one of two hydraulic giants to be installed. The material in the spillway gap is chiefly clay, without sand or rock. The maximum depth of excavation for the spillway will be more than 100 ft. The other giant will work in a borrow-pit where loose rock predominates. The two

FIG. 1.—UP-STREAM SIDE, CONCRETE CORE-WALL TERRACE DAM.

FIG. 2.—LOOKING UP STREAM THROUGH THE GORGE IN WHICH THE TERRACE DAM IS BEING BUILT.

streams of material can be directed and combined in any desired proportion, and rock and clay can be mixed from either pit at the head of each of the two pipe lines delivering material to the dam.

At Dam No. 3, conditions are somewhat different, and the comparative scarcity of loose rock in the clay bed to be used as the principal borrow-pit may make it necessary to bring rock by other mechanical means to fill out the slopes with the desired quantity of stable material. It is not intended that at either dam shall there be the slightest possible chance for a sloughing of the slopes, or an outward movement of the mass during construction by reason of a lack of a sufficient thickness of large rock to give ample drainage and friction, and consequently ample stability, for the slopes will be flatter than the natural rough angle of repose of such rock. Fig. 2, Plate LII, shows the foundation of Dam No. 3, and Plate LIV is a contour plan of the site.

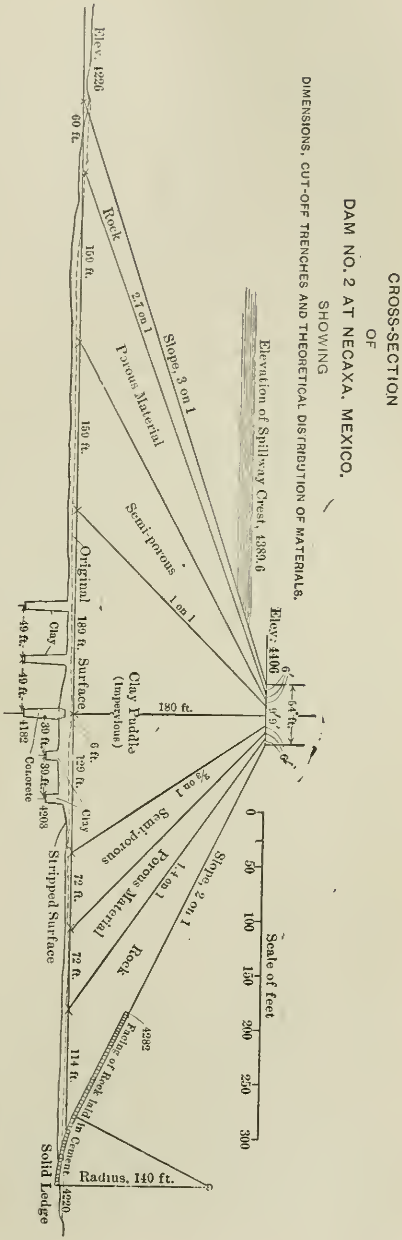


Fig. 5.

Dam No. 1 is a low barrier being built by the company on the Tenango River, a parallel stream. It is to be used for diverting the water through a 3 000-ft. tunnel, into the Necaxa River, immediately below Dam No. 3. Its maximum height is about 30 ft. The earth is being placed by ground-sluicing from a ditch, and filled in between slope levees built up by hand from the boulders and gravel of the creek bed. This is a comparatively unimportant structure, not especially worthy of further notice, except as a specimen of ground-sluicing without water under pressure. Its construction is illustrated by Figs. 1 and 2, Plate L, and Fig. 1, Plate LI.

Dam No. 4 is being constructed near the headwaters of the Necaxa River, 25 miles above the Village of Necaxa, at a place called Laguna, where an extensive reservoir is being formed to store water to be brought from an adjoining stream by a long tunnel, as well as to store surplus water from its own local water-shed. The storage capacity is about 2 500 000 000 cu. ft. The dam will be about 2 880 ft. long on top, 80 ft. maximum height, and will contain 400 000 cu. yd. For temporary storage, the upper toe of the large dam is being built to a height of 40 ft., as a combination rock-fill and earth embankment, with a core-wall of plank, backed by a layer of clay, 10 ft. thick, carefully placed against it on the upper side. The lower half of the dam below the plank core-wall is composed of loose rock to a height of 40 ft., for temporary storage. This will form an integral part of the larger dam, which will be sluiced in place chiefly with the hydraulic monitor. It will be necessary to pump the water for this work, and an installation, similar to that used at Lake Frances Dam, has been planned for this purpose. The elevation at the base of the dam is 2 144 m. (7 032 ft.) above sea level. It is planned to draw water from this reservoir, after the larger dam is built, through a tunnel about 1 500 ft. long, discharging into the adjoining basin of Los Reyes River on the north. About 2 miles down stream from this tunnel a fifth dam is planned in the narrow gorge of Los Reyes River, where similar methods of construction will probably be used. A conduit about 5 miles long will lead from Los Reyes Reservoir to the head of a power drop, where a fall of 2 380 ft. to the level of Reservoir No. 3 is to be utilized for a second installation, where 100 000 h. p. will

Fig. 1.—Ground-Sluicing on Dam No. 1. Tenango River.

Fig. 2.—Method of Distributing Sluiced Materials Through Pipes. Dam No. 1. Tenango River.

be developed in the near future with the auxiliary supply to be diverted from adjoining water-sheds into Reservoirs Nos. 4 and 5, as described.

Projected Hydraulic-Fill Dam on Wichita River, Texas.

An earth dam, 3 000 ft. long on the crest and 80 ft. high, above the river bed, has been projected on the Wichita River, some 50 miles above the City of Wichita Falls, Texas, to be sluiced in place with water pumped from the river. The dam will create a reservoir covering about 10 000 acres, and store about 280 000 acre-ft., to be used for irrigation in the fertile Red River Valley. Borings have shown the existence of a heavy bed of blue and red clay, in the river valley at the dam site, at a maximum depth of 20 ft., overlaid by a stratum of quicksand. As the clay extends up the sides above the water line, the sand stratum is confined to the channel. It is proposed to drive steel sheet-piling across the stream at the center line of the dam to serve as a cut-off for this quicksand layer, and avoid the excavation of the base to the shale bed-rock. The steel piling can be driven into the clay, and will form an impervious barrier to the underflow in the overlying sand. The piles will be allowed to project upward a few feet into the hydraulic-fill, and be enveloped in it.

The earth to be used for building the dam is a red, sandy soil, with a large percentage of clay intermingled with sand and gravel. The top, or surface layer, of the plains above the crest level of the dam, contains an abundant quantity of hard rock nodules of the general sizes of river gravel, but with enough angular protuberances and cavities to show that it was not water-borne, but is the hard residuum of disintegrated rock, which has melted and been carried away as clay or sand, or remains in place as surface soil.

Plans for this dam have been submitted by the writer, and approved by the promoter of the enterprise.

Hydraulic-Fill Dam in Brazil.

The São Paulo Tramway, Light and Power Company, Limited, has a power plant on the Tieté River, in the State of São Paulo, Brazil, 22 miles below the thriving city of São Paulo, where 15 000 h. p. is generated and transmitted to the city. To equalize the flood flow, and make up for shortage in dry seasons, the company is constructing a dam 14 miles above the city, on a branch of the river known as the M'Boy Guassu. The dam will be about 2 000 ft. in length, and from 40 to 45 ft. high, forming a reservoir covering an area of about 5 000 acres, and storing approximately 100 000 acre-ft. of water. The plan proposed contemplates the driving of a row of sheet-piles, from 20 to 30 ft. below the surface in the center of the dam. Steel piles will be used for the river channel, and tongued and grooved wooden piles for the remaining distance. The sod and surface soil to the depth of 2 ft. will be stripped from the base of the dam and built in the form of a levee at each slope footing. Between these levees earth will be sluiced from a hill at one end of the dam, through a flume and pipe laid on a trestle with a grade of 3% from the hillside to within 300 or 400 ft. of the extreme end. This trestle will be built on the center line to the full height required to complete the dam without further addition. It will be 75 ft. in maximum height. Lateral flumes will extend to the slopes on each side at intervals of 50 ft. The materials consist of disintegrated granite, containing a sufficient admixture of clay to yield a water-tight material for the core, and give sufficient coarse, porous and gritty material for the slopes to enable them to have the required friction to stand where placed without sloughing down to flatter angles of repose. The disintegration of the surface rock has left a solid bed of red and gray clay covering the surface to a depth of from 5 to 15 ft., but in the interior of the hill it will probably be necessary to loosen the material by blasting in advance of the water jet.

For protection from muskrats and other burrowing animals, a corrugated-iron fence will be carried up from the sheet-piling to the top along the longitudinal center of the dam. This fence will not be made so tight that water may not pass through it, so that there can be no great inequality in the level of the water on either side of it in the pond on top of the dam, as the work progresses, and therefore no pressure upon the plates. These will be nailed to longitudinal girts bolted at intervals to the center line of posts in the trestle.

The water used for hydraulicing will be pumped under a pressure almost equal to a maximum head of 450 ft., including friction

Fig. 1.—Liquid Earth Being Deposited Through Pipes on Outer Slopes of Dam No. 1, Tenango River.

Fig. 2.—Looking Up Stream at Site of Dam No. 2. at Necaxa. Showing Stripped Abutments for the Dam on Each Side.

in the pipe and the height of the position of the monitor above the river bed, giving a pressure of about 150 lb. per sq. in. at the nozzle of the monitor. A volume of 10 cu. ft. per sec. will be used.

The minimum flow of the river is about 10 cu. m. per sec. (353 cu. ft. per sec), and will be carried through several concrete culverts.

The total volume of the dam is computed at 215 000 cu. m.

The work is in charge of Mr. M. M. Murtaugh, whose successful management of construction of the Snake River Dams, in his capacity of Assistant Manager, gave him a great familiarity with hydraulic-fill construction. The plans have been prepared by the writer, after an examination of the site.

Failure of the Snake Ravine Hydraulic-Fill Dam, Illustrating Improper Application of the Principle.

The failure of an engineering work is often quite as instructive to the engineer as the record of successful construction, and a case of this sort may be cited in two disastrous attempts to build a hydraulic-fill dam in Snake Ravine, on the line of the Turlock Irrigation Canal, in Merced County, California, both of which failed by reason of the non-observance of correct methods. An account of this work, prepared for *Engineering News*,* by J. B. Lippincott, M. Am. Soc. C. E., details the main facts, from which the following description has been largely compiled:

The purpose of the dam across Snake Ravine was to save the construction of 1 500 ft. of side-hill canal and flume on the line of the main canal of the Turlock Irrigation District. The capacity of this canal is 1 500 cu. ft. per sec., and it was located to discharge into the basin above the dam, after passing through a divide with a cutting 50 ft. deep and 800 ft. long. The material of this cut was auriferous gravel in sizes varying from 1 in. to 2 ft., embedded in red clay, with some sand. In its original position it is extremely firm and impervious, standing for years when mined, in walls nearly vertical, as high as 50 ft. A canal used for hydraulic mining, with a capacity of 24 cu. ft. per sec. was already in existence 100 ft. above the grade of the irrigation canal and ½

* October 20th, 1899.

mile distant. All these conditions were simply ideal for the cheap construction of a first-class dam by the hydraulic process.

It was intended to make the dam with the material of the big cut, sluiced down by the water of the mining canal.

The dam required to be 294 ft. long on top and 64 ft. high in the center. The top width was made 12 ft., at a height of 4 ft. above the maximum water line of the canal, with slopes less than 2 to 1.

The first trial was made in 1893, but the dam went out when it had reached the 30-ft. level. The second dam was then built, and stood for some time, but went out when first tested, on June 14th, 1898. The second dam was built upon the wreck of the first, and in the same manner.

The simplest and most elementary precautions for any dam building appear to have been neglected, and no attempt was made to connect with bed-rock or remove the hydraulic mining tailings with which the ravine was filled before construction.

The plan adopted was to build up the lower toe with scraper teams, keeping it 2 ft. above the remainder of the dam. Material was then washed down with a hydraulic jet at the big cut and from the adjoining side hills of the basin, and sluiced to the dam site in the natural watercourses. An inclined shaft of wood, 2 by 4 ft. in the clear, was built inside the outer slope for carrying off waste water, and a vertical shaft of similar character was carried up at the upper toe, both connecting with a waste box of 2-in. Oregon pine, 4 ft. square, passing through the dam at the base. As no flume or pipe was used to carry the material to the outer slopes, where the gravel and boulders would be deposited, the result was that the heavier portion of the sluiced material was dropped on the way, and only the fine silt reached the dam. Without any provision for drainage, this was always in an unstable condition, constantly shaky and vibrating when jarred. It finally slid out when it had reached a height of 30 ft.

The second attempt was made with sufficiently greater care to permit of its completion to the top, but the waste box or culvert crushed in when the structure was only 40 ft. high, and could not be used thereafter. The dam was then finished to the top with carts. With water standing only a few feet deep in the basin the leakage through the dam amounted to 1 cu. ft. per sec. There was

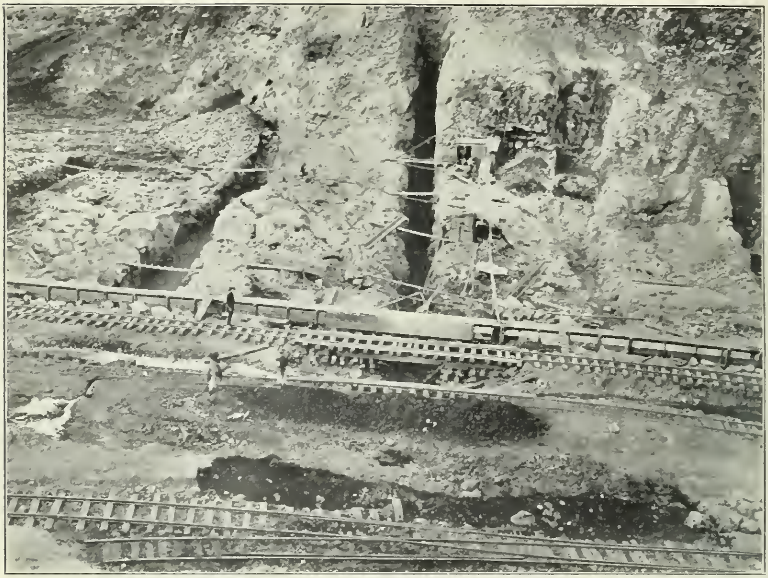


Fig. 1.—Excavation for Cut-off Core-Wall and Lateral Trenches, Dam No. 2, Necaxa.

Fig. 2.—Foundations and Double Eight-Foot Outlet Pipes, for Dam No. 3, at Tezcapa.

more leakage through the material deposited by carts than through the stratified material that was sluiced into position.

The intake canal passes near the east end of the dam, and a wooden box, 4 ft. square, for a new wasteway, was put in the west end of the dam 10 ft. below the top. The intake canal is located on a steep hillside. It soon leaked and sloughed off a piece of the hillside immediately next to the dam, which then began to leak around that end. The waste box, which was also leaking, was then opened in the west cut, and washed out a large hole from the side hill and dam on its lower side. The dam was leaking at its base also. Attacked and weakened on both flank and center, the entire structure gave way and was pushed *en masse* down the ravine at a velocity of from 6 to 10 ft. per sec. It thus traveled about 1 000 ft., and was dumped into the Tuolumne River, damming the stream temporarily to a depth of 20 ft. The Superintendent and two dogs were on the dam when it gave way, and they were carried entirely across the river unhurt, although, on the trip, the man's shirt and pockets were filled with sand and mud.

After this accident the idea of building a dam was abandoned, and the ravine was crossed with a flume.

The case is cited as an example of the misuse of a most favorable opportunity for building a first-class dam with the greatest possible economy and with the best of materials, for lack of experienced and intelligent supervision. The work was done by a so-called "practical" contractor, without an engineer.

Tyler, Texas, Hydraulic-Fill Dam.

The hydraulic-fill dam built at Tyler, Texas, for water-supply storage for that city, to which allusion was made in the early part of this paper, is 32 ft. high, 575 ft. long, and contains 24 000 cu. yd. It was built entirely with water pumped through a 6-in. pipe by a Worthington steam pump of 750 000 gal. capacity, which happened to be located exactly at the dam site. It was the original city supply pump. The entire cost of the dam was $1 140, including the spillway, outlet gate and pipe. This was an average of only 4¾ cents per cu. yd. Earth, from an adjoining hill 150 ft. high, was loosened by water forced through a 1½-in. nozzle, under a pressure of 100 lb. per sq. in., and carried down to the dam, through a pipe. The material was 65% sand and 35% clay. The dam has been in service for nearly 12 years.

La Mesa Dam.

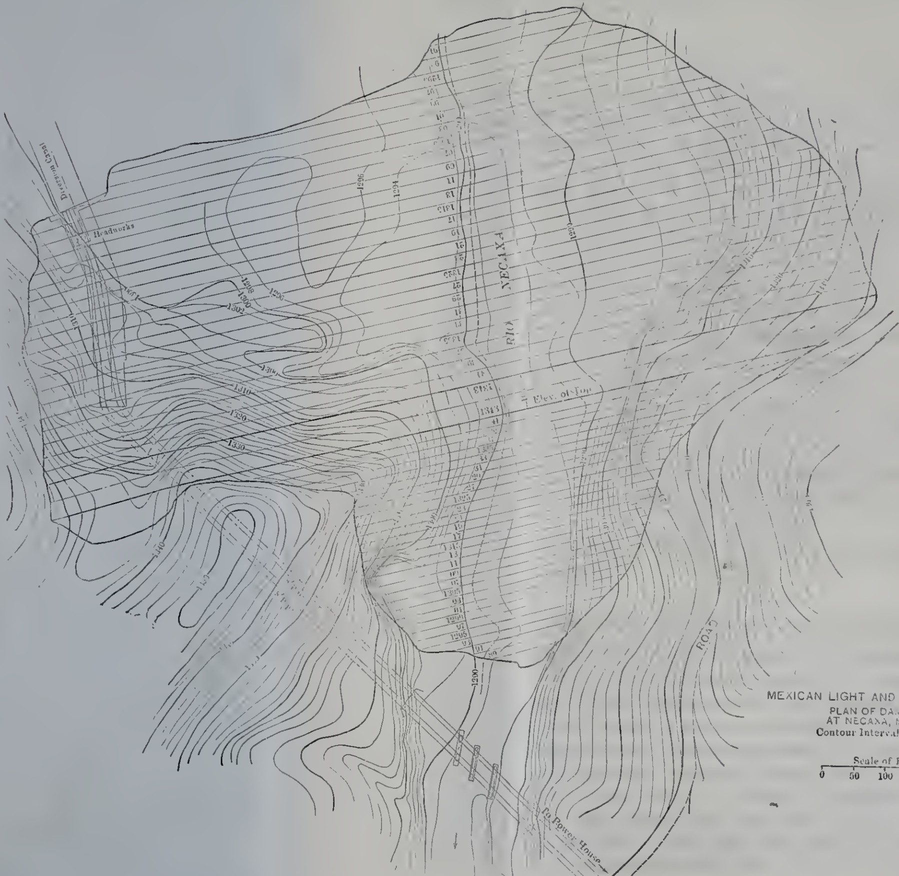
This hydraulic-fill structure is located near San Diego, Cal., and was built for storing waste and surplus water from the supply of the San Diego Flume Company. It is 66 ft. high, 470 ft. long, and contains 38 000 cu. yd., consisting of soil, gravel and cobble stones, loosened, transported and deposited by water. The dam was begun in February, 1895, and in the first 30 working days 55% of the work was finished. The remainder of the work took much longer, as the material became scarcer, and the distance to move it became greater, but it was completed and put in service the same year, and has been used constantly ever since. There has been a leakage of clear water coming out at the base ever since the completion of the dam, amounting to 0.2 cu. ft. per sec. when the water reaches the 54-ft. level, but it was never a source of alarm, or considered in the least menacing to the safety of the structure, and there are very good reasons for believing that it comes through the crevices of the bed-rock cliff at the north end of the dam, and not through the hydraulic-fill at all. The inner slope was covered with asphalt concrete, in an attempt to stop this leakage, but without success.

Both the La Mesa and Tyler Dams were designed and built by Mr. J. M. Howells.

The Proposed Lake Como Hydraulic-Fill Dam, in Montana.

Almost ideal conditions for the economical construction of a large dam by the hydraulic process are presented at Lake Como, in the Bitter Root Valley, Montana, where the Dinsmore Irrigation and Development Company is planning to construct a storage reservoir for irrigation supply. The dam projected will be about 2 500 ft. long on top, and have a maximum height of 93 ft. With the dimensions planned, the volume of material to be moved will be 800 000 cu. yd. The valley at the dam site, as well as the precipitous mountain slopes on either side, are composed of practically one class of material, consisting of glacial moraine gravel, boulders,

PLATE LIII.
TRANS. AM. SOC. CIV. ENGRS.
VOL. LVIII, No. 1043.
SCHUYLER ON
HYDRAULIC-FILL DAMS.



MEXICAN LIGHT AND POWER CO LTD
PLAN OF DAM NO. 2
AT NECAXA, MEXICO.
Contour Interval, 2 Meters

Scale of Feet
0 50 100 150 200

sand and clay, or glacial flour. A ditch already built passes the dam at an elevation of 300 ft. above it, with ample water supply for sluicing. The dam is to be built during the current season of 1906, under the supervision of Mr. H. R. Lord, Chief Engineer, and the writer as Consulting Engineer.

Lake Como is a beautiful sheet of water, covering 196 acres, and surrounded by pine forests reaching to the water's edge. The water is clear, cold, and full of trout. The lake is said to be several hundred feet deep. The addition of 75 ft. to its depth will give the reservoir an area of 1 000 acres, and a capacity of 60 500 acre-ft.

Conclusions.

The general principles involved in successful dam building by the hydraulic process are to be inferred from the various experiences in actual works detailed in the foregoing pages. The writer's only apology for describing these experiences with such minuteness is in order that the application of these principles, and the manifest deductions to be drawn from each distinct case, may be more clearly elucidated to those unfamiliar with this class of construction, which is not so common as to be very widely known.

In the various dams described, a great range of quality of materials has been encountered, each of which requires a special treatment, and a great variety of conditions has been met, necessitating the exercise of ingenuity and good judgment. As far as the writer's experience has gone, it is his opinion that the best material for hydraulic-fill dams is rounded gravel, sand and boulders, intermixed with about 25 to 35% of clay. The rounded rocks roll more readily than broken angular chips, and clay acts as a lubricant to assist in transporting the heavier materials.

The most difficult material with which to build such a dam is pure clay unmixed with sand, because it is unstable until the water is drained from the mass, and drainage is very slow. The shrinkage, therefore, is much greater than in other materials, and there is greater likelihood of the opening of shrinkage cracks, months after the work is finished, through which leakage could start if the dam were put in service too soon after completion. When clay is finally consolidated, however, it makes a dam which can have no superior for water-tightness.

As a core for coarser and more stable materials, clay is extremely desirable, and its presence, with sand, gravel and rock in sufficient proportion to form one-fourth to one-third of the volume of the dam, to be segregated by water and placed in the center of the mass, makes it more easily worked, more useful, and more valuable for safe dam construction than in any greater proportions.

While clay is doubtless the most impervious of all earth, it is not indispensible in the building of water-tight earth dams. The volcanic ash soil of the Snake River Valley has none of the charactertistics of clay, and yet it is so finely divided as to make a water-tight embankment when properly moistened and compacted. Glacial flour, which has no resemblance to clay, will also make an equally good dam.

The essential condition of any earth dam is to secure a sufficient mass of materials having particles so fine and so closely placed in juxtaposition as to oppose friction to the water seeking under pressure to penetrate between these particles to an insuperable degree. There is no limit, except one of cost, to the height which it is possible and safe to build earth dams, if they be made of sufficient dimensions to fulfill the requirement that frictional resistance to the passage of water shall be practically insuperable, or if water in moderate amount does find its way through the mass, that it shall be robbed of all velocity and power to transport any of the particles of the embankment with it, or in other words, that it shall be rendered harmless, and issue as water issues from a filter.

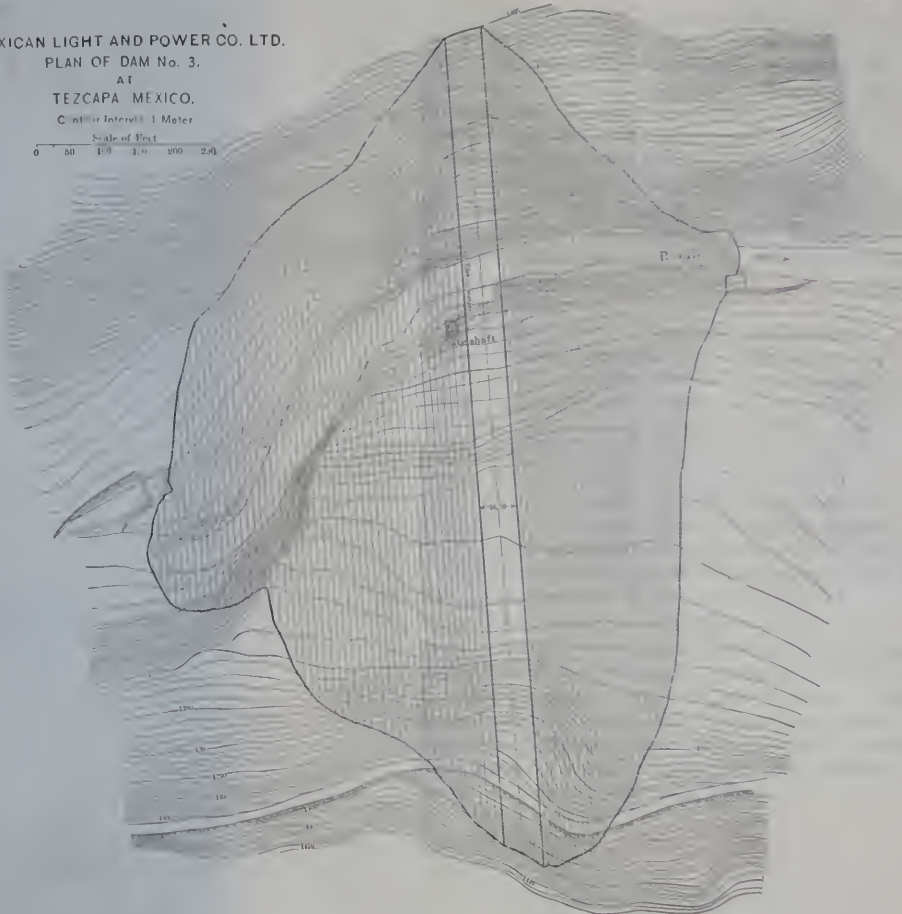
The hydraulic-fill process cheapens the cost of handling and compacting earth in dams so greatly as to extend widely the practicable limits of dam building, by making it feasible to increase the bulk of any dam far beyond the usual dimensions, without exceeding reasonable limits of cost. The great dams proposed in connection with the Panama Canal may possibly be much simplified in design, cheapened in cost, and increased in bulk and stability by the application of the principles herein outlined.

MEXICAN LIGHT AND POWER CO. LTD.
PLAN OF DAM No. 3.
AT
TEZCAPA MEXICO.

Contour Interval 1 Meter

Scale of Feet

0 50 100 150 200 250



DISCUSSION.

Mr. Herschel.

Clemens Herschel, M. Am. Soc. C. E. (by letter).—To the principles laid down at the beginning of this paper, the writer would add one more; namely: an earth dam should have, to be safe for an indefinite period of time, somewhere in its cross-section, a durable element or member of construction, capable of resisting the attacks of burrowing animals not infrequently experienced from the up-stream, as well as the down-stream, side, and from beneath the surface, for the time being, of the water; one, at the same time, capable of arresting the further progress of any veins of water that may penetrate the interior of the dam, whether caused by settlement, sand strata, as mentioned by the author, drying-out cracks, burrowing animals, or any other cause.

There appears to be no logical reason why an earth dam, built by the hydraulic-fill method for excavating, conveying and solidifying the earth of which it is composed, in preference to steam, or any other kind of shovel work, carting or railroading, and spreading, sprinkling and rolling, should be built according to materially different principles for insuring safety and permanency in one case than in the other.

The author of the paper places the reason for using the hydraulic-fill method in a saving of cost by that method for the operations just described, and no other; a reason very well founded in a multitude of cases, and making the examples and data given of the highest interest to all concerned with the construction of reservoir dams.

Still, the method has its defects and limitations. It needs, that it may be adopted, a body of earth situated at a higher elevation than the body of the dam to be built, and not too far away. Possibly some day we shall be able to pump a heavy mixture of earth and water up hill; at present the most that can be done is to pump up the water, and let it and the earth mixed with it run down again.

It necessitates flatter slopes, that is, a greater volume in the earth dam, than dams built by the ordinary method, thus offsetting, to some extent, the saving effected in price per cubic yard of material deposited in the dam by the hydraulic method.

It seems to require, for one reason or another, a longer time than would otherwise be necessary.

No one is probably more disposed than the writer to admire the resourcefulness and skill of the Pacific Coast hydraulic engineers in general, exhibited these 40 years in their works, and of the author in particular, but the nature of the demands made upon them has insisted that they meet those demands in nearly though not in all cases with works built with a minimum expenditure of labor

Mr. Herschel. and of cost, even if at the expense of a certain degree of permanency.

Thus, the entrancing looking wooden flumes of the cañons have already rotted out to such an extent that the proprietors and the water-buying public, no less than the engineers, have been made ready to consider more permanent structures. Signs are not wanting that wooden stave pipes are going the same way that the wooden flumes did. And similarly, all forms of dam construction will have to be designed and built with every regard given to permanency, as in the older settled portions of the country; the more or less temporary forms of structure having served their high and useful purpose of enabling the country to be settled.

These remarks, of course, do not refer to the earth and stone of the dams shown, as there can be nothing more durable than earth and stone, but are made in behalf of a durable core-wall of some sort, always to be placed in any and every earth dam, but which appears to be missing in all the designs illustrated. There is an instance of a plank fence in one case, but that cannot be considered as a durable core-wall, either against the tooth of time, or, when placed up to the flow line, against the tooth of the animal creatures that inhabit such reservoirs.

In the case of water-power canals, some 10 ft. deep, canal banks built with a plank core-wall have remained water-tight after the planks had been converted into a streak of loam; but with the danger ever present, and occasionally occurring, that by reason of some burrowing animal, the canal bank would begin to leak, or even give way.

In the discussion on "Reservoirs with High Earthen Dams in Western India,"* the writer has given the results of his experience in the construction of earth dams, and his ideas of the essentials of such structures. These are, that an earth dam is a structure composed of an unstable water-tight curtain fastened in a water-tight manner across the water-tight surface of a valley; extending up to a horizontal line, and made stable or supported on each side by an earth fill, properly placed and protected against wave and rain wash on the outside and top. There would be no need of a core-wall, if every man at work in the construction of the dam had the intelligence, will and ability to put into it nothing but perfect work. But such embankments are the outcome of the work of hundreds, or of thousands, of the very lowest class of labor, and, strive as the directing mind may, imperfections will creep in, and remain in the completed work. If water once commenced to penetrate into an earth dam, or to percolate, the channels of percolation could readily enlarge, unless there was somewhere a reliable water stop;

* *Minutes of Proceedings*, Inst. C. E., Vol. CXXXII, p. 252, 1897–98; *Engineering News*, Sept. 7th, 1905.

Mr. Herschel.

and to furnish this is the office of the core-wall. Even if the core-wall should crack, it would still fulfill an important part of its duty; for at that point the channel of percolation could not enlarge, thus limiting the amount of water that could percolate, and in all probability causing the leak to close itself instead of enlarging.

None of the arguments ever brought against the use of a core-wall in earth dams have had any weight when tested by experience, or an analysis; and nothing weightier than argument unsupported by experience ever has been brought against them.

The Canistear Dam, in New Jersey, described in the paper before cited, was built between June 10th and November 1st, 1896, less than 5 months, and has been in use ever since, with only nominal repairs. All the work on this reservoir, several dams included, cost about $341 000, and would have cost about $295 000, if not built by night and day work; if built only by day work, as it has been practiced on works built under the supervision of the writer, it would have required more than double the time stated. Considering the bill of quantities which follows, the writer would claim for this work the record time anywhere, and at least a fair record for low cost for that quality of work on the Atlantic Coast. This may also serve to prove that the core-wall, and the spreading, sprinkling and rolling method of building earth dams does not necessarily entail years of time in which to construct them. On the contrary, it may in settled localities excel, in this frequently very important particular, the hydraulic-fill method.

The bill of quantities of the Canistear Dam was as follows:

Main dam and reservoir, clearing......	440	acres.
Main dam and reservoir, grubbing....	440	"
Excavation on dam site...............	11 000	cu. yd.
Gate-house cut, rock..................	1 200	" "
Gate-house cut, earth..................	5 700	" "
Core-wall excavation, rock............	2 450	" "
Core-wall excavation, earth............	2 100	" "
Core-wall, concrete	8 000	" "
Dam, earth, including paving..........	98 000	" "
Gate-house masonry, brick............	100	" "
Gate-house masonry, stone.............	1 500	" "
Gate-house masonry, concrete..........	150	" "
Gate-house fittings, pipes, valves, stairway, etc.		
New road	11 562	ft.

OVERFLOW DAM.

Excavation, earth	2 700	cu. yd.
Excavation, rock	1 450	" "
Masonry	4 500	" "

Mr Herschel.

AUXILIARY DAM, No. 1.

Clearing site of dam..................	850	cu. yd.
Excavation for core-wall, rock.........	80	" "
Excavation for core-wall, earth.........	150	" "
Core-wall, concrete	300	" "
Earth dam, including paving..........	1 400	" "

AUXILIARY DAM, No. 2.

Clearing site of dam..................	950	cu. yd.
Excavation for core-wall, rock.........	20	" "
Excavation for core-wall, earth.........	600	" "
Core-wall, concrete	750	" "
Earth dam, including paving..........	2 450	" "

Closely allied with the question of time is that of the necessary volume of the dam. That excess of volume may merely embarrass construction, and, at the same time, in no wise contribute to the safety of an earth dam, is proven, among other ways, by the cross-section proposed for the Gatun Dam shown on Plate XIV of the Report of the Board of Consulting Engineers for the Panama Canal.

This is the type of dam presented only by the lock-canal minority of that tribunal, the Board having held that one form of reservoir-dam construction was appurtenant to a lock canal and another form was appurtenant to a sea-level canal; precisely the same members of the Board having voted for each form of dam construction according as they voted for a lock canal, or a sea-level canal.

But the history of the origin of this monster dam design (the dam has a base of over half a mile, to retain 85 ft. of water) is necessary, before it can be at all understood how it could have arisen; and, at the same time, before the fallacious reasoning by which it is now justified can be related. Mr. Schuyler has suggested the hydraulic-fill method for building this dam and others at Panama, but no advantage can be taken of the suggestion, so long as the above-described cross-section prevails, and the putting into it of waste earth from the canal excavation, is insisted on.

The origin of the cross-section referred to is to be found in Massachusetts, and begins with the idea, that removing the top soil a foot or more in depth from over the several square miles of a reservoir site will materially improve the quality of the water to be stored and drawn, as compared with the same water if stored in such a reservoir built as they have been hitherto built in hundreds of examples. This has been called an idea because no proof has ever been offered that such results would follow, or have followed; nor can such proof be furnished except solely by practice.

Argument may indeed be made on the other side, but this feature of the case is beside the present question. Mr. Herschel.

In one instance, at least, this so-called stripping work has been done, and has resulted naturally in the accumulation of a very large amount of waste earth—several million cubic yards of it. This all had to be put somewhere, and, as an earth dam formed part of the works, the choice was presented of making a monumental mound of it, or of disposing of it on the down-stream side of the earth dam, and as a part of it; sloping it off to a similarly enormous distance to that already described, and with a slope so flat as to resemble a lawn or golf ground more than the outside of a reservoir dam.

Thus far no harm has been done to designs for reservoir dams in other places, but when this dam of superfluous dimensions is cited as an example of design to follow; whether there be an excess of excavation to dispose of or not; whether, having an excess of excavation to dispose of, it can be disposed of in a less expensive place and manner, or not; it is proper to protest against the general construction of earth dams of these startling and excessively costly, yet useless, dimensions.

To support the same, a new theory of the action of water under pressure, when flowing through the earth, has been brought out, which constitutes the fallacy to which reference has been made. This is, that such water under pressure—in this case starting from the up-stream reservoir—when moving through or let loose in the earth, is bound to proceed in a direction parallel to the surface of the down-stream slope of the dam which has been described, or in a nearly horizontal direction, and thus be compelled to move very slowly; when in fact, and as any observer of a spouting spring, or of the escaping water of a break in a pipe has seen, water under pressure will always seek the surface by lines of least resistance. If it need turn a square corner and escape vertically, instead of continuing to flow horizontally, and thus make wholly useless the half mile of tail or fringe to the dam which has been described, it will not hesitate to do so; leaving the many millions of cubic yards of this fringe, distributed in layers, sprinkled and rolled at great expense, or else put in by hydraulic-fill, wholly wasted below it; or even proceeding to flow over and cut into the surface of such a half-mile fill.

This notion of water flowing only in horizontal straight lines is a hard one for the modern engineer to get rid of. In his youth he was taught all about the "threads" and the "filaments" in which water moves, and it will not be until these conceptions of the school books are rooted out of the books and out of past scholars, that engineers will become, on these lines, less gullible. To this day the

Mr. Herschel outlet gates of reservoirs for a domestic supply of water are placed at several elevations for the alleged purpose of enabling different "layers" of the contained water to be drawn; whereas everyone knows, or by reflection may know, that water is as likely to reach such an outlet from above, or from below, or from any direction bounded by a hemisphere within the reservoir, as from a point in the same horizontal plane as the outlet opening.

It is this same class of false reasoning which would make us believe that water from up stream, or from within a dam, is compelled by inherent predilection to follow lines of resistance that stretch out for half a mile down stream parallel to the surface of a slope piece of that width that has been added to the already sufficient ordinary down-stream slope of an earth dam, rather than turn a square corner and spout out vertically; and that will argue for the construction of such a dam.

For there is also no other valid argument in favor of it. It is a piece of landscape gardening, nothing more. It has been claimed for it, that its great weight will compress the soil for great depths, thus making secure an otherwise insecure foundation. The writer does not think so. It is weight per square foot that counts, not gross weight, and the greatest weight per square foot is under the center of the dam. Even there, the weight of the dam can affect the consistency of the sub-soil for only a very small depth; and certain it is, if such action is not effective and sufficient under the body of the dam, that adding a fringe a half a mile wide down stream from the body of the dam, will not mend the situation.

Before the author can get a hearing for the hydraulic-fill method which he has suggested for the building of the Gatun and other of the Panama reservoir earth dams, it will be necessary to resolve to dispose of the waste excavation from the canal anywhere and in any manner in which it can be disposed of most economically, rather than save it, in order that it may be placed in an earth dam with a half-mile base by means of a 30-mile haul, to hold 85 ft. of water; about six times the base, and twenty times the volume, for the same depth of water, of the already unusually large and voluminous Necaxa Dam described by the author. This done, a cross-section of these dams more nearly normal may be adopted, and estimates and a plan of campaign made, how most expediently to construct them.

Mr. Butcher. William L. Butcher, Assoc. M. Am. Soc. C. E. (by letter).—One point brought out by Mr. Herschel, although having no particular bearing on the matter of hydraulic-fill dams, seems to demand some comment. Speaking of the fallacy of supposing that filaments of water move in straight lines, he says:

"To this day the outlet gates of reservoirs for a domestic supply of water are placed at several elevations for the alleged purpose of enabling different 'layers' of the contained water to be drawn; whereas everyone knows, or by reflection may know, that water is as likely to reach such an outlet from above, or from below, or from any direction bounded by a hemisphere within the reservoir, * * *" Mr. Butcher

Inasmuch as practically every reservoir of any considerable size has the above-mentioned feature incorporated in its design, the question of its utility is a matter of some moment. While Mr. Herschel's contention would doubtless be true if the water in an open basin were of uniform density throughout its mass, there is an abundance of experimental data to prove that such is not the case.

The temperature changes with the depth and the season, thus producing a varying density which gives rise, in deep ponds, in the more northern latitudes, to the well-known phenomena of the spring and fall overturn, during which the water from the bottom is brought to the surface, and *vice versa*. Furthermore, chemical analyses show that during seasons of more or less uniform temperature, the water becomes stratified, the organic matter in the bottom "layers" rapidly exhausting the oxygen in the water and producing foul conditions which remain until a change in season and temperature causes vertical circulation.

Of course, the wind is a disturbing factor to ideal stratification, but an examination of the weekly temperatures of a number of reservoirs indicates that even where the maximum depth is not more than 20 ft., the water of least density is on top for a large proportion of the time. Therefore, there is apparently a very practical reason for designing outlet gates so that the water may be drawn from any desired level; and, under normal conditions, water would be drawn from that level and no other.

T. G. Dabney, M. Am. Soc. C. E. (by letter).—The writer has read this paper and Mr. Herschel's discussion with much interest. Mr. Dabney.

In his observations concerning the design of the great Gatun Dam as projected, Mr. Herschel, to the writer's apprehension, appears to have fallen into a fallacy in endeavoring to point out what he regards as a fallacious idea in the minds of the designers of the dam.

Mr. Herschel objects to the extension of the base of the Gatun Dam ½ mile down stream, and insists that the base width should be limited to the proportions adopted in common practice for reservoir dams, with an impervious "core-wall" extending down to bedrock beneath the dam foundation.

It is not the writer's purpose to undertake to justify the design of the Gatun Dam as a whole, not being in a position to pronounce upon it intelligently; but the particular objection made by Mr.

Mr. Dabney. Herschel appears to be based upon an erroneous conception on his part. Apparently he has neglected consideration of the particular conditions that controlled in the adoption of the plan proposed, namely, the presence of a mass of more or less permeable material—clay, sand and gravel—upon which the dam must rest, which is represented as of such great depth as to forbid the introduction of the usual "core-wall" down to bed-rock.

It is certain, then, that under a head of 85 ft. a considerable volume of water must be forced through the permeable medium underlying the dam, to control which the extension of base was designed. It is difficult to understand upon what ground Mr. Herschel assumes that this extension of base was adopted on the theory that the underground water would move only "in a direction parallel to the surface of the down-stream slope of the dam which has been described, or in a nearly horizontal direction."

On the contrary, the very means designed to control the flow, namely, placing a great mass of earth over a wide space behind the dam, is obviously intended to combat the tendency of the water to escape in a vertical direction, and in full recognition of the rather elementary argument that water must flow along lines of least resistance.

The underground water when forced under hydrostatic pressure through the medium beneath the dam, must meet with a cumulative resistance to its horizontal movement as it proceeds further from the source of pressure; and, if prevented from escaping upward by a resistant body of earth, must reach a point at such a distance from the source of pressure that resistance to its further passage will counterbalance the velocity head, where further movement must cease; which point, in theory, should mark the limit of extension of the base of the dam.

The upward pressure must be greatest in the region next behind the axis of the dam, and here the superposed earth is thickest. Proceeding away from the central axis, both the hydrostatic pressure and the thickness of the opposing earth prism are proportionately diminishing, until a common point is reached where, theoretically, both these factors become zero. Under these conditions, there can be no flow of water "parallel to the down-stream slope of the dam," nor horizontally, nor otherwise, but all movement must cease, and the body of underground water must become stationary, or have so little energy of motion as to become negligible for practical purposes.

The attainment of this condition seems to have been contemplated in the design of the dam, and whether the requirements outlined above have been fully met by the plan proposed is the only question involved, in so far as the sufficiency of the dam alone is to be considered.

Mr. Dabney.

In the writer's experience as a levee engineer on the Lower Mississippi River, it has been found expedient, as well as much cheaper, and quite effective, to cover treacherous areas behind levees with water instead of earth.

The Mississippi River levees, in many localities, are underlaid by thick sand strata, of very permeable character, which are usually overlaid by several feet of tenacious clay, locally called "buckshot." These sand beds become fully water laden during flood stages of the river, and, not infrequently at such times, it happens that, under hydrostatic pressure, formidable eruptions occur behind the levees, in the form of dangerous "boils" or fountains, with alarming displacement of sub-surface material, culminating in some cases in a "blow out," and collapse of the levee.

The usual mode of combating this danger during floods is by building temporary "cisterns" or pools, with earth-filled sacks, around the erupting crater, ponding the water over the orifice to a depth of several feet. By this device the velocity head is reduced, and, at the same time, the dead "water cushion" over the "boil" stops the displacement of solid matter below the surface of the ground.

The writer has invoked this principle quite generally, by building permanent "sub-levees," around such treacherous expanses, and in cases where the inclosed basins embrace several acres, the process of filling them with water when under flood pressure, is hastened by the use of siphons placed over the levee. In one such case a wide channel is traversed by a large levee, the Hushpuckena Bayou, which at that point is 1 000 ft. wide. The flood head of some 30 ft. of water is distributed between the main levee and two sub-levees in series, the latter located across the channel about 400 and 1 500 ft., respectively, below the main levee, narrow channel widths being selected for these locations.

By this means the alarming symptoms of foundation instability are completely controlled, and the situation is rendered secure; and in all similar cases the results have been entirely satisfactory.

The sub-levees are provided with suitable spillways, and also with sub-drains to empty the basins after the floods subside.

The suggestion presents itself that the same principle might perhaps be invoked profitably, under favoring conditions, to afford security to doubtful foundations of reservoir dams; such favoring conditions being facilities for covering the eruptive area below the dam, in either the bottom or banks of the channel, with any required depth of water and to any desirable or necessary distance below the main structure, either in a single pool, or a series with diminishing depths and surface elevations, each separate structure being proportioned to resist only the head it has to carry, with a considerable factor of safety.

Mr. Dabney. Whether or not this principle can be applied to other situations, the writer has found it indispensable for the security of his own levee lines, where instability of foundations cannot be corrected by other means.

Mr. Stearns. FREDERIC P. STEARNS, PAST-PRESIDENT, AM. SOC. C. E. (by letter).—The author has rendered a valuable service to the Society by furnishing a paper which describes so clearly and fully the recent practice in hydraulic-fill dam construction, covering many works completed and under construction, and some of them much greater in magnitude than any that have been built in the past; and in enunciating principles which should be followed in the construction of dams built by this process.

The paper is especially appropriate at this time, when, with the rapid development of power plants in the West, and the reclamation of the arid lands, more important dam construction is in progress than ever before in America.

There seems to the writer to be hardly a question that when local conditions are favorable the hydraulic-fill dam will be the most economical and efficient that can be constructed. It is, of course, not adapted to all places and classes of material, and the same may be said of any other type which may be devised. Good engineering requires that in dam building, as in other work, the problem should be studied locally, and that the type of dam adopted should be that best suited to local conditions.

The writer agrees in general with the principles laid down, but, like all general rules, apparently there should be some exceptions.

The first principle is that the dam "must be founded on an impermeable foundation, and form a water-tight connection with the rock or clay bed on which it rests," and this is a principle which can be followed in nearly all cases in the far West. It would have been impossible, however, to build the dam of the Hempstead Reservoir, on Long Island, under these conditions, as the foundation is the well-known permeable Long Island sand, which extends to a great depth, but, nevertheless, a stable dam has existed at this place for a great many years. It is feasible, by fitting the design of the dam to the location, to build a safe dam almost anywhere.

The author probably would not wish the fifth principle which he lays down to be taken too rigidly. It is that the dam "must not settle, crack, or show any sign of change or movement after final completion, and when put into service." The process of making a hydraulic-fill dam is such that the earth when first deposited is not compact, but, gradually, as the water drains out under the pressure of the superincumbent earth, it settles and solidifies.

The author speaks of this action, and in one case says that the earth packed so readily that four days after sluicing was suspended a team could be driven over the sluiced material without sinking in.

It is probable that this process of settling and solidifying goes on to a minute extent after the completion of a dam and after it is put into service, and that the settling makes the dam more solid and water-tight but does not tend to produce cracks unless there are exceptional conditions, such as, for instance, an abrupt change in the height of the embankment at any place, due to a buried vertical wall of rock. Mr. Stearns.

In another place he refers to the volume in the completed dam as "measured several months after completion, when settlement had ceased."

The writer is not as favorably impressed with the suggestion of the combination of a rock-fill and hydraulic-fill dam, in which the rock is dumped in first and the earth is prevented from passing through the voids of the rock by a wooden partition. Such partitions are not wholly permanent, and, when their usefulness has ceased, there is a danger that the fine material used for filling will gradually sink into the voids in the rock and weaken the dam. Of course, a dam may be built of such dimensions that it will have ample stability, even if a large amount of earth should find its way into or even through the rock fill.

The arrangement of the cross-section of Dam No. 2 at Necaxa, Mexico, seems to the writer much more satisfactory, because the rock rests on the finer material instead of the finer material resting on the rock.

Many of the dams referred to in the paper, including the largest, are now under construction, and it is earnestly to be hoped that, after they are completed and in use, the author will present to the Society another paper on this subject.

Referring now to the discussion of the paper: The writer finds in Mr. Herschel's discussion many principles and statements with which he cannot agree. One principle with which he would agree as a general rule, applicable especially to small dams, but from which he dissents when applied indiscriminately, is that:

"an earth dam should have, to be safe for an indefinite period of time, somewhere in its cross-section, a durable element or member of construction, capable of resisting the attacks of burrowing animals not infrequently experienced from the up-stream, as well as the down-stream, side, and from beneath the surface, for the time being of the water; one, at the same time, capable of arresting the further progress of any veins of water that may penetrate the interior of the dam, whether caused by settlement, sand strata, as mentioned by the author, drying-out cracks, burrowing animals, or any other cause."

Subsequently he gives his ideas of the essentials of earth dams:

"These are, that an earth dam is a structure composed of an unstable water-tight curtain fastened in a water-tight manner across

Mr. Stearns. the water-tight surface of a valley; extending up to a horizontal line, and made stable or supported on each side by an earth fill, properly placed and protected against wave and rain wash on the outside and top."

These are appropriate views for many places, but they would clearly be inapplicable in the case of the Hempstead Dam, already referred to, or in the case of many of the levees along the Mississippi River, where it is impracticable to provide a water-tight curtain extending to the water-tight surface of a valley, and in many other places.

Again, he says:

"None of the arguments ever brought against the use of a core-wall in earth dams have had any weight when tested by experience, or an analysis; and nothing weightier than argument unsupported by experience ever has been brought against them."

The writer does not believe that this statement is warranted. One of the arguments against a core-wall is its cost, and there are cases where a core-wall is inappropriate and unnecessary, and to include one in a dam would be poor engineering. Several instances have occurred in the writer's experience where a core-wall would have been inappropriate. Two of these instances relate to dams built while raising and improving Spot Pond for a distributing reservoir for the Metropolitan Water-Works, of Massachusetts. In all, eight dams were required to retain the water of the pond. At six of these a foundation of rock was available at moderate depth, and they were built with concrete core-walls. At the sites of the other two, the rock was so far below the surface that core-walls founded upon it would have been very costly, and, if founded upon the earth at a higher level, a core-wall of the usual width would not have been as efficient as the much wider cut-off of fine earth which was used, and was very much less expensive.

At both the South and North Dikes of the Wachusett Reservoir the local situation was such that core-walls were not appropriate. At the South Dike an estimate was made of the cost of a dam designed on conventional lines with a core-wall, but a more efficient dam, without a core-wall, was subsequently built for two-thirds of the estimated cost of the core-wall dam. To have used a core-wall in the whole length of the North Dike would have added nearly $1 000 000 to its cost, without increasing its stability or diminishing the seepage through the material under the dam.

The North Dike of the Wachusett Reservoir and the proposed Gatun Dam, with their vast dimensions, seem especially to have disturbed Mr. Herschel, as he speaks of the Gatun Dam as a "monster dam design," and says that the "excess of volume may merely embarrass construction, and, at the same time, in no wise contribute to the safety." In another place he refers to "earth dams of these startling and excessively costly, * * * dimensions." He says that to support or justify these dams "a new theory of the action of water under pressure, when flowing through the earth, has been brought out," which is fallacious. Continuing, he adds: Mr. Stearns.

"This is, that such water under pressure—in this case starting from the up-stream reservoir—when moving through or let loose in the earth, is bound to proceed in a direction parallel to the surface of the down-stream slope of the dam which has been described, or in a nearly horizontal direction."

The writer, as the designer of one of these dams and a participant in the design of the other, has never before heard of this theory or any like it, and fully endorses the view that it is fallacious, but he also regards as fallacious the substitute view, explained at considerable length in the discussion, that the water "instead of continuing to flow horizontally" may escape vertically "and thus make wholly useless the half mile of tail or fringe to the dam." The portion referred to as useless is the main down-stream slope of the Gatun Dam, which falls 1 ft. in 25, and at its upper end is 3 ft. above the level of the water in the lake.

Engineers have frequently expressed the view that, where clay or other very fine material is available, the embankment of a dam can be made impermeable, but they sometimes express a doubt as to the impermeability of the character of the material beneath an embankment, which cannot be seen, but has to be explored by borings and test-pits. As there is, in the Canal Zone, an abundance of clay and other impermeable material, which is available for this dam, and in view of its exceptional thickness, the embankment cannot fail to be water-tight, which would make it impossible for water to escape vertically or any other way through the down-stream slope of the dam, but it is also true that it would not escape in this way if the material in the embankment and foundation were uniformly permeable, because the surface of the "tail or fringe" of the dam is well above the line of the hydraulic gradient of water filtering through a dam of uniform porosity.

The laws governing the filtration through earth are in many respects the same as those governing the flow of water through pipes. If a pipe of uniform section and smoothness were to extend through the Gatun Dam, having a free outlet at the down-stream toe and the full pressure of the reservoir upon its up-stream end, there would be a hydraulic gradient from the water in the reservoir to the outlet of the pipe which would be a straight line. In the same way, if the material in and under the dam were equally permeable in all parts and had a constant area of cross-section, the hydraulic

Mr. Stearns. gradient through the earth, above which no water would rise would be the same as in the pipe. A reduction, however, in the area of either the pipe or the cross-section of the permeable material toward the lower end would result in a hydraulic gradient which would not be straight, and would be higher than the straight hydraulic gradient under the first supposition.

At the Gatun Dam the straight hydraulic gradient from the reservoir to the toe of the dam has an inclination of 1 in 28. The surface of the down-stream slope of the dam at its upper end is 14 ft. above this hydraulic gradient, and the distance between the two diminishes gradually to substantially nothing at its lower end. The reduction in area of cross-section of filtering material could not raise the hydraulic gradient in the upper half of the "tail or fringe" above the surface, even under the unfavorable conditions which have been assumed as a basis of discussion.

By making an assumption still more unfavorable to a dam of this cross-section, that its up-stream quarter be built of porous material and the down-stream three-fourths of impervious material. so that water filtering through the upper fourth would be forced to the surface of the down-stream slope, as Mr. Herschel has suggested, the down-stream "tail or fringe" would even then be far from useless, as it would, by its resistance to filtration, hold the water back to such an extent that the filtration through the upper fourth of the dam would be that due to a hydraulic gradient not more than one-fourth as great as if the tail were removed.

The writer finds that engineers who, in discussing the discharge of a pipe or channel, would always refer to the hydraulic gradient of the pipe or the slope of the water in the channel, and who would never think of using the total fall in a pipe or channel in such a discussion, will frequently refer to the filtration or seepage through or under an earth dam as being due to the total depth of water in the reservoir, without any reference to the equally necessary dimension, the distance or length through which the water must filter. This is probably due to the fact that many dams have a core-wall or other thin water stop which is assumed, theoretically, to be water-tight, although frequently found in practice to be otherwise, and even dams which have no such water stop are assumed to be impervious, although in practice they are not strictly so. That the filtration through earth is governed by the hydraulic gradient as much as the flow of water through pipes, instead of by the total head, has been shown by many experiments, and may also be deduced by reasoning. Doubling the length of a pipe doubles the amount of resistance to the flow of water through it, and it is equally true that doubling the distance through which the water must filter doubles the amount of resistance to be overcome.

Experiments on the movement of water through capillary tubes, sand, and earth, show that the quantity of water discharged is directly proportional to the hydraulic gradient, while in pipes of ordinary sizes the quantity is proportional to the square root of the hydraulic gradient. As the Gatun Dam is fully four times as thick at the base as a conventional dam of liberal dimensions, the hydraulic gradient tending to produce filtration beneath it is obviously very much less than it would be under the conventional dam. The material beneath the dam is such that it is not expected that there would be any appreciable filtration under a dam of usual dimensions, but, whatever the amount may be, the great thickness of the base of the Gatun Dam will diminish the amount. Mr. Stearns.

The writer cannot agree with Mr. Herschel that there is any part of the Gatun Dam which does not add to its safety, except on the basis that, having reached a factor of safety which is sufficient, any additions to a design beyond a reasonable factor of safety do not contribute to the safety and are useless.

The writer realizes that the Gatun Dam may be criticised because of its excessive dimensions and consequent excessive cost, and will explain to some extent the reasons which governed the design.

In the first place, this dam is to hold the water of a lake of about 110 sq. miles, and any failure of the earth dam caused by the forces of Nature or the act of man would be very disastrous, as, until rebuilt, the canal would be useless, and the rebuilding would require several years. This, in itself, was a sufficient reason for going to extremes in designing a dam with very large factors of safety.

In the second place, it was known that any plan which could be introduced would be attacked by interests inimical to the canal, and the past history of the canal, including the engineering discussions upon the subject, made it obvious that the attack upon a lock-canal would be against the dams and locks.

For these reasons it seemed desirable that every force which has ever affected or which may affect the safety of a dam be considered, and that ample provision be made, regardless of cost, against all these forces. It was also desirable that a barrier of sufficient dimensions should be created so that even a legislator or a layman, without engineering knowledge, but possessed of a reasonable amount of common sense, would recognize its stability.

Carrying the crest 50 ft. above the water line, and giving it a width of 100 ft. at the top and 374 ft. at the water level provided so great a mass of material above the water level that it was thought no men would ever attempt to destroy the dam by cutting a self-eroding channel through the earthwork. The great weight of this mound of earth above the water level would, during construction,

r. Stearns. consolidate the parts of the embankment below the water level, and, even in the event of an earthquake, this weight would tend to consolidate these parts of the dam and prevent cracks. It is obvious that the great height above the water line is sufficient to prevent water from flowing over the top of the dam, which has been a frequent cause of the failure of earth dams.

No pipes or channels of any kind are to be permitted in the earth embankments, thereby avoiding a feature which has caused more failures than any other. The thickness through the dam 5 ft. below the lake level is 514 ft., which, with other precautions, furnishes a sufficient factor of safety against burrowing animals. A few dams have failed by the sloughing of the down-stream slope of the embankment, but such failure by sloughing is impossible at the Gatun Dam, where the down-stream slope has an inclination of 4%, and rock is to be used liberally at the toe of the dam.

The filtration of water through the embankment can be readily prevented, as already indicated, because of the excellent material available at this place for building a water-tight embankment, and filtration beneath the embankment will not take place to an appreciable extent because the material resting on the rock of the uplands is impervious clay, and for about 200 ft. down from the surface in the alluvial valleys the material is nearly all a mixture of sand and clay which would not permit an appreciable amount of filtration under a dam of ordinary section, and a still smaller amount would filter under a dam where the hydraulic gradient is diminished by giving the dam a thickness of half a mile at the base.

The soil in the alluvial valleys, to the depth mentioned, is practically water-tight, and it is not expected that it will be compressed to any great extent by the great weight of the embankment to be placed upon it. There will probably be some settling of the soil in these valleys as the embankment is built over them, and, to the extent that the material compresses it, it will become more compact.

It is proposed to build the lower portion of the dam by pumping earth into it, making this portion a hydraulic-fill dam, notwithstanding Mr. Herschel's doubt that it is feasible "to pump a heavy mixture of earth and water up hill," and his statement that "at present the most that can be done is to pump up the water and let it and the earth mixed with it run down again." He seems to be unaware of the many instances in which earth pumped with water has been used, for filling lands and for other purposes, at Washington, Boston, Galveston and many other places. Within two months the writer has seen a dam under construction at Los Angeles where the earth was being pumped into it, and has observed that a pier of the Southern Pacific Railroad, at Oakland, Cal., has been filled with earth by the same method. Hopper suction dredges have been used at Liverpool for many years to lift material from the bottom of deep channels, and the work has been done rapidly and economically. In 1898 the *G. B. Crow,* having a carrying capacity of 3 000 tons, dredged and conveyed to the dumping place 4 309 350 tons of material, at a cost, not including interest on cost or depreciation, of 1.2 cents per ton. Mr. Stearns.

Mr. Herschel points out some other alleged fallacies which are not so directly connected with the theory of dam construction. In one place he refers to the outlet gates of reservoirs for a domestic supply of water:

"placed at several elevations for the alleged purpose of enabling different 'layers' of the contained water to be drawn; whereas, everyone knows, or by reflection may know, that water is as likely to reach an outlet from above, or from below, or from any direction bounded by a hemisphere within the reservoir, as from a point in the same horizontal plane as the outlet opening."

In this criticism he has failed to recognize the fact that in a deep reservoir the water is stratified in summer, the water near the bottom being colder and having greater density than that above, and that both theory and experience show that the water can be drawn from near the bottom without taking any of the warmer water near the surface.

He criticises the view, which he says has its origin in Massachusetts, that removing the top soil a foot or more in depth "will materially improve the quality of the water to be stored and drawn, as compared with the same water if stored in such a reservoir built as they have been hitherto built in hundred of examples." It would be out of place to extend this already long discussion to furnish the proof, which is available, that the view which he criticises is correct. The writer, in the past twenty years, has had an extended experience with reservoirs from which the surface soil has been stripped and with those not stripped. Most of this time regular microscopical and chemical examinations of the waters of these reservoirs have been made, many of them under his direction. As a result of this experience, he knows that the water stored in the stripped reservoirs is much better in quality than that stored in similar unstripped reservoirs.

James D. Schuyler, M. Am. Soc. C. E. (by letter).—In the absence of any direct contributions of actual experience in lines of construction similar to those described in the paper, which it was hoped would be elicited, the writer feels indebted to Mr. Herschel for his good-natured criticisms of the methods and prin- Mr. Schuyler.

Mr. Schuyler. ciples involved, although he has evidently labored under a misapprehension as to the facts in the following statement:

"The author of the paper places the reason for using the hydraulic-fill method in a saving of cost by that method for the operations just described, and no other."

While economy in first cost is certainly a most potent factor in governing the choice of methods of doing any work, it is by no means the only reason which has led the writer to advocate a widespread and general use of the hydraulic-fill method of dam construction. He regards its superiority as consisting largely in the ability afforded to utilize materials which would be otherwise unfit or unsuitable, due to the assorting, grading and separation of different classes of material, by reason of the dissolving action of moving water and its varying velocities, which are entirely controllable, and which permit the deposition of the several grades of materials where they will be most useful and most serviceable in making a stable dam. By this means, the coarse, friction-bearing, stable materials may be placed on the exterior slopes, and the finer particles may be assembled in the center of the mass to serve as a puddle core, and the ease and simplicity with which this may be done constitutes one of the strongest possible reasons, aside from that of economy in cost, for using this method. In fact, by this process it becomes practicable, with care and skill, to build a safe and stable dam of materials which would otherwise be considered valueless, as Nature does not always mingle her materials in the proper proportions to make them fit for forming a water-tight embankment without proper segregation. This selection and separation cannot usually be made by the ordinary processes at practicable cost.

Again, Mr. Herschel says of the process:

"It necessitates flatter slopes, that is, a greater volume in the earth dam, than dams built by the ordinary method."

The writer must challenge this statement as entirely unwarranted by his experience or by any of the sections submitted with the paper on dams built or building by this process. Where the materials may be handled at considerably less cost, the natural tendency of the engineer is toward liberal factors of safety in dimensions, but, in all his experience, the writer has not been led to any special excess in slopes or volume by reason of the exigencies of the method. Mr. Herschel's statement, therefore, that flatter slopes are necessitated by this method, must have been evolved from an experience which he has not cited, or have been the product of his imagination. A comparison of the sections of the dams described in the paper with those of well-known earth dams built by

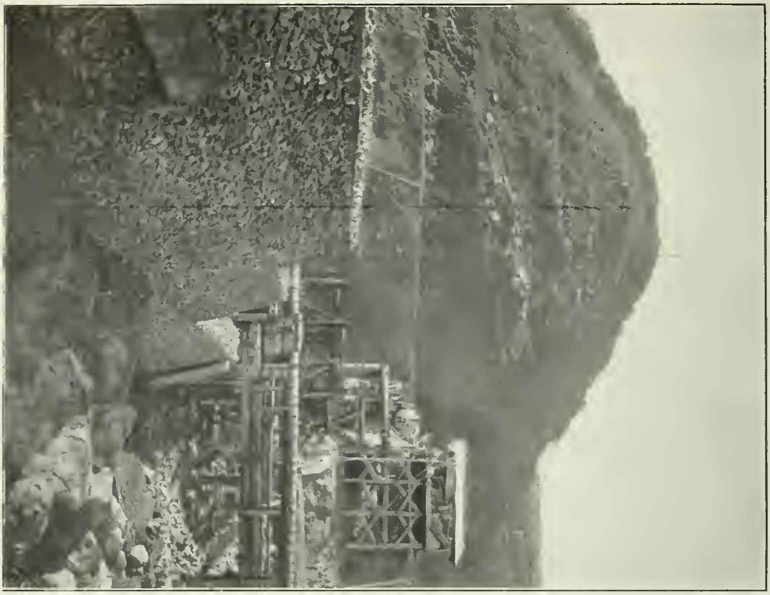


FIG. 1.—CORE-WALL OF DAM NO. 2, NECAXA, SHOWING ROUGHENED SURFACE FOR BONDING WITH HYDRAULIC FILLING.

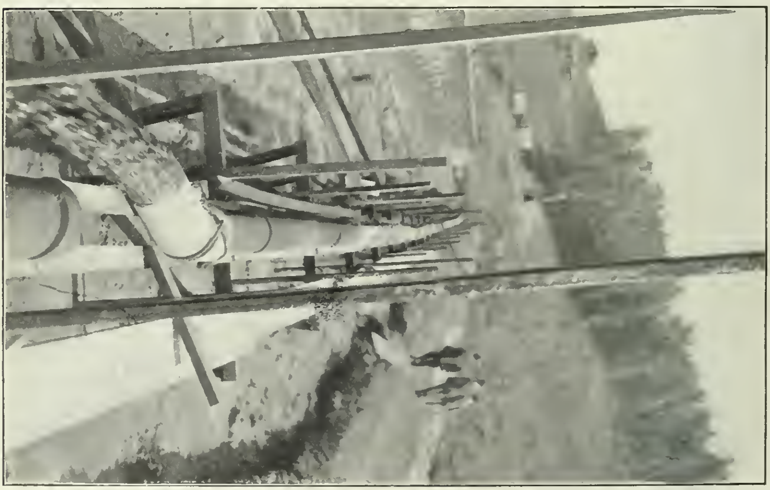


FIG. 2.—PIPE LINE DELIVERING EARTH FOR SILVER LAKE DAM.

Mr. Schuyler.

the ordinary methods hitherto in vogue will certainly fail to support Mr. Herschel's allegation of greater volume in hydraulic-fill dams.

Mr. Herschel further states that:

"It seems to require, for one reason or another, a longer time than would otherwise be necessary."

This impression seems to have been derived from a reading of the accounts of the delays and special difficulties accompanying the construction of many of the dams described; but to conclude that the process as a system is necessarily a longer one than the building of earth dams by the old methods is certainly erroneous. Wherever the conditions are favorable, and the materials and water supply abundant, and the dam does not contain an excessive amount of clay, which settles and drains more slowly than any other material, there is no reason why dams should not be built quite as rapidly by this method as by teams and scrapers, and oftentimes a much greater rate of progress may be achieved.

Among other "defects and limitations" which Mr. Herschel has discovered in this method, he mentions a most surprising fallacy:

"It needs," he says, "that it may be adopted, a body of earth situated at a higher elevation than the body of the dam to be built, and not too far away. Possibly some day we shall be able to pump a heavy mixture of earth and water up hill; at present, the most that can be done is to pump up the water, and let it and the earth mixed with it run down again."

It is almost incredible that an engineer of Mr. Herschel's extensive practice and experience should be unfamiliar with the achievements of modern dredging, where earth, gravel and cobblestones are taken from considerable depths by suction pumps and delivered "up hill" through pipes from 1 000 to 5 000 ft. or more in length. If he will visit Los Angeles at the present time, he will be able to see an earth dam, 50 ft. high and several hundred feet long, being built in large part by the hydraulic-sluicing method, the material being loosened by the hydraulic giant, with water under pressure, and delivered by gravity to a pump, from which it is forced "up hill," under a pressure of 32 lb. per sq. in., to the dam.

The writer has recently examined the site and reported on a dam to be built in one of the Western States, to contain several million cubic yards, and to be more than 140 ft. high. This dam is to be built by the hydraulic-sluicing process, from materials entirely within the reservoir area and not at a higher elevation than the dam.

The writer is unwilling to accept as essential or in all cases suitable or desirable in earth dams, the principle which Mr. Herschel

Mr. Schuyler. desires to add to those laid down at the opening of the paper, namely, that

"An earth dam should have, to be safe for an indefinite period of time, somewhere in its cross-section, a durable element or member of construction, capable of resisting the attacks of burrowing animals not infrequently experienced from the up-stream, as well as the down-stream, side, and from beneath the surface, for the time being, of the water; one, at the same time, capable of arresting the further progress of any veins of water that may penetrate the interior of the dam," etc.

The fixed objection which the writer has to a core-wall of masonry or concrete in the center of an earth dam, especially of large dimensions, is that it introduces an element entirely foreign to the nature of the body of the earth structure, acting as a huge knife to sever it in half, destroying its homogeneity, and preventing the complete knitting of the mass as a whole in a manner that must best subserve the purpose for which it was built. A core-wall must necessarily be built comparatively thin to be reasonable in cost, and must be subject to the minor defects of construction incident to all such work, and subject also to rupture from the possible uneven settlement of the earth on either side of it.

Where a core-wall is built in a dam, the engineer in charge must naturally have a tendency to place rather too implicit reliance upon it for water-tightness, to the neglect of the bond between the earth and the wall. Settlement of the earth and the slipping of the mass downward along the wall, may either rupture the core or leave minute channels along which water may creep to find exit at some crack or fracture in the core-wall. If a core-wall acts to any useful purpose, except as a stop for burrowing animals, it can only do so by assuming that the upper half of the dam is pervious to water, and will permit the percolation of water to the line of the wall. If the lower half of the earth fill should be made in a similar manner, and be equally pervious, the stability of the dam, according to the theory of the core-wall advocates, must then depend wholly upon the integrity and perfection of the core-wall—a slender reliance, surely, subject to all manner of vicissitudes. In the opinion of the writer, a core-wall of moderate height in an earth dam founded on bed-rock, is a most useful and necessary element to form a bond between the earth embankment and the bed-rock on which it rests, but for this purpose it does not need to be carried higher above the rock surface than is reasonably necessary to form such a bond. After effecting this union, the interior mass of the embankment should be built as nearly homogeneous as possible. Any division by an intermediate wall must tend to an undesirable separation of the two halves, tending to produce inequality of settlement, rupture of the core-wall, and a more dangerous condition than would otherwise exist.

FIG. 1.—DAM NO. 1, TENANGO RIVER. THE MATERIALS FOR THE DAM WERE GROUND-SLUICED FROM THE BORROW-PITS IN THE BACKGROUND.

FIG. 2.—SILVER LAKE DAM. LOS ANGELES. THE EARTH, LOOSENED BY THE HYDRAULIC JET, IS CONVEYED TO THE PUMPING STATION SHOWN, AND PUMPED TO THE DAM.

Mr. Schuyler.

By the hydraulic-fill method of construction, where the materials consist in part of stone and boulders, or coarse gravel, the deposit of these upon the outer slopes would ordinarily suffice to resist the ravages of burrowing animals, which only burrow in search of roots which should not be permitted to grow in an earth dam, while the fine materials constituting the body of the dam would act to arrest the progress of percolating water.

Mr. Herschel says:

"If water once commenced to penetrate into an earth dam, or to percolate, the channels of percolation could readily enlarge," etc.

The writer's observation of the velocity of percolating water passing through fine sand and clay, such as would ordinarily be selected for the building of a dam, is that it is so exceedingly slow as to be incapable of enlarging its channels, which can only be accomplished by the removal of particles by the current. If perfect drainage is given to such percolating water by means of gathering wells in the outer slopes of earth fills to prevent the super-saturation of the exterior, or by the placing of a blanket of rock or gravel on the slopes, such as results from the hydraulic-fill process where the material contains a suitable proportion of rock or gravel, it is impossible that an earth dam can fail as the result of such percolation. The writer wishes to make the point clear that, in his conception, no earth dam composed of loam, clay or fine sand, or any materials really suitable for a dam, can possibly fail by the percolation of water through it, provided these threads of percolating waters are intercepted at a sufficient distance back from the exterior slope by free and open channels of drainage to prevent the super-saturation of the outer third. If this principle is recognized as true—and it is so easily demonstrated that it should not be seriously questioned—the combination type of dam, consisting of a loose rock embankment for the down-stream half, and an earth embankment for the up-stream member, sluiced in place with water, and thus absolutely consolidated, would seem to offer ideal conditions for stability. The objection urged by Mr. Stearns to this type is apparently based upon the possible results of the decay of the wooden partition placed in the center of the rock-fill as a temporary stop for the soft earth. He fears the danger that the fine material used for filling will gradually sink into the voids in the rock and weaken the dam. This objection may be overcome by omitting the plank partition, as is being done in the Zuni Dam, or by surrounding it on either side with such a thick layer of finely-broken stone or gravel as to form a perfect filter and prevent the future passage of fine earth through it, in case the plank should finally decay.

The writer has had an experience in the treatment of an earth

Mr. Schuyler. dam, 40 ft. high, with a full reservoir behind it, from which it was in imminent danger of failure by percolation and the saturation and sloughing of the exterior slopes. The method used was to sink a well in the outer slope, at a considerable distance back from the toe, but less than half-way up the slope, from which a drain pipe was led out to the outer toe of the dam. The well was about 6 ft. square, curbed in a permanent manner with timber, and carried to a depth of some 3 ft. below the water plane level which was established by it. All percolation through the dam, which had previously appeared over an area some 200 ft. long and from 10 to 15 ft. high, where sloughing was rapidly going on and water was issuing in small streams, was immediately concentrated in this well, and passed off through the drainage pipes, leaving the face of the dam dry and stable. The principle is often used to concentrate the seepage water of irrigation, which sometimes appears over large areas of lower lands, rendering them unfit for cultivation until tile-drained. Open trenches and lines of tile in such cases often prove inefficient until a well of quiet water is established, from which the overflow can be led away in a tile or pipe. The water rises like a spring through the soil in the bottom of this well without carrying any particles with it. This action is apparently quite similar to that referred to by Major Dabney in his interesting discussion describing the leakage through the Mississippi levees, and the methods used for checking the leaks by forcing the little streams or "boils" to rise through wells, pools or artificial cisterns of water.

In view of the interest taken in this paper, as manifested by private correspondence and inquiry, the writer may be pardoned if he takes up a little additional space with further notes on the subject. The Terrace Dam (Colorado), described on page 237, *et seq.*, has been changed in plan since the paper was written, and is to be increased in height from 180 ft. to 225 ft. above the lower toe, or 205 ft. above the upper toe. The crest will be 25 ft. wide, at a height of 15 ft. above the spillway level. The length on top will be 605 ft. These dimensions will exceed those of any earth dam ever built, and will be considerably beyond the highest hydraulic-fill dam yet projected. About 100 000 cu. yd. of material were sluiced in place from August 1st to November 1st, 1906, when work was suspended on account of severe freezing weather. The depth of filling at the down-stream slope was 55 ft., at the up-stream toe, 43 ft., and in the center, 18 ft. The cost of the sluice ditch and flume, and all the piping and plant for sluicing, was $34 500. Water is delivered to a penstock at a height of 325 ft. above the crest of the dam. There are two pressure pipes for delivering water to the hydraulic giants, each 15 in. in diameter, and of riveted steel, No.

FIG. 1.--SILVER LAKE DAM, LOS ANGELES, FROM THE LOWER SIDE, SHOWING THE PUMPING STATION WHENCE THE LIQUID EARTH IS PUMPED TO THE DAM.

FIG. 2.—SILVER LAKE DAM, WITH LEVEES BUILT UP DRY WITH SCRAPER TEAMS.

16 gauge. The best average work for 30 days was 46 000 cu. yd., delivered at a cost of $6\frac{1}{2}$ cents per cu. yd. for labor. Estimating the volume of the dam at 900 000 cu. yd., the cost of the plant for the entire work may not exceed 4 cents per cu. yd. Mr. Schuyler.

The concrete core-wall described in the paper was not founded on bed-rock, and was discovered to be so poorly built as to be practically valueless. Several deep trenches were made to bed-rock, through boulders on either side of the wall, and filled with choice puddle clay, sluiced in place, and the wall was perforated with large openings to allow the soft material to pass freely through it from side to side.

The greatest volume of water used in sluicing was 25 cu. ft. per sec. No determination has ever been made as to the percentage of solids delivered by the water. The sluice boxes from the borrow-pits are laid on grade of 7 per cent. Wood boxes have been used during the past season, but will be replaced by steel sluices or pipes in the future. It is expected to complete the dam during the season of 1907. Mr. E. W. Case was superseded as Engineer in charge prior to the work of 1906, and the engineering is now under the supervision of T. W. Jaycox, M. Am. Soc. C. E., State Engineer, who is practically directing the work.

Work on the Zuñi Dam, New Mexico, the combination rock-fill and hydraulic-fill, described on page 235, has been suspended for the winter, but will be completed in 1907. The rock-fill was finished as a carefully hand-laid wall, at a cost of about $2.00 per cu. yd. Hydraulic sluicing with a pump was started in October, and satisfactory progress was made until heavy rains rendered the roads too soft for the delivery of fuel. This dam will be finished in 1907.

Work on the hydraulic-fill dam No. 2, at Necaxa, Mexico, described on page 240 *et seq.*, has reached a stage where active sluicing will begin about March 1st, 1907, and material is expected to be delivered at a rate of not less than 10 000 cu. yd. per day. The center core-wall trench was excavated to a final depth of 40 ft. below the stripped surface, across the bottom of the valley, cutting through various layers of hard rock and soft porous material, and finally ending in a bed of solid yellow clay. This trench has been refilled with concrete, extending about 2 m. high above the stripped surface, as shown by Fig. 1, Plate LV. Two trenches above the center trench, and parallel with it, have been excavated to the clay floor of the valley, and will be refilled with puddle, sluiced in place. The two lower trenches, down-stream from the center, have not been carried as deep as the others. The stripping for this dam has required the removal of more than 260 000 cu. yd. of loose material from the middle third, of which possibly 20% has been deposited at the upper and lower toes.

Mr. Schuyler. Dam No. 1, referred to on page 244, was completed in the summer of 1906, and six months subsequently its condition was examined by means of a vertical wood-stave pipe, 30 in. in diameter, which had been left in the center for inspection purposes. Water was found standing in the pipe within 9 ft. of the crest. When this had been baled out, holes were bored through the pipe at different levels and samples of the material were taken with an auger. It was found to be plastic and evidently impervious to water. The dam had settled next to the wall of the spillway less than 3 in., where its height was nearly 40 ft. It showed no sign of settlement elsewhere, and was manifestly water-tight. The accompanying photograph of the completed dam, Fig. 1, Plate LVI, is taken from near the same point as Fig. 2, Plate L. The inspection pipe referred to is shown near the center.

The dam referred to by Mr. Stearns in his discussion, as under construction in the City of Los Angeles, is to form a distributing reservoir with a capacity of 156 000 000 gal. The dam has a concrete core-wall extending to bed-rock at a depth of about 40 ft. below the natural surface, and reaching above the surface from 3 to 6 ft. only. The wall in the deepest part was built between parallel lines of steel sheet-piling, driven about 6 ft. apart.

The dam is to be about 54 ft. high and 900 ft. long on the crest, and will contain 140 000 cu. yd., more than half of which will have been put in by the hydraulic sluicing process. At the outset, the material was loosened and dissolved by "ground sluicing," the water used being taken from a grade line pipe passing nearby at a low level. The water with its load of earth flowed by gravity to a centrifugal pump, by which it was forced through a pipe some 1 500 ft., and deposited on the dam by a flume running longitudinally along the line of the core-wall, and small branch pipes with loose, open joints, leading from the flume at right angles toward the outer slope of the dam. (The plan has since been changed, and pipes substituted for flumes. The branch pipes have also been eliminated.) When the material below the grade line pipe was exhausted, a second pump was installed, and water was forced through a nozzle, or hydraulic giant, under a head of 150 ft., by which the earth at a higher level was loosened and transported to the pump which forces it to the dam. The combined friction head and lift amount to 32 lb. per sq. in., with the dam 24 ft. high. From this height on, to the completion of the dam, all the material will be handled with water, except the low slope levees on either side, which are built up with scraper teams.

The volume of water used in this work is only 0.9 cu. ft. per sec., with which the delivery to the dam averages 300 cu. yd. per day of 9 hours. The proportion of solids carried, therefore, averages nearly 28 per cent. The average cost of the work, in-

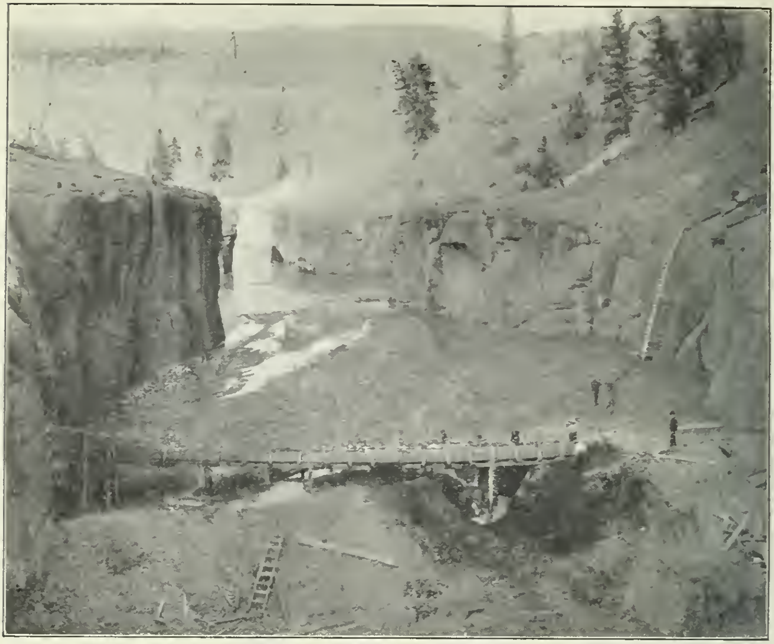


FIG. 1.—TERRACE DAM. COLORADO. LOOKING UP-STREAM. ILLUSTRATING GRADATION OF MATERIAL TOWARD THE CENTER OF THE DAM FROM THE POINT OF DEPOSIT AT THE SLOPES.

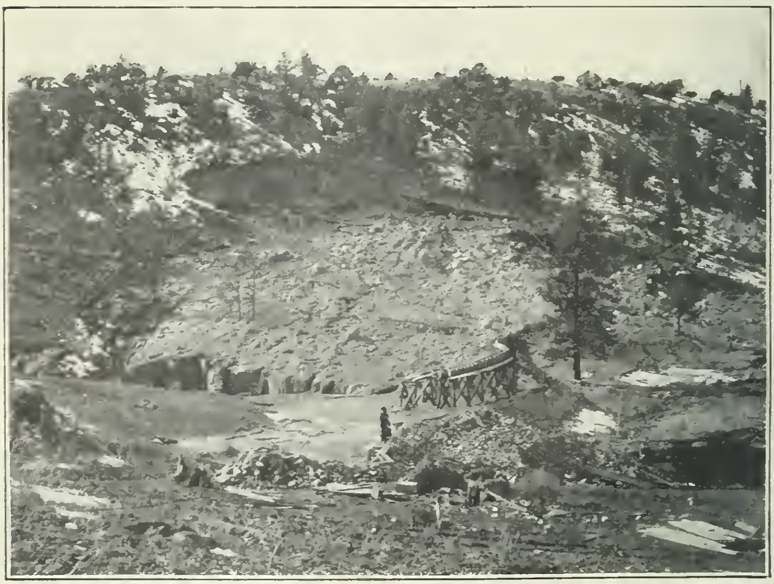


FIG. 2.—TERRACE DAM. DEPOSIT OF HYDRAULIC FILLING ON THE UPPER TOE SLOPE.

Mr. Schuyler.

cluding depreciation of plant, fuel and labor, is 11 cents per cu. yd. The plant is a temporary makeshift, made up from materials on hand, and could be duplicated for $3 300. Mr. William Mulholland, Superintendent of the Water-works, states that the entire dam would have been built by the sluicing process in preference to the use of teams for any portion of it, but for lack of time, as it was essential to get the reservoir in service early in the spring of 1907.

Since the paper was written, the writer has designed a combination rock-fill and hydraulic-fill dam in Mexico, which will have a maximum height of 235 ft. and impound nearly 2 000 000 acre-feet of water in a reservoir covering more than 22 000 acres, and feels confident that this type of construction is amply secure for a work of such bold dimensions. There is a strong probability of this work being carried to an early completion for power and irrigation service. It is unnecessary to call attention to the fact that the reservoir will be practically double the capacity of the Tonto Basin Reservoir in Arizona, to be formed by the Roosevelt Dam, which is one-third larger than the Assouan Reservoir on the River Nile, now the most capacious artificial basin in the world.

A dam is under construction in the mountains north of San Bernardino, Cal., on the headwaters of the Mohave River, by the Arrowhead Reservoir Company, which will have an extreme height of 200 ft. when finished, and has already reached a height of more than 100 ft. The material consists of disintegrated granite soil, which is loosened and loaded on cars by steam shovels, delivered to the outer slopes by railway tracks, and thence washed into the body of the dam, on either side of a masonry core-wall, by water pumped through pipes, laid along the slopes, inside the tracks. The core-wall is of masonry laid in Portland cement, is deeply founded on solid granite, and is carried up all the way to the top, starting with a thickness of 20 ft. and tapering to 3 ft. at the water line. The length of the dam on top will be approximately 800 ft. This interesting work, which is being done by contract, is in charge of Mr. E. H. Kellogg, Chief Engineer. This was a case where a core-wall was quite appropriate, for the reason that the percentage of fine material in the soil available for building the dam is comparatively small. Some tests made by the writer several years ago indicated that there was less than 10% of clay in much of the soil. The work is being done practically in accordance with plans recommended by J. M. Howells, M. Am. Soc. C. E., and the writer in 1901. The dam cannot be finished before the season of 1908, as work has to be suspended each winter on account of snow and cold weather. This work is worthy of a visit by those who are investigating the subject of hydraulic-fills, with or without core-walls.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852

TRANSACTIONS.

Paper No. 1044.

LIGHTHOUSE CONSTRUCTION IN THE PHILIPPINES.*

By Spencer Cosby, M. Am. Soc. C. E.†

With Discussion by Messrs. Charles E. L. B. Davis and Spencer Cosby.

In a country like the Philippines, made up of many hundreds of islands separated from one another by deep and wide arms of the sea, and provided by Nature with numerous good harbors, the water routes are, and must continue to be, the main highways of trade and travel. To them as trunk lines the roads and railroads serve as necessary feeders, and are generally built with a view to providing the shortest and cheapest means of reaching the nearest good harbor. This access to a port is the commercial question of first importance in a country whose home market for its chief products is small, and whose prosperity and very existence as a civilized land depend upon its trade with nations overseas.

The importance of the navigation interests and the need of fostering them by improving the water routes and their terminals were early recognized by the American government of the Philippines. One of its first formal acts was to provide for the prompt comple-

* Presented at the meeting of February 6th, 1907.

† Captain, Corps of Engineers, U. S. A.

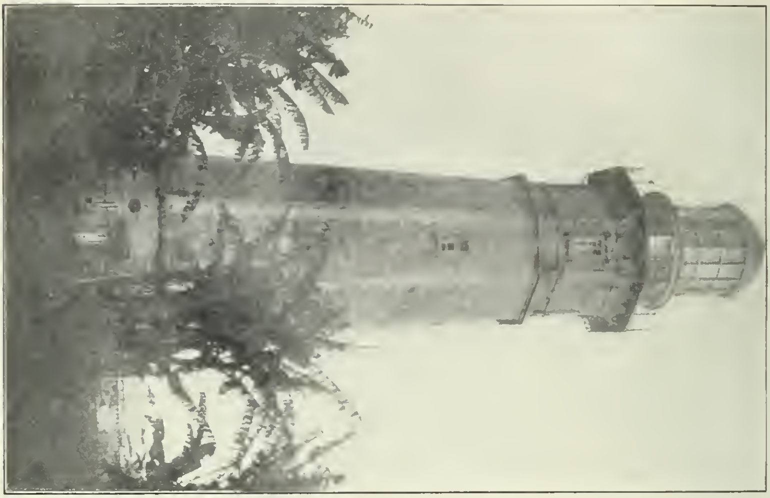


Fig. 1.—Cape Melville Tower.

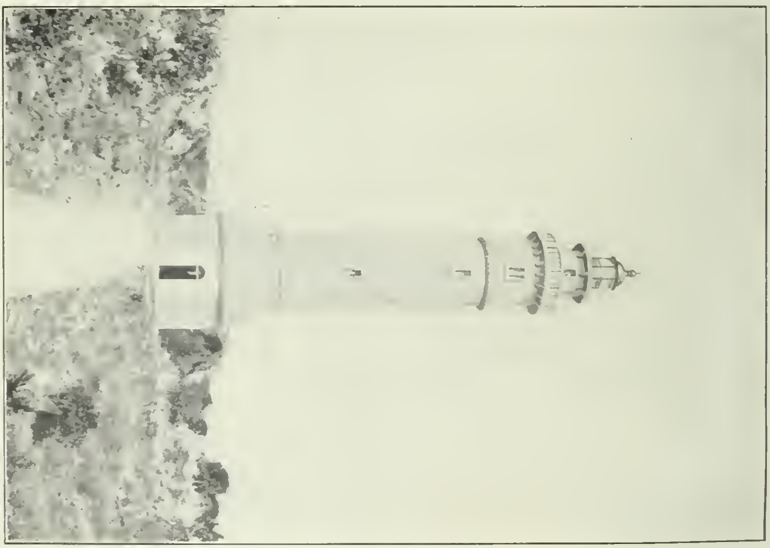


Fig. 2.—Maniguin Island Tower.

tion, at a cost of several million dollars, of the protected deep-water harbor of Manila, begun many years before by the Spaniards. This was followed later by the creation of a Bureau which, as one of its chief functions, had charge of the lighthouse service of the Islands. The new officials proceeded at once to organize the lights already in existence, on the model of our home establishment, and to draw up plans for putting up new lights where most urgently needed.

When the American engineers first took up this matter of lighthouse construction, they were confronted with conditions differing greatly from those met in similar work in the United States. In some respects the problem appeared to be easier of solution. There are practically no fogs along the coasts of the Philippines, and both the transparency of the atmosphere and its refractive power are greater, according to the observations of the Spanish engineers, than along the Atlantic coast in temperate latitudes. These conditions make the lights themselves more effective, and render unnecessary the fog signals which are the most unsatisfactory parts of our modern lighthouse system. Then, too, a working season extending over the whole year, the absence of frost, and the cheapness of labor were factors which promised to be of material advantage in carrying on construction work. To offset these, however, were others, of the opposite kind—typhoons of frequent occurrence and of cyclonic violence along the paths of their centers; earthquakes of a mild character taking place constantly, but in certain districts reaching at times the most destructive intensity; the presence everywhere of the termite or white ant (the great destroyer of woodwork); the tropical humidity adding to the difficulty of protecting iron and steel properly; and the increased cost and long delays in obtaining many of the tools and materials of construction.

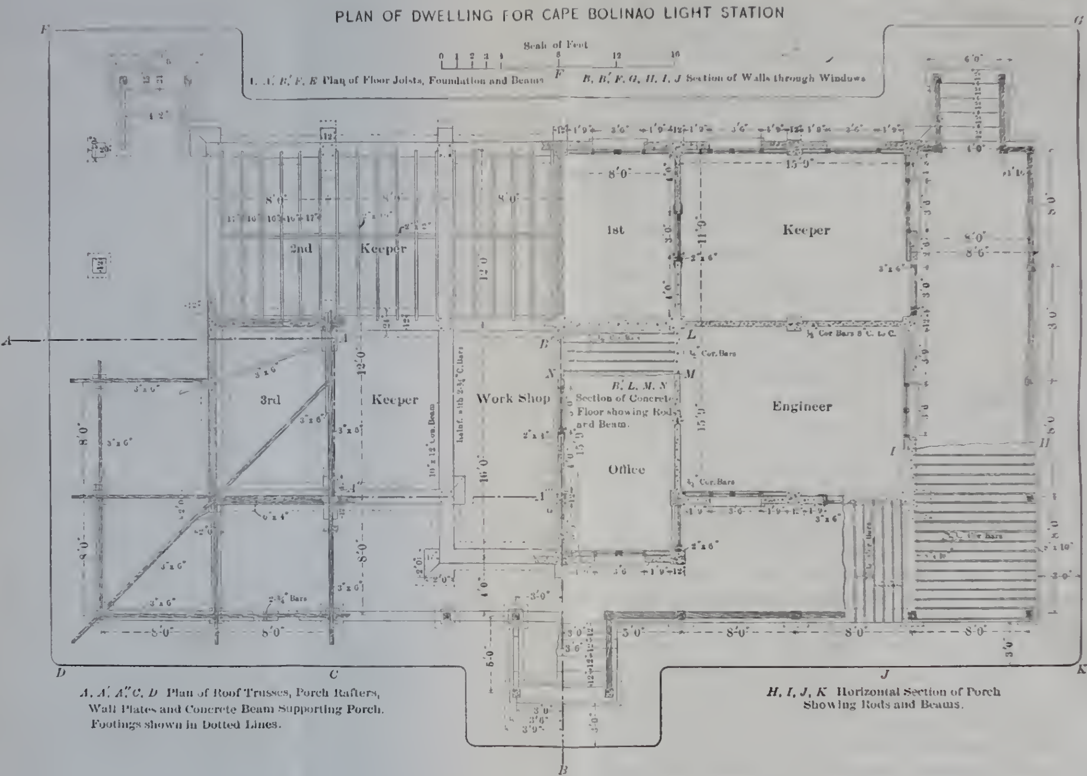
Labor was found to be cheap in about the same degree that it was efficient. A native carpenter, mason or blacksmith, at a dollar a day, will not do more than from one-half to one-quarter as much work as an American skilled workman, and will require more constant and careful supervision. Unskilled laborers, at from 20 to 50 cents per day, perform just about that much worth of work, according to American standards. One peculiar difficulty was met in securing workmen to go to some of the isolated lighthouse stations. The Filipino is strongly attached to his family, and many of the most desirable men refused to leave home for even a few months unless they could take their entire families with them. The difficulty of transporting and housing at the site such a mob of men, women, and children dwindled, however, when it was found that each family usually carried in a small sack its entire belongings, including the cooked rice to be eaten on the voyage, that they expected nothing more in the way of accommodations than sufficient room to lie down anywhere above or below deck, and that a few sticks of bamboo and bundles of "nipa" (palm leaves used for thatching walls and roofs) provided them with all the materials needed for building a home.

It should be mentioned that the labor conditions improved materially as time went on, as the lazy and incompetent were weeded out, and as those retained became used to steady work of the kind required. With fair treatment and proper training, it is believed that the Filipino can be developed into a more efficient and more reliable laborer than the negro of the South.

At the time the first insurrection broke out in the Philippines, in 1896, the Spanish Government was actively engaged upon an extensive programme of lighthouse construction, based upon carefully matured plans, which in the course of time would have provided an excellent system of lights. The disordered state of the country and the need of using all available funds for more urgent purposes caused the practical suspension of work in the latter part of 1896. It was not actively resumed by the Americans until six years later, and the first year was spent almost entirely in completing stations left in an unfinished condition by the Spaniards, and in repairing those wrecked or damaged by the insurgents.

The Spanish plans and records were turned over to the United States, so that when it came to new work the natural course was to carry out the projects they had so carefully worked out and which had back of them the benefit of the makers' long experience. The structures already in existence, at the stations which had not been disturbed, were nearly all of massive masonry—towers, keepers' dwellings, and even the oil houses and detached kitchens. All the buildings were of one story. They had resisted well the earthquakes and other destructive forces at work, except the white ant, the ravages of which had practically destroyed many of the floors, and

PLAN OF DWELLING FOR CAPE BOLINAO LIGHT STATION



A, A', A'', C, D Plan of Roof Trusses, Porch Rafters, Wall Plates and Concrete Beam Supporting Porch. Footings shown in Dotted Lines.

H, I, J, K Horizontal Section of Porch Showing Rods and Beams.

rendered insecure some of the wooden ceilings and roof trusses. A view of one of the best of the Spanish towers is shown by Fig. 1, Plate LIX, and an elevation and sections of a similar tower are shown by Fig. 1.

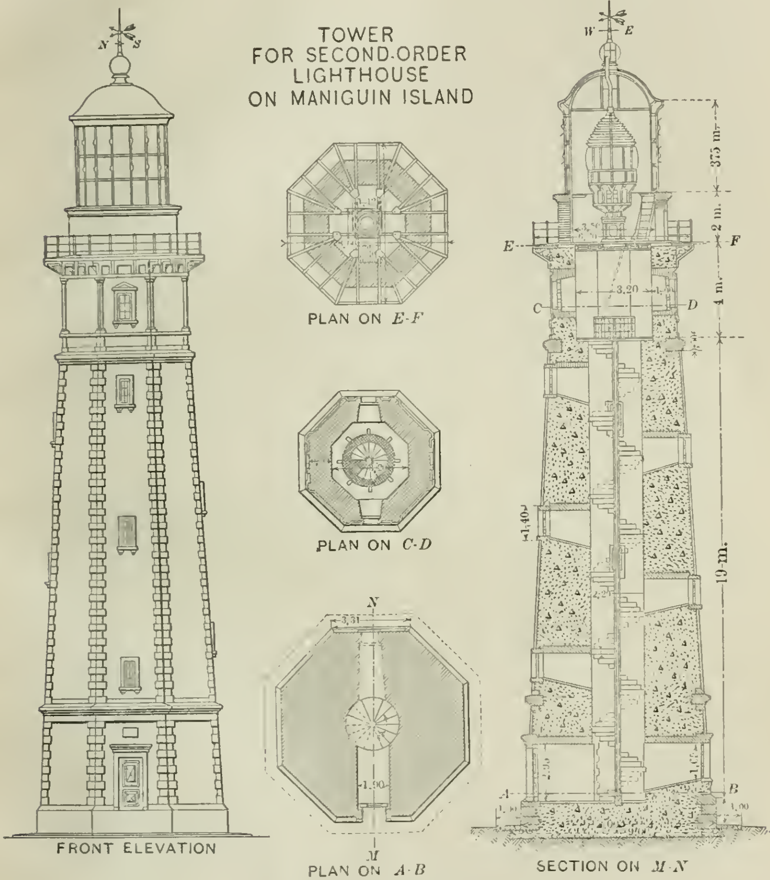


FIG. 1.

The plans of the first new dwelling to be erected called for walls 3 ft. thick and 15½ ft. high from foundation to roof truss, and this was typical of all. Such a structure naturally had nothing to fear from earthquake or typhoon, but the immunity was secured only by the expenditure of a large amount of time and money, and economy in both was essential.

The Spaniards proceeded with their work in a more deliberate fashion than the writer felt justified in adopting. An interval of 4½ years elapsed between the time when work was started on the first-order light station shown by Fig. 1, Plate LIX, and the date when the light was first displayed. Even the smaller lights were usually from 18 to 30 months or more in building—and this after a year or two had been spent in making very elaborate plans and estimates, in having them passed on by various officials and boards, and in calling for bids.

The Spanish lights are excellent structures, well fulfilling their purpose, but it had taken more than 10 years to build the greater number of them, and many a dark gap remained along the principal routes of commerce. It was decided that it was necessary to adopt methods which would provide these gaps with efficient lights as quickly as possible from the limited insular funds available for the purpose. One result of the system adopted may be mentioned here —the number of effective lights was increased by 60% in the two years, 1904 and 1905.

Attention was first directed to reducing the cost of the keepers' dwelling and the minor buildings at each station. The height of ceiling was lowered a few feet, the thickness of walls was decreased, the quality of mortar was improved by doing away with the use of lime; and concrete was used extensively in place of stone masonry. Two dwellings were built of timber, but the result was not satisfactory. The saving in first cost was not sufficient to offset the increased expense of maintenance and repairs, or the risk of destruction by fire, or of serious damage by the white ant, though special precautions were taken to reduce this last risk to a minimum by allowing no wood whatever to come in contact with the ground, by having an open, easily accessible and well-lighted space between the floor and the ground, and by making the lower parts of the posts, which were sunk in concrete piers, of the native hardwood "Molave." This wood has remarkable properties which make it an invaluable building material. It is very tough and durable, seems to be impervious to decay, even when placed on or in damp ground, and resists the attacks of both the white ant and the teredo. Its only

PLATE LXI.
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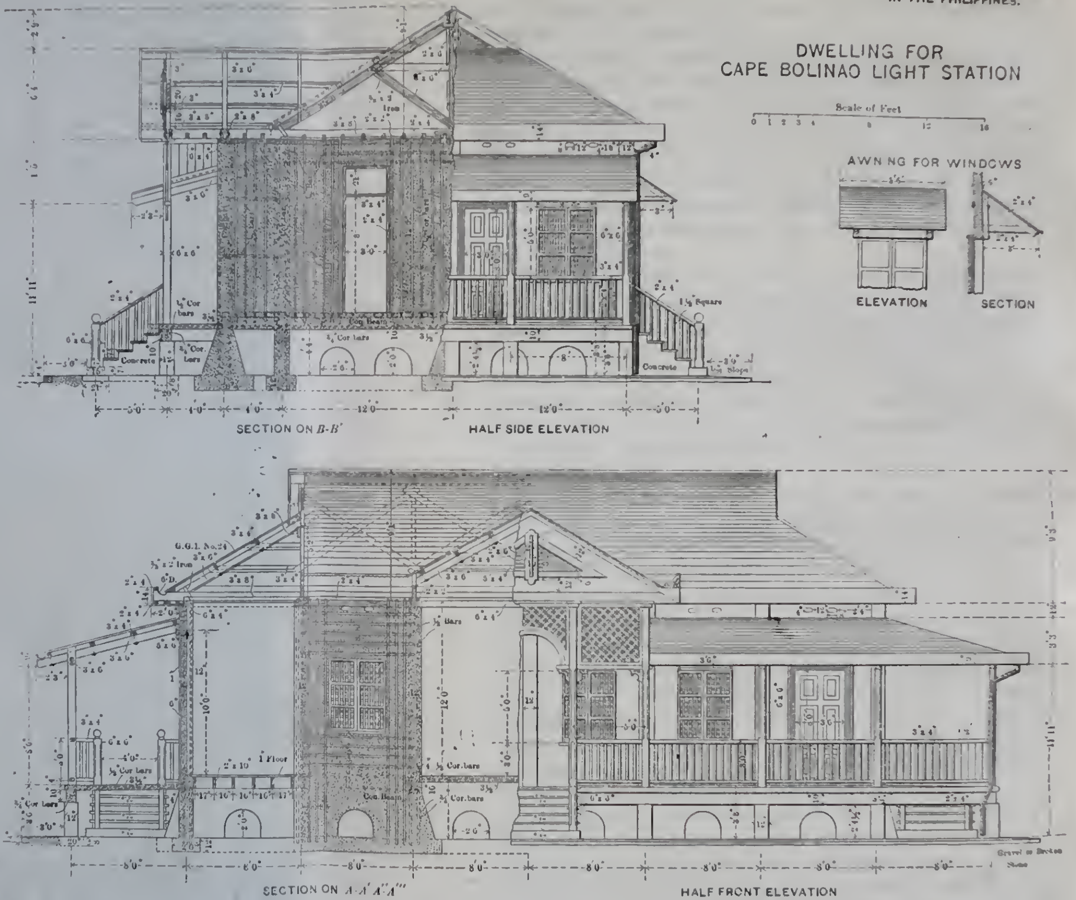

drawback is the high price it commands, in spite of the fact that it is fairly plentiful throughout the Islands.

The next step was to reduce still further the thickness of the walls, making them of concrete and strengthening them with steel rods, in other words, adopting reinforced concrete construction. This was followed by the present type of dwelling, shown on Plates LX and LXI, which consists essentially of columns of reinforced concrete, spaced 8 ft. apart, supporting the roof trusses, and joined by curtain walls of sufficient thickness to keep out rain and heat. It is believed that this form of construction will prove fully as strong and enduring as the more massive and costly Spanish style. It is designed to resist both typhoons and earthquakes. There is comparatively little woodwork, and that is placed so as to render it difficult for a fire to make headway rapidly or for the white ant to gain access to it. Repairs should be needed only at long intervals, and the cost of upkeep should be slight. Three keepers and their families are to occupy the dwelling; the kitchens, store room, oil room, etc., are in small detached buildings near it.

The following is abstracted from the instructions given to the workmen:

The drawings must be followed closely as to dimensions, materials and methods of construction. Should a departure from them be at any time deemed advisable, the overseer will ask the Lighthouse Engineer for authority to make the change, giving his reasons and a full statement of the facts in the case.

All changes made must be reported and shown in red on the blue prints.

Concrete for foundation work, piers, walls of buildings and cistern shall be mixed in the proportions of 1 bbl. of cement, 3 bbl. of sand, and 6 bbl. of broken rock having no dimensions greater than 2 in., excepting partition walls above the floor line, where the dimension of the rock shall not exceed 1½ in.

Concrete for floor beams shall be mixed in the proportions of 1 bbl. of cement, 2 bbl. of sand, and 4 bbl. of broken rock having no dimension greater than 1½ in.

The moulds under the concrete beams shall be supported rigidly for at least 3 weeks after the concrete is placed, and until all danger of sagging is passed. Posts on wedges may be used under the mould planks so as to obtain exact alignment. The concrete in the beams shall be mixed in the proportion, 1:2:4. All other concrete shall be mixed in the proportion, 1:3:6.

The construction of the concrete of each beam shall be completed in a single day. The concrete shall be mixed wet, and as the moulds are filled, in 6-in. layers, cement mortar is to be first placed against the face to secure a smooth exposed surface. All concrete corners, both horizontal and vertical, shall be either rounded or beveled.

Forms for walls shall be made in narrow horizontal sections, and, immediately before the concrete is placed in a form, a coating of mortar (1 cement and 3 sand) shall be placed against the sides of the forms. The concrete shall then be thoroughly rammed.

The roof covering shall be of corrugated galvanized iron, No. 24, B. w. g.

The more important question of tower design was not taken up until after that of the dwellings, for the reason that the parts (nearly complete) of three large steel towers, which had been bought in France by the Spaniards, were found on hand and were used by the Americans in three of the first new stations to be built.

The disadvantages of structural steel as compared with concrete, under the conditions existing, soon became apparent. The expense and length of time required to ship heavy steel members from Manila to the station, to unload them on a coast surrounded by coral reefs—usually in ships' boats or small scows and through the surf—and then to transport them from the beach to the site added largely to the already considerable first cost. The riveters available—Chinese, Japanese and Filipinos were all tried in turn—worked slowly, and even with constant watching did not do first-class work. The frequent handling resulted in much rubbing and scratching of the paint, and the appearance of many spots of rust; and the native painter, to whom looks are everything, could not be made to see the necessity of scraping off the rust before applying the paint. It is this tendency of the native mind that makes the durability of the steel towers doubtful, as the lighthouse keepers, who are charged with keeping them properly painted, are all Filipinos.

Concrete towers, on the other hand, require but little attention, and cost practically nothing for repairs, an important consideration

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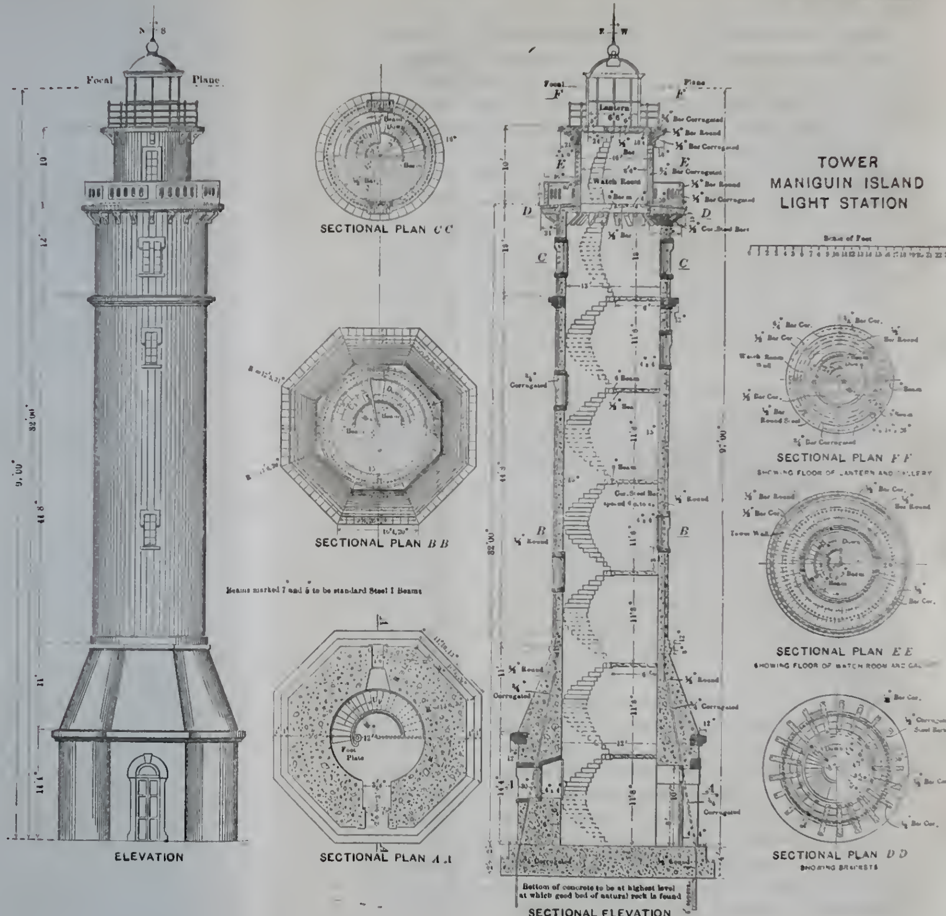
LIGHTHOUSE CONSTRUCTION
IN THE PHILIPPINES.



when all materials and most of the workmen have to be transported long distances in special boats. Then, too, all the projected lighthouses were to be erected either on high and rocky sites or on low-lying coral islands, and in both cases all the materials for concrete, except cement, were to be found in the immediate vicinity. By using steel rods to reinforce the concrete, the total mass of the latter, in a tower of given height, could be safely reduced, the upper walls of the tower could be made thin and light, and the base section broad and heavy, thus lowering the center of gravity and increasing the stability against overturning by earthquakes. The steel rods also served to bind all parts of the tower solidly together, and thus enable it to resist better the quick vibratory movements, made up of vertical as well as horizontal components, which have proved most destructive in severe earthquakes.

All these considerations led to the adoption of reinforced concrete as the material to be used in the construction of the higher towers to be ercted. The accepted designs of three of these are shown on Plates LXII, LXIII and LXIV, and a view of the first in its finished condition is shown by Fig. 2, Plate LIX.

This tower was erected in place of the one designed by the Spanish engineers, shown by Fig. 1. Though its focal plane is 5 ft. higher than the latter, it contains less than half the quantity of masonry required by the Spanish plan. The cost of the metal reinforcement amounts to barely one-tenth of the cost of the concrete saved. The Spanish design, however, gives a factor of safety against overturning by wind pressure of more than 10, while the new tower has a factor of only 7, though this is believed to be ample. On the other hand, its center of gravity is about 7 ft. lower than that of the lower Spanish tower, giving it a greater stability in the case of earthquakes.

The following details of the Maniguin Island tower may be of interest: The vertical reinforcement throughout is of $\frac{3}{4}$-in. corrugated steel bars, the spacing being about 6 in. in the walls, up to the floor of the watch-room, and about 12 in. from center to center in the wall of the watch-room. All these bars are set about 3 in. from the outer surfaces of the walls.

The circular horizontal reinforcement throughout the tower is of $\frac{1}{2}$-in. round steel bars, spaced 12 in. from center to center, next to and inside of the vertical bars. The lantern gallery floor is reinforced with two ½-in. round steel bars placed 3 in. and 15 in., respectively, from the outer edge of the gallery.

The horizontal reinforcement in the floors of the lantern and watch-room, and in all stair landings is of ½-in. corrugated steel bars, spaced 6 in. from center to center, and 1½ in. above the bottom of each floor. These bars extend about 12 in. into the walls on each end.

The radial reinforcement of the watch-room gallery is of ½-in. corrugated steel bars, spaced as shown on the sectional plan on *EE*, that is, about 6 in. from center to center at the interesctions with the vertical ¾-in. corrugated bars.

The brackets under the lantern and watch-room galleries are reinforced with ½-in. corrugated steel bars.

The reinforcement of the parapet on the watch-room gallery consists of one ½-in. corrugated steel bar over each bracket vertically, and one circular ½-in. round steel bar 3 in. from the top.

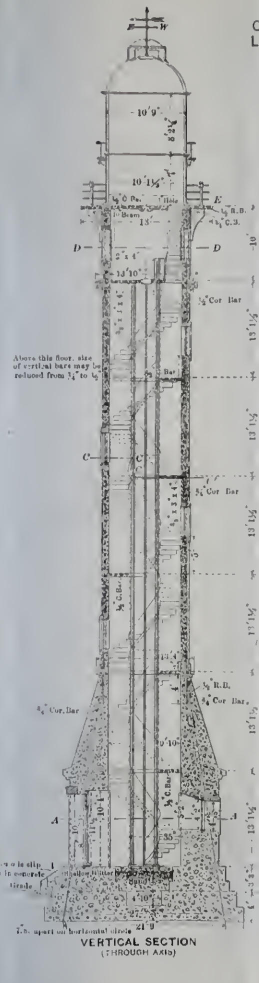
The proportions of the concrete in the blocks, brackets, door and window trimmings, etc., were 1 part cement, 2 parts sand, and 4 parts broken stone. The proportions in other parts were 1 part cement, 3 parts clean sand and 6 parts broken stone or gravel, no piece of which had a greater dimension than 2 in. The concrete was carefully and thoroughly mixed, and was placed in 6-in. layers, each of which was well rammed. Sufficient water was added to make the concrete so wet that it quaked moderately under the rammer. All concrete surfaces more than 12 hours old were wet and then covered with a ½-in. layer of mortar before fresh concrete was placed on them.

The steel bars were not painted, but all rust scales were ordered to be scraped off just before the bars were placed in the concrete. In every case the metal was covered with at least 1½ in. of concrete.

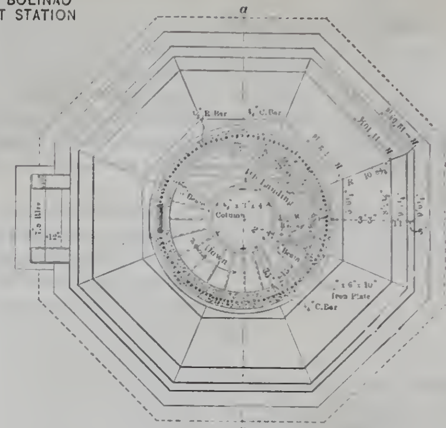
The vertical corrugated bars were in lengths varying from 20 to 27 ft., and their joints were staggered so that adjacent ones occupied several different planes. All bars, vertical and horizontal, overlapped from 24 to 30 in. at the ends, and were wired together. Great care was taken to make all vertical rods absolutely straight and to align them accurately. The horizontal and vertical bars were wired together at each intersection with one another.

TOWER
CAPE BOLINAO
LIGHT STATION

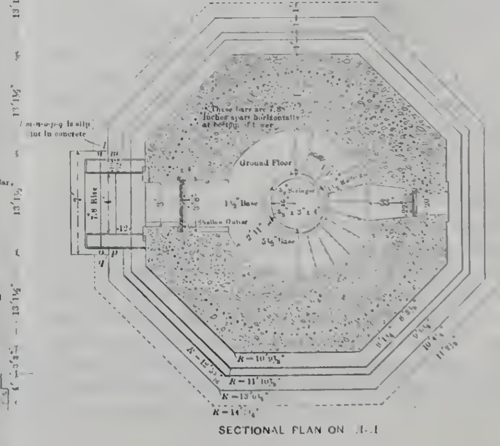
ELEVATION



VERTICAL SECTION
(THROUGH AXIS)



SECTIONAL PLAN ON C C-C'C'



SECTIONAL PLAN ON A-A

The erection of the second tower (Plate LXIII) has just been completed at Cape Bolinao, on the west coast of Luzon. It is not built on solid rock, like the first, which has its vertical reinforcing bars prolonged below the base to act as anchor rods. The masonry portion of the two towers is of the same height, but the second is of simpler design, and requires a somewhat smaller quantity of concrete. Simplicity of construction was a factor having paramount influence in determining the lines of the tower. Laborers skilled in reinforced concrete construction were unobtainable, therefore men of the regular force had to be educated to the special requirements of this kind of work; even to the American overseers in charge it was comparatively new, and, as the site is isolated and difficult of access, frequent inspections from the head office could not be made. Lumber is expensive, and to require many or elaborate forms would add materially to the cost, and delay the time of completion. The design of tower selected, consisting essentially of a long, hollow, concrete cylinder resting on a spreading base, each face of which was a plane surface, required comparatively few forms, and of such a simple character as to give little excuse for mistakes of any kind. As an additional guard against these and against defects due to carelessness or ignorance, however, all factors of safety were made large, and the quantities of concrete and of metal reinforcement were both increased beyond what would be required in ordinary construction. The cornices, the door and window openings, and the bracketed balcony of the lantern are the only features that relieve the simplicity of the design or call for the exercise of more than the most ordinary care or ingenuity in making the forms and setting the steel bars.

The upper part of the tower was given a circular rather than a polygonal cross-section in order to diminish the effect of the wind, which, for purposes of computation, was assumed to exert a pressure on the cylindrical surface equal to half that exerted on a vertical section through the axis normal to the direction of the wind. The maximum wind pressure was assumed as 70 lb. per sq. ft. This is undoubtedly high, but as a wind velocity of more than 120 miles per hour has been recorded at the Manila Observatory during a typhoon, and as the tower is to stand in an exposed position, it was thought advisable to be on the safe side. The Spanish engineers sometimes adopted a maximum wind pressure as high as 80 lb. per sq. ft. in justifying their designs. With a pressure of 70 lb. per sq. ft. the smallest factor of safety of any section against overturning, under the assumptions made, is about $6\frac{3}{4}$, while the greatest compression on the concrete (at the foot of the cylindrical section) would be 186 lb. per sq. in. In all these calculations the weight of concrete was taken at 140 lb. per cu. ft., which is a pound or two less than the weight found by actual measurement of concrete at other stations where the sand and stone are about the same as those to be used in this tower.

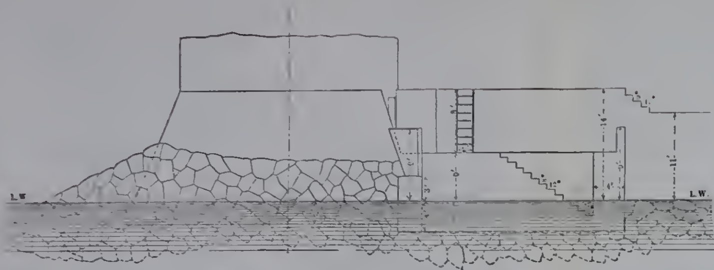
When it comes to earthquakes, so little of a definite nature is known of them, and their effects vary so much with the locality, that it is out of the question to attempt to calculate even approximately the ability of the tower to resist them. In small earthquakes, the vertical component of the motion is often imperceptible, but in destructive shocks, buildings are not only subjected to horizontal stresses, but are also tipped and rocked, and the ground may describe elliptical or other curved paths. The destruction wrought in high structures is usually the result of the rapidity with which vibratory movements are communicated to them. When the lower part of a tower is suddenly moved forward, the inertia of the upper part makes it tend to remain at rest, and a fracture between the two may result. The lighter the upper part the less danger is there of damage from this cause and of overturning from tipping. Hence the importance of placing the center of gravity as low as possible. The stability of a well-built tower, against earthquakes, may be considered as an inverse function of the height of the center of gravity above the ground, and as a direct function of its horizontal distance from the edge about which overturning may take place. The results of numerous recent experiments in Japan on the stability of columns is summed up by Baron Kikuchi as follows:

"For given values of height and external dimensions, a hollow column has a greater seismic stability than the corresponding solid one, the sides or walls in the former column being sufficiently thick and rigid for it to be regarded as a single structure."

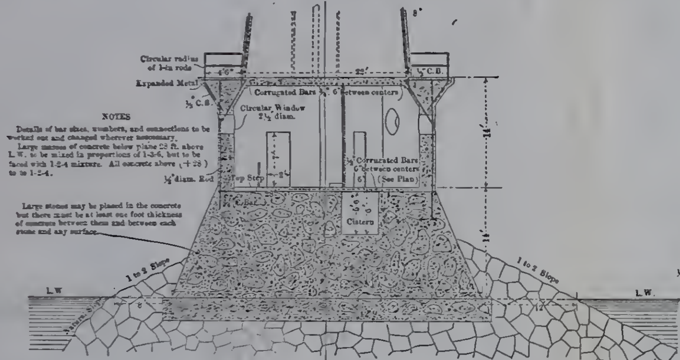
The effort was made to keep the foregoing considerations well in mind in designing the Cape Bolinao tower, at the same time having due regard to simplicity and rapidity of construction.

PROPOSED LIGHTHOUSE TOWER AT OUTER END OF MANILA BREAKWATER.

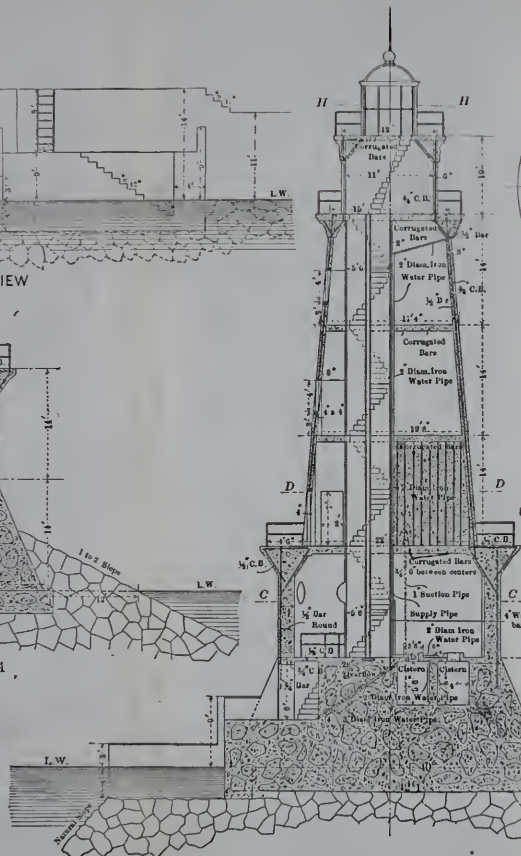
Scale of Feet 0 5 10 15 20



HARBOR SIDE VIEW



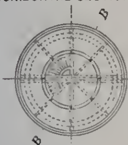
VERTICAL SECTION ON *A-A*



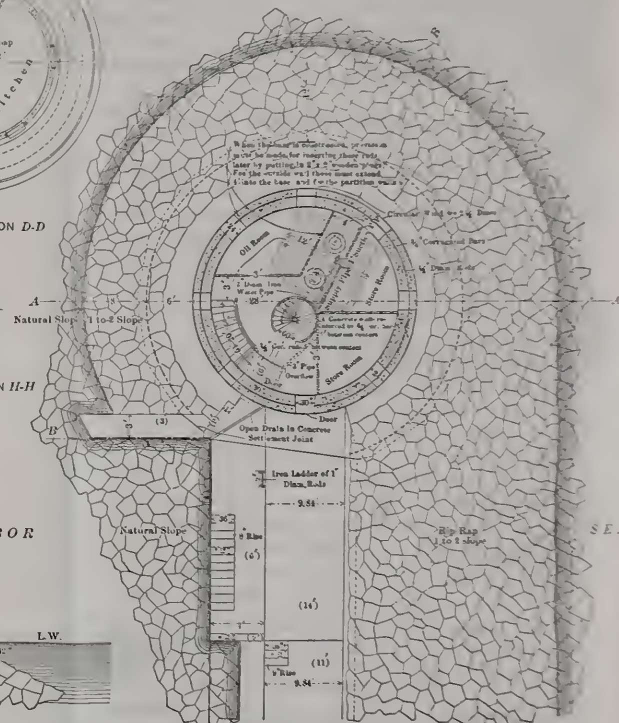
SECTIONAL ELEVATION ON *B-B-B*

HARBOR

HORIZONTAL SECTION ON *D-D*



HORIZONTAL SECTION ON *H-H*



HORIZONTAL SECTION ON *C-C*

As already stated, the motions due to an earthquake are of too complicated and uncertain a nature to attempt to calculate the resisting strength of the tower; but, as a check on the quantity of reinforcement, the following method was used in computing the factor of safety against a simple tipping movement.

An old instrument in the Manila Observatory once recorded an inclination of 13° from the vertical, due to an earthquake, and this was the greatest inclination of which an authentic record could be found. The observers, however, place little reliance on it, considering it much too great; undoubtedly a good part of the movement of the instrument was due to the throw imparted by the quickness of the impulse given; but, in the absence of more reliable data, and as a basis for calculations making little pretense to accuracy, this inclination of 13° was assumed as a fair measure of the simple overturning force to be resisted.

Let W = the weight of the tower,

G = the height of the center of gravity above the base,

D = the distance from the center of the base to the edge about which overturning may occur.

Then, when the tower is inclined 13° from the vertical,

$W \sin. 13° \times G$ = overturning moment.

$W \cos. 13° \times D$ = resisting moment.

Therefore, the factor of safety $= \dfrac{W \cos. 13° \times D}{W \sin. 13° \times G} = \dfrac{D}{G} \cot. 13°$

$$= \frac{13.5}{25.6} \cot. 13° = 2\tfrac{1}{3}.$$

This is the factor of safety against overturning about an edge of the base.

At any other section, as at the base of the cylindrical portion, both the longitudinal component of the weight and the tension of the reinforcing bars act to resist overturning, and the factor of safety must be computed in a different manner. This is done by using the formula, $P = \dfrac{My}{I}$, in which

P = the tension (or compression) on a unit area of the section,
M = the weight above the section $\times$ sin. 13° $\times$ the height of the center of gravity of the portion above the section,
y = 5 ft. 5 in. = the radius to the center of the sectional ring,
I = the moment of inertia of the sectional ring.

Substituting the proper values in the formula, $P = 31\,100$ lb. per sq. ft. is obtained for the maximum tension due to the inclination of the tower. Part of this is taken up by the pressure per square foot, due to the weight above ($10\,655 \times \cos. 13°$), leaving 20 808 lb. to be resisted by three ¾-in. corrugated bars, having a sectional area of 0.37 in. and a minimum elastic limit of 50 000 lb. per sq. in. These give a factor of safety of 2.7, which is slightly greater than at the base.

It may be noted that, under the above assumptions, the maximum compression on the concrete at any edge would be a little more than 300 lb. per sq. in., which is within the allowable limit. It should be repeated, however, that none of these computations pretends to measure the resistance of the tower to earthquake shocks—they were useful in proportioning its parts.

Plate LXIV shows the adopted design of a reinforced concrete lighthouse, on a rubble-mound base, which is to form the outer end of the new Manila breakwater. In designing this, the same assumptions as before were made, except that the weight of unsubmerged concrete was taken as 150 lb. per cu. ft., as a heavier stone is to be used for the aggregate. The frictional resistance of the mass to sliding was taken as 65% of its weight, allowing for submergence up to 14 ft. above mean low water, and the maximum wave pressure as 1 700 lb. per sq. ft. Both figures, of course, are mere assumptions based on the values found at places where the conditions were similar to those at the locality in question.

It is the intention to allow ample time for the consolidation of the breakwater substructure, which was only recently built, before beginning the erection of this tower. The construction of its foundation, however, has been completed.

Too short a period has elapsed since the completion of the other two reinforced concrete towers to warrant claims being made that experience has proved the soundness of their design, though one of them has already passed unscathed through a typhoon during which the recorded velocity of the wind in Manila exceeded 100 miles per hour. That they will successfully withstand the attacks of typhoon and earthquake, and cost little for maintenance, the writer has no doubt, provided, of course, reasonable care and skill were used in their construction.

DISCUSSION.

Charles E. L. B. Davis, M. Am. Soc. C. E.* (by letter).—The writer has read this paper with much interest, and wishes to offer some criticism on the asserted indestructibility of the native hard woods by the teredo and the white ant. Col. Davis.

In the harbor works of the Port of Manila there were two dredges of the endless-chain bucket type, built by the Spaniards and used by the Philippine Government in dredging the mouth of the Pasig River. These dredges dug up at various times pieces of native wood, and several pieces of molave were in the office of the works of the port on the Malecon Drive in Manila when the writer left the islands. These pieces had probably dropped from the decks of passing boats, and, like most of the native woods, being heavier than water, had sunk to the bottom. These pieces of molave were honeycombed by the teredo. The appearance of the wood indicated that it had not been long in the water. In addition to the boring of the teredo, there were several perfectly round, smooth holes, about ⅝ in. in diameter, at right angles to the direction of the fiber of the wood and passing entirely through the specimens. Apparently, these were made by the same animal that bores similar holes in the soft adobe stone used by the Spaniards in the construction of the sea-walls at the mouth of the Pasig River, several specimens of which the writer obtained alive from the cavities of this stone. It is one of the family of Pholadidæ, a bivalve mollusk that bores into wood and stone.

It is for this reason that the wharves or slips which are to be built for the accommodation of vessels in the harbor inside of the breakwater at Manila are to have concrete piers on pile foundations, with steel superstructures.

With reference to the statement that no wood was allowed to come in contact with the ground, and that the lower parts of the posts were sunk in concrete piers, as a precaution against the attack of the white ant, the writer will relate his experience. During a portion of his stay in Manila he lived on Nozaleda Street, in one of the buildings erected by the Spaniards for occupancy by the engineer officers of their army. The United States Quartermaster's Department had built, over the doors and windows, rain sheds of corrugated galvanized iron; those over the windows were supported by brackets and those over the doors by posts on concrete blocks, as shown by the photograph, Plate XLV. These posts were of native hardwood, but, as they were painted, the writer does not know whether or not they were of molave.

On the further post, as can be plainly seen in the photograph,

* Colonel, Corps of Engineers, U. S. Army

Col. Davis. was a crack, either a wind-shake or a sun-check. Sitting on the porch one day, the writer observed a winged ant flying around this crack, evidently examining it very carefully, alighting frequently and going into it as far as possible. A few mornings later, it was seen that a white ant gallery or tunnel had been built from the ground up over the cement base and about a foot or more up the post in a direct line toward the crack. The writer scraped the tunnel off and killed the ants by stamping on them; but, a few mornings afterward, he observed that the same thing had taken place, a tunnel had been built during the night a little higher up the post than before. This time the writer scraped off the gallery and poured kerosene over the ants, which killed them instantly; he then saturated the ground with the oil, which seemed to discourage the ants, for he saw no further attempts to enter the post during his stay in the house.

Evidently, any opening, however small, will afford an entrance to this pest, which will go to great trouble to reach the opening, and great precautions will have to be taken to ensure freedom from the attacks of this insect.

Johnson's Cyclopedia, in the article on Termites, says there are several kinds of individuals in each nest: (1) sexual individuals, kings and queens; (2) neuters, abortive males and females, workers and soldiers; (3) larvæ of males and females, workers and soldiers; and (4) nymphs of the same. The males and females when they come to maturity have wings. The workers measure from $\frac{1}{6}$ to $\frac{4}{6}$ in. in length, have soft, white bodies, and no eyes. They habitually avoid air and light. When they desire to reach a certain point, a covered gallery is immediately built to that point, that they may reach it unseen.

The above description from the Cyclopedia corresponds to the writer's observations as far as they extended. The ants he saw, that is, the workers, looked like white, blind grubs, about $\frac{1}{4}$ in. long, and were very active when disturbed. The gallery was made of earth moistened with some secretion of the ant, and was about the color of potter's clay when fresh; it becomes very hard in time. It is about the size of a lead pencil in exterior diameter, and adheres very firmly to wood and stone. They only work on the gallery at night or in the dark.

Mr. Cosby. SPENCER COSBY, M. AM. SOC. C. E. (by letter).—By the writer's statement that the natural hardwood, "molave," resists the attacks of both the white ant and the teredo, it was not meant that it is not at times attacked and injured by these agencies. The writer's experience, however, has been that molave has little to fear from the white ant. The latter will occasionally build its tunnels or galleries along the outside of a stick of molave or in one of the

Porch of Residence in Manila, Showing Crack in Post Attacked by White Ants.

cracks which abound in that wood, or will even burrow a short distance through it, but it will nearly always be found that its purpose is to reach some softer and less resisting wood. In the case of the porch, cited by Colonel Davis, it is likely that the ants were trying to reach the beams of the roof, which were probably of pine, than which no wood seems to be more rapidly destroyed by this insect. In once instance which came to the writer's notice, the carefully laid pine floor of an inside room was honeycombed by ants to such an extent as to be unsafe eight months after it was placed. In similar cases, molave posts and beams in contact with pine were hardly touched. Mr. Cosby.

At every lighthouse station in which any of the structures were built of wood, the keepers have received instructions to make a close examination of each post and pier at least once a week, to destroy carefully all ant galleries found, and to pour kerosene over the ground around their holes.

As to the vulnerability of molave to the teredo, the evidence is more conflicting. Although the writer heard of several instances similar to those cited by Colonel Davis, in which pieces of this wood were badly honeycombed by the teredo, there were also cases in which molave piles, which had been standing in exposed positions for years, had hardly been touched by them. The only explanation the writer has heard advanced is that there are several varieties of molave, some of which resist the attacks of the teredo far better than others.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1045.

THE FATIGUE OF CONCRETE.

By J. L. Van Ornum, M. Am. Soc. C. E.

General.

Since submitting the paper on "The Fatigue of Cement Products,"* which showed that, for neat-cement blocks in compression, repeated loading would definitely cause failure where the load was greater than about half the load necessary to crush the specimen with one application, the writer has been investigating the same phenomena where concrete was the material used. This research has included both concrete blocks in compression and reinforced concrete subjected to transverse loading; and, in developing this law of fatigue, about 600 tests have been made.

A large number of tests was absolutely necessary in order that the unavoidable variability in strength of individual test specimens, which occurs in such a material as concrete, even when the greatest care is taken in its fabrication, should be eliminated as much as possible by averaging a sufficient number of variable results, so that the average might represent, as closely as might be, the real value desired. It sometimes happens that, in testing concrete, values vary 20%, or even 30% or more, from the average value of the same concrete specimens. If, then, a specimen tested for the 100% load, causing it to fail with one application, happened to be 20% weaker

* *Transactions*, Am. Soc. C. E., Vol. LI, p. 443.

than the average strength, and an average strength specimen to be tested by repeated loadings at a 75% load was actually loaded to 75% of the weaker load, the actually existing percentage of load in such test would have been 60% instead of the intended 75%, which latter value must be the recorded one. In other words, if the average compressive strength of a number of concrete specimens is actually 30 000 lb., and the trial specimen failed at 24 000 lb., the 75% loadings would erroneously be 18 000 lb.; and this specimen would actually take about 4 000 repetitions before failing, rather than about 500, as it evidently should. Of course, an opposite condition of test values would have a contrary effect; and there is evident a necessity of averaging a very considerable number of tests in order to minimize the effect of such influences.

In addition to the consideration just mentioned, precautions were taken to secure uniformity of material and treatment, as far as was practicable. While, even then, there were large variations in the strength of different specimens, such variability was probably decreased by always mixing in one batch enough material to make eight blocks or four beams, of which usually half would be tested for the 100% (or ultimate) strength and the remainder for fatigue.

All the concrete for all the tests was a 1:3:5 mixture, by volume. The cement was a standard American Portland cement, and was tested completely according to the specifications recommended by the Special Committee of the American Society of Civil Engineers on Uniform Tests of Cement*, and thus proved to be of general good quality. The sand was the Mississippi River sand, ordinarily used in the vicinity of St. Louis, Mo., for mortars and concretes. It is a water-worn sand, rather fine, and varying considerably in important characteristics with different consignments, as will be seen from the following facts: On a No. 10 sieve, from 5 to 10% was retained; passing a No. 10 sieve and retained on a No. 20 sieve, there was from 10 to 15%; No. 20 to No. 30, there was 15 to 27%; No. 30 to No. 40, 20 to 25%; and passing a No. 40 sieve, 23 to 50 per cent. The voids in the selected sizes of sand varied from about 38 to 30%; and for unscreened sand as delivered, from 33 to 26 per cent. The weight of the sand was about 99 to 110 lb. per cu. ft., when dry and moderately shaken.

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* *Proceedings*, Am. Soc. C. E., January, 1903.

The stone for the concrete was the broken limestone in the vicinity of St. Louis, Mo., crushed to pass a 1½-in. screen. About one-half was larger than the 1-in. size, and about one-tenth was less than the ½-in. size. It weighed from 80 to 95 lb. per cu. ft., and its voids varied from 42 to 48 per cent. The percentage of voids in the 3 parts sand to 5 parts broken stone was from 16 to 19. While, from the theoretical point of view, it would probably have been more scientific to have screened all the sand and to have used selected proportions of the different sizes, thereby undoubtedly securing an increased uniformity of test results; still, for practical reasons, it was determined to use the unselected and unscreened materials, just as is done in engineering works of concrete, in the proportions 1 : 3 : 5, which would seem to satisfy fairly well any combination of the variable conditions of the materials mentioned above.

The weight of the resulting concrete when dry averaged 146½ lb. per cu. ft., the extreme variation being about 3 per cent. After fabrication, the concrete compression specimens were left in the moulds in air for 1 day, and the reinforced concrete beams for 2 days; when removed from the moulds, all specimens were placed in water for 2 weeks, and were then stored in air, protected from drafts, until tested.

As it was realized that the contemplated series of tests would require repetitions in number running into the hundreds of thousands (actually, the number of loadings reached a total of considerably more than half a million), it was deemed essential that an apparatus be devised which would release the load on the specimen automatically when it reached the desired limit, and then at once allow the same load to be imposed again. This arrangement not only saved an immense amount of labor, but also permitted the tests to proceed uninterruptedly, night and day, until the specimen failed, the register, of course, tallying the number of repetitions so that this essential record should not be lost. The device was an electrical attachment to an ordinary Riehlé universal testing machine of the hydraulic type, an oil pump forcing oil into its cylinder to effect the loading. Whenever a specimen broke in the night, pumping would continue until the movable cross-head, which brings the pressure on the specimen, was lowered an inch or two be-

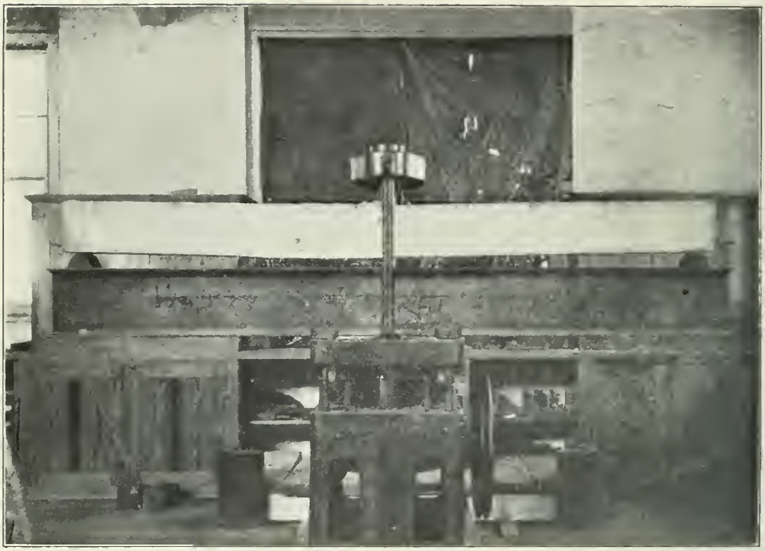


FIG. 1.—REINFORCED CONCRETE BEAM, AS SUBJECTED TO REPEATED LOADINGS.

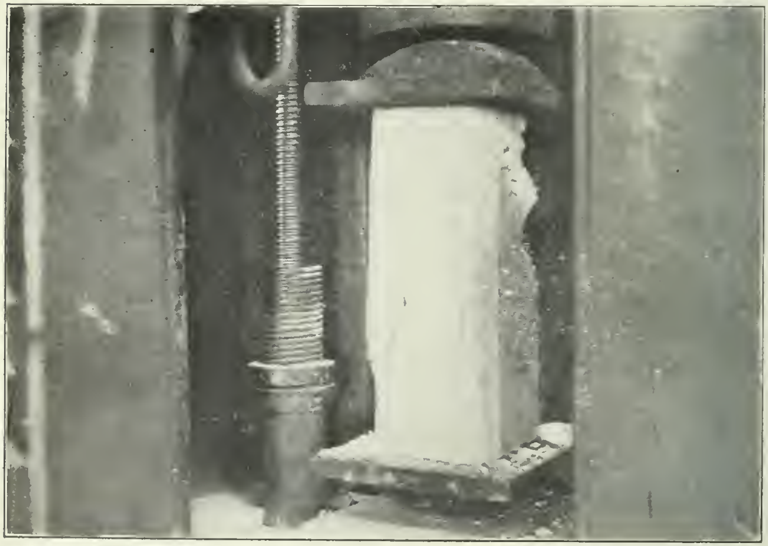


FIG. 2.—CONCRETE PRISM, AS SUBJECTED TO REPEATED LOADINGS.

low its normal height; when this occurred it tripped a lever connecting with the handle of the starting compensator, thus shutting off the current from the motor driving the oil pump.

In the ordinary operation of the device, the flow of oil into and out of the cylinder of the testing machine, and therefore the repetition of loading, was effected by a piston (or plunger) valve which was actuated by the successive reversals of a rotating shaft threaded at the end to engage a female screw attached to the plunger. Rotation was communicated to the shaft by a pair of magnetic clutches built in the form of loose pulleys mounted on the shaft, and belt-driven in opposite directions, the clutches engaging successively with a disk keyed to the shaft, and situated between the two clutches. The necessary end play of the disk was permitted by fitting it to the shaft and key with a loose sliding fit.

The normal position of the plunger valve was such that the pump forced oil into the testing machine. When the load on the specimen reached the value set by the locked poise, the rising of the beam closed a circuit through one of the clutches, thereby attracting the disk and causing it to rotate in such a direction as to drain the oil from the testing machine by the opening of the plunger valve. The consequent dropping of the beam then opened the circuit, thus releasing the disk and stopping the travel of the valve.

The fall of pressure in the oil system then actuated an adjustable pressure valve which was arranged to close an electrical circuit through the other, or reversing, clutch, when the load on the specimen reached a predetermined minimum amount. The plunger valve was thereby closed, thus building up the load on the specimen. At the moment the plunger became fully seated, the circuit through the clutch was broken by the opening of a contact attached to the reciprocating nut of the plunger. The two magnetic clutches then revolved unmagnetized, ready for a repetition of the cycle, as already described. An automatic register was attached to this automatic device in such a way that it recorded the forward motion of the revolving shaft each time it released the pressure on the specimen.

Compression Tests.

The concrete prisms on which these tests were made had the proportions already given, and series of tests were made at ages of 1 month and 1 year. These prisms were approximately 5 by 5 in. in section, and 12 in. high; the corners being rounded, the net section averaged (with very slight variation) 23.6 sq. in. Each end was capped in the making with about 1 in. of mortar in order to prevent a concentration of pressure where otherwise a piece of stone might extend to the surface. Still further to insure uniformity of loading throughout the cross-section, as soon as the prisms were removed from the water, the two ends were ground to plane surfaces in a specially constructed abrasion machine. When placed in the testing machine, after standing in air until the test was due, these plane surfaces were embedded in a thin layer of plaster of Paris (between sheets of dry oiled-paper) which was allowed to harden for at least 2 hours under a pressure of about 20 lb. per sq. in., and spherical bearings were used, both above and below the test specimen, to eliminate errors that would otherwise result from slight lack of parallelism of the two ground surfaces of contact. The writer considers the use of plaster of Paris of doubtful value where care is used in the grinding of bearing surfaces; but he does consider the use of spherical bearings essential in compression tests; although the labor of correctly centering the specimen is increased thereby.

The initial load, of perhaps 500 lb., represents the minimum loading of all specimens, the maximum repetition load being fixed by the position of the poise of the weighing beam. This minimum load was necessary to prevent the falling out of the specimen, which, of course, would have occurred if the load had been entirely removed, as the specimen stood on a spherical bearing. The rate at which repetitions of the maximum loading occurred depended upon the percentage which this load bore to the compressive strength of similar specimens as ordinarily obtained by simple crushing, being less for the higher percentages. This rate varied from 4 to 8 per minute. As a rule, when repetitions were begun upon a prism, the test was continuous night and day until failure occurred.

Generally, from each "mix" of concrete, two blocks were crushed in the ordinary way with one application of load, the average of the two recorded results being taken as the true "100%" load. If these two values were quite different, or if the following repetition tests, on the remaining specimens of the same "mix," were so erratic as

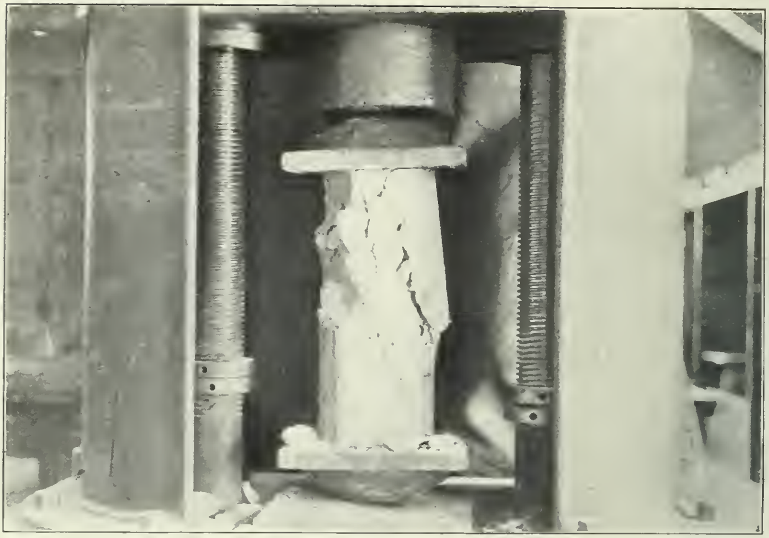


FIG. 1.—CONCRETE PRISM. JUST BEFORE COLLAPSE.

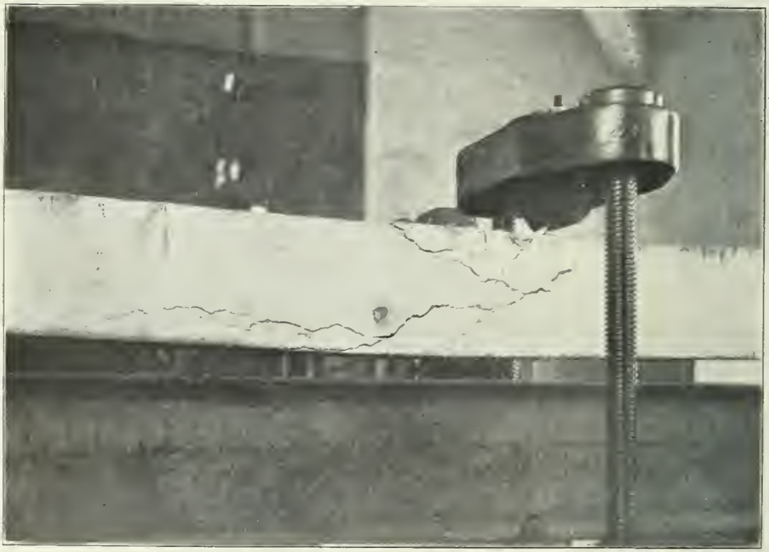


FIG. 2.—REINFORCED CONCRETE BEAM, JUST BEFORE COLLAPSE.

to indicate a probable erroneous value of this "100%" load, others were crushed, until an apparently average "100%" value was secured. Then a certain percentage of this "100%" value was taken as the repetition load for fatigue tests, and the testing was accomplished as indicated above.

Tables 1 and 2 give the results of the 1-month compression tests, the average crushing strength of the concrete at this age being 1 200 lb. per sq. in. Tables 3 and 4, similarly, exhibit the results of the 1-year tests, the average 100% strength being 1 580 lb. per sq. in., an increase of strength of about one-third more than the strength at the age of 1 month. While these repetition results can in no sense be considered as giving a precise average, because of the very great range in values as given (due to unavoidable causes already mentioned), still it is thought that the combined results do definitely indicate, and rather closely approximate, the true law of the fatigue of concrete.

In order to determine whether or not the rate of application of the repetitions has a noticeable effect upon the law of fatigue, tests have been made in which the rate of repetition was never less than 4 per hour, and averaged (due to interruptions of night hours, holidays and other periods of rest) one loading in about 5 hours.

TABLE 1.—One-Month Compression Tests. 100% Loads.

30 000	32 610	23 500	31 650	37 280
20 000	30 100	30 500	32 230	26 780
31 140	31 280	30 120	33 200	21 020
25 820	23 000	26 000	26 400	26 500
25 650	25 000	24 000	27 610	19 800
25 130	23 020	36 560	35 480	22 850
27 270	23 100	26 000	30 840	24 780
27 400	20 970	25 800	31 220	34 150
28 450	26 240	31 200	29 950	27 660
27 640	23 200	25 800	24 200	30 810
30 900	22 540	26 700	28 190	30 200
19 280	26 000	28 040	35 160	26 800
31 090	37 500	25 720	31 880	26 800
27 230	22 740	28 100	37 000	30 020
23 250	23 210	30 200	25 400	37 060
34 400	30 780	32 500	27 800	23 090
33 800	30 100	29 750	26 600	23 480
23 330	27 400	28 000	36 000	30 870
32 930	35 600	29 700	21 500	25 400
36 860	26 200	31 570	24 000	23 430
30 040	28 000	34 160	19 800	29 340
35 240	24 800	26 720	32 150	32 040
31 360	33 000	28 410	25 700	30 500
25 380	32 900	35 760	25 220	25 620
26 800	32 110	33 810	32 290	29 050
24 640	24 600			22 940

Average of 128 = 28 350 lb.

TABLE 2.—Repetition Tests.

95%	90%	85%	80%	75%
6	102	131	123	826
92	51	20	85	101
4	2	25	145	233
3	79	1	57	384
3	151	750	109	1 119
16	3	206	79	416
31	14	26	1 527	168
150	29	198	28	
13	91	91	128	
54	1		656	
16	418		3	
3	206		37	
175	226		124	
8	35		22	
9	77			
3				
Av. of 16 = 37	Av. of 15 = 99	Av. of 9 = 161	Av. of 14 = 223	Av. of 7 = 464

70%	65%	60%	55%	50%
135	2 227	2 016	443	10 711
332	185	666	5 025	21 070
4 405	246	596	1 113	19 800
280	207	1 431	165	
350	13	994	431	
324	32	356	15 629	
1 020	1 881	10 711	173	
1 345	4 005	9 210	2 064	
359	62	7 200	52 440	
	27			
	185			
	5 304			
	510			
	9 957			
	5 525			
	247			
	1 875			
Av. of 9 = 950	Av. of 17 = 1 911	Av. of 9 = 3 687	Av. of 9 = 8 609	Av. of 3 = 17 194

TABLE 3.—One-Year Compression Tests. 100% Loads.

34 260	33 400	33 750	36 680	44 900
46 670	26 440	29 150	44 840	38 650
48 400	45 200	44 100	48 160	49 600
36 200	26 800	30 200	36 540	35 570
39 200	39 860	35 330	36 000	32 720
43 040	31 840	38 700	35 690	32 830
32 000	35 150	35 660	42 500	37 740
30 200	37 500	33 900	44 950	42 700
34 400	37 600	34 600	27 100	39 000
43 350	34 150	27 350	37 000	32 000
33 800	38 600	37 850	40 490	45 960
39 240	41 400	31 000	37 700	44 400
				35 600

Average of 61 = 37 370 lb.

TABLE 4.—Repetition Tests.

95%	90%	85%	80%
3	11	79	175
2	23	14	1 951
3	115	179	192
27	2	137	15
2	9	94	265
4	10	38	1
98	94	811	57
4	19	191	328
3	3	8	600
31	1	345	40
	12	139	
		90	
		3	
		713	
		47	
Av. of 10 = 18	Av. of 11 = 27	Av. of 15 = 193	Av. of 10 = 362

75%	70%	65%	60%	55%
971	2 145	1 168	7 867	2 480
1 138	3 279	2 378	450	83 159
183	332	1 600	10 673	4 237
110	275	2 067	1 360	40 302
542	16	3 731	3 579	
	1 130			
Av. of 5 = 589	Av. of 6 = 1 230	Av. of 5 = 2 189	Av. of 5 = 4 786	Av. of 4 = 32 545

In 50 tests made for this purpose (of which about three-fourths were compression specimens and the remainder were beams), in which the load percentage was intended to be between 85 and 92%, and averaged 89%, the average number of repetitions was, for the compression specimens averaging 90% load, 35 repetitions; and for the beams averaging 88% load, 62 repetitions. By comparing these results with those given in the tables preceding and following, and also with the general curve, there will be noticed no systematic difference in the relation between "percentage" and "number of repetitions," and certainly there is not evident any considerable increase in the number of repetitions necessary to cause failure even when the fastest rate of loading is reduced a hundred times, and where the average rate-ratio becomes as 1 to 1 500.

It is true that some other tests of effect of rate of loading* show a noticeable reduction of strength (or in number of repetitions

* As cited in Falk's recent work on "Cements, Mortars and Concretes," p 67 *et seq.*

necessary to cause failure) with increased rate of application of load; but, in such cases, the frequency of repetitions has been much greater than any used in the tests by the writer; and he believes that where the rate of loading is greater than about 10 per minute there is produced upon the strength of concrete noticeable effect, which rapidly increases as the rate of repetition is increased; while, if the rate of loading is less than 8 or 10 per minute, there is probably little or no effect noticeable in varying such repetition rate.

Reinforced Concrete Beams.

The reinforced concrete beams were 6 ft. long between supports, the extreme length being about 4 in. greater. In cross-section they were 4 in. wide and 6 in. high. The reinforcement consisted of two bars of plain steel, ½ in. square, and placed with their centers 1 in. above the bottom of the beam and 1 in. inside the sides of the beam, respectively. The elastic limit of the steel was 29 000, and its maximum strength 59 000 lb. per sq. in. It will be noted that the percentage of steel was somewhat greater than 2; this large amount was used in order to make certain the failure of the concrete alone, essentially unaccompanied by any weakening effect that high unit stresses in the steel would occasion. The concrete was of the character already described. The treatment after fabrication, and until the test, was similar to that of the compression prisms.

For the tests, the reinforced concrete beams were supported on knife-edges and bearing plates rounded sufficiently to prevent crushing; the load was applied at the center of the supported length through a similar knife-edge and rounded bearing plate. The automatic apparatus already described controlled the repetitions of load, which varied from the determined maximum to a minimum of zero. The rate of application of the loads, of course, depended partly on the "percentage" (or test) load, varying from 2 to 4 per minute.

Tables 5 and 6 give details of the tests at the age of 1 month, and Tables 7 and 8 give similar tests at greater ages. It will be noticed that, in the latter tables, tests at 6 and 12 months are combined, partly because the results at these two ages contain no systematic differences, and partly because the number of tests at the age of 12 months are too few for independent use.

TABLE 5.—ONE-MONTH TESTS, REINFORCED CONCRETE BEAMS. 100% LOAD.

2 800	2 950	2 050	2 440	2 400
2 860	3 250	2 930	2 610	3 460
3 760	3 940	2 470	2 460	2 920
2 860	2 680	2 360	2 920	2 600
3 530	3 050	2 080	2 200	3 820
3 760	3 650	3 490	3 060	2 720
3 500	3 150	2 980	2 630	2 060
2 400				

Average of 36 = 2 909 lb.

TABLE 6.—REPETITION TESTS.

90–99%		80–89%		70–79%		60–69%		50–59%	
%	Repetitions.	%	Repetitions.	%	Repetitions.	%	Repetitions.	%	Repetitions.
91	36	80	388	75	92	63	6 476	53	4 170
91	9	85	96	70	480	64	570	58	2 205
91	1	80	56	74	3 880	60	1 384	59	5 300
94	31	85	1	72	555	66	59	56	3 655
92	1	80	256	70	13	64	120	53	2 538
		80	9	75	14	66	1 630	56	3 560
		80	585	70	86	64	21	59	2 227
		80	23	75	760	68	6 316	50	1 527
		85	7	70	79	68	165	59	5 400
		85	75	70	155	60	162	59	7 670
		80	227	77	516	65	2 507	53	14 080
		86	334	77	1 600	62	34		
		86	74	77	39	69	2 016		
		80	1	70	24	67	1 622		
				74	1 310				
Av. of 5 = 92%; 16 repetitions.		Av. of 14 = 83%; 152 repetitions.		Av. of 15 = 73%; 640 repetitions.		Av. of 14 = 65%; 1 649 repetitions.		Av. of 11 = 56%; 4 757 repetitions.	

From a study of the results of these fatigue tests at different ages, and both in compression and transverse loading, it is thought that the actual law of fatigue of concretes is fundamentally independent of such variations of conditions, the number of repetitions necessary to cause failure depending essentially upon the percentage which the test load bears to the full strength of the concrete. This proposition is borne out by the fact that such variation of values as exists in the summarized values of Tables 2, 4, 6 and 8 are far within the limits of accidental errors. The writer, therefore, has drawn the mean curve, Fig. 1, to represent graphically the summarized law of fatigue of concretes as variously set forth in the four numerical tables just mentioned.

TABLE 7.—SIX-MONTH AND TWELVE-MONTH TESTS, REINFORCED CONCRETE BEAMS. 100% LOADS.

5 100	3 890	3 400	5 130
3 300	3 540	4 750	4 640
3 020	4 230	5 140	4 940
4 080	3 440	5 000	5 210
3 860	3 610	4 350	5 170
4 660	4 060	4 970	3 180
3 090	3 600	*4 100	*4 700
*4 120	*4 850	*4 100	*4 000
Average of 32 = 4 226 lb.			

* Results of tests at age of 12 months; all others made at age of 6 months.

TABLE 8.—REPETITION TESTS.

80–89%		70–79%		60-69%		50-59%	
%	Repetitions.	%	Repetitions.	%	Repetitions.	%	Repetitions.
81	14	75	709	60	1 777	56	4 117
81	* 18	72	492	67	2 970	55	*4 190
80	*140	70	1 643	65	2 375	59	*4 915
80	* 2	78	27	68	9		
89	* 9	75	312	68	2 608		
		79	113				
		70	317				
		74	* 297				
		78	*1 634				
		70	* 507				
Av. of 5 = 82%; 37 repetitions.		Av. of 10 = 74%; 605 repetitions.		Av. of 5 = 66%; 1 948 repetitions.		Av. of 3 = 57%; 4 407 repetitions.	

* Results of tests at age of 12 months; all others made at age of 6 months.

It will be noted that the curve as drawn would become horizontal at about the 50% load, which would fix the limit at which fatigue can affect concrete at about half the strength of the concrete as usually determined. There is some uncertainty as to the exact value of this limiting percentage, but it is thought to be not great. The somewhat indefinite value of this limiting percentage results from the fact that the tediousness of tests at low percentages, and other adverse conditions, prevented making such tests great enough in number practically to eliminate accidental results.

It will be noted that the result of the three compression tests made at 50%, at the age of 1 month, show an average failure at somewhat more than 17 000 repetitions. The writer thinks this result to be accidental, for the reason just given, because the 55% average of the 1-year compression tests is nearly double that value, and also because the three tests in which the repetitions exceeded 40 000 did not fail normally, but were each arbitrarily crushed at the repetition recorded when they gave no indications of weakening after such a large number of repetitions. In detail, the "52 440" specimen given in the 55% column of Table 2 was crushed, after the recorded number of repetitions, under a load 55% in excess of its repetition loading; the "40 302" specimen recorded in the

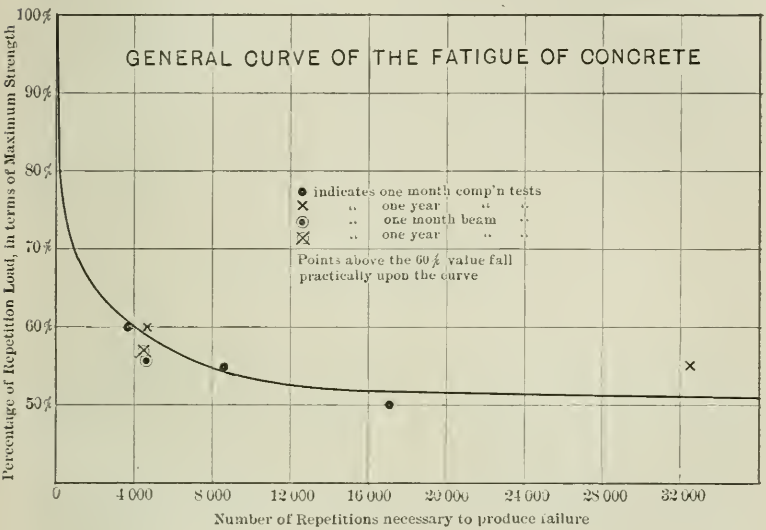


FIG. 1.

55% column of Table 4 was similarly crushed, to end the test, under a final load 81% above its repetition limit, or only ½% below its 100% load; while the "83 159" specimen, when arbitrarily crushed after bearing that number of repetitions, finally failed at a load of three times its repetition value, or a load 65% greater than the 100% value, as determined for this test. These three 55% specimens would probably have withstood an infinite number of repetitions.

These facts illustrate very clearly the inherent difficulty and, in fact, the real impossibility, of determining exactly the 100% load value of any specimen treated by repeated loadings, and lead plainly to the inference that the 17 000-value of the 50%, 1-month compression tests is misleading. Therefore, the writer considers the true location of the curve to be above this low point, and in the vicinity where it is drawn.

The phenomena of failure of these beams involved first the development of tension cracks, then usually (but not always) diagonal cracks (often described as "shear cracks," "web tension cracks," etc.), and last, a compression failure at the top of the beam near the point of application of the load—all these indications of weakness beginning with very minute manifestations, gradually and progressively increasing in size until failure occurred, and usually developing in the sequence just given.

The typical failure of the greater number of these beams is indicated in Fig. 2, which shows schematically the condition at the

Fig. 2.

moment of complete failure. The notched rectangle in solid black is at the center of the beam. The diagonal cracks usually began in the region below the half-height of the beam, extending in an oblique direction and developing gradually downward toward the nearer support, and becoming more horizontal with increase in length until, as a rule, they became horizontal in the plane of the upper side of the reinforcing bars; though sometimes they crossed the bars and united with tension cracks, and sometimes tension cracks extended across these bars, developing into real diagonal cracks of the form just mentioned, the latter in that case growing from below upward toward the load-point instead of in the contrary direction.

While a diagonal crack develops typically, as just described, during the middle period of the test, it also shows an increasing tendency to develop upward toward the load-point. When this failure is developed sufficiently to weaken the beam greatly and to restrict the compressive area, failure by compression begins, and progresses relatively rapidly to a complete collapse. Occasionally, instead of compressive failure manifesting itself so definitely and vitally, the whole prism of concrete (shown in Fig. 3, on the right of the load-point, the figure being drawn just before final failure) above the curved diagonal crack, which finally extended just above

FIG. 3.

the reinforcing bars to the very end of the beam, broke completely away from the remainder of the beam; thus destroying its load-carrying capacity.

Sometimes, the beams, somewhat weakened by the tension cracks first developing, finally failed without the intervening development of diagonal cracks of a serious nature, as indicated in Fig. 4, drawn somewhat before collapse occurred.

FIG. 4.

In the large majority of cases, those in which the development of diagonal cracks formed an essential factor in the fatigue of the beam, it appears that the gradual and progressive destruction of the adhesive bond of the concrete to the steel produced a vital influence upon the phenomena. Of course, usually, but not invariably, this destruction of the adhesion originated near the center (of length) of the beam, and progressed toward, without reaching, the ends. This slipping of the bars in the concrete was shown to be closely related to the development of the diagonal cracks, by breaking open beams, previous to final failure, both before and after the appearance of these diagonal cracks, the evidences of slipping being present after diagonal cracks developed, but usually not evident before.

These evidences of slipping were various. On breaking apart the beams, there was found at the ends a noticeable and effective tendency of the concrete to adhere to the steel; while in the middle portion there was a space over which this adhesion was entirely lacking. The evidence alone is not convincing, but when considered in connection with the phenomena following, it would seem to be conclusive. Close examination disclosed the fact that, where the bond remained effective, the surface remained dull; but it had a polished, glassy appearance when the bond seemed broken. Often such surfaces revealed minute, but plainly discernible, horizontal striations. These must have been caused by a relative longitudinal movement taking place between the surfaces of the two materials, thus etching the smooth concrete surfaces by small projections from the steel. Such striations were rarely found when essentially a compressive failure occurred, but they were invariably present when diagonal cracks were prominently in evidence. Certainly, this relative longitudinal movement could not occur while the adhesive bond was effective.

Again, occasional, very shallow, cavities (perhaps $\frac{1}{4}$ in. long, half as wide and $\frac{1}{100}$ in. deep) in the reinforcing bars were found, in one end of which there was a most finely pulverized cement dust, where the above-mentioned evidences of slipping were also present. This cement dust was invariably at the side of the depression toward the nearest place of unbroken adhesion, the remainder of the cavity being empty, as indicated in the normal section, Fig. 5. Evidently, this phenomenon must indicate a rela-

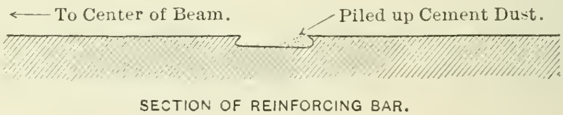


Fig. 5.

tive longitudinal movement of the two materials, necessarily taking place only after the bond was destroyed, the sharp edge of the depression scraping off particles from the concrete and receiving them as repetitions continued.

Another phenomenon, which the writer can explain only by the same proposition, was the occurrence sometimes of a succession of distinct, metallic reports, plainly heard when the load was being applied, followed by a continuous low, rasping sound as the beam was relieved of its load. This certainly proceeded from the interior of the beam, as made evident by careful auditory scrutiny and by the fact that no external deterioration of the beam was discoverable at the same time. It is believed that it was caused by the sudden breaking in detail of the adhesive bond at points where its unusual strength brought a concentration of stresses upon them.

The adhesive strength of concrete to steel, low in value at best, is undoubtedly severely tried by repeated application and relief of load, and the consequent successive production and relief of the various internal stresses which tax so severely this essential and vital factor of reinforced concrete design and construction. Passing without comment the acknowledged fact that scale or thick rust will seriously impair the adhesion, it may be said that numerous critical examinations plainly indicated that any rust on the metal (while completely absorbed by the concrete and so effectively preventing further corrosion) did materially lessen the normal adhesive power of the concrete; the bond was often found lacking opposite the rust discolorations on the concrete, while remaining firm on each side where rust had been entirely absent; and, where the adhesive bond was destroyed in the middle portion of the beam, this destruction habitually terminated in a discolored section, apparently indicating the encountering of an increased adhesive resistance at the cleaner portions of the steel.

Another fact that has escaped deserved attention is the probability that a material excess of water used in mixing the concrete apparently lessens its adhesive power. It is realized that a moderately wet mixture is desirable, in order to prevent voids in the concrete as ordinarily placed, and especially to secure sufficient plasticity to insure a complete filling of the space around and below the network of reinforcing steel; but there seems to be a real danger that the reaction against dry concrete is being carried too far. An excessively wet concrete, not only contains numerous globules of water which, when absorbed, leave the concrete porous, but these, also, especially weaken the adhesion of the concrete to the steel, because there is a tendency for such water globules to seek the surface of the reinforcement, particularly on the under side. The weakening of the bond from this cause was evident in certain beams in which the adhesion was noticeably weak, the water cavities being apparent at the bottom and sides of the steel bars.

Elastic Behavior.

In order, if possible, to learn more of the contributing and complementary phenomena in the fatigue of concrete, observations were frequently taken to disclose any changes that might occur in the elastic properties of the test specimens. About 100 such beams and prisms were thus observed, the number of observation readings involved in this investigation reaching perhaps 10 000.

In this experimental research upon the compression prisms, dial compressometers were clamped upon the specimens by steel collars placed 8 in. apart. The dials read to ten-thousandths of an inch. Readings were taken during the first application of the load, during the first repetition of the load, and at intervals during the life of the specimen under the repetition treatment. These observed strains, with the corresponding stresses as recorded, were reduced to unit strains and stresses, and plotted as in the lower part of Figs. 6 and 7. In determining the value of the modulus of elasticity at different stages of the fatigue-history of the prism, it seemed that the only practicable valuation for comparative purposes was to divide the total unit stress (at the repetition limit) by the total unit strain (occurring between the zero load and the repetition limit) regardless of any intermediate variations in the slope of the stress-strain curve. As thus computed, the variation in value of the modulus of elasticity was determined for various compression specimens and plotted, as illustrated in the upper parts of Figs. 6 and 7. These are mean values; the extreme range of all the tests was about 30 per cent.

The graphs of Fig. 6 are typical of the observations on compression specimens where the repetition load was great enough to cause final failure by fatigue. Nearly all the compressive elastic observations were of this character. The particular prism represented in this diagram failed on 332 repetitions, and six readings were taken on it, as indicated. It will be noticed that the plot for "first loading" is practically straight in its lower part, but convex (to the left) in its upper portion; while the second loading (first repetition) has removed this convexity, giving a practically straight line about parallel to the straight part of the first line. This reduction of a variable modulus of elasticity to a constant modulus throughout the load-range is characteristic of the first elastic stage. The second stage is characterized by a gradual increase in the slope of the plotted line, or, in other words, a reduction in the value of the (constant) modulus of elasticity, the plotted curve, however, still remaining straight for its whole length. This second stage bears a very close relation to the number of repetitions necessary to destroy the specimen. In prisms tested at a high percentage, where the number of repetitions to cause failure are few, this stage does not long continue; but when the test is at a low percentage, requiring a relatively great number of repetitions to rupture the specimen finally, the far greater part of this increase in repetitions is spent in passing through this second stage. The third stage begins rela-

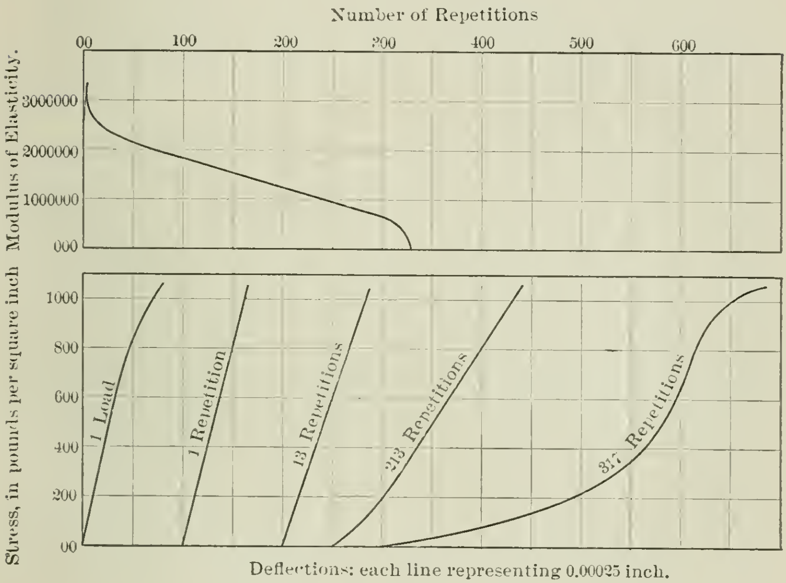


Fig. 6.

tively near to the failure of the specimen, and seem to indicate the beginning of the breaking down of its structure; it is characterized by a concavity developing in the lower part of the diagram. The last stage develops an additional curvature in the upper part of the plot, this being convex. The concave and convex curves rapidly increase in length, and then join, thus forming a reverse curve, which, at the same time, continually increases in its horizontal reach until complete failure occurs. These characteristics are plainly indicated in the five curves in the lower part of Fig. 6, the second stage being represented by two straight lines with somewhat different slopes, while each of the other stages is represented by one curve for each.

In the upper part of Fig. 6 is graphically represented the change in the modulus of elasticity. Characteristic facts are the sudden increase in its value at the first repetition, its rapid reduction to about its original value in the first few repetitions, its straight, sloping portion (which occupies a relatively greater part of the whole length of such curve when the number of repetitions to produce failure are greater), and the final increasingly rapid downward curve of the graph as the end is approached, finally reducing in value to practically zero.

At the beginning there seems to be an attempt of the concrete to adjust itself to the load conditions. Then quickly comes the critical stage, where, as in such cases as now described, the material is unable to endure the burden successfully, as is made evident by the regular diminution of the (constant) modulus of elasticity; though this diminution takes place slowly for the smaller loads. Then, near the end, comes the increasingly rapid reduction in its value, the modulus here becoming very noticeably of variable value from the beginning to the maximum value of any one loading. It was the writer's intention to attempt to determine whether these phenomena of elastic behavior were due to a breaking-down of the adhesive strength or of the cohesive strength of the cement, or possibly to a combination of these causes or to other causes, by extended microscopical examination of the structure of the prisms at critical periods of their behavior; but unavoidable adverse conditions prevented.

A few compression prisms were similarly observed where the imposed repetition load was too low to cause final failure, and these showed a radically different elastic behavior. In general, the first two stages mentioned above were again manifest, while the third and fourth were wanting. Fig. 7 shows these facts graphically for a specimen that withstood more than 30 000 repetitions, and was then crushed at a load as great as that of the average of the other three twin-specimens broken without repetition, the graphs being drawn similarly in every way to those of Fig. 6. The fact is noticeable that all the stress-strain graphs (except perhaps the first) are straight lines; and that the modulus curve, which, as before, suddenly increases and then rapidly decreases in value, quickly approaches horizontality in direction, with its asymptote having about two-thirds its maximum value, which implies that an infinite number of repetitions would not reduce the modulus of elasticity below this two-thirds value. It would probably be a valuable investigation to impose a large number of repeated loadings varying in intensity from one-eighth to one-third of the 100% load (instead of about

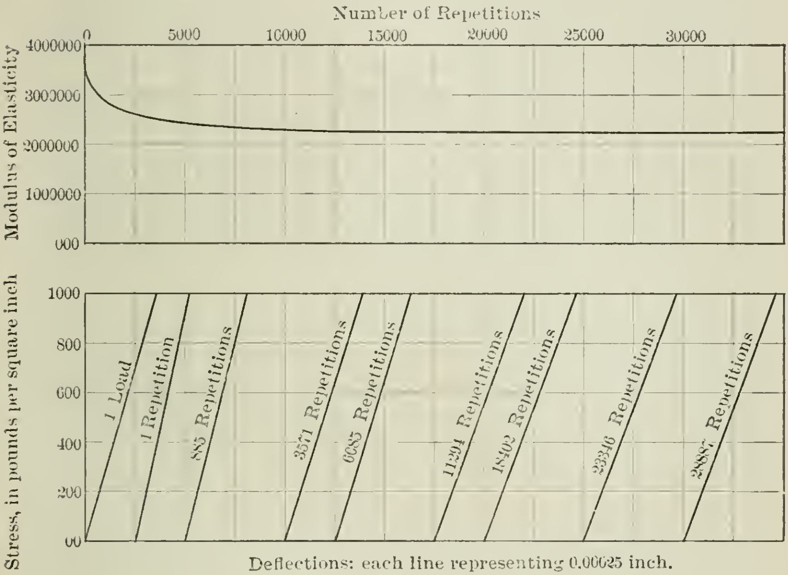


Fig. 7.

one-half load as in this case), to determine whether this two-thirds value of the modulus is general; it is likely that the fraction would be somewhat larger as the percentage of the repeated load becomes less.

In addition to the above-mentioned phenomena, there always occurred a permanent set in all the specimens as a result of the first few repetitions. In the case of the few observations represented by Fig. 7, typifying infinite endurance, evidences of permanent set rapidly disappeared. In the case of the greater number of tests, representing prisms which finally failed under repeated loadings, as typified by Fig. 6, the permanent set became comparatively small during the second stage, but again became more rapid in the third and fourth stages, and increasingly so toward the end.

All these phenomena will assume added definiteness if interpreted according to the theory of resilience. In the case of the specimens subjected to repetitions less in intensity than that load representing what may be called the elastic limit of the concrete (the actual or veritable elastic limit as typified in Fig. 7, not the artificially high one illustrated in Fig. 6, which is sustained for a time but cannot be maintained), the material speedily adjusts itself to the condition which enables it to give back all the work done upon it. When the repeated loadings of the prisms are greater than the elastic limit of the concrete, some of the work done on the specimens is absorbed, resulting in elastic changes that progressively accumulate and necessarily culminate in the collapse of the specimen. This consideration had its due weight in deciding the writer to draw the general curve of fatigue (Fig. 1) tangent to the 50% line.

A considerable number of the reinforced concrete beams were similarly observed, readings being taken to determine the aggregate "load off" and "load on" deflections at different stages of the fatigue test. The results of these observations are illustrated in Fig. 8, which is the plotted representation of the complete results with one of the beams, and is fairly representative, though not typical. Ordinates to the "total deflection" curve represent the maximum deflections when the full repetition load is on, measured downward from the original level of the center of the beam before the first loading was applied; likewise, ordinates to the "permanent set" curve represent deflections, similarly measured, when the beam is relieved of the repetition load. The vertically shaded portion of the diagram represents, then, the vertical range of deflections from the first to the last repetition of the load.

The curves indicate five different elastic conditions of the reinforced beam. The first is where the curves slope downward at the beginning, as the beam is adjusting itself to load conditions, and small tension cracks form. The second is characterized by only a slight downward slope of the lines, indicating a considerable range in which internal conditions are but slightly changing. The third is shown by a rapid downward slope of the curves caused by the progressive breaking of the bond between the concrete and its reinforcement, with the accompanying manifestations of rapidly enlarging tension cracks or diagonal cracks, or both. The fourth is characterized by another nearly horizontal segment of the curves resulting from a second state of nearly perfect elastic equilibrium. The fifth condition is shown by another violent downward slope to the curves, structurally manifesting itself in the progressive breaking down of the concrete in compression, and ending in the complete collapse of the beam.

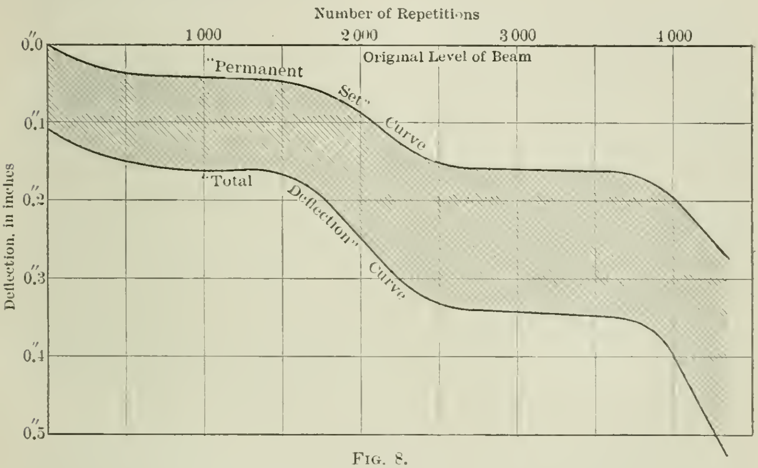


FIG. 8.

These curves should be considered broadly characteristic, but by no means minutely typical. Often, one or more of the five characteristic sections just described is blended with others, as the corresponding structural changes they indicate may be taking place simultaneously. Sometimes these changes overlap so that the resulting curves are much more regular. Occasionally, another change of direction is added, as a new factor of weakness or resistance may manifest itself; and sometimes one of these characteristic conditions of curve-slope may disappear entirely because of the entire absence of its cause. For a beam loaded with less than a 50% load, the first two segments only of the curves exist, the first horizontal portion rapidly becoming truly horizontal and the vertical intercept between the curves not increasing in length as repetitions continue.

Fatigue of Bond of Concrete to Steel.

For these tests, $\frac{5}{8}$-in. square, plain steel bars were embedded in concrete of the previously-mentioned proportions, the bar being placed with its center 1 in. from the tension face of the embedding prism and midway between the sides. These embedding prisms were 4 by 4 in. in cross-section and 15 in. long. These specimens were made with great care, and were very thoroughly tamped. They remained in the moulds for 2 days, were then placed in water for 7 days, and then stored in air for 3 weeks, thus being tested at an age of 1 month.

The machine devised to act upon the specimens consisted of two parallel steel struts, traveling in guides, to which a reciprocating movement was given. Beyond the guides, a metal cross-head was attached to these struts, faced with $1\frac{1}{2}$ in. of oak. In front of this oak face the specimen was clamped into a quite rigid frame, the attachment of the specimen to its independent frame being by heavy screw-clutches clamping the projecting ends of the reinforcing bar just beyond the ends of the concrete. The specimen was thus supported by the metal reinforcement, and a combined blow, pressure, and the accompanying vibration was imparted to it by the machine acting upon the surface of the embedding concrete. The number of blows delivered per minute was 150.

The specimens were set in their adjustable supports so as to encroach upon the extreme reach of the striking head $\frac{7}{100}$ in. Measurement developed the fact that each blow imparted to the specimen about 740 in-lb. of work; of this amount, somewhat less than one-tenth was found to be expended in initial impact, an unknown amount was lost through the beam supports, and the remainder was imparted to the specimen in producing bending stresses, vibration, etc. The effect on the test specimens produced by this treatment, therefore, is seen to be very severe, especially producing those effects which are often thought to prove destructive to the bond between the concrete and the steel; no one watching and hearing the action of the machine upon the test piece could doubt this severity of treatment.

In eighteen specimens the rods were pulled from the concrete without subjection to such preliminary fatigue treatment. For this case the average value of the strength of the bond was 150 lb. per sq. in. of surface of contact, the extreme range from this mean value being about 35%; after the initial rupture of the bond, the weighing beam was again balanced as the bar continued to slip from the surrounding concrete, giving a frictional or sliding resistance averaging 100 lb. per sq. in. of surface.

Of the thirty specimens subjected to the fatigue treatment of the machine, the number of blows given to each averaging 50 000, the average initial unit value of the bond was found to be 125 lb. per sq. in. of surface, and the later frictional resistance, 90 lb. per sq. in.; the range in values was about as before.

The rigorous treatment described reduced, then, the value of the bond somewhat less than 20% in average value, and in no case as much as 50 per cent. However, it is not supposed that this reduction in value would in no case be more than the percentage given, for it might easily be greater or less. It will be noted that the total value of the bond consists of two parts, the adhesion proper and the frictional resistance or "grip" of the concrete to the steel, independent of its adhesion. This frictional resistance is much the larger of the two, the value of the adhesion, proper, apparently being about 50 lb. per sq. in. The "grip" of the concrete upon the steel is particularly dependent upon any effect of change of volume of the enveloping concrete, during both its setting and its hardening; and, consequently, its hygroscopic condition must constitute an important factor. The writer believes that very variable values would result from variations in the treatment of reinforced concrete, especially in connection with the presence or absence of water after it sets.

The same considerations may explain the low value found for the bond in the uninjured specimens, as compared with the greater strength reported by many experimenters, but agreeing more nearly with the lower values indicated by Considère, Hartmann and others. Variations due to proportions of ingredients, age, conditions of materials, and other considerations have received much attention; probably many discrepancies still remaining in values obtained by different experimenters would be explained if the moisture conditions of the specimens, from fabrication to test, were carefully scrutinized in their effect upon change of volume of the concrete as affecting the intensity of bond.

Correlated Considerations.

Inasmuch as neat cement prisms and concrete are definitely affected by fatigue, the inference naturally follows that stone, brick and other engineering materials of the same comminuible character must also be similarly affected. Such scattering tests as other experimenters have published all corroborate this proposition, but results, so far, are too meager to indicate whether or not they present essentially the same law in detail as that herein stated for concrete.

Another natural inference would be, that, as concrete is subject to fatigue through the agency of repeated loadings, it would be affected similarly by a constant, static load of corresponding intensity. Tests of this kind are rare. The writer compressed two concrete prisms with a static load of 90% of their crushing strength, and each of the two specimens withstood this test successfully for a month, with no appearance of failure in that length of time. A slate beam, similarly, resisted a 90% load for about 2 months. A few concrete blocks failed in compression in a few hours under constant pressure of higher percentage. Three such cases are recorded* where stone was the material tested. Because the research work in this particular line is extremely difficult, on account of the time element involved, the writer has thought that the situation might best be indicated by comparing elastic behavior. A reinforced concrete beam thus tested under a 90% load failed in 10 months; its "total deflection" curve is similar to the first portion of a similar curve (the first half of it, but with deflection ordinates about twice as great) of a 90% beam tested by repetitions, as illustrated in Fig. 8. At the present time, only the most general conclusions (and these subject to possible later modification) can be drawn with reference to this subject, which is of some scientific and practical interest. Undoubtedly, such materials are subject to fatigue under a constant load of high intensity. Probably such effect does not exist below a considerable load, such as a 50% load. Apparently, its effects are produced very slowly, as it takes days and perhaps weeks or months to produce an effect corresponding to that of each repetition where the loading is intermittent. An unexpected phenomenon is that

* In Buckley's "Quarrying Industry of Missouri," 1904, p. 299.

(at a certain fixed, constant deflection) the poised weighing beam occasionally rises, instead of (as usual) gradually falling.

Of course, certain facts mentioned in this paper, such as the sequence of the cracks in the reinforced concrete beams, result from the particular details of their construction, and therefore are not general. The writer trusts that reporting such occasional results of limited applicability will in no way obscure the essentially general import of the main phenomena presented. Among the facts developed in these experiments the writer would invite particular attention to the following:

The fatigue of concrete under loads of high percentage is a definite fact; its influence is limited to an intensity of about 50% of the compressive strength of the material, as usually determined. Combining this fact with an allowance of 30% for range in values of compressive strength of concrete of supposed uniform composition and treatment, it is seen that designers of concrete or reinforced concrete must necessarily use a factor of safety of more than three. This consideration does not affect the practice of conservative design; but it does convey a definite warning against the use of working stresses which are too high.

With reinforcing bars having no mechanical bond, the reduced value of the bond caused by great excess of water in mixing the concrete, by rust, scale, etc., is considerable, and, especially, the effect of moisture condition, in expanding or contracting the concrete and so affecting the strength of bond, seems to be a consideration worthy of attention and of more precise and extended investigation.

While the elastic behavior of concrete within the fatigue limit is interesting and valuable in aiding in the more complete understanding of the phenomena of fatigue, the elastic properties of concrete repeatedly stressed to lower percentages is particularly pertinent. The writer's results agree with other experimental results in finding that concrete thus stressed has imparted to it a definite elastic limit, within which stresses are proportional to strains; his results extend this fact in indicating that repetitions indefinitely repeated, if below the fatigue limit, do not alter those propositions. A logical deduction from these facts would seem to be that all formulas for reinforced concrete design, which are intended to give stress values under service conditions, should be based on this hypothesis of direct proportionality of stresses to strains, leaving strictly all parabolic or other curved stress-strain ratios for their legitimate application to conditions existing on the first application of load or at failure.

The tests and results would seem to add a third value for the modulus of elasticity of concrete. Many experimenters have used a value derived from conditions existing at the first application of the load. Others have made use of a value similarly secured after a permanent set had been given the concrete by a previous loading, and sometimes corresponding to the "first repetition" curve of Fig. 7, and giving a numerical value, perhaps 20 or 30% greater than that first mentioned. While it would seem to the writer that, although special circumstances would sometimes require the use of one or the other of the values just mentioned, ordinarily, engineers should use in all formulas and studies for working conditions, where a modulus of elasticity is used, the value to which it is reduced by a great number of repetitions; that is, the asymptotic value as typically indicated in the upper part of Fig. 7, which value may be found to be perhaps 80% of the initial values as frequently derived, or about 60% of the maximum value.

The experiments herein described were made in the Civil Engineering Laboratory of Washington University, during the past three years, and largely as thesis work. Acknowledgment is gladly made of the co-operation of Messrs. Hans Schantl, W. W. Horner, M. Schuyler, C. K. Traber, C. H. L. Cassell, J. M. Bischoff, and S. F. Jones, all former students, to whose interest and work much credit is due.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1046.

UNIFORMITY OF REQUIREMENT AND CLEARNESS OF SPECIFICATION IN AGREEMENTS FOR THE GRADUATION OF RAILROADS.*

By W. F. Dennis, M. Am. Soc. C. E.

With Discussion by Messrs. George L. Dillman, J. L. Campbell, M. S. Parker, Halbert P. Gillette, James H. Kennedy, F. Lavis, T. Kennard Thomson, E. H. Beckler, Harold Bouton, Samuel Tobias Wagner, C. O. Vandevanter and W. F. Dennis.

The construction and betterment of the subgrade of railroads is a large business, affecting the expenditure of hundreds of millions of capital and calling for the services of thousands of engineers and contractors.

A somewhat extended experience in this class of construction, partly as engineer and partly as contractor, has impressed upon the writer matters which, in his opinion, are worthy of consideration in the preparation of agreements for such work. The magnitude of the business, the large number of individuals interested, and the benefit to their business relations which would result from any advance toward uniformity of requirement, increased clearness of specification, and reduction of friction in carrying agreements into effect, are the reasons for presenting this paper, in which some of the matters covered in contracts for graduation will be considered.

* Presented at the meeting of March 6th, 1907.

CLASSIFICATION.

Nearly all railroads find it useful to retain classifications in their forms of agreement. Such classifications give a solid rock material at one end, and an earthy material at the other, with, generally, an intermediate material called loose rock, and frequently an additional hardpan classification, formerly more common than now.

Classifications grew out of a recognition that different materials varied in cost of removal. Each classification was an attempt at segregating material into groups of approximately similar cost. The theory was that, "not being able to see under the ground," if the contractor, in making a proposal for certain work, had to guess at his unit costs for each material, and, in addition, had to approximate the relative quantity of each, he would have to provide against a double hazard. Having a classification, he could guess with some accuracy, from his experience, the cost of the different materials under the given conditions, and eliminate the hazard of their proportion.

Where the experience of the subordinates—who do the actual classifying—of the management, and of the contractors, is fairly well established for a particular section, the theory works out well, with very little friction, and, in the writer's opinion, with cheaper final results to the railroad and more satisfactory results to the contractor—cheaper and more satisfactory because of the elimination of part of the risk.

Twenty-five years ago an unclassified price was exceptional; today much the larger volume of excavation is paid for unclassified. If any tendency can be discerned, it is strongly in the direction of letting all excavation unclassified; or of limiting it to one classification for rock, and another for all other materials.

On the part of the railroads there is the argument that classification is prolific of lawsuits, and there is an unfounded belief that an unclassified price prevents them. Besides this, and outside of all other considerations, the growth of business brought about by consolidations and extensions leaves scant time for investigation, and it is increasingly difficult in practice for the Chief Engineer really to go into or review any classification.

There is also a process of extension of operations and centralization in management in the contractor's business. On different roads he observes classifications which are not exact, even by definition, largely modified by custom, and finds that custom varying with individual engineers in charge, and, from time to time, even upon the same railroad. However exact the contractor's estimate may be, on what he considers a proper classification, he is practically at sea as to what is to be paid. Should he bid upon an anticipated favorable classification, it might be changed during the progress of the work; if he assumes the unfavorable, possibly he may lose the work. In addition, he thinks that he has clearly detected instances where classification was influenced downward by financial considerations, pressure from the management, preliminary estimates being exceeded, and so on.

As a result of these uncertainties, the writer believes that most of the responsible contractors of to-day, in weighing the two hazards —their own both as to proportion and cost of materials, or the other as to what kind of classification they will get from the engineer—generally prefer to take their own risk upon an unclassified price. In making an unclassified price, either from necessity or choice, the contractor not uncommonly finds difficulty in getting bidding information which is sufficiently clear as to quantities and disposition of material. Some engineers present a profile, but refuse to give quantities in detail, being afraid that claims may be made if there be any change. Contractors, to keep their business going, are compelled to bid in the form in which the engineers, their customers, require. Under such circumstances they make the best approximation possible; but when full information is not available there is less competition, and the work goes, not to the best-informed or best-equipped competitor, but to the most optimistic. It is economically wrong to let the work too high or too low, and the engineer is inexcusable who does not secure and present clearly every piece of information that will eliminate the risk of the work as much as possible.

In spite of the apparent satisfaction of both sides with the unclassified price, the writer believes that it is not right in principle, and that paying by a proper classification is, in the long run, the most equitable and reasonable method.

It is worth while to find out if there may not be some position that will shape the theory into a more workable practice. The inquiry can cover only two points: is the average classification sufficiently explicit; and, if not, can it be made more so?

As before stated, the basis and aim of classification are to associate in one group classes of material for which the cost of handling is supposed to be approximately the same. The class, location and magnitude of the work, and the kind of equipment which is best adapted to it, have a very important bearing upon the cost, even of the same material. The association of one material with another on the basis of similar cost, therefore, might be proper in one case and not in another. Confusion from this influence, and attempts at designating material by names and qualities at the same time, cause uncertainty in existing classifications. For work suitable only to scrapers, it is inconsistent to embrace, in the same classification, earth, sand, gumbo and cemented gravel as apparently equivalent. If it is assumed that the work is to be done by steam shovel, many of the items of loose-rock cost for hand work will approximate the cost of an equal quantity of earth.

The company agrees to pay definite prices for definite services in a definite time. The contractor thereupon proceeds to perform the service, providing such appliances as his judgment, particular plant, or financial ability, will justify. If he knows exactly how he is to be paid, from a well-understood classification, or similarly with an agreed unclassified price, he can then make his calculations as to heavy-plant expense and lower working cost, or the reverse, and longer or shorter time. It would be in the highest degree inconsistent if large plant investment should cut down the classification; yet one specification, the writer remembers, classified a material as loose rock if moved by hand, and as earth if moved by steam shovels. If a classification is desirable in itself, this principle seems to be clearly wrong. The classification of a given material should be rigid, for removal by any appliance, and irrespective of quantities or location. If special appliances can reduce cost, competition will make the reduction appear in the price for the classification. From every point of view, the classification should be made, not by names and descriptions of material, but by some definite physical characteristic susceptible of uniform test.

Is it practicable to make a test upon the materials generally found in excavation for public work. As a first criterion, a simple, measurable test, easily applicable, and defining what should be properly in the "earth" classification, is whether or not the material can be plowed in its natural state by a definite plow pulled by a definite number and weight of stock. Whether this material is moved by scraper, grader, cart, car, wheel-barrow, or steam shovel, what is meant is clearly described, namely a material which a designated plow will produce in shoveling condition. This description excludes from the earth classification some material included in some earth specifications, and includes some material which, in others, is classed as loose rock or as hardpan. As will be seen later, earthy material, not included in the "earth" classification, goes to an intermediate classification, for convenience and other considerations, termed "loose rock."

The reason for placing the earthy material, sometimes included in earth and hardpan classifications, in the loose-rock classification, is the obvious one of similarity of cost. If the material is too wet to be plowed, as in case of swamp muck, quicksands and some gumbos; or is too hard to be plowed, like hardpan, cemented gravel, etc., holding to the proper theory of grouping by rough similarity in cost, no designation by name can properly make it "earth," in a cost sense, for all appliances, although it might be for some. Additional costly work may be required to get the material loaded or transported. In some cases the cost of unplowable earthy material may approximate and exceed that of solid rock; but, speaking generally, the cost is somewhat similar to the cost of loose rock, and such material is most fairly included in that classification.

Preliminary to the consideration of a physical test for solid and loose rock, the following definitions have been abstracted from current specifications:

Solid Rock.

New York, New Haven and Hartford.—"All rock or stone containing one cubic yard or more." (All other material is earth.)

Erie.—"Rock in masses exceeding one cubic yard, which cannot be removed without blasting."

Pennsylvania.—"Rock in masses exceeding one cubic yard, which cannot be removed without blasting."

Baltimore and Ohio.—"Rock in solid beds or masses, which may be best removed by blasting."

Chesapeake and Ohio.—"Rock in ledges and detached masses exceeding one-half cubic yard, which may best be removed by blasting."

Norfolk and Western.—"Rock in masses which may best be removed by blasting."

Southern.—"Rock in masses of more than one cubic yard, which may be best removed by blasting."

"Big Four."—"Stone in solid masses or ledges."

Chicago, Burlington and Quincy.—"Stratified rock weighing more than 140 lbs. per cubic foot, which can only be removed by blasting."

Chicago and Alton.—"All stratified rock and rock occurring in masses which can only be removed by blasting * * * must ring under hammer."

Great Northern.—"Rock in place, in removing which, it is necessary to resort to drilling and blasting."

Atchison, Topeka and Santa Fé.—"Rock in solid beds or masses in its original or stratified position * * * other material which can be removed without continuous drilling and blasting, but which is as difficult * * * as solid lime or sandstone."

Illinois Central.—"Rock in solid beds or masses in its original position * * * which may best be removed by blasting." (Everything else classed as "common excavation.")

Northern Pacific.—"All rock in masses that cannot be removed without drilling and blasting."

Missouri Pacific.—"Rock in solid beds or masses, in its original position, which can only be removed by continuous blasting."

What is "rock" and "stone"? Notice the following definitions:

Standard Dictionary.—*Rock.*—"The consolidated material forming the crust of the earth * * * not excluding beds of clay or sand * * * a rock may consist of one mineral species, as limestone, or of several intermingled, as granite * * * massive rock, a rock that does not exhibit foliation or schistose structure."

Stone.—"A small piece of rock. Rock as a material, a piece of rock shaped for a specific purpose. Synonyms, boulders, cobble, mineral, gem, pebble."

Century Dictionary.—*Rock.*—"The mass of mineral matter of which the earth, so far as accessible to observation, is made up; a mass, fragment or piece of the crust, if too large to be designated as a stone. The unconsolidated stony materials which form a considerable part of the superficial crust, such as sand, gravel and clay, are not commonly designated as rock or rocks; the geologist * * * includes under the term rock * * * all of the consolidated materials forming the crust, as well as the fragmental or detrital beds which have been derived from it."

Stone.—"A piece of rock. The name rock is given to the aggregation of mineral matter of which the earth's crust is made up. A small piece or fragment of this rock is generally called a stone."

Webster's Dictionary.—*Rock.*—"Any natural deposit forming part of the earth's crust, whether consolidated or not."

Stone.—"Concreted, earthy or mineral matter * * * also any particular mass of such matter. In popular language, very large masses of stone are called rocks; small masses are called stones; and finer kinds, gravel or sand."

Gillette's "Rock Excavation."—"Rocks are aggregates of one or more minerals, or the disintegrated products of minerals."

These definitions do not help to clear up any uncertainties there may be in railroad classifications.

Loose Rock.

Erie.—"Slate, shale or other rock which can properly be removed by steam shovel, pick or bar, without blasting, although blasting may be resorted to on favorable occasions to facilitate the work * * * detached masses, 3 cu. ft. to 1 cu. yd."

Pennsylvania.—"Stone and detached rock lying in separate and continuous masses containing not over one cubic yard; also all slate or other rock that can be quarried without blasting, although blasting may be occasionally resorted to."

Baltimore and Ohio.—"Slate, coal, shale, soft friable sandstone and soapstone, detached masses 3 cu. ft. to 1 cu. yd."

Chesapeake and Ohio.—"Shale, slate, ochre, which can be removed with pick and bar, and is soft and loose enough to be removed without blasting, although blasting may be occasionally resorted to. Detached masses 3 cu. ft. to 1 cu. yd."

Norfolk and Western.—"Shale, soapstone, and other rock which can be removed by pick and bar, and is soft and loose enough to be removed without blasting, although blasting may be occasionally resorted to. Detached masses 1 cu. ft. to 1 cu. yd."

Southern.—"Soapstone, shale and other rock which can be removed by pick and bar and is soft and loose enough to be removed without blasting, although blasting may be occasionally resorted to. Detached stone 1 cu. ft. to 1 cu. yd."

"Big Four."—"Shale, coal, slate, soft sandstone, soapstone, conglomerate stratified limestone in layers less than 6 in.—detached masses 3 cu. ft. to 1 cu. yd."

Chicago, Burlington and Quincy.—"Stratified rock which can be removed by pick and bar weighing more than 140 lbs. per cu. ft. Detached masses 3 cu. ft. to 1 cu. yd."

Chicago and Alton.—"Stratified rock which can be removed by pick and bar * * * and masses between 3 cu. ft. and 1 cu. yd."

Great Northern.—"Slate and other rock, and loose enough to be removed without blasting, although blasting may be occasionally resorted to—detached masses 3 cu. ft. to 1 cu. yd."

Atchison, Topeka and Santa Fé.—"Hard shale or soapstone * * * in original or stratified position, boulders in gravel, cemented gravel, hardpan * * * and other material requiring * * * use of pick and bar or which cannot be plowed with 10-in. plow and 6-horse team."

Illinois Central.—(No loose rock. Everything but solid rock classed as common excavation.)

Northern Pacific.—"Slate, soft sandstones, or other rock than can be * * * removed without blasting * * * detached rock between 1 cu. ft. and 1 cu. yd."

Missouri Pacific.—"All rock * * * which requires for its removal steam shovel or pick and bar, without blasting, although blasting may be resorted to at the option of the contractor. Detached masses 1 to 18 cu. ft."

A composite view of the several descriptions of rock and loose rock would reduce to about this: Rocky material which can be removed without blasting is loose rock; and that which cannot is solid rock. That word, "can," is the whole of the question, the uncertainty of the answer to which causes most of the disputes about classification. The word "can," is complicated by the fact that it is nearly always cheaper to blast so-called loose rock than pick or bar it apart, and, further, undeniably solid rock can be disintegrated by slow degrees without blasting. The word "can," then, for reasonable interpretation, depends upon some rate of removal. A solid mass of rock might be seamy to the extent of permitting wedging or barring out 1 or 2 cu. yd. per day, but that possibility does not make it loose rock. On the other hand, a certain material might be dug slowly by a steam shovel without blasting. If it could be loaded faster, as a result of blasting, it would be taken out customarily by blasting, but, under the specification, that fact would not necessarily make the material solid rock. On the other hand, the same material, under nearly all specifications, might be solid rock by reason of the impracticability of removing it with pick or bar with reasonable rapidity.

Taking a general view, the difference between materials in a construction sense is obtained by the writer from consideration of the operations necessary in loading such material on their transporting appliances. Earth is a material which can be reduced to loading condition by plowing, or equivalent inexpensive picking or blasting. Loose rock is a material which generally can be put into handling shape by picking, barring and light sledging, or, in lieu thereof, by moderate blasting, but it is not quite as easy to load as earth. Solid rock is a more refractory material, requiring drilling, strong explosives, and general sledging, and, with this additional expense, is not capable of reduction to a loading condition as favorable as the other materials. In seeking for some limit between solid and loose rock, it is impracticable to fix any rate of drilling or quantity of explosive. The condition after blasting cannot be taken as a guide, because that will depend upon the foregoing, as well as upon the cohesion of the material. The requirement of ringing under the hammer is inapplicable; the expression, "continuous blasting," is inexact, and can be worked as a swindle by either side.

Can a physical uniform test be applied? It is known that certain soft or fractured rocks can be picked or barred apart with reasonable rapidity, and customary specifications state the fact, but do not state the rate. By definition of that rate the classifications of rock can be clearly defined. The writer thinks that, keeping close to current practice in classification, the rate of disintegration for loose rock should be within the performance of two men thus employed. A material requiring more than two men working with pick and bar to keep one shoveler busy is certainly a material that "may better be removed by blasting" and which "can only be removed by blasting," in a reasonable sense.

It may be objected that this criterion would throw some material, occasionally understood as loose rock, into the solid-rock classification. The writer does not think this would be the case, if the ordinary "loose-rock" specifications mean what they say. But, if so, the point is, that the cost of material under any classification is not a fixed cost, but varies with many factors. For instance, a material unquestionably solid rock, under certain circumstances, might be loaded for 40 cents per cu. yd.; under other circumstances, identically the same rock, physically and chemically, might cost $1, $3, or more, per cu. yd. Fixing a standard by which the contractor can know clearly how the material will be paid for is not going to make the price for the classification. That price will be made on the average cost of the materials included, as modified and governed by the specific conditions of place and removal, embracing the whole subject of quantity, appliance, situation, credit and labor.

A consideration of importance is the size of the rocky mass that must be exceeded in order to constitute a solid-rock classification. In hand-work an isolated mass of 3 cu. ft. can be handled without much difficulty; but any larger mass will require disintegration before loading. The expense of this disintegration per cubic yard will be higher than that for disintegrating masses of the same material which, under any size limit, would still be solid rock. In steam-shovel work very considerable masses can be loaded without disintegration, and, consequently, without much real extra expense. An objection to a small size limit would be an apparent necessity for more particularity of measurement. As to that, the separate quantities in mixed material, in practice, are approximated percentages, and are as easy to calculate with one size limit as another. Bearing in mind the theory of trying to fix classification by similarity of cost, the writer thinks that 1 cu. yd.—the limit most frequently specified—is too high; 3 cu. ft., although right in one view, is probably too low; and that the compromise limit of ½ cu. yd. would be about right. This limit was formerly common, and is still retained in some specifications.

In an endeavor to set forth the foregoing more clearly, the writer proposes the following as an outline classification:

Excavation, excepting foundation pits for structures, elsewhere classified separately as foundation excavation, shall be either classified or unclassified, as may be determined at the time of the contract. If classified, the following classification shall apply:

Earth.—Material which in its customary natural condition can be plowed—or is equivalent to a material which can be plowed—with a plow cutting a furrow 10 in. wide and 10 in. deep, drawn by a team of 4 horses, or mules, each having an average weight of 1 100 lb., and moving at a reasonable plowing speed, shall be classified as earth.

Loose Rock.—The following shall be classified as loose rock: Earthy or mixed materials, not susceptible of plowing under the foregoing test; soft, fractured, disintegrated or other rocky material, soft or loose enough in its natural condition to be barred or picked apart by two men thus employed serving one man shoveling or loading by hand; solid rock in separate masses exceeding 1 cu. ft. each, and not exceeding ½ cu. yd. The continuous or occasional use of explosives, at the contractor's option, shall not affect the classification, but it shall be governed solely by the test above set forth.

Solid Rock.—The following shall be classified as solid rock: Rocky material in masses exceeding ½ cu. yd., which cannot be broken apart, or displaced from its natural position, except by the use of explosives; and other rocky material which cannot be picked or barred apart by two men thus employed serving one man shoveling or loading by hand.

Where any excavation contains material of more than one classification, the relative percentage of each shall be determined by measurement and observation during the progress of the work.

The principal point aimed at is clearness of test and classification. Experiment will determine whether or not the material can be plowed, and actual picking or barring of the material will determine whether or not two men can keep one shoveler going. Classification becomes, not a matter of opinion, but of demonstration. As illustrating the application of these tests to bidding: if, under the test, the principal earth material upon a piece of work could not be plowed, and, therefore, would be classed as loose rock; and if the intention was to move it by steam shovel; and if competing contractors considered that with their appliances there was little difference in cost between the given material and earth, they would bid about the same price for loose rock and earth. Under other conditions they might justifiably find that the price of the given material had to be two or three times that of plowable earth, and bid accordingly; the classification, in other words, does not make the prices. Rigidity of classification simply makes clear how a given material is to be paid for. With this information, the price for the material can be made intelligently by the contractor in view of his experience with similar material and his knowledge of local conditions.

For heavy steam-shovel work, the writer's opinion is that there is no especial benefit in a distinction by classification between loose rock and earth, and, for that class of work, a classification for solid rock and another for all other material would be sufficient; but nearly all steam-shovel work involves more or less miscellaneous accessory work for team and hand appliances, and the loose-rock classification is needful for them; furthermore, classification on all work would become better established by its uniform practice.

Foundation Excavation.

Almost universally, specifications provide payment for excavating foundation pits for structures. Formerly, these were generally paid for on a percentage basis; but there was frequent opportunity for dispute, mainly on account of differences of opinion as to the kind and extent of appliances to be furnished and the proper compensation for them.

Some specifications provide for payment for the material in coffer-dams as well as for excavation. Here the dispute comes as to the necessary quantity of excavation and the form, material and size of the coffer-dam, all of which are under unspecified control. Other specifications settle this by requiring the engineer to make plans for the coffer-dams. This is impracticable on account of delay, and throws the responsibility on the engineer, who is not generally as well qualified as the contractor to make an economical design. The most frequent custom, and apparently the best, is to leave the material and design of the coffer-dam or support to the contractor, make him responsible and pay by some definite stipulation. Past experience should give the contractor a fair idea of the cost of ordinary foundations, and the conditions of more important emplacements can be studied before bidding; so that, on the average, there should be sufficient information in sight to make a fairly reasonable estimate. Experience shows that the reliable unit to be used in computation is the cost of the excavated material per cubic yard. Specifications do not generally define the quantity of material which shall be measured. The following seems to the writer to be reasonable practice and definition:

Excavation for foundations of pipes, masonry, or other structures, shall be classified as foundation excavation under the following heads:

Dry-Foundation Excavation.—Material of whatever nature, excepting solid rock, found above water level;

Rock-Foundation Excavation.—Material, elsewhere defined as solid rock, found above water level;

Wet-Foundation Excavation.—All material below water level.

By "water level" is meant the average or mean level during construction at which pumping or bailing becomes necessary in the work of excavating. The quantity of wet excavating shall be computed as a prism having a height equal to the distance between the average level of the bottom of the foundation pit and the water level and a base equal to the area of the foundation course plus 4 ft. all around. The dry and rock excavation quantities shall be computed on a base equal to the bottom area of the wet excavation as above defined, with the necessary slopes to the natural surface.

Wet excavation shall include the cost of excavating, piling, coffer-dams, pumping, bailing, leveling off the bottom, and the expense, of whatever nature, necessary to complete the foundation pit from low-water level to the level finally determined for the bottom, and to maintain the foundation pit open until the structure shall have been placed therein, not, however, including the placing of iron, timber, or piles, in permanent artificial foundations, these items being paid for under a separate schedule elsewhere described.

The prices for all classes of foundation excavation shall include the cost of removing the spoil, and depositing it in adjoining fills, or of wasting the spoil, if such deposit in fills be not required by the engineer; and also the cost of removing such portions of coffer-dams as the engineer may require, for appearance or for reducing obstruction to the waterway.

Measurements.

Every now and then textbook misinformation is seen in attempts to measure excavation by the prismoidal formula. For that formula to apply strictly, every point of the surface between the end cross-sections should be capable of generation by a straight line moving on the two end areas. No natural ground corresponds exactly to this theoretical requirement. The middle area is the dominant factor in the calculation, and how may this be obtained correctly? If not cross-sectioned, it must be computed from the end-area dimensions, and this calculation produces an area which is entirely imaginary and certainly out of place in a rigid formula. If a middle area is measured, thereby doubling the number of cross-sections, it will still have to be demonstrated that the solid is a correct prismoid. Using each section in turn as a middle area and halving the totals is theoretically preposterous. The ordinary measurement, by averaging the end areas, cannot, in Nature, represent the mathematical mean area. Neither this nor the prismoidal formula, therefore, gives absolutely rigid measurement. Decision must be made as to which approximation is to be taken. Averaging end areas is simple, quick, and natural in principle, easier to apply, has the sanction of custom, and is on the whole fairly exact. Cross-sections for any formula have to be located by the judgment of the field engineer. Any reasonably experienced man will locate sections sufficiently close together on rough ground to reduce the theoretical error of the end-area computation to a small percentage. The writer has a strong feeling that, in staking out work, the ordinary record of the measurement will vary from the accurate delimitation of the true solid more than the difference that will appear by juggling the record between the two methods of computation.

In the absence of a precise specification for the use of the prismoidal formula, custom would legally govern the method of calculation. Even in the event that the prismoidal formula was specified, the writer does not believe it would hold, because it could not be demonstrated to be mathematically correct in its application to construction cross-sections. With a specification calling for what could be proved to be an incorrect method of measurement, the customary method, it is believed, would control in any controversy. Using the prismoidal formula in the computation of construction quantities is an attempt at substituting something apparently ultra-scientific for a very simple computation. The use of "corrections" to approximate end-area computation to the prismoidal formula appears to be still more objectionable, not having the alleged rigid accuracy of the prismoidal formula, and, as far as the writer can see, requiring nearly as much extra work, with very free chances for error.

Right or wrong, the kind of quantities produced by averaging end areas is the kind sanctioned by custom, and in some States required by law. Any use of the prismoidal formula in construction computation is unwarranted as a matter of right, and very wrong as to policy. The contractor will surely put it down to "skinning," although the only motive may be a desire to be more scientific than others.

Extra Items Included in Prices for Excavation.

In the cost of excavation many requirements are frequently included which do not properly belong there, such as fencing right of way during construction, maintenance of road crossings, temporary bridges, saving and piling rock, opening "horseback" roads, taking material across streams, rip-rapping slopes, benching embankments, spreading material in layers, packing around masonry, and, worst of all, including the cost of foundations in the price of masonry. All these matters cost money. They may or may not be done. Quantity and cost are indeterminate.

The specifications appear to assume that their cost is considered and added to the excavation prices; but the quantity of excavation is rarely known accurately in advance, and, generally, the extent and amount of the extra requirements are absolutely unknown. Under this theory, therefore, a portion of the excavation price is for something which does not belong to it. The engineer, therefore, is either paying for something which is not done, or not paying properly for it, as the case may be.

The general rule of business should apply here. Where a definite thing is to be done, it should be paid for directly. Matters problematical in extent should not be called for, except with the promise of definite reimbursement. One matter should not carry the cost of another matter, unless the latter be uniformly a concomitant of the first and is specified so definitely that its quantity and cost can be calculated. A simple, direct way is to pay stipulated prices if these matters are wanted; or, in lieu of that, to pay for them as extra work.

The current objection to settling these matters as extra work is strongly ingrained, but the authority for them being granted, where is the real difficulty? The argument will be made that the general office has to pass upon an extra bill which cannot be checked. In answer to this, it would seem reasonable to claim that the construction engineer who is reliable enough to classify and measure quantities running to tens and hundreds of thousands of dollars would be competent to investigate correctly occasional bills running to a few hundreds, and that his signature should settle the account.

The human nature side is that the contractor, when he does these things, sees that he has to do them without direct compensation. In preliminary cost estimates, no such accuracy is possible as to have these matters valued correctly in advance. If that view be correct, the engineer, by paying these extras direct, would get his excavation price lower.

A liberal policy in all these matters, as against a rigid requirement, will affect the aggregate of the contract to a very small percentage; but it will eliminate much useless friction and dispute, and the engineer will benefit his company in the long run by establishing a reputation for liberality and fairness at a very small temporary expense and a real saving.

Making Embankments.

The survival of an error which once gets into print is well illustrated by the frequent specification: "Embankments must be carried up in layers of 2 ft."; "materials shall be deposited and distributed as the engineer may direct." The writer has known two instances where inexperienced resident engineers attempted to enforce the requirement, but has never yet seen a railroad fill made in 2-ft. layers. The absurdity of such a requirement is well illustrated by considering steam-shovel work. It is believed that this requirement, in actual expense and delay, would mean that the dumping expense alone on heavy earthwork might cause an extra cost of more than the contract price. If the question was between the ordinary method and this requirement, would the engineer with such a specification feel warranted in paying the extra cost and carrying out the requirement, from any supposed advantage to be thereby obtained? A similar question arises as to the segregation of materials. It is absolutely impracticable, under working conditions, to do anything else than take the material and dump it as it comes. Where top or slope rock dressing on fills is called for, it can hardly be gotten, as a general proposition, except by dumping selected material and reloading. Would any engineer be willing to arrange all his work with the top dressing and pay the cost as an extra? These requirements are not generally enforced, and competition compels the contractor to disregard such problematical expenditures, even though the right to them be specified clearly. Every now and then the requirement is enforced, and it is a clear case of the contractor doing extra work for nothing.

These requirements in railroad construction are for things absolutely without any practical value; they are unmeaning when not enforced, and a dead expense when required, and should be eliminated. On the occasions when such requirements are really necessary, the fair way is to pay for them, and not try to get them done for nothing under a blanket clause.

Overhaul.

The remarks as to specific compensation apply partially to clearing, grubbing and overhaul, but the latter are more definitely ascertainable by the contractor before bidding. On new lines, generally, there is often insufficient information and time for minute examination. Relocation of the line and grade frequently destroys any calculation. With no price for these items, where is the equity between the company and the contractor where extensive changes are made during construction? As an extreme case, suppose a steam-shovel section 5 miles long, with no overhaul clause; suppose the location of the borrow is not shown at the letting; suppose that available borrow exists, with a high-priced or unobtainable right of way near a fill at one end, and that a long-haul borrow, with no cost for land, is available at the other. Has the company a right, in saving right-of-way expense, to force the contractor into hauling, say 4 miles, under a stipulation of no payment for changes or for overhaul?

The overhaul is like the unclassified price—legally enforceable only upon a specific line shown to the contractor at the letting of the work. Speaking in a general sense, any hardship that comes upon the contractor, from changes, will legally release him from the contract, unless they are adjusted to his satisfaction. With no overhaul specified, there is no base agreed upon for change.

The writer admits that there is complication in the overhaul question, arising from what is a proper overhaul distance to specify. A 1 000-ft. overhaul on steam-shovel work may be immaterial, but it is a very serious matter for team haul. The difference could be taken up by specifying that locomotive overhaul price shall apply beyond a certain distance, and on team overhaul beyond a shorter distance. On work which, by reason of location, small magnitude, or for other cause, naturally requires hand or team appliances for excavation, the overhaul price would be made for these appliances, and would generally be high enough to require wasting and borrowing before the proper limits of steam-shovel haul were reached. Where steam shovels, wholly or in part, are anticipated, the overhaul price will come down to such an extent that there will be no controversy as to appliances, because it will be uneconomical to make any team overhaul for the price of locomotive overhaul. In sections where scrapers are the ruling appliances, specifications give properly low limits for the commencement of overhaul—300, 500 and 600 ft.; 1 000 ft. is common elsewhere, and, occasionally, 1 500 and 1 600 ft. For team appliances, 1 000 ft. is too high; 1 500 ft. entirely unreasonable. The nonsensical stipulation, of an overhaul after an average haul on the section or contract, has about disappeared, so that nothing need be said in criticism. For steam-shovel overhaul, there is no standard of custom. Some roads pay under the general overhaul; some include it in the yardage price. Practically, it is a question of track cost as one item, variable per cubic yard, with the yardage per unit of track; of maintenance-of-way; and of the point where extra trains become necessary. The latter distance is dependent upon factors not connected with length of haul, facility of dumping, and facility of loading and speed.

To have more than one overhaul price or distance might be conducive to conflict between the contractor and the engineer as to the proper equipment for the work. Inasmuch as the contractor, in making the proposition, must have in mind the appliances by which he expects to move the material, he should have the ability to adjust differences and work out an overhaul price that will represent the average. The handicap to this is the customary lack of full bidding information. Whichever view is taken, it is not of great importance, provided the overhaul principle is accepted and a clear description is given of how it is to be calculated.

There is considerable difference in the practice of estimating overhaul, and confusion as to just what it means. The customary meaning of overhaul is, that every yard of material moved, over and beyond the free-haul distance, is subject to the contract allowance for overhaul. This carries the implication, as is also the common understanding, that the free overhaul distance shall be measured on the route actually taken by the hauling appliance used; that the route be of grades customary for such appliance; and that the appliance be appropriate to the particular case. In the event of, say, a 100-ft. cut into a 100-ft. fill, it can be seen that the question of proper grades and appliances might become very complex. To avoid controversies, the method of measuring overhaul should be stated clearly so that the contractor, in bidding, could cover the ordinary hauls, as well as the special hauls called for by exceptional conditions. The simplest method would be to locate the stations of free haul, and then deal with the cut and fill on either side by locating the station of the center of mass of each. These points could be identified and understood during construction, and the overhaul computation could be checked at any subsequent period. Unless the method of calculating the haul be defined, the common usage will prevail, the effect of which is that, at a later date, no man can check another, or even himself.

Clearing and Grubbing.

There is nothing to be said specially on these items; as a business policy, they should be paid for specifically, on the ground that they have no uniform relation to grading.

Sub-Contracting Work.

It is as usual to see agreements specify that no work is to be sublet as it is to see, in the execution, sub-contractors more or less peacefully performing their assigned tasks. What is the purpose of the prohibition? It is generally a dead letter in the enforcement. The writer cannot think that experienced engineers consider it a system of double profit. If it is, is not the contractor, having obtained his work in competition, entitled to share his profit with the sub-contractor, if he is so generously inclined? The sub-contractor exists because his services in detail, as foreman, manager, or contractor, coupled with his financial interest in results, and his special plant, are at times more efficient than can be obtained in other ways.

If sub-contractors are eliminated entirely, how would the contractor class be recruited? Most successful contractors develop as sub-contractors before they qualify for a larger field. If the prohibition, except on consent of the engineer, exists, and the sub-contractor is there, in any controversy that may result, the engineer's knowledge of that fact, the giving of instructions, or any form of recognition, will be as binding upon the company as a formal written permission. The inclusion in the contract of a prohibition to be waived in ordinary practice does not safeguard the engineer in the event of litigation. The writer believes it to be against the interest of the company to try to exclude subletting. The only objection that can be brought seriously against the practice is in case of failure, or litigation on account of the sub-contractor; but, in this case, under any customary contract, the company still holds the contractor for the financial end.

The most reasonable view is not to prohibit a perfectly business-like arrangement, but frankly admit it, and, in admitting it, control the form. A stipulation that would seem to cover the whole case would be to require the sub-contractor's agreement to read something like this:

"The company and the contractor shall have all rights and power as to the sub-contractor, that the company has as to the contractor, in its contract with him; and the work assigned to the sub-contractor shall be done subject to all the conditions stated in the specifications of the company, or embodied in the contract between the company and the contractor."

In practice, the sub-contractor's agreement would be a duplicate, except as to principals and price, of the agreement between the company and the contractor, with the foregoing provision added.

As the more successful sub-contractors become contractors and then competitors, the only people really interested in opposing them are the contractors themselves. They might be said to be opposed to subletting work as a principle, but in actual practice they find that, for portions of their work, the arrangement is the least costly and most efficient method of producing a certain result.

If the engineer proposes clearly and firmly to prohibit any form of sub-contracting for an individual piece of work, consideration of policy on the part of the contractor would generally prevent serious opposition, because, no matter what his rights or desires might be, the engineer would have the power to withhold future invitations. On extensive constructions of 20 or 100 miles of railroad, or more, necessitating simultaneous construction from a number of camps, there is no price obtainable by the engineer as advantageous to the company as that which he will get from contractors competing with the expectation of sub-contracting considerable portions of the work. The engineer, by letting a great many small contracts, may think that he will get a lower price; but those engineers who try it do not generally endorse this method.

Contractors have a notion that a contract is a sort of property; that they have the right to make performance by the most suitable method; and that they have as much natural right to make payment by quantity as by time, for services of individuals or groups of individuals. Some of these rights appear to be pretty strong. Sub-contracting occasionally gives the engineer some personal trouble—it rarely gives his company any financial trouble—where there is a sub-contractor there is generally an economic reason. All this means that enforced prohibition of all sub-contracting by everybody will raise the price of work, in the long run; and the engineer cannot justifiably take the position that, in order to avoid some detail and the possibility of annoyance, he may take the risk of increasing the cost of the work. The point will be made clearer by considering the architects: Their clients have chances for liens, suits, etc., to a far greater extent than railroad companies have from sub-contractors, yet it is well known that the average building has as many sub-contractors as there are trades represented; and they are there because that is the cheapest arrangement for the owner.

Right of Way.

One of the scandals of railroad construction is the absolute disregard of plain business methods in the acquisition of right of way, and the consequent unfair saddling of useless expense on the contractor. It is a notorious fact that, in general, the construction contract is let with little or none of the right of way secured. The contractor sometimes gets on the ground before the right of way agent. That agent's interest is to get right of way for the least money. He may consider that a $20 difference may justify withdrawal for a month, or throwing the case into court in order to sustain his prestige. He is held responsible for the results, has a hard job, and must play all the politics he knows. From his standpoint, his action may be right. Suppose the contractor is held up at a critical point of the work; the other's right is not right for him. His contract says he must wait the other's convenience, whether he likes it or not, and shall lose the time and expense of waiting. This is not a fancy picture, but a daily occurrence.

Many engineers allow themselves to be hustled into starting work with immature plans and without right of way. They would have an easier time, save money for everybody, and do the right thing, if they consistently refused to let contracts until these matters were thoroughly in hand.

Insurance.

Agreements generally contain amplified stipulations that construction must be maintained by the contractor until the final acceptance by the engineer. This seems fair enough on its face, but the customary wording does not make equitable distinctions. It is easy enough to see that if the contractor, in such an act as transporting material to the work, suffers loss or damage of that material before it is put into the structure, the company has equitably nothing to do with the damage.

Further, it is not unreasonable that a structure, such as a building, completed in advance of other work, all subject to acceptance at one time, should be insured by the contractor. The question becomes somewhat involved in the instance of, say, trestle-work which has been finally completed, in the event of destruction from a cause, not the fault of the contractor, before acceptance of the work as a whole. For railroad embankments washed away by water; for the destruction of masonry already laid according to specifications; for subsidence of embankments and collapse of slopes, and re-dressing necessary from that cause; and, for similar matters, it is clear to the writer that there is no equitable reason why these risks should be forced upon the contractor. If the contractor places the material where he is directed, no stretch of imagination could make it fair for him to be responsible for destruction by the elements when he has no control over the selection of the location or plan.

As illustrative of the way in which equities are destroyed by such clauses, the writer recalls a case where a retaining wall which was too light fell down when the embankment was placed against it. The wall was rebuilt, payment for it denied, and the contractor sued. He established the fact that the wall fell because of improper design, and that the workmanship was according to contract. He lost on the ground that his contract was not only to build, but to maintain until acceptance, and his remedy would have been to have notified the engineer that the design was insufficient and to have made formal protest to building it.

The writer recalls a personal experience in building a 30-ft. arch. This arch was built on a cemented gravel foundation which was perfectly satisfactory to the engineer and would have been all right ninety-nine times out of a hundred. The side-walls and four cross-walls were carried well down, and the bottom was paved with heavy stone on edge, well laid. The whole work was built under continuous inspection, and the destruction hereafter referred to revealed no fault of workmanship or material.

This arch was accepted on a Friday. In the meantime the fill had been completed for part of its width over the arch, damming the small valley except for the arch opening of about 25 by 30 ft. On the following Tuesday, a cloudburst occurred, and the arch ran full and discharged as a pipe. The paving was dug out by the current, then the gravel was eroded, the side-walls fell in, and the disrupted structure was spread over three acres of ground.

The first question asked by the management was whether or not the contractor was responsible under the "insurance clauses." This was disposed of by the technicality that the arch had actually been accepted a few days before. Suppose the accepting engineer had been delayed on Friday and calculated to get there on Tuesday, instead; then a strictly technical construction of the agreement would have involved the contractor in replacing 3 000 yd. of masonry at his own expense. What had the contractor done, or not done, that could have made any real change in the equity of his position?

According to the teaching of the case first cited, the contractor's legal duty was to have formally notified the company of what was not a fact—that the foundation was bad—and to have refused to build on the accepted foundation, except under protest. It seems to the writer that the simplest investigation of how unfairly these clauses can work out is amply sufficient to show the necessity of eliminating most of them and very carefully restricting the others.

Indemnity.

All agreements are elaborate in provisions to save the company harmless against any acts of the contractor during construction. These provisions are proper, provided the contractor is legally liable; they are wrong where the company withholds payments to force the contractor into settlements, whether proper or not. Many agreements recite that in the event that the company is a party to the action, the company may settle without the consent of the contractor and charge against him, together with such legal expenses as the company may choose to fix as its cost of defense. If the contractor, as an individual, is sued jointly with another individual, he must defend himself as best he can. There is no method by which he can recover the expense of defense, either against the party who sued or on account of whom he is sued.

The writer can see no fair ground why the company should be more favored in the case of construction contracts than in any of its other agreements where it takes its chance according to the general law. The fair principle is that, if the railroad pays the contract price, it should get the goods; if the contractor incurs liability, then, by agreement, he should indemnify the company against loss, and the obligation should end there. In accepting the contractor, the company should consider his ability to save it harmless, just as it considers and accepts his ability to perform the work. If the company is sued jointly with the contractor, it may either let the contract or defend, looking to his satisfying any decision, or, if it elects to defend also, it should do so at its own expense, as it would in any other action.

Power and Authority of the Engineer, and of the Company.

The following extracts are from current specifications:

New York, New Haven and Hartford R. R.—"Right to change line or grade. * * * Should change affect cost of doing work, engineer shall determine price to be paid either above or below, so as to do substantial justice between the parties. * * * Work to be done under direction of engineer; his decision of the true construction and meaning of specifications and drawings final. * * * Contractor shall not sublet. * * * Company reserves the right to suspend, to close and settle up. * * * The cancelling shall not entitle the contractor to any claim for damages. * * * Should contractor be delayed by other contractor, by stakes, damage by fire, etc., time shall be extended."

Erie R. R. and Baltimore and Ohio R. R.—"Quantities and disposition of excavation may be changed by alteration in grades, curves, or alignment. * * * If by such changes work is made more or less expensive, contractor shall have such allowances or deductions therefor as engineer considers just. * * * Contractor shall commence, prosecute and complete under direction of the engineer, who shall have power to direct the application of the forces; to change beginning and ending point of sections; to order the forces increased or decreased. * * * If methods or appliances appear inefficient, engineer may order the contractor to increase and improve. * * * Should right of way not be procured where contractor desires, he shall distribute his forces at other points without any claim for damages. * * * Contractor shall not sublet. * * * Engineer may make allowances he deems just for loss or damage to the contractor, resulting from right of way or other delays of any kind occasioned by the company. * * * Engineer shall have the right to regulate wages. * * * Contractor agrees to do work to satisfaction of engineer. * * * The classification of all excavation, masonry, etc., shall be made by the engineer; decision in regard to same shall be final and binding. * * * The company shall have the right at any time to suspend or annul the contract; such shall not give any claim for damages."

Norfolk and Western R. R.—"In case delayed through negligence or incompetence of a contractor for other work, no claim against the company shall be made. * * * When this agreement is completely performed, performance accepted, certified in writing by Chief Engineer, final estimate shall be paid. * * * Such final estimate shall be conclusive unless reversed or modified by the President of the company upon appeal by either party. * * * Contractor shall not assign or sublet * * * Right to make any alteration in location, line, grade, plan, form or dimensions. * * * No claim for damages, on any ground whatever. * * * No compensation to contractor for hindrances and delays, but extension of time, provided contractor gives notice in writing. * * * Company reserves the right to suspend, terminate or restrict. * * * No claim for consequential damage or anticipated profit or damage of any kind. * * * All questions, differences or controversies in any way whatever pertaining to said work shall be referred to Engineer, and his decision shall be in the nature of an award, final and conclusive upon both parties, unless reversed or modified by the President, and compliance on the part of the contractor with every decision of the Engineer, shall be a condition precedent to the right to receive any payment hereunder."

"Big Four."—"By failure to secure right of way, company is not liable for damages, but gives extension of time. * * * Engineer at liberty to make any changes. * * * Provisions of this contract apply to all changes. * * * Contractor shall prosecute work in such manner, at such times and points as engineer shall direct. * * * No portion of the work to be sub-contracted. * * * Final estimates upon the full completion, to the satisfaction, approval, and acceptance in writing of the engineer."

Chicago, Burlington and Quincy R. R.—"No damages from default of the company, but extension of time. * * * Engineer may make changes. * * * Addition or deduction to price to be made as engineer deems just and equitable. * * * Final estimates, including quantity, quality, classification and price, shall be subject to revision and adjustment by Chief Engineer. * * * Company has the right to terminate contract on giving final estimate to date. * * * To prevent disputes or misunderstandings between the parties hereto in relation to matters of fact, but not of law, the engineer is hereby constituted and appointed the umpire to decide finally, his decision as to the quality, character, kind and classification to be in the nature of an award, final and conclusive."

Missouri Pacific Ry.—"No damages from any default of the company. * * * All provisions of contract to apply to changes. * * * Prosecute work as engineer shall direct. * * * Work shall not be sub-contracted. * * * Final payment upon the full completion to the satisfaction, approval and acceptance in writing of the engineer. * * * Company has right to suspend, or reduce force. * * * After suspension, company can order work resumed. * * * No claim for damage. * * * Engineer shall be, and hereby is made, constituted and appointed, the sole arbiter to decide all questions and matters, and said engineer shall determine and set forth in the final estimate, the quantity, character, kind and classification, under the contract, and his decision and determination, as to any and all matters, shall have the force and effect of an award, and shall be final, binding, and conclusive at all times and places."

Great Northern Ry.—"Contractor agrees not to relet. * * * Work at such points as engineer directs. * * * Engineer made umpire to decide all matters arising or growing out of the contract. * * * Company has right to stop any of the work or diminish force, no claim for damages. * * * Right to make changes, but price to be paid per cubic yard shall cover the risk of any change to the disadvantage of the contractor, and he shall have the benefit of any alteration that may operate in his favor. * * * The decision of the engineer on any point or matter touching this agreement shall be final and conclusive. * * * Said parties waive any and all right of action, suit, or other remedy, in law or otherwise. * * * Final payment when the certificate and estimate shall have been furnished by the engineer."

Chesapeake and Ohio Ry.—"Executed under the direction and supervision of the engineer, or his authorized assistants, by whose measurements and calculations, quantities shall be estimated, and whose determination shall be conclusive. * * * Final estimate on engineer's certificate that the whole work is completely and acceptably finished within the time specified. * * * Contractor shall not sublet. * * * Right to make any alteration in location, line, grade, plan, form or dimensions. * * * No claim for anticipated profits on work, altered or dispensed with. * * * Nothing construed into liability for damages from hindrances and delays, but contractor shall have extension of time. * * * All questions, differences or controversies shall be referred to the engineer; his decision final and conclusive to both parties."

The foregoing extracts are sufficiently illustrative of the powers stipulated for the companies in their construction agreements.

The powers claimed on the average may be summarized as follows:

Contractor to do the work where and when the engineer shall direct, whether the procedure is, or is not, a reasonable one in economical organization, and whether or not the procedure is fair.

Where the company is in default from any cause, the expense shall be borne by the contractor, his relief is in extension of time, provided he gives notice and the engineer considers the point well taken.

The contractor shall equip his work with such forces and appliances as the engineer shall direct. In case he does not, the company holds the right to employ the force and charge the expense; to annul the contract in whole or in part; to seize the contractor's plant; and to withhold any unpaid sums of money which may then be due. The contractor's employees are subject to discharge by order of the engineer. A reservation of 10% is withheld from the contractor's payments in addition to withholding, by custom, another part at the discretion of the subordinate engineer. The company, of its own motion, without default on the part of the contractor, holds the right to terminate the contract at any time, to suspend the work, to hold the contractor to resume, with stipulated denial of the contractor's right, not alone to damage, but even for recovery of expense. The contractor must obey all orders of the engineer and accept his determination as final, at a time when the engineer holds the relation of an employee and agent of the company; the contractor knowing at times that the engineer has no detailed knowledge of the work, and that the information which he certifies as his final judgment is, in fact, the work of an assistant, with whom he may not be even personally acquainted.

Requirement that the company shall have the right to save itself in case of claims against the contractors; to make an *ex parte* examination, to settle the controversy and charge the contractor with the award and such expense as the company shall set, without consultation with the contractor, and without his acceptance. Requirement that the contractor shall exhibit complete receipts, showing that all accounts have been paid. Requirement that the contractor shall save the company harmless from every matter growing out of the construction.

Requirement that, before the contractor can get final payment for matters not in dispute, he shall accept the engineer's estimate as a total and release each and every matter and sign in full settlement, with a more drastic wording in some contracts; that the contractor shall have accepted the final, as handed out by the engineer, before the company shall have incurred any indebtedness to the contractor.

Requirement that, after the contractor shall have done work which is satisfactory to the inspector appointed for it, the work may still be condemned and required to be done over.

The stipulation in favor of the contractor is, that he shall have the right to accept without protest the sum of money the engineer shall say is due him.

It is safe to say that in no other business relation between men are such one-sided agreements customary; in no other relation is a man conceived to be clothed, by reason of a written instrument, in a mantle of infallibility, as is the engineer in customary railroad contracts. In political matters, some thinkers hold that an intelligent despot can give the most efficient government; so, in the case of the engineer, granting an untrammeled, industrious and able man, subjecting the contract to his exclusive decision may work out as the best arrangement all around. As far as the writer's observation goes, the average and general result is good, without much genuine offset; but every now and then there is an instance of gross tyranny and outrageous wrong under these powers. The delays, safeguards, and forms appropriate for a peaceful civilization would paralyze an active army. Railroad contract work requires somewhat of the army's autocratic directness of control, but the control should be within well-defined and reasonable lines.

The objection in a practical sense, however, comes, not so much from arbitrary or unfair use of the engineer's power, but from his carrying out, or being forced into carrying out, requirements which are too broad or are unreasonable, which he may have thoughtlessly included in his agreement, put there from mere copying of precedent, or at the suggestion of a legal department which considers only its side of the case. Ordinarily, these clauses are unnecessary. In spite of them, most engineers have to and do make fair adjustments, and settlement for the majority of work goes through with mutual satisfaction. With these clauses too strongly drawn, the engineer, in spite of a personal desire to be fair, may be forced by his company into an opposite policy, in accordance with the stipulations of the contract. In no class of cases is there greater real damage done than when organized work is suspended or stopped.

In the old days, when contractors' equipment was carts, scrapers and stock, the suspension of work was a serious blow. At the present time the organization and plant for work is multiplied many times more than the requirements of twenty years ago. The expense, effort and time sunk in organizing, very often with special plant, has only a partial relation to the work which may be performed at a particular date. It is a general expense to prorate over all of it. Any suspension or stoppage cuts off the profitable part of the work, and where no compensation is made for the stoppage, the contractor is in effect robbed, whether it be by agreement or not, presuming that the contract was profitable in itself.

It must be further considered that the contractor, in making agreement for a given piece of work, has taken the risk, and if it can be shown that the uncompleted work would have been profitable, upon suspension of this work, he is legally and fairly entitled to that profit, subject to such offset as would be made by an earlier release of services. The no-damage clause offsets this or any other claim.

As the writer understands the obligation of contracts, in matters of measurement, classification, workmanship, meaning and application of specifications, and the like, which have to be decided by an expert, and for which the engineer is nominated by agreement as such an expert, his finding and decision will be held final in the absence of fraud. All other matters are at least open to court review. Therefore, a clear, fair contract interpreted by the engineer has a better chance to be upheld in its final than one which is unfair and extravagant in its stipulations in favor of the company. The time has about come for the companies to be willing to assume the risk of their own acts and plans, and not to saddle these risks further upon contractors.

In conclusion, the writer would say that the main thought in this whole discussion is: That a contractor and a company enter into an agreement for mutual benefit. Every matter not clear, subject to whim and opinion in its working out, unfair in its intent, or in the nature of a "strangle hold," is unfair to one side and reacts upon the other, and that the business of both parties is best served by a fair agreement.

Construction methods, throughout the United States, at least, are fairly uniform for like classes of material. There is, however, the widest variation in customs, requirements, specifications, and general clauses for the agreements for such work. There is no valid reason why the agreements should not be standardized. The consideration and preparation of a standard form would be a very useful and important work for this Society, and of benefit to the greater number of its members, who have more or less to do with carrying out these agreements at one time or another. A standard form would not interfere with special modifications which might be necessary at times. These matters could be covered by special specifications, attached to the standard form.

DISCUSSION.

Dillman. GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—Many of the objections by Mr. Dennis to modern railroad grading contracts are well taken.

Regarding classification, the writer has worked under specifications with from two to eleven classifications. It is his opinion that three classes are better than two, or four, or more. As to the defining clauses, it is hardly material to the contractor what they are, if he knows what they are when making his prices. It it much more important to know who will make the rulings.

The following definitions are suggested as being in line with recent practice:

"Solid Rock" shall include all rock, in ledges or masses of more than 1 cu. yd., which requires blasting.

"Loose Rock" shall include all detached masses of rock measuring more than 1 cu. ft. and less than 1 cu. yd.; also, all shale, slate, soft sandstone and hardpan which can be removed with bar and pick, though blasting may be resorted to.

"Earth" shall include all material not classed above as solid or loose rock.

The plow test never appealed to the writer as a proper test at all. He has seen ten mules stretched out and pretending to pull hard, but not pulling a pound. He has seen solid rock in ledges torn loose with a plow, not exactly plowed, but loosened cheaply in that way. The ordinary engineer is not a mule expert, to know whether or not the animals are pulling.

With foundation excavation, no specification applies to two connective cases. Such work is usually taken out, or at least trimmed up, under the direction of an inspector working under orders; and cost plus a percentage is the only way to eliminate hazard to the contractor or excessive cost to the company. At the letting of a contract, seldom have the investigations proceeded to such a point that a price per unit, or any other way, would represent cost plus a fair profit.

The author is certainly right in cutting out the useless refinement of the prismoidal formula. If a contractor gets his banks full and his cuts of full width, he has done many times as much work outside of the computed prisms as he is paid for by the error of end-area computations.

Overhaul, with a specified free haul, was originally an attempt to even up the cost of grading, the haul being unequal. Beyond a 300-ft. free haul, the writer is of the opinion that it does not serve its purpose, and, when the free haul is extended to 800 or 1 000 ft., it is absolutely pernicious. On the other hand, there seems to be no reason for paying the same for train haul in steam-shovel work as for team or light-car haul. If the method of working is understood in a general way, it will be fair to pay for team haul per 100 ft. and train haul per mile, per cubic yard. Mr. Dillman.

There is certainly no justice in making a contractor pay for work by forcing the haul clause to save right of way. Each case should be considered by itself, and the rule should be applied so that the total economy should be greatest; *i. e.*, the cost of the adjacent right of way plus cost of short haul should be placed against the cost of distant right of way plus long haul. The fact that the company paid one item and the contractor had to pay for the other should not affect the decision of the engineer.

The writer sees no objection to the clause regarding sub-contractors. It is used in order to have only one party with whom to deal in a legal sense, so that, in case of differences, there will be fewer parties to the controversy. In operation, it is never abused. Building contracts are not similar. The various lien laws require certain published statements, sometimes the contracts in full, which would be inexpedient in railroad contracts, and more obnoxious to the contractor than the company.

The writer does not agree that the engineer has too much authority. It is true that in a few cases such authority has been abused, for human nature varies; but, put the authority with the contractor and see where railroad contracts would end.

The object of a contract is to get work done. For its execution, the contractor is one tool, the engineer another. Authority must be given to one or both of them. A divided authority is never efficient. In the interest of ideal fairness, this authority might be divided, but in the direction of attaining the desired end, full authority should be left just where it is. Responsibility is fixed by it, and could be in no other way.

The author has left out the matter of shrinkage. Of course, some material will shrink in banks, and this fact should be reckoned with in grading. The question is, who should pay for it? A clause by which the contractor agrees to put on shrinkage up to 10%, if required by the engineer, makes a straight difference of 10% to the contractor, depending on whether the engineer does or does not order it. This 10%, gross, may mean doubling his profits, or making a losing contract, as the case may be.

Where practicable, quantities paid for should be measured in excavation. Where it is not practicable to measure them in this way, and the payments are made on bank measurements, the writer suggests that extra material, put on for anticipated shrinkage, be paid for at the same price as other yardage.

A contractor objects when shrinkage is first demanded, objects

Dillman. when it is enforced, and may get so used to it that he quits objecting; but he never gets over the fact that the requirement is unfair, as, in fact, it is. Like an abnormal free haul, it is an attempt on the part of the company to get something for nothing, which, in time, reacts.

The writer, however, fully agrees that uniformity in grading specifications is desirable, and, while not agreeing with some of the details suggested, admits that uniformity, with some bad details, is better than the present lack of it.

Campbell. J. L. CAMPBELL, M. AM. SOC. C. E. (by letter).—The writer believes that the prevailing classification of earth, loose rock, solid rock, and overhaul should be eliminated, and that part of the work should be covered by a single unit price per cubic yard.

If a classification is to be retained, the writer suggests the desirability of eliminating the terms, "earth," "loose rock," and "solid rock," and substituting therefor the designations, Class 1, Class 2, and Class 3, and making the specifications read somewhat like this:

Class 1 shall include all material which can be broken and loosened by a standard No. 1 railroad plow into a condition in which it can be removed by standard railroad scrapers, the plow and the scrapers being drawn by power sufficient to develop their full capacities.

Class 2 shall include all material which cannot be broken and removed as specified for Class 1, but which can be loosened and removed by picks and bars, or by a standard steam shovel, rated at 50 tons or more, and without the use of explosives.

Class 3 shall include all material which cannot be removed without being broken and loosened by the use of explosives.

The mere use of the means specified for Classes 2 and 3 will not of itself be evidence of, nor shall it necessarily fix, the classification of the material, because the contractor will frequently find it to his advantage and will use the means prescribed for the higher class to remove material of the lower class. It must be shown that the means used are actually necessary for the removal of the material in question, and, in case of doubt or question about the class of material, the company's engineer shall require, and the contractor, under the direction of said engineer, shall make, such conclusive tests as will determine the class to which such material belongs. Whenever possible, such tests of such material shall be made before the latter is removed.

The writer suggests this, not as a perfect specification, but merely as a change in the right direction. The terms, "earth," "loose rock," and "solid rock," may be entirely misleading as a specification, for each may, and frequently does, include a great variety of material which, in the abstract, is neither earth, loose rock, nor solid rock. Mr. Campbell.

The customary overhaul clause should be omitted, either with or without a classification.

M. S. Parker, M. Am. Soc. C. E. (by letter).—Mr. Dennis strikes a popular chord when, in the conclusion of his most seasonable paper, he says: Mr. Parker.

"There is no valid reason why the agreements should not be standardized. The consideration and preparation of a standard form would be a very useful and important work for this Society."

The writer, having had many years of experience, both as an engineer and as a contractor, heartily concurs in this sentiment. The specifications now in use on many of the great railroad systems of America are as old as the systems themselves. They are old forms, handed down from generation to generation, with an occasional new feature added. It matters not that many clauses may have been ruled against by the courts, the same clauses still appear, either in the hope that they may "stick" in some particular case, with benefit to the company, or from absolutely careless disregard of the rights of the contractor. The author has made a strong argument in favor of standardizing railroad grading specifications, and it seems to be almost unnecessary to add more on this subject. All must admit the desirability of united action toward such uniformity. The railroads of America are expending more money yearly in construction than all other industries combined. When such men as J. J. Hill, F. Am. Soc. C. E., President of the Great Northern Railroad, estimate that, under the present conditions of traffic, there is needed on the railways of the country an expenditure of $5 500 000 000 within the next five years, some idea of the enormous extent of the railroad contracting interests can be obtained.

Many of the clauses in the specifications for grading on large railroad systems are meaningless, and have no legitimate place there. Were these clauses to be enforced, no reputable contractor would consider for a moment the advisability of taking the work offered. All the specifications of the various railroad systems are similar in many respects; clauses regarding the subletting of work; the infallibility of the engineer in all decisions, whether right or wrong, as clearly outlined by the author, and many other dead-letter clauses. Why should not the position of the engineer, as judge, be defined in some more rational manner? Why not substitute such a clause as the following?

The decision of the engineer, on any point or matter touching this agreement, shall be final and conclusive during the progress of the work. This, however, shall not deprive the contractor of any

. Parker. of his rights of redress in case such decisions are palpably wrong in intent or through ignorance of the facts.

Engineers, as a rule, are inclined to be just and fair in their judgment, but they are not infallible. They are subject to error, as are other mortals.

The *bête noir* of contractors on railroad grading work, and also a source of great annoyance to engineers, is classification and overhaul. Engineers vary in opinion as to the proper classification of material under the different specifications, and, to the contractor, overhaul, as calculated by the various methods, is an indefinite quantity.

Clauses in specifications governing the classification of material cannot be expressed too carefully. The terms, "earth," "loose rock," and "solid rock," are a source of much juggling when one indulges in recourse to the courts. The writer has long held the opinion that these terms, used so generally in railroad specifications, should be eliminated, and "classes of material" substituted, the material constituting each class being clearly defined. This method has been adopted by the United States Government in its grading specifications, and railroads might well follow the example, and avoid all complicated technical misunderstanding of words. The terms, "earth," "loose rock," and "solid rock," in themselves, do not express the character of the material to be moved, and the explanations following them in specifications do not change the fact. To designate as loose rock such material as cemented gravel, hardpan, soft sandstone, soapstone and disintegrated granite (which can only be disintegrated through the process of blasting) is absurd.

Solid rock, defined as "rock in solid beds or masses which can best be removed by blasting," would appear to cover the ground intended by the term "solid rock." Such a definition, however, followed by a stipulation that it must ring under the hammer, is ridiculous. This clause, appended to a "solid rock" provision, would place solid sandstone and certain other formations of solid rock in the "loose rock" class, instead of in the "solid rock" class where they rightfully belong. Disintegrated granite boulders, after long exposure to the weather, become so hard that tools will barely cut them. They will ring under the hammer, but a little dynamite will shatter them so that they can be handled as gravel. Solid granite formations which can only be disintegrated by blasting should be classed as solid rock.

The following classification is suggested as fully covering the subject, where three classes are provided:

Class 1.—All rock in solid beds or masses which can best be removed by blasting.

Mr. Parker.

Class 2.—Slate, shale, hardpan, cemented gravel, soapstone, soft sandstone, and all other rock loose enough to be removed without blasting, although, to facilitate handling, blasting may occasionally be resorted to.

Class 3.—All material which in its natural condition can be plowed with a plow cutting a furrow 10 in. wide and 10 in. deep, drawn by four horses or mules, each weighing not less than 1 100 lb., and moving at a reasonable plow speed.

Where any excavation contains material of more than one classification, the relative percentage of each class shall be determined by actual measurement and observation during the progress of the work.

If this last clause is observed by the engineer, the classification will not be so much a matter of opinion as is too often the case under present practice. It is folly to attempt to classify, with any degree of fairness, any excavation containing the various classes of material, after the excavation has been made. This method, so often followed, is unfair to both parties. It resolves itself into a guess as to classification, and the question as to who gets the best of it depends on the guessing ability of the engineer.

The writer has in mind an instance of the varying opinions of engineers as to classification: A cut containing 20 000 cu. yd. was divided in the middle on two engineers' divisions, each getting 10 000 cu. yd. The cut was uniform in depth on both sides of the dividing line. The character of the material was the same throughout the cut, and was hard sandstone overlaid with a layer of earth too thin to be removed by plowing and scraping. The entire cut had to be blasted continually. One engineer turned in his final estimate of 9 500 cu. yd. of solid rock and 500 cu. yd. of earth; the other estimated 500 cu. yd. of earth, 3 500 cu. yd. of loose rock and 6 000 cu. yd. of solid rock. This is one of many instances which the writer could give, which show clearly a difference of opinion in matters of classification, where no difference should be possible.

For moving classified material at mean fair prices a contractor may make a reasonable profit on his work, or he may lose money, depending entirely on the judgment or opinion of the engineer; and, under many specifications, there is so much room for error in opinion that a contractor never knows how he is to fare in the final settlement. This condition should not exist. In the writer's opinion, the fairest and most reasonable grading specifications divide the material into two classes: solid rock, or material of the first class; and earth, or material of the second class, which covers all other materials. The work should be paid for in embankment and also in excavating, or "pay both ways," as it is usually termed.

. Parker. Eliminate the overhaul provision and make the extreme haul 2 000 ft., with a special price per cubic yard for any haul of more than that distance. A contract drawn on these lines prevents all controversies between engineers and contractors in reference to classification and overhaul, much to the relief of both parties.

The overhaul provision in specifications is a source of never-ending controversy. In the early days of this provision, overhaul was largely a matter of guess, as with classification, especially with railroad building in the West. The writer is familiar with many methods of calculating overhaul. Many of the graphical methods are so obscured in mathematical detail that, to the average grading contractor, they are absolutely unintelligible.

The method now used extensively is to take the distance between the centers of mass in excavation and embankment for the average haul, and deduct therefrom the free haul. This, when used properly, appears to be about as reasonable a method as can be devised.

All methods of calculating overhaul known to the writer are erroneous to a greater or less degree. Ordinarily, no consideration is given to the shrinkage or swell of material from excavation. It is assumed that a certain number of cubic yards of excavation in place will make the same number of cubic yards of embankment. The error is evident. For example, 10 000 cu. yd. of earth are hauled from a cut to make 10 000 cu. yd. in an embankment upon which the overhaul has been calculated. The excavation falls short of making the embankment by 1 000 cu. yd. The contractor is paid for the overhaul as calculated and puts in the extra 1 000 cu. yd. at his own expense, or, rather, receives the overhaul price instead of the embankment price for the extra 1 000 cu. yd. Such instances are common, and have come under the writer's observation frequently; they are not exceptions.

In the excavation of rock formations, the swell of the material varies greatly, depending upon the class. When, from an excavation, the embankment quantities are made and the extreme limit of haul is reached, the remainder of the excavation must be wasted. The contractor receives pay for overhaul for the yardage in the embankment; but, for the waste material, he receives no compensation for hauling, unless he elects to haul past the haul limit and accept embankment price for it. As a result, he often hauls from an excavation a large percentage of the material without any compensation for such haul. In many cases this waste material must be distributed along the embankment, and this becomes an unjust burden on the contractor. In the writer's experience, no instance is recalled where the length of overhaul has been calculated on the route taken from cut to fill. The writer has in mind several instances of 100-ft. cuts into 100-ft. fills, as mentioned by the author. Some were filled from suspension bridges, and others by using a long and elaborate system of tracks and trestles. In such cases the ordinary allowance per cubic yard for overhaul does not begin to cover the expense of hauling, and, as a matter of fact, from the writer's experience, the average price of 1½ cents per cu. yd., paid for overhaul of more than 500 ft. and up to 1 500 ft., rarely pays for the expense of hauling, under the most favorable circumstances. Under the method of estimating overhaul from center of mass to center of mass, the writer has known instances where the engineers construed these centers as being the limit of haul. Under a clause in a contract limiting the haul to 1 500 ft., the writer has hauled material for twice this distance, or until the centers of mass were 1 500 ft. apart. If the overhaul clause is to remain a part of grading specifications, the writer quite agrees with the author that the stations of free haul, and also the stations of center of mass in cut and fill, should be established. Overhaul computation on this basis can always be checked at any subsequent time. Unless some such arbitrary method of calculation be adopted and set forth clearly in the specifications, the long-drawn-out controversy over the question will always remain a subject of annoyance and misunderstanding. **Mr. Parker.**

To strike out the overhaul clause from railroad grading specifications would be for the best interests of both parties, and would inaugurate a system of square dealing. As remarked before, the only fair way to pay for grading is to pay for the excavation and also for the embankment. With uniformity of classification and the elimination of the overhaul clause, the contractor would always know what conditions he had to meet, and his bid could be arranged intelligently. Railroad contracting would then become a straightforward business proposition, instead of a hazardous risk, under which, to be absolutely safe, the contractor must place his figures high enough to cover the contingencies that may arise under the classification uncertainties.

Halbert P. Gillette, M. Am. Soc. C. E. (by letter).—The author has done a service to the profession of engineering, as well as to the business of contracting, by collecting so many examples of ambiguous specifications for excavation. It would be difficult, however, to select any work so hard to define in words as the classes of excavation. Earth merges by insensible degrees into hardpan or shale; hardpan and shale merge into conglomerate and slate by equally insensible degrees; geologically, there is no dividing line between what is called "earth" and what is called "rock." This fact is well illustrated in the dictionary definitions cited by the author, and it is shown even better by the definitions found in textbooks on **Mr. Gillette.**

Gillette. geology. It may be seen, therefore, that some arbitrary test should be prescribed to differentiate rock from earth when different prices are to be paid for each. The ancient "plow test" has many adherents, but in its usual form it is probably more productive of legal trouble than any other clause ever devised by an engineer. Some years ago the State Engineer of New York suffered most unjust criticism because of supposedly unfair classification of excavation by the plow test, and his retirement from office was probably due in large measure to this unjust criticism.

The writer, while on the editorial staff of *Engineering News,* in 1903, wrote a series of editorial articles criticizing specifications, and he recalls having written one suggesting an earth classification test somewhat similar to that proposed by the author. Instead of specifying a furrow 10 by 10 in., however, he suggested a minimum number of cubic yards loosened per 10-hour day by a 6-horse plow. It still seems to the writer to be a much better plan to specify in cubic yards, for the cubic yard is the unit of cost, and, after all, the object is to secure some definite cost classification. A 10 by 10-in. furrow cut by four horses means nothing very definite unless the amount of useful work is specified, either by naming the average speed of cutting, or the average number of cubic yards to be loosened in a given time; but why limit "earth" to such easy material as can be loosened by four horses? Ten-horse plows are very common in the West, where driving with a jerk-line is practiced. There is a serious objection to the plow test wherever work is to be done with steam shovels, and the objection is that it is practically impossible to apply the test in many cases. In a through cut, for example, the top 4 ft. of material may be loam, below which may lie an indurated clay of hardpan consistency. The steam shovel exposes a vertical face upon which no plow test can be made; unless this 4-ft. stratum is stripped, the plow test is of no use on the surface. The bottom of the pit may be solid rock. Of what practical use is the plow test under such conditions?

Many other conditions might be mentioned to show the exceeding difficulty of applying a plow test in a satisfactory manner. One more will suffice. In soil of glacial origin, lenses of hardpan are frequently encountered, surrounded by gravel, sand, or shot clay. It is impracticable to strip these lenses in steam-shovel work for the purpose of using a plow test, and, without stripping, no such test is possible.

The plow test, therefore, may serve in plow work, but it is practically useless in much of the work done by steam shovels.

What test, then, shall be applied? In the writer's book on earthwork,* the suggestion is made that excavation be classified by sam-

* "Earthwork and Its Cost," p. 21.

ples taken from specified locations on the profile. No practical method of specifying with any degree of exactitude seems possible to the writer, and a varied experience, embracing excavation at different places across the American continent, has served to emphasize this conclusion. Mr. Gillette.

It is true that this method of "specifying by samples" involves digging test-pits and sinking bore-holes, but the writer is firmly convinced that no engineer ever spent a dollar that returned a greater dividend than the one spent in ascertaining the character of the excavation before the award of the contract.

On any extensive piece of excavation, earth should be dug, and rock should be blasted, by the engineer, to ascertain its quality, as well as to determine the relative quantities of each class. The engineer who cannot persuade his employer to go to this extra expense is hardly fit to be in charge of the work; or, if he is fit, he does himself an injustice in not resigning if his advice is ignored.

The writer agrees with the author in recommending the "end-area" formula, provided the field engineer is taught how to space his cross-sections.

The author, in common with some other writers, conveys a wrong impression by saying that the "end-area" formula is compulsory in some States. It is not compulsory in any State, but laws do provide that, "in the absence of any specified agreement as to measurement," the "average end-area" formula must be used.

In the matter of specifying that embankments shall be built in layers, the writer is in hearty accord with the author, and has already expressed himself at some length elsewhere. It is the acme of asininity to specify that embankments shall be built in layers when there is no intention of enforcing the specification.

As to sub-contracting, the writer also agrees with the author. In the writer's judgment, the ideal method of doing all work in the world is by some sort of contract system, and the further down the line the method can be carried, the better. A sub-contractor is a superintendent or foreman whose wages are paid in the form of profits. A man working by piece-work, or under a bonus system, is a sub-sub-contractor, and his wages, in turn, depend directly upon his energy and skill.

Engineers will always have more or less trouble with contractors; but he is a weak engineer, or a lazy one, who permits such troubles to lead him into the wasteful method of doing work by day labor when it is possible to do it by contract, by sub-contract, or by sub-sub-contract. The writer has recently examined extensive records of cost of railroad work done by contract and by company forces working on the day-labor plan. It is no unusual thing to find that company work has cost 200% more than contract work

Gillette. under precisely the same conditions. For example, it has cost one railway $1 600 per mile to lay and surface track, where $500 would have been sufficient under a contract. The grading cost was equally disproportionate.

If any engineer believes he can do work cheaper for a company by day labor than by contract, he will be doing the company the greatest possible service by resigning and taking the contract himself. Then the knowledge that he must make his brains make the muscles of his men work, or no pay for his time will be had, is certain to test his ability to effect economies as no other condition can.

Then, let there be an end to these absurd clauses prohibiting sub-contracting. Now that engineers, in rapidly increasing numbers, are entering the business of contracting, there is reason to hope for more contributions such as this admirable paper. Let the discussion be one of hard blows, if need be, for, as long as it brings out the honest opinions of both parties as to a contract, it is bound to result in more reasonable prices, fewer lawsuits, and greater economy for all concerned.

Kennedy. JAMES H. KENNEDY, M. AM. SOC. C. E. (by letter).—This paper is most interesting and instructive, as the author appears to view the subject principally from the standpoint of the contractor, and presents the generally accepted views of contractors on contracts, specifications, classifications, etc., in a very able and lucid manner.

This view of the subject will be of great value to many engineers who have never been called upon to look at these matters as the contractor sees them. A somewhat limited experience as a contractor's engineer, in cases where estimates failed to cover working expenses, leads the writer to agree with most of the conclusions arrived at by Mr. Dennis. It may also be added that these conclusions will be found to be of great help in understanding human nature as exhibited by the contractor who digs deeper and deeper into his pocket to meet current expenses, not knowing where he is likely to get off, or whether or not he is getting justice at the hands of the engineer.

The usual railway contract is, no doubt, a very one-sided agreement, altogether in the interests of the railway company, from the contractor's point of view, and its conditions generally clothe the engineer with autocratic powers. It is so one-sided, in fact, that its provisions seldom require the engineer to take advantage of them, and so unreasonable that the contractor often disregards the objectionable clauses of his contract entirely, and, sizing up the engineer, takes his chances of being allowed to avoid such clauses on account of his knowledge of the kind of engineer with whom he has to deal. Nevertheless occasions arise when it is necessary, in the interests of the railway company, to take advantage of one or all of these rigid clauses of the contract; consequently, it is doubtful whether they will ever be eliminated entirely. The writer cannot agree with the author, however, when he says: Mr. Kennedy.

"The time has about come for the companies to be willing to assume the risk of their own acts and plans, and not to saddle these risks further upon contractors."

It is not clear that the contractor assumes any risk except his own, with the exception of an occasional change of location which is unfavorable to him; this, however, is usually provided for in the contract.

Again, in the matter of classification of material, where there is a classification varying from earth at one end of the scale to solid rock at the other, and with intermediate classes of hardpan, loose rock, etc., between, there is no trouble in classifying the earth and rock, as all specifications are somewhat clear and definite; but this is not so with the intermediate classes, especially where it is the custom to allow a percentage of each class, varying with the hardness of the material or the cost of handling. It may be safely stated that, not only will different engineers seldom give exactly the same classification, but the same engineer is likely to vary his classification somewhat on different days when passing over the work, and, if notes be kept and compared, he will be surprised at the different classifications which he has made from day to day for the same work; consequently, there is no definite and fixed standard by which all engineers will classify in the same way; but each must follow his individual ideas. In time these ideas become fixed, so that, in the mind of the contractor, they become what may be called the engineer's personal equation. Every engineer who follows railway work has this personal equation, whether he knows it or not; consequently, contractors, when tendering for work, give more attention to obtaining information as to who is to be the engineer, than to studying up the requirements of the contract and specifications. This will always be the case, to a certain extent, whether or not all work under the same standard specifications.

In regard to the extra items connected with all contracts, the author, no doubt, has the correct idea, and the writer fully agrees in the opinion that:

"One matter should not carry the cost of another matter, unless the latter be uniformly a concomitant of the first and is specified so definitely that its quantity and cost can be calculated."

As an illustration of the meaning here intended, an item of clearing may be assumed to be a single large spreading tree in the middle of a cleared field, which has cost the contractor $10 to remove, and his contract price for clearing to be $20 per acre. The

Kennedy. writer's idea is that this should be returned in the estimate, as a sufficient area, to amount to $11; this is the cost plus the usual percentage, independent of actual measurement on the ground. The reason for this opinion is: that there is no way by which the contractor can obtain a definite idea of what he might be allowed for this particular piece of work, and could not have provided for it. It may be objected, that the above case would be reducing the work to a force-account basis, and that it would be unfair to the company to pay for the odds and ends on such account, while the principal part of the work is done by measurement at contract price; but the case here assumed is only intended as an illustration of a case which, in its nature, cannot be covered by actual measurement. In a general way, there is no valid reason why such items as foundation pits, drains, etc., should not be measured up and returned at actual measurements, but it is clear that the railway engineer cannot avoid entirely the use of the force account.

On the whole, the idea of a standard form of contract, which could be used by all companies, is a good one; and if the Society should take up the matter, there is no reason why the Standard form of Contract and Specifications of the American Society of Civil Engineers should not become as well known as the Standard Rail Sections of the American Society of Civil Engineers.

F. Lavis. F. LAVIS, M. AM. SOC. C. E.—The importance and timeliness of this paper, and the necessity of such uniformity in specifications and contracts as is possible under the varying conditions met in this class of work, can hardly be over-estimated, in view of the large interests involved, and the very divergent opinions held by engineers; to say nothing of the ignorance often displayed, not only by young engineers, who, working for salaries of $90 and $100 per month, have the responsibility of deciding questions of classification involving many thousands of dollars, but also by those higher up, who blindly copy specifications and contracts, no matter by whom they were written or what conditions were originally intended to be covered, and containing provisions which have been the cause of endless trouble both in and out of the courts.

The speaker is obliged to differ from the author, however, as to his final conclusion that a contract providing for classification, which is properly written, would be productive of better results than one under which a straight price is bid for all material excavated. Under this latter condition, however, there would possibly be cases where it might be advantageous to ask for separate bids on different sections of a long line where conditions might vary so much as to seem to warrant it, and also, of course, a separate price for foundation excavation both above and below water.

* Continued from March, 1907, *Proceedings*.

Mr. Lavis.

The author admits that under present conditions, the no-classification principle is rapidly coming into favor, but claims that in such cases engineers do not furnish sufficient information on which bids can be properly based. The argument, therefore, lies between the two alternatives: Whether it is better for the railroads to furnish the contractor with full, reliable information in regard to the work, and do away with classification, or, whether an attempt should be made to frame a specification which will eliminate all disputes. In view of the fact that even engineers are not always infallible, and that a decision as to classification has generally to be made on the spot by young and oftentimes inexperienced engineers, and also that, as a sound engineering proposition, proper information should be furnished to the contractor, there is little doubt in the speaker's mind that the former is preferable.

The necessity of providing full and accurate information to prospective bidders on new construction has been pointed out and emphasized by the speaker in a paper* read before this Society, and amplified further in his book: "Railroad Location, Surveys and Estimates." One of the necessary results of a location survey is therein stated to be:

"To have on the completion of the survey complete right-of-way maps, estimates of quantities, and costs, showing in detail the exact nature of the work, so that contractors can bid intelligently and work be started at once."

The value of an accurate map showing the accessibility of the work, its relation to the surrounding country and lines of communication, roads, trails, etc., on a convenient scale for use by the contractor in looking over the situation on the ground, was also pointed out.

This information is of just as much, if not more, importance to the railroad company as to the contractor, and, undoubtedly, should be obtained in all cases. On important construction, such as, for instance, the New York Subway, or the work which the New York Board of Water Supply has on hand, there is no question as to the propriety of spending all the money necesary to obtain as accurate information of sub-surface conditions as the state of the art warrants. The speaker does not for a moment advocate diamond drill borings to determine beforehand the nature of the material in an ordinary proposed cutting on a new railroad, but he believes that sufficient information should be obtained by auger borings or test-pits, or by any process by which the character of the excavation may be estimated with sufficient accuracy for the purpose in hand.

* "Methods of Location on the Choctaw, Oklahoma and Gulf Railroad," *Transactions*, Am. Soc. C. E., Vol. LIV, p. 115.

·. Lavis. This is necessary, not only for the benefit of the contractor, but also for that of the railroad company, in order to determine, in many instances, the proper location of the line and, in all cases, the position of the grade line. The fact that a cut may be all solid rock, or 50% solid rock, or all earth, has a great influence on this latter, as, in one case, the material might swell and make more embankment and, in another, shrink and make less. Therefore, before the location can be finally settled, and before a grade line can be established, which will give the best balance between cuts and fills, the necessary information as to the probable nature of the material to be encountered in the cuts should be unquestionably obtained. The speaker is inclined to think that railroad managers are becoming more and more convinced of the necessity of doing this, and that there is more promise along this line than in attempting a revision of the classification specifications in order to put them on an equitable basis and beyond the possibility of misunderstanding and dispute.

The speaker has prepared a specimen profile, Fig. 1, which shows substantially the information which should be given to contractors on new construction or, for that matter, on any class of work involving the graduation of railroads, though modifications to suit special conditions will suggest themselves to the experienced engineer. In all cuts and at all foundations, the results of borings or tests-pits, where taken, are shown, samples being on view at some point near the work; the yardage in all cuts and fills is shown, together with the amount of swell if any; and the amounts of both side and cut ditches; the amounts of excavation in foundations; changes of channels, etc., and the disposition of all excavated material; the length of average haul by scale, and the amount hauled, the latter being especially important, as noted later. At all openings schedules of quantities, both of materials and excavation, are shown. In cases of important structures, where the character of the foundation is uncertain, a well-drilling outfit will obtain the information at moderate cost. At the bottom of the profile should be shown. not only the alignment, but also the width of right of way which it is proposed to acquire, with the division lines and names of property owners; and at the top, there should be notes as to the character of the country, *i. e.,* whether wooded, pasture, etc., as the case may be. There should be a summary of excavation quantities for each mile, showing the pay quantities and the balancing of the cuts and fills.

This information, together with a map of the line on a scale of 5 000 ft. to 1 in., as shown in Plate XII* of the speaker's paper previously referred to, will give the contractor all the information necessary, so that when he goes over the ground he can note the

**Transactions*, Am. Soc. C. E., Vol. LIV, p. 128.

Mr. Lavis.

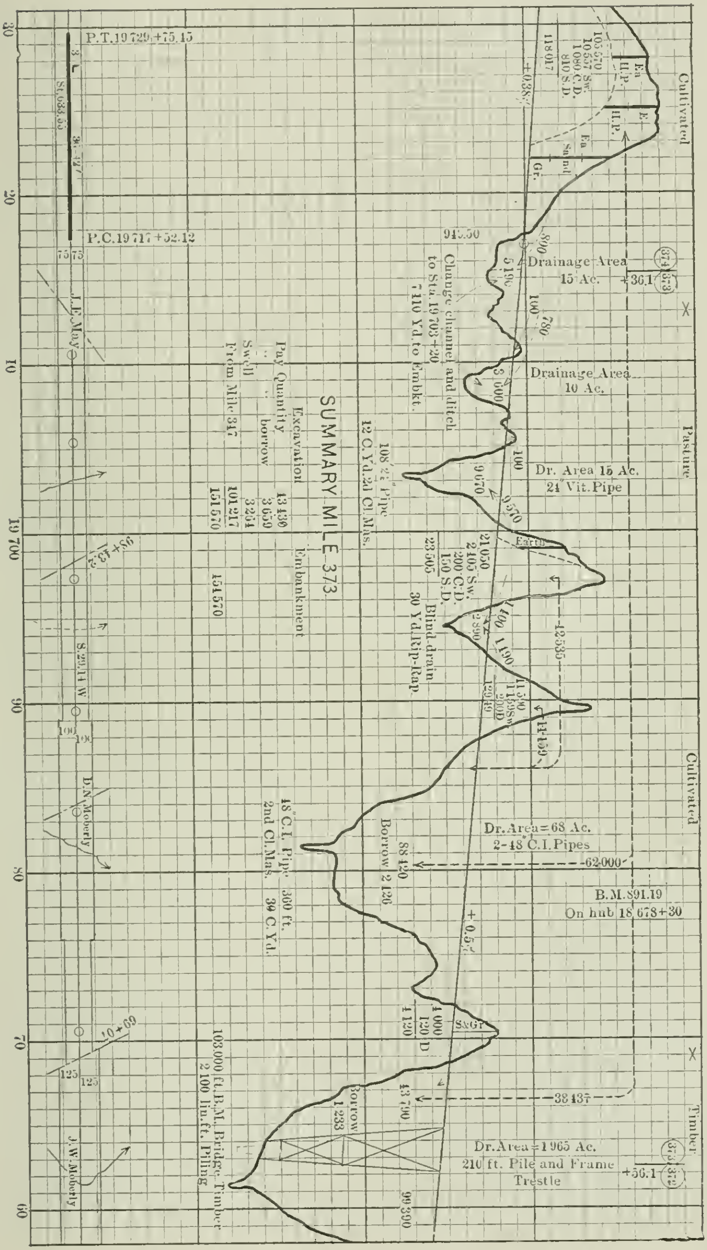


Fig. 1.

r. Lavis. whole situation properly and make an intelligent bid. The map, showing the general relation of the line to the surrounding country and lines of communication, is very useful and necessary for the proper location of sites for construction camps and for estimating the haul of structural materials, supplies, etc.

Of course, all this information is given for what it is worth, and it is believed that there should be no more difficulty in a railroad company protecting itself against any claims of a contractor that he was furnished with misleading information, than the Board of Water Supply, for instance, finds in protecting itself because a contractor might be misled as the result of a wrong interpretation of the information shown by the diamond drill borings. All information of this kind must necessarily be used with judgment, and there is no doubt that the present tendency is toward a freer interchange of ideas between engineers and contractors. No responsible contractor should think of trying to "hold up" a railroad company because he has misjudged the information furnished by the engineers; and, on the other hand, the engineer should provide as full and reliable information as possible, explaining just how it has been obtained, so that it can be appraised at its proper value. It is believed that under these conditions a desirable contractor would come nearer to an equitable bid on a no-classification basis, on account of his large experience in judging conditions, than he would be able to if he had to take into consideration the unknown factor of the untrained judgment of a number of young and inexperienced engineers.

It should be remembered that the man on whom the responsibility of making the decision falls is the resident engineer, a young man with a salary of $100 to $125 per month, in charge of 8 or 10 miles of work. The division engineer usually gets over the work once a week, often only once or twice a month, and, usually, decisions have to be made on the spot, from day to day; he, therefore, is seldom in a position to determine accurately the merits of the controversies which arise, and too often falls back on an indorsement of his subordinates on what are called "general principles," that they should be supported anyway, right or wrong.

The information on the profile showing the proposed distribution of material from the various cuttings in the embankments, the speaker regards as most important, both to the railroad company and to the contractor, and especially to the former. In no other way, than by actually making this distribution as carefully as circumstances will permit, can either the locating engineer or the railroad company arrive at a true estimate of the value of the location, or realize the practicability of the scheme of work.

The cost of the work to the contractor, and this always means sooner or later to the company, is very often increased by impractical conditions of haul which might have been avoided had the distribution of quantities been thoroughly threshed out beforehand. For instance, where possible, a solitary, heavy, steam-shovel and car proposition should be avoided, in the midst of country which can be more economically handled by teams; or, if it cannot be avoided, it may be found that considerable improvement of line and grade may be obtained by introducing sufficient additional heavy work, on adjoining parts of the line, which can be handled by the same outfit, so that the total actual cost of the work, due to the fact that the cost of installing the shovel and its equipment is distributed over a larger amount of excavation, will be so low as to justify the improvement. Many other instances, among others, the necessity of making proper provision for borrow-pits, will suggest themselves, which show the practical utility of thoroughly working out this distribution, besides the obvious advantage to the contractor of being able to estimate at least with a fair degree of accuracy what haul he has to provide for, and thereby minimizing one of the most fruitful causes of dispute and dissatisfaction, and enabling him to lay out his work ahead properly. Mr. Lavis.

The author's remarks on overhaul emphasize the speaker's contention for more and better work on the location. There is undoubtedly a general impression that the line as originally located is likely to be changed so much during construction as to be hardly recognizable by the time the road is built, and the various changes of line and grade have been made in a frantic endeavor to balance the cuts and fills, and through a belated realization of the fact that the location has not been thoroughly threshed out, and that there is a chance to save thousands of dollars here and there, provided the construction has not proceeded too far to prevent it.

No one realizes better than the speaker the absolute impossibility of locating a line so that no advantageous changes are possible; but it is unquestionably a fact that a proper location, based on full and accurate information, properly studied and worked out to its logical conclusion, would reduce the changes to a minimum; and he believes that improvement is to be looked for more in the line of a thorough threshing out of the problems to be encountered before the work is started than by an attempt to overcome the defects of this lack of preliminary investigation by endeavoring to provide a means of paying for what may turn up after construction is started.

The difficulty of writing such a specification may be judged from one case, namely, that in which Mr. Dennis proposes, as a

Mr. Lavis. test for earth, the ability to plow it with four horses or mules of a certain weight. The speaker believes this to be impractical of application, although he is aware that it is very frequently specified. He knew of an instance where a contractor kept a special team for this test. The horses were kept fat and sleek, but they could hardly pull an empty scraper; and, besides this, the man behind the plow may make a great difference. In any event, there are so many instances where earth merges imperceptibly into hardpan, or from any one classification into another, that either the engineer would have to make the contractor keep his team and plow busy all the time making tests, or fall back on his own judgment. Anyone who has been connected with the ordinary contractor's outfit on actual work knows how difficult it would probably be to get any kind of a test made, unless it was at the company's expense. As Mr. Dennis himself points out, under the heading of "extra work," the contractor would consider that this was work which he was compelled to do, but for which he received no compensation. Even supposing the cut were taken out by taking off the top stripping with plows and scrapers—and by no means can a contractor be expected to use this method if some other seems more feasible—there would still be the theoretical necessity of testing the whole surface of the harder material with the team of the standard weight and pulling capacity. In large cuttings, especially where the material is being handled with steam shovels, and in cases where the overlying material is found in pockets, there is an increasing tendency to use well-drillers to bore holes through both earth and rock, or even through hardpan alone, in some cases, and then to blast the whole together. Classification, under such a method, would be almost impossible, in any event very difficult, and there is continually a tendency to increase the size of the cuttings considered feasible, in order to get the required good alignment and low grade necessary to handle the ever-increasing traffic of the railroads.

On betterments of existing railroads there would seem to be a still better argument for no classification, as the nature of the country can be very readily ascertained by an examination of the cuttings on the existing roadbed; in fact the largest amount of work being done under the no-classification specification is of this class, and it is an argument in itself in favor of the fact that better results can be obtained on that basis, provided the contractor can obtain reliable information as to the nature of the work.

In regard to the question of engineering expense in making such preliminary investigations as might be necessary in order to make a proper estimate of the cost of doing the work, it might be pointed out that in all probability this would amount to but little, if any, more than that necessary to keep track of the measurements defining the limits of the various classes of materials, provided this were done in such a manner as to avoid any chance of dispute. Mr. Lavis.

There can be but little question as to the endorsement of the author's position in regard to the application of the prismoidal formula to the calculation of earthwork; the method of average end areas is undoubtedly as accurate as any measurements which are taken of the surface of the ground. It is usual on some roads to instruct resident engineers to take sufficient measurements so that the difference in the center heights of adjoining sections will be within certain limits, say not more than 3 or 4 ft. Using the prismoidal formula on earthwork is on a par with the practice, on one road, of giving the degree of curve down to hundredths of a second, on account of a fancied idea that the ground is being fitted better.

The author's remarks in regard to extra items included in prices for excavation are also to the point. The reputation of an engineer or railroad company for fairness, or even liberality, in dealing with contractors is an asset which should not be underestimated by any means, and no matter how the contract is drawn up, a contractor will think he is being imposed on if he is compelled to build a wagon road for which he is not paid, even though it may be to a certain extent for his own benefit.

The fact that contractors are often compelled to bid on work, and accept contracts, regardless of the form in which they are drawn up, in order to keep their business going, as noted by Mr. Dennis, is something which is beginning to be recognized by thoughtful engineers who have gotten away from the old-fashioned idea that their business is to try and "do up the contractor," and who recognize that the interests of both contractor and employer are becoming more and more identified with each other. With this idea in mind, the speaker believes that both parties to the contract should recognize the necessity of the fullest frankness with each other, the engineer providing as full and accurate information as is possible in regard to the work to be done, and the contractor recognizing the limitations of all such information as to sub-surface conditions.

The old-time railroad contractor, whose equipment is such as to permit him to take only a small section of work which he is able to look after personally, and who, himself, has generally risen from the ranks, is giving way to the modern complicated business organization, often a corporation, in some cases owning seventy or eighty steam shovels, with all the necessary complement of equipment which that involves, and employing high-salaried civil, me-

[Mr. Lavis.] chanical and electrical engineers on their staffs; and it is believed by the speaker that a no-classification specification, accompanied by the fullest information as to the work, would appeal to such organizations rather than one providing for classification.

While differing from Mr. Dennis in this respect, however, the speaker feels that he is to be highly complimented on the very practical nature of his paper, and for the very fair and skillful manner in which he has presented the matter, both from the engineer's and contractor's standpoints. His plea, for a fair, clear contract and for a better mutual understanding between both parties, is entirely in line with the speaker's ideas on the subject.

[Mr. Thomson.] T. Kennard Thomson, M. Am. Soc. C. E.—The question of classification has come up frequently in foundations in New York City, especially in caisson work. In nearly all the lower part of the city, quicksand is found, from near the surface to the hardpan, from 30 to 60 ft. below, and, where caissons have penetrated the hardpan, it has been found to vary from 2 to more than 30 ft. thick, and, in some places, it extends to the rock. In other places, sand, boulders, etc., varying from 1 ft. to 30 ft. in thickness, are found between the hardpan and the rock. Occasionally, the hardpan contains an almost solid mass of big boulders, and sometimes only small pebbles. The big boulders generally have to be blasted. In some places the hardpan can scarcely be removed by the pick, and, in others, it is so soft that it crumbles readily in the fingers.

Architects in New York usually have "wash-borings" made. These are carried down to what is called on the plans "bed-rock or boulders," but which is, in reality, the top of the hardpan. In one case, a contractor looked at these plans and signed a contract agreeing to go to "bed-rock." Although he had to pass through from 5 to 12 ft. of hardpan, which the plans did not indicate, he went to "bed-rock," but lost $20 000 thereby.

Wash-borings generally indicate the top of the hardpan very accurately, but, as they are made for this class of work, they do not show its thickness, nor the elevation of rock, so that the only fair way to let a lump-sum contract is to specify a lump sum for excavation to the average elevation of the top of the hardpan, and specify that everything below that depth will be paid for at so much per cubic yard, which price should be increased, probably for each additional 5 ft. of depth.

In passing through the material above the hardpan, as much as 20 ft. vertical per day can be excavated, while below the top of the hardpan, generally the excavation will not be more than from 2 to 4 ft. per day, and in cases where there is much blasting, 6 in. is the limit for a day's work. The result is that the engineer can calculate very closely what it will cost to go to the top of the hardpan, as given by the wash-borings, and he can calculate the extra cost for each additional foot of depth; but, if he tries to distinguish between good hardpan and poor hardpan, or boulders, or the material often encountered in the hardpan or under it, he will get into trouble, for all these materials blend into each other and vary for every caisson on the work. Mr. Thomson.

Hardpan has been described as a natural concrete, in fact the speaker once heard two engineers disputing as to whether a piece of very good hardpan was or was not Portland cement concrete. As has been said, poor hardpan is sometimes so soft that it cannot be picked up without breaking.

E. H. Beckler, M. Am. Soc. C. E. (by letter).—This paper bears upon a subject to which the writer has given much thought in recent years. One's opinion, while attempting to discuss some of these questions, is certain to be influenced greatly by the point of view, and one's ideas are also likely to be affected by the vision. Everything is green or blue, when seen through green or blue glasses. To the best of the writer's knowledge, the eye-glasses he is wearing while attempting to discuss some of the points at issue are perfectly colorless. The writer, however, has no thought of being anything but loyal to the profession which stands first in producing men who are able in character and attainments, and whose success is never built upon the miseries or misfortunes of others. Mr. Beckler.

There are some conditions in agreements, and some requirements in specifications, which have become obsolete through age, changes in business relations, and by reason of the use of modern appliances and methods of doing work. Other drastic clauses, giving authority to the agents of one contracting party, are inoperative, except in cases of financial embarrassment, attempted wilful wrongdoing, or default; and their presence in the agreement injures no one who proceeds with the execution of the work in a spirit of fairness and uprightness. It is true that one's pride is sometimes touched, but the knowledge that sentiment is not business should bring relief. It is certain that the presence of objectionable features which will not stand the test of law can do no harm. The same conditions apply to sub-contractors, who are frequently men of less intelligence, with weaker views of integrity, than the principals; and it is quite likely that a principal, who objects to the stringent requirements binding him, feels, at times, quite thankful that somebody's forethought has placed restrictions which are benefits in disguise.

The nature of the business is such that the contractor's pecuniary interests are closely represented on the work. He is in daily contact with the agent of the railway company, and is ever ready to present his side of a situation. The young men representing

Mr. Beckler. the railway company are, perhaps, too much taken up with the engineering and structural work, and are too inexperienced to comprehend and realize the judicial functions of the position. The specifications are for the guidance of such men, and it is better that there be not too much looseness, for that may make trouble. It is expected that there will always be near at hand some engineer in charge, who knows what is fair between man and man, and to him the contractor will look, more than to the chief engineer. In these days, where there is an operating department, the near-at-hand duties of the chief engineer often prevent him from giving much personal attention to a new piece of construction, until he can ride over the work in a car. Men at a distance must be held in check.

It is quite desirable that there should be greater uniformity in the general requirements, as contractors go from one road to another, acquire habits, and draw comparisons. When they meet unusual requirements, they are apt to say that the railway company is close, or even go so far as to say that they are being robbed, because of some variation from a custom practiced elsewhere. There are especial features with nearly every piece of work, and these should be specifically mentioned, in order to give assurance that the contracting parties knew the conditions, and were prepared to meet the situation. This prevents distrust and, possibly, litigation.

Method of Letting.

Lump Sum.—This method is without unit prices, or classification. It is possible that the information concerning a small piece of work may be sufficiently accurate and complete to enable fair tenders to be made. It precludes all modifications in plans during construction.

Unclassified Unit Prices.—The information for this method of letting must be nearly as complete as in the case of the "lump-sum" method. The usual clause requiring the contractor, in making his bid, in this and the foregoing method, to certify that his knowledge is obtained through his own agencies, if effective, compels much expense in the examinations; if waived, it is a foundation for a law-suit. Changes of grade or alignment make changes in the material encountered, in the character of the work, in haul, and in other ways, so that change in compensation must be allowed. Generally, changes operate favorably to the railway company. They are made for its benefit, are generally a saving, and, naturally, are against the contractor—in loss of profit, if for no other reason. For a no-classification bid, the contractor's risk appears to be larger, and the price is correspondingly higher, though there is a chance to catch the unwary or irresponsible. The claim that it avoids disputes is not substantiated. The study of the character and attainments of the engineer in charge should be a large factor in preparing a tender on this plan. Mr. Beckler.

Classified Unit Prices.—The writer agrees most heartily with the author in his preference for classification; he endorses his arguments and commends to the attention of all engineers the clearness with which this point is covered.

A talk at one time with an engineer brought out the idea that the classification covered a definite material, and bore no relation, which an engineer should recognize, to the schedule of prices. He claimed that it would be disastrous to class material by relative cost (when properly handled), where the description did not definitely apply. The usual specification refutes this argument by describing loose and solid rock, and then naming earth as "all other material of whatsoever nature," with no attempt at description.

It would seem best to divide all excavation material into three groups in order to average the range of cost, although sometimes the middle price may be omitted. There are certain kinds of material, such as loose and cemented gravel, hardpan, gumbo, and plastic clay, which cannot be excavated at the usual earth prices. The unfortunate contractor who encounters these materials, and asks the engineer for a suggestion for economical excavating, or a hint as to the probable classification, draws little comfort from the reply.

As the author has well said, it is not difficult to describe the material most easily reduced to the loading condition. There is not much trouble in determining rock as such, although the writer once classified a frozen sand bank as rock, much to his chagrin a few months later. More mature thoughts have, in his mind, justified the unintentional benevolence. The portion of a ledge or mass of rock which can be removed economically without blasting is trifling. The seamy disintegrated capping is more troublesome than the solid mass beneath, and it is better not to attempt to separate it in the classification.

The words, earth, loose rock and solid rock, in the classification, are misleading. They prevent many, and especially young engineers, from perceiving that classification is an attempt to fit prices to the character of the material as judged by its resistance to removal.

Classification.

First Class.—This should include material which, in its customary natural condition, can be plowed by a team of 4 horses, each weighing 1 500 lb., with reasonable rapidity for economical handling by scrapers or shovels, but does not require the use of picks or bars.

Second Class.—This should include earthy or mixed materials, Mr. Beckler. those not susceptible to plowing under the foregoing test, and boulders, embedded in first-class material, between the sizes of ½ cu. ft. and 10 cu. ft. Plowing with more than 4 horses, the use of picks and bars, and other hand work, and drilling and blasting are assumed to be some of the methods of loosening this material.

Rock.—This class should include all rock in masses larger than 10 cu. ft., and gravel containing cementing material and requiring the methods used for loosening rock. Drilling and blasting are assumed to be requisite for loosening and fracturing rock economically.

The foregoing leaves the opportunity to introduce a "third class," which might be necessary in some localities, to make the scheme more flexible. The price, with competitive bids, will fix the question of soft and hard rock. Under such a specification, the young engineers in the field will know that rock is rock.

Measurements.

The difference in volume between "end-area" and "prismoidal-formula" methods is due to the two pyramids in the slopes when there is a difference in end heights. The end-area method treats the pyramid as a wedge. This difference in volume may be found as follows:

Let S = the ratio of the base to the height,
D = the difference in the end heights,
L = the length of the prism,
X = the correction = the end-area volume minus the prismoidal-formula volume.

$X = \frac{S\,D^2\,L}{6}$ = volume for both sides for a level cross-section.

Substituting the proper values for

$S = 1\frac{1}{2}$ to 1, 1 to 1, $\frac{1}{2}$ to 1, and $\frac{1}{4}$ to 1;

then $X = \frac{D^2\,L}{4}, \frac{D^2\,L}{6}, \frac{D^2\,L}{12}, \frac{D^2\,L}{24}$, respectively, for these slopes.

It is seen that this correction varies directly as the length and as the square of the difference in end height. It is zero for any height of embankment equal at the two ends, and reduces to a small quantity as the slopes become steep in rock excavations. In earth slopes, short prisms make the error small. The difference in volume between end-area and prismoidal formula, with $1\frac{1}{2}$ to 1 slopes, for a 50-ft. prism, with 4 ft. difference in height, is 7.4 cu. yd. The error of one-tenth, in the cross-section height for an embankment, 6 ft. high and with a base of 20 ft., is 7.1 cu. yd. It is not likely that the slope stakes, on ordinary ground, are accurate to within one-tenth. The natural surface of the country is convex over the ridges and concave in the depressions. The contractors lose many times the prismoidal-formula correction in the convexity of excavations and the concavity of embankments. To be consistent in applying the prismoidal formula, the ground should be planed or leveled before cross-sectioning, and the rod should be read to hundredths. There are steep gulches where it is inconvenient or unnecessary to take frequent cross-sections. The formula can be applied, but the methods used for determining the middle area are inaccurate and tedious. It is better to use the end area, and apply the little formula, previously mentioned, for a correction to the volume. Since the error is all in the slope, it is only necessary to use the side heights. A volume table for D may be quickly prepared. Mr. Beckler.

Some specifications call for the measurement of all work in excavation. It seems to be a better plan to measure the embankment, when it is built from side-borrow. Frequently, the pits are not measured up for months. They become filled with mud and water, and when the quantities determined by pit measurement differ materially from the cross-section quantities, the engineer tries it again. He is delayed in rendering his final estimate, by this borrow-pit figuring, at a time when the contractors are impatient, and he is needed for other work.

Overhaul.

The question of haul is one of the most important under consideration, whatever plan is used, as regards compensation for the work, lump sum, unclassified, or classified unit prices. The exhibit shown at the letting, on which the compensation is based, cannot be changed, in right of way, grade, alignment, diversion channels, sidings, road-crossings, and incidental construction outside of the roadbed, without affecting the question of haul and the contractor's profits. It costs to move material, although not always in direct ratio to the distance or volume. The character of a piece of work may be so changed as to require a change of appliances for transportation at points where the delivery of the proper outfit is worth the price of the work.

On a "pay-both-ways" proposition, some cuts aggregating more than 100 000 cu. yd. were to be deposited in embankment where the ground was unfit for borrow. The line was shifted laterally to skirt a sidehill. A borrow-pit was made in the place of the original location from which material was hauled to the fill, thereby cutting out one price.

A contract was made for a line which had been located during dry, autumn weather. The free haul was 1 000 ft. The profile showed a wide right of way—secured for the purpose of side-borrow—at all places where hauls of considerable height were to be

Mr. Beckler. built. In the following spring most of the low places were filled with water, and remained so for months. The contractors were obliged to haul from the high places, at scraper prices, using cars for the work. The original estimates indicated 60% side-borrow; the final estimates gave 70% hauled. A protest to the engineer in charge brought a shrug of the shoulders, and it is doubtful if the matter ever came to the attention of the chief engineer by more than casual mention by the engineer in charge.

A contract without any haul price included a section of 4 miles with a cut at one end and a long fill. In sub-letting, this section was undertaken at a price greatly above the principal contractor's average price for the entire work. The chief engineer consented, afterward, to let the sub-contractor waste some 30 000 cu. yd., and then made up an estimate including both waste and borrow, thereby taking $2 000 from the principal contractors and more from the railway company. These things show that there is indifference or a failure to understand the railway company's obligation.

It is best to have a price for hauling all material based, preferably, on the cubic yard and 100-ft. distance. This should apply to all roadbed excavation (no free haul), and to all borrow hauled more than 200 ft. If the price for haul is sufficient, the minor changes incident to construction do not cause loss. In justice to the contractor, hauling should not apply to extra work, which is not in evidence at the letting, when the excavations are off the right of way and below grade.

The methods of calculating overhaul are quite varied. When center of mass is mentioned, there is always doubt in the writer's mind as to his understanding of what is meant. The lexicographers, in definitions of center of gravity, mass, figure, volume, magnitude, etc., have made some confusion. Some engineers have shown surprise when told that the point where they add up one-half the yardage in a cut is not the center of mass. The true center, being a product of a unit by a distance, is only the center of figure in regular solids. There is no occasion to determine the center of mass. Unless one wishes to go into calculus, it is sufficient to use the prisms extending from one cross-section to another as units. Consider these units to be hauled to some selected point, as the end of the cut; ascertain the point in the embankment to which the same volume will make the fill; determine the haul to each of the embankment cross-sections, until the above fixed point is reached. The sum of the cut haul and the fill haul gives the required overhaul quantity. By this means, interpolation to find how far a given unit in excavation will reach, in the embankment, is reduced to the one operation of determining the extreme limit of the haul. Some mass-diagram methods are inaccurate, and, practically, the above scheme is used for those which are correct, permitting the eye to assist and check the mind. Where a borrow is overtopped by an excavation, the overhaul should apply to the distance actually hauled, and, if the bank is built up in layers, the usual plan of placing the nearest cut in the nearest fill is unfair. A strip of ground is often bought for borrowing the entire length of a fill, but, in calculating overhaul, the engineer will put the cut in at one end and the borrow at the other. Mr. Beckler.

Sub-Contractors.

If the stipulation prohibiting sub-contracting is to be enforced, the engineer must let his work in shorter pieces. There are no outfits in existence capable of handling some of the larger pieces of work. Prices would go soaring, if one contractor had to own all the outfit. If there are several principal contractors on the same line, there will be continual squabbles over labor, supplies, etc. With sub-contractors acting as foremen, there is better progress and workmanship, and more economical management. While it is not nice to sign an agreement not to sub-let, and immediately proceed to cover a line with foreign parties, it is possible that a modification of the stipulation which definitely states that sub-contractors may be employed, and that the principals will be expected to shoulder all the expenses, of whatsoever nature, that the often-misguided "subs" may choose to incur, may be like "jumping from the frying-pan into the fire." It is better to "let silence give consent."

Insurance.

Sometimes the clauses relating to the protection of the railway company are worded in a way to touch the sensibilities of a self-respecting man. Except for the fact that the work is more hazardous, there is no occasion for greater restrictions than in the purchase of material and equipment. On account of the hazard, it is doubtless best that the railway company insists that accounts shall be properly kept. The inspection of the books and pay-rolls, and the assumption of management of the contractor's forces, are not the company's business until there is evidence of incompetence and probable failure.

Delays.

Contractors are entitled to compensation for delays occasioned by the other party. Extension of time does not pay for feed and idle labor. The claim for delays ought to be made promptly, and its merits determined immediately by a competent person. A tender was once made for a piece of work on which the contractor was to load cars to be hauled away by the railway company. The bidders

r. Beckler. proposed having a timekeeper, employed by the company, to record idle time after any 15 min. delay for cars, and payment was to be made for the time less the 15 min. The bid was rejected, because, as the chief engineer said, it left a chance for dispute. The object was to avoid dispute. The party who took the contract, without such provision, afterward told the writer that he had lost money because of the delays.

The Chief Engineer as Umpire.

There is some probability of the chief engineer being biased in his opinions, on account of estimated cost and pressure by the management. There is greater probability of the contractor assuming this to be the case, because the engineer is an employee of the other party. This opinion is more likely to be the fact, as regards the subordinate engineers. It is quite likely that many chief engineers would like to have the duty of being umpire taken from them. It would be well to have every contract state that some engineer, not an employee of either party, shall be the umpire to decide all questions pertaining to changes made during the progress of the work, or to adjust all claims of contractors for extra compensation. The compensation for the service of umpire should be borne equally by the parties. The chief engineer is frequently prevented from making changes because of the uncertainty as to the feeling of the contractors, and, if it were known that an umpire was agreed upon to settle any question of compensation, the engineer would be less troubled in arranging for work introduced by unforeseen difficulties. It is best that much of the extra work not provided for in the contract at unit prices, or work not plainly indicated in the beginning, should be done at cost, with a percentage for the use of tools and superintendence. Such bills should not be juggled into yardage, especially when the work is done by a sub-contractor. His price is not that of the principal, and he is entitled to the full amount of his bill.

Some of the objectionable features of contracts, from the contractor's point of view, have been inserted by the attorneys directly in the employ of the railway company, who have been told to draw up something binding. The contractor is surely at fault for signing an agreement containing conditions which are humiliating, or which are too one-sided. Such matters can be made satisfactory before the papers are signed. It would be well to have the railway contract forms of agreement and specifications revised, like the American protective tariff, by its friends, that is, engineers and engineering contractors; and a committee of the American Society of Civil Engineers is the proper vehicle to use in making this needed reform.

Mr. Bouton.

Harold Bouton, Assoc. Am. Soc. C. E. (by letter).—The scope of this interesting and important paper is too great for a thorough discussion by the writer. Many of the questions raised require a knowledge of the law of contracts as well as the facts. Many of the evils and defects of which the author complains, however, are only unfair, though a number are illegal; but whether unfair or illegal, they are expensive to the employer, as the author points out. As long as contractors must take contracts as they find them, and as long as competent parties are allowed to contract to do anything not illegal or against public policy, until then will contractors, who are alive to their interests, rely on ambiguous clauses and engineers' interpretations thereof to relieve them; and they will liquidate their damages by resort to a court, or insure against such damages by a high price, or both. Contractors who do not bid high under such conditions are, as Mr. Dennis says: "not the most reliable, but the most optimistic."

The writer is familiar with the subject, and, having considered it from several points of view, the causes of the defects in the present system seem to resolve themselves into two general classes, which would seem to be confirmed by this paper:

1.—Insufficient or unreliable data for the inspection of bidders, and by which they are to be bound;

2.—Lack of definition and limitation of the province of the engineer in drafting the specification and the contract.

Economic reasons would seem to be sufficient to prompt every competent engineer to make the circumstances and conditions affecting each contract as certain as possible for the inspection of the bidder; at least, when these have been, or can be, made reasonably certain. Lack of time or funds is the excuse usually given, but the time devoted by the average engineer to drafting the contract in supposed legal form will generally refute this, and the same time spent in intelligent investigation and collection of data would render unnecessary the usual attempt to shift the responsibility for not having done so; and the first cost of precision will bear favorable comparison with the final cost of uncertainty.

As Mr. Dennis says, general clauses are a confession of weakness. Precise, clear statements need no definition or protection. Ambiguous and uncertain clauses, inserted to avoid the legal consequences of uncertainty or worse, need definition, and usually get it, at the expense of the employer.

There is a reason for the insertion of these clauses which, unfortunately, often controls. It arises from a desire to confirm a preliminary estimate of the project, made on a superficial survey of the location, facts and circumstances, and on the basis of the

Mr. Bouton. "average price of similar work," at a time when the authorization for the work depends on the estimate.

Improvement can hardly be expected until the fundamental differences and essentials of cost and price are more generally recognized and satisfied.

Limitation of the province of the engineer is perhaps the most important topic of discussion suggested by this paper, particularly because of its very declared purpose and its title.

Of all defects in the present system, the usual method of drafting contracts and specifications is the most glaring and the most productive of litigation. With insufficient or withheld data as a foundation, a scrap-book structure is erected from other specifications and contracts, themselves seldom original, which have been drawn to satisfy work of a different character, or at least affected by radically different circumstances and conditions, and in another section of the country; and great effort is made to satisfy legal as well as engineering requirements.

This fault, no doubt, arises mainly from the fact that the technical knowledge necessary to draft a contract and specification is usually beyond the province of the lawyer, as it varies with and in each contract; but the fundamental essentials of the law of contracts, necessary to make it binding upon the parties thereto, are generally constant. The error results from not segregating the matters of fact from those of the law. The engineer and employer should agree on the facts, and separate the technical from the general. The technical will vary with each contract. The general facts will be more constant, but will also vary to some extent. It is feasible, however, to compile a schedule of general facts incidental to the average engineering contract and place it in the possession of every engineer, so that a choice of these may be made for each new contract, with the addition of any other facts which the circumstances may suggest. When selected by engineer and employer, they should be referred to one who is qualified to draft them in legal form; but to go further than to standardize the facts of general clauses would be a grave mistake.

There remains that portion of the contract which engineers should not try to draft, and that is the purely legal part wherein is recited and defined the parties, general subject matter, and consideration. The effect of a standard form of agreement, even if this were made with legal assistance, would be disastrous, for many would assume its infallibility. Such a standard form would not satisfy even all railroad contracts, and if insertions or other changes were made by a person not conversant with the fundamental essentials of a contract, the result would be very dangerous

Mr. Bouton.

and unfortunate. A knowledge of the law of contracts is a desirable qualification in every engineer; not for the purpose of drafting contracts, but that he may understand the importance of clear, concise and exact terms, and avoid the use of clauses so general as to seem to be designed to permit the engineer to avoid any consequence of his own neglect or error.

The interest and discussion, when these questions arise, show that the study of legal principles is taking the place it should in an engineering education. It seems ridiculous to attempt the legal without a knowledge of the successive relations in a contract, namely: Formation, Operation, Interpretation, and Discharge. Further, there is no contract unless the following five essentials of Formation are present and are satisfied:

1.—Offer and Acceptance.
2.—Form and Consideration.
3.—Capacity of Parties.
4.—Reality of Consent (Mutuality).
5.—Legality of Object.

An admirable discussion of these questions may be found in the paper* by Albert J. Himes, M. Am. Soc. C. E., but a brief reference to each of the contract relations may be excused because of their extreme importance.

Formation.—This corresponds to the Proposition and Execution of the contract, and is written evidence of the limits to the rights and obligations of the parties. Its successive requirements are:

I.—That the acceptance must be in the precise terms of the offer, and unconditional;

II.—That written form is necessary, and the consideration real, though not necessarily adequate;

III.—That the parties must be not only legally and mentally capable, but acting within their authority, if representatives;

IV.—That, for mutuality, the minds of the parties must have really met on the same subject-matter;

V.—That while legality may seem self-evident, regulations or ordinances, charters, statutory and other law, unknown and unsuspected by an engineer, may result in the illegality of the contract.

Operation.—Operation, corresponding to Performance, tests the limits of the rights, liabilities and relations of the parties and of the subject-matter; including the very important ones of sub-contracting, sureties and bankruptcy.

* *Transactions*, Am. Soc. C. E., Vol. LVI, p. 104.

·. Bouton. *Interpretation.*—Interpretation means definition, and is regulated by the intent of the parties, as shown by the written instrument and the surrounding circumstances; but, while parol evidence may explain, it cannot vary or modify the written terms.

Discharge.—Discharge or Termination may take place by waiver, substituted agreement, substantial performance, breach, impossibility of performance, and by operation of law.

The important question of classification, to which the author devotes much of his paper, deserves consideration.

The classifications, "Earth," "Loose or Soft Rock," "Solid or Hard Rock," as the author says, seem to be founded on a desire to isolate in each of these classes those materials for which the unit cost of handling is approximately the same. The author then proceeds to make his suggested improvements on a similar basis, though the paper, in its analysis, shows that while the factors of cost and price include that consideration, others are quite as important if not controlling. There seems to be another reason for the present system of classification. It is that of uncertainty and a desire to limit to some class, materials for which the engineer feels unable to estimate the cost; for example, "quicksand," in the "Earth" class.

Believing that the system is fundamentally wrong, the writer cannot agree that the author's suggestions are an improvement. Also, it does not appear to him that the suggestions for determining the class will work satisfactorily in practice, or lessen the usual friction and differences of opinion. The author's suggested classification is, indeed, on a basis of loading condition, and approaches closely the true economic relation; but his method of ascertaining this condition would seem to accomplish nothing more than could be done by tests more scientific in principle and quite as practicable. Teams and plows do not seem to be in accord with the present rigid tests for many materials. "Horse power" is no longer recognized as a measure of the working force of an average horse; and how much less will "two men working to keep one man busy" measure a constant standard of work, when so much depends on their strength, temperament, inclination and other characteristics? Nor is there any accurate measure of the power of explosives to overcome the resistance of materials. Should there not be as fair and exact tests and definitions of materials for destruction furnished by the employer, as of materials required of the contractor for construction? Any improvement should provide more definite information for the bidder, lessen the ambiguity as to what price will be paid, and thereby lower the price.

The present system of postponing quality as well as quantity of all excavation materials, until after the execution of the contract,

Mr. Bouton.

induces honest differences of opinion which tend to weaken the mutuality; and it is because Mr. Dennis proposes to continue what seems to be a fundamental defect, that the writer disapproves of his recommendations, even if his criteria are practicable.

A responsible contractor, whatever the classification specified, endeavors to determine exactly the quality, quantity, location and disposition of the material, and then, according to his resources, experience, and the specified and other conditions, he will add to his estimated cost an amount which he hopes will include a profit. He looks for cause first and at effect next; but the engineer asks for bids on effect first, and ascertains and defines cause later. The contractor's time to investigate is usually short, and if the data provided by the engineer are insufficient or unreliable, and his own examination is unsatisfactory, the uncertainty is paid for in the price. The price of the usual class is not even an average of quality, though the materials are reasonably ascertained, for the party who pays does not determine their quality at the time the contract is executed; and, instead of paying a definite price for definite materials, the employer more often pays a price safe to bid, on what may prove to be a group of unlike materials, the quality and quantity of which the engineer will ascertain and determine after the price is made.

Rock formations, for which the usual classification strives to produce a lower price, might be said to be confined to very definite kinds of deposits, generally speaking, stratified rock and those which are imperfect because they are in process of formation or disintegration. The railroad classification clauses quoted by the author refer—by name and character of deposit—to well-known materials, as criteria of the very quality they attempt otherwise to define in the same clause, and thus show the futility of trying to classify quality. If a material is named, its quality is at once apparent to a person of average intelligence.

Geology provides a classification that is standard throughout the world and satisfies every requirement of practicability, exactness and inflexibility. The names given to individual rocks and deposits of Nature are so invariable that their physical, if not their chemical and mineralogical, characteristics are known to most parties likely to be interested, and further information regarding them can be readily obtained.

With this as a natural, scientific and established basis, and also the very one by which the contractor consciously or unconsciously determines costs, the writer, without assuming to tabulate all the materials that should be found in each class, proposes the following scheme of classification:

Soil.—Soil shall be rock or like material, the particles of which

Mr. Bouton. are not cemented together, but are in a granulated or powdered condition when dry; and shall include material, wet or dry, as found in its natural state. The various soils shall include sand, earth, clay, gravel, muck, gumbo, quicksand, and others that satisfy the definition in this clause.

Indeterminate.—Materials in the indeterminate class shall be rock, or like material in process of formation or deterioration, the constituent particles of which are sufficiently cemented together to support their own weight without breaking or cracking either when dry or wet; also, rock of any kind found in masses exceeding 1 cu. ft., but not exceeding ½ cu. yd., in volume, as a result of fracture or any other natural cause. The materials in this class shall include indurated clay, cemented gravel, hardpan, weathered, decayed or otherwise deteriorated rock, boulders or fractured rock not exceeding ½ cu. yd. in volume, and others satisfying the definition in this clause.

Rock.—Rock shall be mineral masses composed of materials, the particles of which are in a state of perfect cohesion, as usually determined geologically for its kind, and not yet deteriorated to any appreciable depth from any given natural exposed face. They shall be classified generally as follows, for masses exceeding ½ cu. yd. in volume:

I.—Of physical character similar to that of deposits known geologically as aqueous, including shale, slate, sandstone, limestone, etc.

II.—Of physical character similar to that of deposits known geologically as metamorphic and igneous, including trap, granite, gneiss, syenite, marble, mica-schist, and some sandstone, etc.

Extraordinary.—Extraordinary shall be special work of any character to be defined as occasion arises, before letting the contract if possible, and paid for on a basis regulated by the circumstances. For example:

Foundation.—The foundation shall be of a character preliminary and consequent to the placing and location of permanent work and materials below or above the average water level of the site, and shall include all false-work, materials, labor, and the use of plant and appurtenances that the contractor deems necessary, provides and uses, for successfully completing and maintaining the conditions of preparation and location of permanent work built and left as finished at the particular site in question. Foundations shall be classified as:

(*a*) Dry—Above water level;
(*b*) Wet—Below water level.

On this basis, a schedule should be formed from the country rocks and soils expected or likely to be found at the location. If the rocks and soils are of unusual character, the schedule should include a reference to materials usually found elsewhere, as a basis of comparison. Under the general heads just given, each material should be mentioned by name in its proper place, and a bid should be asked for each separate item scheduled. For each or any material which may be found in course of excavation, but which is not expected, or is of doubtful quality, it should be specified that it shall be placed in one of the general classes and be paid for at the price bid for the material or rock which it most nearly resembles. It should be specified that disputes shall be settled by submitting standard samples to a third competent party for such classification, or, if necessary, by tests of the same general character as those now provided for natural and artificial rock, stone and materials used in construction, as building stone, sand, cement, concrete, brick and the like. The standard tests might include and provide for a selection of the following for each material or rock: Heat, absorption, specific gravity, fracture, attrition, tension, compression, cohesion, adhesion, formation and other physical, chemical and mineralogical attributes. Mr. Bouton.

The test of heat, absorption, compression and tension would be likely to remove any reasonable doubt as to whether any material—such as hardpan—should be classed as "Soil," "Intermediate," or "Rock."

It might be advantageous, in many instances, to use Class 1, Class 2, etc., or Class A, Class B, etc., as alternative headings for those given in the suggested classification herein; but, whenever the present general practice of classification is used, classification by numbers or by letters is far preferable to the terms "Earth," "Loose or Soft Rock," and "Solid or Hard Rock." The latter terms, almost invariably, designate a classification including materials distinctly different, and the continued use of such terms is certain to perpetuate the past and present difficulties of interpretation.

Rigidity is not suggested, and modification will be necessary in each particular instance; but the advantage of a certain, established, reasonable custom, in the rules of evidence, cannot be overestimated.

How far this or a similar classification may have been advocated, or adopted and rejected as impracticable, in the past, the writer is unaware, but will be pleased to learn.

Samuel Tobias Wagner, M. Am. Soc. C. E. (by letter).—The writer is decidedly in favor of an unclassified price for excavation, under most conditions of a general character, and more particularly where municipal work is concerned, or where a municipality is interested in the contract. Where such price is unclassi- Mr. Wagner.

Mr. Wagner. fied, he considers that the contractor is entitled to all the data which can reasonably be obtained by the engineer. General plans showing the roads, railroads, and means of access to the work, and profiles showing the cuts and fills, and the approximate distribution of the material, should be given. Water-jet borings should also be made at reasonable intervals. While borings of this class, as a rule, are very unsatisfactory for showing the character of the rock formation, they can be used to show a reasonable line of demarcation between earth and rock, and thus furnish a basis by which the contractor, in making up his bid, can determine the relative proportions of each.

The writer has in mind a large piece of municipal work executed under his charge, involving about 1 000 000 cu. yd. of excavation, on which an unclassified price was used, and on which frequent water-jet borings were made. As the work progressed, the borings were found to show, almost invariably, the approximate line of the rock, and the contractor was able to make a price which was equitable to both parties.

The compilation of the practice on different railroads, made by the author, is interesting and startling in the differences shown in the specifications for the classification. On the road with which the writer is connected (The Philadelphia and Reading Railway), the practice varies. When classified, the following specification is used:

"Excavations shall be classified under the following heads: Solid Rock, Loose Rock, Earth, and Excavations in Water.

"Solid rock shall include all rock found in ledges and in detached masses, exceeding 1 cu. yd. each, which, in the judgment of the Chief Engineer, may best be removed by blasting.

"Loose rock shall include all kinds of shale, soapstone and other rock which can be removed by pick and bar, and is soft and loose enough to be removed without blasting, although blasting may occasionally be resorted to: also detached stones of less than 1 cu. yd. and more than 1 cu. ft.

"Earth shall include all materials of an earthy nature, of whatever name and character, not unquestionable rock, as above defined.

"Excavations in water shall apply to foundation pits under water, and deepening of channels in running water; it shall include drainage, bailing, pumping and all materials and labor connected with such excavations: also the necessary dressing of rock for the base course of masonry. The materials removed from excavations in water shall be classified and determined in the same manner as materials removed from other classes of excavations, *viz.*, as solid rock, loose rock and earth."

The writer fears that the classification proposed by the author will be very difficult to apply practically, and will not simplify the question.

Mr. Wagner.

One trouble with classified work is the difficulty in verifying the actual conditions if a dispute arises after the work is done. This calls attention to the necessity of the engineer and contractor taking up and settling on the ground from day to day such questions, and not waiting until the progress of the work has obliterated all traces of the actual conditions.

It is a great mistake to include, in any price bid by the contractor, anything more than the actual items which are distinctly appurtenant thereto, unless the specifications can indicate precisely what is to be included. Even then, in most cases, it is better to have in the proposal special items for the extra work. Nothing is more unfair than to ask a contractor to do something which cannot be distinctly shown on the drawings or specified in advance.

The writer does not believe in extra work—that is, force-account work—where it can be avoided, but in some cases it is the only fair method to be pursued. In such cases it would seem to be wise to have, in the proposal, prices agreed upon to be paid for the several classes of labor, and if the quantities are checked daily by the engineer and contractor it is certainly an equitable method of procedure, and open to no serious objection. In such cases it should be understood that such prices include a necessary percentage for the use of ordinary tools, cost of superintendence, and profit.

The writer thoroughly agrees with the author that nothing should be asked in a specification which is not practicable in the work. Such clauses should be omitted by all means. Many clauses of this kind creep into specifications because they have been in previous ones, and their presence is due to the shears and paste pot more than to anything else.

It is unfortunate that much work has to be put under contract without the preparation of thoroughly digested plans and detailed specifications, but conditions often require such work to be done. Often, too, the right of way in certain localities cannot be obtained before the work is put under contract. In such cases, the responsibility of the engineer, in determining the equity between his employer and the contractor, is very great. There are few engineers who would not rather wait until these points are cleared up before asking for bids, but too often the conditions will not allow it.

Some specifications require the contractor to be responsible for any damage which may occur to property, etc., during the progress of the work. This clause has interested the writer for many years. His experience has been that, no matter how the contract be worded, so as to make the contractor responsible, the courts will not hold him so, if it can be shown that he has carried

. Wagner. on his work in the usual manner in which such work is carried on by a good and successful contractor. It is necessary to prove negligence in order to hold him, and such proof is not easy. The writer is of the opinion, therefore, that it is better to omit such clauses, direct the contractor how to proceed in detail in cases requiring care, and take the responsibility, than to retain the clause and have the contractor add a contingent item when making his bid in order to cover imaginary cases which may never occur.

The author is to be congratulated on the preparation of a paper so rich in problems which are of frequent occurrence, and it is to be hoped that the discussion will be fruitful in clearing up many points upon which an improvement can undoubtedly be made.

Ir. Vandevanter. C. O. VANDEVANTER, M. AM. SOC. C. E. (by letter).—It seems unfortunate to the writer that there has not been more discussion of Mr. Dennis' valuable paper from the engineer's viewpoint. The writer has long been dissatisfied with the classification used in the ordinary specifications. In the early days of railroad construction, when earth was loosened by picks or plows, and explosives were only used where rock occurred, a classification based on the use of explosives may have been satisfactory, but the writer's experience has been that progressive contractors of the present day use explosives on all classes of material, even where they themselves would not claim a higher classification than earth.

The method of determining classification, proposed by the author, would seem to the writer simply to shift the authority from the engineer to the contractor. It will hardly be disputed that there is as much difference in laborers as there is in the material to be classified. Without considering his loyalty to his employer, self-interest would cause a foreman to try to get as high a classification as possible, and, in consequence, it would be only natural for him to select his worst men for pickers and his best for shovelers, if thereby he could raise the classification of the work done by him. Then, as pointed out by some of those who have discussed this paper, the plow test is incapable of being applied in a great many cases, and is not reliable even where it can be used. The writer has seen four horses, seemingly straining to pull a large railroad plow when being dragged back on its side and not cutting any furrow at all. It seems to the writer that classification, if used at all, should depend on some characteristic of the materials themselves. Of course, the different grades, no matter what method of determining them may be adopted, will often merge into each other so gradually that it will be impossible to determine a definite line of demarcation. Under such circumstances, the writer has been accustomed to fix points on his cross-sections, as close together as he thought would undoubtedly show each class, and then take a point midway between them as the one limiting the different materials. Of course, where the materials are mixed up so that the different classes cannot be shown on the cross-sections, and a percentage method is the only way to arrive at the quantities included in the different classes, it is reduced to a question of judgment, and it is not apparent how this could be eliminated under any method of determining the classification. Mr. Vandevanter.

The specification so often used for loose rock—"All shale, slate or other rock that can be removed by pick and bar without blasting, although blasting may be occasionally resorted to"—has always seemed to the writer to be meaningless. Why are slate and shale selected as samples? It is undoubtedly true that some slate and some shale can be removed without blasting, but this is also true of gneiss or schist, and the writer has met with both slate and shale which fulfill all the requirements of solid rock. The difficulty in using such a specification is to determine what is meant by "can be removed." It would hardly be contended that it means "when it is possible to remove the material with pick and bar without considering the rate at which this can be done," as, under that ruling, almost all, if not all, thinly stratified rock would come under the loose rock specification. If some rate is necessary, then what rate? If the classification is made to depend on the rate at which work is done, then the question of management comes in, and the contractor who has the most efficient and best handled forces gets the lower classification.

It is not clear to the writer why there should be a separate specification for dry foundation excavation. Usually, in excavating for pipes, and for shallow pits for masonry, the material is simply cast outside the pit, or, at the worst, is a short wheel-barrow haul, and would seem to be one of the cheapest excavations on the work. Wet foundation excavation, however, is different, usually requiring two or more lifts of the material or some special arrangement for handling, in addition to the increased cost of loosening and loading the material, and provision for taking care of the water. This classification should include all excavation below water level, whether in foundations or the changing of streams.

In regard to the use of the prismoidal formula, the writer would go even further than the author, and say that he thinks the quantities gotten by averaging end areas, supposing of course that the cross-sections have been properly taken, are more nearly correct than they would be if the prismoidal formula were used. The latter is based on the assumption that the figure is bounded by planes, which is not the case in earthwork. Nature always uses curves instead of straight lines, and, taking into account the fact that one side is bounded by a curved surface, the prismoidal formula will be

Mr. Vandevanter. found to give quantities which are too small, both in excavation and embankment.

In reference to sub-contractors, the writer's understanding of the clause, usually inserted in specifications, is not to prevent the contractor from sub-letting the work if he desires, but to make the relations between the company and the sub-contractor that of a foreman for the contractor. If the company is to recognize and deal with each sub-contractor, it is not apparent what benefit it would receive from the percentage the sub-contract pays the original contractor. Under such conditions, it would be as satisfactory and more economical to let the work in smaller contracts; nor would the company seem to have, should they authorize and legalize the sub-contracts, the same right to require the contractor to give it the benefit of his experience and abilities, by staying on and supervising the work, for which it is paying him 10% or more.

As for the authority conferred on the engineer to interpret specifications, decide classifications, etc., it is absolutely essential that this power be conferred on someone, and there would seem to be no one as well qualified as the engineer in charge of the work, who has no direct interest in the cost. The fact, admitted by the author, that the arrangement usually works satisfactorily, would seem to indicate that it is not an unreasonable one. The powers conferred by some specifications may be too broad, but, in the writer's experience of more than 30 years, he has never known them to be exercised arbitrarily.

The writer will be glad to see the Society take up this question if thereby a set of specifications can be obtained which will be clear as to their meaning, and of universal application.

Mr. Dennis. W. F. DENNIS, M. AM. SOC. C. E. (by letter).—Since the presentation of this paper, the writer's attention has been called to the "Manual of Recommended Practice," published by the American Railway Engineering and Maintenance of Way Association. On page 16 of that manual the following will be found:

"17. All material excavated shall be classified as 'Solid Rock,' 'Loose Rock,' 'Common Excavation,' or such additional classifications of material as may be established before the award of the contract.

"18. Solid Rock shall comprise rock in solid beds or masses in its original position which may be best removed by blasting, and boulders or detached rock measuring one cubic yard or over.

"19. Loose Rock shall comprise all detached masses of rock or stone of more than one cubic foot and less than one cubic yard, and all other rock which can be properly removed by pick and bar and without blasting; although steam shovel or blasting may be resorted to on favorable occasions in order to facilitate the work."

Common excavation is everything else.

As far as the writer knows, this specification is the only one having the endorsement of an engineering society. Its present form is doubtless the result of its having been proposed, discussed in committee, reported to the association, and adopted; its authority, therefore, is high. Mr. Dennis.

Upon the point of clearness, the writer finds in this specification little to criticize. Very ordinary experience would differentiate loose and solid rock, and what else there may be is "common." At the same time, this carefully weighed specification still turns upon the opinion of a fact: it refers to rock, "which may be best removed by blasting," and to loose rock, "which can be properly removed by pick and bar," and, therefore, again illustrates the difficulty, freely acknowledged by the writer, and freely commented upon in the discussion, of fixing the intangible gradations between the materials.

The writer, however, thinks that this specification is wrong in principle, in associating too many kinds of material in the "common" classification.

The trouble with the "everything else" specification is that, on the average, it means "earth;" that earth means, to the contractor, a material equivalent to one reasonably plowable; and prices are made and accepted with that implied understanding; and whenever occasional material is actually found to be of a different nature and more expensive to handle, the "everything else" prevents an assignment of the occasional material to a classification more nearly representing its cost. If all the material in a certain section were known to be hardpan, the cost of it and probably the price would be the same, whether the material were called loose rock, hardpan, earth, or common. Further separation of the "common" excavation is needed to apply to particular parts of the line having material costing more than ordinary earth, where the price for "common" excavation, on the average, would apply only to ordinary earth.

The writer cannot endorse too strongly the demand by several discussors for complete information, in order to assist the contractor in making up his bid, particularly in the case of unclassified excavation. Mr. Lavis' specimen profile (Fig. 1) contains the kind of information needed, and the writer feels free to state that, in the last twenty years, he can recall no profile information submitted to bidders which shows so clearly what has been planned by the engineer for execution by the contractor. Nevertheless, to the writer, this profile shows clearly the inelasticity of the "no classification" as against the "classification" method. Let the engineer study the method of handling the work; let him test the material; let him design the profile with reference to the probable appliances by which it will be excavated, the utility of which is clearly shown

Mr. Dennis. later in Mr. Lavis' discussion; after all, he has not done the work, and he would never do it precisely as he has blocked out. Furthermore, in the preliminary study of the work, he has to get some idea, at least for estimating purposes, of the relative cost of the different classes of material. He cannot reasonably jump at an unclassified price, any more than can the contractor, and, like the contractor, he must perforce classify hauls and material, and work out an average which finally appears as an estimated unclassified price.

Without a knowledge of prices for materials, obtained by observed classification, how can he do this? What is the logical reason why the method on which the unclassified price is based is not equally good for paying? The writer believes that, in a section where the material is mainly of one class, the unclassified price, based upon reasonable information, is of benefit in saving detail and argument; but, where the materials are much mixed, and are not clearly differentiated in location, the actual work, with the changes inevitable even after painstaking planning, will produce an inequity, either to the contractor or the company, in the working out of an unclassified price.

Referring to the claimed inelastic quality of an unclassified price, consult Mr. Lavis' profile. In the piece of work shown, cuts, swells, and borrows are estimated to make up an estimated embankment. The writer does not think Mr. Lavis will claim that these calculations will be verified even approximately by the results. Suppose it is discovered during construction that the fills are subsiding and sloughing very freely? Suppose the management decides to cut out the trestle at Station 63, and have a solid fill; that line and grade cannot be changed, and that the profile fills of 151 000 cu. yd. become 250 000 cu. yd., and that, therefore, inferentially, the borrow increases by 100 000 cu. yd.? What will be done to balance the actual with the preliminary?

It is perfectly evident that the contractor in estimating would multiply each cut and borrow quantity by a figure representing his idea of the price of each, including its haul, and would get the average price by summing the products and dividing by the number of yards affected.

Under the change, will he be allowed the cheapest borrow and haul? Will he be forced to take a long haul at the farther end? Will he be forced to double-track the section? Will the engineer be called upon, under a clause of the contract, to investigate and report a change in the contract price by reason of the work being rendered "better or worse than anticipated from the information furnished?"

Suppose the engineer throws an earth excavation line across a valley, and substitutes a through rock cut? All contractors have seen such changes; they have to be adjusted. Mr. Dennis.

With classification and haul applied to the work, changes create no question to be settled outside of the contract. The writer maintains that the legal proposition is that an unclassified price holds in the absence of material change; but let there be change of quantities and distribution, and the contractor cannot be compelled to execute the work; or, in other words, if he be of the "hold up" class, he has an exceptional opportunity; and, without that disposition, he must of necessity often be entitled to seek adjustment, and yet the contract does not provide a principle by which to do it.

In the writer's opinion, the whole question comes to this: First, on average work and with average information, about equal chances for legal difficulties exist, whether the work be classified or unclassified. Second, classification and haul permit free readjustment and change during construction, without changing the original equities of the contract. Third, the real objection to classification is reduced to two heads: increased work and experience required on the part of the engineers; and the incitement to discussion and controversy about the classification. Fourth, the latter objections are correctable in large part by greater uniformity, justice, and particularity of specification.

There must always be some inexactness, but, for work as a whole, there is need for no controversy about the greater part of the material. The differences on the disputed point will bankrupt neither side, and disputes are not necessarily hurtful.

Just here the writer will comment upon the following sentence in Mr. Lavis' discussion, referring to profile information:

"Of course all this information is given for what it is worth, and it is believed that there should be no more difficulty in a railroad company protecting itself against any claims of a contractor that he was furnished with misleading information, than the Board of Water Supply, for instance, finds in protecting itself because a contractor might be misled as the result of a wrong interpretation of the information shown by the diamond drill borings."

If the information is correct, and the contractor misleads himself, there is no controversy; but suppose that the information itself proves to be wrong in the working out? Invitations give "Information," with the expressed purpose "for the information of bidders," and then say that the principal does not guarantee the correctness of the information. If the principal furnishes the information and then benefits by a price bid in accordance with the acceptance of that information as true, no reservation, contract, or contention should seek to make the contractor the financially guilty party, if the information is unreliable. The principal, in common honesty, should stand good for his information and acts.

Mr. Dennis. Referring to comments upon overhaul, the agreement of several gentlemen in this discussion has been to the effect that the question is puzzling and perplexing to the engineer, and would therefore better be left to the contractor to work out. Other writers frankly admit the overhaul principle, and show the legal effect of changes.

Coming back to the plow test, Mr. Gillette and others cite instances where this is uncertain in application. This is freely acknowledged, but is it better to throw out the advantages of classification, if such be genuine, or to struggle with the same indeterminateness in a plow test? The writer does not feel that the question, of whether material is plowable with a given plow and team, is insurmountable in practice. The majority of earthy material clearly is plowable; some of it clearly is not. The proportion of it on the dividing line is bound to be inconsiderable. Objection has been made on the ground that, by reason of location, much earthy material cannot actually be tested by the plow. This objection is correct, and it was sought to cover this by saying "equivalent to" plowable earth. Surely, in the long run, no damage can be done by opinion as to whether these relatively small quantities of untested material do or do not correspond to others which can be tested. Whether the test be by a designated plow and team, or by a measurement of the quantity plowed, is immaterial, provided the basis be stated clearly, if measurement is to be the test.

Several of those taking part in this discussion advocate classification, but bring out strong reasons why the customary rock, loose rock, and earth nomenclature should be replaced by a segregation of the material into classes by number or letter. In each case, the material itself is supposed to be defined explicitly, and the name by which the definition is labeled is essentially nothing but a reference word. The nomenclature, however, is admitted to be worthy of discussion. One reason for holding to the customary terms as class labels is that the general use of these terms is established in the broad lines, and, in seeking improvement, the line of least resistance would be to improve first the essential things.

In conclusion, the writer feels that the original paper is somewhat disconnected, and the discussions of the several subjects were more discursory than he would like, but they are illustrative of the cardinal thought, that the common form of railroad contracts contains crudities and principles which should be threshed out. Fair consideration should be given to the manifest difficulties of both sides of the agreement; and the instrument which forms the legal basis of work costing hundreds of millions of dollars every year is worthy of any study looking toward its perfection. The writer's comments, suggestions and arguments were not so much to establish an opinion which he individually might have, but to show that there are arguments and questions, and, by inference, that the ordinary form of contract is an instrument worthy of attention. Mr. Dennis.

Mr. Bouton brings out the point that there are portions of the contract and specifications which the engineer alone should write; and that there are other parts which, in each individual case, should be turned over to the lawyers; therefore, he argues against a standardized form. The writer does not reach this conclusion. The general clauses of the contract are for the protection of contract relations between the parties thereto, and these parties are capable of getting these relations into words. Further, such relations are mostly uniform. There are, outside of this, certain general clauses which should be drawn only by lawyers, and also, at times, certain special clauses. The counsel in each individual case could embody these as special additions to the agreement, just as proposed in the case where special specifications are necessary. The resulting agreement would then consist of say 95% of crystallized general experience in matter and form, and say 5% for the special case, the experts—lawyer and engineer—adding to it in the form of the special clauses or specifications. Each corporation, in preparing its own agreements, goes through precisely this process, establishing a standard for its own property, as far as it can, according to the varying opinions of its experts from time to time. There is just one more step, and that is, to get the several standards welded into one; not made up from the partial experience of an individual, but combining, crystallizing, and classifying the experience of all. Thousands are confronted with this question every day; all engineers are at some time. Why not reduce the friction? The writer feels that a standardized construction agreement for railroads should be proposed by this Society. There will always be the classifiers *versus* the non-classifiers. Provide for both of them—everything else can be agreed upon.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1047.

THE DESIGN OF THE NEW CROTON DAM.*

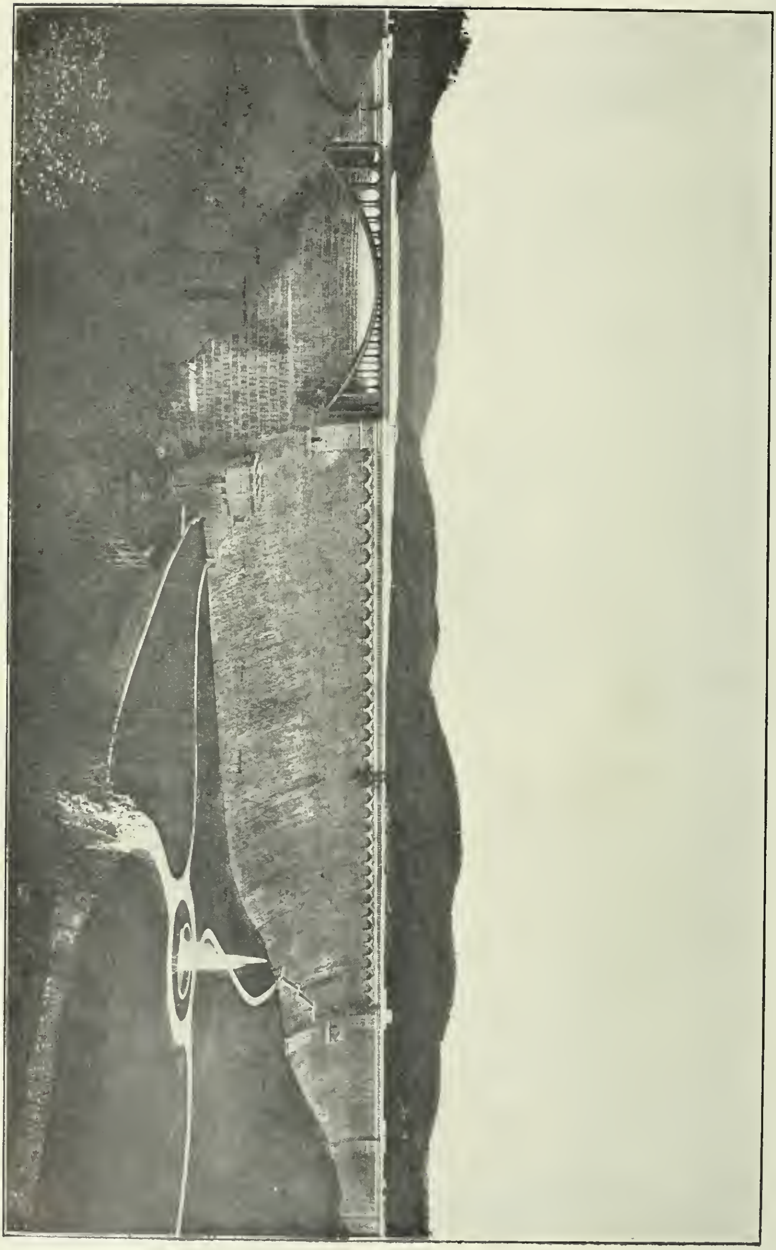
By Edward Wegmann, M. Am. Soc. C. E.

With Discussion by Messrs. Charles S. Gowen, George L. Dillman, Luther Wagoner, William Cain, and Edward Wegmann.

The New Croton Reservoir was constructed in 1892-1906 to store an additional supply of water from the Croton water-shed for the City of New York. An aqueduct, 33.12 miles long, having a capacity of about 300 000 000 gal. per day, was built to convey the water from this reservoir to the city.

The reservoir was formed by constructing a high masonry dam (Plate LXVIII) across the Croton River, about 6 miles above its mouth. According to the original plans, the dam was to be built across the Croton River a short distance above the old "Quaker Bridge," the proposed structure being called the Quaker Bridge Dam. Owing to the great opposition to this project that arose at the "public hearings" held for a discussion of the proposed plans, the Aqueduct Commissioners of the City of New York, who had charge of the construction of the new reservoir and aqueduct, decided to build the dam about 1¼ miles farther up stream, on lands belonging to A. P. Cornell and others. For this reason the dam was first called the "Cornell Dam," but this name was soon changed to "The New

* Presented at the meeting of February 20th, 1907.



THE NEW CROTON DAM.

Croton Dam," as an older dam, built 3¼ miles farther up stream in 1837-1842, was known as the Old Croton Dam.

The plans for the New Croton Dam were prepared under the direction of the late Alphonse Fteley, Past-President, Am. Soc. C. E., Chief Engineer of the Aqueduct Commissioners, who had charge of the construction of the work to January 1st, 1900, when he resigned on account of ill health. He was succeeded as Chief Engineer by the following gentlemen, all of whom are Members of the American Society of Civil Engineers: William R. Hill, January 1st, 1900, to October 14th, 1903; J. Waldo Smith, October 15th, 1903, to August 1st, 1905; Walter H. Sears, August 1st, 1905, to date.

Charles S. Gowen, M. Am. Soc. C. E., was in immediate charge of the construction of the dam as Division Engineer, from the beginning to August 31st, 1905, when he resigned and was succeeded by Mr. F. B. Rogers.

The contract for constructing the dam was awarded on August 26th, 1892, to James S. Coleman, the lowest bidder, for $4 150 573. Mr. Coleman assigned his contract, on January 2d, 1895, to the firm of Coleman, Ryan and Brown, who, on July 13th, 1898, assigned the contract to Coleman, Breuchaud and Coleman. The original contractor was the senior member of both these firms. Ground was broken for the construction of the dam on September 20th, 1892, and the work was practically completed by February 1st, 1906, at a cost of $7 631 185.69,* the increase in cost above the original bid being principally due to a number of changes in the plans for the dam.

According to the contract plans, the dam was to consist of:

First.—A masonry waste-weir, about 1 000 ft. long, along the rocky hillside forming the north slope of the valley;

Second.—A masonry dam about 600 ft. long, extending from the waste-weir, almost at right angles thereto, across the valley, and well into the south slope; and,

Third.—An earthen dam with masonry core-wall, about 600 ft. long, forming a continuation of the masonry dam to the south side of the valley.

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* This amount includes, besides the cost of the masonry dam, the expense of constructing about 20 miles of new highways and of reinforcing about 3 miles of the old aqueduct submerged by the new reservoir.

The plans were modified subsequently by building the whole dam of masonry with the exception of a length of 128 ft. at the south end, which was made an earthen dam with a masonry core-wall, according to the original plans. The maximum height of this earthen dam above the surface of the ground is only about 26 ft.

The general plans for the dam and the construction of the foundations have been fully described by Mr. Gowen.* The principal changes made in the plans have also been discussed by Mr. Gowen.† These changes consisted in the substitution of a masonry structure for the proposed earthen dam. An account of how the dam was built may be found in Mr. Gowen's papers; the writer will consider herein only the design adopted for the dam.

The Design of the Dam.—Extensive studies and calculations were made in designing the proposed Quaker Bridge Dam. The profile and plan adopted for the New Croton Dam were based entirely upon the plans prepared for the Quaker Bridge Dam. Therefore the writer will explain fully how the latter was designed.

The first plans for the Quaker Bridge Dam were prepared by the engineers of the Department of Public Works of the City of New York, under the direction of the Chief Engineer, the late Isaac Newton, M. Am. Soc. C. E., who was assisted by the late E. S. Chesbrough, M. Am. Soc. C. E., the late J. W. Adams, Past-President, Am. Soc. C. E., and Mr. B. S. Church, as Consulting Engineers. In this connection Mr. Chesbrough inspected the principal high masonry dams in Europe, but, unfortunately, he died before he could write his report on the subject.

The profile designed by the engineers of the Department of Public Works is shown in Fig. 1. No report is on record explaining why this form was adopted, but, from statements made by the engineers who assisted in preparing the plans, it appears that the profile was determined by a graphic method given by A. de Beauve in his "Manuel de l'Ingénieur des Ponts et Chaussées,"‡ a limiting pressure of 100 lb. per sq. in. being assumed for both the up-stream and the down-stream faces of the dam to the river-bed. Below the river-bed, both faces were made vertical. The maximum pressure

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* *Transactions*, Am. Soc. C. E., Vol. XLIII, p. 469.

† *Transactions*, Am. Soc. C. E., Vol. LVI, p. 32.

‡ A translation of that part of the "Manuel de l'Ingénieur" which gives the graphic method was made for the Department of Public Works by the late E. Sherman Gould, M. Am. Soc. C. E.

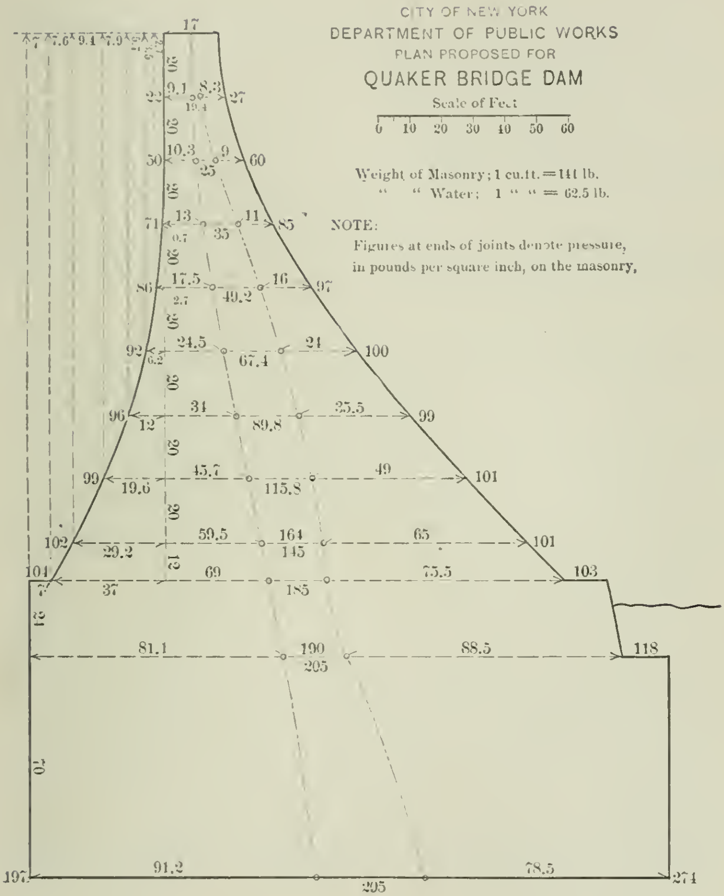

CITY OF NEW YORK
DEPARTMENT OF PUBLIC WORKS
PLAN PROPOSED FOR
QUAKER BRIDGE DAM
Scale of Feet
0 10 20 30 40 50 60
Weight of Masonry; 1 cu.ft. = 141 lb.
" " Water; 1 " " = 62.5 lb.
NOTE:
Figures at ends of joints denote pressure, in pounds per square inch, on the masonry.

FIG. 1.

at the foundation was calculated to be 274 lb. per sq. in. at the down-stream face, and 197 lb. per sq. in. at the up-stream face.

The reason which induced the engineers to make both faces of the dam vertical below the river-bed is not apparent; nor is it clear why they were willing to subject the masonry at the down-stream face of the foundation to a pressure of nearly 20 tons per sq. ft. The following explanations of these points have been made unofficially by some of the engineers who assisted Mr. Isaac Newton in making the designs:

1.—It was assumed that below the river-bed the water pressures against the up-stream and down-stream faces of the dam balanced each other. This view is evidently incorrect, as the water pressure on the down-stream face (assuming the river-bed to be saturated with water) would only be equal to the increase in water pressure on the up-stream face below the river-bed.

2.—The masonry below the river-bed, being supported laterally by the back-filling, could support safely much greater pressures than the masonry above the river-bed. Although this statement is probably correct, it would be impossible, in the present state of knowledge, to estimate how much additional strength would result from this support. It was also stated that the friction between the masonry below the river-bed and the back-filling would relieve part of the pressure on the masonry.

The plans for the Quaker Bridge Dam, received from the Department of Public Works, were referred by the Aqueduct Commissioners to their own Engineer Department, at the head of which they had placed Mr. B. S. Church as Chief Engineer, Mr. A. Fteley as Deputy Chief Engineer, and Joseph P. Davis, M. Am. Soc. C. E., as Consulting Engineer. These engineers made a very thorough investigation of the subject of high masonry dams. The mathematical part of the studies was assigned to the writer, who, at the time, was Assistant Engineer of Construction in the service of the Aqueduct Commissioners.

At that time (1884) there was only one high masonry dam in the United States, the Boyd's Corners Dam, 78 ft. high, constructed in 1866 to 1872, in Putnam County, N. Y. Several high masonry dams had been built or were under construction in France, notably the Furens Dam, 164 ft. high, built in 1862 to 1866, near St.

Etienne. Very little had been published about high masonry dams in English engineering literature, but a number of articles on this subject had appeared in the *Annales des Ponts et Chaussées.* The first search made by the engineers of the Aqueduct Commission was for methods of proportioning the profile of a masonry dam, and the following were found:

1.—De Sazilly's method, in his memoirs on "A Type of Profile of Equal Resistance for Reservoir Walls." (*Annales des Ponts et Chaussées,* 1852.)

2.—Delocre's method, in his memoir on the form of profile to be adopted for high reservoir walls. (*Annales des Ponts et Chaussées,* 1866.)

3.—Professor W. J. M. Rankine's method, in his "Report on the Design and Construction of Masonry Dams," (*The Engineer,* January 5th, 1872.)

4.—Bouvier's formulas, in his memoir on "Calculations of Resistance of High Masonry Dams." (*Annales des Ponts et Chaussées,* 1875.)

5.—Pelletreau's method, in his memoirs on "Walls which Sustain the Pressure of Water." (*Annales des Ponts et Chaussées,* 1876 and 1877.)

6.—De Beauve's graphic method, given in his "Manuel de l'Ingénieur des Ponts et Chaussées." (Paris, 1878.)

The methods mentioned, with the exception of some minor variations, are all based on the principles first pointed out by De Sazilly. Prior to the appearance of his memoir, masonry dams had not been designed according to scientific principles. In fact, it has been shown that some of the dams built would be stronger if their positions were reversed, the up-stream face being turned down stream. De Sazilly stated that, in order to be safe, a masonry dam must comply with the following two conditions:

a.—The pressures on the masonry or on the foundation must not exceed certain safe limits; and

b.—There must be an ample margin of safety against the dam sliding on its foundation, or shearing apart.

He was unable to devise a formula satisfying these two conditions, but found that if a profile were designed to satisfy the first condition, the pressure on the masonry and the foundations being limited to from 8 000 to 12 000 lb. per sq. ft., there would be no possibility of the dam sliding or shearing apart. If a dam should be designed to sustain greater pressures on the masonry and foundation, and if it were found, on trial, that it did not have sufficient resistance to sliding or shearing, De Sazilly recommended that the profile be recalculated with a lower limit of safe pressure, until a profile was obtained that satisfied both the above-mentioned conditions.

Resolving the inclined resultant of the water pressure and the weight of the masonry into horizontal and vertical components, De Sazilly considered the former component to be resisted at the base of the dam by the friction between the base and the foundation, and by the adhesion of the masonry to the foundation. He assumed the vertical component to be distributed on the base of the dam as a uniformly varying pressure. This distribution can be represented graphically by plane figures, the centers of gravity of which lie in the line of action of the vertical component. Four cases may arise:

I.—When the vertical component intersects the base of the dam at its center, the distribution of pressure is shown by a rectangle, the area of which represents the total pressure.

II.—When the vertical component intersects the base between its center and the limit of its center-third, the distribution of pressure is shown by a trapezoid.

III.—When the component intersects the base just at the limit of its center-third, the trapezoid of distribution becomes a triangle.

IV.—When the line of action of the vertical component crosses the base between the limit of its center-third and the nearer extremity of the base, the distribution is shown by two triangles, one representing a negative strain, or tension, and the other a positive strain, or compression.

Formulas can readily be derived by calculating the maximum pressures resulting from the above-mentioned distribution of pressure.* For the trapezoid diagram, which changes from a rectangle

* "The Design and Construction of Dams," by Edward Wegmann. M. Am. Soc. C. E., p. 10.

to a triangle as the line of action of the vertical component moves from the center of the base to the limit of its center-third it will be found that:

$$p = \frac{2W}{l}\left(2 - \frac{3u}{l}\right) \dots\dots\dots\dots\dots\dots\dots\dots (A)$$

in which

W = the vertical component of the resultant pressure;
u = the distance of the line of action of W from the nearer edge of the base;
p = the maximum intensity of pressure on the base;
l = the width of the base.

When $u = \frac{l}{3}$, the trapezoid of reaction becomes a triangle, and

$$p = \frac{2W}{l} \dots\dots\dots\dots\dots\dots\dots\dots (B)$$

When $u < \frac{l}{3}$ there should be, according to the laws of a uniformly varying stress, a positive and a negative triangle, the former representing the pressure on the foundation, the latter the tension on the base; but as it would be unsafe to depend upon the tension in the masonry, it is best to neglect it in calculating the pressure on the foundation, which may be found by the formula:

$$p = \frac{2W}{3u} \dots\dots\dots\dots\dots\dots\dots\dots (C)$$

Formulas A, B and C can be used for calculating the pressures in the masonry at any horizontal plane by assuming a horizontal joint at this plane. A dam should be built as nearly monolithic as possible, by having the stones break joints in all directions, yet it is assumed, for convenience in making the calculations, that horizontal joints occur at regular intervals.

The distribution of pressure given by Formulas A, B and C has been thus far assumed by all writers who have proposed methods of calculating profiles for masonry dams. The late J. B. Francis, Past-President, Am. Soc. C. E., in a paper on "High Walls or Dams to Resist the Pressure of Water,"* drew attention to the fact that, owing to the elasticity of the masonry, the real distribution of pressure is probably somewhat different from what is given by the above-

* *Transactions*, Am. Soc. C. E., Vol. XIX, p. 147.

mentioned formulas, the pressures at the faces of the dam being less, and those near the center of the dam greater, than the values calculated by these formulas. While this statement is probably correct, no rational formula, taking the elasticity of the masonry into account, has yet been proposed. As it is probable that Formulas A, B and C exaggerate the amount of pressure near the faces of the dam, where the masonry is weakest, they are evidently safe.

De Sazilly calculated the resistance of a dam to sliding or shearing, in the following manner:

In Fig. 2:

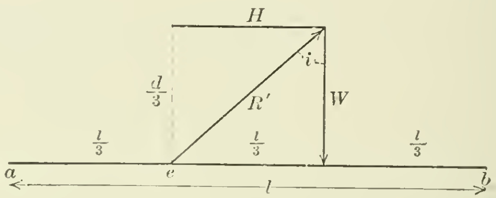


Fig. 2.

Let H = the horizontal thrust of the water;
d = the depth of the water;
l = the length of the base of the dam, $a\ b$;
W = the total vertical pressure on the base;
R = the resultant of H and W;
f = the coefficient of friction;
e = the adhesion of the base to its foundation per unit of length;
i = the angle made by R with a vertical line.

The dam cannot slide on its foundation without overcoming the friction which would result from this motion and, also, the adhesion of the base of the dam to the foundation.

Equilibrium is assured when

$$H \leqq f\,W + e\,l \qquad \text{(D)}$$

This formula may be applied to any assumed horizontal joint above the base; in which case e will represent the cohesion of the masonry. The value of f, as taken by different writers, has varied from 0.67 to 0.75: the value of e, while considerable, has not yet been determined. De Sazilly recommended neglecting $e\,l$, as an additional margin of safety, and writing:

$$H \overset{=}{<} f\,W \text{.............................. (E)}$$

whence,

$$f \overset{=}{>} \frac{H}{W} = \tan.\, i \text{...................... (F)}$$

which gives the least value of f which will prevent sliding. Formula E can be used for calculating the resistance of the dam to shearing at any assumed horizontal joint; in which case f represents the resistance to shearing.

The foregoing method of determining the resistance of the dam to sliding or shearing has been followed thus far by all writers on the subject.

De Sazilly stated that, in making calculations for determining the profile of a dam, two extreme cases should be considered, namely, reservoir full and reservoir empty. In the first case the pressures at the down-stream face of the dam reach their greatest intensity, while in the second case the pressures at the up-stream face reach their greatest values. To obtain the most economical profile for a dam, the design should be made so that the maximum pressures at both faces are equal to the adopted limit of safe pressure. De Sazilly called this type a profile of equal resistance. He obtained differential equations for determining the faces of such a profile, but was unable to integrate them. Close approximations to the theoretical profile of minimum area may be obtained by substituting for curved faces either polygonal outlines or steps. De Sazilly adopted the latter alternative, and designed a stepped profile, Fig. 3, for which he devised equations of the fourth degree.

The next writer on the subject, M. Delocre, who made the theoretical studies for the Furens Dam, in France, adopted all the principles laid down by De Sazilly, but gave the profile of the dam polygonal outlines, Fig. 3, as the steps proposed by De Sazilly involve a waste of masonry and an additional expense in cutting them. Delocre's method is quite laborious, and involves the solution of equations of the sixth degree.

About 1871, Professor W. J. M. Rankine was requested, in connection with some proposed reservoirs for Bombay, India, to make a mathematical investigation of the best form of profile for a masonry dam. Rankine adopted the same general principles as De Sazilly and Delocre, but stated that, in addition to limiting the pressures in the masonry, it was important to keep the lines of pressure, reservoir full or empty, within the center-third of the profile, in order to avoid tension in the masonry.

COMPARISON OF PROFILE TYPES.

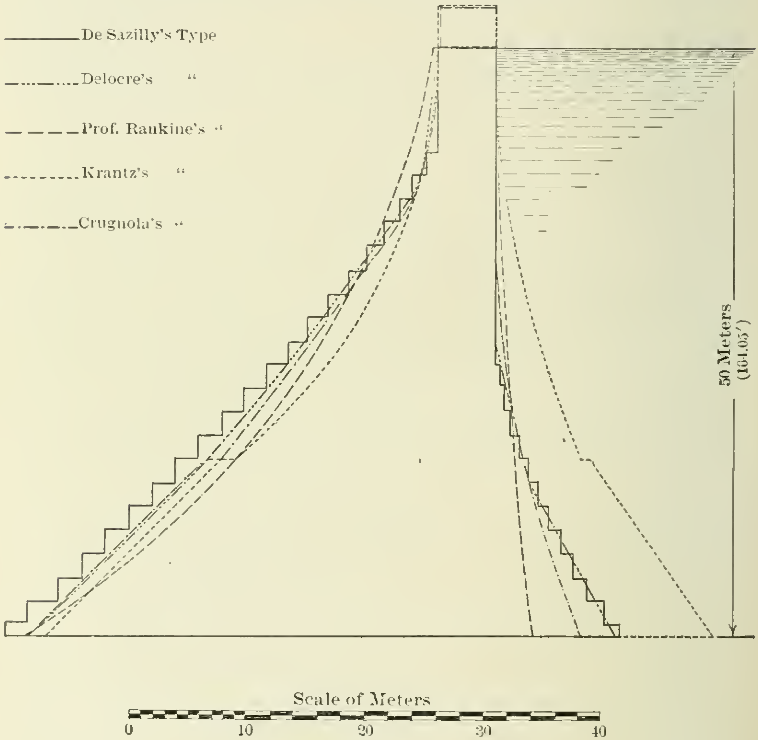


FIG. 3.

In calculating the maximum pressures in the masonry, the French engineers assumed the same limit of safe pressure for the up-stream and down-stream faces, and, for the case of "reservoir full," they only considered the vertical component of the weight of the masonry and of the horizontal thrust of the water, in calculating the distribution of pressure at any assumed horizontal joint. Rankine, however, states in his report:

"It appears to me that there are the following reasons for adopting a lower limit at the down-stream than at the up-stream face. The direction in which the pressure is exerted among the particles close to either face of the masonry is necessarily that of a tangent to that face; and, unless the face be vertical, the pressure found by means of the ordinary formulæ is not the whole pressure, but only its vertical component; and the whole pressure exceeds the vertical pressure in a ratio which becomes the greater, the greater the 'batter' or deviation of the face from the vertical. The down-stream face of the dam has a much greater batter than the up-stream face; therefore, in order that the masonry of the down-stream face may not be more severely strained when the reservoir is full than that of the up-stream face when the reservoir is empty a lower limit must be taken for the intensity of the vertical pressure at the down-stream face than at the up-stream face * * *."

In the present state of knowledge, Rankine did not consider it possible to deduce by mathematics the ratio which the limits of pressure at the up-stream and down-stream faces ought to bear to each other, and, therefore, he fixed the limits which he adopted in designing a profile entirely by what experience had shown to be safe, in connection with masonry dams. The limits of pressure which he adopted are:

	Limits of vertical pressure, in pounds per square foot.
Down-stream face	15 625
Up-stream face	20 000

The same reasoning which led Rankine to recommend a lower limit of vertical pressure for the down-stream than for the up-stream face induced him to make the vertical pressures at the down-stream face diminish as the batter increases. Basing this diminution on practical examples, he designed his profile type so as to have, at a depth of 150 ft., the maximum pressure that occurs in the Furens Dam at the same depth, namely, 6½ kg. per sq. cm., or about 6.65 tons (of 2 000 lb.) per sq. ft. Below this depth, the maximum vertical pressures diminish gradually in Rankine's profile-type.

The conditions given by Rankine do not determine the form of the profile, but when there is added the practical requirement that, with a view to economy, the profile should have the minimum area consistent with safety, the choice of profile becomes limited. The methods of De Sazilly and Delocre involve lengthy and complicated calculations. Rankine devised simpler formulas for determining the form of a profile. Owing to his having assumed a higher limit of pressure for the up-stream than for the down-stream face, the batter of the down-stream face becomes much steeper than that of the French types. The weight of the water resting on the steep up-stream face of such a profile can add but little to the stability of the dam. Rankine neglected this force in his formulas, as the slight error thus introduced is in the direction of safety. This omission simplifies the formulas. The profile recommended by Rankine is described by him as follows:

"In choosing a form in order to fulfil the conditions, without any practical important excess in the expenditure of material beyond what is necessary, I have been guided by the consideration that a form whose dimensions, sectional area, and center of gravity, under different circumstances, are found by short and simple calculations is to be preferred to one of a more complex kind, when their merits in other respects are equal, and I have chosen logarithmic curves for both the inner and the outer face, the common subtangent being 80 ft. for both." (Fig. 3.)

The formulas given by Rankine for this profile are exceedingly simple, but they give only one profile, the dimensions of which might as well be calculated once for all. If a change is made in the weight of masonry, the limiting pressures, etc., upon which this profile is based, the formulas given are incapable of solution by any direct method, and can only be solved approximately by a process of trial and error involving the higher mathematics. If such a method would give the most economical profile for a dam, there might be no objections to using it; but such is not the case. The logarithmic profile proposed by Rankine is only a rough approximation to the profile of minimum area fulfilling the given conditions, and, if it be continued for a greater depth than 150 ft., the down-stream face becomes so flat that it cannot be used for a practical design.

Rankine appears to have been the only English writer who proposed formulas for calculating an economic profile for a masonry dam. Since the appearance of his report on the "Design and Construction of Masonry Dams," some additional memoirs on this subject have been published in the *Annales des Ponts et Chaussées.*

Bouvier proposed to modify Formulas A, B and C so as to calculate the distribution of the whole resultant of the water pressure and the weight of masonry on an assumed joint at right angles to the line of action of the resultant.

Pelletreau advanced a method of determining a profile-type. By the higher mathematics, he found a simple series expressing the thickness of the dam at any depth, as long as the up-stream face remains vertical. He did not succeed, however, in finding a general formula for that part of a dam where both faces are battered.

A graphic method of finding a correct profile for a masonry dam is given by de Beauve in his "Manuel de l'Ingénieur des Ponts et Chaussées" (Paris, 1878). It gives accurate results, but is laborious.

Krantz proposes some profile-types (Fig. 3) in his book entitled "A Study of Reservoir-Walls" (Paris, 1870), but he gives no formulas for determining their dimensions.

Crugnola, the Italian engineer, likewise has given profiles for masonry dams (Fig. 3), without formulas, in his book on dams (Turin, 1882).

Molesworth, in his pocketbook for engineers, has given empirical formulas for determining the profile of a dam.

The methods of determining the profile of a dam and the proposed types mentioned above are all that the writer was able to find in 1884, when the calculations for the proposed Quaker Bridge Dam were being made. They are all very laborious and more or less complicated, when used for calculating profiles based upon different data as regards weight of masonry, limiting pressures, etc.

In calculating the dimensions of the first profiles for the Quaker Bridge Dam for different data, the writer used a simple process of trial calculation instead of the complicated methods described above. The profiles were based upon the following conditions:

1.—The lines of pressure, reservoir full or empty, must be kept within the center-third of the profile.

2.—The maximum pressures on the masonry or on the foundation must not exceed certain safe limits, a greater limit of pressure being assumed for the up-stream than for the down-stream face.

3.—The dam must offer sufficient resistance to sliding or shearing.

The distribution of pressure and the resistance to sliding were calculated by the formulas given by De Sazilly.

The width of the profile was determined by trial calculations at regular intervals, usually of 10 or 20 ft. A vertical axis of moments was taken at a convenient distance up stream from the dam, and the positions of the lines of pressure, reservoir full and reservoir empty, were determined at each assumed horizontal joint by taking moments about the axis. The top width of the dam being given, both faces were kept vertical until a joint was reached, at a depth found by trial, at which the line of pressure, reservoir full, just reached the limit of the center-third of the profile. Below this joint the down-stream face was battered so as to keep the line of pressure, reservoir full, just on the limit of the center-third of the dam, while the up-stream face was kept vertical until a joint was reached where the line of pressure for reservoir empty was at the limit of the center-third of the profile. Below this joint both faces of the dam were battered so as to keep the lines of pressure, reservoir full and reservoir empty, just on the limits of the center-third of the profile.

The upper part of the profile was based solely upon the condition that the lines of pressure, reservoir full or empty, should be within the center-third of the profile, as the pressures on the masonry are inconsiderable. After a number of courses had been determined by this principle, a joint was found at which the maximum pressure at the down-stream face reached the given limit. For the next courses, the down-stream face was given more batter so as to keep the maximum pressures at the down-stream face on the given limit, but the batter of the up-stream face was continued so as to keep the line of pressure, reservoir empty, just on the limit of the center-third of the profile, until a joint was reached at which the maximum pressure at the up-stream face was found to be equal to the assumed limit. Below this joint both faces were battered so as to keep the maximum pressures at the faces on the given limits.

From the process described above it will be seen that the joints of the dam are not based upon the same condition. We have, commencing at the top:

1.—A part of the profile where both lines of pressure, reservoir full and reservoir empty, lie within the center-third of the profile;

2.—A part where the line of pressure, reservoir full, lies just on the limit of the center-third of the profile, while the line of pressure, reservoir empty, is within the center-third;

3.—A part where the lines of pressure, for reservoir full and empty, both lie on the limits of the center-third of the profile;

4.—A part where the maximum pressures at the down-stream face just reach the given limit, while the limit of pressure is not yet reached at the up-stream face; and,

5.—A part where the maximum pressures at both faces are kept just on the given limits of pressure.

After the first profiles had been determined for the Quaker Bridge Dam by trial calculations, as described above, the writer devised a simple analytical method of determining the width of the profile of a masonry dam at regular intervals, which was used in the final calculations. It was developed in the following manner:

Assuming the highest water level at the top of the dam, and taking, for convenience in the calculations, 1 cu. ft. of masonry as the unit of weight or pressure:

Let $a =$ the top width of the dam;
$x =$ the unknown length of a joint of masonry;
$l =$ the known length of the joint above x;
$h =$ the vertical distance between l and x;
$u =$ the distance of the line of pressure, reservoir full, from the down-stream edge of x;
$n =$ the distance of the line of pressure, reservoir empty, from the up-stream edge of x;
$m =$ the distance of the line of pressure, reservoir empty, from the up-stream edge of l;
$v =$ the distance on the joint, x, between the lines of pressure, reservoir full and reservoir empty;
$d =$ the depth of water at the joint, x;
$r =$ the specific gravity of the masonry;

$H = \frac{d^2}{2\,r}$ = the horizontal thrust of the water above x;

$M = \frac{d^3}{6\,r}$ = the moment of the water relative to x;

W = the weight of the masonry resting on x;

w = the weight of the masonry resting on l;

R = the resultant of H and W;

p = the limiting pressure per square foot at the down-stream edge of x;

q = the limiting pressure per square foot at the up-stream edge of x.

The vertical component of the water pressure on the up-stream face of the dam is neglected, as recommended by Rankine, as the trifling error thus made is in the direction of safety.

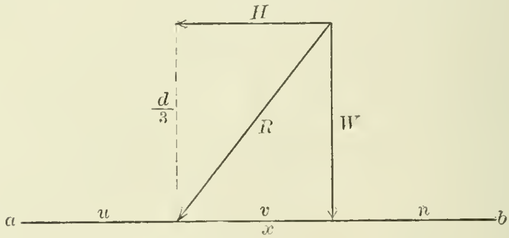


FIG. 4.

The lines of pressure, reservoir full and empty, divide each assumed horizontal joint of the dam into three parts (Fig. 4). For the joint, x:

$$x = u + v + n \text{ . . . . } (1)$$

As $$M = W\,v, \text{ then } v = \frac{M}{W}.$$

Assuming each course of masonry to be bounded by straight lines, then

$$W = w + \frac{(l + x)\,h}{2}.$$

Substituting in Equation 1 for v the value, $\frac{M}{W}$, and for W the value $w + \frac{(l + x)\,h}{2}$, we obtain

$$x = u + \frac{M}{w + \frac{(l + x)\,h}{2}} + n \text{ . . . . } (2)$$

By substituting in Equation 2 for u and n the proper values resulting from the given conditions, the minimum thickness of a dam can be calculated at intervals as close as desired. In the upper parts of the dam, where the only requirement is to keep the lines of pressure within the center-third of the profile, we must write:

$$u = \frac{x}{3}, \text{ and } n = \frac{x}{3}.$$

When the limiting pressures, p and q, are reached, the values of u and n are obtained from Formula A, namely:

$$u = \frac{2x}{3} - \frac{p\,x^2}{6\,W}; \text{ and } n = \frac{2x}{3} - \frac{q\,x^2}{6\,W}.$$

As far as the thrust of the water is concerned, the top width of a dam might theoretically be 0; but, as the dam must also resist the shocks caused by waves and floating bodies, the top must have a certain width, and this is to be determined solely by practical considerations. Generally, the top is made sufficiently wide to serve as a roadway. Whatever the width assumed, it gives the dam an excess of strength at the top to resist the thrust of the water. In order to reach the minimum width of profile to resist this thrust, both faces of the dam must be kept vertical until a joint is reached at which the line of pressure, reservoir full, is just at the limit of the center-third of the profile. The depth at which this joint will be found can be ascertained from Equation 2 by making the following substitutions:

$$x = l = a;\ u = \frac{a}{3};\ n = \frac{a}{2};\ h = d;\ w = 0;\ M = \frac{d^3}{6\,r}.$$

After reducing,

$$d = a\sqrt{r} \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

For the next courses, the down-stream face must be battered so as to keep the line of pressure, reservoir full, on the limit of the center-third of the profile; but the up-stream face must be kept vertical until a joint is reached where the line of pressure, reservoir empty, lies on the limit of the center-third of the profile. Between this joint and the one found by Equation 3, there will be several joints having $u = \frac{x}{3}$, and $n > \frac{x}{3}$. The value of n may be found by taking moments around the up-stream face of the dam and placing the moment of the whole weight equal to the sum of the moments of its parts. In this manner it will be found that

$$n = \frac{(x^2 + l\,x + l^2)\,\frac{h}{6} + w\,m}{w + \frac{(x + l)\,h}{2}}.$$

Substituting in Equation 2, $u = \frac{x}{3}$, and the above value of n, we obtain, after reducing:

$$x^2 + \left(\frac{4\,w}{h} + l\right)x = \frac{6}{h}\,(w\,m + M) + l^2 \ldots\ldots\ldots\ldots (4)$$

This equation may be used for a number of joints until one is reached where the line of pressure, reservoir empty, is on the limit of the center-third of the profile. Below this joint both faces of the dam must be battered so as to keep the lines of pressure, reservoir full and reservoir empty, on the limit of the center-third of the profile.

For these joints we must place

$$u = \frac{x}{3}, \text{ and } n = \frac{x}{3}, \text{ in Equation 2.}$$

After reduction, we obtain:

$$x^2 + \left(\frac{2\,w}{h} + l\right)x = \frac{6\,M}{h} \ldots\ldots\ldots\ldots\ldots\ldots (5)$$

Equation 5 can be used until a joint is reached where the maximum pressures on the masonry reach either the limit, p or q. As it has been assumed that $p < q$, according to Rankine's recommendation, the limit of pressure will be reached first at the downstream face. For these joints, Formula A gives

$$u = \frac{2\,x}{3} - \frac{p\,x^2}{6\left[w + \frac{(x + l)\,h}{2}\right]},$$

and

$$n = \frac{x}{3}.$$

Substituting these values in Equation 2, and reducing, gives

$$x^2 = \frac{6\,M}{p} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

This equation may be used for a number of joints, until one is reached where the pressure at the up-stream edge equals the limit, q. Below this joint the pressures, both at the down-stream and the up-stream faces, must be kept at the given limits, and therefore, from Formula A:

$$u = \frac{2\,x}{3} - \frac{p\,x^2}{6\left[w + \frac{(x + l)\,h}{2}\right]},$$

$$u = \frac{2\,x}{3} - \frac{q\,x^2}{6\left[w + \frac{(x + l)\,h}{2}\right]}.$$

Substituting these values in Equation 2, and reducing, gives

$$x^2\,(p + q - h) - 2\,x\,\left(w + \frac{l\,h}{2}\right) = 6\,M \ldots\ldots\ldots (7)$$

From Equations 3 to 7, inclusive, may be calculated the width of the profile at intervals. If these intervals are sufficiently small (about 10 ft.), the change from one equation to the next comes so gradually that there is no difficulty in knowing which to use. Equations 6 and 7, which apply to that part of the profile where both faces are battered, give the lengths of the joints, but do not fix their position with reference to a vertical axis, for which purpose the following two auxiliary equations are required. They give the amount of batter of the up-stream face for the course in question. This batter, which is denoted in the formulas by y, is found by taking moments about the up-stream edge of the joint, x.

For the joints found by Equations 5 and 6:

$$y = \frac{2\,w\,(x - 3\,m) - h\,l^2}{6\,w + h\,(2\,l + x)} \ldots\ldots\ldots\ldots\ldots (8)$$

For the joints found by Equation 7:

$$y = \frac{w\,(4\,x - 6\,m) + l\,h\,(x - l) + x^2\,(h - q)}{6\,w + h\,(2\,l + x)} \ldots\ldots (9)$$

Equations 3 to 9, inclusive, were used for determining the profile of the Quaker Bridge Dam above the river-bed. For the part below the river-bed, the equations were modified so as to include the weight of the saturated gravel resting on the dam. The modified equations were derived by the late Ira A. Shaler, M. Am. Soc. C. E., who calculated the final profile adopted for the Quaker Bridge Dam. They will not be given here, as they are very lengthy.

The profile of the Quaker Bridge Dam was based upon the data and conditions stated in Table 1.

TABLE 1.—QUAKER BRIDGE DAM.

	Elevations.	
	Croton datum.*	Referred to bed-rock.
Top of dam	210	262
Highest water level	206	258
River-bed	35	87
Bed-rock	—52	0
Base of dam	—58	—6

Top width of dam........20 ft.
Weight of masonry, per cubic foot........156¼ lb.†
Weight of water, per cubic foot........62½ lb.
Weight of saturated gravel and sand in river-bed. per cubic foot........145.88 lb.‡
One cubic foot of masonry was taken as the unit of weight.

* Mean tide at the mouth of the Croton River.
† Corresponding to a specific gravity of 2½.
‡ This weight was found by assuming the sand and gravel to weigh 125 lb. per cu. ft., and assuming one-third of its bulk to be filled with water.

Limits of Pressure.—From the top of the dam to Elevation 110 the pressures in the masonry, at the up-stream and down-stream faces, were limited to 20 625 and 16 400 lb. per sq. ft., respectively. (Corresponding to 10 and 8 kg. per sq. cm.)

From Elevation 110 to the base of the dam, the pressures were allowed to increase gradually, reaching a maximum of 30 000 lb. per sq. ft. at both faces of the base.

The full water pressure was assumed to act as far as the base of the dam, but no upward pressure under the base was taken into account. The vertical component of the water pressure on the up-stream face of the dam was neglected, as recommended by Rankine.

In order to form a practical design, the theoretical profile found in the manner described above was modified slightly, as follows:

1.—The top width was increased by corbeling out to give the width required for a roadway.

2.—The outlines were simplified by substituting a few simple batters for the many changes occurring in the faces of the theoretical form.

3.—In the final calculations the dam was given a superelevation of 4 ft. above high water, and the thickness of the profile, for a depth of 40 ft. from the top, was increased slightly to make it more symmetrical, and to provide additional strength to resist the shocks from floating bodies.

4.—The sharp triangle at the down-stream toe of the dam was

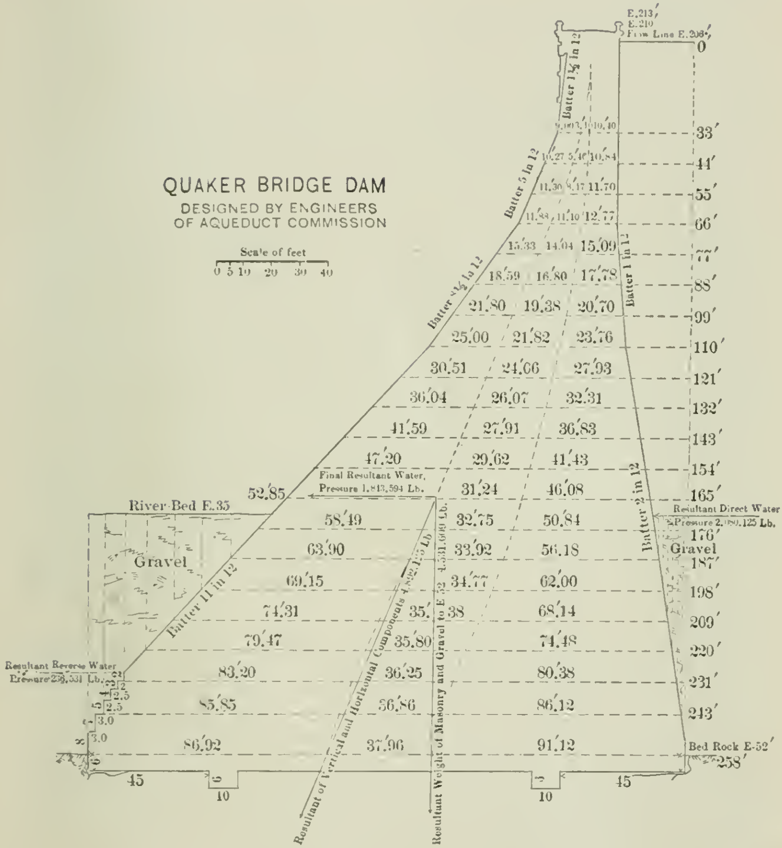

QUAKER BRIDGE DAM
DESIGNED BY ENGINEERS
OF AQUEDUCT COMMISSION
Scale of feet
0 5 10 20 30 40
River Bed E.35
Gravel
Batter 11 in 12
Batter 5 in 12
Batter 1 in 12
Batter 2 in 12
Final Resultant Water,
Resultant Direct Water
Bed Rock E-52′
0
33′
44′
55′
66′
77′
88′
99′
110′
121′
132′
143′
154′
165′
176′
187′
198′
209′
220′
231′
243′
258′
45
10
10
45

FIG. 5.

cut off, as it was not considered to have sufficient strength to distribute the pressure of the masonry, and the face was carried up in steps for a certain height. This reduced the width of the base from 230 to 216 ft., and increased the pressures on the masonry at the down-stream face to 33 266 lb. per sq. ft. Fig. 5 shows the profile finally adopted.

Having designed the profile by this method, the next question considered was whether the dam should be straight or curved in plan. Good examples of each plan of construction existed. For a curved plan, calculations were made to ascertain whether the area of the profile could be reduced on account of the additional strength that might be due to arch action. That such action might take place was shown by the fact that two curved dams—the Zola Dam, in France, and the Bear Valley Dam, in California—resisted the water pressure successfully, although the lines of pressure for reservoir full fell outside of the profiles of the dams. These dams, therefore, owed their stability solely to arch action, as they were unable, by gravity, to resist the water pressure. In both these cases the dams were constructed across narrow valleys.

The Zola Dam was built about 1843. Its height is 119.76 ft. above the foundation, and its width is only 41.82 ft. at the base. Its length is 205 ft. on top, and it is curved in plan, the radius at the crest of the dam being 158 ft. When the reservoir is full, the line of pressure falls about 11.50 ft. outside of the base of the foundation.

The Bear Valley Dam was constructed in 1884 in the Bernardino Mountains, in California. The profile adopted was so thin that the line of pressure, reservoir full, fell almost wholly outside of the dam. This dam, which has been replaced by a stronger one built below the first site, had a maximum height of 64 ft. above the foundation, and was 3.17 ft. wide on top and 20 ft. wide at the base. It was curved up stream to a radius of 335 ft.

While these two examples proved that in the case of a narrow valley a dam might resist the water pressure by acting as a horizontal arch, calculations showed that in a curved dam built across a wide valley, like that of the Croton River, arch action would produce far greater strains in the masonry than those which would occur in a dam designed to resist the water pressure by weight alone.

The pressures in a curved dam were ascertained by supposing the dam to be divided into horizontal courses, each of them forming a ring composed of a number of voussoirs. The pressure in such a ring, which is exposed in a horizontal direction only to the water pressure acting normally to its surface, is found by the well-known formula:

$$T = p\, r,$$

in which

$T =$ the uniform thrust in a circular ring;

$p =$ the pressure per unit of length of the ring;

$r =$ the radius of the ring's outer surface.

This formula applies to a circular ring of masonry, but, evidently, it may be imagined that part of the ring is removed and replaced by the practically rigid sides of the valley, in which case the formula would still be applicable.

The authorities consulted agreed that very little reliance could be placed on calculations treating a dam as a horizontal arch, and that a curved plan for a dam should only be adopted for a narrow valley.

Delocre states that a dam will cease to act as an arch when the width of the dam equals one-third of the radius of the curve, or is larger.

Pelletreau advises that the arched plan should not be adopted when the dam is more than 131 ft. in height and length.

Krantz gives the limit of span for an arched dam as 131 ft., with a radius of not more than 65.6 ft.

Rankine says that "in the present state of science, the calculations of stability, treating the dam as a horizontal arch, are so uncertain as to be of doubtful utility." He recommends that a dam be always built sufficiently strong to resist the thrust of the water by its own weight, and that in the case of a narrow valley the plan be curved so as to add some additional stability to the structure.

In their reports to the Aqueduct Commissioners on the design and construction of high masonry dams, dated July 28th, 1887, both Mr. B. S. Church. Chief Engineer, and Mr. A. Fteley, who had resigned as Deputy Chief Engineer and was Consulting Engineer at the time, advised that the plan of the Quaker Bridge Dam be made straight, for the following reasons:

1.—A dam of the length of the proposed Quaker Bridge Dam (1 350 ft. at the crest) would be exposed to excessive strains if it acted as an arch.

2.—If designed to resist the water pressure by its weight alone, as recommended by Rankine, it would not yield sufficiently to cause arch action.

3.—If no arch action should occur, the curving of the plan would involve a waste of money, as it would increase the volume of masonry.

The conclusions arrived at by Messrs. Church and Fteley were fully concurred in by the engineers of the Department of Public Works.

The next important feature considered was the superelevation to be given the dam above the highest assumed flood level (Elevation 206). This was determined from the data of existing dams and from practical considerations. The top of the dam was placed 4 ft. above the flood level, and the top of the stone parapet 3 ft. higher, which was considered sufficient to prevent the highest waves that might occur from breaking over the top of the dam.

One of the most important questions to be decided was the length of the overflow weir. At the Old Croton Dam, which is about 4½ miles above the site selected for the Quaker Bridge Dam, the maximum flood recorded produced a flow of 8 ft. 2 in. deep over the overflow weir, the length of which is 251.4 ft. The water-shed above the dam contains 338.82 sq. miles. It was decided to make the overflow weir of the Quaker Bridge Dam sufficiently long to be able to discharge in 24 hours a volume of water represented by a uniform thickness of 6 in. over the whole water-shed, which contains about 361 sq. miles. The length of overflow corresponding to these conditions was calculated to be about 1 300 ft. In view of the fact, however, that the dam was to be provided with blow-off gates, and that one, if not both, of the Croton Aqueducts would be drawing water from the reservoir, it was decided to give the overflow weir a length of 1 000 ft.

In reference to the kind of masonry: the plans contemplated the use of uncoursed, broken-range rubble masonry, built of quarry stone of irregular sizes, laid with full beds and joints. Cut stone was only to be used for ornamentation, where required.

On account of the importance of the proposed Quaker Bridge Dam, and the opposition to its construction, the Aqueduct Commissioners, on March 7th, 1888, appointed a Board of Expert Engineers to take into consideration the plans prepared by the engineers of the Commissioners and the modifications which had been or might be suggested by others, either in plan or in cross-section, and to advise the Commissioners fully on these subjects. Joseph P. Davis, M. Am. Soc. C. E., J. J. R. Croes, Past-President, Am. Soc. C. E., and Mr. Hermann Schussler, were appointed on this Board. Mr. Schussler could not serve, and Mr. William F. Shunk was appointed in his place. After a thorough investigation of the questions submitted, the Board of Expert Engineers, on October 1st, 1888, made its report to the Aqueduct Commissioners. The forces to which the dam might be exposed were classified in this report, as follows:

1.—The quiescent and ever-acting forces, such as the weight of the masonry and the pressure produced by the impounded water;

2.—Forces produced by the expansion of ice in place, or by floating masses;

3.—Forces produced by waves of translation, the possible cause of such waves being the giving way of a dam above, or an extensive land slide;

4.—Earthquake shocks.

The experts assumed the following data:

Top of parapet of dam	Elevation 214	
Highest water level	" 202	
River-bed	" 35	
Bed-rock	" —52	
Weight of water per cubic foot	62.5	lb.
Top width of dam	20	ft.
Weight of masonry per cubic foot	146.25	"
Ice pressure per linear foot	43 000	"

With reference to the pressure of ice, the experts stated:

"In our search for information upon the expansive force of ice in place, caused by increase of temperature, we found little of value recorded; but we obtained valuable, though somewhat conflicting, information by correspondence and personal interviews, which information, supplemented by experimental data, concerning its strength, elasticity and rate of expansion under a rising thermometer, has led us to the opinion that the dam should be proportioned to resist a thrust at the highest ice line of about 43 000 lb. per lin. ft.

"More positive information was available regarding the force exerted by ice flow. Under certain unfavorable conditions, when ice jams form in a quick running current, it appears to be almost irresistible by direct opposition. But, as in the case of the Quaker Bridge Dam, the water current, when there is one, will tend to direct the flow away from it, and direct impact can be produced only by sheets of ice driven by the wind, we have concluded that, if the dam be proportioned to resist the pressure of 43 000 lb. per lin. ft., above mentioned, it will be of ample strength to withstand the attack of floating masses."

To secure the dam from injury by waves of translation, its upper portion, where the effect of such waves would be greatest, was designed to have a coefficient of at least 2 against overturning, when the level of the water might be at the top of the parapet.

Earthquake shocks may vary from a slight tremor to an immeasurable force. The experts stated that, if the dam be proportioned to resist the forces already mentioned, it would have ample stability to withstand all but shocks of the severest nature.

The profile designed by the experts was made to comply with the following conditions:

1.—The factor of safety against overturning shall be at all points at least 2;

2.—The ratio of the weight of masonry above any horizontal plane or point to the maximum force tending to cause sliding or shearing along the plane shall not be less than 3 to 2;

3.—The maximum quiescent stress on the down-stream end of the joints at the river-bed (Elevation 35) shall not exceed 20 000 lb. per sq. ft.;

4.—Below the river-bed, when the strength of the masonry to resist crushing is aided by the lateral pressure of the earth, the maximum quiescent stress shall not exceed 28 000 lb. per sq. ft.

5.—The pressures on the joints may be somewhat greater at the up-stream face than at the down-stream face, as they will be permanently reduced as soon as the reservoir begins to fill.

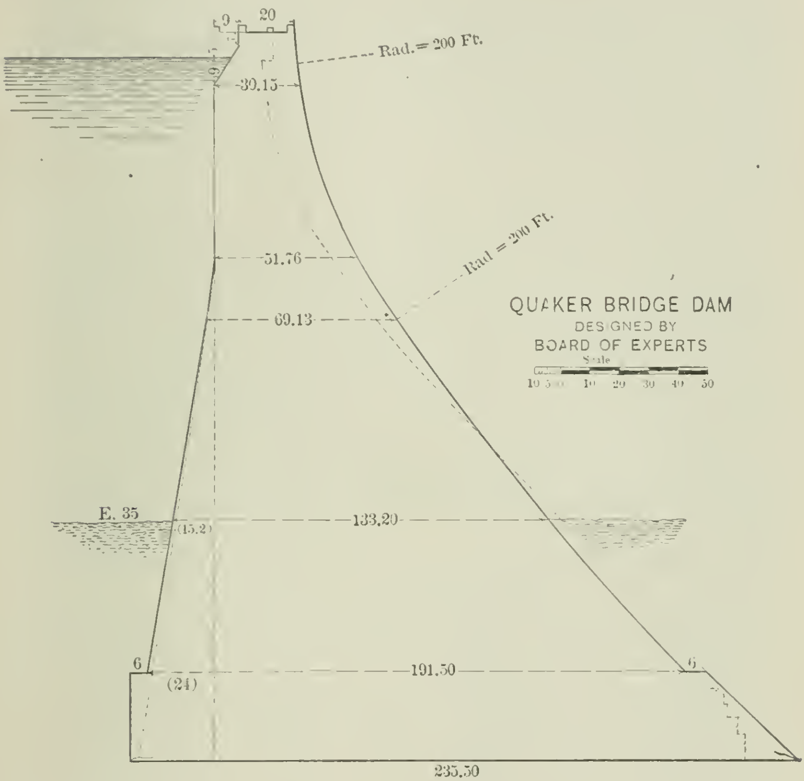


FIG. 6.

The profile designed by the Board of Experts to comply with the conditions given above and, also, the profile recommended by the engineers of the Aqueduct Commission, are shown in Fig. 6.

The former profile is thicker in its upper part than the latter. The experts thought this increase in thickness necessary to give the dam sufficient strength to resist the shock of ice and excessive freshets.

In making the final location of the dam, the engineers of the Aqueduct Commission had staked out the axis of the dam on two straight lines, respectively 326 and 1 077 ft. long, the angle between the two lines being about 45 degrees. This was done, in accordance with the information obtained by test borings, to make the excavation for the foundation as small as possible. The experts recommended that the plan for the dam be curved to a radius of about 1 200 ft., although this would have involved a slight increase of about 10% in the volume of masonry. The reasons that made them decide in favor of a curved plan in preference to a straight location were:

1.—The curved form accommodates itself better to changes of volume due to changes of temperature.

2.—While a curved plan would add nothing to the stability of a dam having a "gravity section," as long as the dam stood, it might be of great advantage in case the masonry should yield.

3.—In a structure of the magnitude and importance of the Quaker Bridge Dam, the question of producing a pleasing architectural effect is only second to that of structural stability. Such an effect can be obtained better by a plan curved regularly in a long radius than by a plan composed of straight lines with sharp angular deflections.

In this paper the writer has given the conclusions of the engineers of the Aqueduct Commission, and also those of the Board of Experts, to show how eminent engineers may differ in their views in designing an important structure. The plans recommended by the experts were not adopted by the Aqueduct Commissioners, and the drawings for the New Croton Dam were based upon those prepared for the Quaker Bridge Dam by the Commissioners' engineers.

Discussion of the Plans Adopted.—Considering the protracted studies made in designing the profile for the Quaker Bridge Dam, one would expect to find the profile finally adopted about as perfect as was possible with the knowledge available at the time. Such, however, does not seem to the writer to be the case, for reasons which he will give. The profile was based on the conditions proposed by Rankine, but, on account of the unprecedented height of the dam, greater limits of safe pressure had to be assumed in the lower parts of the structure than had been recommended by Rankine. One of the points in which Rankine had improved on the methods adopted by the French engineers was in assuming a lower limit of pressure at the down-stream than at the up-stream face of the dam, the limits adopted by him for a dam 180 ft. high being, respectively, 15 625 and 20 000 lb. per sq. ft. In designing the profile for the Quaker Bridge Dam, the maximum pressures assumed in the calculations were, according to Mr. Fteley, Consulting Engineer at the time and later Chief Engineer of the Aqueduct Commission,* for a depth of water of 110 ft. or less, 16 400 lb. per sq. ft. at the down-stream face and 20 625 lb. per sq. ft. at the up-stream face. From a depth of water of 110 ft. to the base of the dam the maximum pressures in the masonry were allowed to increase gradually until they reached a maximum of 30 000 lb. per sq. ft. at both faces at the base of the dam. The pressure at the ends of the different assumed horizontal joints of the theoretical profile, as given by Mr. Fteley,† are shown in Table 2.

TABLE 2.—THEORETICAL PROFILE FOR QUAKER BRIDGE DAM.

Elevation of joint. Croton datum. Feet.	Depth of water at joint, in feet.	PRESSURES AT ENDS OF JOINTS, IN POUNDS PER SQUARE FOOT.	
		Down stream.	Up stream.
171.3	34.7	13 031	6 516
156.	50.	14 156	11 328
136.	70.	15 234	15 234
116	90.	15 984	15 984
96.	110.	16 391	17 453
76.	130.	16 384	18 462
56.	150.	17 078	19 930
35.	171.	18 219	21 822
15.	191.	20 084	23 641
—5.	211.	22 680	25 466
—25.	231.	25 691	27 313
—52.	258	30 000	30 000

The profile to which Table 2 applies seems to be somewhat inconsistent, for the following reasons:

1.—Greater pressures are allowed at the up-stream than at the

* Report to the Chief Engineer, July 25th, 1887.

† Table II of Mr. Fteley's Report.

down-stream face, until the base of the dam is reached, where the pressures are made the same at both faces. Why should not a difference in pressure be made at the base, as in the upper parts of the dam?

2.—The pressures at the down-stream and up-stream faces, respectively, were limited to 16 400 and 20 625 lb. per sq. ft., to a depth of 110 ft. of water, which was assumed arbitrarily. Below this depth, the pressures were allowed to increase gradually. What object was accomplished in limiting the pressures as described above to a depth of 110 ft. when, below that depth, they were allowed to increase? Would it not have been more logical to have designed the upper part of the profile solely with a view of confining the lines of pressures, reservoir full or empty, to the center-third of the profile, and to have allowed the pressures to increase gradually until some assumed limits of maximum pressures for the down-stream and the up-stream faces of the dam were reached, below which point the profile should have been designed so as to keep the pressures at the ends of the joints exactly at the prescribed limits?

The theoretical profile discussed above was slightly modified to meet some practical requirements, as already explained. The modifications changed the distribution of pressures in the masonry somewhat; these are given in Table 3* for the final profile.

TABLE 3.—Profile Adopted for Quaker Bridge Dam.

Elevation of joint. Croton datum. Feet.	Depth of water at joint, in feet.	Pressures at ends of joints, in pounds per square foot.	
		Down stream.	Up stream.
173	33	8 588	6 578
151	55	12 425	11 973
129	77	14 716	14 960
107	99	15 891	16 797
85	121	16 544	18 291
63	143	16 966	19 688
41	165	17 997	21 578
19	187	19 672	23 609
—3	209	22 406	25 563
—25	231	26 156	27 609
—52	258	33 266	30 828

* Table III of Mr. Fteley's report.

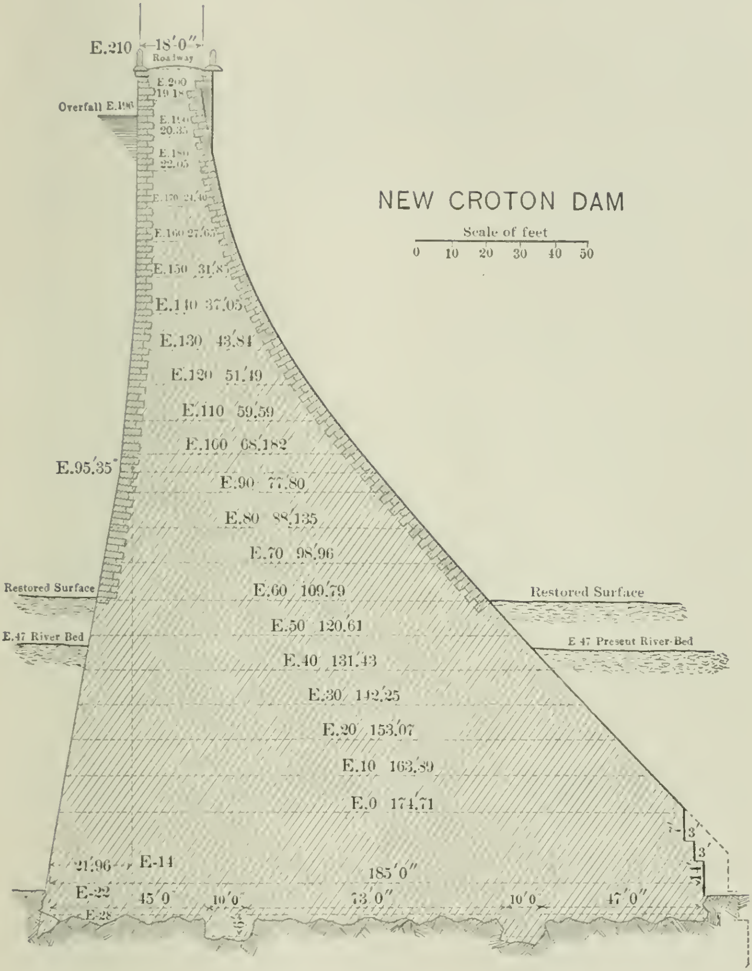

NEW CROTON DAM
Scale of feet
0 10 20 30 40 50
E.210
18′ 0″
Roadway
Overfall
E.150 31.85
E.140 37.05
E.130 43.84
E.120 51.49
E.110 59.59
E.100 68.182
E.95.35
E.90 77.80
E.80 88.135
E.70 98.96
E.60 109.79
E.50 120.61
E.40 131.43
E.30 142.25
E.20 153.07
E.10 163.89
E.0 174.71
Restored Surface
E.47 River Bed
Restored Surface
E.47 Present River-Bed
21.96
E-14
E-22
E-28
185′ 0″
45′ 0″
10′ 0″
73′ 0″
10′ 0″
47′ 0″
3′
3′

Fig. 7.

According to Table 3, the distribution of pressure is worse in the modified than in the theoretical profile, the pressure at the base, in the former, being greater at the down-stream than at the up-stream face. This is principally due to the fact that part of the toe of the dam at the down-stream face was cut off and replaced by steps. The writer does not think it was necessary to make this change. The pressure to be distributed at the face is not vertical, but, according to Rankine, parallel with the face; the batter immediately above the steps is the same as at the toe of the theoretical profile, and there appears to be no good reason why the face should be broken into steps at the down-stream toe, while remaining battered immediately above the steps.

The contract plans for the New Croton Dam were not made until about five years after the studies for the Quaker Bridge Dam had been concluded. No further calculations were made. Mr. Fteley, who had become Chief Engineer of the Aqueduct Commission, based the profile upon the design he had prepared for the Quaker Bridge Dam, the only difference being that the down-stream face of the profile for the latter was rounded off in its upper part by the introduction of more changes of batter, the area of the profile being also slightly increased. The profile of the New Croton Dam given in the contract drawings (Fig. 7) has polygonal outlines, which, in the upper part of the down-stream face, approach closely to a curve. In constructing the dam, the Division Engineer in charge fitted curves to the angular line intended for the down-stream face. Three circular curves separated by two short tangents of 22 and 26 in. were used. It would certainly have been better if, in preparing the contract drawing, the profile had been slightly changed by adopting a compound curve for the upper part of the down-stream face.

The up-stream face of the dam was built vertical from Elevation 140 to the top of the dam (Elevation 210). Below Elevation 140 the face was battered. A profile could easily have been designed with the up-stream face vertical from the top of the dam to the refilling (Elevation 70), and would have been more easily followed in construction. The water pressure against a dam is represented graphically by a triangle, and the theoretical profile of minimum area to resist the water pressure is also a triangle until

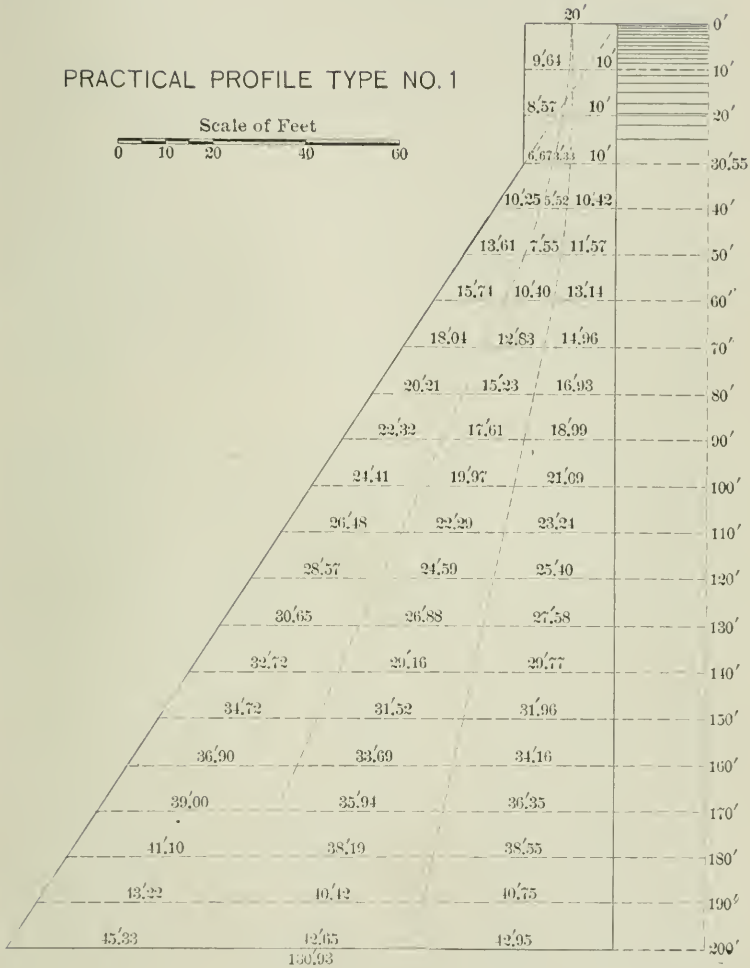

PRACTICAL PROFILE TYPE NO. 1
Scale of Feet
0 10 20 40 60
20′
0′
9.′64 10′ 10′
8.′57 10′ 20′
6.′67 3.′33 10′ 30.′55
10.′25 5.′52 10.′42 40′
13.′61 7.′55 11.′57 50′
15.′71 10.′40 13.′14 60′
18.′04 12.′83 14.′96 70′
20.′21 15.′23 16.′93 80′
22.′32 17.′61 18.′99 90′
24.′41 19.′97 21.′09 100′
26.′48 22.′29 23.′21 110′
28.′57 24.′59 25.′40 120′
30.′65 26.′88 27.′58 130′
32.′72 29.′16 29.′77 140′
34.′72 31.′52 31.′96 150′
36.′90 33.′69 34.′16 160′
39.′00 35.′94 36.′35 170′
41.′10 38.′19 38.′55 180′
43.′22 40.′42 40.′75 190′
45.′33 42.′65 42.′95 200′
130.′93

FIG. 8.

the pressures in the masonry reach the given limits (Fig. 8). The nearer the practical profile approaches a triangle, the greater the economy will be.

To resist the action of waves, and shocks from floating bodies, a dam must have a certain thickness at the top, which may have to be increased if it is to have a roadway. The effect of this increased width is to augment the dam's strength to resist overturning or sliding; it adds only a trifling amount to the pressures in the masonry. If the thickness added at the top of a dam be taken into account in the calculations, the most economical profile type is that shown by Fig. 9, but as the area of this profile, down to a depth of 150 ft., is only about 5% more than that of the simple triangular profile type shown in Fig. 8, the writer thinks the latter is to be preferred on account of its simplicity. Such a profile, if continued to a depth of 200 ft., would produce a maximum pressure of 30 320 lb. per sq. ft., if the masonry has a specific gravity of $2\frac{1}{3}$.

As regards the location of the New Croton Dam, it is greatly to be regretted that the site was changed from Quaker Bridge, $1\frac{1}{4}$ miles up stream, to that known as Cornell's. At the former site the dam would have been founded entirely on gneiss; the overflow would have taken place at a depression in a spur north of the dam: about $1\frac{1}{2}$ sq. miles would have been added to the water-shed, and the storage capacity of the reservoir would have been increased.

At the Cornell site the waste water could not be discharged through a depression of the hillside, and therefore Mr. Fteley designed the waste weir to adjoin the main dam. In order to keep the waste channel in rock, which was only to be found near the surface at the north side of the valley, and to obtain also the desired length for the waste weir (*i. e.,* 1 000 ft.), the weir was located almost at right angles to the main dam, along the north hillside, and joined by curves with the hillside and with the main dam. This location, while it provided an excellent waste weir and channel, involved considerable expense. In constructing the weir and channel, about 120 000 cu. yd. of earth, and about 240 000 cu. yd. of rock were excavated, and about 90 650 cu. yd. of masonry were laid.

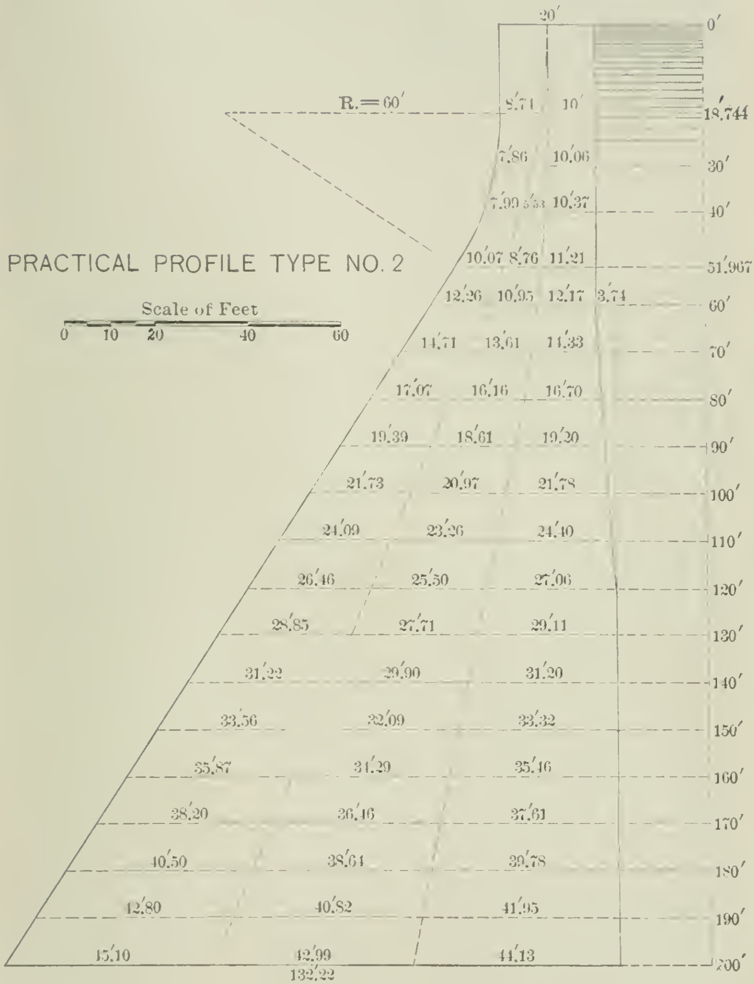

PRACTICAL PROFILE TYPE NO. 2
Scale of Feet
0 10 20 40 60
R.=60′
20′
0′
18.744′
30′
40′
51.967′
60′
70′
80′
90′
100′
110′
120′
130′
140′
150′
160′
170′
180′
190′
200′
132.22′

FIG. 9.

The writer has given herein some criticisms of the profile of the New Croton Dam, considered as an engineering design. In doing so, however, he does not mean to imply that the dam is not sufficiently strong to resist the pressures to which it is exposed. The reservoir has been filled to Elevation 182 (14 ft. below the top of the depressed spillway), and the dam has been found to be satisfactory in every respect.

DISCUSSION.

Charles S. Gowen, M. Am. Soc. C. E.—Mr. Wegmann's paper is comprehensive, in so far as it relates to the history of the design of the New Croton Dam, and to the account of the mathematical work involved, and is of value as showing in detail the methods used in the calculations of the theoretical profile of the dam section, and in its references to certain practical questions involved in the design of such a structure. Mr. Gowen.

In a sense it completes the records in the *Transactions* of this Society as to the planning and construction of an unprecedented and notable engineering work.

The literature referring to this design, however, is not lacking, but is embodied in the various reports of the Aqueduct Commissioners, which have been referred to and quoted by the author. These reports deal very fully with all the points involved in connection with the design of the Quaker Bridge Dam profile as finally adopted by the Aqueduct Commissioners' engineers, as well as with the profile recommended by the special board appointed to consider the question. However, as these reports are not generally available to the profession and the public, Mr. Wegmann's paper may be considered as especially valuable, both as a matter of record and as a matter of information.

The design finally adopted by Mr. Fteley for the New Croton Dam section was, as stated by the author, the Quaker Bridge section slightly modified. The results derived from the study which resulted in the Quaker Bridge profile were considered final and conclusive, as might be expected, the change of location involving no questions of theory and consequent additional research. The speaker was somewhat familiar with this question at the time the contract drawings were being made, and recalls that the calculations of pressures were examined in view of the possible changes in the weight of masonry that might be used in the dam construction.

As to the author's criticisms of the profile adopted, the speaker, in reviewing them, is disposed to take exception to them as follows:

First.—As to the statement that in the Quaker Bridge profile the maximum pressures at a depth of 110 ft. were assumed to be 16 400 lb. per sq. ft. and 20 625 lb. per sq. ft. for the down-stream and up-stream faces, respectively. In Table 2, for the theoretical profile, as quoted by the author, the calculated pressures are 16 391 and 17 453 lb. per sq. ft. for the down-stream and up-stream faces, respectively, while in Table 3 the calculated pressures in the adopted profile are correspondingly 16 217 and 17 544 lb. per sq. ft.

Tables 2 and 3 show, as the author states, greater pressures on the up-stream than on the down-stream face for a depth which

Mr. Gowen. ranges from 110 to 231 ft. in Table 2, but it will be noted that the pressures at a depth of 110 ft. at the two faces do not vary very materially, although the pressure on the up-stream side is somewhat greater. It seems to the speaker that it is to be assumed that the ultimate up-stream face pressures should be limited in the same way as those on the down-stream face; hence the 30 000-lb. per sq. ft. limit, as shown in Table 2. This limit is somewhat increased in Table 3 owing probably to cutting off the down-stream toe, which, as the author observes, has increased the calculated pressures at that point. The increase in the toe limits, as shown by Table 3, is not significant, in view of the generally admitted proposition that the toe pressures at the base of such a profile are less by a considerable amount than those calculated. It is well known that such an actual condition results in errors on the side of safety when profiles are calculated on theoretical lines, and, through this condition, the cutting off of the down-stream toe, as shown in the profile, and to which the author objects, would seem to be justified, even if the practical consideration that masonry at the extreme end of such a toe would not have mass or strength sufficient to be effective, be ignored. This contention, that the toe should have remained, seems to be based on the assumption that the line of pressure at the down-stream face is parallel to the face of the dam, and Mr. Wegmann states that experiments lately conducted tend to prove this.

That such a condition has for a long time been assumed is evident, as certain high masonry dams in Europe and America, built since the determination of the New Croton Dam section, were planned to have selected stones, approximately rectangular, laid on inclined beds which approach the normal to the profile inclination near the toe. While, in the speaker's opinion, it is a question as to whether the extra expense involved in such construction warrants the results obtained, there is certainly no disposition on his part to criticise the possible effectiveness of such an arrangement of the masonry. But that such arrangement warrants or necessitates the carrying of the toe to the rock bottom, in accordance with the theoretical lines, does not appear, and certainly the toe thus carried out in its outline could only be effective if sufficient excavation were made in the foundation rock to afford an adequately extended skewback, such as is always provided in arch construction. Fig. 10 may illustrate the speaker's point more clearly than the text.

In the case of the New Croton Dam, *A A A A* shows the curtailed toe and the lines of the rock foundation below; certainly it would seem that, if the toe were left in, to accord with the author's idea, a special excavation in the rock bottom approximating the lines, *A B B B*, would have to be made, in order to render the small retained triangle of masonry effective. It does not seem to the speaker that such work is called for, particularly when it is shown that toe pressures with the toe abbreviated cannot be excessive and are, as is generally admitted, less than the calculations shown. **Mr. Gowen.**

Second.—As to the author's contention that there was an arbitrary assumption of toe pressures at a depth of 110 ft. and that it

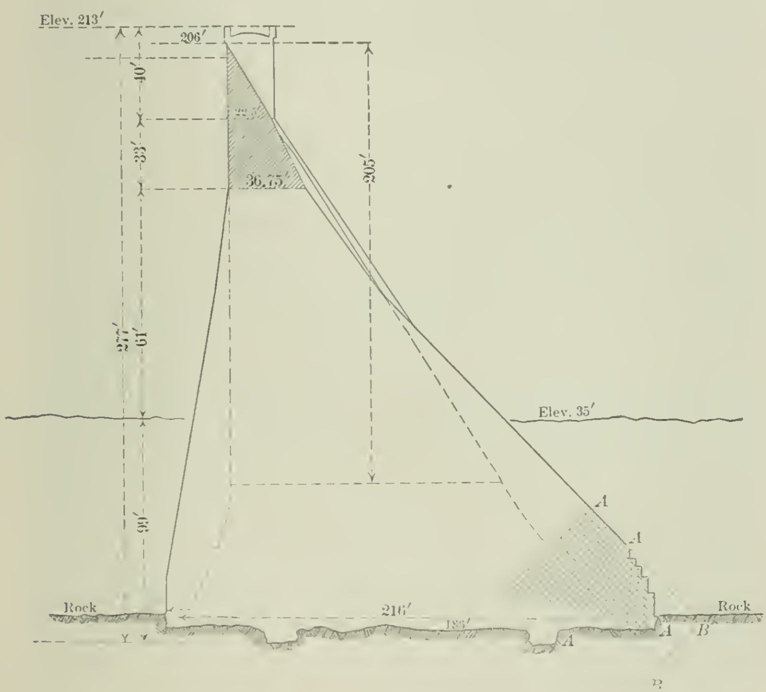


Fig. 10.

was illogical, in view of this and of the gradual increase of pressures below the 110 ft. depth, not to have designed the upper part of the profile solely with a view of confining the pressure lines just within the middle third of the section, and then to have allowed a gradual increase of pressures until a maximum limit should have been reached and the profile continued below with this maximum limit not exceeded, it would seem to the speaker that these conditions, practically, were all kept in mind in the design

Mr. Gowen. of the theoretical profile, as the following points had to be kept in view from the beginning:

(*a*) The top of the dam had to be of a certain thickness, say 20 ft., to afford a roadway.

(*b*) The top of the dam had to be built at a certain height above high-water mark.

(*c*) It was essential that lines defining slopes and batters of the profile should show easy and gradual changes from the vertical, in order that the gradually cumulative pressures resulting from the elevation of the water behind the dam might be properly transmitted to the base without undue strain at any particular point. This point was dwelt upon particularly in Mr. Fteley's report on the Quaker Bridge profile, and it is this that accounts for the up-stream pressures between the depths 110 and 258, in Table 3, which are criticized by the author as being higher than on the corresponding lines on the down-stream face. It is this consideration, also, taken in connection with the surplus of section and weight due to the necessary height of the dam and the planned roadway, which results in the somewhat narrow limit between the pressure lines in the middle third at a depth of 110 ft., and the resulting pressures at the joint ends, of about 17 000 lb. per sq. ft., which, while they may be less than theoretically allowable, are not inconsistent as between the up-stream and down-stream faces, while they are certainly moderate. Different assumptions in calculating the theoretical profile, made in accordance with the author's expressed ideas, would have resulted in sharper angles in definition of the up-stream batter below the 110-ft. depth, and the same would have been the case with the outline of the down-stream face, which would have shown too marked an angle between the upper and lower sections.

In fact, in the speaker's opinion, the contention that the profile is consistent can be easily sustained by an inspection of the calculated factors of safety against overturning, due to the profile as adopted in Table 3.

These are as given in Table 4.

TABLE 4.

Depth of water, in feet.	Factor of safety.	Depth of water, in feet.	Factor of safety.	Depth of water, in feet.	Factor of safety.
66	2.07	132	2.38	198	2.99
77	2.09	143	2.49	209	3.10
88	2.10	154	2.60	220	3.22
99	2.12	165	2.69	231	3.29
110	2.15	176	2.79	243	3.33
121	2.27	187	2.88	258	3.29

As Mr. Wegmann's contentions are based upon certain assumptions regarding pressures and the resulting consequences, and not upon methods of derivation and calculation, it would seem that Table 4 shows that such assumptions as were made gave consistent and logical results. Mr. Gowen.

Third.—Mr. Wegmann states that it would have been better if the contract drawings had shown a compound curve to define the upper part of the down-stream face instead of the right lines used, which were later modified in construction by curves joined with short tangents. In reply to this, it may be said that the contract drawings showed definitely and properly the section planned, and the curves substituted later were simply to smooth the general effect of the face, and the tangents joining these curves are not discernible.

Fourth.—As to the author's statement that a profile could easily have been designed with the up-stream face vertical, and that this face could have been continued to a depth of 200 ft. before reaching a pressure of 30 000 lb. per sq. ft., and that such a face would have been preferable on account of simplicity of construction, it must not be forgotten that in the building of the dam there was a depth of nearly 300 ft. to be provided for, and that the necessary batter below 200 ft. to provide for the increase of section required would be too great, for the reasons given above.

Finally, as to the contention that the dam should have been built at Quaker Bridge, it may be said that this site was carefully investigated and finally rejected, as it was deemed that the abutting hillside at the north end was not adequate to stand the pressures and probable leakage that would obtain. The Cornell site was not ideal, nor desirable, and was not recommended by Mr. Fteley as Chief Engineer of the Aqueduct Commission, when its construction was decided upon. The dam, however, has been built and is serving its purpose, and it would be a difficult matter to prove that the site on the whole is inferior to the original Quaker Bridge location.

In presenting his paper, Mr. Wegmann referred to the extended and prolonged foundation work at the New Croton Dam site as one of the reasons why the Quaker Bridge site would have been preferable. This reference is misleading, and for the following reasons:

The foundation conditions, as developed according to the original plans, proved to be as anticipated when the work was begun, and were dealt with adequately. The drastic change by which the masonry section was extended into the side hill, involved a very large increase in foundation work and expense, which was not anticipated by the people who advocated and insisted

Mr. Gowen. upon the change. The speaker feels free to say that this change was unnecessary and resulted only in loss of time and in undue expense, and he certainly cannot concede that the conditions thus developed are of any weight in support of the original Quaker Bridge site.

Mr. Dillman GEORGE L. DILLMAN, M. AM. SOC. C. E. (by letter).—The writer would call attention to the fallacy in argument by all masonry dam analysts who have developed or adopted uniform section types. Their argument (expressed or implied) is that because a uniform construction of a safe profile insures stability, uniformity is necessary to stability. This is not only not so, but the type suggested by the writer some years ago of a buttressed wall is more stable than any uniform section type, and incidentally contains considerably less masonry.

The Croton Dam is a great structure. The writer has no doubt of its stability; but a finer structure, architecturally, and a safer structure, against ordinary failure or earthquake effects, could have been built at materially less cost as a buttressed wall.

It is time the engineering profession cast off the fetters of antiquity and ceased to be bound by tradition. De Sazilly took a great step in advance. Rankine did great work in his various analyses. Engineers can study their work with profit, yet need not unquestioningly follow them. Analysts since their time have only refined their methods, whether with improvement being a matter of opinion.

Another thing: Masonry is more or less permeable. In the mathematics of Mr. Wegmann's analysis it is presumed that the upper face is tight. Suppose it is not. For the sake of safety, presume the tight surface to be one-half or three-quarters of the distance to the down-stream face. Where does all this analysis lead? It proves the dam to be absolutely unsafe. In one case the line of pressure (reservoir full) falls outside the middle third. In the other case the factor against overturning is less than one. The attempt to make a dam tight from face to face is a mistake. If the upper face is tight it will do no harm. But if the upper face is not tight and water is stopped at some other place, tightness at the second place is an element of weakness, and may be fatal.

The foregoing hypothesis is perfectly possible. The masonry may be permeable to some point below the up-stream face; it may be permeable to near the down-stream face. In the former case, the dam is less safe than the analysis shows. In the latter, a rational analysis would prove the dam absolutely unsafe.

Such a result could not obtain with the buttressed-wall type. In case of permeability, the factor of safety against overturning would be reduced, but never to less than 1, and the line of pressure would not be outside the middle third (reservoir full).

LUTHER WAGONER, M. AM. SOC. C. E. (by letter).—The historical part of this excellent paper, such as changes of plan, boards of experts, with attendant delays, reminds one of the struggle with the water problem in San Francisco during the past 30 years. The Croton Dam (a municipally owned and managed one) was discussed for about 10 years, and was constructed in the next 14 years; thus in 24 years, with ample engineering talent and plenty of money, New York has a completed dam; while San Francisco has advanced no further than the preliminary or talking stage, and the present average mental condition is that a new water supply is needed, but public opinion is still unformed as to how, when and where, it is to be obtained. Mr. Wagoner.

Persons interested in municipal ownership, *pro* and *con*, can read Mr. Wegmann's paper with profit.

The writer has read with much interest the reasons given for building a straight dam instead of a curved dam, as advised by the Board of Experts in 1888. One of the reasons given—that curvature might set up dangerous strains—deserves a passing notice, and, in the writer's opinion, this is due to the author's application of the hoop tension formula to the case of curved dams. In this formula,

$$T = pr$$

Where $T =$ the uniform thrust in a circular ring,

$p =$ the pressure per unit of length of the ring,

$r =$ the radius of the ring's outer surface.

While this formula is true as regards an external force such as would collapse a tube, it certainly does not apply correctly to dams, either curved or straight, for in the latter case, r, the radius, is infinite, hence the thrust, T, is infinite. But, as straight dams exist, and as it is also certain that they are not acted upon by infinitely great forces, it should be clear that the formula in question does not apply correctly. To hold that it does apply to curved dams, suggests the old problem of an irresistible force meeting an immovable body.

Another objection is urged against curvature, that it increases the quantity of masonry without any beneficial result. On page 426 the author states that with approximate dimensions of radius 1 200 ft. and a crest length of 1 350 ft., there would have been an increase of volume of masonry of about 10 per cent. This corresponds to a central angle of about 65 degrees. Within the limits of any useful curvature for curved dams, the excess of length of arc over chord varies as the square of the arc, or the percentage of excess of the arc over the chord $= 0.0013 1^2$, in degrees, which, for 65°, is about 5.5%, but, at the center there is no increase, and it should be remembered that the major portion of the

Mr. Wagoner. volume is ordinarily embraced by smaller angles than the crest. Taking the section used, and combining it with the profile in Mr. Gowen's paper, it appears that the foregoing curvature would have added about 2.8% to the volume.

Within the limits of useful curvature, it may be said that the increase in volume varies from 1¼ to 3%, depending upon the cross-section of the valley.

In collaboration with Mr. Hubert Vischer, the writer studied the Bear Valley Dam, and the results were published under the title "On the Strains in Curved Masonry Dams."*

In that paper some notice is taken of the author's hoop compression theory, previously mentioned. As to the further reason given for avoiding curvature—on account of the difficulty or rather impossibility of computing the stresses—the writer believes that in the paper on the Bear Valley Dam by Mr. Vischer and himself, methods of computation of such stresses have been set forth with sufficient clearness to give an approximate solution of such questions as may arise in practice. In that paper it has been shown, for the Bear Valley Dam, that the moment at the lower toe is reduced about 20%, this relief being due to arch action, while the increase in the volume of masonry is not much in excess of 1 per cent.

Elsewhere in that paper it is shown that much greater economies would have resulted from a better selection of radius and profile.

On page 420 of Mr. Wegmann's paper, it is stated that the Bear Valley Dam, built in 1884, has been replaced by a stronger one; this is an error, as the original dam is still in use. Some slight work has been done about 50 ft. down the stream for the foundations of a new dam, but it in no manner relates to the original work, which is apparently in good condition to-day, after 23 years of use.

As a matter of interest, Fig. 11 is introduced, the left-hand part being Mr. Wegmann's Practical Profile Type No. 2 to a depth of 80 ft., and the right-hand shaded figure being the profile of the Bear Valley Dam. The writer does not offer it as an example of good practice, but only as confirmatory of his final remarks about the necessity for a revision of current ideas about permissible unit stresses.

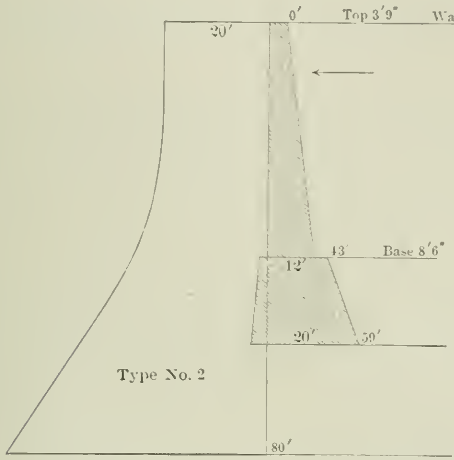
Arch action is always worth much more than its cost in volume of masonry, for by its use the strain upon the lower toe can be relieved, and this in turn is sufficient warrant for some reduction in the cross-section of the dam.

Aside from æsthetic considerations, it also provides for automatic relief from stresses due to temperature changes, and renders a dam safe against ice thrust and wave action, and probably all but the most severe earthquakes.

* *Proceedings*, Technical Society of the Pacific Coast, December, 1889.

As an illustration, take the case of the Austin Dam. Had this dam (a long, low one) been arched, perhaps to a central angle of not more than 30°, involving about 1% increase in the volume of masonry, there can be no reasonable doubt that, even had it cracked under the sudden load due to the flood, its parts could not have been overturned, or pushed bodily by sliding down stream, as occurred. Mr. Wagoner.

The writer, being a known advocate of curvature for masonry dams, has been criticised for the use of both a gravity section and curvature in his design for the La Grange Dam, but, in that case,



COMPARATIVE PROFILES OF MR. WEGMANN'S TYPE NO. 2 AND THE BEAR VALLEY DAM.

FIG. 11.

the conditions were different. The water-shed above the dam is 1 500 sq. miles, and the high-water mark of the flood of 1862 was found near the dam site, and at several points above it, from which it was computed that 130 000 cu. ft. per sec. might have to pass over the crest, and fall 110 ft. Such a flood would make about 19 ft. of water over the crest. The plans were competitive, and it was known that the consulting engineers did not regard favorably any reduction from a gravity section. The owners desired security against any accident, and it appeared that the possible loss due to a failure of the dam might be a thousand fold more than the cost of extra volume due to curvature. Any one of the above reasons are deemed sufficient answer for the choice.

Mr. Wagoner. The methods of computation set forth by Mr. Wegmann will be of convenience to the profession, as far as methods of procedure go, but it has long been in the mind of the writer that too much weight is attached to precedent as to permissible loads.

Reading Mr. Gowen's paper, which carefully sets forth in detail the methods used in setting the stone in the New Croton Dam, it would appear to be entirely possible that there is a factor of safety of from 10 to 15 for any possible load.

How long shall engineers continue to worship the "Sacred Cow," or permissible loading of about 200 lb. per sq. in. for good solid masonry?

In structures of concrete, twice this value is recognized as good practice. For the nearly constant and quiescent load upon a dam, is there any good reason why a lower factor of safety could not be used?

This is a very important subject for those who plan dams, because in many cases it is the extra height gained that determines the storage capacity. Many new engineering projects might become possibilities if higher unit loading was allowed. Finally, does not the whole subject of permissible loads upon first-class masonry require revision?

Mr. Cain. William Cain, M. Am. Soc. C. E. (by letter).—The author has derived a very convenient set of formulas for the design of a high dam when subjected to the five conditions imposed. Exception can be taken, however, to some of the conditions. Thus, if the center of pressure on any horizontal joint for quiescent forces (water pressure and weight of masonry) reaches the middle-third limit, then, by the common theory (provisionally adopted), any accidental or extraordinary forces added will cause tension at one end of the joint. If the joint there should crack and open, admitting water, the structure would be materially weakened. Such accidental forces may be due to wind and wave action, floating ice or other bodies, earthquakes, and the expansion of ice under an increase of temperature.

The writer* has given the results of "a preliminary study" on this subject for a dam 258 ft. high, subjected to water pressure on one side. As in the Quaker Bridge Dam, the weight per cubic foot of the masonry was taken as $2\frac{1}{2}$ times that of water. The diagram used in that communication, with dimensions, unit pressures and factors of safety marked on it, is reproduced herewith as Fig. 12.

The claim was made that—in addition to the three universally imposed conditions, no tension, safe unit stresses, and no possible sliding for any horizontal joint—a fourth condition must be im-

* *Engineering News*, June 23d, 1888.

posed, *viz.*, that the factors of safety against overturning and sliding shall increase gradually from the base upward to allow for the proportionately greater influence, on the upper joints, of the accidental forces mentioned above. Mr. Cain.

It was found that this could easily be done by taking the well-known theoretical triangular type, with some additions at the top to allow for a roadway and to aid in counteracting the accidental and extraordinary forces.

The upper figure of any one of the "factors," marked in Fig. 12 in the form of a fraction, gives the so-called factor of safety

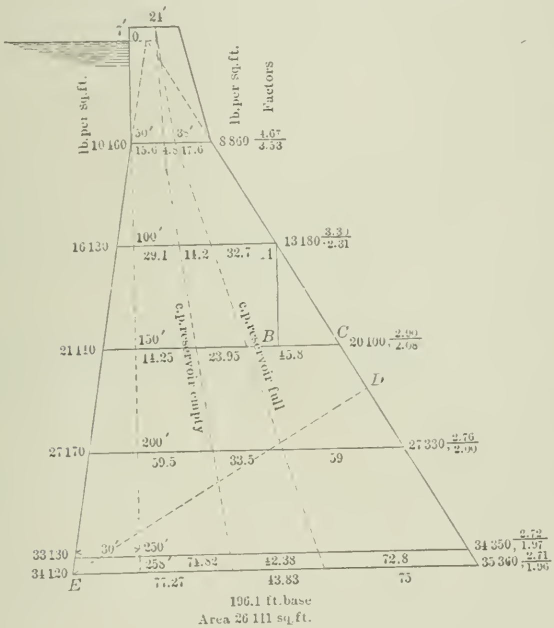


FIG. 12.

against overturning, or the factor by which it is necessary to multiply the horizontal thrust of the water to cause the resultant to pass through the outer edge of the joint considered. The lower figure gives, for any joint, the ratio of the weight of the masonry above a horizontal joint to the thrust of water corresponding, which is, in a sense, a factor of safety against sliding on a horizontal joint.

The maximum vertical pressure per square foot at the outer toe, as computed by the author's Formula A, is 35 360 lb., or 246 lb. per

Mr. Cain. sq. in., which is less than the pressure to which concrete is subjected in current practice every day.

As the cross-section has about the area of the Fteley design of the Quaker Bridge Dam, and a less spread at the base, it should cost less in construction. If the maximum unit stress is admissible with perfect safety, then it is submitted that the design is stronger, for forces ordinary and extraordinary, than the Fteley design, where the factors against overturning actually decrease from near the base to 66 ft. below the surface of the water, above which they increase again.

Of course, this design, which was only a first study, could be improved, perhaps, by changing the batters and top widths, also by rounding off with a curve the down-stream face at the angle, but it is submitted that the principle, that the factors of safety against overturning and sliding should decrease from the top down, should be observed where the accidental or extraordinary forces are not originally included.

In the beautiful design, Fig. 6, by the "Board of Experts," the accidental forces are included from the start, leading to a very strong dam. Here, an ice pressure of 43 000 lb. per lin. ft. acting at the surface of the water was included from the first, which required an additional section supposed to be ample to resist any other accidental forces, including earthquakes.

It is hoped that additional information with regard to forces produced by ice expansion will come out in the discussion on this paper. An item in *Engineering News,* June 30th, 1888, on the authority of Thomas C. Keefer, Past-President, Am. Soc. C. E., is to the effect that:

"An ice bridge of about 90-ft. span, between two fixed abutments, expanded so from a rise of temperature as to rise 3 ft. in the center."

For ice 1 ft. thick, the horizontal thrust resulting would then be about 21 000 lb.

The distribution of the vertical component of the pressure on any horizontal joint is given approximately by the author's Formula A; but, as Rankine points out, this is not the whole pressure, including the horizontal component likewise; in fact, the direction of the normal pressure close to a battered face must be parallel to that face. Hence, he recommends, that there be taken a lower limit to the unit pressure (as given by Formula A) at the down-stream face than at the up-stream face.

The usual hypothesis, that, for any reservoir wall, plane horizontal sections before stress remain plane sections after stress, is open to doubt. For a wall with vertical faces, subjected only to its own weight, the hypothesis is true, since the weight of each particle can be transmitted vertically to the foundation, giving rise to a uniform stress on each horizontal joint. Mr. Cain.

If a load, uniformly varying from face to face, is added at the top, the stresses corresponding can again be transmitted vertically, and the stress on any horizontal joint is a uniformly varying one. In these cases, horizontal plane sections before stress remain plane sections after stress, always supposing the limit of elasticity of the material not to be exceeded. But, for a wall with inclined or curved faces, the stress is not transmitted vertically, and there is a well-grounded belief that the stress on any horizontal joint is not uniformly varying, and that Formula A gives an excess of vertical unit pressure at the face considered. Thus, consider the triangular wedge-shaped portion, *A B C,* Fig. 12: Its lower face is subjected to an upward pressure; it thus acts as a cantilever, and, of course, offers less resistance than a rectangular cantilever, corresponding to a stepped face. If the lower horizontal joint is to remain plane after strain (as the theory supposes), the stress at the lower edge of the wedge, for the battered wall, must be less than the corresponding stress for the stepped wall; since, for the same deformation, the stress is less for the weaker cantilever. Thus, even if the stress be supposed to be uniformly varying for the stepped wall, it cannot be so for the battered wall, and *vice versa.* This shows no more than that the assumption of uniformly varying stress on plane horizontal sections is not universally true for all kinds of faces.

The matter is still further complicated, where, in addition, the thrust of the water is considered. Although the stresses on a horizontal section probably increase up to a certain point and then decrease to the face, it is certainly safe to use Formula A—which supposes a uniform increase—in finding the maximum vertical unit pressure at either the up-stream or down-stream face of the reservoir.

It might be thought that, by combining this vertical (normal) unit stress, thus found by Formula A, with an assumed unit shear on the horizontal joint (perhaps the average), the total normal stress, next the face and acting parallel to it, could be found; but, by known formulas, this procedure will generally lead to shear on a plane perpendicular to, and therefore along, the battered face, which is absurd.

As a last resource, an upper limit could be found by taking sections, as *E D,* Fig. 12, perpendicular to the battered face. On this assumption, we combine the entire thrust of the water for the total depth to *E,* with the total weight of the masonry lying above the plane, *E D.* If the resultant strikes *E D,* at the distance, u, from *D,* and we denote by l, the length *E D,* and by W, the normal

Mr. Cain. compound of the resultant on $E D$, then, adopting the usual hypothesis, that plane sections, as $E D$, before strain, remain plane sections after strain, it follows that the author's Formula A will give the required normal stress at D, acting parallel to the face at D. The error in this procedure lies in adopting the hypothesis of the conservation of plane sections, as $E D$, before and after strain. In fact, suppose a horizontal section at D: After strain, if the stress at the up-stream face is zero, the vertical deflection of its center should be half that at D; similarly for the section, $E D$. But, if the up-stream face is vertical, this entails equal vertical deflections of two points in the same vertical at different elevations, which is absurd. The vertical deflection at the center of $E D$, and therefore the stress there, should be greater than at the center of the horizontal section through D. The resulting stress parallel to the face at D, as computed above, is thus too great.

As a numerical illustration, the normal stress thus found, acting parallel to the down-stream face, at the point where the vertical unit stress is 20 400 lb. per sq. ft., was found to be 28 800 lb. per sq. ft., which is doubtless excessive. At present, therefore, we are forced to depend upon Formula A for horizontal sections. If this is supposed to give the whole unit stress at the down-stream face, as well as at the up-stream face, then, as Rankine suggests, the permissible unit stresses at the latter should be taken larger than at the former.

The ratio between the two can only be a matter of judgment. Of course, it leads to more nearly vertical up-stream faces.

A word may appropriately be added about earthquakes, for a high dam is more likely to be rocked and subjected to tension along the upper joints than a low dam. Milne says,* that during the Yokohama earthquake of 1880, when the maximum amplitude of the earth's motion was probably less than ¾ in., the upper story of a house rocked from side to side 1 ft. out of its vertical position. The writer witnessed a similar swaying of the brick walls in the third story of a very large building during an earthquake in Charleston, S. C., though the amount was not measured.

Fortunately, the tops of dams are not as free to move as houses, because they are connected with the sides of the valley; but an earthquake must cause some motion, which may cause tension in some joint which has been pared down until the resultant on it just passes through a middle-third limit. In the writer's opinion, such a wall has not a sufficient factor of safety. In fact, considering that the extraordinary forces, as earthquake shocks, ice expansion, etc., cannot be known beforehand, or measured, and further, considering the fact that in the present state of science, even the

* In his book on "Earthquakes," p. 114.

maximum unit stresses for quiescent forces alone cannot be computed exactly, it would seem to be unwise to enter into too great refinements as to the profile, provided the dam has sufficient factors of safety everywhere. Mr. Cain.

One method of failure of loosely constructed dams is by sliding along some inclined plane. In the writer's "Retaining Walls,"* is given a graphical treatment of this case. In the case of the Habra dam, 116 ft. high, the section having been pared down to a minimum, and the dam not being constructed of the best materials, it broke along a plane passing nearly through the outer toe, and making the angle of friction of masonry on masonry with the horizontal. From the method of construction of the New Croton Dam, there should be no fear of such sliding. Still, if this dam should ever fail, it will probably be from cracking first on the up-stream face from too great tension, and this will be followed by shearing along an inclined plane, as in the case of the Habra Dam.

It may be inferred from what precedes that the writer prefers the design of the Board of Experts, Fig. 6, to the Fteley design. It was a great step in advance to include the extraordinary forces from the start, and the single horizontal force assumed acting at the top can not only provide for ice expansion but for earthquakes, though the action is not exactly the same. The main point is that such an assumption leads to increasing factors of safety from the bottom up, where only quiescent forces are considered, which increase provides, in a rude way, for earthquake shocks, ice action, etc. When this is done, the conditions imposed, that there shall be no tension or sliding, and that the unit stresses everywhere shall be safe, will probably be realized for both quiescent and accidental forces.

As to the safe unit stresses, the writer has long been of the opinion that the upper limit usually adopted is entirely too low. Why is not a compressive stress of 300 lb. per sq. in. permissible?

Granting that it is, then the great spread at the base, as in the various designs, can be avoided, and a uniform batter can be used below certain levels, for both down-stream and up-stream faces.

Where this is done, the theoretical triangular form can be used, with some additions at the top, to give a satisfactory design, as first suggested by the writer. He is glad to see the trend in that direction, as evidenced by the author's Fig. 8, as it is in the line of simplicity.

EDWARD WEGMANN, M. AM. SOC. C. E. (by letter).—The points raised by Mr. Gowen call for some further explanations: Mr. Wegmann.

First.—As regards the assumed limits of pressure:

On the drawing of the profile finally adopted for the proposed

* Van Nostrand's Science Series, No. 3, p. 167.

Mr. Wegmann. Quaker Bridge Dam (which is on file in the office of the Aqueduct Commissioners) there is the following statement:

"Notes, Data, etc.

"Theoretical Profile. Limiting Conditions.

"1. Both resultants to intersect every horizontal joint within its middle third.

"2. Limits of pressure above El. 96 .... { Down-stream face, 16 406 lb. per sq. ft. / Up-stream " 20 625 " " " "

"3. Below El. 96 both faces to have a straight batter to bed-rock (El. —52).

"Pressure at latter point (El. —52) to equal 30 000 lb. per sq. ft.

"Weight of gravel, vertically over base (El. —52), to be taken into account.

"Equations for theoretical profile derived by Wegmann's method."

The conditions and limits of pressure given in the foregoing statement agree with what the writer has stated on page 418, with the trifling difference of 6 lb. per sq. ft. in the limit of pressure at the down-stream face above Elevation 96.

It so happened that at Elevation 96 the pressure at the down-stream face had practically reached the assumed limit, while the pressure at the up-stream face was more than 3 000 lb. below the limit adopted for that face.

Mr. Gowen points out that the differences between the calculated pressures at the down-stream and up-stream faces, to a depth of 110 ft. from the top, are not great, although, below this depth, the differences are somewhat greater, and he says it seems to him that:

"The ultimate up-stream face pressures should be limited in the same way as on the down-stream face; hence the 30 000-lb. per sq. ft. limit, as shown in Table 2."

This is going back to the first writers on the design of masonry dams—De Sazilly, in 1853, and Delocre, in 1866—who assumed the same limits of pressure for the down-stream and up-stream faces of the dam. As stated on page 409, Rankine, in 1871, recommended that a lower limit be adopted for the maximum pressure at the down-stream face of the dam than for that at the up-stream face, the limits assumed by him in his logarithmic profile being, respectively, 15 625 and 20 000 lb. per sq. ft.

The correctness of Rankine's view of this matter has been generally accepted, and recent writers on the subject of masonry dams, such as Bouvier, Guillemain, Hétier, and Lévy, accomplish practically the same result as Rankine by considering, for the case of "reservoir full," the distribution of pressure on inclined instead of on horizontal joints.

By cutting off the thin down-stream toe of masonry, the maximum pressures at the base at the down-stream and up-stream faces were changed from 30 000 lb. per sq. ft. to 33 266 and 30 828 lb., respectively, thus reversing Rankine's conditions that less pressure should be allowed at the down-stream face than at the up-stream face. Mr. Gowen claims that: Mr. Wegmann.

"This increase in the toe limits, as shown by Table 3, is not significant, in view of the generally admitted proposition that the toe pressures at the base of such a profile are less by a considerable amount than those calculated."

The writer agrees with Mr. Gowen that the formulas used for calculating the pressures at the faces of a dam, which are based on the assumption that the pressures in a dam at any assumed joint are distributed as "a uniformly varying stress," exaggerate, in all probability, those pressures, but this is just as true for the up-stream as for the down-stream face; and, if a pressure of 33 000 lb. per sq. ft. is permitted at the latter face, then, just as safely, according to the present state of knowledge, about 38 000 lb. per sq. ft. can be allowed at the former face.

With regard to cutting off the thin toe of masonry at the down-stream face, Mr. Fteley says, in his report of July 25th, 1887, on the proposed Quaker Bridge Dam, that this was done because no dependence could be placed on the strength of the sharp triangle of masonry forming the down-stream toe of the theoretical profile, and because it presented the additional advantage of lessening the width of the expensive excavation to be made in the bed of the river. The very manner in which vertical steps were substituted for the inclined face, at the toe, indicates that the distribution of a vertical pressure on the base of a dam, and not the distribution of inclined pressures parallel to the face, was considered.

Rankine states that:

"The direction in which the pressure is exerted among the particles close to either face of the masonry is necessarily that of a tangent to that face."

This principle has been proved mathematically by Lévy. The conditions of pressure at the down-stream toe of a dam, therefore, are more like those in an arch at its skewback than like those of a pier or wall having only vertical pressures to distribute. The writer thinks that, for the above-mentioned reason, the down-stream batter should have been continued to bed-rock, in order to bring the inclined pressures at the face to a solid skewback of rock.

In a paper, "On Some Disregarded Points in the Stability of Masonry Dams,"* Messrs. L. W. Atcherley and Karl Pearson have

* Published in "Drapers' Company Research Memoirs," Technical Series II, London, 1904.

Mr. Wegmann. shown that, in testing the stability of a dam, it is not sufficient to make calculations at different horizontal cross-sections of the dam, as is usually done, but that they should also be made for vertical cross-sections. They found, both theoretically and by experiments on models of dams, that a dam collapses first by tension on the vertical sections of the toe, and that the shearing of the vertical sections over each other follows immediately after this opening up by tension. The models on which they experimented, some of horizontal, and others of vertical, layers of wood, had reinforcements at the down-stream toe. In accordance with the conclusions of Messrs. Atcherley and Pearson, the down-stream toe of the New Croton Dam should have been reinforced instead of being cut off. The writer refers, of course, to that part of the toe which is above bed-rock.

Second.—As regards the maximum pressure at the faces of the dam, the early writers on the subject of masonry dams (De Sazilly and Delocre) designed what they called "profiles of equal resistance," the maximum pressure at the faces remaining constant at an assumed limit of safety, except near the top of the dam, where the pressures were necessarily less than this limit.

In the design for the Quaker Bridge Dam the limits of pressure assumed above Elevation 96 were not reached until at about Elevation 76 at the down-stream face, and at about Elevation 56 at the up-stream face (Table 2). From these elevations down, the pressures increased until the maximum of 30 000 lb. per sq. ft. occurred at the base. As a matter of fact, the pressures increased steadily, although somewhat irregularly, from the top to the base. What was accomplished, therefore, by assuming limits of pressure above the arbitrarily assumed Elevation 96? Would it not have been more logical to have designed the profile from the top downward simply by Rankine's principle that the lines of pressure for reservoir full or empty should be kept within the center-third of the profile, until some assumed limits of maximum pressures at the down-stream and up-stream faces had been reached, after which the profile should have been designed to keep the maximum pressures at the assumed limits?

By keeping the limits of pressure within the center-third of the dam a factor of safety of 2 against overturning is secured, and sufficient resistance to sliding and shearing. Nothing is gained by having the factors of safety against overturning increase to 3.29 at the base of the dam, as shown in Table 4. Below the river-bed, where the dam has support from the refilling, the structure is less likely to be overturned than above the river-bed. If any differences were made in the factors of safety against overturning, these factors should be made greater in the upper part of the dam than at the base, in order to provide ample safety against ice pressure and shock from floating bodies. Mr. Wegmann.

In recent years three dams have been built in France (Echapre, Cotatay and Ondenon), with profiles according to Fig. 13, which would indicate that French engineers appreciate the merits of the simple triangular profiles shown in Fig. 8, which is based solely on the principle of keeping the lines of pressure within the center-third of the dam.

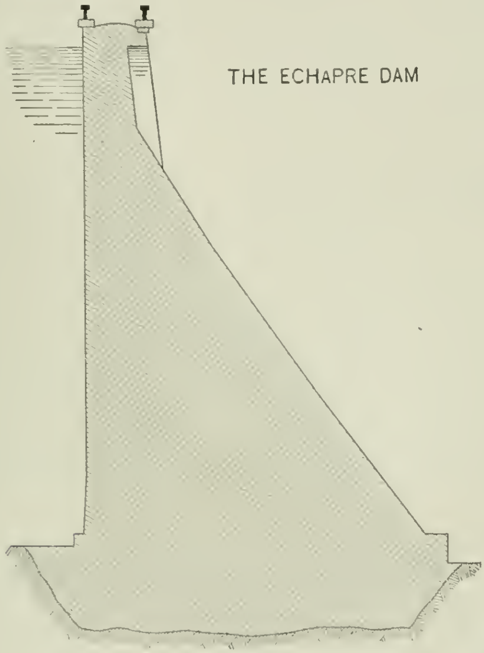


FIG. 13.

The Echapre Dam has a maximum height of 37 m. (121.4 ft.) above the foundations. Its up-stream face is vertical, and the down-stream face is vertical for a short distance at the top and is then built on one batter, except that the angle made by this batter with the vertical face is rounded off by a circular curve. The Cotatay and Ondenon Dams have similar profiles. What could be simpler than these profiles?

Third.—As regards the outlines of the profile adopted for the New Croton Dam, which is shown by Fig. 7, the down-stream face consists of a polygonal line with batters changing about every 10

Mr. Wegmann. ft., except below Elevation 90, where two batters, differing but slightly, are adopted. The part of the face above Elevation 90 looks very much like a compound circular curve, and, in actual construction, the engineer in charge adopted three arcs of circles for the face above that elevation, joined by two short tangents—one of 26 in. and the other of 22 in. Since the dam has been built, a compound circular curve, without any tangent, has been fitted in the Drafting Bureau of the Aqueduct Commissioners to the downstream face of the dam above Elevation 90. This compound curve does not differ more than ¼ in. at any joint from the dam as actually built. Would it not have been a little more in keeping with the profiles usually adopted for masonry dams, if the compound curve had appeared in the contract drawings, instead of the numerous changes of batter?

Fourth.—As regards the adoption of a vertical up-stream face, the writer thinks that that face of the dam could have been safely made vertical from the river-bed to the top—a height of 163 ft.—according to the contract drawings. If this had been done, the pressure at the up-stream face at the river-bed would have been about 25 000 lb. per sq. ft., and from this point a straight batter could have been adopted to make the pressure at the base 30 000 lb. per sq. ft., or even 35 000 lb., if the former pressure were allowed at the down-stream face. The writer appreciates the necessity that existed of reducing the excavation below the river-bed as much as possible, but, instead of doing this by cutting off part of the down-stream toe, he would have made the up-stream face steeper, approaching as nearly as possible to the simple profile shown by Fig. 8.

Fifth.—As regards the change in the location of the dam: After thorough explorations by diamond drills, the location of the proposed dam at the Quaker Bridge site, which was originally recommended in 1883 by Chief Engineer Isaac Newton, of the Department of Public Works, and his Consulting Engineers, E. S. Chesbrough and Julius W. Adams, was accepted in 1887 by Chief Engineer Church of the Aqueduct Commissioners, and his Consulting Engineer, A. Fteley, as well as by a Board of Experts, consisting of Joseph P. Davis, J. James R. Croes and William F. Shunk. In the reports by these prominent engineers, all of whom, with one exception, were Members of this Society, there is not a word said about the hillside at the north end of the dam being inadequate to withstand the pressures, or that leakage might occur at this point, nor is this fact mentioned by Mr. Fteley (who had become Chief Engineer) in his final report of October 8th, 1890, to the Aqueduct Commissioners on the subject of building a high dam across the Croton Valley somewhere between the Old Croton Dam and the Quaker Bridge.

In this report, Mr. Fteley discussed three possible locations for the proposed dam, including the one at Quaker Bridge and one about 1¼ miles farther up stream, known as Cornell's, where the dam was finally built. All Mr. Fteley said about the latter site was that it should be carefully considered, before deciding on a final location. Mr. Wegmann.

Without any other recommendation than this, as the records show, and without further investigation, the Aqueduct Commissioners decided, on January 22d, 1891, by a divided vote of 5 to 2, to build the proposed dam at the Cornell site. Mr. Fteley was very much surprised at this action, and informed the writer that this change was not made for engineering reasons.

It was well understood, at the time, that this action was in the nature of a compromise with an influential citizen who, for reasons of his own, strongly opposed the construction of the dam at the Quaker Bridge site, but had no objection to a high dam if it were built farther up stream. The advantages possessed by the Quaker Bridge site have been mentioned on page 432.

Mr. Dillman refers to the advantages obtainable by building a dam as a buttressed wall or as a number of piers joined by arches, as recommended by him in his paper entitled "A Proposed New Type of Masonry Dam."* Henry Goldmark, M. Am. Soc. C. E., has described† a design for a dam 100 ft. high, consisting of piers and arches, which was to be built near Ogden, Utah. The bids for this dam, designed as mentioned, and for one designed as an ordinary gravity dam, showed a saving of about 15% in favor of the former plan. Water-tightness was to be secured by covering the whole up-stream face of the arches with steel plates. This dam was never built, but a dam of similar design, about 60 ft. high, has been constructed at Belubula, New South Wales.‡

The writer has had occasion to design a dam of this type, 160 ft. high, and agrees with Mr. Dillman that a saving in material may be effected by building piers and arches instead of a dam of uniform cross-section. Whether such a dam can be made as water-tight as one built with a uniform cross-section remains to be proved by actual construction. The dam of the future may be built as suggested by Mr. Dillman, but in 1892 there was no precedent that would have warranted designing the New Croton Dam—which was to have a height of nearly 300 ft.—for piers and arches.

Mr. Wagoner objects to the application of the hoop-tension formula to the case of curved dams. That formula was proposed by Rankine for curved dams, and was used by Mr. Goldmark in calculating the strains in the arches of the proposed dam at Ogden,

* *Transactions*, Am. Soc. C. E., Vol. XLIX, p. 94.

† *Transactions*, Am. Soc. C. E., Vol. XXXVIII, p. 290.

‡ *Engineering News*, Vol L, p. 500.

Mr. Wegmann. previously mentioned. Although this formula may not be applicable to curved dams of considerable length, it may safely be used, in the writer's opinion, for small arches like those proposed for the Ogden dam.

The writer agrees with Mr. Wagoner that much greater pressures than those usually allowed in masonry dams may be safely sustained by good masonry. That is one of the reasons which inclines him to recommend the adoption of the simple triangular profile shown by Fig. 8 to a depth of 200 ft., or even more.

The writer thinks that Professor Cain is correct in saying that the factors of safety against overturning and the resistance to

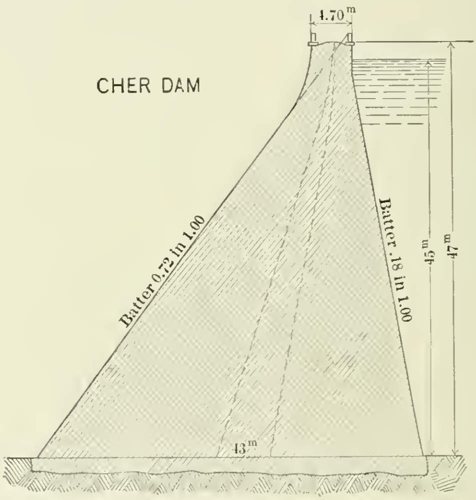


FIG. 14.

shearing should be greater for the upper than for the lower joints of a dam. The fact that the factors of safety usually increase toward the base is not due to design, but to the necessity of drawing the lines of pressure nearer to the center-line of the profile at the base, in order to reduce the pressures at the faces.

The simple design for a dam, proposed by Professor Cain (Fig. 12), has been adopted for the Cher Dam in France (Fig. 14), which is now under construction and will have a height of about 148 ft. The writer is of the opinion that, for the conditions which have hitherto been generally assumed in the design of masonry dams, the profile shown by Fig. 9 is the most economical, but he prefers the profile shown by Fig. 8 on account of its simplicity. Neither of these profiles complies with the additional conditions proposed by Professor Cain, *viz.*, that the factors of safety against overturning and shearing should increase from the base to the top. With these conditions, added to those usually adopted, Fig. 12 may be found to be preferable. Mr. Wegmann.

As regards the question of ice pressure, the writer thinks that, although ice confined, as between bridge piers of short spans, exerts a very great pressure, such a pressure could not occur in a reservoir of any great extent. With few exceptions, the masonry dams now in existence were designed without considering ice pressure, and thus far no damage has been recorded on account of this omission.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1048.

RAINFALL, AND RUN-OFF IN STORM-WATER SEWERS.*

By Charles Emerson Gregory, Assoc. M. Am. Soc. C. E.

With Discussion by Messrs. William C. Hoad, A. Marston, W. B. Ruggles, Emmet A. Steece, L. J. Le Conte, Charles B. Burdick, and Charles Emerson Gregory.

The object of this paper is to amplify and bring together matter contained in previous discussions of this subject, and to propose a more rational method of solution.

Unfortunately, the experimental data presented are not entirely conclusive, and it is hoped that this paper may inspire the making of further experiments, and induce other investigators to submit additional data for comparison.

Among the earliest data which gained the notice and, for a long time, the approval of the engineering profession are the observations of rainfall and run-off made from gaugings in the sewers of London, England, in 1854, by Roe, Bidder, and Bazalgette, and interpreted by Hawksley. Very little is definitely known about the details of these experiments. Two definite conclusions, however, seem to have been reached. One was that the run-off during a storm is about 40 to 54% of the rainfall; and the other, that 1 in. of rain per hour is a safe maximum for which to provide. The latter conclusion, especially, seems to have been for a long time generally accepted both in America and in England, and still has such a firm hold on

* Presented at the meeting of February 20th, 1907.

the fancy of those who have not studied the matter deeply that they cannot look at the subject from any other viewpoint.

These general statements have been reiterated by authorities in many textbooks, and in most cases without the qualifications necessary for a proper understanding of their meaning. It is not usually stated whether the 40 to 54% run-off is a rate of run-off (the fact most necessary to know in designing a drain); or a total run-off; or whether it is a percentage of the maximum rate of rainfall; or of an average rainfall; or of the total rainfall in a single storm; or of the rainfall in an hour. The maximum rate of 1 in. per hour was apparently taken to be of equal weight and effect, whether it fell at a uniform rate, or whether it all fell in 15 or 20 min.; and was supposed to be the same for all water-sheds of whatever size or shape.

From these two general conclusions, supplemented probably by personal observations, was developed the empirical formula, of the type used by Hawksley and Adams, in which the run-off varies with the size and slope of the water-shed, but in which Jupiter Pluvius was so firmly bound by the inch-per-hour tradition that it was never suspected that he would deviate from the path prescribed for him.

The following notation will be used throughout this paper to designate the various factors:

Q = Discharge, in cubic feet per second;
A = Drainage area, in acres;
S = Slope of surface, or of sewer, in feet per thousand;
R = A rainfall coefficient, sometimes considered to be a maximum rate for a short period, in inches per hour; sometimes as the average rate per hour; but usually not well defined;
N = The length, in feet, in which the sewer falls 1 ft.;
d = Diameter of circular sewer, in inches;
D = Diameter of circular sewer, in feet;
r = Hydraulic mean radius;
v = Velocity, in feet per second;
s = Sine of slope of sewer;
C = A coefficient, depending upon the extent of impervious surface, or the ratio of rainfall to run-off;
i = Intensity of rainfall, or rate, in inches per hour;
t = Time, in minutes.

Hawksley's formula, in its original form (derived from Roe's tables*), is as follows:

$$\log. d = \frac{3 \log. A + \log. N + 6.8}{10}.$$

If N is replaced by its equivalent $\frac{1}{s}$, and the formula is divested of its logarithms, it becomes

$$d^{10} = 6\,309\,574 \frac{A^3}{s},$$

and, if d is reduced to feet, then $d = 12\ D$, and

$$D^{10} = 0.0001019 \frac{A^3}{s}.$$

This formula is still used in the Borough of Brooklyn, New York City, except that, for the large sewers, the last term of the numerator is changed from 6.8 to 8.0, and would therefore appear as

$$d^{10} = 100\,000\,000 \frac{A^3}{s},$$

and $$D^{10} = \quad 0.001615 \frac{A^3}{s}.$$

This formula may be transcribed, so as to give Q in terms of A and S, by substituting in $v = c \sqrt{r\ s}$, the fundamental formula for the flow of water, thus:

If $c = 100$, then $v = 50 \sqrt{D\ s}$, and $Q = \dfrac{A - D^2\ r}{4}$.

Therefore, $Q = 39.27 \sqrt{D^5\ s}$,

and $$D^{10} = \left(\frac{Q}{39.27}\right)^4 \times \frac{1}{s^2}.$$

But, as $$D^{10} = D^{10} \left(\frac{Q}{39.27}\right)^4 \times \frac{1}{s^2} = 0.0001019 \frac{A^3}{s},$$

and $$Q = 3.946\ A \sqrt[4]{\frac{s}{A}},$$

and since $$s = \frac{S}{1\,000},$$

$$Q = 0.7\ A \sqrt[4]{\frac{S}{A}}.$$

In the same manner,

$$D^{10} = 0.001615 \frac{A^3}{s}$$

* Report of Commission on Metropolitan Drainage, London, 1857.

reduces to $Q = 1.4\ A \sqrt[4]{\frac{S}{A}}$, which is twice as great a value for Q as in the original form. This change may be explained by introducing a rainfall coefficient, the value of which in the original form was unity and now becomes 2. The general form may then be written:

$$Q = C\ R\ A \sqrt[4]{\frac{S}{A}}$$

in which $C = 0.7$ and $R = 1$ in the original form, or 2 in the amended form, and is supposed to provide for a rainfall of 2 in. per hour, but the length of the period is not stated.

From this formula as a basis have been developed nearly, if not quite, all the later modified forms of the empirical exponential formulas.

The formula by the late Julius W. Adams, Past-President, Am. Soc. C. E., as published in 1880, was deduced as follows: In the expression,

$$D^5 = \left(\frac{Q}{39.27}\right) \times \frac{1}{s},$$

as previously developed from the formulas for the flow of water, in the demonstration of the Hawksley formula, the exponent of D is changed from 5 to 6, and for Q is substituted $\frac{A}{2}$, on the assumption that $\frac{1}{2}$ in. of rain per hour will reach the sewer, thus giving

$$D^6 = \frac{A^2}{6168\ s}.$$

and, for any other value for R than unity, substitute $\frac{A\ R}{2}$ for Q,

then $$D = \sqrt[6]{\frac{A^2\ R^2}{6168\ s}},$$

but D is also equal to $\sqrt[5]{\frac{Q^2}{1542\ s}}$,

therefore $$\sqrt[6]{\frac{A^2\ R^2}{6168\ s}} = \sqrt[5]{\frac{Q^2}{1542\ s}},$$

whence $$Q = 1.035\ R\ A \sqrt[12]{\frac{s}{A^2\ R^2}}.$$

If $\frac{S}{1\ 000}$ be put in place of s, then

$$Q = 1.837\ R\ A \sqrt[12]{\frac{S}{A^2\ R^2}}.$$

The Bürkli-Ziegler formula is identical in form with the reduced Hawksley formula, and has been used extensively in Europe. It differs from the original Hawksley formula only in that the values of the constants, C and R, have been modified to a greater or less extent.

As American engineers began to investigate this important subject on their own account, and as designs made on the old theories began to prove inadequate, the so-called rainfall coefficient, originally put down as unity, was increased in order to fit more nearly the new conditions. These changes were brought about by several more or less complete scientific investigations, and, while no attempt will be made to give herein an accurate or complete history of this part of sewer design, a few important mile stones along the way will be noted.

One of the first investigations was by R. E. McMath, M. Am. Soc. C. E., on the sewers of St. Louis, Mo. The details* will not be given here, but only a brief summary of the essential points and the results.

A tabulation was made showing all existing trunk sewers and branch sewers, with their capacities calculated by Kutter's formula, the number of acres tributary, the average slopes of the flow lines, and brief descriptions of the surface conditions. The St. Louis sewers had been built from time to time under the direction of different engineers, each of whom had his own peculiar ideas of what was required; consequently, the ratio of area drained to sewer capacity had a wide range.

Next, a record of the adequacy of each sewer was made, the capacities, in cubic feet per second, being plotted as ordinates, and the tributary acreages as abscissas, and all inadequate points were marked. A curve, including all inadequate points, was then drawn, and its equation was found to agree with the following formula, now commonly known as the McMath formula:

$$Q = A\ C\ R \sqrt[5]{\frac{S}{A}}.$$

To make this formula fit the curve described, the following values were given to the different factors:

$$C = 0.8,$$
$$R = 2.75,$$
$$S = 15.$$

* *Transactions*, Am. Soc. C. E., Vol. XVI. p. 179.

Below this curve there was a kind of neutral zone in which some points were inadequate and some were adequate, on account of flat slopes, undeveloped areas, etc.

In most cases it was found that, by applying the correct values for the slope and the surface coefficient, the formula gave approximately correct results.

This formula, as far as the writer knows, is only an empirical modification of Hawksley's exponential formula, and is typical of those most generally used.

Soon after this, two other investigations were made: one by E. Kuichling, M. Am. Soc. C. E., and the other by Rudolph Hering, M. Am. Soc. C. E.

Mr. Kuichling's observations related to sewers draining well-developed territory, a moderately large percentage of which was impervious; territory only partially developed; and residential neighborhoods having a small percentage of impervious area. The observations, like those by McMath, were made without the aid of clock gauges for either the rainfall or the depth of flow in the sewers, but the rate of rainfall for short intervals was carefully timed by observers stationed at the gauges. The depths of flow in the sewers were recorded by a simple device which gave maximum depths only. The observations, however, were taken with such a clear understanding of their significance and limitations that the essential features were obtained with as great accuracy as was possible with the apparatus used. The results of many of these gaugings are shown in Tables 2 and 3.

Mr. Kuichling was the first to give a rational method of analysis of the problem, and the following conclusions are condensed from his paper:*

1.—The percentage of rainfall discharged from any given drainage area is nearly constant for heavy rains lasting equal periods of time.

2.—This percentage varies directly with the area of impervious surface.

3.—This percentage increases rapidly, and directly or uniformly with the duration of the maximum intensity of the rainfall, until a period is reached which is equal to the time required for the

* *Transactions*, Am. Soc. C. E., Vol. XX, p. 1.

concentration of the drainage waters from the entire area at the point of observation; but, if the rainfall continues at the same intensity for a longer period, this percentage will continue to increase at a much smaller rate.

4.—This percentage becomes larger when a moderate rain has immediately preceded a heavy shower on a partially permeable territory.

5.—The sewer discharge varies promptly with all fluctuations in the intensity of the rainfall.

6.—The sixth conclusion shows the unreliability and the danger of making general applications of the standard exponential formulas, and suggests the following method of solution, in which a, b, and c are empirical constants, and t is the duration of the rain, in minutes:

First. $Q = A\ C\ R.$

Second. $C = a\ t.$

Third. $R = b - c\ t.$

Fourth. $Q = A\ a\ t\ (b - c\ t).$

In the latter expression, Q and t are the only variables, and, placing the first derivative equal to 0,

$$A\ a\ (b - 2\ c\ t) = 0, \text{ and therefore:}$$

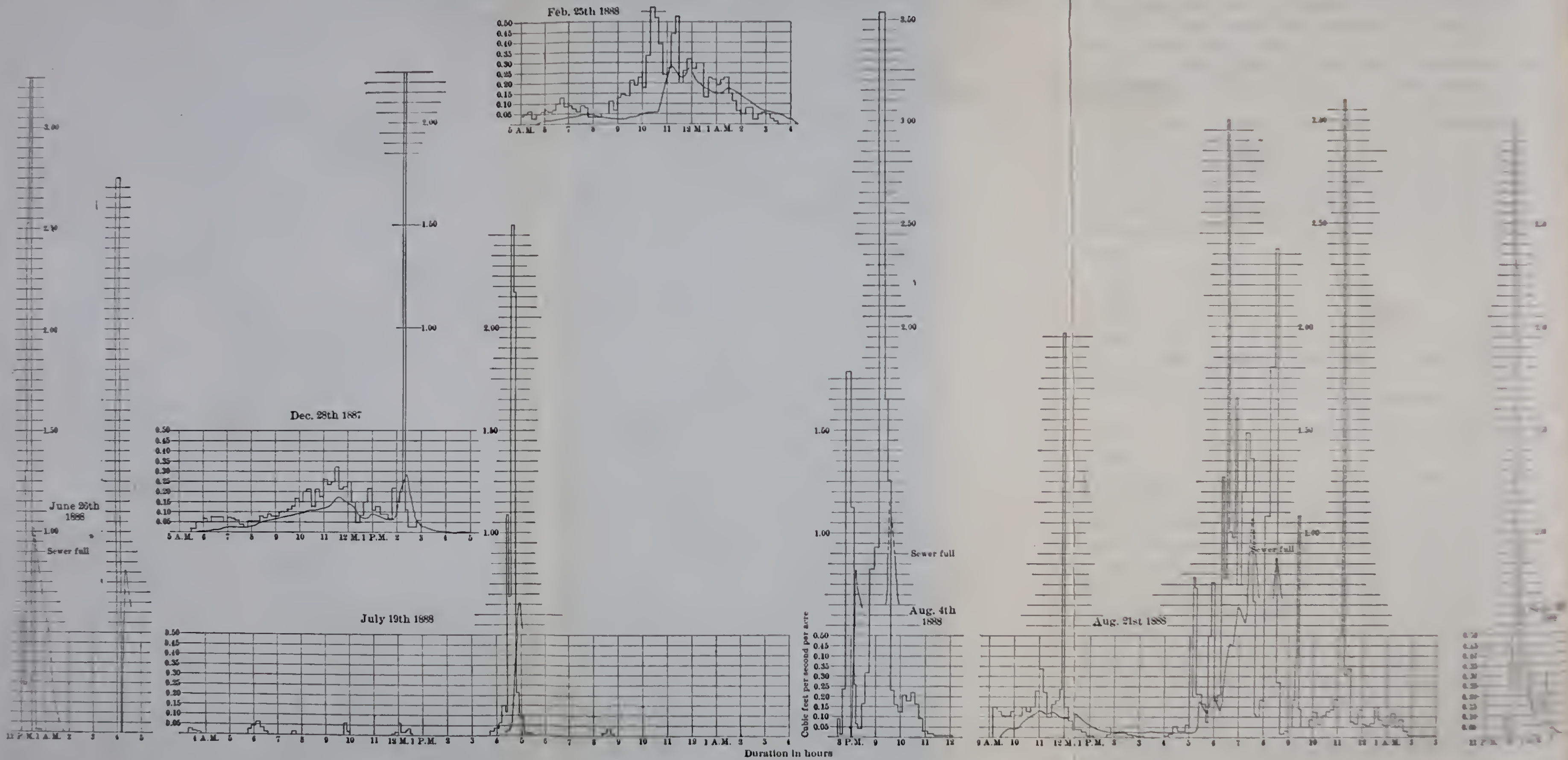
Fifth. $t = \frac{b}{2\ c}$ = the time for the absolute maximum rate of run-off.

The results of Mr. Hering's investigation, made in New York City, in 1888, have not been published. The writer believes that Mr. Hering was the first to use clock gauges for both rainfall and run-off. His observations are very briefly described as follows:

"A typical, built-up and paved section* in the lower part of New York City was selected. A self-registering gauge was erected to record the flow from an area of 221 acres. The interior of the sewer was carefully examined, and found to be regular and suitable for the safe application of a slope formula, and a value for the coefficient of roughness was determined. A survey of the water-shed was made, and its slope was found to be regular and the average grade was 7 ft. in 1 000. The average length of the sewers from the periphery of the district to the gauge was 2 886 ft., and the maximum distance about 4 800 ft. The total area of the dis-

* This section was drained by what is known as the Sixth Avenue sewer, and, roughly, is bounded on the south by First Street, on the north by Sixteenth Street, on the east by Broadway, and on the west by Eighth Avenue.

DIAGRAMS OF RAIN STORMS AND RUN-OFF IN SEWERS, SIXTH AVENUE DISTRICT, NEW YORK CITY.



trict is made up of 96.64 acres of roof area, or 43.5%, 103.32 acres of paved area, or 46.5%, and 21.76 acres of grass area, or about 10 per cent. The population was estimated at 37 700 persons. Rain gauges were placed on roofs in various parts of the district, and were read after each storm and compared with the self-registering gauge located more than 2 miles to the north in Central Park, from which the various rates of fall during each storm were taken. The results from the rain gauges were then plotted on cross-section paper, with time, in hours and minutes of the day as abscissas and the recorded intensities as ordinates. On this same sheet was plotted the rate of run-off, in cubic feet per second, as ordinates to agree with the recorded times of day as abscissas. Table 1 is a record of the complete series of observations."

The first part of Table 2 shows seven of the most severe storms of the year, with average rates. These storms, plotted as described above, are shown on Plate LXIX. The wide range and sudden changes in intensity are in this way clearly shown, together with the corresponding flow in the sewer.

A study of Table 1 and the diagrams on Plate LXIX shows that there is a wide variation in the rates of rainfall and in the percentage running off, and makes it evident that there is not much to be gained by considering the light rains, with the object of determining the maximum run-off.

Mr. Hering, after comparing his results with the McMath data, concluded by recommending diagrams plotted for a formula like McMath's in form, but with slightly changed exponents, and constants. With the usual notation, this formula is:

$$Q = C\,R\,A^{0.83}\,S^{0.27},$$

in which he recommends $CR = 1.64$ for metropolitan districts, and $CR = 1.02$ for suburban districts.

In 1900 two sewers were gauged by Mr. L. M. Hastings, City Engineer of Cambridge, Mass., and were mentioned by John R. Freeman, M. Am. Soc. C. E., in his report on the Charles River Improvement. The water-sheds of both sewers were small. The Sherman Street District contained 68 acres, with a slope of 32 ft. per 1 000, and 28% of impervious area, the remainder of the surface being nearly all grassland with steep slopes; the subsoil consisted of clay. The Shepard Street District contained 56 acres, with a slope of 28 ft. per 1 000, and 36% of impervious area. The soil was of a loose sandy nature, and the slopes were light. The most noticeable thing brought out was the great difference in the run-off from these two water-sheds, evidently due to the nature of the so-called pervious area. The maximum rate of run-off recorded was about 0.7 in. per hour for the Sherman Street, and 0.51 in. per hour for the Shepard Street water-shed, and both were caused by a sudden downpour which lasted only 8 min. While sufficient data are not given to calculate accurately the time required for the water to concentrate from these water-sheds, it was probably about 15 min. for each. The diagrams published by Mr. Freeman, while very complete for the purpose for which he used them, were drawn on too small a scale to enable one to estimate accurately the greatest average intensities for periods of 15 min. or less, as the unit of time seems to have been 1 hour, except for the storms of August 27th, when 0.6 in. fell in 8 min., and July 5th and August 10th, when 0.5 in. fell in 5 min. The storm of August 10th (Table 2) is difficult to explain when compared with that of August 27th, unless these sudden dashes of short duration were local, and very much less actually fell on the Sherman Street than on the Shepard Street water-shed, or where the rain gauge was located. The nearest automatic rain gauge was about 1 mile from the area gauged.

The next greatest run-off was caused by a long storm, of varying intensities, on May 3d, in which about 2 in. of rain fell, and the greater part of this in 2 hours. The maximum run-off from this storm (Table 2) was 0.6 in.

In justification of the design of the Walworth sewer, and the proposed formula,

$$Q = A\ C\ R \sqrt[6]{\frac{S^{\frac{2}{3}}}{A}},$$

in which C equals unity for impervious areas, W. C. Parmley, M. Am. Soc. C. E., gives the following experimental data:* "A sudden rain dash, not so severe as to cause more than passing remark, and not lasting more than 20 min., caused the 14 ft. 9-in. sewer to flow one-third full." This storm may have had an intensity of 2.5 in. per hour, and, therefore, from the 1 434 acres, may have reasonably caused a run-off equal to 640 cu. ft. per sec., about as observed, or it may not have done so. Information of this kind is of little value, and is apt to lead to erroneous conclusions.

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* *Transactions*, Am. Soc. C. E., Vol. LV, p. 412.

The results of the foregoing experiments are collected in Tables 2 and 3 for comparison and study. Table 2 contains data relating to storms of great intensity, but only of sufficient duration to allow all parts of the water-shed to contribute to the flow at the gauge. Table 3 contains data relating to longer storms, Column 8 indicating the length of the continuous rainfalls, and Column 10 giving the value of C if the pervious area is considered to have no run-off.

A short statement and summary of some of the doubtful points in each set of experiments will be made, in order to give a correct idea of their relative importance.

The writer knows of no detailed account of the experiments on which Hawksley based his formula, but the general acceptance of his conclusions must have been due to the knowledge his contemporaries had of the experiments, or to confidence in the ability of their author. Later experience has shown that he erred more in his assumptions of the rainfall than in its relation to run-off.

The Bürkli-Ziegler was an adaptation of Hawksley's formula, but with better determined coefficients.

The McMath experiments were made in a very rough general way, and were really only a compilation of casual observations. They can be used only as a rough empirical check on similar conditions.

Mr. Kuichling's Rochester experiments, however, were made in a scientific manner, but lacked the precision that automatic gauges give to such work. Although the flow in the sewers was measured by pairs of gauges, placed so as to determine the slope of the water surface, the value in each case was impaired because they did not give the time of maximum for each, and, consequently, it is not known that the two summits occurred simultaneously. All details, however, were carefully worked out, and all conditions fully stated. Where sewers ran under a head, they were given credit for only their capacity when running full, which may or may not have been correct.

Mr. Hering's gaugings of the Sixth Avenue sewer, in New York City, were made with a single clock gauge, which recorded the depth of flow in the sewer; the water surface slopes, therefore, had to be assumed. Mr. Hering states in his report that conditions were suitable for the safe application of a slope formula, and it may

TABLE 1.—COMPARISON OF RAINFALL, AND DISCHARGE FROM AREA OF WHICH

From the Report on the Sewerage System of New York City.

No.	Date.	Rain Data.							
		Total fall.	Duration. Hours.	Duration. Minutes.	Total rainfall, cubic feet per acre.	Maximum fall.	Duration, in minutes.	Rate per hour.	Cubic feet per second, per acre.
1	Dec. 15th, 1887..	0.692	9	00	2 511.96	0.031	10	0.186	0.188
2	" 28th, 1887..	1.170	10	45	4 247.10	0.149	4	2.235	2.253
3	Jan. 25th, 1888..	0.830	9	30	3 012.90	0.097	10	0.582	0.587
4	Feb. 7th, 1888..	0.250	5	40	907.50	0.021	3	0.420	0.423
5	" 25th, 1888..	1.650	10	30	5 989.50	0.094	10	0.564	0.565
6	Mar. 3d, 1888..	0.030	1	20	108.90	0.024	10	0.144	0.145
7	" 11th, 1888..	2.090	27	05	7 586.70	....	......	....	....
8	" 21st, 1888..	0.744	16	50	2 700.72	0.100	10	0.600	0.605
9	" 27th, 1888..	0.190	2	20	689.70	0.044	10	0.264	0.266
10	April 5th, 1888..	1.560	15	00	5 662.80	0.070	10	0.420	0.423
11	" 14th, 1888..	0.110	6	50	399.30	0.019	5	0.228	0.230
12	May 11th, 1888..	1.694	20	35	6 149.22	0.128	10	0.768	0.774
13	" 14th, 1888..	0.680	22	45	2 468.40	0.049	5	0.588	0.593
14	" 18th, 1888..	0.537	13	25	1 949.31	0.098	4	1.470	1.482
15	" 24th, 1888..	0.371	13	00	1 346.73	0.029	10	0.174	0.175
16	" 26th, 1888..	0.226	4	00	820.38	0.026	10	0.156	0.157
17	June 15th, 1888..	0.301	2	00	1 092.63	0.076	8	0.570	0.575
18	" 26th, 1888..	0.953 0.456	1 0	05 10	3 459.39 1 655.28	0.583	13	2.691	2.711
19	" 28th, 1888..	1.768	27	23	6 417.84	0.118	4	1.770	1.784
20	July 9th, 1888..	0.454	21	45	1 648.02	0.019	10	0.114	0.115
21	" 19th, 1888..	0.650	24	20	2 359.05	0.394	10	2.364	2.384
22	" 27th, 1888..	0.198	7	30	718.74	0.018	10	0.108	0.109
23	Aug. 4th, 1888..	1.844	4	00	6 693.72	0.590	10	3.540	3.569
24	" 12th, 1888..	0.420	3	45	1 524.60	0.079	10	0.474	0.478
25	" 21st, 1888..	3.750	20	00	13 612.05	0.396	10	2.376	2.396
26	Sept. 11th, 1888..	1.182	17	10	4 290.66	0.200	10	1.200	1.210
27	" 13th, 1888..	0.873	2	25	3 168.99	0.299	7	2.563	2.583
28	" 19th, 1888..	1.424	22	40	5 169.12	0.410	16	1.537	1.550
29	Oct. 6th, 1888..	2.035	22	10	7 387.05	0.145	15	0.580	0.584
30	" 12th, 1888..	0.204	6	30	740.52	0.024	10	0.144	6.144
31	" 16th, 1888..	0.327	25	15	1 187.01	0.090	5	1.080	1.089
32	" 19th, 1888..	0.395	20	35	1 433.85	0.088	10	0.528	0.528
33	" 27th, 1888..	0.562	18	25	2 040.06	0 048	10	0.288	0.288
34	" 28th, 1888..	0.132	2	05	479.16	0.020	10	0.120	0.120
35	Nov. 8th, 1888..	1.832	49	00	6 661.05	0.187	5	2.244	2.263
36	" 15th, 1888..	1.222	14	20	4 438.86	0.068	9	0.453	0.457
37	" 19th, 1888..	0.496	14	10	1 800.48	0.037	10	0.222	0.222
38	" 25th, 1888..	0.770	40	05	2 795.10	0.010	10	0.060	0.060
39	Dec. 9th, 1888..	0.243	9	00	882.09	0.015	10	0.090	0.090
40	" 11th, 1888..	0.302	4	33	1 096.26	0.020	10	0.120	0.120
41	" 17th, 1888..	2.290	24	05	8 312.70	0.108	5	1.296	1.307
42	" 27th, 1888..	0.495	18	22	1 470.15	0.050	4	0.750	0.756

THE SIXTH AVENUE SEWER, NEW YORK CITY, THE DRAINAGE IS 221 ACRES.

by Rudolph Hering, M. Am. Soc. C. E., May 31st, 1889.

Flow Data.					
Discharge of rain, in cubic feet per acre.	Percentage of rainfall reaching sewer.	Cubic feet per second for maximum flow. Total.	Cubic feet per second for maximum flow. Per acre.	Percentage of maximum rainfall reaching sewer.	Remarks.
1 500.0	0.597	19.732	0.089	0.473	
2 892.0	0.681	64.296	0.290	0.129	
648.0	0.212	12.638	0.057	0.097	$1\frac{1}{4}$ in. of snow.
324.0	0.357	10.864	0.049	0.116	
3 312.0	0.553	61.635	0.278	0.492	
4.2	0.039	0.665	0.003	0.021	
....	....	....	....	....	Severe snow storm (blizzard).
2 760.0	1.022	35.474	0.160	0.264	
267.6	0.388	18.624	0.084	0.314	
3 099.0	0.547	79.594	0.359	0.848	
276.0	0.691	19.067	0.086	0.374	
3 894.0	0.633	102.430	0.462	0.597	
1 419.0	0.575	60.305	0.272	0.158	Two storms.
1 272.0	0.653	121.275	0.547	0.368	
864.0	0.642	25.275	0.114	0.651	Two storms.
492.0	0.600	14.633	0.066	0.420	
450.0	0.412	41.460	0.187	0.325	
1 692.0 684.0	0.489 0.413	199.982	1.022	0.377	Very heavy thunder showers.
3 678.0	0.574	180.915	0.816	0.457	Local storm. (Note.) The fall at our gauges much greater than at Professor Draper's.
600.0	0.364	9.977	0.045	0.391	
878.5	0.372	147.659	0.663	0.279	
240.0	0.334	5.321	0.024	0.220	
5 157.0	0.770	199.982	1.162	0.326	Storm of greatest intensity during year.
540.0	0.354	22.836	0.103	0.216	
9 600.0	0.705	199.982	1.026	0.428	
981.0	0.229	138.791	0.626	0.517	
1 284.0	0.405	146.550	0.661	0.256	
....	....	157.414	0.710	0.458	Sewer record not complete. Float caught.
4 620.0	0.625	89.347	0.403	0.690	
555.6	0.750	17.293	0.078	0.542	
1 131.6	0.953	69.839	0.315	0.289	
903.6	0.630	58.753	0.265	0.502	
1 896.0	0.929	24.166	0.109	0.378	
276.0	0.576	10.420	0.047	0.392	
6 247.2	0.938	199.973	0.902	0.399	
3 680.0	0.830	76.263	0.344	0.753	
1 500.0	0.833	31.926	0.144	0.649	
2 683.2	0.960	12.415	0.056	0.983	Snow and hail $1\frac{1}{2}$ in. on 25th.
759.0	0.860	9.311	0.042	0.467	
1 008.0	0.919	16.849	0.076	0.633	
5 756.0	0.692	107.746	0.486	0.372	
1 339.2	0.911	23.057	0.104	0.138	

TABLE 2.—SHORT-TIME STORMS.

(1) Location and designation of water-shed.	(2) Number of acres.	(3) Date of storm.	(4) Percentage of impervious area.	(5) Time of flow through longest line of sewer on water-shed, in minutes.	(6) Average rate of rain, in inches per hour, for time stated in Column 5.	(7) Observed maximum run-off rate, in cubic feet per second per acre.	(8) Ratio of run-off to rainfall, C.	(9) Remarks.
New York City—6th Ave. District......	221	June 26, 1888	90	15	2.367	1.022	0.431	
	221	July 19, 1888	90	15	1.850	0.666	0.360	
	221	Aug. 4, 1888	90	15	2.910	1.162	0.399	End of storm.
	221	Aug. 21, 1888	90	15	2.180	0.880	0.404	Beginning of storm.
	221	"	90	15	1.347	0.470	0.342	End of storm.
	221	Sept. 13, 1888	90	15	1.570	0.661	0.421	
	221	Dec. 28, 1887	90	15	0.730	0.290	0.390	
					Average......		0.391	
Cambridge, Mass.—Shepard St. Dist...	56	July 25, 1900	36	5	4.8	0.33	0.070	
	56	Aug. 10, 1900	36	5	5.4	0.36	0.066	
					Average......		0.07	
Cambridge, Mass.—Sherman St. Dist..	68	July 25, 1900	28	5	3.6	0.45	0.125	From Cambridge gauge.
	68	Aug. 10, 1900	28	5	4.8	0.20	0.042*	" Harvard "
					Average......		0.125	
Rochester, N. Y.—District I..........	357	April 5, 1888	15	34	0.24	0.025	0.11	
	357	May 26, 1888	15	34	0.57	0.086	0.15	
	357	June 24, 1888	15	34	1.40	0.140	0.10	
	357	June 24, 1888	15	34	1.55	0.165	0.105	2 hours later.
	357	July 11, 1888	15	34	0.41	0.056	0.14	
	357	July 18, 1888	15	34	0.36	0.058	0.16	
	357	Aug. 4, 1888	15	34	0.4	0.046	0.115	
	357	Aug. 16, 1888	15	34	0.95	0.08	0.09	
	357	Aug. 17, 1888	15	34	0.60	0.072	0.12	
	357	Aug. 26, 1888	15	34	1.11	0.10	0.09	
					Average......		0.12	
Rochester, N. Y.—District IV.........	128.7	June 24, 1888	30	18	2.62	0.55	0.21	
	128.7	July 11, 1888	30	18	0.66	0.12	0.18	
	128.7	July 18, 1888	30	18	0.5	0.09	0.18	
	128.7	Aug. 4, 1888	30	18	0.73	0.098	0.134	
	128.7	Aug. 16, 1888	30	18	1.41	0.21	0.15	
	128.7	Aug. 26, 1888	30	18	2.02	0.31	0.15	
					Average......		0.166	
Rochester, N. Y.—District X..........	25.12	July 18, 1888	50	10	0.75	0.19	0.25	
	25.12	Aug. 17, 1888	50	10	1.33	0.25	0.19	
					Average......		0.22	
Rochester, N. Y.—District IX........	133	May 26, 1888	38	15	0.84	0.117	0.14	
	133	June 24, 1888	38	15	2.62	0.31	0.12	
	133	July 11, 1888	38	15	0.72	0.148	0.20	
	133	July 18, 1888	38	15	0.544	0.077	0.14	
	133	Aug. 4, 1888	38	15	0.8	0.137	0.17	
	133	Aug. 16, 1888	38	15	1.6	0.31	0.19	
	133	Aug. 17, 1888	38	15	0.95	0.12	0.13	
	133	Aug. 26, 1888	38	15	2.23	0.31	0.14	
					Average......		0.16	
Rochester, N. Y.—District XVII......	92.3	May 26, 1888	30	16	0.88	0.187	0.21	
	92.3	June 24, 1888	30	16	2.62	0.34	0.13	
	92.3	July 11, 1888	30	16	0.76	0.166	0.22	
	92.3	July 18, 1888	30	16	0.57	0.111	0.19	
	92.3	Aug. 4, 1888	30	16	0.83	0.15	0.18	
	92.3	Aug. 16, 1888	30	16	1.616	0.288	0.18	
	92.3	Aug. 17, 1888	30	16	0.99	0.16	0.16	
	92.3	Aug. 26, 1888	30	16	2.34	0.34	0.15	
					Average......		0.17	

* Too small: throw-out.

be only a matter of opinion whether or not this was true. The most serious objection is that the automatic gauge from which the rainfall rates were obtained was 2 miles distant, in Central Park, and these rates were checked with the rain gauges on the area, for total amounts only.

The experiments made by Mr. M. L. Hastings, seem to have only one serious fault, and that is that there were no automatic rain gauges on the areas, the rain rates being taken from two gauges: one, giving only total amounts, located between the water-sheds, and one automatic gauge about a mile away. Between these gauges there has been a difference of 50%, and, consequently, a possible error of that much in the quantity of rain falling on the areas. The flow in the sewers was measured by pairs of gauges 600 ft. apart.

A single gauging of a very severe thunderstorm in Hartford, Conn., by F. L. Ford, M. Am. Soc. C. E., is typical of the storms of great intensity in New England, and is the only one given in detail in his report. Two spring freshet storms are recorded, but as it is so evident that the run-off was influenced to a very great extent by the condition of the sewer outlet, or by other unknown local complications, they are considered of little or no value in deducing general laws. There were automatic rain gauges on three sides of the area gauged, two of them on the outskirts of the water-shed, and the other about ½ mile away, at the City Hall, and the flow in the sewer was measured by two clock gauges about 3 000 ft. apart. In this observation the greatest lack is accurate knowledge of the characteristics of the water-shed, such as the total area actually tributary, the percentage of impervious area, etc.

Since making some of the investigations reported above, a few general laws have been quite generally accepted. Their acceptance, probably, is due quite as much to their broad reasonableness as to experimental determination. They may be stated as follows:

The quantity of run-off from any catchment area is a function (1st) of its area; (2d) of the rate of rainfall; (3d) of the slope; (4th) of the percentage of impervious area; and (5th) depending on the shape of the water-shed. It has been expressed by Messrs. E. Kuichling, A. N. Talbot, and others, in the following general

Rochester, N. Y.—District I.	357	Dec. 10, 1888	15	34	0.31	60	0.043	0.11	
	357	May 9, 1888	15	34	0.93	70	0.21	0.22	
	357	June 28, 1888	15	34	0.46	105	0.14	0.30	
	357	Sept. 16, 1888	15	34	0.47	50	0.093	0.20	
Rochester, N. Y.—District IV.	128.7	May 10, 1888	30	18	0.31	60	0.072	0.23	
	128.7	June 28, 1888	30	18	0.45	105	0.23	0.51	
	128.7	Sept. 16, 1888	30	18	0.47	50	0.18	0.40	
	128.7	May 9, 1888	30	18	0.77	35	0.26	0.33	
Rochester, N. Y.—District X.	25.12	May 26, 1888	50	10	1.00	13	0.32	0.32	
	25.12	Aug. 16, 1888	50	10	1.62	15	0.40	0.25	
	25.12	Aug. 26, 1888	50	10	2.5	14	0.84	0.34	
	25.12	June 24, 1888	....	......	2.62	20	0.85	0.32	
	25.12	June 28, 1888	....	......	0.8	20	0.29	0.36	This summit may have been the result of the longer rain.
	25.12	Dec. 10, 1888	....	......	0.31	60	0.18	0.6	
	25.12	June 28, 1888	....	......	0.45	105	0.29	0.8	This summit may have been the result of the shorter rain.
Rochester, N. Y.—District IX.	133	Dec. 10, 1888	....	......	0.31	60	1.28	0.41	
	133	Jan. 28, 1888	...	......	0.45	105	0.28	0.62	
Rochester, N. Y.—District XVII.	92.3	Dec. 10, 1888	....	......	0.31	....	0.08	0.26	
	92.3	June 28, 1888	....	......	0.45	....	0.30	0.66	
Hartford, Conn.—Franklin Ave. Sewer.	280	July 11, 1900	45	17	3.4	17	0.34	0.10	0.40

TABLE 3.—LONG-TIME STORMS.

(1) Location and designation of water-shed.	(2) Number of acres.	(3) Date of storm.	(4) Percentage of impervious area.	(5) Time of flow through longest line of sewer on water-shed, in minutes.	(6) Equivalent uniform rate, for duration of rain considered.	(7) Duration of continuous rain, in minutes.	(8) Rate of run-off at end of continuous rain.	(9) C, as observed.	(10) C, if pervious area has no run-off.	(11) Remarks.
New York City.—6th Ave. District.	221	Aug. 21, 1888	90	15	1.36	15	0.47	0.35	0.39	
	221	"	90	15	1.20	30	0.65	0.54	0.60	1.40 rate for last 15 min. 1.07 rate for preceding 25 min.
	221	"	90	15	1.07	73	0.90 (reduced)	.... 0.84	.... 0.93	0.33 rate × 0.39 = 0.13. 1.03 − 0.13 = 0.90 reduced rate.
	221	Feb. 25, 1888	90	15	0.49	30	0.28	0.57	0.63	
	221	"	90	15	0.36	68	0.27	0.75	0.83	Average for last 38 min. immediately preceding summit = 0.36.
	221	Dec. 28, 1887 (short)	90	15	0.73	15	0.29	0.39	0.43	
	221	Dec. 28, 1887	90	15	0.25	60	0.18	0.72	0.80	
Cambridge, Mass.—Shepard St. District.	56	May 3, 1900	36	5	0.50	30	0.13	0.26	0.72	
	56	"	36	5	0.38	70	0.13	0.34	0.95	
	56	Sept. 18, 1900	36	5	0.4	30	0.10	0.25	0.70	
	56	"	36	5	0.54	80	0.14	0.26	0.72	
	56	"	36	5	0.48	150	0.17	0.35	0.98	Average for last 50 min. immediately preceding summit = 0.48.
Cambridge, Mass.—Sherman St. District.	68	Sept. 18, 1900	28	5	0.58	30	0.20	0.34		Averages of gauges at Harvard and Cambridge.
	68	"	28	5	0.52	60	0.32	0.62		" " " " " " "
	68	"	28	5	0.70	120	0.61	0.87		" " " " " " " for last 30 min.
	68	May 3, 1900	28	5	0.43	60	0.23	0.53		Averages of gauges at Harvard and Cambridge.
	68	"	28	5	0.38	90	0.25	0.66		" " " " " " "
	68	"	28	5	0.55	120	0.44	0.80		" " " " " " " for last 60 min.

form: $Q = A\,C\,i$, in which Q is the run-off, in cubic feet per second; C is a coefficient, supposed to be a constant, expressing the relation between run-off and rainfall on a particular water-shed, and a function of impervious area; i is the intensity or rate of rainfall, in inches per hour, lasting a sufficient time to cause all parts of the area to contribute to the flood wave at a given point at the same time, and, therefore, is a function of the slope and length of the water-shed; and A is the number of acres drained.

For comparison with each other, and with the two formulas proposed by the writer, the exponential formulas most extensively used have been reduced to the same form and notation, as follows:

Hawksley .... $Q = A\,C\,R\sqrt{\dfrac{S}{A}}.$ in which $C = 0.7$, $R = 1$ to 2

Bürkli-Ziegler ...... $Q = A\,C\,R\sqrt[4]{\dfrac{S}{A}}$ in which $C = 0.7$ to 0.9, $R = 1$ to 3

McMath .... $Q = A\,C\,R\sqrt[5]{\dfrac{S}{A}}$ in which $C = 0.1$ to 0.8, $R = 1$ to 2.75

New York diagrams.. $Q = A\,C\,R\sqrt[6]{\dfrac{S^{1.62}}{A}}$ in which $C\,R = 1.62$ to 1.05

Parmley .... $Q = A\,C\,R\sqrt[6]{\dfrac{S^{\frac{3}{2}}}{A}}$ in which $C = 0$ to 1, $R = 4$

Adams............. $Q = A\,C\,R\sqrt[6]{\dfrac{S^{\frac{1}{2}}}{A\,R}}$ in which $C = 1.837$, $R = 1$

Proposed .... $Q = 2.8\,A\,\dfrac{S^{0.186}}{A^{0.14}}$ for impervious surfaces.

Proposed .... $Q = C\,\dfrac{\;\dfrac{105}{1}\;}{84\sqrt[5]{A\,S^2} + 25}$, in which $C = 0.10$ to 0.54

The establishment of automatic rain gauges has made it possible to obtain rainfall intensities for all parts of a storm. From time to time various curves have been deduced to show the relation between rain intensities and duration. These curves are generally drawn from plottings of the rates of rain, in inches per hour, as ordinates, and the time, in minutes, as abscissas, by inclosing the points thus plotted. This curve usually takes the form of the hyperbola. The curves by A. N. Talbot, M. Am. Soc. C. E., for the

Eastern United States, Fig. 1, seem to have been most generally adopted for both ordinary and maximum rainfall.

Each locality, however, may have, and many have been shown to have, a somewhat different curve, as is shown by the diagram,

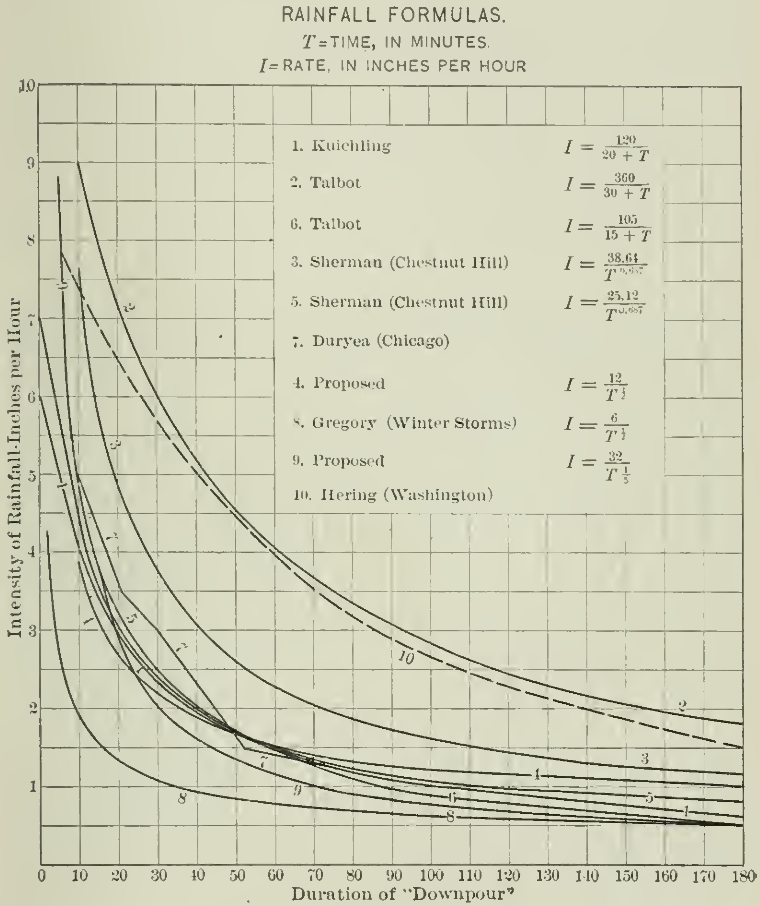

FIG. 1.

Fig. 1, on which are drawn curves deduced by competent authorities for different localities. These curves show the necessity for a solution of this problem which will give the relation between any rainfall curve and the consequent run-off, rather than an empirical formula which gives the same result for different localities, regardless of the rain curve.

Curve 9 is proposed in order to obtain results for long-time rains which will be more like Talbot's ordinary curve, as it is the writer's opinion that curves based on the records of a single gauge give results which are too great for the average over large areas, and, consequently, too great for long-time storms.

The diagram, Fig. 10, shows a curve for rainfall during the winter, and includes all but two storms which have occurred between November 1st and April 1st, which have been reported as severe at the places indicated, covering long periods of time. This curve has values which are just half of those given by the proposed curve, $i = \frac{12}{t^{\frac{1}{2}}}$, for ordinary severe rainfalls for the entire year.

Having tabulated and briefly discussed the better known data relating to this problem, and very briefly reviewed the practice for the past fifty years, an attempt will be made to analyze these data and, as far as possible, harmonize them into a rational, workable method of design, applicable to all conditions and places, and to do it so that, not only maximum conditions may be predicted, but that, given any storm of more than moderate intensity, and the factors of the water-shed, the run-off, can be estimated with a precision corresponding with the accuracy of the information given.

The almost universal failure of the inch-per-hour formulas and designs is easily explained by the now well-established fact that the rate of 1 in. per hour is quite often exceeded, and especially for water-sheds requiring but a short time for water to flow from the most remote part to the outlet, called short-time water-sheds. The other conclusions and data may be made to harmonize to as great an extent as their accuracy warrants, if rationally treated, even, perhaps, to reconciling Hawksley's 54% with W. C. Parmley's 100% run-off.

Even a very casual study of the data in Tables 2 and 3, and the methods of experiment, would suggest the need of further and more reliable experimental data, as each set of experiments has its own peculiar factors which are open to doubt, and also for the reason that there have been no long-time or very short-time water-sheds gauged.

The first part of Table 2 shows seven storms in the Sixth Avenue sewer district, in New York City. The percentage of run-off in each storm varies only 5% from the average of 39% of the average rainfall for 15 min. (the time required for the water to flow from the most remote part of the water-shed); or from 50% if this time be taken as 25 min. This, it may be noted, might explain and agree with the 54% run-off reported by Hawksley. The variation of 5% may be explained by inaccuracy in measuring the rainfall, as only one automatic gauge was used, and the average rate on the water-shed might easily vary that much, but would tend to equalize in the series. The condition of the 10% of pervious or grass area might also vary the result slightly, but the 90% of impervious area should give a constant ratio between actual rainfall and run-off, as it apparently does.

The Shepard Street district, in Cambridge, Mass., gives for two short intense storms (if judgment is exercised in determining the actual rate of rain on the water-shed) a very uniform ratio of 0.07 for 36% of impervious area. The Sherman Street district gives a ratio of 0.125 for 28% of impervious area, showing that the so-called pervious areas on each are of very different natures. This is borne out by the facts of the case, the Shepard Street district having flat slopes and sandy subsoil, while the Sherman Street district has steep slopes and a clay subsoil.

Following this there are several storms from Mr. Kuichling's Rochester gaugings, designated by districts. Here the pervious areas seem to be more uniform, and the run-off ratio follows the percentage of impervious areas more closely. The average value for C for each district is deduced as designated.

It is reasonable to assume that all impervious areas, under similar conditions, should give a constant ratio for all storms of equal duration and intensity. In practice, however, all water-sheds and all rain storms are different; therefore, all storms and all water-sheds must be reduced to a common basis of comparison. The maximum flow from any water-shed, except one of unusual shape, obviously is at the time when all parts of the area are contributing their average maximum at the point of observation. For any shorter period only part of the area is contributing, and for a longer period the intensity of the rain is less. The average maximum run-off occurs, obviously, and as is shown later, a few minutes after the time required for concentration. The exceptions to this general law are either when the shape or state of development of the water-shed are such that the increment from the farthest points is less than the corresponding decrease in intensity; and, for very short-time water-sheds, where the decrease in intensity is less than the corresponding increase in the value of C. This time required for concentration is not, as has often been stated, the total time elapsed between the beginning of the rain and the time of observed maximum run-off, but rather is the time elapsed while the water is traveling through the sewer from the most remote part, plus the time required for the water to reach the sewer from the roofs, pavements, and gutters, at the remote points, minus the time required to reach the sewer from near-by roofs, etc., as water that has not yet reached the sewer obviously cannot influence the maximum which occurs at that time. This is shown clearly on the diagrams of the Sixth Avenue sewer, where the maximum run-off occurs usually after the intense rain has ceased. This latter time will perhaps usually be about the same as the time at the most remote parts, and, consequently, where there is not known to be a difference, the time of concentration may be taken as the time that the water is in the sewer and passing from the most remote point to the gauge. Taken in this way, the time of concentration in the Sixth Avenue sewer was 15 min., and the ratios for each storm are shown in Table 2 to average 0.391. Each set of experiments is shown in the same way, and the average ratio is stated for the time of the water-shed.

Table 3 contains records of storms of long duration and of fairly uniform intensity. These storms, while not of very great intensity, caused a run-off nearly as great as many shorter storms of much greater intensity. A typical long-time severe rain, which occurred on August 21st, 1888, in the Sixth Avenue district, is shown on Plate LXIX. This storm and also those of February 25th, 1888, and December 28th, 1887, are shown by the diagram, Fig. 8, on which is plotted the time, in minutes, as abscissas and the ratios of run-off to rainfall as ordinates; the different summits of the flow in the sewer are taken and the extreme rain rates above the average are reduced by giving them the same ratio as the short-

DIAGRAMS SHOWING RELATION OF LONG-TIME UNIFORM STORMS TO PROPOSED CURVES, FOR FIVE DISTRICTS IN ROCHESTER, N.Y. AND FOR THE SHERMAN ST. DISTRICT IN CAMBRIDGE, MASS.

DISTRICT XVII—30% IMPERVIOUS.
TIME OF WATER-SHED, 16 MINUTES.

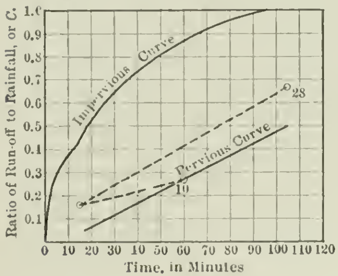

Fig. 2.

DISTRICT IX.—38% IMPERVIOUS.
TIME OF WATER-SHED, 15 MINUTES.

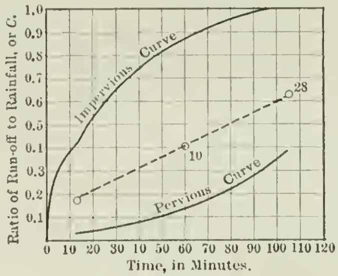

Fig. 3.

DISTRICT I—15% IMPERVIOUS.
TIME OF WATER-SHED, 34 MINUTES.

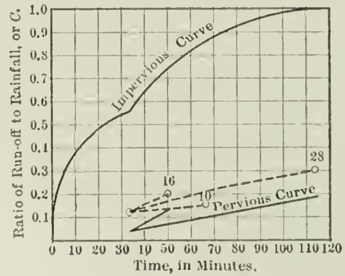

Fig. 4.

DISTRICT X—
TIME OF WATER-SHED, 10 MINUTES.

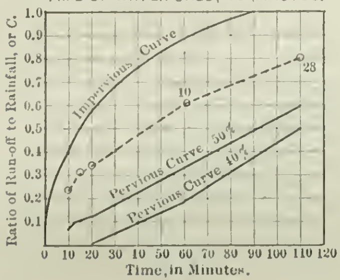

Fig. 5.

DISTRICT IV—30% IMPERVIOUS.
TIME OF WATER-SHED, 18 MINUTES.

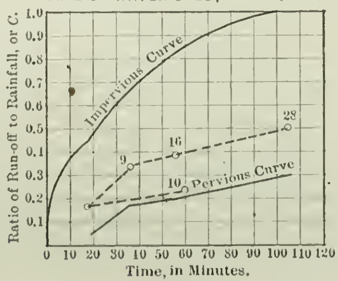

Fig. 6.

SHERMAN ST., CAMBRIDGE, MASS.
WATER-SHED 28% IMPERVIOUS.
TIME OF WATER-SHED, 5 MINUTES.

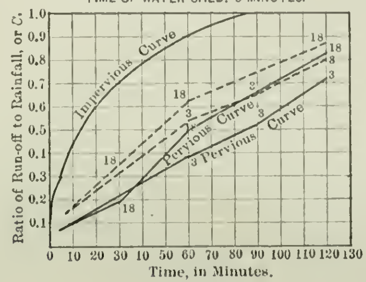

Fig. 7.

time rains and deducting the result from the total run-off before determining the ratio equivalent to a uniform rain. These ratios show a very regular increase. Each point is connected on the diagram by a broken line. If the 10% pervious areas, being small, be counted as 0, then the ratios for impervious surface will be indicated by the solid lines, as shown. In the same manner, on Fig. 8, are shown three storms in the Shepard Street district, Cambridge, Mass. Here, the pervious areas, although a much greater proportion of the total, seem to have absorbed the rain as rapidly as it fell; therefore these areas may also be made equal to 0, and this gives the points representing the ratio for impervious areas.

The severe storm at Hartford, Conn., is treated in a similar manner. The pervious areas are not as easily eliminated on the other water-sheds, and they will be analyzed by applying the apparent relations of the four storms just described. The ratio of run-off on both impervious and pervious areas for the different localities and times where the maximum occurred immediately following the intense rain, may be found by assuming that the ratio of run-off for similar times is the same on each, and selecting areas on which the pervious portions are similar in nature but different in relative extent, as follows: District IV, Rochester, N. Y., has 30% impervious area, and the Sixth Avenue district, New York, has 90% impervious area; let x = ratio for impervious area and y = ratio for pervious area:

then $$0.90\,x + 0.10\,y = 0.39$$

and $$0.30\,x + 0.70\,y = 0.166$$

eliminating y,

$$x = 0.42.$$

In this way it will be found that, wherever the pervious areas are similar, there is a remarkable uniformity in the values of C for impervious areas. These values, and the proposed curves for the long-time storms just mentioned, both point to the fact that the ratio, C, is not a function of the impervious area alone, but has a wide range of values between 0 and 1.00 for totally impervious areas. They also show that the variation is not uniform, and that it is different for storms having a greater duration than the time required for concentration, although the actual time may be the same; that the ratio increases very rapidly during the first minutes of a storm and then more gradually. The ratio for the time for concentration is necessarily less than the ratio for a greater time, because at this time part of the area is contributing the rain from the first minutes of the storm with a consequent small ratio. This relation was first roughly shown by the straight-line curves in Mr. Kuichling's paper.

This change in the ratio as the rain continues is due to many causes, among which is capillary action for small depths, and the inertia of the water. It is readily observed that, during very intense precipitations on surfaces slightly inclined, considerable depth is attained before the run-off equals the rainfall. This same phenomenon is true throughout, to a greater or less extent. In order to approximate the curve of the changing ratio for impervious area, it must be remembered that (for a unit of area) each actual observed ratio is an average of all ratios during the time required for concentration on the water-shed. Not yet knowing the law of change, assume that the rate of change is uniform for the time of the water-shed; then these average ratios will be points on the curve at a time that is less by one-half of the time for concentration than the time of the observed ratio. For illustration, take the last summit for the storm of August 21st, 1888, on the Sixth Avenue territory; the ratio for 73 min. is 0.84, and if the 10% pervious area is considered as equal to 0, the impervious area has a ratio of 0.93; this 0.93 is not plotted for 73 min. directly above the 0.84, but at 65½ min., which is 7½ min. (or one-half of the 15 min.) less than 73 min. Each point is treated in this way, and some of the averages for the time of the water-shed from Table 2 are also plotted in a similar manner.

After considerable study, the cubic parabola, $C = 0.233\, t^{\frac{1}{3}}$, was found to enclose these points and therefore express the rate of change of the value of C for an elementary area. The ordinates of this curve give the ratios for each succeeding increment of t, and therefore the average ordinate for any time, t, is the area between the axis and the curve, $\frac{3}{4}$ $(t\, C)$ divided by t, or $\frac{3}{4}$ of $C = \frac{3}{4}\,(0.233\, t^{\frac{1}{3}}) = 0.175\, t^{\frac{1}{3}}$. These curves are shown on the diagram, Fig. 8. The value of C, in the curve, $C = 0.233\, t^{\frac{1}{3}}$, becomes unity when $t = 80$, or, in other words, it is proposed that where rain has fallen for

80 min. continuously on a unit of impervious area, the run-off has become equal to the rain, and, therefore, for all areas of greater time than 80 min., the average run-off must be expressed as follows:

$$\frac{(80 \times 0.75) + (t - 80)\,1}{t} = \frac{t - 20}{t},$$

a curve with an asymptote to C = unity, or, in other words, the value of C can never quite reach unity for a rain lasting only the time of the water-shed, however long the time.

PROPOSED CURVE SHOWING RATIO OF RAINFALL TO RUN-OFF FOR A UNIT AREA.

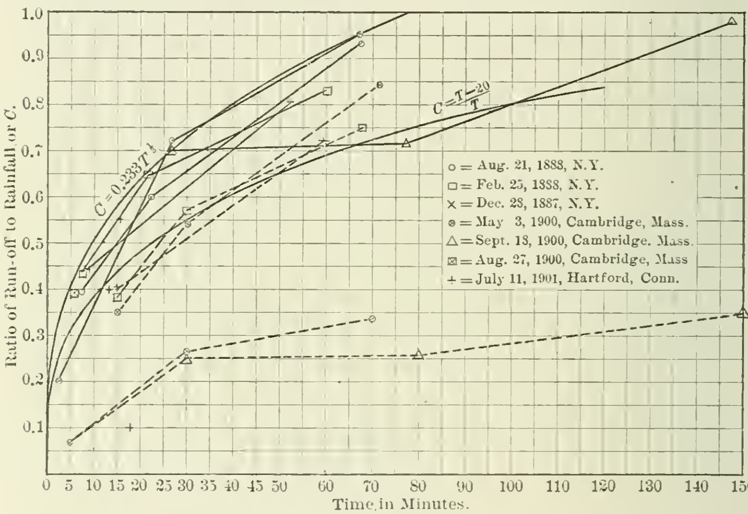

Fig. 8.

To show the general application of this method of analysis, all the storms in Table 3 not used to deduce the relation may be tested as follows:

On the diagram, Fig. 7, are plotted the storms on the Sherman Street district, Cambridge, the first point being the average of the storms for the time of the water-shed, and the other points being summits for the times indicated, those of September 18th being designated by 18, and those of May 3d by 3. These points are joined by broken lines. Then, from the curve for impervious areas, Fig. 8, values for C are taken, as follows: For the time of the water-shed, say 5 min., the value of C for the $\frac{3}{4}$-curve is 0.2. This is plotted as the first point in the impervious curve. Next take, say, 10 min., then the rain has continued on the most remote part for 5 min., and on the nearest for 10 min., and the average ordinate or value for C is $0.175 \frac{(10^{\frac{4}{3}} - 5^{\frac{4}{3}})}{5} = 0.47$. Continue in this manner until a sufficient number of points on the curve are located. These values for impervious areas are then applied to the 28% of impervious area on this water-shed, and the result is deducted from the observed ratio, the remainder being the run-off from the so-called pervious area; divide this by the percentage of pervious area, 0.62, and it becomes the ratio for this particular pervious area and duration. These points are joined to show the increase in the ratio as the ground becomes saturated. The diagrams, Figs. 2, 3, 4, 5 and 6, show storms of similar character on sewerage districts in Rochester, N. Y., from Mr. Kuichling's gaugings.

These pervious curves show that, under favorable circumstances, where grass areas have steep slopes and clay subsoil, rains having a duration of 90 min. may cause a rate of run-off nearly 90% as great as the rate of rain for that period; and, also, that, under conditions favorable for the absorption of the rain, the run-off may be practically zero.

The pervious curve from each diagram is then plotted on the diagram, Fig. 9, and two curves are drawn, one maximum to enclose all, and another ordinary to cover all the Rochester storms. The purpose of this diagram is to aid the judgment in estimating the run-off from pervious or grass areas, during the summer. The values lie probably anywhere between the maximum curve and zero.

Having now determined the law of change for C, the rational method of solution for any given case is as follows:

Calling cubic feet per second per acre equivalent to inches per hour, the run-off, in cubic feet per second, must equal the area in acres multiplied by C, determined from the diagram, Fig. 8, for the time of the water-shed, t, or the time of the duration of the rain, T, multiplied by the rate of rain for the time, or $Q = A\ C\ i_1$ in which, for impervious areas, $C = 0.175\ t^{\frac{1}{3}}$ for a storm of duration equal to the time for concentration, or, if for greater duration,

$$C = \frac{0.175}{t}\left(T^{\frac{4}{3}} - (T - t)^{\frac{4}{3}}\right).$$

and i, the intensity from any rainfall curve, as, say, $\frac{120}{T + 20}$. The simplest case is where T is equal to the time for concentration, t. Substitute for C and i their values, and

$$\therefore \quad Q = A\, 0.175\, t^{\frac{1}{3}} \times \frac{120}{t + 20},$$

then, for the maximum rate of run-off,

$$\frac{Q}{A} = \frac{19.8\, t^{\frac{1}{3}}}{t + 20}.$$

Place the first derivative equal to 0, and $t = 10$, the time for the maximum rate of run-off for this particular rainfall curve.

If, however, $i = \frac{12}{t^{\frac{1}{2}}}$, or any exponential formula of similar form, then

$$Q = \frac{A \quad 2.19\, t^{\frac{1}{3}}}{t^{\frac{1}{2}}} = \frac{2.19\, A}{t^{\frac{1}{6}}},$$

a function which always decreases as t increases, and there are no maximum or minimum points, except at the limits.

The solution for a maximum run-off for any rain curve is as follows: It is evident from Fig. 8 that the ratio, C, increases very rapidly during the first minutes of a storm, and probably more rapidly than the rate of rain decreases, as was shown by the maximum run-off rate at 10 min. Therefore, it is easily shown that the greatest run-off will occur at some time between 10 min. and 0 after the time of flow through the longest line of sewer on the water-shed, this value being 10 when $t = 1$, and 0 when $t = 0$. Fig. 11 shows the curve of the rate of run-off if $i = \frac{120}{t + 20}$ for water-sheds on which the time of flow through the longest line of sewer is 5 min., 30 min., and 70 min. From these and similar curves the locus of each maximum point for each time is plotted, with the times required for concentration as abscissas, and the maximum rates of run-off as ordinates. This curve is shown on Fig. 11 by a concave line. From this curve, the time being known, the maximum rate of run-off from any water-shed may be read off directly. Beyond $t = 80$, the time of concentration of the water-shed may be taken as a maximum without serious error. Having determined the time for the maximum run-off, the values of C and i are taken from

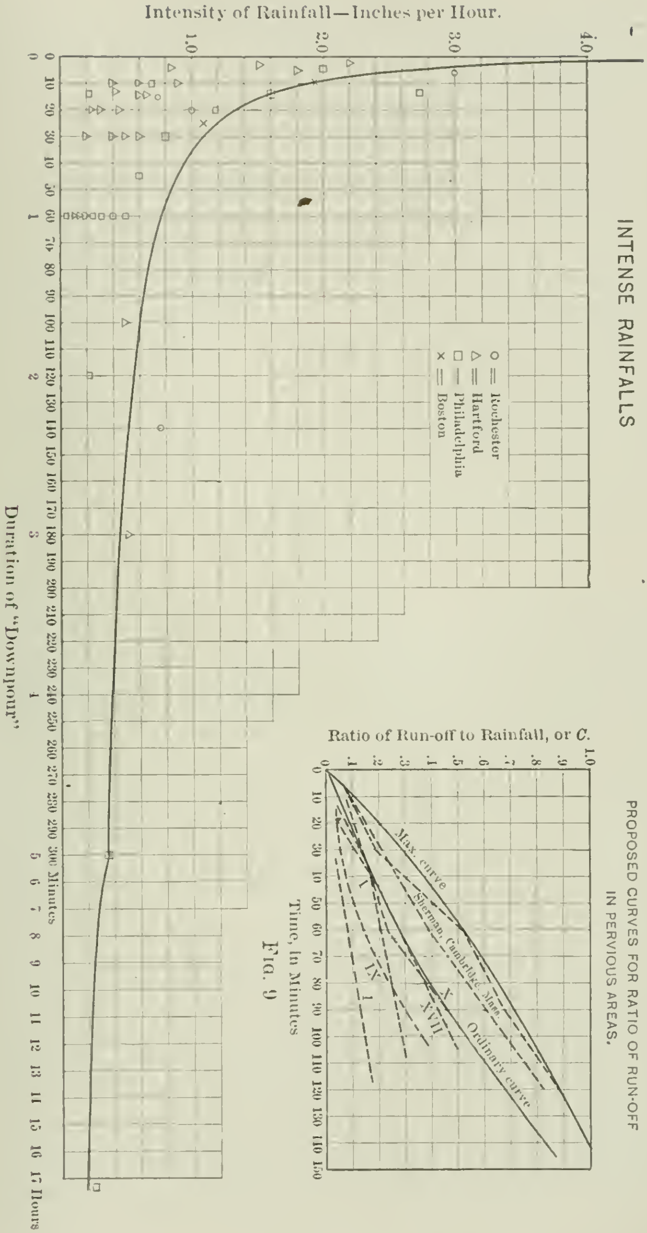
INTENSE RAINFALLS
Intensity of Rainfall—Inches per Hour.
○ = Rochester
△ = Hartford
□ = Philadelphia
× = Boston
Duration of "Downpour"
Minutes
Hours

Fig. 10

PROPOSED CURVES FOR RATIO OF RUN-OFF IN PERVIOUS AREAS.
Ratio of Run-off to Rainfall, or C.
Max. curve
Sherman, Cambridge, Mass.
Ordinary curve
Time, in Minutes

Fig. 9

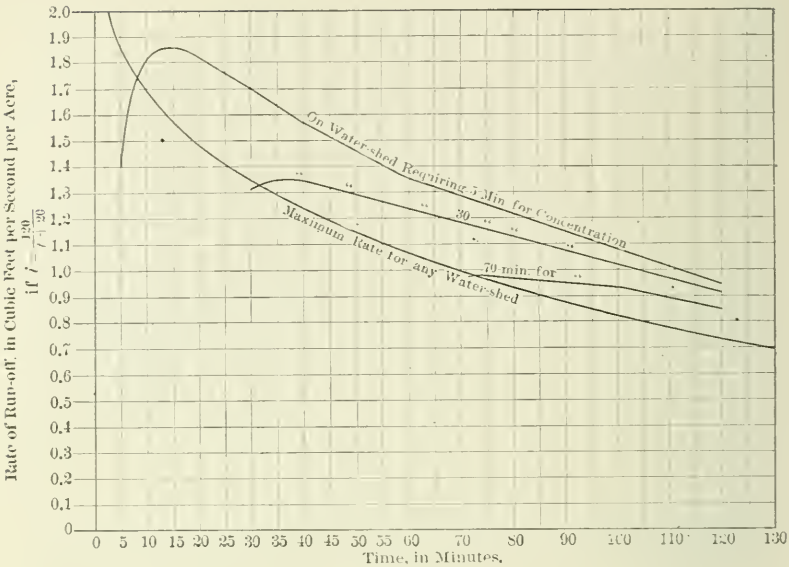

FIG. 11.

their respective curves, or from the curve on Fig. 11, and the result is obtained for the impervious area. The run-off from the pervious areas must be estimated separately, as outlined above.

Any special case, the rain rate being given, would be solved by taking the value of C from the curves in one, or all, of the foregoing ways, and applying it to the rainfall for the rate of run-off.

If an exponential curve that gives smaller values for long-time storms, like Curve 9 on Fig. 1, $i = \frac{32}{t^{\frac{4}{5}}}$, be used to show the relation between the maximum rainfall and run-off, a curve somewhat similar to the McMath form may be deduced from the foregoing laws. This curve shows the relation of the McMath curve to the foregoing deductions better perhaps than can be done by substituting rational values for the time and the rainfall in the original formula, as was done by the writer in the discussion of the paper* by W. C. Parmley, M. Am. Soc. C. E., on the Walworth sewer.

If $Q = A\ C\ i$, for C substitute its value for the time of the water-

* *Transactions*, Am. Soc. C. E., Vol. LV, p. 341.

shed, $C = 0.175\, t^{\frac{1}{3}}$ inasmuch as Curve 9 gives values too great for short periods, this excess will be offset by taking the value of C at the time for concentration, instead of the time of maximum run-off, which occurs a few minutes later if the time for concentration is short. For a rainfall in which $i = \dfrac{32}{t^{\frac{4}{5}}}$, in which $t = \dfrac{l}{V}$, V being the average velocity in feet per minute, and l the greatest length of sewer, in feet, in a rectangular district twice as long as wide, is equal to $369 \sqrt{A}$.

For an average value of V substitute in the standard formula for flow, $v = c\sqrt{r\, s}$, express s in terms of S; give c an average value of 80; and, as the relation between Q and the diameter is expressed by the equation of a parabola, the average hydraulic mean radius is two-thirds of the greatest hydraulic radius, and $60\, r = V$. By substitution, these values reduce to $V = 84 \sqrt[5]{A\, S^2}$, and, by further substitution,

$$Q = 2.8\, A^{0.86}\, S^{0.187},$$

or

$$Q = 2.8\, A \frac{S^{0.187}}{A^{0.14}}$$

for totally impervious areas. This gives a curve which approximates the McMath curve in form, but in which the values are somewhat greater, probably due to an irrational method of modification of the McMath formula to apply to perfectly impervious surfaces, the experiments having been made on areas with considerable pervious surface. It agrees more closely with the average values of the New York diagrams and Mr. Parmley's curve, and shows that the foregoing formula can be made to fit roughly the experimental data on which they are based, as well as explain the results of any individual storm. The great difficulty with formulas like the preceding, as has been demonstrated, is to modify the coefficients to fit all conditions rationally. The assumed values and significance of C and R being only the roughest of approximations, the factors affecting their values are quite generally misunderstood, especially in the Hawksley, Bürkli-Ziegler, McMath, and Parmley forms. If C is made to represent only the percentage of impervious area, and is considered constant for all times, then it does not represent the ratio between rainfall and run-off, but is simply a coefficient of area, and whatever variable factors this ratio contains are thrown into the so-called rainfall coefficient, R, which has but little relation to the actual rainfall, and the modification of which modifies other factors than the rainfall.

In order to modify the formula deduced above for different amounts of impervious area, the A, in the original form, $Q = A\ C\ i$, must be made equal to the amount of impervious area, but the A, the equivalent for l, and in the denominator, should be given the full value of the total area. Any other direct modification is misleading, and would apply only roughly in special cases, where the percentage of impervious area is large. All formulas of this kind are solutions of special cases, and, consequently, are limited in their application. The formula,

$$Q = \frac{C\ \frac{105}{l}}{84\ \sqrt[5]{A\ S^2}} + 25$$

suggested by the writer in a previous discussion, is more rational and general, but does not provide for the variable, C, and, therefore, is inaccurate for very large and for very small water-sheds.

The only safe method of designing, if anything more than a rough approximation is desired, therefore, is to analyze each case by itself, applying the principles and relations as deduced above, and being guided by the general form, $Q = A\ C\ i$.

The results of each method or formula, for the areas and slopes indicated, are compared in Table 4.

In order to make the comparison of the formulas on the merits of the general form, rather than on the special value of any coefficient, and also to serve as a guide in the selection of proper coefficients, the letter, C, in each case, is taken to represent the percentage of equivalent impervious area, and the letter, R, to include all other constant factors of rainfall and run-off; but, C' indicates the relation or ratio of the rainfall to the run-off. The value of R is determined as follows: C is given a value of 0.5, $A = 100$, and $S = 10$; the value of Q is obtained by working out Formula 9 for $A = 100$ and $S = 10$, and is 90. These values are substituted in each formula, and the equation is solved for R. The values thus obtained are given in Column 3. With this value for R, and 0.5 for C, the results in Columns 4 to 7 are obtained.

TABLE 4.—COMPARISON OF RESULTS OF RUN-OFF FORMULAS, ETC., FOR THE AREAS AND SLOPES INDICATED.

(1)	(2)	(3)	(4)		(5)		(6)		(7)		(8)
No.	Formula.	Values of R. If $C = 0.5$, $Q = 90$, $A = 100$, $S = 10$	10 ACRES.		100 ACRES.		1 000 ACRES.		10 000 ACRES.		Maximum value of $C\,R$ ordinarily used.
			$S = 1$, $t = 9.0$	$S = 10$, $t = 3.5$	$S = 1$, $t = 17$	$S = 10$, $t = 7$	$S = 1$, $t = 35$	$S = 10$, $t = 14$	$S = 1$, $t = 69$	$S = 10$, $t = 27$	
1	$Q = A\,C\,R\sqrt[4]{\frac{S}{A}}$	3.2	9	16	50	90	285	507	1 600	2 850	1.4
2	$Q = A\,C\,R\sqrt[4]{\frac{S}{A}}$	3.2	9	16	50	90	285	507	1 600	2 850	2.7
3	$Q = A\,C\,R\sqrt[5]{\frac{S}{A}}$	2.85	9	14	57	90	357	572	2 258	3 570	2.2
4	$Q = A\,C\,R\sqrt[6]{\frac{S^{1.62}}{A}}$	2.08	7	13.7	48	90	330	614	2 236	4 170	1.62
5	$Q = A\,C\,R\sqrt[6]{\frac{S^{1.5}}{A}}$	*1.00 2.18	13.6 7.4	21 13	92 59	165 90	634 342	1 120 613	1 300 2 346	7 660 4 175	.... 4.0
6	$Q = A\,C\,R\sqrt[6]{\frac{S^{\frac{1}{2}}}{A\,R}}$	4.04	11	13.2	74	90	505	612	3 450	4 180	1.837
7	$Q = A\,C\,R\,\frac{S^{0.187}}{A^{0.14}}$	*2.8 2.23	10 8.0	16.0 12.4	73 59	113 90	533 424	819 650	3 856 3 071	5 931 4 723	.... 2.8
8	$Q = A\,C'\,\frac{105}{\frac{t}{81\sqrt[5]{A\,S^2}} + 25}$	....	10.5	12.0	83	111	595	916	3 798	6 865	....
9	$Q = A\,C'\,i$, and $i = \frac{120}{t + 20}$	....	8.6	9.5	78	90	640	800	5 000	6 960	....
10	$Q = A\,C'\,i$, and $i = \frac{32}{t^{\frac{1}{5}}}$	....	10	16.0	73	113	533	819	3 856	5 931	....

* Value given by author of formula.

In each case an ideal water-shed of rectangular shape, and twice as long as wide, is assumed.

Table 4 does not show how much difference in the rate of run-off would be made by a change in shape, or in resistance to flow. For example, in Column 5, if the 100 acres with $S = 10$ had been of such shape that its length was $2\frac{1}{2}$ times as great as for the area considered, the run-off given by Formulas 8, 9 and 10 would have been changed to the figures given by the flat slope, $S = 1$, while the results from Formulas 1 to 7 would remain unchanged. Formulas 9 and 10 show the relative value of the two rainfall curves used.

Column 8 gives the maximum values of $C\ R$ in ordinary use, but, except in Formulas 4 and 7, they are not intended to provide for perfectly impervious surfaces.

The writer desires to express his thanks and appreciation for data, aid, and suggestions, kindly furnished by Messrs. E. Kuichling, Rudolph Hering, F. L. Ford and M. L. Hastings.

DISCUSSION.

William C. Hoad, Assoc. M. Am. Soc. C. E. (by letter.)— The fact that the run-off per unit of area from small areas is greater than that from large areas is clearly recognized, and is conceded to be due to the higher maximum rates of rainfall obtaining for the short period of time necessary for the concentration of the flood waters from all parts of the small area than for the longer period required for concentration in the case of the larger area. This is taken account of, in many of the formulas, by introducing some power of the area, A, or of both the area and the general slope, S, into the expression for the run-off, thus assuming that the time of concentration is a function of the area. Even when the rational formula, $Q = ACI$, is used, the time of concentration, for which the intensity, I, is taken, is oftentimes determined by a subordinate formula involving the area and the slope. The writer has in the past spent considerable time in attempting to obtain a satisfactory expression for the time, T, in terms of some function of the area, A, and its general slope, S, with the idea in view of incorporating such an expression with a rainfall formula and a variable run-off factor, but has finally given it up. He had come to believe, moreover, that this cannot be done with as close an approach to accuracy as is demanded by the importance of the results involved. Mr. Hoad.

The time of concentration of any given area is a function of the distance through which the water falling upon the extreme limits must pass through in order to reach the point in question, and of its velocity of flow in passing through this distance. This distance is not a function of the area, for areas of the same size but of different shapes give different distances, as may be seen by Fig. 12 and Table 5. Here, with the assumed directions of flow, indicated by the arrows, the ratios between the distances and the square root of the area are seen to vary from 296 to 482. Moreover, it is a matter of common observation that, on account of local topographical conditions, the routes taken by sewer lines are often more or less circuitous, and this constitutes another important source of irregularity with which the area has nothing to do. In the sewer map and tables accompanying the paper by W. C. Parmley, M. Am. Soc. C. E., on the Walworth Sewer at Cleveland,* the twenty-three sub-areas tributary to this sewer are shown, and the number of acres, and, also, "the number of feet from the remotest lateral to the point of discharge into Walworth Sewer" are given for each area. The ratios of these distances to the square root of

* *Transactions*. Am. Soc. C. E., Vol. LV, p. 341.

Mr. Hoad. the number of acres in their respective areas vary from about 100 to more than 600. Of the twenty-three areas, five give ratios of less than 250, and eight of more than 450. The mean of the largest 25% of the ratios is 2.8 times the mean of the smallest 25 per cent. The figures given neglect the relatively small distance that the storm-water must run over the surface of the ground before reaching the street inlet; but, since the variation in the ratios does not coincide with the variation in the size of the areas, this omission should not vitiate the above comparison.

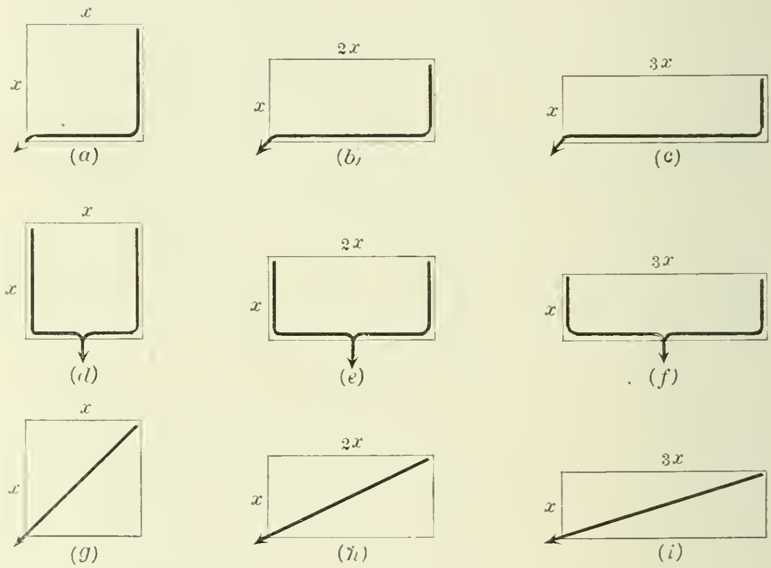

FIG. 12.

TABLE 5.—RELATION BETWEEN AREA AND DISTANCE, FOR DRAINAGE AREAS OF DIFFERENT SHAPES, AND FOR DIFFERENT GENERAL ROUTES FOR SEWERS, AS INDICATED IN FIG. 12.

(a) $D = 417 \sqrt{A}$	(b) $D = 443 \sqrt{A}$	(c) $D = 482 \sqrt{A}$
(d) $D = 313 \sqrt{A}$	(e) $D = 296 \sqrt{A}$	(f) $D = 302 \sqrt{A}$
(g) $D = 296 \sqrt{A}$	(h) $D = 330 \sqrt{A}$	(i) $D = 381 \sqrt{A}$

Neither is the velocity of flow—the other factor of the time of concentration—entirely dependent upon the general slope of the surface, or of the sewer, or even upon a combination of the slope and the area. It is, however, chiefly determined by the slope and Mr. Hoad. the quantity of water flowing, but the area is a very inaccurate measure of this latter because of the widely different degrees of imperviousness possessed by different areas. Taken on the whole, the slope, S, and the area, A, are at best only uncertain guides for the determination of the proper rainfall rate, R, to use.

The writer is strongly of the opinion that, ordinarily, it is preferable to determine the time of concentration directly in each particular area; that is, to scale the distance from a map, estimate it carefully, or measure it out on the ground, and to compute the time from this and the calculated velocity of flow, the velocity being determined with sufficient exactness from the known slopes and a preliminary estimate of the quantity of water. A more or less arbitrary addition of a few minutes should then be made to this, to allow for the time required for the water to reach the sewer. With the time, T, determined, the intensity of rainfall, I, proper for this area, should be taken from a curve or formula, the run-off factor should then be decided, and the quantity, Q, computed.

This method, of course, has been used to a considerable extent. Its chief advantages are its flexibility and its rationality. It may be used with essential accuracy on an area of any reasonable shape, slope and size, and under any conditions of rainfall, provided that these conditions of rainfall and topography are known; if they are not known, and have to be guessed at, is at least as accurate as any of the other formulas. The advantage of a rational over an empirical method is obvious.

The leading disadvantage that has been urged against this method is that its application is tedious and involves preliminary estimates of distance and time. This can hardly be considered a serious objection, however, as the small amount of extra office work is quite insignificant when compared with the importance of an accurate determination. Moreover, the thorough knowledge of the local conditions pertaining to each area, which is a requisite to the proper use of the method, is certainly conducive to more intelligent design. Probably a more frequent, though less recognized, obstacle is the fact that the use of the method requires a knowledge of the local rainfall curve. In case very little is known of the local rainfall rates, other than those for a 1-hour period or longer, it is much easier to shift the whole responsibility to some formula that does concern itself with variations in the rainfall rate.

The writer has developed for himself a method of using the rational formula which avoids some of the disadvantages mentioned, and which may be of some benefit to others.

Mr. Hoad. The general formula, as usually stated, is:

$$Q = ACI \quad \ldots\ldots (1)$$

in which

Q = the run-off, in cubic feet per second;
A = the area, in acres;
C = the ratio of rainfall to run-off; depending largely upon the imperviousness, but modified to some extent by the length of time required for concentration;
I = the rainfall intensity, in inches per hour, for the time of concentration, T.

The writer, in common with many others, has collected and studied numerous rainfall data, particularly those for the Middle West. For those storms for which it is ordinarily desirable to provide, in designing storm sewers, he has expressed the relation between the intensity and the duration of the downpour by the equation:

$$I = \frac{160}{20 + T} \quad \ldots\ldots (2)$$

which is of the form used by Professor Talbot, Mr. Kuichling, and others, but with the numerical constants changed to suit local meteorological conditions, and also to exclude very high rates of rainfall of infrequent occurrence.

The relation between the intensity of rainfall and the duration of the downpour—in other words, the rainfall curve—varies in different sections of the country; moreover, even in the same section, different engineers will place different valuations upon the importance of providing for the less frequent rates. It may, indeed, be entirely appropriate in a single city to design one sewer to provide adequately for the heaviest storm of a generation, and at the same time design others with such capacity that they may be expected to be surcharged every year or two. These variations in the rainfall curve may be expressed with essential accuracy by a coefficient. Owing to the manner in which precipitation records have been reported, data regarding the maximum rate lasting for 1 hour, in different localities, are more commonly available than those for any other period, and probably most engineers have a much more accurate idea of the ordinary maximum hourly rate for their own localities and of the frequency of its occurrence than of the rates for shorter periods of time. It is believed, therefore, that a coefficient based on the rate of rainfall lasting for 1 hour may be easily and accurately applied. The rainfall curve given in Equation 2 may then be written

$$I = R\,\frac{80}{20 + T} \quad \ldots\ldots (3)$$

Mr. Hoad.

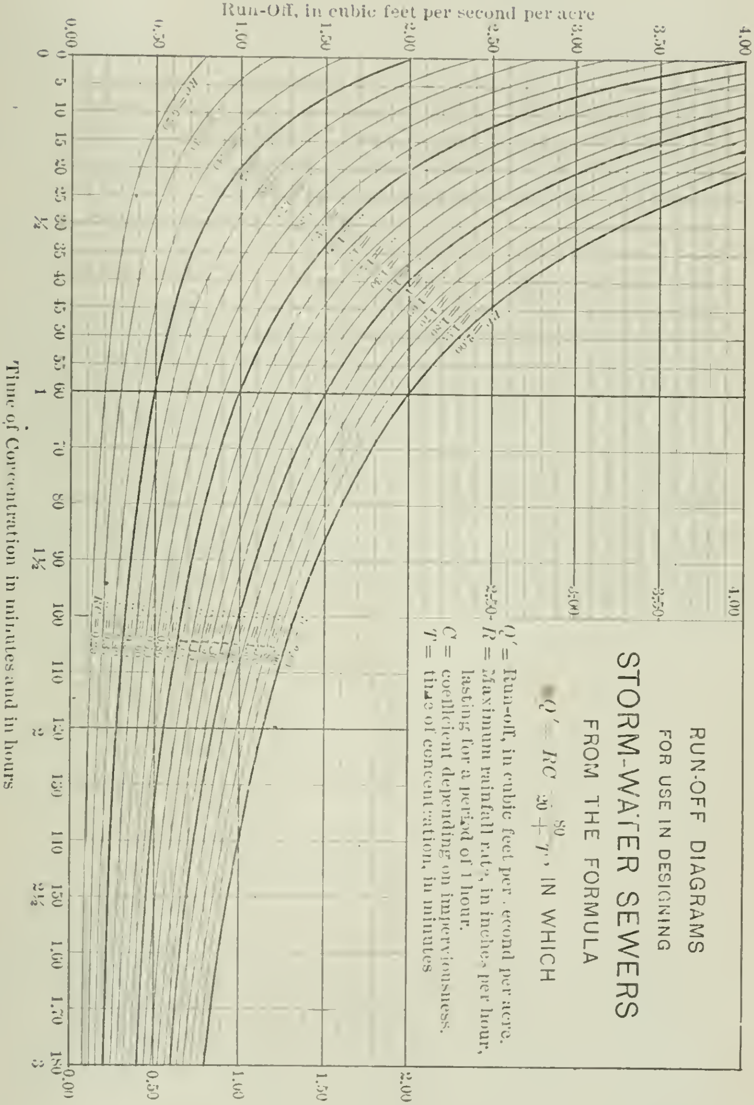

Fig. 13.

Mr. Hoad. in which

R = the maximum rate of rainfall which it is deemed expedient to consider, and lasting for the period of 1 hour.

The value of R, in the writer's curve, is 2. If it be taken as 1.5, the curve will be identical with that of Mr. Kuichling, shown in Fig. 1.

Calling Q' the run-off, in cubic feet per second per acre, and substituting $R \frac{80}{20 + T}$ for I, Equation 1 may be written

$$Q' = C R \frac{80}{20 + T} \ldots\ldots\ldots\ldots\ldots\ldots (4)$$

which is the writer's run-off formula.

In this equation, R will probably vary for different localities and conditions from as low as 1.0 to 2.5, or perhaps more, and C may range all the way from 0.10 to nearly if not quite 1.00; this will give RC a range of from perhaps 0.20 to 2.00, as outside values. In Fig. 13 curves are drawn for this equation giving the run-off, in cubic feet per second per acre, for varying periods of time, and for values of RC varying by tenths from 0.20 to 2.00. The use of diagrams of this kind is to be recommended, as it not only saves time, but, which is of much greater importance, also enables the user to see something of the significance of the assumptions which he perforce must make in any calculation of run-off.

TABLE 6.—VELOCITIES, IN FEET PER MINUTE, IN SEWERS FLOWING FULL. FROM KUTTER'S FORMULA, WITH $n = 0.015$.

S	Diameter of Sewer, in Feet.									
	1.5	2	2.5	3	3.5	4	5	6	8	10
	Diameter of Sewer, in Inches.									
	18	24	30	36	42	48	60	72	96	120
1	90	110	130	150	170	190	220	250	300	350
2	130	160	190	220	240	270	310	350	430	490
4	190	230	270	310	350	380	440	500	600	700
6	230	280	330	380	430	460	540	610	740	860
8	260	330	380	440	490	540	620	710	860	990
10	300	370	430	490	540	600	700	790	960	1 100
15	360	450	520	600	660	730	850	970	1 160	....
20	420	520	600	690	770	850	990	1 120	....	....
25	470	580	680	780	860	940	1 110	....	....	....
30	510	630	740	850	950	1 040	....	....	....	....
35	550	680	800	920	1 020	1 120	....	....	....	....
40	590	720	850	980	1 090	....	....	....	....	....

The derivation of the time, T, may be somewhat tedious, but the determination of this factor with essential accuracy is believed Mr. Hood. to be of sufficient importance to warrant the expenditure of the small amount of time necessary for it. The method already used by Mr. Parmley and others, and suggested by Mr. Gregory, of computing the actual time of flow in the tributary sewers, the length, grade and (approximate) quantity of water being previously known, and of adding to this an allowance of a few minutes for the time required for the water to reach the inlet, is certainly to be recommended. Even if these data are not available, as in the case of areas only partially sewered, they may be estimated with as much accuracy as the "average" slope, S, which is somewhat indefinite.

As a help in determining the time, T, Table 6 is given. This shows the velocities, in feet per minute, when "flowing full" in circular sewers from 18 in. to 10 ft. in diameter, on grades of from 1 to 40 per 1 000. They are adapted from a table computed from Kutter's formula, in which n has a value of 0.015.

The manner of using the diagrams, then, is first to determine as nearly as practicable the time of concentration, T; then, from the diagram, take out the run-off, in cubic feet per second per acre, corresponding to this time and to the value of RC previously decided upon; then multiply this by the area in acres.

For the purpose of a comparison with the formulas given in Mr. Gregory's Table 4, the writer has computed Table 7, which

TABLE 7.—RESULTS OF USE OF RUN-OFF FORMULA,

$$Q' = C\,R\frac{80}{20 + T}$$

FOR THE AREAS AND SLOPES INDICATED.

	10 ACRES.		100 ACRES.		1 000 ACRES.		10 000 ACRES.	
	$S = 1$	$S = 10$	$S = 1$	$S = 10$	$S = 1$	$S = 10$	$S = 1$	$S = 10$
Average velocity of water, in feet per minute	120	300	160	420	220	580	300	780
T, after adding 8 min.	20	....	36	....	72	....	156	....
T, after adding 5 min.	....	10	....	16	....	29	....	62
Q, when $R = 0.80$, and $C = 0.50$	8	11	57	90	350	650	1 820	3 900
Q, when $R = 1.50$, and $C = 0.50$	15	20	107	167	650	1 220	3 410	7 320

gives similar values for the formula just developed. The distances were estimated at $443\sqrt{A}$, the areas being twice as long as broad, Mr. Hoad. and the velocities were assumed as indicated. Arbitrary additions of 8 min. in the case of the 1 to 1 000 slope, and of 5 min. in that of the 10 to 1 000 slope, were made in order to allow time for the water to reach the sewer. In order to make the run-off for 100 acres with the 10 to 1 000 slope equal to 90, and thus fulfill the conditions required of the other formulas, R was put equal to 0.80, certainly a lower value than should ordinarily be assumed in practice. In the lower part of the table are given similarly computed quantities for a value of $R = 1.5$, C remaining the same, which probably more nearly represent average conditions than those above.

Mr. Marston. A. Marston, M. Am. Soc. C. E. (by letter).—The most important part of this paper seems to the writer to be the general presentation of the following principles:

First.—The water falling on the so-called "impervious" area of an ordinary sewer water-shed does not all run off as fast as it falls, but part of it accumulates in increasing quantity on the surface during downpours of moderate length, such as cause the maximum discharges from sewer districts of ordinary size.

Second.—As the storm continues at the same rate, the ratio of run-off from the "impervious" area to the rate of rainfall increases, owing to the increased depth and velocity of the surface flow toward the sewer, until finally, if the storm lasts long enough, the rate of run-off from the "impervious" areas becomes equal to 100% of the rate of rainfall.

Third.—The "pervious" area becomes more and more saturated with water as the storm continues, and there will be a percentage of run-off from "pervious" areas increasing from 0 for short storms to quite a large percentage for storms lasting several hours.

There can be no question as to the general truth of these three principles, but when the author attempts to go further and present definite curves and formulas, as in Fig. 8, purporting to show the exact laws of change in the percentage of run-off with relation to the time elapsed since the beginning of the storm, the writer believes he is going beyond what the present meager data from gaugings of storm sewers warrant.

These data are not very extensive, and the author himself points out their many defects. For many years engineers have been criticising the old run-off formulas, and new formulas, based on insufficient data, should be avoided.

All that engineers are at present warranted in doing is to make some deduction from 100% run-off from the "impervious" areas, for short storms in favorable cases, and some increase above 0% (say varying up to 20% for 1-hour storms, with average soil and slopes) in the run-off from the "pervious" areas for long storms, both the deduction and the addition being at present left to the judgment of the engineer, in view of his general knowledge and his familiarity with local conditions. Mr. Marston

The gaugings shown on Plate LXIX are of considerable value and interest, though they are not reliable as to relative times of rainfall and sewage flow fluctuations, because the automatic rain gauge was 2 miles away.

The storm of August 21st is especially interesting, on account of the three successive downpours, giving three principal discharge maxima, approximately at 7, 7.30, and 8.30 A. M. These illustrate very clearly the danger of adopting any theory which would make the rate of run-off start in at 0 at the beginning of the downpour which causes the maximum discharge, as indicated by the curves and formulas of Fig. 8.

The fact is that there is always a possibility that at the beginning of the maximum downpour which determines the maximum rate of run-off, an immediately preceding downpour, only a little less in rate, may have already heavily flooded the water-shed, and increased the percentage of run-off from both "impervious" and "pervious" areas. Thus the downpour from 7 to 7.30 A. M., on August 21st, superposed its run-off upon the afterflow of the preceding downpour, and brought the maximum rate of run-off up to approximately 78% of the average rate of the preceding 30-min. rainfall.

This is close to the total percentage of "impervious" area of the sewer district, figured as recommended by Emil Kuichling, M. Am. Soc. C. E., by taking only 80% of the area of first-class pavements.

One is forced to wonder what the discharge of the sewer would have been from the 30-min. downpour from 9.10 to 9.40 A. M., on August 4th, if it had been closely preceded by a heavy downpour, as was the case on August 21st.

The percentage of maximum run-off is also greatly affected by the distribution of the rain through the downpours, as may be seen by comparing the 30-min. rains of June 26th, and August 4th. The first gave approximately 71%, maximum run-off, and the second only 53½%, the difference being attributable to the fact that the heaviest rainfall occurred at the end of the June 26th, and at the beginning of the August 4th downpours.

In regard to the "time of concentration," it seems to the writer that the author is in error in assuming that usually no addition need be made to the time required for flow through the longest sewer to allow for the time required for water to reach the sewer from the gutters and roofs at the most remote point. He argues that this would be balanced by the time required for the water to

Mr. Marston. reach the sewers from the gutters and roofs nearest to the outlet, but this cannot usually be the case, for the increased quantity of water from the surface immediately around the nearest street inlet must usually begin to enter the sewer within 1 min. of the beginning of a heavy downpour preceded by a lighter rain. The curves on Plate LXIX, which he cites, are not reliable, as regards relative times of rainfall and flow, for the rain gauge was 2 miles away, and he does not state whether the sewer gauge was not some distance from the nearest street inlet.

Alva J. Grover, Assoc. M. Am. Soc. C. E., on Plate IV of his paper "Flood Waves in Sewers, and Their Automatic Measurements,"* presents some simultaneous automatic gaugings of rainfall and sewage flow at Omaha, Nebr., which appear to be perfectly reliable, as regards relative times, and show a practically instantaneous response of the sewer flow to the rainfall.

In regard to new rainfall curves and formulas, it would appear that a number of curves and formulas can be found, all representing the known facts fairly well. The case is similar to that of formulas for safe loads on columns for bridge specifications. In each case the formulas which make the calculations most simple have important advantages over the others. Exponential functions do not appeal to the writer as being convenient for calculations; compare, for example, Talbot's well-known $\frac{105}{T + 15}$ with the proposed $\frac{32}{T^{\frac{4}{5}}}$.

The writer has yet to find any general rainfall curves better fitted to published rainfall data, or simpler to use, than Talbot's well-known and simple formulas.

Mr. Ruggles. W. B. Ruggles, M. Am. Soc. C. E. (by letter).—The author of this excellent paper quotes the deductions from Mr. Kuichling's experiments:

"1.—The percentage of rainfall discharged from any given drainage area is nearly constant for heavy rains lasting equal periods of time.

"2.—This percentage varies directly with the area of impervious surface."

It is not the writer's purpose to dissent from these conclusions as applied to any roof-covered or paved areas—although, even in New York City, 10% of the area upon which Mr. Hering's experiments were made was composed of grass—but to suggest a guard against hasty application of these rules (clearly not intended by the author or experimenter) to drainage districts having areas largely of cultivated lands, or covered with vegetation, especially the dense, heavy-bladed, cup-like vegetation of the tropics, and with a light soil of vegetable mould. Taking as an example the drain-

* *Transactions*. Am. Soc. C. E., Vol. XXVIII.

age of areas along the Panama Canal, it has been observed that at the end of the dry season, of, say, 4 months, when everything is parched, a rainfall of 3½ in., from 12 to 9 P. M., followed next day by a like amount from 12 to 4 P. M., did not yield nearly so great a run-off at a large pipe opening, readily observed, as would a little more than half as much (during the same time) later in the season when everything was saturated by the daily rains. Even at the latter period, area for area, it would hardly be expected to yield such a run-off as occurred in the Ohio River Valley in the record flood of that stream (which, in 1884, at Cincinnati reached a stage of 71 ft. 0¾ in. above low water) when the rainfall was mainly upon frozen, sleet-covered, but sometimes temporarily thawing, ground, corresponding in condition much more nearly to that of paved areas than could otherwise exist over so large a region. The writer has not at hand the figures for the area or volume of that flood, but, if available, they would probably lend support to the conclusions reached by Mr. Kuichling. Mr. Ruggles

TABLE 8.—HEAVY RAINFALLS IN CUBA, NICARAGUA, AND PANAMA.

Place.	Date.	Rainfall, in inches.	Duration of rain.	Remarks.
Santiago, Cuba........	Sept. 3, 1900...	10.58	12 hours.	Ending 8 P. M. } By U. S. Weather
" "	Sept. 6, 1900...	22.20	60 "	Ending 8 A. M. } Bureau Reports.
Matanzas, Cuba.......	Apr. 6, 1901...	4.7	1½ "	Between 7.30 and 9 P. M.
" "	1901 or 1902*..	6.92	24 "	Almost exactly the same rainfall was reported for Havana on the same day.
Greytown, Nicaragua.	July 18, 1891...	8.17	9 "	
" "	Dec. 5, 1892...	8.75	24 "	
La Boca, Canal Zone..	Nov. 17, 1906...	7.3	24 "	Most of this precipitation occurred in 12 hours
Culebra, Canal Zone..	Oct. 24, 1890...	4.73	24 "	On the Continental Divide.
" " " ..	Dec. 3, 1906...	5.55	24 "	
Empire, Canal Zone..	Dec. 3, 1906...	6.15	24 "	Of this, 5.6 in. fell in 14 hours.
Culebra, Canal Zone..	May 10-20, 1897.	20.3	10 days.	The total fall for May, 1897, was 32.86 in.

* Memorandum of exact date mislaid.

Now that the requirements of sanitation are leading to new or improved sewerage systems in Panama, Cuba, Porto Rico and other tropical countries, more or less under the direction of American engineers, occasion is taken to call attention again to the greater rainfall of those countries, as shown by Table 8.

EMMET A. STEECE, M. AM. SOC. C. E. (by letter).—This problem, so ably handled by the author, is one which engineers have tried to solve in only a desultory way for years, and its importance is worthy of more consideration than it has received. Mr. Steece.

Mr. Steece. The writer is of the opinion, however, that a more thorough and systematic investigation of the subject will be necessary before any convenient comprehensive formulas can be written.

The various formulas, including those by the author, contain definite expressions of the slope of the surface, without considering the element of perviousness except by indefinite factors.

In the average impervious district the portion affected by slope is rarely more than 50 per cent. What effect, then, has the slope of the surface upon the remaining 50%, covered by structures of every conceivable shape, and with rain-water conductors of directions as various?

Let it be fully considered that any formula which does not have an arrangement of factors which will make it applicable to different climates is also without material value, for, in the North the entire surface often becomes impervious, as when the ground is saturated and frozen, and in the South the surface conditions change but slightly in comparison.

The writer has made many observations of drain discharge of rainfall, but has not considered them fit for publication for the reason expressed by Mr. Gregory in his comments on some found in engineering literature, namely, their incompleteness.

The writer believes that the best solution of the problem will be accomplished by as complete an investigation as has been given to the properties of engineering materials; fully illustrated by photographic views of the site of each experimental district, rather than formulas based upon a few disconnected individual experiments, widely scattered and vaguely stated.

Mr. Le Conte. L. J. Le Conte, M. Am. Soc. C. E. (by letter).—The writer is much impressed by this elaborate paper. The author's collection of data and his criticisms of the various formulas are very interesting. From a sense of duty, however, the writer feels compelled to call attention to the fact that, in his opinion, it is utterly hopeless for anyone to attempt to deduce a useful and practical general formula suitable for application to any case that may come up in practice. It is the writer's opinion that a general formula, which will inspire complete confidence, can never be prepared. This fact is due primarily to the large number of entirely independent variable factors necessarily involved in the final result. In all cases like this, the only formula which has any practical value is an empirical one, based simply on facts and observation. Of course, such a formula has naturally a very limited application; and great mistakes are always being made, in trying to stretch it so as to give it a wide field of application. Nothing could be more unfair or irrational, and yet this is being done every day.

The data really needed are more reliable observations and more

Mr. Le Conte.

facts as to the actual maximum discharge of sewers, noting carefully the time interval between the crest of the heavy rainfall and the crest of the maximum sewer discharge following the heavy rainfall.

All the physical facts directly connected with the case should also be noted as fully as possible. When this is done carefully, the results can be used safely, but within narrow limits. The troubles encountered in this problem in hydraulics call to mind the same difficulties in computing the flow and discharge of streams. Nothing is so unsatisfactory to-day, as the various formulas for the flow and discharge of rivers—and yet the variables in those formulas are not nearly as complicated and erratic as those in the storm-water sewer problem.

All the river formulas bear a very close relation to the slope per mile, since gravity is the moving force; but every student of river hydraulics knows that there is nothing so variable and unreliable as surface slope per mile. The channel cross-section is continually changing, the velocity is continually changing, and the slope per mile is also subject to the most erratic changes. The only absolutely constant factor is the discharge, which is uniform. The most remarkable instance, as to slope per mile, is to be noted on the Mississippi River between Red River and Donaldsonville, where there is a stretch of river, extending for at least 40 miles, along which the surface slope is reversed, and is up stream instead of down stream, and yet the great river forges ahead down stream, just as if the slope were in that direction. This instance is only mentioned as a caution to engineers, and to remind them how careful they should be in taking surface slope per mile as a reliable factor. Nothing is more unreliable, and yet all river formulas compel its use as an important factor.

A great many of the older members of this Society, no doubt, have valuable notes bearing upon the subject of this paper, but are chary about putting them in print, because they are a little uncertain about some of the variable factors. The writer thinks this is a great mistake. The more observed facts that can be obtained, the better can the whole category of facts be examined, and in due course of time engineers will arrive at more intelligent conclusions.

In the case of Hawksley's original formula, although the rainfall was considered in a general way, yet it is noticed that when he came to frame his formula for practical use he discarded rainfall entirely. As the writer understands it, his original formula contained no rainfall factor. He simply considered the area of the tributary water-shed and the average slopes of the ground surface, and then he carefully gauged the storm-water discharge

Mr. Le Conte. of the sewers after heavy rainfalls. He also noted the height of the flood wave as it passed the gauging station, and the time length of the same wave, as near as might be. He fully appreciated the fact that it was the shape of this progressive flood wave that he was after, and that this really fixes the proper size of the storm sewer. It may be a long, flat wave, which calls for a comparatively small sewer, or it may be a short, high-crested wave, which calls for a much larger sewer.

By what process of reasoning can one arrive at the conclusion that the rainfall is a direct function of this flood wave? The only direct functions are the physical features of the tributary water-shed. The most that can be made out of the rainfall is a secondary function, and, at all events, it cannot be placed in the same category with the direct physical functions peculiar to the water-shed, which alone influence the shape of the progressive flood wave. Hawksley wisely concluded that, at all events, rainfall is not a direct controlling factor in the problem, and therefore purposely omitted it from his formula. It seems to the writer that this was a logical conclusion.

What engineers wish to ascertain is the actual storm-sewer flow, and this can only be found by actual gaugings. This is far preferable to measuring the rainfall at detached points of observation and then theorizing about, and calculating in the office, the storm-sewer flow from a computed average rainfall over the entire water-shed, which, at best, is the wildest kind of a guess. The thing to observe directly, in any case, is the storm-sewer flow, irrespective of whence or how it really originated.

Of course, rainfall, as a secondary function, has a general bearing on the problem, as instanced by the Brooklyn case, where it became necessary to modify the old formula. In considering this case, it is found that the average annual rainfall at London is only 26 in., well distributed, and with maxima of 1.8 to 2 in. per hour; whereas, at Brooklyn, the average annual rainfall is 43.3 in., and with maxima of probably 3 in. per hour, so that the Hawksley formula had to be modified—but, in any event, the best way to find out definitely how much to modify it, would have been to gauge the actual storm flow in existing sewers after heavy rainfalls and note the height and shape of the flood wave passing the gauging station. This is the only rational conclusion. In this way, reliable factors, which could be depended upon in that particular neighborhood, could have been obtained. In applying these results to another city, having markedly different physical conditions, a high order of intelligence and judgment would be required.

Mr. Burdick. CHARLES B. BURDICK, Assoc. M. Am. Soc. C. E. (by letter).—It is likely that no problems confronting the engineer are more difficult of solution than computations relating to stream run-off, or the flow in storm-water sewers. Such means of computation as engineers possess are based upon experiment. Definite statements of actual flows under stated conditions are rare, and it is for this reason that a compilation of run-off data, such as has been presented by the author, is particularly valuable. Whether the data presented are sufficient to justify a new formula, of general application, is perhaps subject to doubt. Mr. Burdick.

The problem is subject to approximate solution only, and, fortunately for those dealing with it, a close approximation is sufficiently accurate for ordinary purposes.

As the expenditures based upon these computations are very large, it follows that each particular problem warrants the most careful study, in the light of all the data that can be brought to bear upon it, particularly such local data as can be secured.

It has proved instructive to the writer to summarize such conclusions as have resulted from the study of this problem in various localities, and the diagram, Fig. 14, and Table 9, which is explanatory, are presented more with the idea of showing the wide variation in the necessary allowances under different conditions, than to present data of value for local application. The data from which the curves were plotted have been secured from reports and miscellaneous sources in which much of the information bearing upon the reasons for adopting the run-off allowances is lacking, or is not as complete as would be desirable. These curves, however, are the result of conscientious study, and, in a number of the cases, are based upon observations of sewers previously constructed in the cities referred to. With due allowance for personal equation, the effect of local conditions is seen at a glance to be large.

The author (page 474) has conveniently reduced the best known run-off formulas to the same nomenclature, and it will be seen that, for areas ranging in size from small to medium, the results by the use of the various formulas do not differ greatly. The Bürkli-Ziegler formula, or its modification by McMath, is perhaps used to as great an extent as any. The meat of the proposition, however, lies in the coefficients, so that, in reality, all these formulas are in themselves useful, principally in determining the relative rates of run-off upon different areas.

The formulas used most frequently are deficient in that they do not make sufficient allowance for variations in surface slopes. Corrected in this respect, the question of permeability still stands between success and failure. It makes little difference what coefficients are used, provided the results are reasonable, and it has been the practice, in applying the Bürkli-Ziegler and similar

Mr. Burdick. formulas, to alter the coefficient of rainfall, or surface allowance, or their product, in such amount as seemed to fit the requirements best.

Although the source of run-off is rainfall, slope and permeability have equal effect, for, without slope, no flow can occur, and, with a sufficiently porous surface, no water can reach a sewer except through seepage. The rainfall is much the least troublesome factor in the problem, and, while its study is of importance in accounting for an observed flow, it has little significance in comparing the run-off allowances shown upon the curves presented in Fig. 14. A. N. Talbot, M. Am. Soc. C. E., some years ago,* showed that high rates

TABLE 9.—EXPLANATION OF FIG. 14.

City.	Conditions, etc.	Formula.	R.	C.	S.
Baltimore, Md.....	Recommendation of Commission.	McMath.	3.5	0.80	15.
St. Louis, Mo.. ..	Clay soil; steep slopes.	McMath.	2.75	0.75	15.
Elgin, Ill..	Glacial drift; black soil surface; good drainage.	McMath.	2.75	0.50	15.
Peoria, Ill..........	Largely steep slopes, and considerable clay soil.	Bürkli Ziegler.	2.	0.50	10.
Boston, Mass......	Stony Brook. Recommendations of Commission.	Bürkli-Ziegler	1.0	0.75	20.
Paducah, Ky......	Clay soil; largely flat.	Bürkli-Ziegler.	2.75	0.65	1.
Terre Haute, Ind..	Sandy subsoil; thin stratum of black earth.	Bürkli-Ziegler.	1.5	0.62	1.5
Gary, Ind..........	Sand, no black soil; flat.	Bürkli-Ziegler.	1.	0.60	1.
Chicago, Ill........	For urban districts; sandy subsoil; flat slopes.	Bürkli-Ziegler.	1.	0.80	1.
New York, N. Y...	Formula mentioned in Mr. Gregory's paper Q (total quantity) $= C\,R\,A^{0.83}\,S^{0.27}$, in which $C\,R = 1.64$ and S is taken at 7.				

of rainfall vary little from Maine to Louisiana, although the annual rainfalls differ materially; he also showed that, from the data available at that time, there appeared to be no relation between annual rainfalls and high rates, further than the possibility of more frequent heavy precipitations in localities of high annual rainfalls.

The variations in municipal allowances must be accounted for most largely by variations in the slope, and (in the writer's opinion, more important still) the permeability of the surface. The latter factor, of course, is of less significance in a closely built business district, where artificial conditions may offset those of Nature; even in outlying districts, provision for the future must tend toward the same artificial condition.

* *The Technograph*, No. 6.

Mr. Burdick.

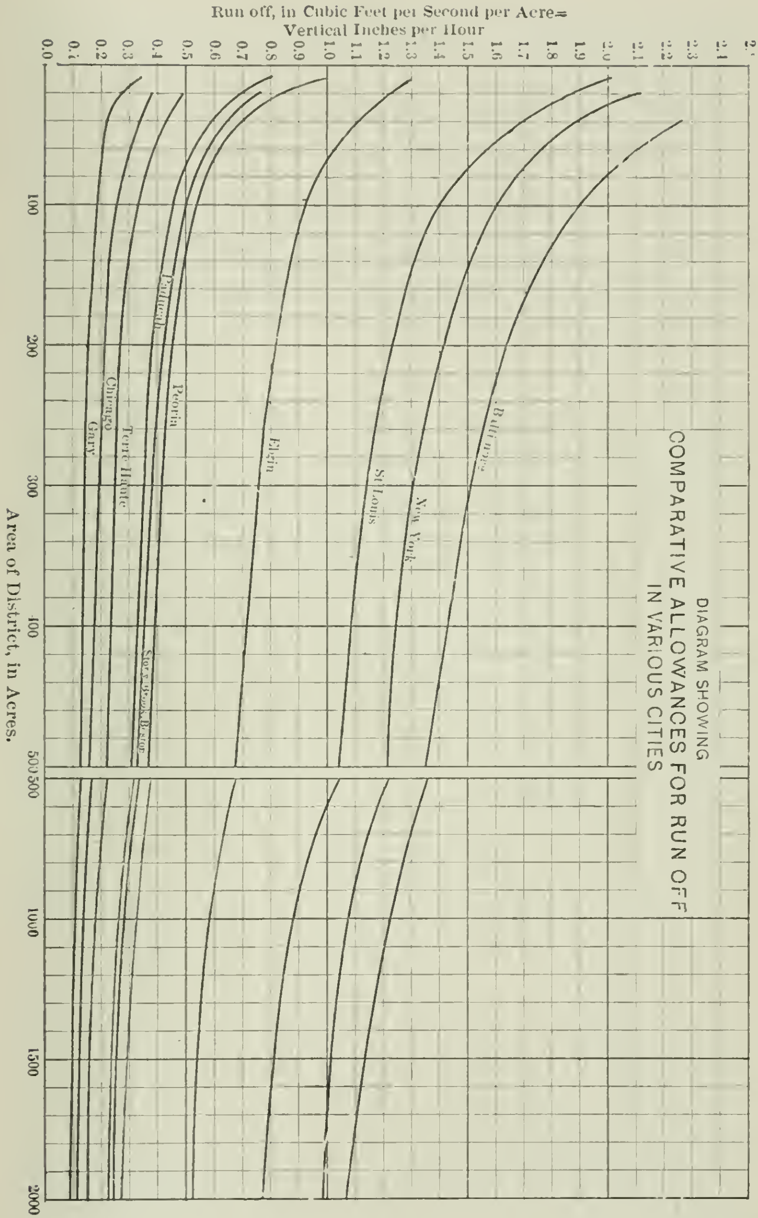

Fig. 14.

Mr. Burdick. It is frequently stated that rainfall or melting snow upon frozen ground produces a condition closely approaching that of the highly improved district. It is believed, however, that the effect of this condition is frequently exaggerated. The writer has in mind a flat, sandy locality where storm sewers, except within a business district of a few acres, have received no storm water except through seepage, and no doubt the catch-basins will remain unfilled until the streets and dooryards assume a very different aspect from that at present. Even with relatively impervious surfaces, where the ground is flat, large storage areas are brought into use, resulting in relatively small run-off rates, particularly if the street drainage is indifferent. This undoubtedly accounts largely for the low run-off rate in Chicago.

There is another factor, which, if not to be considered in the computation of run-off, must be given due weight in proportioning the run-off vehicle, namely, the ability of the community to withstand the first cost of the improvement. It is hardly necessary to state that a semi-urban community can better afford to withstand the effect of an exceptional storm than can a closely built district in New York or Chicago, with the deep basements and valuable merchandise and furnishings of those cities. Undue provision for the future is the frequent cause of delaying sanitary improvements for years. It is customary to design sewer systems, particularly in the smaller cities, with a liberal allowance for the future. In most cases this can be done without undue expenditures. It is wise, at least, to know how the future will be cared for when it arrives, but it is bad finance and bad engineering to make expenditures for a future which is problematical, if the interest upon the investment consumes the advantage gained by the present expenditure, or if the future can be almost as cheaply cared for when the need arises.

In the design of storm-water sewers, this should not be overlooked. There are many cases where a relatively low run-off rate can be applied, and the future can be cared for by a few lines of low intercepting sewers constructed when improvements on streets and lots cause the run-off rate to approach the impermeable.

In the application of a run-off formula to a particular problem, the value of local data is of first importance. If the city is partly sewered, a study of these sewers, as regards adequacy, is particularly profitable, if studied in the light of the controlling factors. If no sewers exist, a study of the culverts and streets and railroads is profitable in at least furnishing negative information, a minimum below which the discharge cannot fall. Culverts or waterways deficient in size furnish the best data, as their carrying capacity, particularly when "headed up" can be computed with reasonable accuracy.

What has been said herein must not be interpreted to minimize in any way the valuable data which the author has presented; it is just such data that enable the engineer to take full advantage of any scraps of local information that may be obtained, particularly if local conditions approximate those mentioned in the paper. The general application of local data, however, must be made with caution, or the results will not be fruitful of good. Mr. Burdick.

Charles Emerson Gregory, Assoc. M. Am. Soc. C. E. (by letter).—There is one point at least upon which the writer and apparently all who have contributed to this discussion are agreed, and that is the need for more experimental data in the form of careful and systematic observations covering a long period of time, before either the constants or variables of this problem can be determined finally and accurately. Mr. Gregory.

The writer would say to Mr. Marston that he does not claim, and has not claimed, that the curves presented show final and exact laws. But it is claimed that they represent a convenient approximation, which shows the relation, and explains many superficially apparent discrepancies between the data presented. Therefore, as far as any conclusion can now be reached, they show probable values.

The storm of August 21st was analyzed carefully, and the results were explained by the curve for impervious areas on Fig. 8, or rather, this storm with others was used to establish the curve.

Mr. Marston apparently misunderstands the writer's statement as to the "time for concentration." The water that falls during the first part of the storm, near the point of observation, has no influence on the ultimate wave crest, as it has run away before the large flow from the remote parts arrives. The time to be deducted is that which elapses between the moment when the downpour ceases and when the crest of the wave reaches the point, or that time required for the last of the rain to reach the sewer. While it is usually about the same as the time at the remote parts, it is not always so. If, however, the rain did not cease at this moment, the wave would continue to rise, as shown on Fig. 8.

Formula 10 of Table 4 was given solely in order to deduce, in a rational manner, a more correct formula of the exponential form for comparison with older forms.

Messrs. Steece and Burdick have helped to emphasize the point to which little space was given by the writer, *viz.*, that the run-off from the so-called pervious areas is influenced by so many factors that they can be approximated only after many more experiments have been made. As was stated by the writer, the run-off from not very radically different areas may vary from 0 to 90 per cent.

The comments of Mr. Le Conte are very interesting and instructive, but they would seem to indicate that he does not appre-

Mr. Gregory. ciate the position, taken by the writer, that a formula cannot express more than the very roughest approximation, and that in preference a general method of design which permits intelligent and correct consideration of the variables should be followed.

Although formulas were presented, they were more for the purpose of comparison and analysis than for use, except where only approximate results are desired.

The writer heartily agrees that more data are needed, and especially data concerning the influence of physical conditions, but the proposition that engineers be guided solely by actual gaugings of sewer flow, is, in many cases, simply putting the cart before the horse, as sewers which have not yet been built cannot be gauged. If only the results of gaugings of sewers in one locality are to be used, they cannot be used safely for a different locality, where the rainfall, as well as the physical conditions, are different, unless there is a fair understanding of cause and effect in the laws of change of all the variables for the changed conditions. The rainfall, instead of being a secondary function, is the primary one, as run-off cannot result without rainfall, and it is the one universally dominant influence. These laws and conditions, as indicated by the meager data at hand, the writer has attempted to state and evaluate. Additional data may materially change the values and possibly further subdivide the influencing conditions, but whether or not they have been stated and evaluated correctly here, it is absurd to expect to make an intelligent and rational design without such knowledge.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

No. 1049.

Discussion.

TYPHOID MORTALITY IN SOUTH BETHLEHEM, PA.*

By Mansfield Merriman and Winter L. Wilson, Members, Am. Soc. C. E.

Referring to the discussion by J. P. A. Maignen, Assoc. Am. Soc. C. E., on the paper by Messrs. Hazen and Hardy, printed in *Transactions,* Vol. LVII, p. 383, the writers desire to present a full record of the typhoid mortality in South Bethlehem, Pa. Messrs. Merriman and Wilson.

Mr. Maignen says "it may be of interest to know the influence which filtration has had on this disease in South Bethlehem," and then he gives figures which show 54 deaths from typhoid and enteric fever in 1903 and 1904, before the installation of the filter plant in November, 1904, and no deaths in 1905 and 1906. Two inferences are evidently intended to be drawn from these statements: First, that the normal mortality from typhoid before 1905 was about 27 deaths per year, or at the rate of about 180 per 100 000 of the present population; and second, that the influence of the filtration of the water supply has been to cause this disease practically to disappear. The first of these inferences is entirely unjustified, as the following statistics will prove; and the second will also be shown to have little or no validity.

The statistics given by Mr. Maignen are incomplete, as they cover only a short period, while also the cases and deaths given by

* This discussion, relating to the discussion by J. P. A. Maignen, Assoc. Am. Soc. C. E., on the paper by Messrs. Hazen and Hardy, "Works for the Purification of the Water Supply of Washington, D. C." (*Transactions*, Vol. LVII, p. 307), was received too late to be printed with that paper in Vol. LVII of *Transactions*.

Messrs. Merriman and Wilson.

him for 1903 and 1904 are much greater in number than those recorded in the files of the Board of Health of the Borough of South Bethlehem. The writers are unable to explain this discrepancy, except by the assumption that the figures include localities outside of the limits of the borough where the same water supply is used. This explanation, however, does not account for his statement that only 8 cases and no deaths occurred in 1905 and 1906, for the official records of the Borough of South Bethlehem show 22 cases and 5 deaths between January 1st, 1905, and November 1st, 1906.

In order that the facts, and all the facts, regarding the mortality from typhoid fever in the Borough of South Bethlehem may be known, Table 1 is presented. This table has been compiled by the writers from the records of the Board of Health.

TABLE 1.—Deaths from Typhoid Fever in South Bethlehem, Pa.

	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906
Jan.	......	......	......	......	......	......	......	......	......	......	......	......
Feb.	......	......	1	......	......	......	......	1	5	1	......	2
Mar.	1	......	1	......	......	......	1	2	4	......	......	......
Apr.	......	......	......	......	......	......	2	1	11	1	......	......
May	......	......	......	......	......	......	......	......	1	2	1	......
June	......	......	......	......	......	1	1	......	1	......	......	......
July	......	......	......	......	......	......	......	......	1	1	1	......
Aug.	......	......	......	......	......	......	......	......	1	......	1	......
Sept.	1	......	2	......	......	......	......	2	......	2	......	......
Oct.	1	......	1	......	2	1	......	......	1	......	......	......
Nov.	......	......	1	......	......	......	1	3	......	......	......	......
Dec.	......	2	1	......	......	1	......	1	1	......	......	1
Totals....	3	2	7	0	2	3	5	10	26	7	3	3

An examination of these figures shows that there were no deaths from typhoid fever in 1898, that the number of deaths increased each year until it reached a maximum in 1903, and has since decreased. There was, in fact, an epidemic of typhoid fever in 1903, in which year 147 cases were reported to the Board of Health and 26 deaths occurred. The cause of this epidemic was attributed by some to the water supply and by others to the milk supply, the weight of evidence being that it was due to the water. Whatever may have been the cause, the epidemic was over in the fall of 1903, and the typhoid mortality for 1905-1906 has been at practically the same rate as in the years 1899-1900. The total number of deaths from typhoid during the years 1895-1902 was 32, or an average of 4 deaths per year, and this may be regarded as the normal typhoid mortality in South Bethlehem, uninfluenced either

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by epidemics or by filtration. The epidemic of 1903, due probably to pollution of the water supply, caused this normal rate of 4 to be raised to 26. The average mortality in the two years, 1905-1906, is 3, but it is questionable whether the decrease from 4 to 3 can properly be ascribed to filtration when it is seen that an average of 2.5 is found for the two years, 1899-1900, and a still lower one for the two years, 1898-1899.

South Bethlehem derives its water supply from the Lehigh River. Allentown, five miles up stream, derives its water supply from springs, and discharges a considerable quantity of sewage into the same river. Table 2 gives the facts regarding the mortality from typhoid fever in Allentown, as compiled by the writers from the records of the Board of Health of that city:

TABLE 2.—Deaths from Typhoid Fever in Allentown, Pa.

	1895	1896	1897	1898	1899	1900	1901	1902	1903	1904	1905	1906
Jan.	.....	2	2	2	1	2	.....	1	1	1	1	......
Feb.	3	1	3	......	2	1	......	3	3	2	2	1
Mar.	2	4	1	2	......	4	....	8	2	......	1	3
Apr.	7	......	1	......	1	.....	4	2	.....	3	1	2
May	3	......	......	......	......	......	2	1	2	1	......	2
June	1	1	.....	2	2	......	2	2	2	2	1	2
July	4	......	......	3	8	......	1	......	2	.....	1	1
Aug.	10	2	2	4	3	......	1	3	......	1	3	3
Sept.	8	1	......	3	3	1	4	2	1	......	1	1
Oct.	7	4	1	3	2	1	1	27	2	4	1	1
Nov.	1	......	2	1	2	2	......	11	1	1	3	2
Dec.	2	5	1	2	......	......	2	5	1	2	3	1
Totals...	48	20	13	22	24	11	17	65	17	17	18	19

Table 2 shows fluctuations similar to those in the table for South Bethlehem, the number of deaths being greater because of the larger population of Allentown. The typhoid epidemic of 1903 in South Bethlehem was preceded by one in Allentown in 1902; the rise of the number of deaths in years prior to 1902, and the subsequent decrease, show a similarity to the fluctuations in the figures for South Bethlehem. The average mortality for the seven years preceding 1902 is 22 per year, and this may be regarded as the normal rate for Allentown independent of epidemics or filtration. The epidemic of 1902 caused the normal yearly mortality rate of 22 to rise to 65, and then, for the four years following, the average rate dropped to about 18. This drop in typhoid mortality after 1902 cannot be attributed to filtration, since Allentown has no plant for this purpose.

In order to compare properly the foregoing statistics of typhoid mortality for South Bethlehem and Allentown, it is necessary to

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reduce them to a common basis, and the number of deaths per 100 000 of population has been computed. In doing this, the population of the two towns for each year was first determined by interpolation from the census records, as follows:

	1895	1896	1897	1898	1899	1900
South Bethlehem..	11 770	12 060	12 360	12 650	12 950	13 241
Allentown........	30 320	31 340	32 360	33 330	34 400	35 416

	1901	1902	1903	1904	1905	1906
South Bethlehem..	13 530	13 830	14 120	14 420	14 700	15 000
Allentown........	36 430	37 460	38 480	39 550	40 620	41 690

The deaths from typhoid fever per 100 000 of population are then as follow for each of the years, 1895-1906:

	1895	1896	1897	1898	1899	1900
South Bethlehem...	25.5	16.6	56.6	0.0	15.4	22.7
Allentown.........	158.3	63.8	40.2	65.9	69.8	31.1

	1901	1902	1903	1904	1905	1906
South Bethlehem...	37.0	72.3	184.8	48.5	20.4	20
Allentown.........	46.7	173.6	44.2	43.0	44.3	45.5

These figures show the average yearly typhoid death rate per 100 000 of population for the twelve years, 1895-1906, to be 43.3 for South Bethlehem and 68.9 for Allentown. Omitting the epidemic year of 1903 at South Bethlehem and those of 1895 and 1902 at Allentown, the average annual death rate per 100 000 of population has been 30.5 for South Bethlehem and 49.4 for Allentown. The epidemic of 1903 in South Bethlehem caused the typhoid mortality for that year to be nearly six times the average rate, while that at Allentown in 1902 caused the rate for that year to be about three and one-half times the average rate. In the second year following these epidemics, the typhoid death rate in both towns dropped below the average. The remarkable similarity of the curves for the two towns leads to the probable conclusion that the typhoid epidemic of 1903 in South Bethlehem might have been due to that which prevailed in Allentown in 1902. This conclusion is strengthened by the facts that the water supply of the Lehigh River is always more or less polluted by the drainage of Allentown, and that the two epidemics were separated only by an interval of about four months. The drop of both curves in the two years following the epidemics, together with the known fact that the Allentown supply has never been filtered, furnishes evidence that the decrease of typhoid fever in South Bethlehem in 1905 and 1906 cannot fairly be attributed to the filtration plant which was put into operation in November, 1904.

AMERICAN SOCIETY OF CIVIL ENGINEERS.

INSTITUTED 1852.

TRANSACTIONS.

Paper No. 1050.

ADDRESS AT THE ANNUAL CONVENTION IN THE CITY OF MEXICO, JULY 8TH, 1907.

THE ENGINEER AS A PROFESSIONAL MAN.

By George H. Benzenberg, President, Am. Soc. C. E.

With the most sincere and profound regret I realize that, all efforts to the contrary notwithstanding, circumstances have persistently shaped themselves in such a way as to make it impossible for me to attend this Convention and to exchange the customary greetings and felicitations with the members of the Society and our hosts, who so kindly and urgently have invited us to hold our Thirty-ninth Annual Convention in the City of Mexico, the Capital of our Sister Republic, which city, let it be remembered, is the oldest permanent city on the American Continent, for it had its organized government, its courts of justice, its sculptured structures, and a people who had advanced in the development of order, skill, and knowledge, centuries before the Europeans even discovered their existence.

Under the long, judicious, and splendid administration of its honest, capable and fearless, yet conservative Chief Magistrate, who was first elected to the Presidency thirty years ago, the Republic of Mexico has made great progress. Order has been brought out of chaos, peace has been maintained, improvements have been fostered, industries encouraged, and the health, comfort, and general welfare of the people have been advanced. The great natural resources of Mexico, which are attracting attention and inviting foreign capital, will, when fully developed and utilized under the direction of the skilled and experienced engineer, add materially to her prosperity, increase her wealth and population, and rapidly advance her standing among the nations of the earth. The convening of the American Society of Civil Engineers within her borders is, therefore, at this time, most opportune, and, I believe, will mark an epoch in the history of the development of the agricultural, mining, and other resources and industries of this great country, and thus result in mutual advantage to both host and guest.

The Constitution of our Society provides that the President shall deliver at the Annual Convention an address, presumably consisting of a general résumé of the most important work in progress and of whatever advance has been made in the engineering art during the past year. Such a statement, however, would be but a repetition of what is being currently presented by the various journals, magazines, and such other publications as are devoting their columns to engineering subjects. All these keep the engineer well informed, not only of all notable work under construction, but also of all that is being projected. Through the perusal of these ably edited and carefully compiled periodicals, we are sufficiently enabled to keep fully abreast of the times, in a statistical and instructive way, without my adding further details by a perhaps tiresome summary at this time. It may not be inappropriate, however, to call your attention to the fact, that during no similar period in the history of engineering in this country have so many works of such gigantic magnitude, requiring the expenditure of such vast sums of money, been undertaken or entered upon as during the past year. Among these, the principal works, probably not over nine or ten in number, are all devoted to improving or enlarging present facilities for navigation, travel and commerce, or to providing new channels and new terminals for the accommodation of such, and involve the expenditure of nearly seven hundred millions of dollars, while nearly as large a sum is to be spent from public funds for the development or improvement of water supplies, the betterment of sanitary conditions, the reclamation of arid lands, and the construction or reconstruction of highways, bridges, and other public works for the health, relief, and convenience of Man.

One can scarcely conceive the influence which these great works and improvements will have upon the health and prosperity of America, the commerce of the world, and the betterment of the condition of mankind.

It is, however, not only the constantly increasing needs of commerce and transportation, and the comfort and welfare of the public at large, that engage the talent of the engineer. The prevailing tendency to acquire profit and benefit by the introduction of more economical processes of construction and manufacture, as well as by the reduction of wastes, makes it imperative to enlist his analytical and inventive skill and his trained and experienced judgment in almost every branch of human activity, and to secure his advice or approval in every important enterprise. Hence, everywhere, immense interests and ever-increasing responsibilities are being entrusted to the engineer by private as well as corporate organizations, and so thoroughly has he been able to meet every expectation, to fulfill every obligation, and to demonstrate his ability to meet every emergency successfully, that his service is considered as so elementary, so absolutely essential, that his counsel is not only invariably sought in the line of design and construction, but very frequently also as to the possibility or probability of any undertaking becoming a profitable venture. This, of course, is not only true with reference to the engineer in America, but is equally true as to his connection with the industrial development in every other country. It is in keeping with the world's practical grasping at every opportunity to profit by the knowledge of the highest and best exponents before entering upon or engaging in any great or important venture. These exponents, therefore, should not only be well informed, accomplished, and gifted in their calling, but they must also be the representatives of the very best and highest type of that which is implied by American Citizenship. Besides being designers and constructors of works for the upbuilding of commerce and every other industry, they should also, to come up to the standard of the profession, become foremost and eminent among their fellow men. When this high station has been reached by honorable and distinguished achievement, the world has always with delight exhibited its admiration and approval, and been pleased to honor the profession. This marked position of trust, influence, and eminence which the profession has attained so signally in the industrial world is undoubtedly due to the fact that engineering has become a science of the highest standard, wonderfully developed as such within the past century and especially during the last fifty years.

This development and progress has been particularly along the line which constitutes the very essence of engineering, and consists in establishing and advancing a thorough knowledge and understanding of the various properties of material, of the forces and latent energies of Nature, and of their economic and intelligent application to the needs and benefit of Man. This present knowledge has been acquired by faithful and patient study, by close and careful investigation, by thorough and systematic experiments and tests, and by compiling and communicating the results thereof by reports and publications, in order that the profession might have the full benefit of such research and continue the same for the information and benefit of its members. There was a time when such information was not to be had. The engineer had but two ways of acquiring professional knowledge, either as an apprentice to some one experienced in engineering, or by his own investigation and experiment, often made at the expense, and sometimes to the injury, of his employer. At the present time, the thoughts, works, and valuable experiences of engineers in every branch of the profession are preserved in the form of reports and papers, that not only describe but illustrate the work of which the author had charge or with which he was associated, and relate in detail the problems and often the difficulties which were encountered, and the manner in which they were successfully overcome. Add to this the thorough and practical discussion which papers of such character usually call forth, revealing often the very best experiences and practice of others, and you have a way of imparting and acquiring knowledge that was not much more than thought of fifty years ago.

Furthermore, the various engineering colleges and technical schools of to-day contain libraries whose shelves are stocked with treatises containing specific and valuable instruction in engineering, as well as with official reports on engineering work. They also possess complete laboratories, thus enabling the student in every branch of the profession to become at least somewhat familiar with the practical end of his study and to train his mind in the proper method of conducting experiments and of investigating and diagnosing the engineering problems which may present themselves, or which may be entrusted to his charge. Moreover, every engineer of to-day, besides his collection of books of reference, keeps himself supplied with all the current literature and with every report and article published pertaining at least to that branch of engineering in which he is especially engaged, in order to remain well informed and abreast with the current conception, practice, and experience of the craft.

The marvelously rapid progress made in every branch within the last 25 or 30 years is therefore directly due to the fact that all the learning and experience of the profession has, by it, been prepared and made readily accessible to the engineer and student, and has thereby enabled him to project and carry out his work in such a manner and with such favorable results as frequently to challenge the wonder and admiration of an appreciative nation.

To promote and foster this service further, and to maintain the high standard of the profession among its members, as well as in the estimation of mankind, the profession in our country organized this Society, as well as many kindred associations relating to the different branches of professional work, for the advancement of the individual engineer.

To the younger generation of engineers, who have or have not had the benefit of a technical education, a membership in this and kindred societies is of inestimable value, for it affords them the opportunity not only of participating in practical discussions, but also of having access to valuable documents, papers, drawings, and publications, containing a great mass of information which otherwise is not to be had, and which the earlier engineers did not possess. The present very rapid and remarkable growth in the membership of this Society demonstrates that this fact is now fully understood and appreciated. This advantage will doubtless lead the younger engineer to still greater achievements in his time, providing he does not entirely lose sight of the fact, that, though at college he may have learned how to proceed in preliminary investigation and study, and how to apply his knowledge, he has not as yet, by practice or experience, acquired the requisite judgment or the necessary discernment and prudence so essential to success in designing and carrying out engineering works.

It may be properly stated, therefore, that the present-day engineer largely owes whatever success he may have achieved, or reputable position he may occupy, primarily to the advantage which the profession has accorded him, through the close study and diligent research of others who have preceded him and have given the benefit of their learning and experience to the craft, and to which he in turn has likewise added his share for the benefit and welfare of others who may follow.

If this be true, and the engineer, because of this service and also the eminence of his profession, occupies to-day a higher position in the world and enjoys a standing and reputation which compares most favorably with that of any other profession, then the question very properly arises, what duty does he in return owe to the profession; what should be his attitude toward it and his brother engineer, and what position must he ever maintain before the community at large, in order to receive always the full confidence to which he and it should and must always be entitled? This query is a pertinent one, which no engineer should disregard or hesitate to answer.

The engineer who is true and loyal to the pure purposes and noble and lofty character of his profession will bring to it the love, pride, and enthusiasm to which it is justly entitled because of its mission and its achievements. His true devotion to duty allows no unworthy sentiment, interest, or motive to influence him in his professional counsel or work. He places his profession always above himself, and carefully avoids every connection with any undertaking of the integrity or honesty of purpose of which he entertains a doubt. He is mindful that every act of his shall reflect creditably and elevate the standing and reputation of engineering, nor does he hesitate to exert his influence, his power, or his authority against any questionable proposition.

With the prevailing and ever-increasing tendency of the present day to acquire wealth rapidly, many temptations are thrown in the way of the engineer. Individuals or corporations engaged in monopolizing natural or artificial advantages for selfish purposes, to the detriment of others, do not hesitate to seek the aid of the engineer in furthering their objects. They tempt him to close his eyes to questionable and reprehensible practices. The engineer who is the head of, or is employed in, a municipal or other government department, or who has charge of work executed under contract, is nearly always exposed to such temptation. Every kind of influence is brought to bear upon him to exercise the duties of his office or of his position for the benefit of some individual, party, or combination. It assumes every conceivable guise, and is frequently used in such an insidious manner that the unsuspicious mind is not aroused. How often has not the engineer on municipal work been importuned to place the inspection of such work in the hands of someone whose sole qualification consisted in nothing better than the possession of political influence, and who was expected to be not over-scrupulous in carrying out the instructions of the engineer? In such case there must be but one issue, consistent with self-respect and with the reputation of the profession, and it is a credit to the craft to state that it is not often that such issue is not raised and carried out by the engineer, though it may often produce temporary embarrassment to himself.

The element of loyalty to duty, of absolute faithfulness to every honorable engagement, which is not infrequently ignored in many professional and business enterprises, must ever be a distinguishing characteristic of every member of this profession. No matter what may be thought, said, or done in any other walk of life, that much, at least, the engineer owes to his calling, which through its combined achievements reflects credit and reputation upon the individual. In whatever form his service is rendered, whether in an official or private capacity, it must be given with honesty of purpose. The world must know that the engineer does not lend himself, his ingenuity, his skill, or his experience to aught but what is right, and that, having the privilege of declining, his acceptance of a commission means that he is not, knowingly, lending himself to any questionable undertaking, and that his sense of probity, his pride in his work, and his devotion to his profession rise above every other influence or consideration.

The engineer, therefore, must not regard his work only as the means for acquiring a competency, but he must bring to it an earnest enthusiasm which will pilot him safely past every temptation to ignore his duty. His sense of honor will guide him in avoiding errors, failures, and disappointments in his work, and cause him not to assume any obligations he is not qualified to fulfill, or which are at variance with professional propriety and dignity.

To his brother engineer he should always entertain the same candor, courtesy, and respect that he does toward the entire brotherhood; he should always be fair minded and honorable. His sense of justice and equity will never allow him for one instant to forget that he himself is not infallible, that there is often more than one way of solving a problem, and that the work of the "other" may be capable of as good results, with as high economy, as his own solution; nor will he forget that his fellow engineer has the absolute and unqualified right to receive the same consideration he would himself expect. He will, with the innate spirit of a gentleman, honor and respect the opinion and judgment of his fellow engineer, if given in good faith, to the same extent that he values his own; for the argument and reasoning of the one may be as sincere and logical as that of the other, and may possibly be founded on as good or even better experience, the relative merits of which can only be determined by unbiased consideration. He should also recognize that the reputation of the individual is inseparably linked with that of the profession, that the honor and glory of one reflects upon the other, and that, therefore, by maintaining the standing and reputation of its members, he is but maintaining his own.

The respect and esteem which a community entertains toward any profession or its members is always in proportion to that which the members of such profession entertain toward one another.

Unfortunately, there are those who do not hesitate to presume upon the general confidence accorded by the community to any profession, and to exploit it or its reputation for purely personal profit and gain. In the other professions the law has been invoked to provide protection against the unauthorized acts and unprofessional practices of members or individuals whose dealings and transactions tend to reflect discredit upon their several occupations.

Happily, there has been no necessity for the enactment of any special statute to protect a community against imposition from the engineer or from one who claims to be a member of our profession, probably because his deeds and works speak for him, while his misdeeds advertise themselves with surprising rapidity.

The engineer is not infallible; he may make mistakes, and it would be remarkable indeed if he did not at some time err in his judgment; but, so long as he is conscientious, exercises every care, and exhausts all his resources in the performance of his obligations, it cannot be said that he is culpable or negligent in his duty. It is necessary, therefore, that he should always be circumspect and vigilant. That the engineer has recognized these obligations toward his profession is evidenced by his standing in the community today, and by the trust and confidence with which every material development has been committed into his hands.

It is not the object to applaud our own work, but to resolve that the engineer shall ever be found worthy of every trust, equal to every emergency, and imbued with the lofty spirit and noble purposes of the profession, with the understanding that it is his mission to study and utilize the elements and forces of Nature in such a way that they will become the agents through which he may contribute his share in the universal upbuilding of a higher civilization.

Though, during the brief period in which engineering has become recognized as a most important factor in this work, an advance has been made greater than all the wisdom of previous ages could conceive, it is probable that not much more than the portal of that temple of fame has been reached wherein the full knowledge of the occult forces and secrets of Nature yet lie concealed.

Enlisted in this glorious service, let the spirit of zeal and noble effort animate every engineer, and the lofty purposes of the profession spur him onward to become a worthy member thereof, bringing to it the ardor and enthusiasm of the artist, the earnestness and perseverance of the student, together with the genius to originate, and thus he will aid in raising to its highest point the standard of professional achievement.

With this thought, mindful of what has been accomplished in the past, and of the great possibilities of the future, I send you my sincerest greetings, and best wishes for an auspicious and memorable convention, and for the success and prosperity of every member of our profession.

MEMOIRS OF DECEASED MEMBERS.

JOHN JAMES ROBERTSON CROES, Past-President, Am. Soc. C. E.*

DIED MARCH 17TH, 1906.

Not many years ago Civil Engineering was unrecognized as a profession, and its few practitioners were not theoretically educated as engineers, but gained their technical education from study, example, and practical experience, much in the same manner as the apprentice learns a trade.

Engineering schools were unknown, and scientific courses in colleges not yet established. In those days, however, there were engineers who cast luster upon the nascent profession—men who, although not so highly educated as specialists, had a much broader general education than the average graduate of the modern schools of engineering, and responded admirably to the exigencies of new and great public works.

John James Robertson Croes was one of the latter class, and one of the brightest of these engineers of the old school. Broadly and thoroughly equipped in general scholarship, he entered the profession at the lowest round of the ladder, and fitted himself by special study and practical experience for a long career of usefulness.

He was the only son of the Rev. Robert Brown and Helen (Robertson) Croes, and was born in Richmond, Virginia, on the 25th day of November, 1834.

At the time of his birth his father was the rector of St. John's Church in Richmond. His paternal grandfather was John Croes, first Bishop of New Jersey, whose parents came from Holland.

His maternal grandfather was James Robertson, a Scotchman, who came to this country at an early age, and became a prominent and highly respected citizen of Philadelphia.

To Bishop Croes may properly be ascribed the successful establishment of Rutgers College, prior to 1825 known as Queens College. This institution, projected in 1767 by the "Coetus" party of the Dutch Reformed Church, held a precarious and languishing existence, in fact, but a nominal one, from 1794 until 1801. During the latter year the Rev. John Croes was called to the rectorate of Christ Church, New Brunswick, and, in addition, was simultaneously invited by the trustees of Queens College to reopen that institution by establishing and taking charge of its Grammar

* Memoir prepared by the following committee: Edgar B. Van Winkle, Rudolph Hering, and Charles Warren Hunt, Members, Am. Soc. C. E.

School. When, after six years, he resigned, the school had become so well founded and prosperous that the trustees determined that, under the old name of Queens College, they would enlarge its scope and reinstitute a complete collegiate course.

Bishop Croes was a veteran of the Revolutionary War, having served three years in the army when a young man, respectively as private, sergeant, and sergeant major, and it is interesting to note that he is the only person rising to the Episcopate in the Protestant Episcopal Church who ever served in the Continental Army.

The boyhood of the subject of this memoir was passed in New Jersey, and at Terre Haute, Indiana, to which latter place his family moved in 1843.

His father—a fine classical scholar—prepared him thoroughly for college, imparting to him in a strong degree his own literary tastes, and laying the foundation for his excellence as a writer.

Mr. Croes in 1850 entered the College of St. James, in Hagerstown, Maryland. On his journey there from Terre Haute, the first seventy miles had to be made, over the National Road to Indianapolis, in a heavy, canvas-covered box wagon, without springs, and drawn by four horses. Indianapolis was at that time the western terminus of the then recently constructed Richmond and Indianapolis Railroad.

He was graduated from the College of St. James in 1853, and at once began the study and practice of civil engineering. This action was much regretted by some of his relatives, who felt that such an "occupation" was unworthy of his fine equipment of intellect and education, which should have been reserved for one of the "three learned professions."

His first employment was in 1854, in New Brunswick, New Jersey, with Israel Smith and Peter Sours upon surveys for the Newark and Bloomfield Railroad, and with the New Jersey Railroad and Transportation Company.

His fine natural abilities and industry were even then so conspicuous that he was selected by the late James P. Kirkwood, Past-President, Am. Soc. C. E., as an assistant on the construction of the first water-works for Brooklyn, New York, and finally placed in charge of the construction of the main reservoir and pumping station at Ridgewood.

In 1860 he took service under another of our Past-Presidents, the late Alfred W. Craven, then Chief Engineer of the Croton Aqueduct Department, of New York City, and was detailed as Principal Assistant to still another of our Past-Presidents, the late General George S. Greene, on the Croton Water-Works Extension, his work being mainly on the construction of the large reservoir and appurtenant works in Central Park, and of the so-called High Bridge Enlargement.

General Greene, being called to service in the War of the Rebellion, was succeeded by the late William L. Dearborn, M. Am. Soc. C. E., who had been Resident Engineer on the High Bridge Enlargement, and, in turn, Mr. Croes was promoted to that position and completed the work.

From 1863 to 1865 he was Principal Assistant Engineer on the Water-Works of Washington, D. C., and had charge of completing its Aqueduct, and the famous Cabin John Bridge, as well as building the dam at Great Falls on the Potomac River. During some five months in 1865 he assisted his old chief, Mr. Kirkwood, in examining and designing improvements for the water supply of both Cincinnati and St. Louis.

In 1866 he was Principal Assistant Engineer to General Greene in designing and constructing the first of the storage reservoirs to be built in connection with the Croton River water supply of New York City.

The dam of this first reservoir, situated at Boyd's Corners, Putnam County, New York, is 78 ft. high and 670 ft. long, and was unique, as far as America was concerned. At that time, nothing on high masonry dams had been published in the English language. About six months after work began, General Greene was called away to assume the position of Commissioner and Chief Engineer of the entire Croton Aqueduct Department, and Mr. Croes was placed in charge of the Boyd's Corners Dam, which he practically completed in 1870, at which time, the Tweed régime having been inaugurated, men of strict integrity and strength of character were an inconvenient factor, and General Greene and Mr. Croes were quickly required to make room for more congenial officials.

In 1872, George H. Norman, M. Am. Soc. C. E., had presented a fund to the Society to provide a gold medal to be awarded annually to the paper which should be judged worthy of special commendation as a contribution to engineering science. The first award of this medal was made to Mr. Croes for his paper descriptive of the Boyd's Corners Dam, entitled "Memoir of the Construction of a Masonry Dam."*

In 1870, upon retiring from the position of Resident Engineer of the Boyd's Corners Dam, Mr. Croes became Principal Assistant to William H. Grant, M. Am. Soc. C. E., in the work of laying out street, sewerage and water systems for that portion of New York City north of 155th Street, as well as certain parts of Westchester County.

In 1872, upon the retirement of Mr. Grant, Mr. Croes was appointed Topographical Engineer of the newly created Department of Public Parks, charged with laying out a system of streets, parks,

* *Transactions*, Am. Soc. C. E., Vol. III, page 337.

etc., for the newly annexed portion of New York City, embracing some 12 300 acres beyond the Harlem River.

The City of New York, at this critical period in its development, was fortunate in securing the services of a man, so able, original, and intelligent, and with the courage of his convictions.

The commissioners who laid out New York in 1811 had adopted a most crude arrangement of streets and avenues, all at right angles, without regard to the topography of the land subdivided or the convenience or rights of the general public. Mr. Croes used the rectangular system of streets where it was economical and convenient, but did not hesitate to replace it by one adapted to the topography, and introduced curved avenues where the grades or the cost of grading could be reduced thereby. From æsthetic consideration, as well as those of utility, he first introduced systems of parkways connecting well-defined centers.

For this departure from the extravagant system of inflexible rectangularity, Mr. Croes was subjected, more or less, to ignorant criticism, and in some cases to the active hostility of land owners who were wedded to the awkward lot unit of 25 by 100 ft.

Being clear as to the correctness of his views, he was not a man to present plans inspired by considerations of temporary expediency. Time has justified his wise foresight, and those portions of the "Annexed District," where his original plans have actually materialized, are more attractive and convenient than any location in the rectangular districts having similar natural topography.

In 1879, Mr. Croes resigned from service in the Department of Public Parks, and thereafter, to the end of his life, devoted himself to developing a private practice, initiated in 1870, in connection with the firm of Croes, Church and Van Winkle. Henceforth, his high professional ability and ripe experience became more and more known; and his work developed largely into that of consultation on matters of hydraulic engineering. His reputation in this capacity was so widespread that he received calls from all parts of the country.

The water supply of Newark, New Jersey, was the subject of an exhaustive examination and report in 1879, and later, for the Lehigh Valley Railroad Company, he conducted extensive investigations which resulted in the execution of a contract for supplying water for Newark, since carried out by another company.

As a notable exception to his consultation practice, he served as Chief Engineer of the Suburban Rapid Transit Company, from its formation until its merger with the Manhattan Company in 1891. Under his supervision some four miles of elevated railroad were built, including the large draw-bridge across the Harlem River at Second Avenue. An unique feature of this road—one strongly advocated by Mr. Croes—was the purchase of a right of way through private property.

In 1889, Mr. Croes made extensive examinations relative to the water supply for Syracuse, New York, and his recommendations and plans, embodied in a valuable report, were adopted and subsequently carried out.

In 1889 he was one of a Board of Experts to report on the Quaker Bridge Dam for the Croton Aqueduct Commission.

In 1899 he prepared for the Comptroller of the City of New York a report on the New York Water Supply; and in 1900 another for the Merchants' Association on the same subject. In these he pointed out the waste of water constantly going on in the city, and urged the necessity for a more general use of water meters.

In 1901 he served as a member of a Board of Consulting Engineers which prepared a report for the Croton Aqueduct Commission on the New Croton Dam and Reservoir.

He prepared, and in several cases carried out, plans for water supply, sewerage, etc., for many small cities, among them those for Lancaster, Pennsylvania, and Lawrenceville, New Jersey.

In 1895, Governor Morton appointed him as one of the Commissioners for the Preservation of the Palisades. He was a most interested enthusiast on this subject, and, although serving without remuneration, threw himself into the work with great energy, sparing neither time nor expense to promote the salvation of this magnificent monument of Nature.

From 1903 to 1905, he served as Consulting Engineer for the New York State Health Department, and in this capacity investigated and reported upon many technical questions.

From 1903 to his death, which occurred just as the work was on the threshold of completion, Mr. Croes was engaged as Engineer of the Construction of the Artificial Lake for Princeton University. At the exercises marking the formal presentation of the lake to the University, President Wilson voiced the sentiment of many when he said:

"I deeply regret the fact that Mr. Croes, the distinguished engineer who devoted himself with so much interest to the execution of this public project, did not live to see its happy completion."

In 1897 a sudden and severe illness prostrated Mr. Croes, and from that time until his death, which occurred on the 17th day of March, 1906, he was never a well man, yet he continued to work indefatigably to the end.

This sketch, which is rather a suggestion than a statement of the work done by Mr. Croes during his fifty years of professional life, would be incomplete without reference to his literary achievements. He wrote easily and well on technical and general subjects. He contributed largely to the editorial pages of *The Sanitary Engineer* (now *The Engineering Record*) during some ten years, and for many years compiled the "History and Statistics of American Water-Works," published by *Engineering News*. He also contributed to *The Railroad Gazette* notable articles on Rapid Transit, reviews of scientific books to the *New York Times*, and prepared many professional papers for the technical societies of which he was a member. He was also the author of a number of articles for "Johnson's Universal Cyclopedia." Among his literary accomplishments was also a poetical ability, which, however, was rarely revealed, and then only to a few intimates.

Upon the reorganization of the American Society of Civil Engineers in 1867, after a dormant period of fifteen years, Mr. Croes was among the first of the younger engineers elected to membership, and throughout the rest of his life he was devoted to its welfare, a conspicuous, enthusiastic, and hard-working member.

Without detracting from the memory of any of the great men who laid the foundation for and started the upbuilding of this Society, it can be truly said that to no one is it more indebted for faithful service and undeviating loyalty than to the subject of this memoir. He served as a Director in 1877, as Treasurer from 1878 to 1887, as Vice-President in 1888, and as President in 1901. As a Past-President he continued to serve as a member of the Board of Direction until his death, and such was his devotion to the Society that, unless restrained by illness, he never failed to attend the meetings of the Board. In 1881, without compensation or assistance, he prepared a catalogue of the railroad section of the Library, consisting of a volume of 188 pages.

Mr. Croes was also a Member of the Institution of Civil Engineers of Great Britain, of the American Public Health Association, and the American Water Works and the New England Water Works Associations.

He was by nature a man of essentially social tendencies, the indulgence of which was limited and constrained by the exigencies of an unusually active professional life. Of fine intellectual gifts, broad education, acquaintance with literature, varied experience with many classes of men, united with a retentive memory and keen sense of humor, he was always an attractive talker.

These qualities, combined with buoyancy of spirits, made him as a young man a delightful comrade, and when ripened with years a most interesting and instructive companion. Enthusiasm in regard to whatever matter he took up, either of professional work, recreation, or controversy, was a dominant feature in his character, and made him a potent advocate and natural leader. He was a good linguist and a discriminating lover of music and art. His Scotch and Dutch ancestry made him a man of positive convictions, and tenacious of his views when once carefully established. When led into controversy, he was a formidable opponent, while, as a friend, he was most steadfast and loyal.

A well-founded confidence in his own mental equipment made him largely free from self-consciousness, and he was a good expert witness, the subject in hand monopolizing his thoughts without embarrassment as to the impression, favorable or unfavorable, he might be making.

He joined the Union League Club in 1869, and, later, naturally gravitated to the congenial intellectual atmosphere of The Century Association, to which club he was elected in 1884, and where he was ever a welcomed and valued member. At the time of his death he was serving as a member of its Committee on Admissions.

Of all the phases of Mr. Croes' life, the one least known to the world at large, but one that was finest, as seen by his intimates, was his domestic life. His spirit was true altruism. He never married, and his devotion to his family can only be described as beautiful. Nothing he did for them was ever felt as a sacrifice. His happiness was only reached through their happiness. Such affection was warmly reciprocated, and his home was made for him a place of charm as well as rest.

In analyzing the life and character of Mr. Croes, two elements are always distinctly traced: strength and intellect. As a young man, he was strong physically, but as conditions changed with age, this attribute greatly lessened, but his mentality seemed to gain more than his physical strength lost.

His power of mind and body was shown in his remarkable capacity for concentration and the accomplishment of work. His reports were always based on the most extensive statistics and sets of observations available, all elaborated and collated with unlimited labor, so that the conclusions reached were clear and logical, and hence convincing. The rapidity of his work was due in a large measure to his power of concentration, to system, and to his habit of economizing time. As an instance of this habit, the paper for which he received the Norman Medal was largely written on scraps of paper while waiting for railroad trains.

He was honest—as befitted a strong man—and no considerations of policy or fear of consequences ever made him swerve in demanding honest work from contractors. His life, in this respect, could be held up to our younger engineers as an example.

His strength made him positive in his convictions, and strenuous in maintaining them, and sometimes led to strong antagonisms. Although at times an active partisan, he was always an honest antagonist, fighting openly and never striking in the back.

In his business dealings it may be said of him as was said of another distinguished member of this Society: "He was not only straight from a business point of view, but he was morally straight." With his strength of character, self-respect and rugged honesty, he could never become a courtier, and the flattery he detested to receive he never thought to apply to others, not even in the form of studied deference which at times might have gained for him professional advantage.

His mind was so receptive that full advantage was easily taken of all opportunities for the acquisition of knowledge, which made him unusually conspicuous as a general scholar among professional colleagues. He kept in touch with the advancement of science and all questions of the day, and could intelligently appreciate and enjoy intercourse with men versed in the many forms of learning.

This, in brief, is the record of an eminent engineer, who for so many years has been a familiar figure at gatherings of the American Society of Civil Engineers, who held positions of trust in its management, who unstintingly worked for its welfare, who unselfishly helped to guide its policy from weak infancy to maturity and world-wide influence, and who loved it beyond all else save his family.

FREEMAN CLARKE COFFIN, M. Am. Soc. C. E.*

DIED NOVEMBER 11TH, 1906.

Freeman Clarke Coffin was born in Boston, on September 14th, 1856. He was the son of Alonzo King and Mary E. Coffin. His early life was spent in Patten, Maine, where he attended the public schools up to the age of fourteen years, when the support of the family fell upon his shoulders, and it became necessary for him to go into business. For a year or two he served in a general store, and then began the manufacture of furniture in a small way. In the next ten years he built up the largest establishment of its kind in the town.

Dissatisfied with the outlook for the future, he moved to Boston, at the age of twenty-six, entered the works of the Coffin Valve Company, and two years later the office of the late M. M. Tidd, M. Am. Soc. C. E., one of the leading civil engineers of New England, who was devoting himself largely to water-works construction. There Mr. Coffin began his technical studies, and laid the foundation for his future success.

In 1894 Mr. Coffin opened his own office in Boston, and in 1905 took into partnership his principal assistant, Mr. Lewis D. Thorpe.

Notwithstanding the character of his early training, and the fact that he did not take up engineering work until middle life, by dint of hard work and constant application, Mr. Coffin, in the course of a comparatively short professional career, made an enviable name for himself among the foremost engineers of New England. His professional work was largely along the lines of water-works and sewerage construction, though he was often consulted and had testified many times in water-works valuation and water diversion cases in different parts of the country.

Mr. Coffin was a Member of the Boston Society of Civil Engineers, the Canadian Society of Civil Engineers, the New England Water Works Association, and the Twentieth Century Club, of Boston. At the time of his death, he was a Vice-President of the Boston Society of Civil Engineers, and Chairman of the Sanitary Section of that Society (for the formation of which Section he was largely responsible), and had been nominated as a Vice-President of the New England Water Works Association.

Mr. Coffin's keen interest in the technical side of his profession, and in the work of the engineering societies, is attested by the admirable papers and discussions which came from his pen. Among

* Memoir prepared by Leonard Metcalf, M. Am. Soc. C. E., and William S. Johnson, Assoc. M. Am. Soc. C. E.

these might be mentioned his discussions before this Society on papers upon "The Financial Management of Water Works" and "The Valuation of Water-Works Property," and his papers before the New England Water Works Association upon "The Financial Management of Water-Works;" "Standpipes and Their Design;" "Friction in Several Pumping Mains;" "Corrosion of Pipes;" "Application of Gas, Gasoline and Oil Engines to Pumping Machinery," and "Covered Reservoirs and Their Design."

One of the most notable of his services to the engineering profession grew out of his paper presented before the New England Water Works Association, containing "A Few Notes on Cast-Iron Pipe," which led to his appointment as Chairman of the Committee of that Society which drafted the "Standard Specifications for Cast-Iron Pipe and Special Castings." These specifications have been substantially adopted by the American Water Works Association, and are now coming into general use. Mr. Coffin also published a handbook entitled "Graphical Solution of Hydraulic Problems" which has won favorable comment, and has passed through two editions.

He died at his home in West Medford, on November 11th, 1906, leaving a wife and four sons.

While the demands of his professional practice prevented Mr. Coffin from attending the meetings of this Society to any great extent, he took a very active interest in the affairs of the local societies, and was widely and very favorably known professionally throughout New England, and in the adjoining States and Provinces. Studious in mind, honest in conviction, and courteous in manner, Mr. Coffin won for himself the respect and confidence of all with whom he came in contact. In his death the profession loses an able and clear-headed thinker; the community, an honest and public-spirited man.

Mr. Coffin was elected a Member of the American Society of Civil Engineers on February 6th, 1895.

CHARLES HUMPHREYS, M. Am. Soc. C. E.*

DIED NOVEMBER 18TH, 1906.

Charles Humphreys was born on May 2d, 1853, in Lowell, Massachusetts, where his father, Joshua, though still in the Navy of the United States, but on leave of absence, was in charge of the Middlesex Mills of that town. His mother was Miss Margaret Chandler, of Georgetown, D. C. His father entered the United States Navy as a Midshipman in 1829, being third in his class. In the War with Mexico he served with credit in the fleet before Vera Cruz, and did other valuable duty, on surveys, etc. He resigned from the Navy in 1853, and was engaged in civil pursuits until the War between the States began. He then entered the service of the Southern Confederacy. Joshua Humphreys died in Fredericksburg, Virginia, in November, 1873, at the age of sixty.

The ancestors of Mr. Charles Humphreys came from Wales as early as 1682 and settled at Haverford, Clinton County, now Delaware County, Pennsylvania, near Philadelphia. His great grandfather, Joshua, was in the military service at the battle of Trenton, and thereafter until the close of the War of the Revolution. He was a member of the Philosophical Society of Philadelphia. This Joshua was one of a group of famous builders of that period; and, in 1792, when the navy had become a matter of general interest, he ventured to send letters, giving his views of the needs of the situation, to Robert Morris and to General Knox, then Secretary of War and Navy. In these suggestions are already to be discerned the principal elements in American success in ship building—originality and adaptation to our own circumstances. Humphreys claimed that as the number of our vessels would necessarily be inferior to that of the European navies, ours must be light enough and fast-sailing enough either to decline a battle or accept it at will; and yet, if our vessels should enter a battle, they must be equal in fighting abilities, vessel for vessel, to any vessel of any size. He proposed, therefore, to make the vessels larger and longer, and yet lower, than the existing type. The same number of guns could then be placed on one deck as had previously been placed on two, and, either in heavy winds or in light, they could outsail existing European ships, could easily overcome those of the same rate, and meet on equal terms those of a higher rate. His suggestions were accepted, and when Congress ordered the building of six ships of war, Humphreys was instructed to prepare the models for them. He did so, and the six vessels, *Chesapeake, Constellation, Constitution, President, Congress,* and *United States,* were built at various

* Memoir prepared by William P. Craighill. Past-President. Am. Soc. C. E.

places on his model, the last in his own shipyard at Southwark, Philadelphia. The ship-of-the-line, *Philadelphia,* launched in 1799, was also built by Humphreys at his own yard. The work done by these vessels fully justified the ideas of their builder. Their model exerted a very great influence, not only on our own subsequent navy, but on that of England and other countries as well.

Charles, the brother of Joshua, was an active and influential member of the Continental Congress from 1774 to 1776.

Samuel Humphreys, the son of Joshua, was brought up in the shipyard, taking charge of the business in 1808, being appointed constructor for the Philadelphia Navy Yard in 1813, and Chief Naval Constructor of the United States in 1826, a position which he held until his death, in 1846, in Georgetown, D. C. He was the builder of the steam frigate, *Mississippi,* Perry's flagship in the Mexican War and in his expedition to Japan. Great pecuniary and other inducements were offered to him to enter the service of Russia, but he preferred to devote his time and talents to his native land.

Major General Andrew A. Humphreys, uncle of the subject of this sketch, after being graduated at West Point, served in the Indian War in Florida, and wón great distinction as a member of the Corps of Topographical Engineers of the United States Army, especially for his studies and writings on the "Hydraulics of the Mississippi River," and the surveys for the locations of the trans-continental railways. In the War between the States he was gallant and prominent as Chief of Staff and as a Corps Commander in the Army of the Potomac. After that War he became Chief of the Corps of Engineers of the Army.

The career of Charles Humphreys was cut short by death in Washington, D. C., on November 18th, 1906.. His health had failed for several years. His disease was incurable by medicine, and an operation would have been fatal. His last illness was tedious, but he was comforted by the devoted attentions of his wife and children, and bore his suffering with the fortitude of a brave man and the patience and resignation of a sincere Christian.

He entered the service of the United States at the age of seventeen, in a subordinate position, and did not have the advantage of a technical, collegiate education; but his natural ability, his strict and faithful attention to every detail of duty, with thorough study of every subject in the profession which came under his care, and a wide and varied experience, made him one of the most capable, trusted and successful civil engineers. He was engaged for thirty-seven years upon the improvement of rivers and harbors and the construction of the fortifications of the country. As a surveyor and draftsman, he was specially expert and careful. He inspected the quarrying, cutting and erection of the beautiful Centennial Monument at Yorktown, Virginia. He was engaged for a number of years on the movable dams of the Great Kanawha and Ohio Rivers; on the improvement by jetties of the bar at the entrance of Winyaw Bay, South Carolina; on the improvement by dredging of the bar at the mouth of Cape Fear River, North Carolina; of the Patapsco River in Maryland; of the harbor of Norfolk, Virginia; and was connected with many other improvements of less importance.

He was held in the highest esteem by all his official superiors, by his comrades, and by those who served under him.

The following extract from a letter of an officer of the United States Corps of Engineers is but a sample of many that have been received:

"In all the relations of life in which I knew or observed him, in his family, among his associates, and in his official duties, he was a man of the most sterling character and qualities. As Assistant Engineer, he was thorough and efficient in the office, in the field, in important surveys, and upon works in the immediate direction and management of men.

"As an engineer, his observation was close and intelligent, and his judgment sound.

"He was utterly unsparing of himself, in his industry and application in the office, and in exposure of himself in the field. Such was his simplicity of character, modesty, and forgetfulness of self, that he was little adapted to obtaining, of himself, what his merits deserved.

"He was a vestryman and warden in a struggling church at Wilmington, North Carolina, and, in his simple, retiring way, was of excellent influence in his community.

"I lament his death, and shall ever bear him in affectionate remembrance."

The following short extracts are from a letter of an associate who knew him very intimately:

"Charley was a man clear through—one of the best kind—a practical, well-informed, affectionate fellow, with no frills, and one who could be relied upon to an unlimited extent. His sympathies were excited by anyone in the least distress, real or imaginary, and it came as natural to him to give comfort as it is for water to run; he could not help but do it. He learned his profession in a practical way while working in the field or at the drafting board, with books as his companions in both places. He was very studious of technical books of the profession he loved, and was assisted in his progress by his aptness with mechanical tools, of which he purchased many and used them in making models of bridges and other engineering structures. In these models he delighted in demonstrating that the structures were symmetrical and the different parts nicely adjusted to the relative strains. In this thorough, practical way he learned his profession; and the way, in my opinion, illustrated the manliness of the man.

"Officially, Charley was unostentatious, but never undignified. His manner was plain and kind, but there was a something, indescribable but recognizable, about him that repelled all presuming upon it to take advantage of his good nature. I always thought him an ideal man and official—not brilliant or dashing, but faithful to the point of utter reliability in every direction."

Many other letters of similar tenor could be added, and the present writer, who knew Mr. Humphreys, as boy and man, for nearly forty years, can heartily concur in all that has been said by others, and wishes to add his own testimony to the purity and excellence of his character as a man, and his ability and unusual merit as an official.

Charles Humphreys married Miss Elizabeth C. Hungerford, a member of an old and prominent family of Maryland. He left three children, two daughters and a son, the last taking up the profession of his father, after being graduated at the Rensselaer Polytechnic Institute at Troy, New York.

Mr. Humphreys was elected a Member of the American Society of Civil Engineers on February 1st, 1905.

WALLACE CLYDE JOHNSON, M. Am. Soc. C. E.

Died December 15th, 1906.

Wallace Clyde Johnson was born in Granville, Massachusetts, on May 21st, 1859. He was a son of James W. and Frances A. Johnson. Mr. Johnson was educated in the public schools of his native town, and in the years 1880-82 attended Williams College. In 1884 he was graduated from the Worcester Polytechnic Institute, and in the same year he accepted a position with the Holyoke Water Power Company, of Holyoke, Massachusetts, as Assistant Engineer. While occuping this position, his ability, energy and engineering skill commanded attention. When Clemens Herschel, M. Am. Soc. C. E., of Holyoke, was requested to recommend an engineer to take charge of the development of the Niagara Falls Hydraulic Power and Manufacturing Company, at Niagara Falls, New York, Wallace C. Johnson was the man recommended.

This was in 1886, and it requires but a casual review of the hydro-electric conditions in the United States to understand fully what this recommendation meant. The period since then has been one of truly wonderful accomplishments in the hydro-electric development of the world.

Mr. Johnson remained Chief Engineer of the Niagara Falls Hydraulic Power and Manufacturing Company until 1900, at which time he became the Consulting Engineer, which position he held until his death. While he was Chief Engineer of that company he designed and had executed, under his supervision, the hydro-electric development of this Company on the banks of the Lower Niagara River, this being the pioneer hydro-electric development of the country, using turbines under a head of more than 200 ft. This plant, as well as many others which he designed, will stand as monuments to his memory.

In 1900 Mr. Johnson accepted the position of Chief Engineer of the Shawinigan Water and Power Company. He designed their plant at Shawinigan Falls, Quebec, Canada, which is the largest water-power development in the Dominion. The water-wheels used in this plant were, at the time of their installation, the largest ever built, and the transmission line from Shawinigan Falls to Montreal was the longest east of the Rocky Mountains.

On June 11th, 1906, Governor Higgins appointed him a member of the State Water Supply Commission, and the broad field of the Commission was particularly interesting to a man of his attainments.

Mr. Johnson's advice was continually sought by companies contemplating water-power development and by those desiring to remodel or enlarge plants already built. He made examinations and reports on water-power propositions covering the United States from Maine to California, throughout the Dominion of Canada, and in Nicaragua.

He held the position of Chief and Consulting Engineer of the Niagara Falls Hydraulic Power and Manufacturing Company, at Niagara Falls, New York; Chief and Consulting Engineer of the Shawinigan Water and Power Company, at Shawinigan Falls, Quebec; Chief and Consulting Engineer of the Bodwell Water Power Company, at Old Town, Maine; Chief and Consulting Engineer of the Hannawa Falls Water Power Company, St. Lawrence County, New York; Chief Engineer of the Empire State Power Company, Chief Engineer of the Chicoutimi Power Company, Quebec; and Chief Engineer and General Manager of the Albion Power Company, Albion, New York.

Mr. Johnson's genial disposition and cordial manner made him a great favorite. He was a member of the American Society of Mechanical Engineers, and of the Engineers Society of Western New York, of which society he was Past-President. He was also a Member of the Canadian Society of Civil Engineers; and an Associate Member of the American Institute of Electrical Engineers; and of the Society of Arts, London, England. He was a member of the St. James Club, of Montreal; the University Club, of Buffalo; and the Tarratine Club, of Bangor, Maine.

On May 31st, 1893, he was married to Miss Eloise Gertrude Murlless, of Holyoke, Massachusetts, who survives him.

Mr. Johnson died at the comparatively early age of forty-seven years, but the work which he accomplished bears testimony to the active life he led. He was constantly and effectively in the field of work, and his life was devoted to his profession. His services were highly valued by the various companies with which he was connected.

Mr. Johnson was elected a Member of the American Society of Civil Engineers on October 5th, 1892.

FELICIAN SLATAPER, M. Am. Soc. C. E.*

DIED SEPTEMBER 11TH, 1906.

Felician Slataper was born in Trieste, Austria, on April 19th, 1828. After his early schooling, he entered the Austrian Naval Academy at Venice in 1841. Leaving this academy in 1844, he pursued his studies until 1848 in the Polytechnic College at Vienna. In 1850, he determined to seek his fortune in a new land, and sailed from Trieste to San Francisco, and from there to the Isthmus of Panama, where he was employed on the rebuilding of the roads across the Isthmus. Here he was stricken with fever, and, while convalescing, set sail for New York, arriving early in 1851.

Owing to his limited knowledge of English, Mr. Slataper found some difficulty in procuring permanent employment in New York; and, in the spring of 1852, he made his way to Pittsburg, and secured employment with the Pennsylvania Railroad as Assistant Architectural Draftsman, under Oliver W. Barnes, M. Am. Soc. C. E., who was at that time the Principal Assistant Engineer in charge of the construction of the Western Division of the Pennsylvania Railroad, and of the Machine Shops and Round House in Pittsburg. Mr. Slataper was engaged chiefly upon the tracings of the plans of these buildings. He remained in this employment a year or more, and then went to the Pittsburg and Connellsville Railroad, serving under Mr. Barnes, who had left the service of the Pennsylvania Railroad Company in the spring of 1853.

Later, Mr. Slataper moved to Washington, D. C., and took a position in the Government service. In 1861, he went to the Downingtown and Waynesburg Branch of the Pennsylvania Railroad, then being constructed by Mr. Barnes as Chief Engineer, and made a topographical survey and map of the line and the adjoining country.

In 1863 Mr. Slataper took service with the Pittsburg, Fort Wayne and Chicago Railway Company, as Division Engineer, continuing in that service until 1869 when that road was leased by the Pennsylvania Railroad Company. He continued with this company, and, in 1870, entered the service of the Pennsylvania Company, organized to operate the Pittsburg, Fort Wayne and Chicago Railway and other lines west of Pittsburg owned or controlled by the Pennsylvania Railroad Company, remaining with that company for 25 years. In March, 1871, he was appointed Chief Engineer, and retained that position until 1889, when he was made Consulting Engineer, which position he finally gave up

*Memoir prepared by Thomas Rodd, M. Am. Soc. C. E.

in May, 1895, having then been on leave of absence since September, 1892, residing in Trieste, his birthplace.

During the years 1863 to 1867 inclusive, Mr. Slataper was identified especially with the design and construction of the Allegheny River Bridge of the Pittsburg, Fort Wayne and Chicago Railway, making nearly all the drawings and superintending its construction, while holding the title of Engineer and Architect under the eminent engineer, the late John B. Jervis, Hon. M. Am. Soc. C. E., then Chief Engineer. This remarkable structure, a double-track, through, lattice bridge with three trusses, endured, until 1903, under the heavy traffic of the railway. In 1893, the enormous 500 000-lb. gun, exhibited by the United States Government at the World's Fair in Chicago, was moved over this structure on its heavy steel special cars, the bridge having no supports but its own stone piers, and being practically in its original condition, save for some unavoidable deterioration. This bridge had a total length of 1 172 ft., composed of five spans of 164 ft. each, and four spans of deck plate-girders, each 88 ft. long. It will be recognized by engineers that this life of 35 years, during that period of heavy traffic and increasing loads, is sufficient testimony to the ability and painstaking care of the man to whom is clearly due the credit of the design and execution of this great work.

On September 23d, 1856, Mr. Slataper was made a citizen of the United States in the United States District Court at Pittsburg, Pennsylvania.

In 1870 the City Councils of Pittsburg elected three Water Commissioners, serving without compensation. They employed hydraulic engineers and made a report, after a thorough investigation, on the problem of water supply for the city. Mr. Slataper was the first man selected, and the only one who had a unanimous vote. Messrs. J. K. Moorhead and George A. Berry were the other members of this commission. Mr. Slataper was very active and efficient in the work of the commission and in the selection of the hydraulic engineers, who were the late E. S. Chesbrough, M. Am. Soc. C. E., and the late Moses Lane, M. Am. Soc. C. E. The report of the commission was duly made, and with modifications and extensions, was carried out, and the city water supply of Pittsburg as it is to-day is the result of that commission. Besides this work, Mr. Slataper was frequently employed in an advisory capacity, as well as in designing works for other railroads.

In 1876 Mr. Slataper was one of the judges in deciding the awards of prizes on railway exhibits at the Centennial Exhibition in Philadelphia.

During the eighteen years of his incumbency as Chief Engineer of the Pennsylvania Company, several new railroads were located and constructed by him, and, owing to difficulties of construction and controversies with other lines seeking to occupy the same ground, some of them were built under very adverse conditions. In all the work of reconstructing the Fort Wayne Railroad, he displayed great resourcefulness and ability, and he always required from others the same diligent and persistent effort which he himself gave to all the work in his charge. While his instincts were naturally kind and his manner courteous, he held every one to a strict accountability, and was content with nothing but the best service from all his subordinates. His unswerving loyalty to duty, and his high ideal of integrity earned for him the affection and respect of his associates, while his pleasant manner and his willingness to adapt himself to his company and surroundings made him a welcome companion in all the circles of his business and social life.

On August 19th, 1853, Mr. Slataper married Eliza Jane Lee, the daughter of Dr. Daniel Lee. The surviving children of this marriage are Daniel Lee Slataper, of Alvin, Texas, and Mrs. S. W. Kerr, of Philadelphia. Mrs. Slataper died in October, 1892. In November, 1893, he married again. His widow and her daughter, Ada, 10 years old, survive him, residing in their beautiful home in Trieste.

During his long and active career, his naturally great care and thought for all his affairs led him to exercise his good judgment in both saving money and investing it carefully, so that at the time of his death he was fairly well provided with this world's goods, of which he made a just and carefully-thought-out division between his surviving heirs on both sides of the Atlantic.

Mr. Slataper was elected a Member of the American Society of Civil Engineers on September 15th, 1869, and while he sometimes attended the Conventions of the Society, his very busy life, as well as his inclinations to follow his duties very closely, left him little or no opportunity to make any contributions to the literature of the Society, nor did his tastes or his inclinations lead him in this direction. He was, however, always faithful, competent, and diligent, gaining the respect and esteem of his friends and associates, rendering devoted service to his employers, and leading such an exemplary life as to be, in all respects, in his family, social, and business life, a worthy example to those who had the privilege of working under or being associated with him.

The writer visited Mr. Slataper in March, 1906, at Trieste, and found him apparently in very good health, tall, with a full head of hair and full beard, both perfectly white. He was slightly bowed at the shoulders, but was active in body and mind, and seemed in a better situation to enjoy the true felicities of life than ever before. But this was his last good time, and thenceforward he declined in health and strength until the time of his death, on September 11th, 1906, in his seventy-ninth year. During all this time he had the affectionate care and devotion of his family, being surrounded by every possible comfort and solaced in his last days by the memories of a well-spent life. His many friends in America will appreciate these particulars.

GEORGE TATNALL, M. Am. Soc. C. E.*

Died September 13th, 1906.

George Tatnall was born in Wilmington, Delaware, on December 27th, 1853. His father was Edward Tatnall, and his mother, Rachel Richards Webb; his grandparents were Edward Tatnall and Margery Paxson, James Webb and Lydia Pritchett Richards. He came of good old Quaker stock, his ancestors on both sides having been members of the Society of Friends, in England, before coming to America.

The first Tatnall came to Wilmington from Leicestershire, England, in 1725. The early Webbs, Pritchetts, Lamborns, and Richards came from England to Chester County, Pennsylvania, at various times between 1685 and 1700, the immediate ancestors of Mr. Tatnall subsequently moving to Wilmington. His grandfather, James Webb, started the first morocco tannery, and this has since become the leading industry in Wilmington. He was a Member of the City Council, and of the Board of Education, and was an ardent anti-slavery advocate.

Mr. Tatnall received his early education at the Friends School, Wilmington, and was fitted for college at T. Clarkson Taylor's Academy. In 1870-71 he was a student in the engineering course, Cornell University. Leaving college at the end of this session, he became a Rodman on the construction of the Wilmington and Western Railroad. During 1872, and part of 1873, he was Assistant Division Engineer for ten months under J. Dutton Steel on the Berks County Railroad, and then Division Engineer on the same road from Maiden Creek to Lenhart.

In September, 1873, he returned to Cornell University and in June, 1875, took the degree of B.C.E., completing the four years' course in three years. On leaving the University he obtained a position with Cofrode and Taylor, of Philadelphia, remaining a year with them, bridge drafting and designing some of the minor buildings of the Philadelphia Centennial Exposition. For the next year he was engaged at the Edgemoor Bridge Works and in private practice. From 1877 he was Topographer on Location and Assistant Engineer on Construction on the Pittsburg and Lake Erie Railroad, on the Division of the Ohio River Bridge at Beaver, and the terminal under the Pittsburg, Fort Wayne and Chicago Railroad, at Beaver Falls, Pennsylvania. On the completion of this work he went to Mexico as Principal Assistant Engineer for an American syndicate to locate and construct the Vera Cruz, Anton Lizards and Alvarado Railroad. After a few months the American

* Memoir prepared by William A. Pratt, M. Am. Soc. C. E.

company withdrew from the work, and he remained as Consulting Engineer with the Mexican company until December, 1879. After engaging in private practice for several months, he was, in September, 1880, appointed Assistant Engineer of the Philadelphia, Wilmington and Baltimore Railroad, and, when the Pennsylvania Railroad acquired this property, he remained in the service of this road until 1904, a record of 24 years, during which time he was engaged in designing and erecting wooden and iron highway and railroad bridges, in preliminary and location surveys, and in the construction of branch roads, of third and fourth track, yards and sidings, freight houses, grain elevators, cold-storage houses, and other heavy buildings, notably the Quaker City Cold Storage Building, and the Philadelphia Market House.

In 1904 he left the Pennsylvania Railroad to engage in consulting practice in Philadelphia, and in 1905 was appointed Principal Assistant Engineer of the New York, Westchester and Boston Railway, locating and building a high-speed four-track electric railway in the Borough of the Bronx, New York City, and in Westchester County, New York, which position he held at the time of his death.

On September 12th, 1900, he was married to Catherin Jewett Coolidge, of Wayland, Massachusetts, daughter of James Coolidge and Henrietta A. Saunders, both of Salem, Massachusetts. His wife and two sons and a daughter survive him.

In 1905 he became interested in a new explosive, called Ferronite, invented by Franz Wartenberger, with which the inventor had been allowed to experiment on the railway with which Mr. Tatnall was connected. Mr. Tatnall, while visiting his wife's family, at Wayland, Massachusetts, was induced by Wartenberger to go to Lynn, on September 7th, to witness a demonstration he was to give some contractors at a quarry at that place. While Wartenberger was loading a hole, Tatnall thought he was doing it in a careless manner, and started to remonstrate, when a premature explosion occurred, terribly injuring both. His first thought, in spite of his own suffering, was for others, and, calling to the spectators to stand back, he tried to extinguish the burning clothes of his companion, and, on the arrival of the ambulance, insisted that Wartenberger should be attended to first. They were both hurried to the Lynn Hospital, but Wartenberger died soon after being admitted.

Mr. Tatnall displayed wonderful vitality, recovering from the shock, and at times giving hope that he might recover. He lingered five days, dying of hemorrhage from a ruptured artery at 6 A. M. on September 13th, 1906. . His wife and two sisters were able to reach him and attend his last hours.

Mr. Tatnall possessed all the sterling qualities of his Quaker ancestors; he was of high personal character, genial with his associates, and devoted to his family.

He was an engineer of ability, painstaking and thorough in his work, and always considerate of others, and leaves a host of friends who deeply mourn his untimely death.

He was elected a member of the American Society of Civil Engineers on May 3d, 1899; and was also a member of the Transportation Club, to which he had contributed several papers.

ARTHUR PRICE LAW, Assoc. M. Am. Soc. C. E.*

Died November 19th, 1906.

Arthur Price Law, Superintendent of Construction for the Holbrook, Cabot and Rollins Corporation, died at his home in East Providence, Rhode Island, on Monday, November 19th, 1906, of typhoid fever.

Mr. Law was born in New York City on July 7th, 1870, and attended the public schools and high school in Flushing, Long Island, and was a graduate from the full civil engineering course of the International Correspondence Schools, of Scranton, Pennsylvania.

He worked as assistant on Government surveys in Florida, and in 1893 was engaged by the New York, New Haven and Hartford Railroad Company, on heavy construction, making a specialty of bridge foundations. He stayed with this company as Junior and Senior Assistant Engineer until 1900, when he entered the service of Holbrook, Calbot and Rollins, as Superintendent of Construction, in charge of deep foundations and substructure work, and was in their employ at the time of his death.

During the seven years of his connection with the Holbrook, Cabot and Rollins Corporation he had charge for them of the foundations for a double-track railroad bridge at Tiverton, Rhode Island, and all the work at Bridgeport, Connecticut, done by this corporation. He started the work on the Housatonic River foundations, and then went to the Connecticut River and completed the work on the deep foundations there. Later, he took up the matter of the foundations for the work at Neponset, Massachusetts, and also for the bridge across the Seekonk River at Providence, Rhode Island. All this work was done for the New York, New Haven and Hartford Railroad Company.

His work was carried through with marked ability and success, and without any trouble or loss either to the Railroad Company or the Holbrook, Cabot and Rollins Corporation, and was done to the utmost satisfaction of both these corporations.

Mr. Law belonged to the Red Men, and was elected an Associate Member of the American Society of Civil Engineers on June 3d, 1903.

* Memoir prepared by J. W. Rollins, Jr., M. Am. Soc. C. E.

HENRY BRIGHAM LOOKER, Assoc. M. Am. Soc. C. E.*

DIED JANUARY 3D, 1905.

Henry Brigham Looker was born in Cincinnati, Ohio, on April 10th, 1858. He was the eldest son of former Paymaster-General Thomas H. Looker, United States Navy, and was of a family which had been for several generations represented in the military and naval forces of the United States, from the time of Othniel Looker, who served as private in Colonel Martin's Regiment of the Jersey Line in 1776, and in other Revolutionary organizations until 1782. Aside, however, from three years' service as a cadet at the United States Military Academy and six months' service as Captain of Volunteers during the Spanish War, Major Looker devoted himself to civil engineering and surveying until his untimely death, which was undoubtedly hastened by the hardships of this short military service.

Major Looker was educated in part at the Maryland Agricultural College, in part at the United States Military Academy at West Point, and in part by private tutors. Leaving West Point in 1881, he began active engineering practice with a year's service in a subordinate capacity on the surveys for the Delaware and Chesapeake Ship Canal, and for the Lockport Tunnel, and Relocation Work for the Lehigh Valley Railroad. From September, 1882, until June, 1887, he was engaged in work of instruction in mathematics, surveying, military engineering, and tactics at the Betts Military Academy at Stamford, Connecticut, and at De Veaux College, Suspension Bridge, New York. In June, 1887, he entered the office of General Herman K. Viele, Civil and Topographical Engineer, Washington, D. C., as Principal Assistant in charge of field and office work relating to surveys, landscape engineering, electric railroads, subdividing and grading suburban land, and general municipal practice. In July, 1890, he opened an office for himself as a civil engineer, working along these and similar lines, and during the five succeeding years, in work of this nature covering the entire District of Columbia, established himself as one of the leading engineers in private practice at the Capital. In March, 1895, at the instance of the leading business men of Washington, he was appointed Assistant Surveyor of the District of Columbia, and in August, 1897, he succeeded Mr. William Forsyth as Surveyor, which office he held until his death, except as noted below.

At the outbreak of the Spanish War he volunteered for service,

* Memoir prepared by Jay J. Morrow, M. Am. Soc. C. E., Captain, Corps of Engineers, U. S. Army.

and was appointed Major in the Brigade of the National Guard of the District of Columbia. As the quota of the United States Volunteer troops from the District was reduced from a brigade to a regiment, he accepted the appointment as Captain of Company H, which was detached to serve as engineers in General Miles' expedition to Porto Rico. He resigned his office of Surveyor of the District of Columbia, to accept this commission, on July 7th, 1898. The company rendered important service at the landing near Guanica, and on July 25th, 1898, captured the first Spanish flag taken in Porto Rico. The company constructed several military bridges in Porto Rico, and, as part of a provisional battalion of engineers, built the road leading to Fort Capron. Major Looker was temporarily appointed Military Governor over one of the military districts of Porto Rico, and served in this quasi-civil capacity with credit. He received honorable mention in reports to the War Department, and his name was included in the list of nominations by the President for brevets, all of which failed of confirmation by the Senate. At the close of the war he was presented with a handsome sword, by the men of his command, as a testimonial of their affection and esteem.

After being honorably discharged from the military service of the United States, Major Looker was reappointed Surveyor of the District of Columbia on November 22d, 1898, which office he held until his untimely death, January 3d, 1905. During the last year of his life it became evident that the exposures incident to the tropical campaign had affected his health, but so sudden a termination was not anticipated, his death coming suddenly and resulting from congestion of the brain. Upon the announcement of his death, his valuable public services, his sterling integrity of character, and the noble Christian virtues practiced by him in life, were recognized publicly by the Commissioners of the District of Columbia through the issuance of the resolutions herewith quoted:

"*Whereas,* The Commissioners of the District of Columbia have learned with deep regret of the death of Henry B. Looker, late Surveyor of the District of Columbia, who died at 3:20 A. M., January 3d, 1905;

"*Resolved:* That the Commissioners of the District of Columbia hereby testify their appreciation of the services rendered by the deceased during his incumbency of the office of Surveyor of the District of Columbia; which extended from August 18th, 1897, to the day of his death without interruption except between July 7th, 1898, and November 22d, 1898, during which time Mr. Looker commanded a company which took part in the Spanish-American War. Mr. Looker was a man of sterling integrity, and eminently qualified to hold the responsible position which he occupied, and his death is a loss to the District of Columbia.

"*Resolved:* That a copy of this resolution be entered on the minutes of the Board of Commissioners, a copy be sent to the relatives of the deceased, and copies be furnished the daily press."

Major Looker was married in 1893 to Miss Catherine Earle, and is survived by her and by one son.

He was elected an Associate Member of the American Society of Civil Engineers on May 3d, 1893.

EDDY ELBERT YOUNG, Assoc. M. Am. Soc. C. E.*

Died June 1st, 1906.

Eddy Elbert Young was born at Wapun, Wisconsin, on January 4th, 1865. His parents were Enoch and Cordelia Young. When he was two years old his parents removed to Lowell, Massachusetts, in which city his boyhood was passed.

He commenced work at the early age of sixteen, having then gone through only part of the course at the Lowell High School. After leaving school, however, he continued his studies, taking up mechanical and architectural drawing in evening classes for several years. Later in life he often expressed regret at leaving school so soon, although his associates considered this regret as needless, for, by his private study and his close and accurate observation, he became in many respects a man of culture as well as an advancing engineer. At the time of his early death he had done much good work, and the indications were that he would do even better.

The following is a brief summary of his professional career, but it is incomplete in that it gives no indication of the generally excellent character of his work.

In the spring and summer of 1881 Mr. Young was a Draftsman and Rodman for the Merrimack Manufacturing Company, on mill engineering. From May, 1883, to April, 1884, he was a Draftsman and Rodman with Melvin B. Smith, Civil Engineer and Surveyor; for the remainder of 1884 he served as a Draftsman and Transitman on short engagements of a miscellaneous character. From March, 1886, to May, 1888, he served as Draftsman and Transitman on the New York and New England Railroad, and from September, 1888, to March, 1889, in a similar capacity, and also as Inspector, for the Boston and Maine Railroad on the construction of the Mystic Terminal Grounds.

During the summer of 1889 Mr. Young was an assistant with George H. Barney, Civil Engineer and Surveyor, at Hyde Park, Massachusetts: and from the autumn of 1889 to the spring of 1890 was an assistant and inspector with Percy M. Blake, Civil Engineer, on the construction of the water-works at Andover, Massachusetts. During the spring and summer of 1890, under the late Albert F. Noyes, M. Am. Soc. C. E., City Engineer of Newton, Massachusetts, he was an assistant on assessor's maps, drafting, surveying, computing, etc.

In September, 1890, he entered the service of the Metropolitan Sewerage Commission, as Draftsman and Transitman, and was en-

* Memoir prepared by Howard A. Carson, M. Am. Soc. C. E.

gaged in the office and on the surveys and construction of the North Metropolitan and Charles River Systems, including tunnel work under compressed air. In October, 1894, he left the employ of the Metropolitan Sewerage Commission, and entered that of the Boston Transit Commission, as Assistant Engineer, where he remained until March, 1898. During this period he was engaged on the construction of the Boston subways, designs for ventilating chambers, grading and sewers for Boston Common, tunnels under thoroughfares, and pile foundations for the Public Garden incline. From April, 1898, to January, 1899, he was Assistant Engineer on sewer assessments in Boston, and from February to April, 1899, Assistant Engineer on designs for masonry structures for the Boston Elevated Railway.

In June, 1899, Mr. Young again entered the service of the Metropolitan Sewerage Commission, as Assistant Engineer, and, until August, 1903, was engaged on surveys and construction of the high-level sewer, involving tunnel work in earth and rock, and the use of compressed air in advancing the headings through quicksand.

In August, 1903, he entered the employ of Messrs. Jacobs and Davies, Consulting Engineers, as Engineer of Alignment on the Hudson Tunnel, having charge of the triangulation, lines, grades, etc. From August, 1904, until January 31st, 1906, he was in charge of drafting for the O'Rourke Engineering Construction Company, and from February 1st, 1906, until his death he was Engineer and Manager of the New York work of the Healey Sewer Machine and Construction Company, which company was engaged in land and subaqueous borings for various railroads, the New York Rapid Transit Railroad Commission, and for the additional water supply for New York City.

As an indication that Mr. Young's work was excellent in character, he was given a bonus of $500 by his employers, Messrs. Jacobs and Davies, on the meeting of the headings of the Hudson River Tunnel, for the alignment of which he had been responsible. In the letter of transmittal, which also informed him of an increase in salary, this bonus was mentioned as "some appreciation" of the value of his work and of the untiring energy with which he had always devoted himself to the interests of the tunnel company.

On February 6th, 1897, he was married to Miss Alice Frances Carter, of Charlestown, Massachusetts, who, with two children, survives him.

For some years prior to his death, Mr. Young's physical condition had been such as to call for a surgical operation of a serious nature. His very energy and ambition here worked against him, leading him to delay too long in seeking surgical relief; however, in April, 1906, he went to Boston, where, in a private hospital, the necessary operation was performed, on the 6th of that month. On April 25th he went back to work in New York, but two days later was forced to give up. He returned to his home in Auburndale, and died there on June 1st, 1906. The immediate cause of his death was acute nephritis, but the original cause was his unselfish devotion to work, which led him to forget himself too completely. He was buried in Lowell, Massachusetts.

Mr. Young's fondness for music, his talent for drawing and modeling, and his appreciation of the beautiful in form and color, indicate his fine taste.

Many will feel that by his death they have lost a valued friend, and the engineering profession an ambitious member who was earnest in whatever he undertook, doing all his work honestly, truthfully and well.

Mr. Young was elected an Associate Member of the American Society of Civil Engineers on January 6th, 1904.

ALBERT HENRY ZELLER, Assoc. M. Am. Soc. C. E.*

DIED NOVEMBER 2D, 1906.

Albert Henry Zeller was born in St. Louis, Missouri, on January 20th, 1867. He was the second son of William Zeller and Christine Haarstick-Zeller. His death occurred in St. Louis on November 2d, 1906, after an illness of several months. He is survived by his mother, two brothers, William F. and Eugene C. Zeller, and one sister, Mrs. F. W. Frerichs, of St. Louis.

When he was three years old his parents took him to Germany for a year, returning to America in 1871. At this early age he developed a love for music, and was allowed to begin his studies on the violin. In later years, without permitting his love for music to encroach upon the time due his chosen profession, that of Civil Engineering, he devoted many of his leisure hours to the violin, to his own enjoyment and that of a few chosen friends.

Mr. Zeller received his early education at the St. Louis public schools, attending the Peabody School at St. Louis from 1873 until 1878, when he was sent to a French school at Lausanne, Switzerland. He remained at Lausanne two years, taking a regular course, but devoting considerable time to the study of the French language, in which he became very proficient.

In 1880 he returned to St. Louis and attended Smith Academy, and later Washington University. He was graduated from the former in 1883 and from the latter in 1887, with the degree of Bachelor of Engineering.

He was particularly fond of mathematics, and was ever ready to apply mathematical solutions to problems coming under his notice.

Mr. Zeller's first employment after graduation was as Draftsman and Assistant Engineer on the St. Louis, Iron Mountain and Southern Railway, at St. Louis, where he remained until April, 1889. From April, 1889, until May, 1891 (with the exception of the period from May to December, 1890, during which time he traveled in Europe), he was engaged as Assistant Engineer with the St. Louis Merchants Bridge Terminal Railway, St. Louis, Robert Moore, Past-President, Am. Soc. C. E., Chief Engineer. From May, 1891, to November, 1892, he was employed in the designing department of the Edge Moor Bridge Works, Wilmington, Delaware. From February to May, 1893, he was Principal Assistant Engineer of the St. Louis Terminal Railway Association, in charge of the erection of the train-shed at the new Union Station. In May, 1893, he was ap-

* Memoir prepared by Edward Flad, E. B. Fay and W. G. Brenneke, Members, Am. Soc. C. E.

pointed Engineer Assistant to the President of the Board of Public Improvements, at St. Louis, a position which he filled for four years with much credit to himself and much profit to the city. Robert McMath, M. Am. Soc. C. E., who was President of the Board of Public Improvements at the time, in writing of Mr. Zeller, states:

"He declined reappointment for another term, intending to go to Europe for a long stay. I was sorry to lose him, for he was worthy of confidence, and our relations were wholly pleasant to both, as were his relations with all who came in contact with him."

Resigning from the service of the city in 1897, he went abroad for two years; on his return, he opened an office at St. Louis for the general practice of his profession, at which work he was engaged at the time of his death.

For some years past, however, he had devoted much of his time to managing the estate of his family; and, though he kept abreast of the time in his profession, and was well posted on the engineering problems of the day, he did not engage in extensive practice.

Mr. Zeller had traveled much, having visited Europe five times. He was a man of broad mind and sound judgment, and without guile or deceit. To know him was to admire him. Justice and truth and gentleness were embodied in his every act and thought. His friends thought of him as one on whom they could depend for assistance in every good cause, and, if need be, for such sacrifices as friendship calls for.

In his untimely death the engineering profession has lost a member whose life could ill be spared. The community in which he lived, and the associates who enjoyed his friendship will remember him and continue to feel the influence of his thought and his life, and he will not have lived in vain.

It seems but proper to add the following testimonial by Mr. William Chauvenet:

"How pleasant it is—after all—to speak of one whose memory must ever be a sweet one and whose life as we recall it will ever help to make us better men.

"It was his gentleness in all his relationships, and his justice in all dealings with men, that most impressed me. He was strong to see the right and fitting thing to be done in any emergency, and his clear vision was ever helpful to his friends.

"His command of himself in all positions, and the absence of any tinge of severity in the exhibition of such command, was one of his most noted characteristics.

"Ever considerate of others, he nevertheless made you feel that he was somehow under obligations to the one he was helping, and this mark of the true gentleman was one of his charms as a companion and as a friend.

"Of his own fortunes, whether good or bad, he never spoke, while his ever-active interest in the good fortunes of others, and his sure sympathy in times of loss, made him one whom we naturally sought out whenever the clouds hung low.

"Assuming nothing, yet was his judgment sound, and in all his relationships with men, and especially with his comrades, nothing could disturb his sweetness.

"I know I am naming the characteristics of a rare man, and one more associated with the 'old school' than with our aggressive and unconventional times, yet such was my friend and dear companion.

"He is to me alive and often present in our midst, because his qualities partook of those things which do not die, but live on, safe within the hearts of those who had learned to love him while here."

He was a member of the Engineers' Club of St. Louis, the University Club, and St. Louis Field Club. He was elected a Junior of the American Society of Civil Engineers on May 7th, 1890, and an Associate Member on April 7th, 1897.

BENNO ROHNERT, Jun., Am. Soc. C. E.*

DIED SEPTEMBER 1ST, 1906.

Benno Rohnert was born at Detroit, Michigan, on September 10th, 1866, and died at North Bay, Ontario, on September 1st, 1906.

He received his education in the public schools of Detroit and in the University of Michigan, being graduated from the Detroit High School in 1883, and from the Civil Engineering Department of the University of Michigan in 1887. While at the University, he was one of the leaders in his class, and during his senior year he was Managing Editor of *The Chronicle,* one of the University publications.

From 1887 to 1889, Mr. Rohnert was Assistant to F. Collingwood, M. Am. Soc. C. E., in the construction of the Dry Dock at Newport News, Virginia. In 1889, he was appointed by General O. M. Poe, Corps of Engineers, U. S. A., as Assistant to E. S. Wheeler, M. Am. Soc. C. E., on the public work of improvement in St. Marys River, and for fifteen years he was identified with this work in various positions of responsibility. He did all the instrument work connected with the construction of the Poe Lock, at Sault Ste. Marie, Michigan; and later, as Assistant to Joseph Ripley, M. Am. Soc. C. E., he had responsible charge of the work of making surveys for the purpose of channel improvements in St. Marys River, covering a length of channel of about 50 miles and involving the expenditure of from \$6 000 000 to \$8 000 000. A part of his duty, as Assistant to Mr. Ripley, was the making of all surveys, measurements, and estimates leading up to the improvement of the West Neebish Channel in St. Marys River, now under construction at a cost of about \$4 500 000. Since 1904, he was engaged in private practice until his sudden death by apoplexy.

Mr. Rohnert was an active member of the Masonic fraternity and a Knight Templar. Among his associates, both business and social, he was known for his honesty and stanch integrity; and he held his friends by his genial, broad-minded, and tolerant disposition. He was elected a Junior of the American Society of Civil Engineers on April 3d, 1889.

* Memoir prepared by Charles Y. Dixon, M. Am. Soc. C. E.

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