The Author. they should be sufficient at any rate to begin with, for he had provided by-passes at each, so that any one could be thrown out of working without disturbing the daily supply. Moreover, the main was of wrought steel, and therefore not subject to the bursts which might be expected in a cast-iron main, and, in addition, the main was kept under as low a head as possible by regulating the outflowing discharge from Bakers Hill, West Northam, Bulla Bulling, and so on, instead of by means of valves at the respective receiving-reservoirs. When the demand increased largely, it would no doubt be advisable to increase correspondingly the capacity of the service-reservoirs. The advantages of the circular form in service reservoirs, to which Dr. Deacon had drawn attention, were more or less generally acknowledged; and where deemed advisable the circular form had been adopted in the service- and suction-reservoirs of the Coolgardie water-supply, though this was not stated in the Paper. The relative advantages of thick and of thin linings, also raised by Dr. Deacon, must always be one for argument; and, personally, the Author preferred to expend £1 at once, and another £1 after 20 years, rather than £1 15*s*., or even somewhat less, to begin with. The general statement, referred to by Mr. Fitzmaurice, as to the improvement of mortar by the use of a proportion of loam in lieu of the same quantity of sand, was, of course, not based on the Author's opinion, but on the results obtained by Professor Sherman. The tests carried out for the Bulla Bulling reservoir confirmed that writer's results, and the Author would like in this connection to draw attention to the universal Indian practice of adding clay more or less burnt to pure limes. The clay was added in the presence of water, and unquestionably a stronger and more hydraulic mortar was produced than if the place of the clay had been taken by an equal quantity of sand. It was possible that similar good effects were produced by combination of clay with the free lime present in most Portland cements, but extreme care was necessary, as Mr. Hawksley had pointed out.

Correspondence.

Professor Campbell Brown. Professor J. CAMPBELL BROWN, of Liverpool, agreed that the theoretical requirements for a good coating for iron pipes were as stated at p. 74. These desiderata, however, were not obtained by the practical means mentioned on p. 80. Not only at Coolgardie, but also elsewhere, Trinidad asphalt was being employed instead of pitch, with gas-tar, for the purpose of coating iron pipes. But

Professor Campbell Brown.

natural asphalt contained only about 14 per cent. of the matter which was really valuable for protecting iron pipes, namely, pitch, the remainder being mainly calcium carbonate. This large amount of mineral matter was good for paving purposes, but for coating iron it was not only useless, but positively disadvantageous. It thickened and hardened the pitch with material which did not help to make the pitch adhere to the pipes, and which was gradually dissolved by large quantities of water containing carbonic acid or other substances which acted upon limestone. Accordingly, hard pitch from gasworks would be worth more than seven times as much as natural asphalt for mixing with coal-tar for the inside coating of water-pipes. For the outside it was not of so much importance, because there the coating might be thicker, and the pipe might be embedded in asphalt of considerable thickness with advantage.

Mr. Bruce.

Mr. A. Fairlie Bruce observed that the proportion of rainfall actually found to flow off the Coolgardie catchment-area was in accordance with the general experience that, with small precipitation, the percentage received in the reservoir was much less than in the case of a heavy fall, as the former generally consisted more largely of light local showers, and the water was re-evaporated or absorbed before it reached the watercourses. No doubt the small percentage collected was also due in part to the gathering-ground being so heavily timbered. Rainfall observations were liable to be very misleading, especially in a large area, unless a considerable number of rain-gauges were distributed over the area so as to give a true average result; as the amount of rainfall was greatly influenced by elevation and exposure. The experience in Bombay had been that the mean rainfall at Tansa Lake, at a height of 362 feet above sea-level, was about 108 inches, at Tulsi Lake, 370 feet high, 104 inches, and at Vehar Lake, 190 feet high, 84 inches. As measured by the rain-gauges, which were situated at or near the dams, about 50 per cent. of the total rainfall reached the Tansa, and almost 65 per cent. the Tulsi and Vehar Lakes; but as much of the gathering-grounds, especially those of the two latter, were much higher than the rain-gauges, it was believed that a deduction of 10 to 20 per cent. should be made from these figures, to obtain correct results. The inaccessibility of the surrounding hills, which were thickly clothed with jungle, prevented rain-gauges from being established on them. The Author was to be congratulated on the general design and water-tightness of the weir, though it appeared to Mr. Bruce that the outlet-works might have been modified with advantage;

Mr. Bruce. for example, by making provision for the fixture of temporary external sluices, to enable the working sluices to be repaired in case of need, as had been done at the Staines reservoirs. The iron stand-pipe in the valve-tower appeared to be somewhat unnecessary. It would have been an improvement had the outer valve-chamber been transformed into a screening-well, as external screens were liable to damage and provided a very small area. The precautions taken to protect the toe of the dam from the effects of scour by water flowing over it were wise, though the impact might have been further reduced had the outer face been stepped or rock-faced. At Tansa, where of course the body of water flowing over the portion of the dam used as a waste weir was much larger, nullahs, 20 to 30 feet deep and 50 to 100 feet wide, had been cut by the water, and sooner or later protection works would become necessary; as already at one or two points the erosion had approached nearer to the foundations than was altogether desirable. The locking-bar pipe adopted seemed to have been in every respect the most satisfactory form of pipe that could have been used, as, in addition to avoiding the loss of strength due to riveting, considerable longitudinal stiffness was imparted by the bars. It would be interesting to know if the coating was proving durable, and if a perfectly smooth internal surface had been obtained. In America where several asphaltic compositions had been employed for coating steel pipes, they were usually baked in an oven to harden and improve the surface. The Author did not mention whether yarn had been used in making the joints, or if not, what method had been adopted to prevent lead from running into the pipes. On the Glasgow waterworks, where the 48-inch pipes were jointed with collars of somewhat similar design, this was effected by means of internal rings, which were removed after the joint was run, when it was staved internally as well as externally. The reduction of diameter at the valves might have been carried even further with considerable advantage. Mr. Bruce had for some years employed 24-inch valves for 48-inch pipes, and 18-inch valves for 32-inch pipes, small valves being cheaper in first cost and much more easily manipulated, repaired, or replaced, than those of the full diameter of the pipe. Kutter's formula was quite unnecessarily complicated for calculating the discharge of pipes, that of Darcy being much simpler and more logical. Applying it to this case with a coefficient of friction of 0·00345, the results obtained by the Author would appear to be about 18 per cent. less than they ought to have been for new and well-coated pipes; and it would be interesting to know if there was anything in the roughness of the coating, or any friction due to the projection of the bar into the cross section of the pipe, which would account for this. The following were the results of some experiments made by Mr. Bruce on the discharge of large pipes:— Mr. Bruce.

No.	Diameter.	Age.	Below Calculated Quantity.	Length.	Situation.
	Inches.	Years.	Per Cent.	Miles.	
1	48	New	1	1	Glasgow Waterworks.
2	48	10	7	11	Bombay ,,
3	32	42	34	13	,, ,,
4	24	24	32	16	,, ,,
5[1]	24	24	19	16	,, ,,
6	24	16	30	13	,, ,,

[1] No. 4 after being scraped.

The covering of the pipes with earth would appear to have been a wise step, though the banks would demand constant attention to prevent them from being washed or blown away. Experience in Bombay had shown that where pipes were laid below ground in dry earth, free from salt or organic impurity, they had remained for nearly 50 years practically unaffected by corrosion; whereas others laid above ground had been more or less attacked in a quarter of that period. They also required frequent caulking, owing to the expansion and contraction due to variations of temperature. Concrete lining on slopes, such as that employed in the service- and regulating-reservoirs, was usually found to give trouble, though it was possible that the system of reinforcement adopted might suffice to prevent the tendency to crack owing to slight subsidence, which generally occurred sooner or later. Had the floor been asphalted the thickness of concrete might have been safely reduced to about 6 inches, without danger of leakage.

Mr. Crowell. Mr. FOSTER CROWELL remarked that the Paper, besides being an instructive record of excellent work, contained useful suggestions of general bearing. For example, the facts cited in regard to the faults discovered in the rock bed at the site of the Helena weir showed the meagre value of preliminary borings and trial-shafts as indicators of the conditions of any rock formation, with reference to its continuity and reliability for the purposes of a dam. This was a not infrequent experience, and had been marked by many striking examples, a prominent one being the New Croton dam, of the New York City waterworks, recently completed, where extensive and careful preliminary explorations had been made, but where great discrepancies

Mr. Crowell. had been disclosed later by the complete excavations.[1] It might be set down almost as an axiom that the suitability of a particular rock formation for the foundation of a dam could not be known definitely until the final excavations were actually made: nevertheless, it was customary for engineers to proceed with the design, make the preliminary estimate of cost, and award the contract, as if the borings and trial-shafts were quite sufficient, trusting to luck or depending on judgment as to whether the indications were comprehensive. If afterwards serious geological faults were disclosed by the excavation, a change or abandonment of site might become necessary, or at least desirable; but usually in such cases one of two courses was adopted, either going deeper—involving increased quantities and cost, and perhaps disclosing new difficulties—or resorting to some more or less unsatisfactory artifice with attendant risks. Guided by past experience, he had adopted some time ago a rule to assign only negative or tentative values to preliminary borings; that was to say, the absence of rock would be a positive indication of unsuitability, but no other indications would be considered positive until corroborated. Carrying out this principle in preparing plans for a masonry power-dam at St. Joseph's, N.Y., in 1903, subsequently built under his direction, he had decided to ascertain actual conditions by uncovering a large part of the proposed site down to bed-rock at the owner's expense, before completing the plans and specifications or inviting tenders. There had been special reasons for precautions here, owing to the fact that the proposed structure was to take the place of a former dam which had failed because of defective foundations. Fortunately, favourable conditions had been found; but even if it had turned out otherwise, the extra expense entailed in that contingency, regarded as insurance charges against a much greater loss, would have been amply justified, and he would advocate a similar course in all important cases. It was of interest to note the growing co-ordination of recognized engineering methods on modern works, throughout the world. It did not often happen, of course, that conditions were precisely the same in different countries; but, especially during the past few years, it had come to pass that, given like general conditions, the treatment resorted to by engineers was apt to be carried out on closely similar general lines. Even in the details, and in the technical terminology employed

[1] C. S. Gowen, "The Foundations of the New Croton Dam." Transactions American Society of Civil Engineers, vol. xliii. p. 469.

Mr. Crowell. in describing results, the same was true; so that in reading and studying in America this Paper, written of work carried out in Australia—which, to Americans, was one of the uttermost parts of the earth—no glossary was required in order to arrive at a full understanding; while the work described was excellent and up-to-date, if, indeed, it was not distinctly in advance in several respects. The pipe-line was noteworthy, in respect of both the use of locking-bar pipe, and the very ingenious and effective caulking-machine; its great length and high working-pressures gave it rank among notable metal conduits; and its very fine results, as shown in the Paper, were matters of congratulation to designer and maker alike. The discharge-capacity of locking-bar pipe compared with riveted pipe of equal diameter, as shown by the tests, was a matter of great interest and importance, and it was to be hoped that the Author would supplement these tests from time to time with others, to determine the effect of use on the degree of roughness and on the capacity. It would be agreed by all interested in hydraulic matters that there was hardly any subject connected with the science of water-supply that was so greatly in need of reliable data as was the question of flow in pipes; and the Coolgardie conduit, with its arrangement of receiving- and regulating-tanks, afforded peculiar facilities for making exact and reliable determinations. There were a number of other points, such as durability and progressive leakage, permanence of pipe-coating, etc., which periodical tests would bring out. So far as Mr. Crowell knew, it was not usual to pay great attention to the procurement of such data; but they would prove of great value if systematically collected and published.

Mr. Fairley. Mr. William Fairley thought that the description of the pumping-machinery might have been enlarged with advantage, considering the very important part it played in the success of the works. It would appear that, from the initiation of the scheme, considerable uncertainty was entertained as to exactly how much water would be taken during the first few years after the completion of the works; the quantity having been put as low as 1 million gallons per day. In these circumstances, while it might have been advisable to construct the conduit to the dimensions necessary to deliver the full quantity provided for in the scheme, the necessity for providing the pumping-machinery on a similar scale was not apparent from the information given in the Paper. In the first four stations there were provided three units, each approximately of 300 HP. and equal to one-half of the required capacity, with one of the units, or one-third of the total power, in reserve.

Mr. Fairley. Boiler-plant was provided on a similar scale, and this arrangement appeared to give a safe reserve and to be particularly suited to the case. In the remaining four stations, however, only two units were provided, each equal to the required full capacity, one of the units being spare, with boiler-plant similar. The reason given for this variation was that economy was effected by having the whole of the steam ends of the engines to the same patterns, thereby reducing initial outlay and the number of spare parts to be kept in stock: but in a scheme of such magnitude, where about twenty sets of engines were required, sufficient saving by having one size of engine instead of two or three different sizes, was not apparent. Certainly from the engine-builder's point of view it was convenient, and might mean not only that he realized some little economy in constructing in the shops, but also that power considerably above actual requirements had to be installed in four stations. The advantage of having all spare parts to the same standard for eight stations, spread over 300 miles of country, did not appear to counterbalance the serious expense in providing additional engine- and boiler-power. It did appear that if three units, as in the first four stations, had been provided in the remaining four, giving each station one-third reserve, a very considerable saving would have been effected; and in any case the smaller units for pumping and for steam-generation would have lent themselves more readily to economical working in the early years, should the demand be less than originally estimated—which Mr. Fairley understood was the case in this instance. Instead of 6,068 HP., as shown provided in the Paper, 5,463 HP. would appear to be sufficient if two standard sizes had been adopted instead of one. The difference, at £48 per HP., amounted to about £29,000. The practice of putting down two units, each equal to half the maximum demand, and adding the reserve third as soon as required, would appear to be more economical, and might have been applied in this instance; there would have been considerable saving in the capital cost and also no difficulty in adding further pumping-power as required in the future. On p. 89 the Author mentioned that the feed-water was passed through a heater placed on the exhaust-pipe leading to the condenser. Mr. Fairley would be glad to know if experience had shown any gain from this arrangement. Also, had the Author found any difficulty in keeping each pumping group doing its exact amount of work, due to the tendency in this type of pumping-engine to run short stroke?

Mr. Folwell. Mr. A. Prescott Folwell, of Easton, Pa., remarked that, while it could hardly be questioned that the yield of each catchment-area was a law unto itself, it was also true that watersheds similarly located had many characteristics in common; and in view of the apparent similarity in several respects between the Helena basin and many of those in the south-western part of the United States, a brief statement concerning these latter might be of interest. Along the Pacific coast of Southern California ran a range of mountains, east of which lay about 80,000 square miles of country, between 4,000 feet and 10,000 feet above sea-level, which had a mean annual rainfall varying between 2·8 inches in the south and 7 inches in the north, with an occasional year of practically no rain at all. As an illustration of the yield in this region, in 1901 one area of 4,000 square miles yielded 0·185 inch, and two others, of about 23,500 square miles each, yielded 0·246 and 0·2785 inch, respectively. In the mountainous region to the west the mean rainfall varied between 5 inches and 60 inches per annum, and several reservoirs were located there. Typical of many of these was the Sweetwater, which was notable in possessing fairly complete records of rainfall, evaporation and yield on the drainage-area, dating back to 1888. The rainfall and the ratio of the yield thereto were given in the following Table, the years extending from September to August inclusive:— Mr. Folwell.

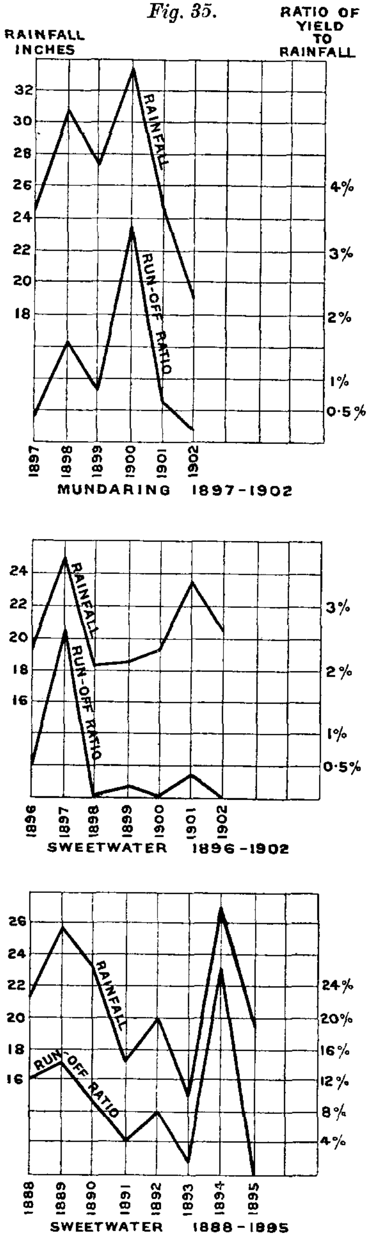
RAINFALL AND YIELD ON THE SWEETWATER CATCHMENT-BASIN.

Year.	Rainfall.	Ratio of Yield to Rainfall.	Year.	Rainfall.	Ratio of Yield to Rainfall.
	Inches.	Per Cent.		Inches.	Per Cent.
1888–89	21·00	12·000	1896–97	24·91	2·670
1889–90	25·71	14·000	1897–98	18·18	0·003
1890–91	23·40	9·000	1898–99	18·51	0·135
1891–92	17·14	4·000	1899–1900	19·10	0
1892–93	20·00	8·000	1900–01	23·65	0·368
1893–94	14·76	1·000	1901–02	20·29	0
1894–95	27·14	26·000			
1895–96	19·54	0·517	Mean	20·95	5·55

Practically all the rain fell between September and April, or during about the same length of time as the rainy season at Mundaring; and it was seen to be approximately similar to the rainfall there in both quantity and variations. The percentage running off, however, averaged almost five times as much as at Mundaring. In *Fig. 35* the rainfall and the percentage of yield for each of these two watersheds was plotted, and a remark-

Mr. Folwell.

Fig. 35.



able similarity between the two was evident, both showing what the Paper referred to, namely, that the ratio of yield to rainfall was even more variable than the rainfall itself; also, that extremes of rainfall were likely to be accompanied by still greater extremes in the yield-ratio, the ratio-curve always moving in the same direction as the rainfall-curve. At Mundaring, however, this ratio had not reached zero, and it was to be hoped that it might never do so; but in all the dry regions of the United States such a condition was likely to occur more or less infrequently. As on the Helena, the high percentages of yield were due to heavy storms, during which one-fourth to two-thirds of the entire season's rainfall fell in 2 or 3 days. The last seven seasons on the Sweetwater conveyed a lesson which would not soon be forgotten in that country, for in only one of these had the yield equalled the evaporation from the reservoir (annually 15 to 20 per cent. of the reservoir's capacity), which had consequently been almost entirely empty for 4 years, although having a capacity of about three times the consumption. It was possible that the apparent similarity between these two cases was but superficial; but if not, the storage allowance at Mundaring, equal to 2 years' consumption, was not only not excessive but might prove disastrously small, and the drainage-area itself insufficient, as the Author considered it to be. Calculation based on *Fig. 5* (p. 59) and Table IV. of the Appendix showed that the Mundaring reservoir would apparently lose by evaporation about 780,000,000 gallons per annum when full, and about one-fifth of this amount when half full. If it had been full at the beginning of 1901, and if the daily consumption had been 5,600,000 gallons, it would have been lowered during 1901 by about 1,267,000,000 gallons, and during 1902 by about 185,000,000, leaving less than 1,500,000,000 gallons to tide over a possible third dry year; while two successive years like 1902 would completely empty the reservoir. It would seem probable, therefore, that the fears of the Author were well founded, and that a study of the rainfall and yield of this and adjacent basins should be continued, so that before the consumption reached 5,000,000 gallons per day definite plans for increasing the storage, and especially for increasing the yield, might be decided upon and carried out. Mr. Folwell.

Mr. JAMES H. FUERTES observed that the yield of streams depended primarily upon rainfall; yet, under ordinary conditions, the fluctuations in yield might be in no definite degree proportional to the fluctuations in rainfall. So far as actual rates of flow were concerned, each stream was a law unto itself. Two streams with similar adjacent watersheds, with the same annual rainfall, and with apparently identical conditions in all respects, might show wide differences in ordinary rates of flow; and attempts to deduce actual monthly flows from rainfall-records could in most cases result only in rough approximations, likely to be in error by anything between 5 per cent. and several hundred per cent., the likelihood of large errors in the percentage being greater with small than with large rainfalls. Within the limits of the United States there existed meteorological and other conditions, which, in different localities, conspired to produce stream-flows ranging from practically the entire rainfall to practically nothing. The Yakima River in the north-west, for instance, had a normal yield considerably in excess of the normal rainfall, as reported, the excess being derived from the melting of the snow in the high mountainous regions. On the other hand, there were streams in the desert lands of the west which were practically dry throughout the greater part of the year. While the conditions indicated by the very low yield of the Helena River and its tributaries were extreme, they had been approached by certain rivers in the United States; and in one at least, the Cimarron River in Colorado and Kansas, the annual rate of off-flow had been very much lower than the Helena gaugings had so far indicated. Mr. Fuertes.

Mr. Fuertes. The Cimarron (with a watershed of 5,200 square miles) discharged in 1896 only 0·004 inch of rainfall from its watershed, this being only one-tenth the rate of flow shown by the Helena in 1902. Similarly, in 1896 the Arkansas River at Hutchinson, Kan., with a watershed of 34,000 square miles, gave a yield of only 0·06 inch of rain, the average rainfall over the watershed for that year being about 20 inches. These wide variations in rates of flow showed, as the Author stated, the extreme caution necessary in estimating stream-flows in dry countries, where important works would depend for their usefulness and efficiency upon the water collected. The most important point for determination, in countries where the normal yield was small, was the minimum annual flow to be expected for different annual rainfalls below the normal. In such countries the amount of storage necessary was usually so great that actual minimum flows, or minimum monthly flows, were not of much significance. In some cases it was also important to know what the minimum annual flow for two or even three successive years might be. In arid countries it was not unusual to have two consecutive years during which stream-flows might be extremely small. For instance, in 1896, 1897 and 1898 the total annual flows of the Arkansas River at Hutchinson were respectively equal to but 0·06, 0·07 and 0·12 inch of rainfall on the watershed. Viewed in the light of the gaugings quoted hereafter, it was therefore quite probable, as the Author suggested, that the 2 years' storage originally provided in the Mundaring project might not prove sufficient, with the present watershed, to tide over a cycle of dry years and to afford a constant daily supply of 5½ million gallons. The annual precipitation and its seasonal distribution appeared to be the two principal factors determining the annual yield of streams; and, of the two, the seasonal distribution undoubtedly had the most direct effect. For instance, the flow of the Muskingum River in 1888, with 42·6 inches of rain, was 10·3 inches from the watershed, while in 1893, with 42·36 inches of rain, the yield was 16·2 inches, an increase of 57 per cent. with a slightly smaller rainfall. In the first case about 25·5 inches of the total precipitation occurred during the months June–November inclusive, and 17 inches in the months December–May, while in the second case the conditions were reversed. The only other condition appearing to exert much influence on the yearly flow was the relative capacity of the watershed to absorb water, and subsequently to give it up with a comparatively small evaporative loss. Dense forests and abnormal ground-storage seemed to be favourable to such conditions, for numerous observations supported the view that from areas covered with large dense forests the

Mr. Fuertes.

yield was uniformly larger than from those not so protected. In order to show this effect, however, the forests must be so thick as to afford dense shade, to maintain the atmosphere near the ground in a moist condition, at a comparatively constant temperature, and to prevent the winds from blowing over the ground; ordinary patches of forest-area, second growth, or scrubby timber and bushes, had practically no effect towards increasing stream-flows. The foregoing remarks applied, of course, only to total annual yield, and apparently the size of the watershed, and its geological and topographical features, had little effect on that yield. These views were substantiated by the following data (p. 148) relating to the annual flows of several rivers. The yields tabulated were the smallest recorded flows, resulting from the stated annual rainfalls, which appeared in such records as had come under his notice. For instance, if on any stream a rainfall of 42 inches occurred in several different years, giving different yields, that year had been selected for tabulation which gave the smallest yield, if it was unusually low for the given rainfall; if the flow was not unusually low it had not been tabulated. Thus a given stream might appear in the tabulation only once, or it might appear several times. The yields tabulated were not necessarily, therefore, the minimum flows of the various streams. For instance, the yield of the Muskingum in 1895, with a rainfall of 29·84 inches, was but 4·9 inches; this, however, was a greater relative yield than 10·3 inches in 1888 from a rainfall of 42·6 inches. For this particular stream none of the annual flows, with the exception of that for 1888, were extraordinarily low in proportion to the rainfall. The 1888 gauging alone, therefore, had been given. The tabulation included rivers with watersheds ranging from 18·9 square miles to 54,900 square miles in area, and at elevations varying between the level of the sea and a general altitude of 6,000 or 7,000 feet above sea-level. In mean annual temperature the districts represented varied between 40° F. and about 65° F., and in mean annual rainfall between 10 inches and 55 inches. Topographically, nearly every conceivable type was exemplified: rivers in mountainous districts, in deforested plains, in ordinary rolling country, in arid deserts, and in flat country dotted with lakes. Streams from heavily-forested areas, in the sense previously described, were not included in the tabulation, there being very few streams issuing from such districts in the United States for which long-period gaugings were available. Since in most cases where stream-flows had to be estimated, only rainfall-records were available, all such estimates must be based on comparisons with other streams which had been gauged, and in

Mr. Fuertes.

EXCEPTIONALLY SMALL ANNUAL STREAM-FLOWS RESULTING FROM CERTAIN ANNUAL RAINFALLS, ON VARIOUS WATERSHEDS.

Stream-flows expressed in Inches of Rainfall *running off from the Watershed* during the Year.

River.	Drainage-Area.	Rainfall.	Yield.	Year.
	Square Miles.	Inches.	Inches.	
Arkansas, Hutchinson, Kan. . . .	34,000·0	20·4	0·06	1896
,, ,, ,, ...	34,000·0	25·0	0·12	1898
,, Rocky Ford, Col. . . .	11,440·0	16·5	0·64	1898
Broad, Carlton, Ga.	762·0	56·0	21·4	1898
Cimarron, Arkalon, Kan.	5,200·0	15·0	0·004	1896
Cobbosseecontee, Gordiner, Me. . .	230·0	47·0	14·0	1892
Lake Cochituate, Mass.	18·9	46·0	14·5	1876
,, ,, ,, ...	18·9	61·5	15·8	1866
,, ,, ,, ...	18·9	58·5	20·0	1869
,, ,, ,, ...	18·9	58·0	22·5	1867
,, ,, ,, ...	18·9	50·5	23·5	1895
,, ,, ,, ...	18·9	56·0	26·5	1898
,, ,, ,, ...	18·9	67·5	27·0	1863
Croton, Croton Dam, N.Y. . . .	339·0	47·8	17·98	1871
,, ,, ,, ,, ...	339·0	56·0	25·1	1882
,, ,, ,, ,, ...	339·0	59·3	28·9	1898
,, ,, ,, ,, ...	339·0	60·3	32·6	1888
Desplaines, Riverside, Ill. . . .	630·0	32·5	3·0	1895
,, ,, ,, ...	630·0	33·5	5·0	1895
Elkhorn, Norfolk, Neb.	2,474·0	26·1	0·94	1897
Genesee, Mt. Morris, N.Y. . . .	1,070·0	40·7	12·80	1896
Green, Blake, Utah	38,200·0	12·5	2·50	1897
Helena, Australia	569·0	19·3	0·04	1902
,, ,, ...	569·0	24·5	0·08	1897
,, ,, ...	569·0	25·0	0·17	1901
,, ,, ...	569·0	27·2	0·23	1899
,, ,, ...	569·0	30·8	0·46	1898
,, ,, ...	569·0	33·3	1·16	1900
,, ,, below weir . .	50·0	35·0	2·30	1899
,, ,, ,, ,, . .	50·0	36·0	3·60	1901
,, ,, ,, ,, . .	50·0	40·0	7·40	1900
James, Buchanan, Va..	2,058·0	41·9	10·8	1896
Jefferson, Sappington, Mont. . .	8,984·0	18·0	0·34	1898
Kansas, Lawrence, Kan.	54,891·0	26·0	0·86	1898
Muskingum, Zanesville, O. . . .	5,828·0	42·6	10·3	1888
Mystic, Mass.	26·9	50·4	16·3	1895
Neshaminy, Forks, Pa.	139·3	44·4	16·6	1896
,, ,, ,, ...	139·3	67·0	35·8	1889
Ocmulgee, Macon, Ga.	2,425·0	50·0	15·4	1898
Perkiomen, Frederick, Pa. . . .	152·0	49·7	17·3	1896
,, ,, ,, ...	152·0	66·8	36·1	1889
Potomac, Great Falls, Md. . . .	11,880·0	51·1	15·3	1886
Presumpscott, Sebago Lake . . .	470·0	59·2	23·8	1888
Rio Grande, S. Marcial, N.M. . .	28,067·0	10·0	0·36	1896
Smoky Hill, Ellsworth, Kan. . .	7,980·0	25·0	0·12	1897
,, ,, ,, ,, . . .	7,980·0	26·0	0·13	1898
Solomon, Niles, Kan.	6,815·0	25·0	0·29	1898
South Platte, Denver, Col. . . .	3,840·0	11·5	0·61	1896
Sudbury, Mass.	78·0	44·5	16·6	1892
,, ,, ...	78·0	52·1	22·3	1895
Tohickon, Pa.	102·2	68·0	42·0	1889

order to render such comparisons of value, the nature of the streams used for comparison should be known. It was proper, therefore, in order to render the foregoing data of practical value, to state briefly the general character of the watersheds from which these streams flowed. In these descriptions, the term "deforested" did not necessarily mean entirely devoid of vegetation, but referred to watersheds which might contain a considerable amount of forest-area, if open, or scattered in patches, as already explained. Mr. Fuertes.

Arkansas River, above Hutchinson, Kan.—Practically totally deforested. Rises in mountains of Colorado, but principal part of watershed is in the rolling prairies of Colorado and Kansas. Since 1887 large quantities of water have been diverted at the head-waters in Colorado for irrigation in that State. The effect of this diversion, while significant, will not account entirely for the very low flows. Not a very large percentage of watershed under cultivation. Mostly ranges. Elevation generally from 2,000 to 3,000 feet above sea-level.

Cimarron River, above Arkalon, Kan.—Totally deforested; at times disappears in ground by percolation at parts of its course, reappearing again above Arkalon. Rate of flow small and generally without much fluctuation. Watershed lies 3,000 to 4,000 feet above sea-level, and is largely devoted to cattle-ranches.

Cobbosseecontee and Presumpscott Rivers, in Maine.—Watersheds comparatively flat land, largely covered with more or less open forests, but large percentage of exposed water-surface in lakes and ponds; about 500 feet above sea-level.

Cochituate Lake, in Massachusetts.—Similar to Sudbury, but a much larger percentage of water-surface and marshy land.

Croton River.—Much broken with high steep hills; considerable second growth and open forest-lands in south-eastern New York, about 500 feet above sea-level. Fair proportion under cultivation.

Desplaines River.—Deforested rolling prairies in Illinois; considerable cultivated areas.

Elkhorn, Kansas, Smoky Hill, and Solomon Rivers.—Streams in Kansas and Nebraska. Rolling prairie country, totally deforested. All 1,000 to 2,000 feet above sea-level. Watershed partly in pasture land, partly under cultivation.

Genesee River, above Mt. Morris, N.Y.—Rolling hilly country in New York and Pennsylvania. Considerable timber on portions of watershed and a good deal under cultivation. Mostly good farming-country. General elevation 1,000 to 2,000 feet above sea-level.

Green River, above Blake, Utah.—Deforested; rises in Rocky Mountains in Wyoming, flowing south to join Colorado. Watershed above Blake, 6,000 to 7,000 feet above sea-level; generally rolling elevated plateaus. Valleys generally very wide and flat, and frequently deeply depressed below the elevated table-lands.

James River, above Buchanan, Va.—In mountains of Virginia. General elevation 1,000 to 2,000 feet; hills and mountains quite generally covered with second-growth timber. Small proportion under cultivation; several broad flat valleys, amounting to perhaps 10 per cent. of watershed, entirely cleared.

Jefferson River, above Sappington, Mont.—At head-waters of Missouri River, in Montana, in mountainous country. Considerable timber and fair proportion of open wide valleys. General elevation 5,000 to 7,000 feet above sea-level.

Muskingum River, above Zanesville, O.—Deforested. Rolling prairies in south-eastern Ohio; a good portion under cultivation, mostly farming-country, about 1,000 feet above sea-level.

Mystic River.—In Massachusetts; generally similar to Sudbury.

Mr. Fuertes. *Neshaminy, Perkiomen and Tohickon Creeks.*—Adjoining watersheds in Pennsylvania; deforested farming-country in rolling hills about 500 feet above sea-level; about 25 miles from Philadelphia.

Ocmulgee River, above Macon, and *Broad River*, above Carlton.—Streams in Georgia. Considerable timber-land on watersheds, but not sufficiently dense to affect stream-flows noticeably; mostly rolling country 500 to 1,000 feet above sea-level.

Potomac River, above Great Falls, Md.—Generally mountainous and hilly, with broad open valleys; considerable proportion of valleys under cultivation mountains largely covered with open timber.

Rio Grande River, above S. Marcial.—Deforested; head-waters rising in mountains. More or less timbered land along stream and its tributaries. Large part of watershed in arid plains. General elevation of watershed about 6,000 feet. Some irrigated areas in crops, but most of the country wild.

South Platte River, above Denver, Col.—Deforested; rises in Rocky Mountains. Sometimes goes entirely dry. Lower part of watershed in rolling country. Watershed generally mountainous, 6,000 to 8,000 feet above sea-level.

Sudbury River.—Deforested; rolling farming-country in Massachusetts.

For many of the streams cited records of rainfall and yield had not been kept for a sufficient length of time to give a reliable statement of minimum monthly or daily flow. In the form in which they were given above, however, the data might be of service as a guide in forming a judgment as to probable minimum annual flows where similar conditions might prevail. It would be noted that for most rates of precipitation few of the streams had reached their probable minimum yields. The Croton records, for instance, extended over nearly 37 years, and the Sudbury 27 years, while the Cochituate records went back to 1863. The Cochituate in the sixties, with very heavy rainfalls, showed abnormally low yields, as compared with the yields for 1898; from which it might be implied that, if the Croton and the Sudbury had been gauged at the same time, lower flows would have been recorded for similar rainfalls than had since been reached. It must be borne in mind that for rainfalls of less than about 40 inches most of the gaugings given covered but short-time observations, many of which extended over only very few years; and somewhat smaller relative flows might be expected than those shown. In dealing with such statistics, the attempt to deduce the laws governing the observed phenomena was a fascinating study, but Mr. Fuertes felt that in the present state of knowledge it was better to avoid such temptations, and hence he was content with merely arranging the few observations which had come under his notice in a form convenient for comparison. In reading over the Author's account of the construction of the concrete dam, or weir, Mr. Fuertes had been much impressed with the fact that daily temperature-changes of 60° F. to 70° F. were of common occurrence for at least 6 months of the year. This might have had something to do with the absence of subsequent cracks due to changes of temperature. In cold climates monolithic masonry or concrete dams nearly always developed cracks in the upper parts on the advent of cold weather. Concrete expanded and contracted about the same amount, for equal changes of temperature, as steel; but while steel had sufficient strength to resist such strains, if anchored at both ends, concrete of proportions suitable for use in dams had not, being, theoretically, only able to stand a fall in temperature of about 13° F. below that at which it took its final set. This consideration would indicate that in large dams as much of the concrete as practicable should be laid in cool weather. Mr. Fuertes.

A new waterworks development for the City of Lynchburg, Va., placed under construction about a year ago, included a pipe-line 21 miles long and a concrete dam across Pedlar River, a tributary of the James River. The present plans were for a supply of 8,000,000 United States gallons[1] per day. The pipe-line traversed very broken country through which hauling with teams was expensive, and, at some seasons, impracticable, except for very light loads; this consideration had had much to do with his recommendations as to the methods and materials to be used in building the pipe-line. As now under construction it was a composite line, 30 inches in internal diameter, consisting of about 18·8 miles of wood-stave pipe, about 1 mile of cast-iron pipe, and 1¼ mile of steel pipe. All straight portions of the steel line were of locking-bar pipe similar in construction and in details of joint to that used for the Coolgardie line, curves being made with riveted steel specials. The line had many summits, as would be understood from the fact that eighty-nine air-valves, four stand-pipes to break the static heads and limit the hydraulic gradient, and two pressure-relief valves, had been provided in the 21 miles. Sluice-valves, with by-passes, had been placed in the conduit at intervals of about 2 miles. Where these occurred at stand-pipes they were placed on the down-stream side of the stand-pipe, between an air-valve, or cluster of air-valves, and the stand-pipe. Air-valves were placed each side of the sluice-valves at all places in the main conduit. Sluice-valves located between the stand-pipes permitted accurate regulation of the flow, for any desired delivery, while maintaining the conduit full, under pressure, at all points. There were three

[1] 1 United States gallon = 0·8331 Imperial gallon.

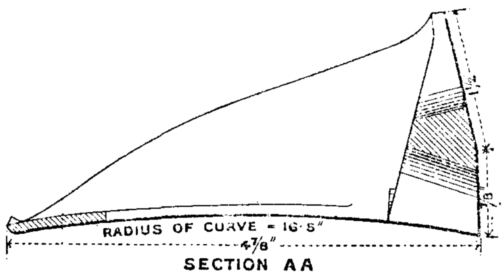
Mr. Fuertes. tunnels on the line, with an aggregate length of 2,000 feet; in all tunnels the roofs and side walls were to be lined with concrete, and the pipe was to be built continuously through them. Where the line crossed the James River, and at certain other important crossings, as well as in the city streets, heavy cast-iron pipe was to be used; at all other places wood-stave pipe would be employed for heads up to 200 feet, and steel pipe for all heads higher than 200 feet. The wood-stave pipe was built in the trench, working simultaneously at different sections, and joining the different sections of completed pipe by springing in staves cut about $\frac{1}{8}$ inch longer than the spaces to be filled. The staves were of California redwood, in lengths varying between 10 and 22 feet, and averaging 15 feet, not more than 10 per cent. being shorter than 12 feet. They were of perfectly clear stock, and only timber first-class and sound in every respect was accepted. The staves were brought from California ready for use. After arrival at the nearest railway-station they were inspected, then hauled out to the line of work, and distributed along the trench in piles about 3 feet wide and 3 feet high, special efforts being made to have the staves built into the pipe as quickly as possible after delivery. It was impossible to get perfectly dry stock, and when the staves were piled where exposed to the sun and wind, the ends were apt to split and the flat surfaces to splinter more or less. All the stock had been kiln-dried and was what would be called well-seasoned. The pipe was covered over as soon as possible after clamping up the band. The staves were dressed from 2-inch by 6-inch stuff, with outer and inner surfaces true to the proper curves, and were $1\frac{1}{2}$ inch thick after dressing. Both edges were plane and radial to the circles, but one had a very small tongue left longitudinally along its centre, about $\frac{1}{8}$ inch wide and $\frac{1}{16}$ inch high; nineteen or twenty staves completed the circle. Where staves abutted end to end, the joint was made by inserting a thin steel plate, $\frac{1}{8}$ inch thick and $1\frac{1}{2}$ inch wide, in kerfs cut in the ends of the abutting staves. This plate was a little longer than the staves were wide, so that in clamping up the bands the ends of the plate would be jammed into the sides of the staves adjoining. All the plates were coated with preservative coating of the kind used for the bands and shoes. Each band had a formed hemispherical head on one end and threads and nut on the other. The bands were made of homogeneous mild steel, having a tensile strength of 58,000 to 65,000 lbs. per square inch. Each rod was required to be capable of being bent back flat upon itself when cold without sign of fracture. The threaded end was not upset, the threads being cold rolled. The specifications required the strength of the headed end and the threaded end to be greater than that of the body of the rod. The rods were shipped from the rolling-mills, laid out straight, and tied in bundles of convenient weight for handling. Rods were taken from each lot at random, and tested to destruction. Before coating, the bands were bent by hand around circular bending-tables, to give them nearly the proper curve to fit the pipe. Each band was then examined carefully, cleaned and freed from rust, and provided with a nut and cut washer $\frac{1}{8}$ inch thick. After bending, the bands were wired together loosely in bundles of about twenty-five, and dipped in a hot bath of melted "Pioneer Mineral Rubber Pipe Coating." This was an asphaltic preparation, a product of "Gilsonite" mined in Utah; and a series of comparative tests conducted by him when seeking a suitable protective covering for the bands and shoes of this pipe-line had shown it to be practically the only coating, out of several tried, that yielded satisfactory results. He had given the subject much attention, because the life of the bands and shoes would determine the life of the pipe-line. The shoes were coated similarly to the bands. After cooling and drying, the bands and shoes were distributed along the line of the excavations, and piled up for use. The piles were examined immediately after distribution, and all parts of the coating accidentally marred or rubbed off during transportation were repainted. After the bands were on the pipe and the final clamping completed, all the ironwork was then again gone over and retouched, where the coating was imperfect, with a thick coat of Smith's durable metal-coating. The shoes were of malleable cast iron having a tensile strength of 40,000 lbs. per square inch. The quality of the iron was tested occasionally by battering a shoe out flat with the sledge, and observing the action of the metal under this treatment. Altogether, about 320,000 shoes would be used, so that a difference of 1 cent. each in the cost represented the sum of $3,200.00 (£657). Considerable care had therefore been taken in the design of the shoe, to produce a form that would have the necessary strength, and at the same time keep the weight down to the lowest limit, without allowing the metal to be less than $\frac{1}{8}$ inch in any part. The details of the shoe as finally designed were shown in *Figs. 36* (p. 154). The shoes weighed, after coating, almost exactly 1 lb. each. They had proved very satisfactory from a practical point of view, as there was no tendency for the bands to Mr. Fuertes.

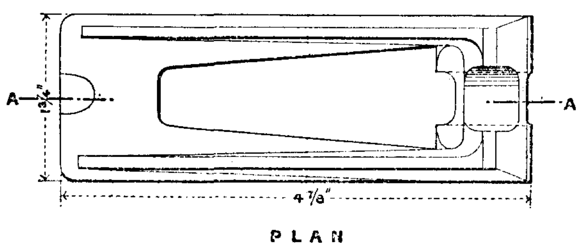
Mr. Fuertes. jump out of the shoes when turning up the nuts, nor for the points of the shoes to dig down into the staves when the shoes slipped around the pipe with the tightening of the bands. Whenever the head on pipe was more than 50 feet the bands were spaced according to the following formula:—

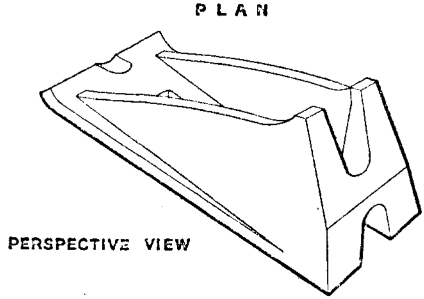
$$N = \frac{330\ HD}{d^2\ S}$$

in which N = number of bands per hundred feet of pipe; H = head

Figs. 36.







on pipe in feet; D = diameter of pipe in inches; S = permissible stress on bands in pounds per square inch of section; d = diameter of bands in inches. In this case, with S = 12,000 lbs. per square

Mr. Fuertes.

inch, the formula reduced to N = 3·3 H. Where the head was less than 50 feet, the bands were spaced closer together than the formula would require, and in accordance with the following Table :—

Head, 0 – 20 ft.	108 bands per 100 feet.
„ 21 – 26 „	114 „ „ „ „
„ 27 – 31 „	120 „ „ „ „
„ 32 – 36 „	127 „ „ „ „
„ 37 – 40 „	134 „ „ „ „
„ 41 – 45 „	150 „ „ „ „
„ 46 – 50 „	165 „ „ „ „

The entire pipe was laid under ground with a covering of earth about 2 feet 6 inches deep. In placing the bands they were first strung over the end of the pipe and accurately spaced, brace-wrenches with a 6-inch sweep being used to bring them quickly to a light tension. While tightening the bands, the staves were coopered with wooden mallets from the inside, and the bands were tapped lightly to allow them to slip around the staves as tightening proceeded. The spacing-gang was followed by another, which completed the clamping, using S wrenches with 10-inch handles as a precaution against over-tightening. The shoes were staggered according to a uniform system, and lay around the top half of the circumference of the pipe. Where wood-stave joined cast-iron pipe, special bell-ends 7 inches deep were cast on the iron pipe and the stave-pipe was inserted therein, the joint being packed with oakum, run with melted lead and caulked in the manner customary in jointing cast-iron pipes. The 30-inch valves also were cast with large bells, and the joints were made in the same way. At the crossing of the James River the pipe was under a head of 410 feet, necessitating liberal provision of air-valves in case the pipe should be ruptured in the valley. Clusters of valves, with sufficient capacity to prevent the reduction of pressure within the pipe from exceeding about 8 lbs. per square inch, under the maximum conditions, were provided at the summits on each side of such places. Vertical and horizontal curves were used, the minimum radius adopted being 200 feet. No difficulty was experienced in springing the pipe to this radius, the staves on the outside of the curve being sledged back endways to make tight butt joints before final clamping. The great advantage accruing from the use of wood-stave pipe of large size, in the matter of cost, was exhibited by the following average prices per lineal foot bid by the successful contractor for wood-stave, cast-iron and steel pipes, 30 inches in diameter each, erected, but not including excavation, which was tendered for separately. The prices for wood-stave pipe

Mr. Fuertes. were, for the staves, in place, and for the bands, per hundred bands in place. The average head on the wood-stave portion was 110 feet, requiring 363 bands per hundred feet.

	Per Lineal Foot.
Wood-stave pipe	\$2·29 (9*s.* 5*d.*)
Steel pipe ¼ inch thick	\$4·00 (16*s.* 5*d.*)
" " $\frac{5}{16}$ " "	\$5·00 (20*s.* 6*d.*)
Cast-iron pipe	\$6·38 (26*s.* 2*d.*)

For higher heads than 200 feet it would be necessary to use thicker staves and heavier bands. For a wood-stave pipe to be successful, the hydraulic gradient must be established with great care, so that the pipe would always be under pressure at all points and for all rates of flow. If this precaution were neglected, the staves in those positions where the pipe was only partially filled would decay rapidly; but when such a pipe was properly laid out, and properly built, its life should be equal to that of other forms of conduit in general use. The advantages in its use lay in the ease with which the materials could be transported through rough, broken country, the rapidity and simplicity of construction, cheapness, and the fact that the cost varied with the head under which the pipe was to work, thus making it possible to cheapen the cost under small heads by spacing the bands farther apart. Pipes of this type had been used largely in the western parts of the United States for many years, mainly in the development of irrigation-works; and, when properly built, they had proved eminently satisfactory and durable. The oldest stave-pipe in use, so far as he was aware, was at Wicopee, on Fishkill Creek, in New York State, where a mill-flume, 30 inches in diameter, 300 feet long, banded with flat hoops, and built in 1850, was still in service and in good condition. In connection with the City waterworks of Denver, Col., there were more than 100 miles of such pipes, the first having been laid in 1889. Stave-pipes, banded with round hoops and using shoes of the style adopted for the Lynchburg pipe-line, were the invention of Mr. C. P. Allen, when City Engineer of Denver, Colorado; and the United States patents having expired, the invention was now public property. The use of wood-stave pipe instead of cast iron for the Lynchburg pipe-line had involved a saving of about \$350,000, nearly \$17,000 (£3,485) per mile on the entire length, or \$19,000 (£3,895) per mile on the length for which the substitution was made. This saving in cost, as well as in interest and sinking-fund charges, would enable the town, in about 20 years, to build a second conduit of equal capacity, thus doubling the supply, at a total cost for both lines of about two-thirds of the cost of a single line of cast-iron pipe with a capacity equal to that of one of the wood-stave lines. It was also claimed, as a result of measurements made several years ago, that wood-stave pipes would deliver larger quantities of water than cast-iron pipes, under similar conditions as to sizes and gradients; but he thought this point should not be given too much weight. He considered the greatest advantage of the wood-stave pipe in respect to delivery to lie in its ability to continue its original rate of discharge after long usage, due to the fact that corrosion and incrustation did not take place, and hence a smooth interior surface was maintained. Mr. Fuertes.

Mr. C. E. GOUMENT considered that, judging from the figures of rainfall and discharges for the years 1897–1902, it seemed doubtful whether the storage-capacity, enormous as it was, would be sufficient to tide over three consecutive dry years, such as 1897, 1901, and 1902 appeared to have been. In India drought generally occurred in periods of 3 years, and if the same conditions held good for Australia, the reservoir would certainly need enlargement if shortage was to be avoided. The rainfall on the areas immediately above and below the weir, and the yield from them, were more favourable than those of the large catchment-area which had actually been utilized; and it seemed to him that a substantial saving might have been effected in the initial outlay on the storage-reservoir, and a more certain supply secured, if the recommendation now made by the Author, to extend catchwater drains into the well-watered and quick-shedding country draining into the Helena below the present weir, had been carried out in the first instance. A smaller reservoir would probably have sufficed. With regard to the class of pipes used for the supply-line, considering the enormous expenditure incurred on this item of work and the fact that the life of thin steel pipes was comparatively short, it seemed open to question whether, financially, it had been a sound decision to use these pipes in preference to ordinary cast-iron pipes which had a much longer life under the same conditions. His experience led him to think that there was no comparison between the two as regarded durability. The life of a thin steel pipe depended to a large extent on the time the coating lasted. Once this was worn through, the pipe soon corroded and became useless. In 1892 some steel tubes, $\frac{1}{4}$ inch thick, were laid down on the rising-main of one of the pumping-stations in the Punjab where the pressures were too heavy for ordinary cast-iron pipes. Some of these pipes had already corroded internally and had recently had to be renewed. Chemical analysis showed no acid in the water which would account for Mr. Goument.

Mr. Goument. excessive corrosion. The tubes had been obtained from a firm of good repute, and the coating had seemed to be in sound condition when they were laid 13 years ago. On the other hand, some cast-iron pipes laid 25 years ago in the same town had been taken up recently and had been found to be in an excellent state of preservation. Some figures to show to what extent cast iron would have been more expensive than steel would have been useful. In India, as a rule, imported cast-iron pipes, for the pressures for which they were suitable, up to a working-head of, say, about 200 feet, cost but little more than steel pipes, even in places in the Punjab where the transport by land from the nearest seaport was over 1,000 miles of railway.

Mr. Hazen. Mr. ALLEN HAZEN, of New York, thought the Paper described conditions entirely unique in waterworks practice. Probably nowhere in the world had water been carried so far, and at such heavy expense in proportion to the quantity. On the other hand, the necessity seemed to have been great, and to have fully justified the project; and the undertaking was an unusually useful one. There were many novel points in the construction, among the most interesting being the use of an entirely new form of pipe. Apparently the weight of the locking bars used for holding the steel sheets together was fully equal to the weight of the steel required for the lap joints and rivets in ordinary riveted steel pipe, so that there was no saving in the total weight. The greater strength of the joint, however, and particularly the greater smoothness of the inside surface, which resulted in less friction of the water flowing in the pipe, were substantial points in its favour, and would seem to ensure to this type of pipe a wide field of usefulness. The lead joints provided at short intervals must act as expansion-joints. American riveted steel pipe-lines were riveted continuously, and American readers would question whether lead joints were really necessary. Changes of 45° or 50° F. in the temperature of water and buried pipes were common in the northern United States; and computations indicated that there would be no serious trouble from the lack of expansion-joints, even though the range of variation of temperature were considerably wider than this. The use of the sleeves and lead joints resulted in greater smoothness of the interior surface, however, and the arrangement seemed to have been convenient and satisfactory in this case. It might not be so readily applicable to the much larger pipes which had been extensively used in waterworks in recent years. The impounding-reservoir was in a region with much greater rainfall than the region served with water. Nevertheless, the record of yield indicated that, generally speaking, even this region was at times exceedingly dry; and it appeared that, to ensure a supply, water must be carried in storage for unusually long periods, that was, for 2 years and more. Under these circumstances, particularly in a warm and sunny climate, it would seem, from American experience, that the water in this reservoir would be sure to become foul, from the growth of algæ and other small plants and animals. It would be interesting to hear what the experience had been in this respect. The passage of the water through the pumps, pipes and reservoirs would no doubt tend strongly to remove such tastes and odours if they existed at the source. The water apparently entered each successive reservoir through a pipe extending above the water-level, and so fell into the reservoir through the air. The water would thus be aerated successively at each step in its progress. With water passing through the pipe and reservoirs at the full nominal rate, about 12 days would be required for it to pass from its source to the point of use. Under the present conditions, with only one-half to one-quarter of the full quantity of water passing, a month or more would be required, and the opportunities for purification were such as did not often occur in waterworks practice. The tastes and odours which might be most troublesome, and the removal of which would be most likely to be effected by this process, could only be determined by direct observation, and their presence or absence was not indicated by the Tables of analyses in the Paper. The cost of the work was surprisingly low. Although a large part of the steel came from the United States, the cost would have been low had the work been done there, instead of on the other side of the globe. The completion of a large enterprise at so reasonable a figure, and in a way so satisfactory in other respects, reflected great credit upon both the engineering and the business ability of those who had had the matter in charge. Mr. Hazen.

Mr. Frank Herbert, of Glasgow, thought that while the Author modified by subsequent comparisons his statement that the price of locking-bar pipes was a little more than that of riveted pipes, he did not sufficiently emphasize the fact that on the basis of such comparisons the locking-bar pipe was actually cheaper. It had an ultimate tensile strength exceeding 27 tons per square inch of plate section, whereas the best double-riveted joint only had an ultimate tensile strength of 19 tons per square inch; and allowing the same factor of safety for both, and adding $\frac{1}{16}$ inch for corrosion, the weight of the one was not more than three-quarters of that of the other; moreover, owing to the lap joints and rivets, the riveted pipe required to be $7\frac{1}{2}$ per cent. larger in diameter to Mr. Herbert.

Mr. Herbert. give the same frictional loss, and as Mephan Ferguson pipe could be made rather cheaper per ton than riveted pipe, for equal efficiency as a pressure-main, Ferguson pipe cost about two-thirds the price of riveted pipe. It would be noticed, from the actual tests described on p. 74, that the loss of water through 295 miles of main, including nine pumping-stations and nine large reservoirs, amounted to less than 3 per cent., allowing the full delivery of 5 million gallons per diem: which spoke volumes for the efficiency of the pipes and joints. No figures were given to show the basis on which the diameter of the Coolgardie pipe-line had been determined, and as this was an important matter, it might be interesting to analyse it. From figures given by the Author (which could, of course, have been ascertained approximately when determining the size of pipes required), the cost of the pumping-plant, including spares, erected, was equal to £61·25 per horse-power. The charge for fuel at 32*s.* per ton (Newcastle, N.S.W.), assumed to give 74 millions duty in every-day work = £19·71 per foot of head pumped against per annum at 5 million gallons supply per day; and each foot of head cost £64·44 for pumping-plant. Each 1 inch diameter of pipe cost erected £62,333. The friction through 307½ miles of pipes delivering 5 million gallons per day (3,472 gallons per minute) would be—

				Difference
For 32-inch pipes,	1 in 2,420	. . . .	671 feet	
				111
31 „ „	1 in 2,075	. . . .	782 „	
				146
30 „ „	1 in 1,750	. . . .	928 „	
				169
29 „ „	1 in 1,480	. . . .	1,097 „	
				215
28 „ „	1 in 1,230	. . . .	1,312 „	

The saving in cost of pipes by reducing from 32 inches to 31 inches would be £62,333; while the loss by extra cost of pumping-plant would be £64·44 × 111 = £7,153 + extra annual cost of fuel = £19·71 × 111 = £2,088, which latter capitalized at 5 per cent., 7½ per cent., or 10 per cent. = £41,760, £31,320, or £20,880 respectively. Thus the saving by reducing from 32 inches to 31 inches would be £62,333, while the loss would be, at 5 per cent. £48,913, at 7½ per cent. £38,473, and at 10 per cent. £28,033. Similarly, the saving by reduction from 31 inches to 30 inches would be £62,333, and the loss would be £66,968, £52,578, or £38,188. The saving by reduction from 30 inches to 29 inches would be £62,333, and the loss would be £77,510, £60,850, or £44,200. The saving by reduction from 29 inches to 28 inches would be £62,333, and the

Mr. Herbert.

loss would be £98,615, £77,425, or £56,235. Thus for a supply of 5 million gallons, and allowing 5 per cent. interest, the most economical size was 31 inches, at 7½ per cent. 29 inches, and at 10 per cent. 28 inches. The rate of interest to be allowed depended on the probable life of the goldfields, and the probable lapse of time before the 5-million-gallon supply was required. From the Author's remarks on p. 53 it appeared that the actual demand up to the present had been somewhat disappointing, and that it would have been well to have allowed, say, 7½ per cent., and thus save £62,333 in the cost of the pipe; but otherwise the size adopted appeared to be about the best, allowing as it did about 6 per cent. interest. For a permanent town water-supply in England a 4 per cent. basis might safely be assumed, while for an individual mine or speculative venture 10 per cent. would be better policy. The fact of this, the longest water-main in the world, being of steel, invoked discussion on what had become of late years a much-debated point, namely, the relative life of cast-iron and of wrought-steel or iron pipes. Mr. Herbert's own opinion was that very little corrosion occurred in either cast or wrought pipes when placed in natural ground, generally clay; but that in artificial ground, particularly ashes, slag, or any decomposing material, rapid corrosion took place in both, and generally much more rapidly in cast-iron pipes. In bad ground the life of the pipe was almost entirely dependent on the efficiency of the coating. Tar and asphalt appeared to take a much better hold on wrought pipes, probably because these thin pipes quickly attained the temperature of the bath, even if they were not previously heated, which was sometimes done. He believed that within the next 10 years wrought-steel pipes would supersede cast pipes almost entirely for water and gas-mains, and in many places would be used for sewage.

Mr. Hering.

Mr. Rudolph Hering observed that the Paper was of special interest to American engineers, owing to some similarity between the meteorological conditions of the territory near Coolgardie and those of the Pacific coast of the United States. The rainfall in California varied between 3½ inches and 42 inches in one year. The prices paid for water near Coolgardie, even at the lowest figure, 25*s.* per thousand gallons, were much higher than the prices paid on the Pacific coast, due partly to the apparently greater difficulties of construction, and partly to the more elaborate structures than were customary in the pioneer works of California. The Coolgardie scheme appeared to be a thorough and efficient solution of the problem, and the work contained a number of suggestions as to

Mr. Hering. details which might profitably be adopted elsewhere. It was not often that 5,000,000 gallons of water must be elevated daily to a point over 1,000 feet higher than, and over 350 miles from, the source, requiring a total dynamic lift at the pumping-station of about 1,700 feet. In this respect the work appeared to be unique. Although practically all the materials for construction had been imported, the work had required less than 5 years to build, which under the circumstances did not seem excessive for its magnitude and the difficulties of the situation. The rainfall-records, as had been the case in California when that State was first developed, were meagre, and much depended on good reasoning. In arid regions the ratio of yield to rainfall varied greatly, being large for large precipitations and small for small ones. In California, with rainfalls less than 10 inches per annum, unless some of the rain fell as sudden showers, there was usually no off-flow at all. San Francisco was obliged for that reason to store its water for 2 and even 3 years. A few years ago the evaporation during 1 year from the storage-reservoirs was greater than the rainfall or the inflow from the catchment-area. In the western deserts of the United States, that portion of the rainfall which did not evaporate percolated to a considerable depth, and continuously fed the rivers, which, like the Colorado River, discharged large quantities of water through the year, although flowing in the midst of a desert. Naturally, such conditions had invited irrigation schemes, which, near the mouth of such a river, would allow the entire flow to be diverted without infringing upon riparian rights. With regard to the quality of the Coolgardie water, the difficulties due to common salt would gradually disappear, particularly if flat and swampy areas on the catchment-area were sub-drained so as to prevent the capillary action near the surface in dry weather from raising the dissolved salts and holding them upon the surface by subsequent evaporation until they were again dissolved by the rain-water. He could not understand how there could be anaerobic bacterial action in the pipes carrying potable water, which should contain a large amount of oxygen. The descriptions of the locking-bar pipe and of the caulking machinery, which seemed to present novelties of value, were of much interest.

Mr. H. C. Hill. Mr. Henry C. Hill, of Philadelphia, communicated an account of the reservoir at the Belmont filtering-station, forming part of the works for the improvement, extension and filtration of the water-supply for the City of Philadelphia. This station was situated about 1½ mile west of the Schuylkill River, and consisted of a settling-reservoir, eighteen covered ¾-acre filters, and

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a covered clear-water basin. The settling-reservoir was divided by an embankment into two compartments, having an aggregate capacity, with 25 feet depth of water, of 72 million gallons. The reservoir was situated on a hillside, mainly in excavation. The area of the east division at the flow-line was 5·4 acres, that of the west division being 5·33 acres. The east division was constructed about equally of cut and fill, while the west division was nearly all in excavation, principally a quartz-like trap rock with some micaceous rock. In making embankments the most impervious materials of excavation were placed next the inner slope, and all materials were rolled in layers not exceeding 6 inches in thickness. Whenever excavation in rock revealed fissures, these were filled with Portland-cement grout, and the irregular surfaces of the rock in the floors and slopes were levelled up partly with clay puddle, and partly with Portland-cement concrete. In preparing the foundation for the rolled embankment all top soil was thoroughly stripped off, and the inclined ground was stepped in horizontal terraces. All embankments and fills were rolled with four 25-HP. traction-engines, each weighing 10 tons or 3,300 lbs. per foot width of roller-wheels; two 18-HP. traction-engines of 8 tons each; and one 10-ton traction-roller. All the rollers were provided with grooved front wheels and cleated rear wheels. Each division of the reservoir was lined on the floor and slope with 18 inches of clay puddle, on which was placed a lining of Portland-cement concrete 6 inches thick (*Figs. 37*, p. 164). On the concrete lining over the whole floor, and extending up the slopes to a point 10 feet vertically below the water-line, was placed a $\frac{3}{4}$-inch mixture of asphalt mastic and grit. In order to prevent slipping of the asphalt mixture on the slopes, the concrete was indented with grooves $\frac{1}{2}$ inch wide and deep, and 4 inches apart. At the end of the dividing embankment was a valve-house, in which were placed all the valves for controlling the water to and from the two divisions, which were provided with separate inlet- and outlet-pipes, and screen-chambers. As the reservoirs were designed as settling-basins and not as storage-basins, the water was pumped through a 48-inch cast-iron main, laid on the floors and extending diagonally across each division of the reservoir, and was discharged through a series of 48-inch spigot-and-faucet tees, set at alternate angles of 45 degrees from the vertical. The ends of the inflow-pipes were provided with 48-inch upturned elbows. The water passed upwards from the tees and elbows diagonally across the basin to a floating discharge-pipe, consisting of a 48-inch riveted steel pipe $\frac{1}{8}$ inch thick, provided

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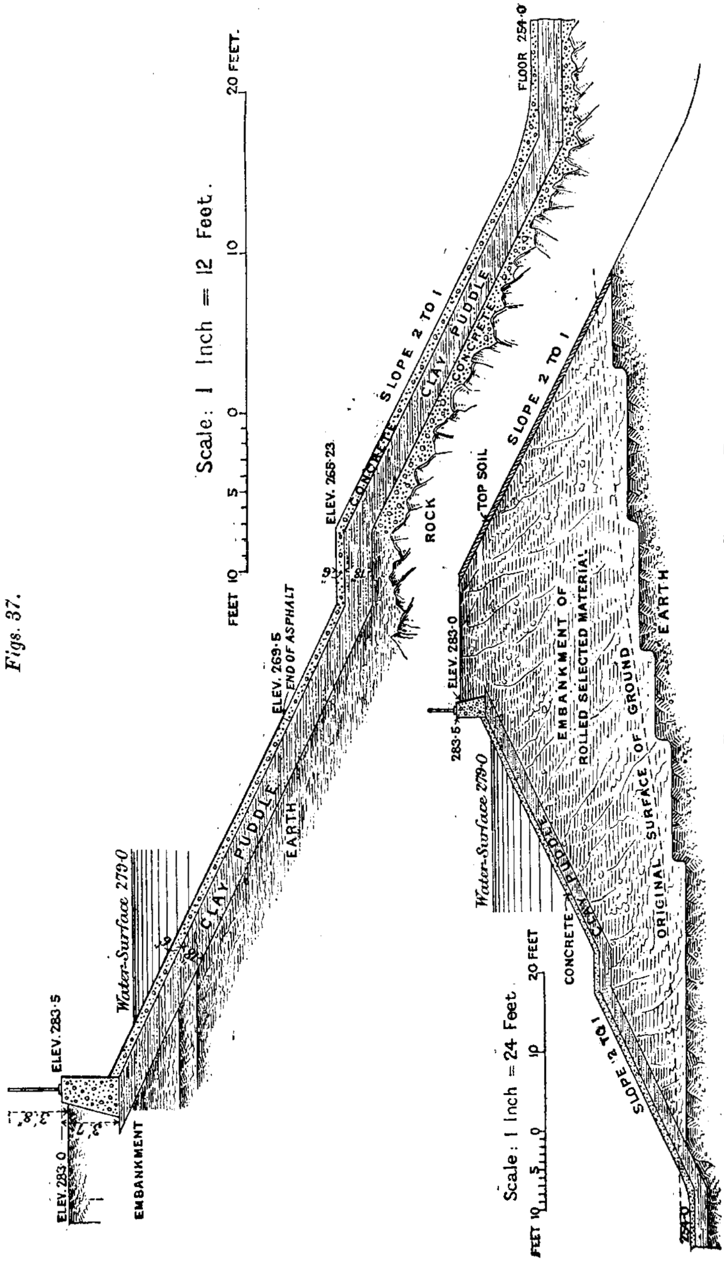
Figs. 37.



Settling-Reservoir at Belmont Filtering-Station, Philadelphia.

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with a swinging joint at the bottom, and sustained in an inclined position by a cylindrical iron float which kept the mouth of the pipe about 2 feet below the surface, so as to admit of drawing off the upper stratum of water. When both divisions of the basin were in service the water was forced to pass diagonally across and upwards through both divisions, before it left the sedimentation-reservoirs on its way to the filters. The reservoir was designed to be water-tight within the limits of the materials employed, and information had been sought on this point, with the result that very few data had been found to be available on the subject of the water-tightness of puddle and concrete-lined structures. Such tests as might have been made elsewhere were not readily accessible in publications, and the leakage assumed to be properly allowable was therefore entirely a matter of judgment. Obviously, structures like earthen embankments lined with concrete, made up of many separate floor- and slope-sections, and containing many thousand lineal feet of comparatively shallow joints, could not, even with the utmost care in construction, be expected to be absolutely water-tight. The puddle used in the works at Philadelphia was composed of equal parts by volume of rich ferruginous clay and broken stone or gravel, moistened with water and thoroughly mixed in a Chambers pug-mill. As much care was exercised in the testing of clays, and in supervising the proportions and mixing of clay puddle, as in the proportioning of materials and the mixing of concrete. All puddle clays were tested by the Ulzer method of rational analyses, and it was found after a long series of experiments that the best results were obtained with clays containing at least 50 per cent. of available clay constituents, or, in other words, the resulting mixture of 50 per cent. of clay and 50 per cent. of broken stone would contain 25 per cent. of available clay constituent, as shown by rational analyses. The concrete lining was composed of 105½ lbs. of Portland cement, 3 cubic feet of sand, and 5 cubic feet of broken stone, the stone ranging in size from ¼ inch to 1¼ inch. In some cases, to ensure more water-tight work, the 3 cubic feet of sand was altered to 1½ cubic foot of sand and 1½ cubic foot of limestone screenings. All concrete was mixed in cubical box mixers of at least 1 cubic yard capacity. The dry ingredients, after being dumped in the mixer, were given eight or ten turns, water was then added, and the mixer was given about twenty-five more turns. All cement was required to fulfil the following requirements before it was incorporated in the work:—

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Specific gravity	3·10
Fineness: residue on—	
50-mesh sieve	0·0 per cent.
100-mesh „	10·0 „
200-mesh „	25·0 „
Anhydrous sulphuric acid	1·75 „
Initial set (Vicat needle)	20 minutes
Briquettes 1 square inch in cross section to develop the following tensile strengths—	
7 days (1 day in air, 6 days in water), 1 part of cement, 3 parts of standard quartz sand	170 lbs. per sq. in.
28 days (1 day in air, 27 days in water), 1 part of cement, 3 parts of standard quartz sand	240 lbs. per sq. in.

All cement to be subjected to the boiling test before acceptance.

Six-inch cubes of concrete taken from batches used in the work were required to withstand the following crushing-loads:—

At 30 days old	1,700 lbs. per square inch.
„ 60 „ „	2,100 „ „ „ „
„ 90 „ „	2,400 „ „ „ „

It was found in practice that the above requirements were readily met, in fact there were quite a number of 60-day cubes which took a crushing-load of 5,000 lbs. or more per square inch, the average of all cubes being about 3,300 lbs. While the clay puddle was relied on for water-tightness, it had been thought advisable to place on the surface of the concrete lining a $\frac{3}{4}$-inch layer of asphaltic mixture. Owing to expansion and contraction during rapid changes of temperature, it was doubtful if the layer of asphalt had contributed much to the water-tightness of the structures. Concrete filter-tanks and clear-water basins, with a lining of puddle under the floor and around the walls, had been proved on a 14-day test, with the rise or fall of the water-level measured with hook-gauges, to be practically water-tight. In these structures, of course, the asphaltic lining was not used, nor had experience shown that it was necessary to secure water-tight work. The composition of the asphaltic lining called for in the specification was—

	Parts by Weight.
Neuchatel mastic	70
Bermudez asphalt	10
Clean, sharp grit and sand	20

This mixture would contain about 21 per cent. of total bitumen. The materials were heated in a kettle to about 280° F., and thoroughly mixed. It was found upon test that the mixture specified contained too high a percentage of bitumen, and was too

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soft for use on the slopes when exposed to the sun. Experiments were then made to ascertain what percentage of total bitumen in the mass would hold on the slopes without creeping when warmed by the rays of the sun. Concrete slabs about 2 feet wide and 4 feet long, placed at the same slope as the sides of the reservoirs, were covered with mixtures containing various percentages of bitumen, and maintained at a temperature of 100° F. Wires were placed across and above the slabs, and tacks were driven in the surface showing the downward movement of the asphalt. Owing to the fact that it was practically impossible to procure Neuchatel mastic in the Philadelphia market, except in very small quantities, Seyssel mastic was substituted, the above tests indicating that it was in all respects equal to Neuchatel. The several mixtures used contained as an average the following weights of materials for each batch :—

Material.	Floor and First Layer of Slope.	Second Layer of Slope.
	Lbs.	Lbs.
Seyssel mastic	585	598
Grit	315	332
Refined Trinidad asphalt	50	33
Refined Bermudez asphalt	50	37

These mixtures gave by analyses an average of 15·5 per cent. of total bitumen for the first mixture, and 13·2 per cent. of bitumen for the second mixture; the slopes requiring, of course, a stiffer mixture to prevent creeping during hot weather. The water-tightness of all structures forming part of the works for the improvement, extension and filtration of the water-supply of Philadelphia, was a requirement of the specification, and searching tests of each structure were made to show compliance therewith. Upon trial it was found that the west division of the reservoir did not come within the limits fixed by the Bureau of Filtration; the water was drawn off, the surface of the concrete above the line of asphalt was thoroughly dried, and several coats of silicate of soda (syrup) were then applied with an ordinary whitewash-brush. Each coat of silicate of soda was allowed to dry thoroughly before the next coat was applied, and this treatment was found to be very effective in reducing leakage through the concrete. The leakage of the west division of the Belmont reservoir upon final test was 36,000 gallons per day of 24 hours, or about 0·1 per cent. of its cubic contents.

Mr. H. C. Hill. This leakage, after allowing for evaporation, represented a fall of 0·02 foot per day in the water-level. The leakage of the east division of the reservoir upon final test was 21,000 gallons per day of 24 hours, or about 0·06 per cent. of its cubic contents. This leakage, after allowing for evaporation, represented a fall of 0·011 foot per day in the water-level.

Mr. J. W. Hill. Mr. JOHN W. HILL, of Philadelphia, felt that the Author, as well as his eminent predecessor, Mr. O'Connor, and his assistants, were entitled to great credit for the conception and successful completion of so bold an enterprise as the Coolgardie water-supply, which certainly deserved more than passing notice. Personally, he was chiefly interested in the pipe-line, presenting as it did novel features in the construction of steel-plate pipe, and suggesting the probable extended use of a new kind of steel pipe, in which might be developed the full strength of the steel plate within the elastic limit. A little more than 2 years ago he had occasion to prepare the plans and specification for Contract No. 28, known as "Lardner's Point pipe distributing-system," one of more than seventy contracts forming part of the works for the improvement, extension and filtration of the water-supply of the City of Philadelphia. Some thought was given to the possible use of locking-bar pipe such as was used in the Coolgardie scheme; but there being then no manufacturer in the United States who had had any experience in the production of pipe of large diameter of this pattern, and the information now contained in the Paper with reference to the Coolgardie pipe not being available, it was not given the same consideration as it would have if the matter were to come before him at the present time. The pipe-system embraced in Contract No. 28 consisted of four lines of 60-inch pipe, reaching from Lardner's Point pumping-station, the principal pumping-station of the new Philadelphia waterworks, to valve-chambers Nos. 8, 9, and 10 at the western terminus of the pipe-line at Torresdale and Kensington Avenues, a distance of approximately 14,606 feet. While it was contemplated to lay four lines of such pipe for the whole distance, for the present four lines were laid only along Robbins Street, a distance of 800 feet from the pumping-station; and from this point three parallel lines of pipe were laid along Tacony Street and Torresdale Avenue to valve-chambers Nos. 8, 9 and 10, a distance of about 13,802 feet. In taking tenders for this pipe due consideration was given to steel pipe, with and without double-riveted butt joints, and to cast-iron pipe. The conditions favouring steel pipe were;—

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1. Ductility of material, and capacity to resist shocks.
2. Rapidity with which the sections of the pipe could be furnished and made up in place in the trench.

The conditions favouring cast-iron pipe were:—

1. Known durability in service (cast-iron pipe had been used in Philadelphia for a period of over 85 years, and except tuberculation of the interior from the iron and lime salts in the Schuylkill river-water, had apparently suffered no serious injury in strength or carrying capacity during this time).
2. Greater smoothness of the interior of the pipe, and consequent reduction of frictional resistance, and increase of carrying-capacity.
3. Convenience of repair in case of rupture in service.
4. In laying such pipe provision could readily be made for expansion and contraction due to temperature-changes, although these, at the depth at which the pipe was laid after filling the trench, were necessarily slight.

Still another advantage sometimes found with cast-iron pipe, laid in the 12-foot length usual in the United States, was that considerable settlement of the bottom of the trench might occur without rupturing the pipe, or creating serious leaks at the lead joints. The terms of the contract under which tenders were received for both riveted steel and cast-iron pipe provided that the sections of the former should be subjected to a hydrostatic test of 180 lbs. per square inch at the place of manufacture, for such length of time as the engineer might deem desirable, to prove the tightness of the riveted and caulked joints. After the sections were made up in the pipe-trench, and before refilling it, they were to be again subjected to a hydrostatic pressure of 160 lbs. per square inch under the same conditions as to length of time of test. The contract further provided that each joint should be absolutely water-tight under these tests. The provisions with reference to cast-iron pipe were somewhat different. After a pipe had been inspected for casting and foundry-work, and carefully checked as to dimensions, thickness of material in the barrel, eccentricity of bore, and diameters of ends, it was to be dipped in the customary bath of hot pitch and linseed-oil, and when the coating had become dry and cold, the pipe was to be subjected in the proving-press at the foundry to an internal hydrostatic pressure of 300 lbs. per square inch for not less than 10 minutes. While

Mr. J. W. Hill. under this pressure it was to be subjected to a thorough hammer-test, and any pipe which showed leaking or sweating at the end of this time was to be rejected. After the pipe was made up in the usual lengths of 12 feet in the trench, the contract required that it should be subjected to a further pressure of 200 lbs. per square inch for a period of not less than 5 hours, this length of time being fixed to enable the inspectors to examine carefully each joint and length of pipe, and if there should be spongy places in the pipe-barrel which had escaped inspection at the place of manufacture, this time was deemed sufficient to bring the water to the surface and enable such spots to be marked and the proper remedy applied. The test in the pipe-line also permitted of a trial of such forms of special castings as could not be conveniently tested at the foundry. All special castings in this contract, excepting curves and man-hole castings, were 48 inches to 60 inches in diameter, and were made up in the valve-chambers with flanged joints. Tees and crosses were flanged castings. Forty-eight-inch to 60-inch taper-pipes were constructed with a flange on the small end and a socket on the large end to join with the spigot of a 60-inch pipe; and all straight castings were tested at the foundry under the same conditions as applied to the cast-iron pipe. At the time this contract was made there was only one hydrostatic pipe-press in the United States of sufficient capacity to apply the required test, and the two other foundries which were employed in the manufacture of the pipe were required to build presses of magnitude adapted to this size of pipe and to the proof-test provided for. At one time (1903) the rate of progress in the manufacture of the pipe was so slow that the city was on the eve of going abroad to buy part of the work from the pipe-foundries of Great Britain and Belgium; and it was a pleasure to state that the English pipe-foundries which were called upon in this matter were entirely willing to undertake to meet the conditions of the contract, which were more exacting than any previously laid down in the United States in work of this kind. In addition to the hydrostatic test at the foundry and in the pipe-trench, it was required that there should be cast from the ladle for each piece of pipe at least one tensile test-bar and two cross-breaking bars. The tensile bars were 11 inches long, and approximately 1 inch square, corrections being made for exact dimensions upon breaking the bar in the testing-machine. The cross-breaking bars were 26 inches long, and approximately 2 inches wide and 1 inch deep, and were broken by placing them flatwise on supports 24 inches apart, and applying the load at the centre. The conditions with reference to metal were that

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the tensile strength should be not less than 22,500 lbs. (10 tons) per square inch, and that the cross-breaking bars of standard dimensions and span should take a load of 2,280 lbs., equivalent to a maximum fibre-stress of 41,000 lbs. (18·3 tons) per square inch of section, and show a centre deflection of not less than 0·34 inch before breaking. When tenders were received for this contract it was found that the difference between the cost of constructing the work with cast-iron pipe and with riveted steel pipe amounted to about $186,000·00 (£38,130), an amount which in his opinion was too small to justify the risk incidental to laying in city streets these large lines of parallel pipes of material of less known durability than cast iron ; and it was gratifying to note that the tests of the pipe at the foundry and after laying in the trench had never developed a fracture. Out of more than 2,500 pieces of this pipe manufactured to the present time, not one had broken in the proving-press under pressures sometimes as high as 350 lbs. per square inch ; and naturally none had broken in the trench under the lower pressure of 200 lbs. per square inch, although the latter might happen should the pipe be injured in transportation. In centering these pipes, instead of following the usual practice of centering from the outside, after the first few lengths were laid this was done entirely from the inside; an inspector inside the pipe indicating by taps of a hammer on the barrel the direction in which the spigot was to be wedged in the socket to bring about concentricity of the spigot end of one pipe and the socket of the next. In the placing of the spigot of one pipe in the socket of the next, an allowance of $\frac{1}{8}$ inch was made for longitudinal motion, partly to compensate for such temperature-changes as might take place in the length of the line before the trench was filled, and partly to allow for inequality in settlement of the pipe after filling, which would not be so well provided for if the spigot of one pipe was brought solidly home against the shoulder of the hub of the next pipe. Valve-chambers were provided at intervals of about 4,000 feet, so arranged with line- and branch-pipe valves as to permit of cutting out of service temporarily any line of pipe which might be injured, and confining the supply during the period of repair to the remaining lines of the system. The worst condition that could arise in service would be the temporary loss of the use of one line of pipe between two valve-chambers, the remaining lines between the two chambers still being serviceable; while of course all the three lines—or four lines when completed—beyond the chambers nearest to the point of rupture would be unaffected by the break. Where the pipe entered a valve-chamber

Mr. J. W. Hill. it was reduced from 60 inches to 48 inches in diameter, partly because the manufacture of 60-inch stop-valves had not been as extensive, nor the use of valves of such size as well established, as those of 48-inch stop-valves, and partly because the height of such stop-valves would have materially increased the depth of the trench in which the pipe was laid. The number of chambers, and therefore of reductions in size from 60 inches to 48 inches in each line of pipe between the pumping-station and the terminal point at Torresdale and Kensington Avenues, was six, and the estimated loss of head by reason of the use of reducers from 60 inches to 48 inches, of 48-inch tees and crosses in the chambers, and of 48-inch stop-valves at each chamber, as computed by the formula given by Weisbach,[1] was less than 0·35 foot for a velocity of 7·39 feet per second through the 48-inch stop-valves and special castings, corresponding with a velocity through 60-inch pipe of 4·73 feet per second: an amount altogether too small to raise any doubt of the wisdom of the reduction of the diameter of pipe from 60 inches to 48 inches while passing through the valve-chambers, in view of the very large reduction in the cost of trenching by the use of 48-inch stop-valves. All valve-chambers were constructed with concrete floors and side and end walls. Upon the walls were placed rolled-steel beams, with arches sprung from the lower flanges of the beams, to support the roadway and paving which was carried over the chamber. Access to the chambers was provided through two manholes at corners diagonally opposite. The special castings in the valve-chambers, tees, crosses and taper-pipes, were made of open-hearth cast steel, having a tensile strength of approximately 60,000 lbs. (26·8 tons) per square inch of section, and the bodies of the 48-inch stop-valves, which were subjected to twisting and oblique stresses during test in the line and in service, were also steel castings, to reduce the chances of rupture. While the lines of pipe were tested to 200 lbs. pressure per square inch upon completion in the trenches, the working-pressure was limited to 125 lbs. per square inch by a series of 4-inch Crosby relief-valves, one of which was placed on each line of pipe where it entered a valve-chamber. In addition to the relief-valves on the pipe-line, each of the pumping-engines (of which there were twelve of 20,000,000 U.S. gallons daily capacity, or a total of 240,000,000 gallons) delivering into these lines of pipe was provided with a 12-inch relief-valve set to lift at 120 lbs. per square inch. Thus the probability of ever producing a water-ram sufficient to injure

[1] J. Weisbach, "A Manual of the Mechanics of Engineering, etc." vol. i. p. 872. London, 1877.

Mr. J. W. Hill. the pipe in service was extremely remote; but anticipating that rupture might occur under unaccountable conditions, and that with the large volume of water flowing through these lines of pipe, great damage might be done to the bottom of the trench, and probably to the parallel lines of pipe, the pipes were everywhere laid upon a concrete floor 9 inches thick, extending from side to side of the trench. It was thought that, if a rupture should occur in service, the washing out of the filling over a pipe would simply open the street and cut the banks at the side, but could not by any chance shift from their positions and injure the adjoining lines of pipe. In constructing the floor, which was laid after the pipe-lines were blocked in position and tested, a concrete cheek, formed as a monolith with the floor and extending up on the curve of the pipe for a vertical height of about 9 inches above the floor, was provided on each side, thus forming a cradle to guard against the lateral movement of the other pipes should one of the lines be fractured in service. In laying this pipe the contractors were required to construct a standard-gauge railway upon the side of the street, and to place thereon a travelling crane with a boom having a reach of 20 feet at right angles to the line of trench, and a capacity, with this reach, of about 10 tons. Each piece of cast-iron pipe weighed approximately 14,000 lbs. ($6\frac{1}{4}$ tons), and each of the 48-inch stop-valves weighed about 22,000 lbs. (9·8 tons), these representing the heaviest castings used in the work. The contract provided that cutting pipes for closures should not be done in the usual manner with a hammer and chisel, but with a machine which would cut the pipe at right angles to its axis, and leave the end of the casting true and smooth, and which would accomplish the work without the possibility of fracture of either section of the cut pipe. In planning this system of pipes it was thought that in time there would be considerable tuberculation of the interior from the iron and lime salts found in the water of the Delaware River, which would be the source of supply for Lardner's Point pumping-station; and manholes were provided in each of the 60-inch by 48-inch taper-pipes through which access to the lines of pipe between chambers could readily be obtained, for the purpose of removing incrustations, and scraping and recoating the pipe. The removal, subsequent to preparing the contract, of a line of 30-inch pipe which interfered with the placing of the first four lines along Robbins Street, indicated that 27 years' use had produced very little deposit and no tuberculation in the pipe, and the manholes which were intended to be placed midway between the valve-chambers were accordingly omitted, although those in the

Mr. J. W. Hill. taper-pipes were retained. Should occasion arise to examine the interior of these lines of pipe and remove obstructions, this could be accomplished by taking out of service one section of the pipe-lines at a time, between any two consecutive valve-chambers. These pipe-lines, with a calculated capacity of 240,000,000 U.S. gallons per day of 24 hours, were pumping-mains intended to convey filtered Delaware River water to several districts of the City of Philadelphia, which at the present time contained a population of about 1,100,000. The filtration of the Delaware water would be accomplished by sixty-five covered plain sand-filters each $\frac{3}{4}$ acre in area, constructed on the banks of the Delaware River, about 3 miles up-stream from the Lardner's Point pumping-station. In the valve-chambers each line of pipe was provided with a $2\frac{1}{2}$-inch relief-valve to be blown by hand. As the vertical undulations in the lines of pipe were not very great between their origin at Lardner's Point pumping-station and the terminus at Kensington Avenue, it was thought preferable to have these air-valves blown from day to day by an inspector, rather than to trust to automatic valves which might fail to act and probably subject the pipe-line to undesirable shocks by reason of accumulated air. At two points it was necessary to carry the pipes over creeks or large sewers, which necessitated the use of flanged pipe and the building of concrete chambers in the line of the sewer or watercourse, to confine the channels to definite widths. Two lengths of flanged pipe were bolted together in the centre of the span lengths, and acted as tubular horizontal girders of 16 feet clear span, of 60 inches internal diameter and 63·4 inches external diameter, calculated with a factor of safety of about 20, for a centre dead load of pipe, water, earth filling and pavement, of 47,256 lbs. (22 tons), and a maximum centre live load of 40,000 lbs. (17·8 tons). An interesting feature of this contract had been the necessity of moving under pressure a line of 48-inch cast-iron pipe about 1,194·45 feet long, which was one of the two pumping-mains from the original pumping-station (Lardner's Point No. 1). To place this pipe in its new position to accommodate the four lines of 60-inch pipe in Tacony Street required that it should be changed 11·5 feet in alignment, and 13·17 feet in level at its lowest point. In carrying out the work the deflection from the old elevation to the new was made on the first 200 feet of the line at each end, the remaining portion being run on a tangent. As about 30,000,000 U.S. gallons of water were being delivered daily through the 48-inch main, it being the principal pumping-main, it was impossible to take

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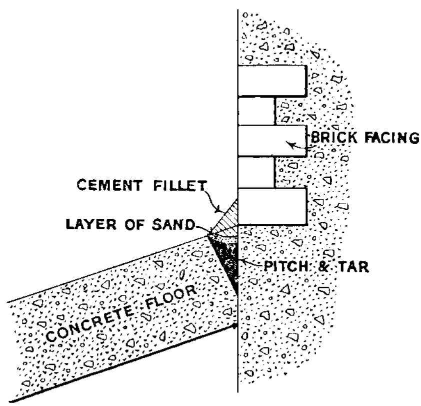
the line out of service, even for a short time, and careful preparations were made for moving it into its new situation while under pressure, and without stopping the pumping-engines. The centre-line of the main in its original position was 1·17 foot shorter than the calculated centre-line for its new position. After it was moved careful measurements showed that the actual draw of the joints had only been 0·93 foot, or 0·24 foot less than was expected. This was very evenly distributed throughout the one hundred joints, and the average movement of the pipe in each joint was slightly more than 0·11 inch. In order to guard against excessive pull in the joints, each pipe was marked before being moved, on the top and on each side; but owing to the fact that some of the pipes rotated 100 degrees, it was impossible to make proper reductions of the plus and minus readings. Small gauges were, however, used continually on these marks, and the lead rings in some cases were drawn out ½ inch or more; but these joints were not disturbed unless a leak occurred, when they were immediately recaulked. Before moving the pipe, the trench for its new position was excavated to line and gradient; excavations were then made under the pipe on about 200-foot sections, and the pipe was gradually lowered to its new gradient, being meanwhile thoroughly braced in position to prevent lateral movement. The greatest depression was reserved until the last, thus allowing any gain to work up to this point. After the pipe had been lowered to the new gradient, wooden skids, upon which iron strips were fastened, were placed beneath each length, and the pipe was then moved laterally into position with screw-jacks. To facilitate moving the pipe the iron strips were kept well greased. The time occupied in moving the pipe was about 1 month, and except for a few hours, when a cracked pipe was discovered, the line was never taken out of service, and was under a uniform pressure of 70 lbs. per square inch. The cracked pipe had probably existed since the main was originally laid, and to avoid delay steel bands were placed around it and tightened up. In the subsequent operations this length of pipe gave no further trouble. To avoid accidents and delays, rigid inspection was maintained both day and night, and men within easy hailing-distance were placed along the line to ensure the immediate closing of the valves at either end in case of accident. After the pipe had been relocated it was allowed to rest for a few days until it had assumed its final position, and all joints were then thoroughly recaulked. The Author's remarks on leakage were not quite clear in the distinction between leakage at the

Mr. J. W. Hill. longitudinal locking-bar joints, and leakage at circular lead joints. The Author stated that the lead joints were tight, but examination of locking-bar joints made in the United States had shown them to be water-tight under considerable pressure; so that the leakage referred to by the Author would seem to come from the lead joints where two sections of the pipe were brought together in the double bell. Mr. Hill's experience at Philadelphia had demonstrated that it was almost impossible with cast-iron water-pipe and circular joints to have absolute water-tightness; and that in a long line under a pressure of, say, 100 lbs. per square inch there was a measurable leakage. He would like to know the results of the Author's tests in this respect.

Mr. Hurse. Mr. A. E. Hurse considered that, provided the material used as ballast was of a hard, compact, and dense nature, and sufficient sand was used to fill the interstices, no object was gained by rigid adhesion to a fixed gauge in mixing concrete. Professor C. E. Sherman's experiments on the presence of clay in sand were well worthy of study, and it would be useful if the resistance to crushing of blocks of concrete, made with unwashed and with washed ballast or gravels of different compositions, could be investigated over a period of, say, 3 years, so as to decide this doubtful point. In his own experience on the construction of a covered service-reservoir of 5,000,000 gallons, built of concrete, the use of unwashed pit-gravel had been necessary on account of local difficulties: first a threatened action at law for the silting-up of the watercourses caused by the washing of large quantities of "terrace-gravel" excavated on the site, and secondly the distance and altitude of cartage of materials. The sand screened from the pit-ballast subsequently used was found to contain 8 per cent. of clay, and the aggregate was of good angular shape. The proportions of the concrete were 5 to 1, and there being sufficient sand in the ballast, none was added. During the construction of the walls with the concrete of unwashed ballast, sample masses were cut out at various points; these were found to be well set even after only a fortnight had elapsed since mixing and laying. He regretted that he had no record of the resistance to crushing, but the good quality of the concrete was also shown by the fact that flat arches of about 13 feet span and 13 inches rise, and 12 inches in thickness, were sufficiently set for the centres to be withdrawn after 14 days. Slight cracks occurred at the haunches, but these were cemented up and caused no trouble. In the first coat of rendering unwashed Leighton Buzzard sand, containing 2½ per cent. of impurities, was used, with no undesirable effects. Close observation of this reservoir for 4 weeks after filling had indicated very satisfactory results. The method of caulking cracks in the floor of a reservoir described at p. 142 was similar to a plan which had been adopted in the floor of a tank of 1,500,000 gallons capacity for a crack resulting from settlement, as illustrated in *Fig. 38*. Mr. Hurse.

Mr. G. H. LIST was of opinion that the salt-impregnated soil in which the thin steel pipe-line was laid would undoubtedly act injuriously on the metal and greatly shorten the life of the pipes. He had had large experience of pressed-steel sleepers in similar soil, and their life was very short. No doubt the coating described would act beneficially if perfect, but his experience was that in the handling of such pipes the best coating got damaged and allowed corrosion to start. The joints would be the most dangerous places, as here the coating had to be removed to ensure a good lead joint. Paying the joints with hot tar-mixture was sure to be carried out imperfectly in many cases, and so corrosion would get a start. The plan of wrapping and re-dipping, which was being followed at Dudley Port for gas-mains, would protect the body of the pipe, but the wrapping must be cut away for some distance at each end to allow the collar to be slipped on and the lead joint to be run and caulked, so that matters would be no better than before. If it were possible at moderate cost to surround the pipes as a whole with some impervious material, then corrosion would be prevented. He would suggest covering the pipes, after laying and jointing, with Callender bituminous sheeting, or preferably Portland-cement grout, thus protecting them as the lining of "tube" tunnels was protected. This might be done by means of a movable casing halved over the pipes, lined with thin parting-paper, cement grout being run in to give a coating about 1 inch thick. The machine designed and used successfully for caulking the joints was most ingenious; but it was bulky and doubtless somewhat awkward to use in the Mr. List.

Fig. 38.



Mr. List. confined space of a pipe-trench, and could only be used on collar joints with a double face. On socket joints such a machine would have to be firmly clamped round the pipe to enable it to press up the lead joint. On riveted pipes it could not be used, and even on the pipes described a certain amount of hand-caulking was necessary. He had just completed a big pipe-laying job, and at one time he had had difficulties about setting up the joints by hand. To overcome the trouble he had suggested the use of pneumatic caulking-tools—a small portable compressor on wheels, driven by an oil- or steam-engine, with sufficient length of hose to command a definite length of track each side, and ordinary long-stroke riveting hammers working on suitable sets. Owing to questions of first cost near the end of the job, and to the men becoming more reasonable, the proposal had gone no farther; but he thought it would have answered well, and would have been simpler to handle than the machine described. His experience agreed with that of the Author regarding the effect of a proportion of loam or clay on mortar. Much expense and delay had been caused recently on a work with which Mr. List was connected by having to go a long distance for sand, which was no whit better in effect than a sand close at hand which had a small proportion of loam in it. The best brickwork he had ever seen or ever done was on the Rapti Bridge near Gorakhpur on the Bengal and North-Western Railway, India. The mortar used was made of kunkur (slow-setting hydraulic) lime (2 parts) burned with wood in a running kiln, drawn hot, and put into a pan-mill with small native bricks (1 part) taken from old ruins. These bricks had been set in mud mortar, and they were not cleaned or selected in any way. Sufficient water was added, and the mixture was ground for 30 minutes and taken to the work. In 10 days it set so hard that it became monolithic in character, and when any portion had to be dismantled it could only be picked to pieces with a pickaxe or crowbar, the mortar being actually harder than the best "first-class" bricks. He presumed that there was no local limestone available on the Coolgardie scheme; but he thought much more might be done than was now done with the old-fashioned lime mortars, at much less cost, and with quite as good results as with Portland cement. Personally, in cases where work was neither under water nor required to set rapidly for tidal work, he would prefer to use good lime mortar in brickwork and for concrete. He was sorry to say that even in India, where cement was very expensive, and lime was both good and cheap, the tendency was to prefer cement. He knew of one instance where, with both limestone and coal on the spot, cement had been used at enormous cost, though he was certain that in the bulk of the work the indigenous materials lime and surki might have been used with safety and economy. The reason was not far to seek: it was unwillingness to take a little trouble, when anything else could be found ready to hand. Mr. List.

Mr. CHAS. F. MARSH remarked that it appeared from the description given in the Paper that only one thickness of caulking-roller was used for each joint. It would be interesting to know whether any trials were made varying the thickness of the tools, as was done with hand-caulking. It might have been better, at any rate, to pass a chisel-edged roller round first, to ease the lead from the barrel of the pipe, before beginning to set up. The average rate of progress for each pipe-laying gang, for the 3-months period, appeared to have been about thirty-eight pipes per 8-hour day, which was very good. It would be interesting to know what was the record for 1 day. The illustration of the lead-melting and joint-running apparatus did not give a very clear idea of the method of working, and it would be useful if the Author would give a further description. Mr. Marsh thought it would have been better to leave the coating on the end 6 inches of the pipes, as it would not materially affect the running of the joint, and in any case, with an 8-inch collar, 6 inches of scraping at each end of the pipe appeared to be too much. The recoating of pipes in position was never very satisfactory, and it was surely not advisable to destroy the coating more than could be helped. Could the Author state what mixture was used for the necessary repairs to the coating and how it was applied? It was curious that surprise was often expressed at the fact that the presence of small dust in sand used for mortar or concrete increased its strength; for all experimenters who had made tests on the value of sand of different sizes had found that dust improved the sand. Exhaustive experiments carried out by Mr. Feret at Boulogne showed that the resistance of mortars increased not only with the amount of cement used, but also with the combined volume of the cement and sand in a unit volume of mortar. Mr. Feret had found that, when even-grained sands were used, this combined volume became less and less as the grains became smaller, whilst it was larger for sands with varying grains than for those with even grains. For sands with varying grains, the combined volume of cement and sand was larger as the sand-grains became more mixed in size, being greatest when there were no medium-sized grains, and the ratio of large to fine grains was 2 : 1, the cement, of course, being included among the fine grains. By large grains those between $\frac{1}{5}$ and $\frac{1}{12}$ inch were meant; medium Mr. Marsh.

Mr. Marsh. were those between $\frac{1}{12}$ and $\frac{1}{50}$ inch; fine grains were those less than $\frac{1}{50}$ inch. Standard sand was medium-grained, the size of grain varying very little, namely, between $\frac{1}{20}$ and $\frac{1}{30}$ inch. Apparently the presence of a small percentage of clay in sand was not always detrimental, but this must depend on the general character of the sand, and each case should be carefully tested for as long periods as possible, since a sand might give better results for short periods than for long. In a Paper[1] read before the American Cement Users' Association at Indianapolis in January, 1905, Mr. J. C. Hain, Engineer of Masonry Construction to the Chicago, Milwaukee and St. Paul Railway, gave the results of some tests carried out by him on clayey sands, as against standard sands. A sand containing 7·7 per cent. of clayey matter, tested against a standard sand with 3·4 per cent. of clayey matter, gave results 40 per cent. and 30 per cent. higher than the standard at 7 days and 28 days respectively. The difference became slightly less with age, the result of 3-year tests being 20 per cent. higher than the standard. Another sand with 15·7 per cent. of clayey matter, tested against the same standard sand, gave results 10 per cent. higher at 7 days and 26 per cent. higher at 3 years. In another series of tests Mr. Hain made one set of briquettes with unwashed sand containing 6 per cent. of clay, and another set with the same sand after washing. The unwashed sand gave results averaging 25 per cent. higher than the washed sand over periods ranging from 7 days to 2 years. These results bore out those of Professor Sherman, mentioned in the Paper, which showed that sand containing 10 to 15 per cent. of clayey matter gave a mortar approximately 25 per cent. stronger than did clean sand. It would be dangerous, however, to generalize on the somewhat surprising results of these tests, and while it could not be denied that sands which appeared worthless might in reality make stronger mortar than clean sands, it would be extremely rash to use any such sand until careful and extensive tests had been carried out, and its superiority firmly established. If long-period tests could not be waited for, and if for any reason it would be economical to use an apparently dirty sand, it would not be advisable to employ it unless it proved to be at least 5 to 10 per cent. stronger than standard or other selected clean sand at 28 days. The construction of the floor of the Bulla Bulling reservoir in two layers would doubtless prevent cracks from extending right through, but if it had been formed in one layer having a

[1] See *Engineering News*, vol. liii. p. 127. Also Minutes of Proceedings Inst. C.E., vol. clxi. p. 396.

Mr. Marsh.

grillage of wire of a smaller mesh than that used, placed near the upper surface, Mr. Marsh believed the cracking would have been prevented, since it had been found that such reinforcement prevented the formation of cracks. It would be interesting to know why barbed wire had been used for the grillages. If it had been adopted with the idea that the barbs were necessary to give sufficient hold to the concrete, such a precaution was entirely unnecessary, as plain wires would have given all the hold required. He did not think that wire in the form of a cable formed a good reinforcement, since the concrete could not properly surround the strands and protect them against corrosion; and he believed that a closer mesh of single-strand wires would have been preferable. With the exception of those at the face, the wires in the vertical walls appeared to be unnecessary, since the walls were apparently not designed as being reinforced: in any case the wires in the centre would not be of much use.

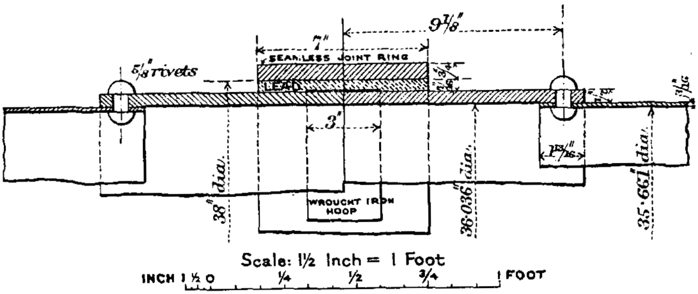
Mr. Moncrieff.

Mr. ALEX. B. MONCRIEFF considered that the Institution was indebted to the Author for a lucid and comprehensive Paper on the important Coolgardie water-supply, and personally he desired to thank him for an opportunity afforded him for examining the works, which had been finished in a workmanlike manner and without unnecessary ornament of any kind. Deep regret was naturally felt by the members of the profession in Australia that Mr. C. Y. O'Connor did not live to see his great work completed. Mr. Moncrieff's own experience indicated that the high price at which water was now sold in Western Australia could be maintained only under extraordinary conditions. In the other Australian States it was generally lower, while in South Australia the price of water from the principal supplies fell as low as 6*d.* per thousand gallons. The price which could be obtained for water governed the amount of capital which it was possible to spend in the construction of the works, and it was important to note that from the Coolgardie waterworks there was practically no reticulation, the scheme consisting of headworks, pumping-plant, and leading main only. This might have a vital effect on the ultimate revenue-producing power of the scheme. The Author had touched upon a burning difficulty confronting engineers in Australia when dealing with the question of gaugings of river-flow and rainfall. In that country there were practically no records extending over sufficient time to be of positive value in preparing schemes for large works, and greater responsibility than would otherwise obtain was thrown upon engineers in designing structures and calculating supply. The difficulty was such that it had to be experienced before it

Mr. Moncrieff. could be appreciated thoroughly. He looked upon the Paper as a valuable addition to the literature dealing with water-conservation in the outlying districts of Australia. The authorities of Western Australia had been fortunate in being able to obtain a clean catchment-area for their great reservoir. This had very frequently not been obtainable in Australia; for the catchment-areas having been alienated and occupied before water-conservation was considered, it was beyond the possibilities of the finances of the States to purchase them back; and in constructing large works Mr. Moncrieff had, where possible, adopted the principle of keeping the reservoirs off the main streams, allowing the flood-waters to pass, and only taking supplies into the reservoirs after the main streams had been scoured. In older countries filtration of this water would be demanded, but the method referred to had proved successful so far. There was not one instance of a large filtration-plant having been constructed in Australia, though there was a small scheme at Broken Hill, N.S.W. The character of the water from the Coolgardie catchment-area was generally recognized as good, but it might ultimately be found necessary to filter the supply for domestic purposes. No provision for meeting this contingency seemed to have been made in the location of the works. The concrete dam at Mundaring was of a very substantial character; but in dealing with almost similar circumstances elsewhere Mr. Moncrieff had preferred to provide a by-wash apart from the dam, as allowing of simpler construction and less liability of damage from excessive floods. He had also found it generally desirable to provide a tunnel in solid ground under the flank of the dam, and to have a separate water-tower, rather than to take the outflow-works through the concrete and build the water-tower as part of the main structure. This was a matter for individual judgment; but it had always appeared to him that to break the continuity of the concrete with an outlet-pipe, or to break the contour of the dam with the excrescence necessary for the construction of a water-tower, was liable to lead to rupture of the concrete under the stresses due to expansion and contraction in the very wide range of temperature obtaining in Australia. Referring to the pipe-line, it seemed to him that no engineer comparing riveted steel pipe with the locking-bar pipe adopted could hesitate to decide in favour of the latter. The punishment of the steel in punching, and the innumerable points at which possible leakage might occur, condemned the riveted pipe at once in comparison with the locking-bar pipe, which consisted of four pieces only. *Fig. 39* showed the method adopted in South

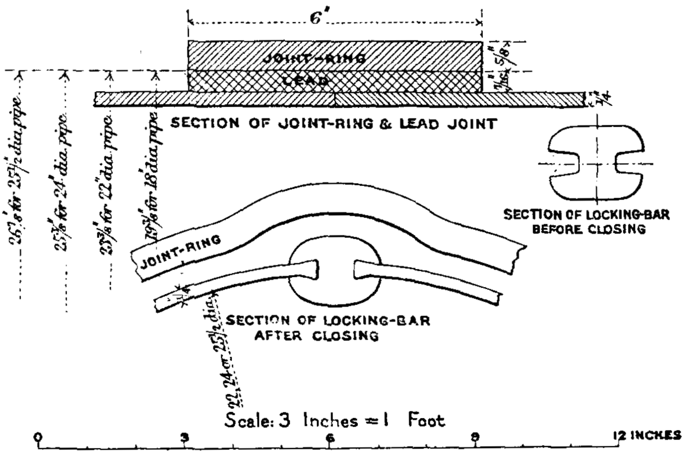
Australia in constructing and jointing a riveted steel pipe of similar diameter to that used for the Coolgardie works. He had found that the plain ring, allowing for through lead filling, as shown in *Fig. 40*, made an excellent connection for the pipes, and Mr. Moncrieff.

Fig. 39.



admitted of curves of comparatively small radius being followed in laying such a main, without specials. The difficulty anticipated from the flowing of the lead through the joints of the pipes was entirely overcome by the insertion of a temporary band of iron

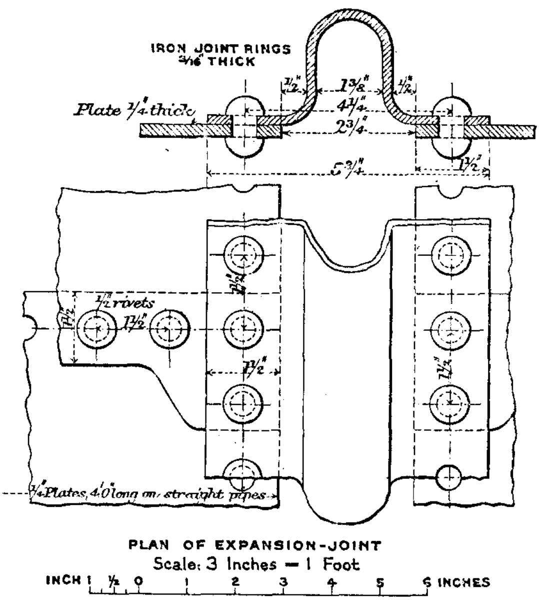
Fig. 40.



expanded in the pipe by means of a screw, and such a joint was certainly not more expensive to caulk than that adopted on the Coolgardie works, while the use of spun yarn—a perishable material—was entirely avoided. With regard to the method of

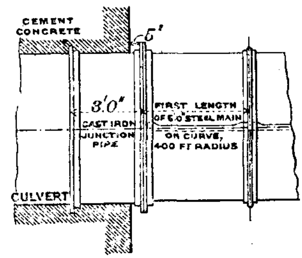
Mr. Moncrieff. making the joints, it was generally acknowledged that the caulking of the lead round the locking-bar required most care, and as this had had to be done by hand at Coolgardie he failed to see the advantage of caulking the balance of the joint by machinery. The prices quoted for hand- and machine-work seemed high. For laying a similar 26-inch main in South Australia, formed of $\frac{1}{4}$-inch plate closed with locking bars, the cost complete was £4,140 per mile; the cost of laying and jointing being for labour £153 per mile, while labour for caulking only ranged from 2*s.* 6*d.* to 3*s.* per joint. It was possible, however, that higher rates of wages had to

Fig. 41.



be paid in Western Australia. The use of locking-bar pipes, coated in a similar manner to those referred to by the Author, had proved a success in South Australia, in which State the new design of pipe was first adopted by Mr. Moncrieff. The Author had been particularly fortunate in being able to lay his main close to the railway, as carting the pipes over rough country was the common experience, in which case the locking-bar pipe again proved its superiority over the riveted. The methods adopted in laying to prevent the pipes from being injured by exposure in the trenches appeared to have been admirable, and it was satisfactory to note that the simple methods adopted had proved Mr. Moncrieff. satisfactory without the intervention of the expansion-joint, the introduction of which had been so freely discussed before the work was undertaken. In this connection he had found that for pipes of large diameter the ring joint was very effective where the pipes were exposed to the great variation of temperature obtaining in South Australia. A long length of pipe 6 feet in diameter manufactured as shown in *Figs. 41* and *42*, resting on cast-iron cradles, had remained in this position for many years without the slightest movement, although the temperature to which it was exposed had varied from freezing-point to 170° F. in the sun. The Appendix to the Paper was valuable, but had it been possible to publish more details of the cost, its value would have been considerably enhanced. Perhaps the Author would furnish these details, which would be of much interest to his professional brethren in Australia.

Fig. 42.



JUNCTION OF CULVERT WITH STEEL MAIN
Scale: 1" Inch = 8 Feet
INS. 12 6 0 1 2 3 4 5 6 7 8 FEET

Mr. George W. Rafter, of Rochester, N.Y., observed that it was Mr. Rafter. unusual to go more than 350 miles for a daily supply of 5,600,000 gallons—or indeed for any quantity; and the combination of gravity and pumping, with the 2-years'-storage requirement rendered the Coolgardie water-supply a project worthy of the best effort of the engineers employed upon it. While the Paper was valuable as a whole, the most interesting portion of it to Mr. Rafter was that relating to the yield from the catchment of the Helena River. The Author stated that the catchment-area above the reservoir at Mundaring was 569 square miles, and that, on the face of it, the allowance was excessive for 2 years' supply; but Mr. Rafter had found that the calculations in such cases were frequently insufficient, and whether or not a catchment of 569 square miles would supply 5,600,000 gallons daily, apart from leakage, was purely a question of what water ran into the reservoir. This the Author had foreseen; he had made provision accordingly; and the outcome justified his judgment. Great conservative care in dealing with this question was necessary in other regions besides Australia, because catchments yielded very different quantities of water for the same or nearly the same rainfalls. Frequently it was considered that when the rainfall sank

Mr. Rafter. below a certain minimum, the yield would be nothing. This minimum varied for different regions and conditions. In the case of the Helena River in 1902, the minimum rainfall from which the yield would be nothing was apparently somewhat less than 19·3 inches—on the supposition that the mean rainfall at Mundaring and at York truly represented that of the entire catchment-area, although there was perhaps some doubt on this point, as stated in the Paper. The ratio of yield to rainfall in that year was only 0·2 per cent., or the depth of off-flow on the catchment was 0·039 inch. In 1897 a rainfall of 24·5 inches gave a ratio of yield to rainfall of 0·34 per cent. (0·083 inch), while in 1900 a mean rainfall of 33·25 inches gave a ratio of 3·5 per cent. (1·6 inch). This was a small quantity for a rainfall of more than 33 inches, from the point of view of one familiar with yields in the United States, where, in the eastern part of the country, for this rainfall yields of about 8 to 15 inches might be expected. Apart from the evaporation observations given in Table IV. (p. 98), the preceding statement alone would indicate that evaporation from the Helena catchment-area was large. The gaugings showed that in 1897 and 1902 the ground-water off-flow of the Helena River must have been little or nothing. Mr. Rafter considered that there was a law, though a somewhat obscure one, governing the yield from catchments in all parts of the world; but this law was not yet very precisely determined, nor was it likely to be, because the absorptive nature of the soil of catchment-areas, evaporation, and other conditions differed so much, that every catchment-area was, so far as its yield as a whole was concerned, a law unto itself; and only when all, or nearly all, the conditions were known would it be possible to deduce a formula expressing accurately the relation between rainfall and yield. Nevertheless, there were some general principles which might be applied, although not admitting of much precision. For example, for the Genesee, Hudson and Croton rivers in the State of New York, the following data were available.[1]

Genesee River.	Inches. Rainfall.	Inches. Off-flow.
Yearly average	40·3	14·20
1895 (minimum year)	31·0	6·67
Difference	9·3	7·53

[1] Much information on the subject of the relation between rainfall and the yield of catchment-areas, including the data here given, will be found in Mr. Rafter's work "Hydrology of the State of New York." Albany, N.Y., 1905. [*Bulletin No. 85 of the New York State Museum.*]—SEC. INST. C.E.

Mr. Rafter.

	Inches. Rainfall.	Off-flow.
1894 (maximum year)	47·79	19·38
Yearly average	40·30	14·20
Difference	7·49	5·18
Comparing the maximum year with the minimum—		
1894	47·79	19·38
1895	31·00	6·67
Difference	16·79	12·71
Comparing the storage-periods—		
Average of storage-period	19·40	10·50
Storage-period of 1895 (minimum)	13·20	5·63
Difference	6·20	4·87
Storage-period of 1894 (maximum)	27·71	15·73
Average of storage-period	19·40	10·50
Difference	8·31	5·23
Comparing any two years—		
1892	41·69	15·42
1896	40·68	12·80
Difference	1·01	2·62
Comparing storage-periods:—		
Storage-period of 1892	19·84	9·38
Storage-period of 1896	17·84	9·25
Difference	2·00	0·13
Comparing growing-periods:—		
Growing-period of 1892	15·30	4·90
Growing-period of 1896	10·28	0·83
Difference	5·02	4·07
Comparing replenishing-periods:—		
Replenishing-period of 1896	12·56	2·72
Replenishing-period of 1892	6·55	1·14
Difference	6·01	1·58

Hudson River.

	Rainfall.	Off-flow.
Yearly average	44·21	23·27
1895 (minimum year)	36·67	17·46
Difference	7·54	5·81
1892 (maximum year)	53·87	33·08
Yearly average	44·21	23·27
Difference	9·66	9·81

Mr. Rafter.

	Rainfall (Inches)	Off-flow (Inches)
Comparing maximum year with the minimum—		
1892	53·87	33·08
1895	36·67	17·46
Difference	17·20	15·62
Comparing the storage-periods—		
Average of storage-period	20·62	16·10
Storage-period of 1895 (minimum)	15·79	11·68
Difference	4·83	4·42
Storage-period of 1892 (maximum)	24·95	22·50
Average of storage-period	20·62	16·10
Difference	4·33	6·40
Comparing any two years—		
1898	48·28	27·12
1891	42·96	20·56
Difference	5·32	6·56
Storage-period of 1898	22·80	16·81
Storage-period of 1891	20·69	16·59
Difference	2·11	0·22
Growing-period of 1898	13·52	3·24
Growing-period of 1891	13·49	2·07
Difference	0·03	1·17
Replenishing-period of 1898	12·19	5·27
Replenishing-period of 1891	8·78	1·90
Difference	3·41	3·37

Croton River.

	Rainfall (Inches)	Off-flow (Inches)
Average of storage-period	23·68	16·83
Storage-period of 1897	20·55	14·64
Difference	3·13	2·19
Storage-period of 1898	28·81	20·08
Average of storage-period	23·68	16·83
Difference	5·13	3·25
Comparing any two storage-periods—		
Storage-period of 1896	24·84	18·01
Storage-period of 1895	19·55	14·78
Difference	5·29	3·23
Storage-period of 1888	30·33	21·74
Storage-period of 1883	19·03	11·37
Difference	11·30	10·37

Mr. Rafter.

NOTES.—*Genesee River.*—This, and one of its tributaries, Oatka Creek, were gauged, with the exception of a certain period, from 1890 to 1898, inclusive. The sequence of these gaugings was as follows: From April, 1890, to November, 1892, gaugings of Oatka Creek (catchment, 27·5 square miles) were made; from December, 1892, to August, 1893, inclusive, gaugings were not kept of either Oatka Creek or Genesee River, and for this period the yield is computed approximately from the rainfall by an application of the principle indicated in the foregoing tabulation; September, 1893, to February, 1897, inclusive, the record is that of Genesee River at Mount Morris (catchment, 1,070 square miles); in March, 1897, the dam at Mount Morris over which the gaugings were made was carried away by a flood and the record for the balance of that year and for the year 1898 is deduced from the record at Rochester, where gaugings have been kept since 1893. The catchment-area at Rochester is taken at 2,365 square miles.

Hudson River.—In October, 1887, daily measurements of the flow of Hudson River were begun and have continued every working-day from that time to the present. These measurements are made at the dam of the Duncan Company, at Mechanicville, where this company works a paper-mill, using the entire flow of the river from a catchment-area of 4,500 square miles. In order to obtain the complete flows, a record has also been kept of the number, size, kind and discharge of turbine water-wheels in use during the same period. The flow of Sundays and holidays, when no observations were taken, has been assumed as a mean between the preceding Saturday and the following Monday. The detailed Table of Hudson River yield, from which the foregoing tabulation is drawn, is for the 14 years 1888–1901, inclusive.

Croton River (catchment, 339 square miles).—This is appropriated as the water-supply of the City of New York. Records of the yield have been kept from 1868 to the present time, but the foregoing statements include only the period 1877–99, inclusive.

The statements are made with reference to a "water year" extending from December to November, inclusive, and divided into a storage-period (December to May), a growing-period (June to August), and a replenishing-period (September to November).

On the Genesee catchment, an average of 10·5 inches runs off in the storage-period, 1·7 inch in the growing-period, and 2·0 inches in the replenishing-period. On the Hudson, an average of 16·10 inches runs off in the storage-period, 3·45 inches in the growing-period, and 3·72 inches in the replenishing-period. On the Croton, an average of 16·83 inches runs off in the storage-period, 2·57 inches in the growing-period, and 3·43 inches in the replenishing-period. The average rainfalls corresponding to these yields can be obtained by examining the tabulation. The Genesee River is distant from the Hudson and the Croton about 240 miles.

The foregoing figures related to rivers in the eastern part of the United States, where annual rainfalls ranged from a minimum of less than 20 inches to maxima of over 60 inches. The rainfall and yield of the Helena River might be contrasted with those of the Loup River in Nebraska. The catchment-area of the Loup River was 13,542 square miles. It issued from a sloping, gently rolling country, with a soil consisting largely of porous sand, into which a portion of the rainfall sank, finally appearing as ground-water flow into the stream. The river was 250 miles in length, Mr. Rafter. elevated 1,500 feet above sea-level at its mouth and about 3,400 feet at its source. The average length of the catchment-area was 230 miles and its width 59 miles. The following were the rainfalls for the years 1891–98, inclusive, as kept at seven somewhat irregularly distributed stations in or near the Loup River catchment-area. The elevations of the stations above sea-level were given under the names, in feet.

Year.	Ainsley. (2,307.)	Bassett. (2,323.)	Burwell. (2,180.)	Lexington. (2,385.)	North Loup. (1,967.)	North Platte (2,841.)	Ravenna. (2,008.)	Average of Seven Stations.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
1891	30·42	24·75	25·20	37·46	30·96	23·36	35·92	29·73
1892	25·36	24·46	21·85	26·71	22·56	20·37	25·60	23·70
1893	15·88	15·74	15·69	24·79	15·46	13·16	18·13	16·98
1894	10·81	12·40	12·77	15·37	14·17	11·21	15·67	13·20
1895	20·94	18·32	20·22	23·94	22·88	14·58	20·26	20·16
1896	21·32	20·07	20·71	30·03	30·37	16·52	27·50	23·79
1897	27·13	25·86	17·70	30·85	28·10	17·09	32·75	25·64
1898	17·86	17·78	18·18	22·80	16·50	15·54	18·50	18·17
Mean	21·21	19·92	19·04	26·49	22·62	16·48	24·30	21·42

The following Table gave the approximate yield, in inches on the catchment-area, of Loup River at its mouth for certain months for the years 1895–1901, inclusive. The yield of this stream was stated to be, within limits, uniform from month to month, and it was on this basis that the column, "Proportionate quantity per year," had been computed.

Year.	April.	May.	June.	July.	August.	September.	October.	November.	Measured Quantity for Indicated Months.	Proportionate Quantity per Year.
	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.	Inches.
1895	0·23	0·25	0·30	0·18	0·19	0·20	0·21	0·25	1·81	2·72
1896	0·34	0·25	0·25	0·23	0·22	0·20	0·23	..	1·72	2·95
1897	0·30	0·21	0·20	0·22	0·15	0·16	0·36	..	1·60	2·74
1898	0·23	0·30	0·33	0·17	0·23	0·17	0·21	0·22	1·86	2·79
1899	0·27	0·26	0·32	0·18	0·20	0·18	0·16	..	1·67	2·86
1900	0·28	0·29	0·29	0·28	0·29	0·28	0·23	0·23	2·18	3·27
1901	0·35	0·39	0·35	0·24	..	..	..	..	1·33	3·99
Mean	0·29	0·28	0·29	0·21	0·21	0·20	0·23	0·23	1·74	3·05[1]

[1] Approximate.

The foregoing rainfalls and yields of Loup River indicated that the minimum rainfall which might occur without any accompanying yield was considerably less than in Australia, and Mr. Rafter. might be not more than 8 to 10 inches. This difference was probably due largely to difference in evaporation, absorptive capacity of the soil, and other physical conditions, although the evaporation in Nebraska was large. At Lincoln, in 1895, it was 48·4 inches; at Omaha, in the same year, it was 54·6 inches; and at North Platte, 48·6 inches. These evaporations were from a free water-surface, as determined by pan measurement at Lincoln, and computed from the readings of wet- and dry-bulb thermometers at Omaha and North Platte. The velocity of the wind was an important element in evaporation, and the following data on this point referred to Omaha for the calendar year 1895:—Average velocity of wind for the month of May, 10 miles per hour; for the month of July, 6·9 miles per hour; and for the entire year, 8·5 miles per hour. In Southern California the conditions approximated somewhat to those in Australia in the region discussed by the Author. As illustrating this, some phases of the yield from Sweetwater River, which had a reservoir upon it, with a catchment-area of 186 square miles above the reservoir, might be considered. The following records were for a "water year" from September to

Year.	Rainfall.	Off-flow.	Evaporation.	Wind.	Temperature of Air.
	Inches.	Inches.	Inches.	Miles per Hr.	°F.
1892-93	20·0	1·61	49·6	4·9	61
1893-94	14·8	0·14	48·7	5·1	60
1894-95	27·1	7·12	46·3	5·2	59
1895-96	19·5	0·101	45·2	5·2	61
1896-97	24·9	0·665	..	5·3	61
1897-98	18·2	0·0005	61·9	5·9	60
1898-99	18·5	0·025	..	5·8	59
1899-00	19·1	..	..	5·7	61
1900-01	23·7	0·087	..	..	62
1901-02	20·3	..	..	..	..

NOTE.—The rainfall is taken as a mean of three stations, as follows:—At two stations in the catchment of Sweetwater River, namely, at the reservoir (elevation 250 feet) and 25 miles east of the reservoir (elevation 3,500 feet), and at a third station (elevation 4,800 feet) 3 miles from the watershed-line between the catchments of the Sweetwater and San Diego rivers. Probably the rainfall of this third station is excessive, and certain corrections have been made which render the deductions somewhat approximate.

August, inclusive, as most nearly agreeing—according to Lippincott's California Hydrography—with the natural division for this

Mr. Rafter. region. The Table showed that for the conditions of Sweetwater River in Southern California the limit of rainfall with no yield varied between about 15 inches and 19 or 20 inches, the yield depending upon the distribution of the rainfall, evaporation, absorptive condition of the soil, and other elements. While the preceding illustrations were rather general, nevertheless, taken in conjunction with the Paper, they fairly substantiated the statements that the yield, while roughly proportional to the rainfall, was not in very precise relation thereto; and that the quantity of rainfall required to produce some off-flow varied in proportion to evaporation, physical conditions of the catchment, etc., in different parts of the world.

Mr. Smith. Mr. CHARLES W. SMITH thought the Coolgardie waterworks might certainly be characterized as one of the boldest schemes ever carried out; and considering the paucity of data regarding rainfall and river-flow at the time of its inception, and the unsatisfactory ratio of yield to rainfall, as ascertained after the commencement of the work, the designers were to be congratulated on the fact that the storage-reservoir had been satisfactorily filled. Engineers with Australian experience, however, and with a knowledge of the variableness of the rainfall in that country, would readily concede the wisdom of providing storage capable of conserving sufficient flood-waters to tide over any protracted period of drought, certainly not less than would suffice for 2 years. The Author anticipated that possibly there might be a flood-discharge over the weir-crest of a depth of 5 feet, but did not state how he arrived at this conclusion. In the absence of accurate information as to the magnitude of floods, what assurance was there that any depth estimated on insufficient data might not be largely increased, as had been Mr. Smith's own experience at the Laanecoorie Weir, on the Loddon River, Victoria? In that case the maximum flood-discharge had been inferred from gaugings made at low states of the river only, and checked by a formula applied to the drainage-area. This had given a maximum volume of $1\frac{1}{4}$ million cubic feet per minute; but actual flood-measurements, made after completion of the weir, had proved it to be as much as $2\frac{1}{4}$ million cubic feet per minute for floods of a few hours' duration. For the passage of the smaller volume only, a depth of 8 feet on the sill had to be reckoned with, and in order that no damage might result from the too sudden discharge of flood-waters over the weir, it had been decided to keep the crest of the masonry 5 feet below full-supply level, and to introduce automatic flood-gates to close the space up to that height. These gates had been designed

Mr. Smith.

somewhat on the lines of those described by Mr. Chaubart.[1] The installation, which had been in work for about 12 years, was most satisfactory, and had allowed the maximum floods to pass without damage either to weir or to flank-works. The adoption of this system appeared to Mr. Smith to be the wiser course when dealing with weirs of this class, where the flood-waters were of uncertain volume. So large a portion of the capital cost of the Coolgardie scheme had been expended on the pipe-line, and the life of this main depended so much on the value of its protective coating, that a few words as to Mr. Smith's experience of the Sydney water-supply works might be of some interest. There many miles of steel pipes were laid, of both the riveted and locking-bar type, and ranging from 72 inches to 8 inches in diameter. In all the coatings applied Trinidad asphalt had been an invariable ingredient, in proportions ranging from 75 to 50 per cent. In the earlier contracts for these pipes coal-tar had been specified to be used in conjunction with the asphalt in equal parts, but later maltha, in the proportion of 25 to 50 per cent. had been substituted for coal-tar. Speaking generally, none of these mixtures could be pronounced a success. Where pipes were laid above ground, as in the case of 9½ miles of 72-inch pipe (forming duplicate mains between Pipe Head and Potts Hill balancing-reservoir), the exterior coating had proved an absolute failure. One of these mains had been scraped and cleaned, and recoated with cement wash, which also had proved ineffective. Recent experiments with a paint composed of boiled linseed-oil and Portland cement in the proportion of 1 gallon of oil to 5 lbs. of cement, as used by the Public Works Department of Queensland for painting water-tanks both inside and outside, had proved so satisfactory that its extensive use, more especially for outside coatings, had been decided on.

Mr. Strange.

Mr. W. L. STRANGE observed that the weir forming the storage-reservoir was one of the highest in existence, while the length of the rising-main and the magnitude of the pumping installation were not equalled in any existing waterworks. The work had been carried out under very difficult conditions in a manner which reflected the utmost credit upon all concerned. The scheme was probably unique in regard to the very small amount of the yield from the catchment—0·20 to 3·50 per cent. of the annual rainfall of 19·3 to 33·25 inches. The originators of the project were to be congratulated on forecasting the amount of the yield so accurately. Apparently the larger part of the catchment-area was very flat as

[1] *Annales des Ponts et Chaussées*, 1855, pt. ii. p. 230.

Mr. Strange. well as very absorptive. The test of the amount of evaporation, made at the reservoir, indicated that the loss on this account, in ordinary circumstances, should be moderate. The remaining factor of the disposal of the rain-water—percolation—should therefore be high, were it not for the densely wooded nature of the catchment-area which would reduce it by increasing the evaporation. Probably the true explanation of the small amount of the yield was, as pointed out by the Author, that the rain-gauge stations registered falls greatly in excess of the average amount. The experience gained from the catchment-area confirmed that obtained in other countries, namely, that it was far better to depend for a gathering-ground upon a smaller area with abundant rainfall than upon a larger one with deficient precipitation. With regard to the proposal to supplement the natural catchment-area by diverting the off-flow from part of the neighbouring well-watered country into the reservoir, it must be remembered that catch-water drains could not be economically made of sufficient size to carry the whole of the off-flow from storms, which were the chief sources of replenishment. The advisability of constructing subsidiary reservoirs to feed the main reservoir might therefore be considered. Possibly it would be practicable to increase the yield from the natural catchment by denuding it, partially or wholly, of its timber. This should improve the potable quality of the water, although it might lead to an increase of silt-deposit in the reservoir. It was not clear from the drawings why the weir had been designed with so high an overfall, seeing that this had apparently entailed the thickening of the section of the structure. It would seem from Figs. 6, Plate 1, to have been possible to provide (with or without undersluices, and with a somewhat increased depth of overflow) a waste-weir of sufficient discharging-power and with a low overfall at the south end of the dam. Doubtless, however, this matter had been fully considered by the designer of the work. In any event, the provision of undersluices would have saved the prolonged flow over the weir which apparently would occur in all years of good rainfall. As was so frequently the case, the foundations of the work had proved worse than anticipated; for this reason, when estimating the cost of such structures, it was always advisable to allow a large amount for contingencies. The foundations were very irregular in cross section, and it was fortunate that this had not led to any cracks due to unequal settlement. The Author did not explain why the outlet was placed at practically the deepest point of the longitudinal section, where also the cross section of the foundations was very irregular, nor why the lowest outlet-valve was located so near to the base of the weir. The contents of the reservoir at this level were inappreciable, and in a few years' time they would probably be much decreased by silting. By raising the sill, apparently a safer position for the outlet could have been selected, and the pumping-lift would thereby have been diminished. The vertical intervals at which the outlet-pipes had been placed were unusually large, and it would have been an improvement if arrangements had been made to draw off the supply at more numerous points. The permanent object of having a scour-pipe at the base of the outlet was not clear, as this would not enable any appreciable amount of silt to be removed from the reservoir-basin, nor could it aid greatly in diminishing the flood-discharge over the weir. The selection of the locking-bar pipe had resulted in much saving, but it was a somewhat bold measure to adopt it on so large a scale with only the limited experience of a very much shorter main as a guide. The Author did not state whether any form of reinforced concrete main had been considered; most likely, at the time, sufficient experience of this form of construction had not been gained. Although it would probably have involved more pumping-stations, so as to reduce the pressure on the main, a pipe of this kind would seem to be peculiarly suited to a line of country heavily charged with corroding salts. It was noteworthy that the long length of the main had exercised a purifying influence on the quality of the water. It was a question, however, if filtration before the water was admitted into the rising-main would not have been advisable, in regard to the population served, as well as lessening the incrustation of the pipes. The "sand cuts" mentioned by the Author were interesting. Presumably the escaping water was partially confined by the filling over the main, and was thus able to give the sand abrading-power. The Author did not say if any permanent arrangement had been made to prevent this damage from occurring: surrounding the pipes with gravel or clinker might be a remedy. Mr. Strange noticed that the total storage-capacity of the reservoirs *en route* and at the end of the main was 28·68 million gallons, or 5 days' supply, of which 12 million gallons, or a little over 2 days' supply, was at the end; this seemed a small allowance for so long a length of main, but it would doubtless be ample as long as the daily consumption remained as low as it was at present. Mr. Strange.

Mr. GEO. B. WILLIAMS thought that, considering the very small margin between the loss from evaporation and absorption and the rainfall, and also the fact that the off-flow depended almost entirely on the few very wet days in the year and on the volume Mr. Williams.

Mr. Williams. of water falling on those days, the irregularities in the ratios of the yield to the total annual rainfall were not surprising. If the average percentage were taken for a series of years, a figure could no doubt be obtained which would be approximately correct for any series of years, providing the periods taken in each case were long enough. It would have been more satisfactory if, when dealing with the discharge, some statistics as to the rainfall on the watershed had been available. Unfortunately no rain-gauges appeared to have been fixed, and this question was dealt with in a perfunctory manner. The reasons given by the Author for assuming that the rainfall on the upper portion of the watershed was less than at York hardly appeared conclusive. If the average loss from evaporation and absorption could have been obtained, expressed in inches over the whole watershed, the result would have been of considerable interest. From Table I. (p. 97) and Table III. (p. 98) and from the map (Fig. 2, Plate 1), the rainfall on the watershed below the weir could be estimated; and for the 3 years 1899–1901 the average loss from evaporation and absorption appeared to have been about 31 inches for the larger watershed of 50 square miles, and 26½ inches for the smaller area of 10 square miles. These losses seemed somewhat large when the discharges were compared with those from the catchment-areas of the Nepean and Cataract Rivers near Sydney.[1] For 6 years the average loss amounted to 25½ inches for the Nepean watershed and 28½ inches for the Cataract. It would be supposed that the loss on the Helena watershed would be less, for the rain all fell in the winter months; and it would be reasonable to expect it to be not much more than 20 inches per annum. In the absence of accurate data a nearer estimate than this could not well be obtained, and it might be somewhat wide of the mark. Assuming that 20 inches was the average annual loss, it would be found from Tables I. and III. and from the Table on p. 57 that the average rainfall for the whole watershed for the 25 years, 1876–1901, would have been nearly 21½ inches, which was less than the mean of York and Mundaring, but more than the Author's figure of 18½ inches on p. 56. This latter figure appeared to be the result of pure guess-work.

Mr. Phipps Williams. Mr. GEORGE PHIPPS WILLIAMS, having had the pleasure of intimate acquaintance with the late Mr. O'Connor during his career in New Zealand was glad of an opportunity of expressing his admiration for the boldness and originality displayed throughout the magnificent scheme designed by him and described in the Paper. Mr.

[1] Minutes of Proceedings Inst. C.E., vol. lxxv. p. 176.

Mr. Phipps Williams.

O'Connor's plans exhibited the same breadth of view and the same mastery of detail for which he was noted when on the goldfields of the west coast of New Zealand, where his water-races and other works remained as monuments of his professional skill. The Paper seemed to deal a death-blow to all arbitrary formulas for the discharge from catchment-areas in relation to the rainfall, or indeed to any formula at all. Countries varied so much in local conditions that only the closest possible study of all the factors in each case could give results that were even fairly approximate. In the Canterbury Plains of New Zealand, large areas had no visible off-flow, and some of the rivers had an underground flow greater than that above ground. Some of the former found outlet as springs on the tops of downs and on slopes lying on the reverse side of the watershed from the catchment-area which supplied them. Some of the flow found its outlet at the bottom of the ocean, having been previously partially tapped by the numerous artesian wells in the neighbourhood of Christchurch. In the present case, where only 0·2 per cent. of the rainfall in 1902 was discharged at the weir, evaporation and not percolation seemed to have been the factor to which the enormous loss was ascribed. There was apparently no percolation at the Mundaring reservoir, though it had been anticipated; but it was not clear whether percolation might not cause loss through fissures elsewhere within the catchment-area. Another point raised by the Paper was an important one, affecting all engineering undertakings, especially in the Colonies, where economic development was often rapid. It was the question how far it was economically politic to design works ahead of actual requirements. In water-supply schemes it was no doubt sound hydraulic practice that a main should have the full capacity of the expected ultimate discharge, and circumstances might perhaps warrant a like course for pumping-station buildings; but with regard to the pumping-machinery itself, it was clearly advisable in a tentative scheme of this sort to cut down expenditure until prospects of increased consumption warranted further outlay, provided sufficient reserve power had been allowed for breakdowns. In the present case it appeared that, although the pumping-engines had been erected about 3 years, the consumption of water had never yet exceeded one-fourth of the amount which they were designed to supply. Their total cost was set down as £290,000; so that, including a proportional part of the reserve of power as estimated, the sum of £72,500 would have sufficed to cover the cost of all requirements and contingencies to date. Further, the actual lift was also much less; because, taking the

Mr. Phipps Williams. discharge at 1,400,000 gallons per day, the velocity in a 30-inch pipe would not exceed 0·53 foot per second, which would reduce the frictional head to an almost nominal amount—something less than 3 inches per mile; or taking it at 80 feet for the 308 miles, and allowing 50 per cent. for deterioration of pipes and waste head, it would amount to only 120 feet. The total natural head, including height to be surmounted and losses at reservoirs, was 1,500 feet, and this added to the frictional head as above gave a total head of 1,620 feet only, which was exactly three-fifths of the total estimated head of 2,700 feet for the full discharge of 5,600,000 gallons. Thus the cost of the engine-power required for the present maximum load might be further multiplied by that fraction, making a reduction again from £72,500 to £43,500, including the same proportion of reserve power as had been provided. This sum subtracted from £290,000 gave £246,500 as the amount by which the cost of pumping-machinery exceeded that of present requirements. The interest on this sum at 4 per cent. amounted to £9,860 a year, and assuming the engines to have been erected 3 years, it appeared on the face of it that at this rate a sum of nearly £30,000 might have been saved to date on this item alone. No doubt in any undertakings like this the engine-power should be kept well ahead of present requirements, even after allowing ample reserve power; but the margin allowed here seemed excessive, and he gathered from the Paper that this excessive first outlay was not contemplated in Mr. O'Connor's original estimates.

The Author. The AUTHOR, in reply, observed that he had perused the Correspondence with much gratification, and he felt that waterworks-engineers were especially indebted to those who, in addition to comment, had furnished valuable data regarding water-mains and the yield from catchment-areas. He would ask for consideration when dealing with some of the comments, as his hands had not been altogether free in construction, owing to orders for material already placed. There were naturally features of the works in which he himself did not concur; and although he had altogether avoided showing this in the Paper, he might not be able in reply to put the case on some points as favourably as his predecessor would have desired. The restricted demand for water, as compared with the original estimate, had been dealt with fully in his reply to the Discussion. The initial report on which the Western Australian Parliament sanctioned construction was of a very general character, so that although the money estimate as a whole was intended to be and had proved practically correct, the details could not be expected to be similarly correct. Engineers of experi-

The Author. ence in responsible colonial positions were aware that the large number of reports which an engineer-in-chief had to submit, on proposals of every description, rendered it impossible to conduct a department economically unless initial schemes were kept of a general character; and therefore in reply to Mr. Phipps Williams it was enough to add that, although the allowance for pumping-machinery in the initial proposals was comparatively small, there was no doubt in the Author's opinion that Mr. O'Connor was right in concurring in a larger allowance for friction-head, etc., and a corresponding increase in pumping-machinery. The works had been carried out by departmental labour, not by contract, and designing had proceeded almost *pari passu* with construction, so that alterations had been possible, except of course in cases such as that of the pumping-machinery, for which orders had been placed early. As stated in the Paper, a considerable amount of reticulation had been carried out, domestic and other services having been furnished throughout in the townships of Coolgardie, Boulder, Kalgoorlie, Boulder Mines, Southern Cross and Northam, and those of Midland Junction and Guildford were now in hand. But as the original proposals did not include reticulation, and as there was nothing very special in these works as carried out, except perhaps the use of a positive meter on every service, he had not entered on any description in the Paper. This has evidently misled Mr. Moncrieff, whose remarks also as to the high price of water in Western Australia could be held to apply only to the Coolgardie scheme. Elsewhere in that State prices were lower—in fact, below those of other parts of Australia; for while the water-rates in Melbourne, Sydney and Adelaide were respectively 6*d.*, 7*d.*, and 1*s.* in the pound, and while the measurement-rates per 1,000 gallons at these places were respectively 1*s.*, 1*s.*, and 6*d.* to 1*s.* 3*d.*, on the other hand the Author had been fortunate in obtaining sanction for the corresponding rates at Fremantle, the principal port of Western Australia, to be reduced to 6*d.* in the pound, and from 1*s.* per 1,000 gallons generally to as low as 4*d.* for certain purposes. Even at these rates there was a handsome return on the capital invested: moreover, in the western State there were several possible irrigation-schemes from which water could be profitably sold at a low figure. It was very interesting to note that the American figures confirmed the small yield possible from catchment-areas in climates such as that of Australia, so different from more humid places. Mr. Geo. B. Williams's statement, that it would have been more satisfactory to obtain statistics as to the rainfall on the watershed before

The Author. beginning work, was no doubt correct in theory when applied to the best-known portions of old countries in the present day; but such procedure in the Colonies would immensely retard development, and would certainly have sadly hampered the great English pioneer engineers of the last century. Although it was out of the question to expect accurate or even regularly correct results by forecasting where long records were non-existent, still some method of estimating was necessary; and the results detailed by Messrs. Folwell, Hering and Rafter seemed to fit the Author's theory better than that of the simple percentage reduction mentioned by Mr. Bruce and referred to hereafter. The point was one of the greatest possible importance, and the Author ventured therefore on the following amplification. The methods of designing on the basis of the yield might be divided into four, namely: (1) when very ample quantities were available and no storage, or very little, was provided, as when a spring or perennial river was tapped; (2) when a whole year's demand was considered and storage was provided accordingly; (3) when a succession of dry years—generally three—were taken as the basis, and the storage was intended to equalize the yield in this period; and (4), the worst case of all, when sufficient reservoir-capacity had to be provided to render the surplus of years of heavy rainfall available for use in dry years over a long period, perhaps 20 years or more. Mr. Fuertes's interesting and valuable contribution regarding minimum yields affected only the conditions detailed under (1) and (2). Of the other gentlemen who had written on this point, all except Messrs. Folwell and Hering appeared to have dealt with it under the comparatively fortunate circumstances detailed under (3), and of course regarding this there were valuable data already extant, as, for instance, Sir Alexander Binnie's Papers on rainfall;[1] but even in the British Isles and other well-watered and humid countries, owing to the increasing demand for domestic, power, and other purposes, the time was coming—if indeed engineers had not already been faced with the question—when the problems must very often be solved under the conditions detailed under (4), and be solved more economically than was possible under those of (3) only. In the Author's experience alone the economies effected by allowance for long-date storage had ranged in certain cases from

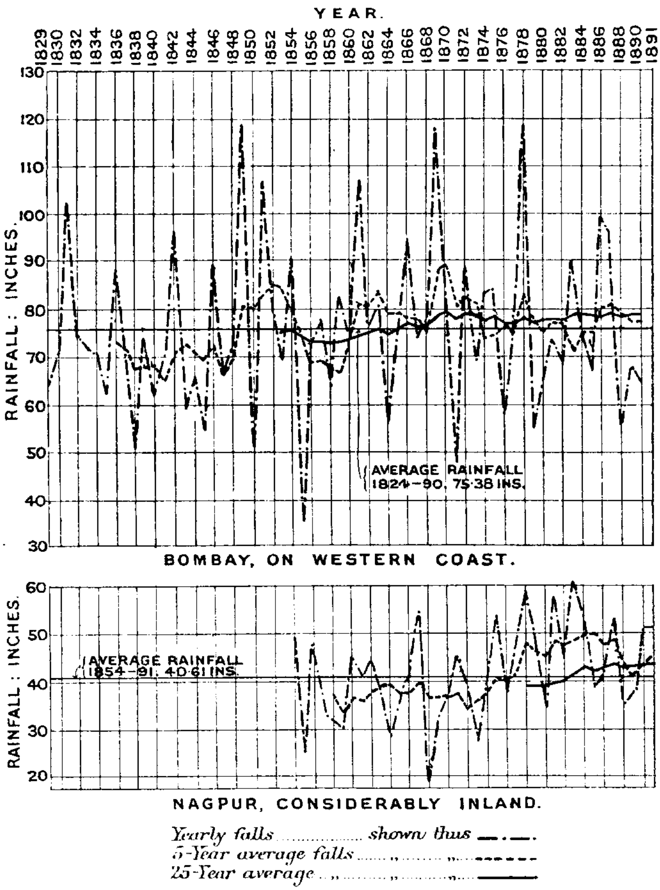
[1] "The Nagpur Waterworks; with Observations on the Rainfall, the Flow from the Ground, and Evaporation at Nagpur; and on the Fluctuation of Rainfall in India and in other places." Minutes of Proceedings Inst. C.E., vol. xxxix. p. 1.

"On Mean or Average Annual Rainfall, and the fluctuations to which it si subject." *Ibid*, vol. cix. p. 89.

£25,000, in one instance, to a 30-per cent. reduction of the cost per 1,000 gallons in another. It was in the hope, therefore, that, as suggested by Mr. Fuertes, other data might be forthcoming later on regarding long-date results, that the Author drew attention to

The Author.

Figs. 42.

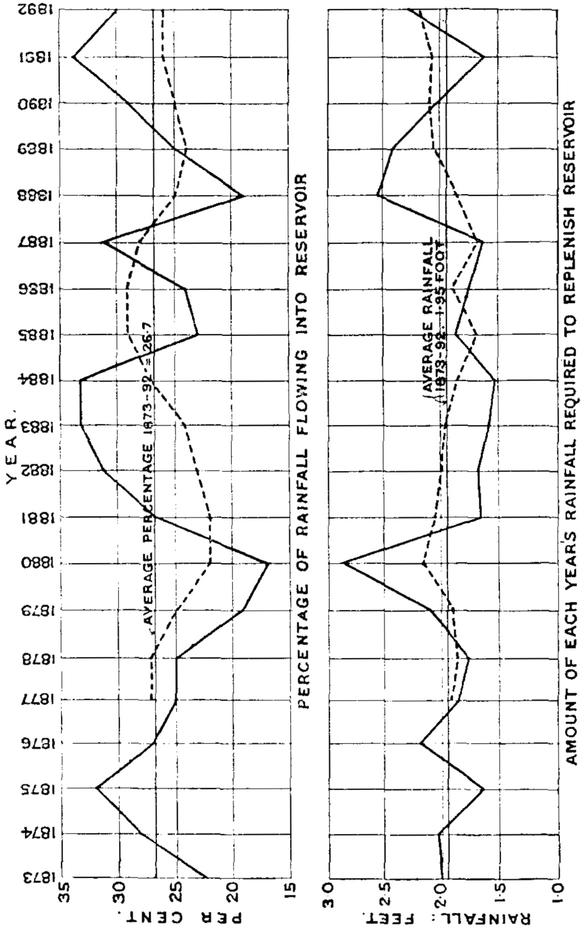


COMPARATIVE RAINFALL OF TWO PLACES IN INDIA SERVED BY THE SOUTH-WEST MONSOON CURRENTS.

Figs. 42 and *43*, worked out several years ago, when considering additions to the Nagpur waterworks. The diagrams, being self-explanatory, were submitted without further comment than that if some margin of safety was allowed, such data, if available at one place, should enable a successful forecast to be made in connection

The Author. with proposed works similarly situated and served. He had not previously considered the possibility of extensive percolation of flowing water through surface soils to underground fissures in granite, as referred to by Mr. Phipps Williams, nor did he think it at all probable that by this means there could be drained away such

Figs. 43.



Yearly quantities shown thus ——— *5-Year averages shown thus* - - - - -

NAGPUR WATERWORKS. WORKING OF RESERVOIR.

immense quantities of flowing water in all the five basins for which figures had been given in the Paper, while in exactly the same class of country the reservoir itself should prove fully stanch with a head of 100 feet of water over its bed. When questioning the utility of catch-water drains, Mr. Strange had evidently considered only the extreme feature of streams draining precipitous country, and liable, therefore, to the heaviest floods. Even in such country a large proportion of the off-flow could be diverted; while the Author's experience in half-a-dozen cases in less precipitous country had amply demonstrated that these drains could be most successfully used in connection with the Coolgardie water-supply reservoir. Moreover it had to be remembered that their aid was required in periods of small yield, and not in bumper years. The maximum flood-discharge anticipated at the Helena weir had been checked by deduction from known floods in adjoining streams, and as there had been settlers in the country for three-quarters of a century, this was a more accurate method than that referred to by Mr. Smith, of forecasting from flows at low states of the river and checking by means of formulas with arbitrary coefficients. The Author's experience was that probable maximum flood-discharges were easier of computation than low, or average, or total discharges. As regarded the minor points raised in connection with the weir, it was stated on p. 63 that the usual temperature and other cracks had duly appeared in the weir. These remarks Mr. Fuertes had evidently overlooked. With reference to the position of spill-water discharge arrangements, Mr. Strange had raised points similar to those of Mr. Deacon in the Discussion, to which the Author had already replied. Undersluices, unless very large, would not have much effect, and large sluices could not be recommended for a position such as that at Mundaring; they would have entailed far more expenditure than they were worth. The position of the lowest draw-off outlet referred to by Mr. Strange was not at the lowest point, which was R.L. 320, but 20 feet above, at R.L. 340, which was a very good allowance for silting in a reservoir 7 miles long, as of course the heaviest silting would naturally occur where the inflowing waters first met those of the lake. The scour was at R.L. 330, or 10 feet below lowest draw-off level. It was of course an exploded idea that a scour-pipe, however large, would denude the bed of a reservoir of silt, or even remove a large portion of it; but on the other hand the scour-pipe should certainly keep the approaches to the draw-off outlets clear, especially if placed so much below the lowest as in the present case. The question of filtration of the water, raised by Mr. Moncrieff, was very important. At present the catchment was rigidly guarded against pollution; but this might not be possible always, and then filtering would be necessary. He had so warned the The Author.

The Author. Government regarding the similarly-situated supply of the Western Australian metropolitan area. It would be easy to introduce filtering-arrangements when necessary in connection with the Coolgardie water-supply, if not immediately below the weir, then along the pipe-line, for instance at Baker's Hill, where there would always be ample head to spare. It might be of interest to note that the water as delivered on the goldfields was 70 per cent. purer bacterially than that at the source. So long, however, as precautions were taken to keep the catchment free from pollution there were no especial dangers; for although, as pointed out by Mr. Hazen, long storage in a warm climate aided in promoting the inferior growths, both animal and vegetable, evidently there were also counteracting influences where catchments were protected, as was shown in the following comparative Table of the average number of bacteria per cubic

Month.	Melbourne Tap-Water. Average Number of Bacteria per Cubic Centimetre.	Perth Tap-Water. Average Number of Bacteria per Cubic Centimetre.
April	?	780
May	?	147
June	?	298
July	132	132
August	99	370
September	76	90
October	89	30
November	60	1,572
December	71	3,470
January	54	?

centimetre. The storage-reservoirs of both towns were comparatively shallow; the Melbourne catchment was rigorously protected, and the water was unfiltered, but that at Perth was subjected to what was designated a filtering-process. The correspondence elicited by the Paper was especially wealthy in information regarding pipes and pipe-lines. In reply to the queries and inferences contained in the remarks of Messrs. Fuertes, Goument, John W. Hill, List and Strange, the Author would first state generally that the desirability of using wood-stave, ferro-concrete, or cast-iron pipe in lieu of steel was duly investigated before preference was given to the last mentioned. As regarded the wood-stave pipes, Mr. Fuertes had himself pointed out a very serious objection, namely, that they should always be kept full, otherwise rapid decay must result: a glance at the longitudinal section of the Coolgardie main showed the impossibility

The Author.

of keeping it fully charged without continuous pumping at the maximum rate. Having recently returned from the International Railway Congress held at Washington, and subsequent inspection of works under the auspices of the American Railway Association, the Author had nothing but admiration for the directness of American engineering aims; but, on the other hand, it was his opinion that American estimates of cost, especially comparative figures, were not of overmuch value for comparison in other countries, unless full allowance was made for the fact that, so far as he could judge, American home works paid more for materials, etc., than was charged for goods intended for export. Thus Mr. Fuertes stated that the tender for $\frac{1}{4}$-inch steel pipe, 30 inches in diameter, for work at Lynchburg, Virginia, amounted to 16*s.* 5*d.* per lineal foot in position. It was not quite apparent to the Author whether the cost of valves and specials, culverts and aqueducts, meters and road-crossings, covering-pipes and contingencies, were or were not included in this figure. Trench-excavation was not included, and if, for the purposes of comparison, the cost of this item and also carriage by rail from the Western Australian sea-coast inland were deducted from the Coolgardie water-supply figures for identical pipes, the net cost arrived at was 16*s.* 6*d.* per foot, inclusive of all the incidental works above mentioned. He considered that this told strongly against the American figures, having in view the fact that the whole of the plates, bars and lead was imported, a very large portion from America itself, and the further fact that wages in that country were 25 to 30 per cent. below those of Western Australia. On the question of the comparative cost of cast iron, raised by Mr. Goument, the Author would direct attention to Mr. John W. Hill's experience at Philadelphia. In the neighbourhood of cheap iron, Mr. Hill found that 60-inch cast-iron pipes, to withstand the low head of 160 feet, cost £4,650 per mile more than steel pipes would have been obtained for. Apart from this, neither cast-iron nor ferro-concrete was in the Author's opinion suitable for the high pressures of the Coolgardie water-supply. These could no doubt have been reduced by introducing more pumping-stations, but unprofitably, owing to the enhanced cost of pumping. The actual life of steel pipes, raised by Messrs. Goument and List, was another question, but it did not enter too largely into the pipes dealt with in the Paper, for, as therein stated, a sinking-fund of 3 per cent. per annum on the whole cost of the works has been provided by Act of Parliament to discharge the loan at a comparatively early date. The Author was glad to note

The Author. Mr. Herbert's good opinion of the locking-bar pipe, but considered that he had rated its virtues somewhat too highly. For the Coolgardie water-supply, at any rate, the price asked, length for length and thickness for thickness, was higher for locking-bar than for riveted pipe. He did not propose to follow Mr. Herbert in his calculations as to the comparative cost and economy of pipes of various diameters, as they appeared to be based on the fallacy that a 31-inch diameter pipe laid complete cost $\frac{31}{32}$ of the cost of a main 32 inches in diameter. This, at any rate, was not the Author's experience. Professor Campbell Brown's statement that asphalt was not suited for use in a composition employed for pipe-coating was too sweeping. The lime contained in the asphalt would no doubt be injuriously affected by some waters, but in numerous cases this material had served excellently, instances being forthcoming in the works mentioned by Messrs. Fuertes and Moncrieff. The coating referred to by Mr. Smith must have been wrongly made or applied, or used in some unsuitable position, to have perished, while exactly similar coating had served well in South Australia, as mentioned by Mr. Moncrieff. The precautions against corrosion suggested by Mr. List would surely prove altogether too expensive for general use, even if they were successful, which was not certain. With reference to the jointing of the pipes, no yarn had been used, the lead being kept from running into the pipe by temporary expansive rings; as the sleeve-rings varied slightly in diameter the caulking-tools used also varied in size; the caulking-machine was in the Author's opinion eminently suited for use on ordinary socketed pipes; and there was no doubt that a machine suitable for caulking at the locking-bar could be devised and would save money. The cost of caulking in South Australia mentioned by Mr. Moncrieff did not appear to have been lower than on the Coolgardie water-supply when due allowance was made for the larger pipe and for higher wages. On the latter work the cost of hand-caulking was 4*s.* 3*d.* on a 30-inch pipe, equivalent to 3*s.* 8*d.* on the 26-inch pipe used in South Australia. The Coolgardie wages were, per day of 8 hours:—Foreman, 16*s.* 6*d.*; hand-caulkers, blacksmith, lead-melter, etc., 12*s.* 6*d.*; labourers, 10*s.*; and the corresponding rates in South Australia were, according to the Author's notes, 10*s.* 6*d.*, 7*s.* 6*d.*, and 6*s.* 6*d.*—less than two-thirds on the average. Reducing the Coolgardie figures for 26-inch pipe even to two-thirds, the cost per joint was less than 2*s.* 6*d.*, the lower of the South Australian figures; and as mentioned in the Paper, machine-caulking cost 1*s.* less per joint, besides saving in depth of manhole and ensuring uniform

The Author. work. In reply to the concluding portion of Mr. John W. Hill's remarks, the Author had obtained in India all but absolute water-tightness in a new cast-iron main 12 inches in diameter and about 2 miles long. It was more than questionable, however, whether it was worth striving after such a condition of things. But even in the rapid work of the Coolgardie water-supply there were lengths from which the leakage was less than the figures given on p. 74. Thus the $36\frac{1}{2}$ miles of main west of station No. 4, gauged statically for 2 days, after being filled for the first time, showed a loss of 119 gallons per mile per diem, and after attention to visible leakage it showed a month later 31 gallons per mile per diem, and another month later 35 gallons per mile per diem, or, say, $1\frac{1}{2}$ pint per joint. The Author concurred with Messrs. Bruce and Hill as to the value of asphalt lining for reservoirs, and he had used it on the Western Australian goldfields for this purpose; but he employed 1 inch of asphaltic mortar without concrete or puddle, and found it sufficient for a head of 15 to 20 feet of water, while the cost of construction was one-fourth to one-third of that of a sufficient thickness of concrete lining. It was not possible to concur in Mr. Marsh's remarks as to the inutility of the wires buried in the concrete lining of the Bulla-Bulling reservoir. This work had been carried out in the height of a hot summer and the lining must have been subjected to great tensile stress when cooled by the admission of water. One after another of several concrete reservoirs on the goldfields had suffered from cracks all over, and consequent leakage, whereas the Bulla-Bulling lining had opened only at the expansion-joints, as already stated. There was no doubt, as Mr. Fairley said, that the capital cost of the pumping-machinery would have been reduced by employing three sets at each of the last four stations, but this would have meant two sets working at one time instead of one as in the adopted scheme; and at West Australian rates of wages this would have meant an appreciable increase in the cost of maintenance. Other advantages would also have been lost. Mr. Phipps Williams's calculations on the possible saving in pumping-machinery, if intended to be taken literally, were, the Author regretted to say, beyond him. Was it intended to imply that small pumps cost the same amount per horse-power to instal as large ones, and that working small pumps three shifts cost no more than doing the same amount of work in one shift with large pumps? How also would the increasing demand (increasing until the ultimate estimated amount should be reached) have been provided for with such small pumps as Mr. Williams considered enough? The small pumps, unless of varying

The Author. power, could not have been placed at the stations decided on as best for the ultimate supply, and nothing but peripatetic pumping-stations would have permitted of uniformity and economy in size of pumps without excessive pressure on the main as the horse-power rose in keeping with the demand. The details of cost of sundry works referred to by Mr. Moncrieff had not been given by the Author, as he was in hopes that they would be supplied to the Institution later on by his principal assistant, Mr. Reynoldson, to whom the Author, being now engaged in London and unable therefore to give effect to Mr. Crowell's suggestion of further tests of friction, etc., from time to time, looked for compliance in this direction also. In conclusion he desired to express his obligations to those who had commented so kindly on the work carried out. Very few indeed, besides engineers, credited the intense anxiety entailed during construction of a work of this kind; and none but engineers appreciated the difference between the reasons for success and failure in connection with Colonial works.