Prof. Kreuter.

that there should be no tension anywhere at the faces of a dam. They had only required that the intensity of pressure at the faces should at no point exceed certain limits. That condition has led to a theoretic type in which both lines of pressure fell within the middle third, in the case of a maximum working pressure of 6 kilog. per sq. cm., in the Furens dam. In the theoretic type for the dam of the Ban, however, which had been calculated by Mr. Montgolfier, and where 14 kilograms per square centimetre had been assumed, the consequence of the above condition was a very considerable deviation of the line of pressure, reservoir full, from the middle third. That uncertainty was the reason why Rankine found it advisable to premise the necessity that the lines of pressure should not leave the middle third of the profile.

As to Mr. A. W. Brightmore's objection, that, by making a part of the inner face overhang near the top, the condition of no tension was violated, he would reply, that that tension was practically nil, the corresponding deviation of the line of pressure. It was therefore not to be compared with the deviation of the line of resistance, reservoir empty, in the Tansa dam. He was aware of the circumstances which still ought to have been noticed in order to arrive at a complete solution of the problem, but was sure that it would be of no further practical advantage to enter into too minute details; and, on the other hand, he thought that any expansion of the theory might be arrived at without difficulty.

Correspondence.

Mr. Marius Bouvier.

Mr. MARIUS BOUVIER considered that some interest might attach to French practice in regard to such structures as those alluded to in the Papers. This was exemplified by the reservoirs in the south of France, an account of which had been recently given by him to the *V^ème Congrès Internationale de Navigation Intérieure*,[1] held in Paris in 1892.

Dr. A. von Braunmühl.

Dr. A. VON BRAUNMÜHL, of Munich, observed that, hitherto, in selecting the most suitable form of masonry dam, either an empirical method had been adopted or a more or less arbitrary section had been accepted; but Professor Kreuter had presented a complete, consistent, mathematical treatment of the subject. An

[1] "Reservoirs in the South of France." By Marius Bouvier. Fifth International Congress on Inland Navigation, Paris, 1892.

Dr. A. von Braunmühl.

analysis of the problem had convinced Delocre that, in consequence of the varying relation of the pressures, continuous profiles, such, for instance, as logarithmic curves, were not possible; he therefore divided the cross-section into three portions. Professor Kreuter remarked truly that, in order for the line of pressure to change from its fixed position in portion No. 1 to its position in portion No. 2 of Delocre's section, an intermediate part should be introduced. This part was of trapezoidal form. The equation for the breadth in the third portion was given as an algebraic expression, that in the fourth (base) portion being determined by a logarithmic curve. The principal difficulty consisted, after the breadth in the entire cross-section of the dam were found, in so joining together the four portions as to ensure the fulfilment of the necessary conditions of equilibrium. This was an easy matter in the first and second portions, but in the last two the equation contained an integral that could only be evaluated step by step. The effect of the vertical component of the water-pressure on the profiles, hitherto neglected, was next considered by a combined graphical and analytical method.

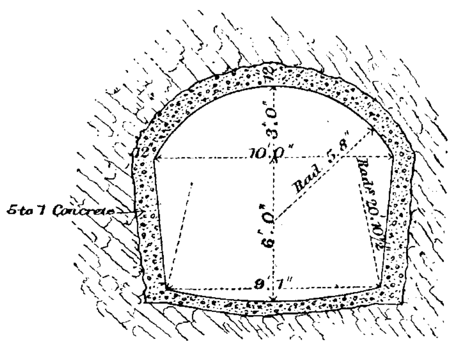
Engineers might, perhaps, consider this theoretical treatment of the question too cumbrous for ordinary use. Two replies might be given to this: first, the work involved in the computation was less laborious than it might at first sight appear to be; and, secondly, the time spent in arriving at a solution of the question was of little importance in relation to the certainty introduced into the design of dams by rigorous mathematical treatment.

Mr. A. Fairlie Bruce.

Mr. A. Fairlie Bruce observed that the strength of the Tansa dam might have been increased had it been constructed with an inner skin of 5 to 1 Portland cement concrete, say 5 feet thick at the bottom, diminishing to 1 foot at the top. That would not have added much to the cost, and would have prevented water from finding its way into the heart of the structure, which, while it might cause little or no leakage, must, by its pressure, impair the stability of the dam. It would have been interesting to know why rubble masonry was preferred to rubble concrete for the work. The crushing strength of the mortar compared unfavourably with that of Portland cement mortar. With cement concrete two weeks old, in which the mortar was composed of 1 part of cement to 3 parts of sand, he had, in arched ribs, obtained nearly as good results as those quoted in the Paper at six to eleven months; and at twelve weeks he had obtained 2,447 lbs. per square inch, which exceeded the maximum Tansa test. Judging from the quantity of water delivered, and the small loss by leakage, the workmanship of the aqueduct must have been of high class; but the section adopted seemed to leave something to be desired. He preferred that adopted on the new aqueduct from Loch Katrine, *Fig. 16*, in which the side walls had a batter of 1 in 12, and the invert had a versed sine of 6 inches. The flat invert of the Tansa aqueduct would be found difficult to cleanse, and silt would accumulate in the unlined portions. In the Loch Katrine aqueduct, a concrete invert was provided throughout the entire length, whether lined or not. As regarded the discharge of the siphon pipes, Mr. Bruce had found by experience that D'Arcy's formula gave a correct result, if a deduction of 4 per cent. were made for the resistance of curves. Applied to the Tansa case, it gave a delivery of 22,488,000 gallons, which corresponded closely with the result obtained by the Author. The subsequent diminution of delivery due to corrosion, unless the water was very soft, amounted to about 1 per cent. per annum. Considerable future trouble and expense might have been saved had a few lengths of the second line of pipes been laid at each of the valve-chambers, and closed with a blind flange, the sluices being afterwards fixed when required. The faucet adopted might have been made lighter had a steel hoop been shrunk on it. The thickness of lead ($\frac{1}{2}$ inch) was greater than that generally used. The usual practice now was to dispense with the bead on the spigot end, a steel or wrought-iron hoop being shrunk on instead, giving greater strength for transit, and capable of being afterwards struck off and sold for about its original cost. Had this been done, a shallower faucet might have been adopted. On the Glasgow Waterworks cant-collars were employed in place of bends on large pipes, and the convex sides of all curves were buttressed with masonry or concrete. For foreign work, where, as in the case of the Tansa works, transport presented a serious difficulty, steel offered many advantages over cast-iron for pipes. Referring to the Baroda Waterworks, the Author had made a great mistake in dispensing with a puddle-wall in the embank-

Mr. A. Fairlie Bruce.

Fig. 16.



LOCH KATRINE AQUEDUCT.

Mr. A. Fairlie Bruce. ment, however good the material might have been. It would have been far better to have dispensed with the second puddle trench. The puddle in the trenches did not appear to have been worked with sufficient care. With layers of 6 inches to 12 inches thick, tramping alone was scarcely enough without cutting and working with proper tools, to amalgamate each layer with that below it. Under the circumstances, it was not surprising that considerable leakage had resulted, and it might tend to increase, till it eventually destroyed the embankment. *Fig. 6*, page 47, did not indicate whether the outlet-culvert was constructed in a trench or merely on the surface of the ground; nor how it was carried across the puddle-trench. If that was imperfectly done, it would introduce an additional source of danger. It was not evident why scrap-iron filters were considered necessary. Surely sand-filters alone would have been sufficient. If, in these latter, 12 inches of gravel, graduated from the size of shot downwards to that of marbles, had been substituted for 2-inch metal, it would have been better. As it was, the sand would be found to wash down through the stones, and they would become choked, and do harm rather than good. It was not made clear how the rate of filtration at the Jeypore Waterworks was regulated; nor was the capacity of the clear-water tank given. The latter should be such that, when the filter became fouled, there should always be sufficient storage for the filtered water when the engines were not running; and it should not be possible, by the pumps supplying more water than the filters could pass, to cause an excessive head on the latter, otherwise the sand would certainly be liable to break up. Mr. Bruce thought, for waterworks purposes, Worthington engines produced less shock in the pipes, and were more economical in working, than beam-engines.

Mr. Collignon. Mr. E. COLLIGNON, of Paris, in reference to Prof. Kreuter's Paper, observed that much importance attached to the condition that the line of resultant pressures should always lie in the middle third of the section. Although it was desirable that the then resultant pressure, reservoir full, should not fall too far from the inner face, it was less important to prevent opening of the joints at the outer face, because the water could not take advantage of it there. On that account no inconvenience arose from causing the curve of pressure to approach the inner face slightly beyond the position assigned to it by the Author. The Author had confined himself to the consideration of horizontal joints, and had not dealt with imaginary joints situated obliquely in the masonry. In that respect, the necessity for considering oblique joints was admitted by the majority of French engineers. Occasionally the limit of resistance, whilst low enough on a horizontal joint, attained a much higher value for an inclined joint. It would be well to introduce this correction for the special section proposed by the Author. Mr. Collignon.

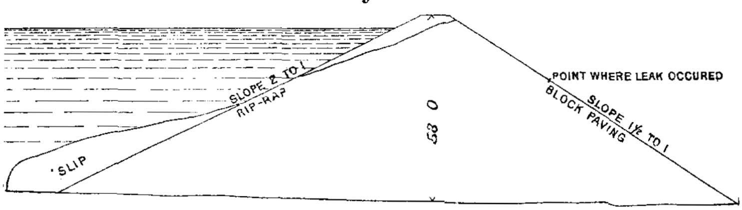
Mr. J. J. R. CROES considered that the methods employed in the construction of the Indian dams were interesting, but were certainly not applicable to any part of Europe or America; and the results in securing water-tight structures did not appear to have been such as would be satisfactory in American practice. Sufficient time had not elapsed to enable a judgment to be formed as to the stability of the Tansa dam, or as to its resistance to the percolation of water. The Author's contention that it was not essential that the mortar used in the interior of a large masonry-dam should be of a strength and hydraulic character equal to the best Portland cement, appeared to be reasonable. The crushing-resistance of mortar used in a dam could not be as great as that of the stone used in the masonry, nor could its specific gravity be as high. In the heart of a large mass of masonry, weight and homogeneity of the mass were the principal objects to be secured, and those could be best obtained by the use of as little mortar as was required to fill all the void space between the stones by a material which would resist solution by water. The practical limit to the use of large stones was governed by the possibility of roughly filling all the void spaces between the stones, and the best masonry-wall was one in which the stones were of large mass, imbedded in good hydraulic mortar, and laid sufficiently far apart to admit of the joints being filled with concrete composed of broken stone and good hydraulic cement, such as the Indian kunkur and the natural hydraulic cements of America. With rock weighing 172 lbs. per cubic foot, 32 per cent. of the mass might be of large stones and the specific gravity of the masonry would be 2·33. The earthen-dams at Baroda and Jeypore did not appear to have been constructed with a view to impermeability, which was essential in reservoirs in which it was important to retain all the water impounded. The cross-section of both of the embankments showed, moreover, that economy of space was not an object in their construction. The slope on the water-face, of 3 to 1 in one case and 4 to 1 on the other, was greater than that ordinarily permissible in reservoir embankments. Mr. Croes.

Such flat slopes possessed advantages in the case of sudden emptying of a reservoir, when a tendency occurred for the inner face of the embankment to slide, owing to the water retained within it. Two instances of such sliding of the inner bank of reservoirs

Mr. Croes. had come under his notice during the past year. At Pottsville, Pennsylvania, a storage-reservoir was constructed in 1886. The embankment was formed of earth, 500 feet long on the top and 60 feet above the natural surface of the ground at the middle. It was 10 feet wide; the outer slope was $1\frac{1}{2}$ to 1, paved with stone 8 inches to 12 inches thick; and the inner slope was 2 to 1, covered with rip-rap.

The reservoir was filled but not drawn down for six years. In August, 1892, a leak appeared at the middle of the embankment on the outer face, about 20 feet below the top. The water was turbid, carrying with it fine sand. The leakage increased rapidly until it was said to have flowed down the slope in a stream 3 inches deep and 20 to 30 feet wide. The waste-valves were opened and the water was drawn off the reservoir as rapidly as possible. Two days were occupied in emptying the reservoir. When it was nearly empty, the inner slope settled and slid down

Fig. 17.



into the basin, causing a depression in the dam about 5 feet deep in the middle and 200 feet long. At the foot of the bank the material slid out about 40 feet, assuming the form shown in *Fig. 17*. This embankment was stated to have been constructed of selected material excavated from the hill-sides in the vicinity, consisting of clay, sand, and a small quantity of gravel—the clay predominating. The material had been hauled on to the embankment and spread in thin layers and consolidated by the passing of the teams and wagons, but was not rammed or rolled. Through the middle of the embankment specially selected material was placed which was wet and cut with spades. Examination of this bank a year after the break occurred showed it to be charged with water. It had taken six years, after the reservoir was filled, for the entire dam to become saturated with water below a plane sloping from the water-surface inside to a point about 20 feet below the top on the outside of the bank. The water then began to ooze out, carrying with it some fine material, and gradually opening channels for the free passage of water. Then, when the reservoir was suddenly emptied, the saturated bank on the inside was so tenacious that the water in the bank could not escape rapidly enough and the whole mass slid down. The other case was that of a service-reservoir at Portland, Maine. The reservoir-bank, 40 feet high, had been constructed with the greatest care under the direction of an experienced engineer. The inner slopes of $1\frac{1}{2}$ to 1 were covered with clay puddle, on which was laid a bed of broken stone faced with a pavement of granite blocks. On the 5th August, 1893, a section of the embankment at one corner of the reservoir gave way in consequence of the water following a line of pipe laid through the bank, making a gap 60 feet wide at the top and 20 feet wide at the bottom, emptying the reservoir in less than an hour. Shortly after the reservoir was thus emptied, the paving on two sides slid down into the reservoir. In this case also the lining was insufficient to resist the thrust of the saturated clay puddle behind the wall. The immediate cause of the sliding of the bank in both instances seems to have been the presence of a mass of retentive material on the inner slope, which had been intended to prevent the penetration of water, but which had become saturated and caused the mass to slide, instead of draining out gradually as would have been the case with sand or ballast. It seemed likely that such accidents as these might be prevented by adopting the course recommended by Col. Jacob for the toe of the outer slope of the Jeypore dam, and making the exterior of the embankment on both sides of sand for 5 feet overlaid by broken stone carrying a pavement of large rubble. To prevent the percolation of water through an embankment, a thin core wall of concrete or rubble masonry was preferable to a wall of puddle. Mr. Croes.

Mr. T. P. S. Crosthwait.

Mr. T. P. S. CROSTHWAIT congratulated the Gaikwar of Baroda on having given a constant supply of water to his capital, and Mr. Jaganath Sadasewjee on the execution of the work. A gravitation scheme had been adopted by the Author, and in that he was, no doubt, much influenced by the caste prejudice against pumping. That prejudice had been overcome in Calcutta and elsewhere, but it proved to be very strong in Baroda, and probably it was not as easy to combat a prejudice of that kind in a native State as in one under British rule. Mr. Crosthwait, having been engaged, in 1878, in the preparation of a scheme for the water-supply of Baroda, had not seen his way to recommend gravitation, owing to the country around the city consisting of a flat plain intersected by water-courses, of which the beds were 30 to 90 feet below the surface of the plain, the banks being nearly vertical and composed

Mr. T. P. S. Crosthwait. of dry sandy clay, with black cotton soil near the surface. Borings showed this to be the general condition, and he had found that any reservoir would have to be very shallow and of large area, from which there would be great loss by evaporation, considerable loss by absorption, and also leakage. The reservoir which has been formed appears to have a general depth of only about 8 feet when full. The leakage under the embankment was first noticed in 1890, and some precautional works were carried out, the leakage increasing to 49,000 cubic feet daily in 1891 and nearly double that quantity in 1892. These quantities had been observed in October, at the end of the Monsoon, when it might be presumed that the reservoir was full or nearly so. It would be interesting to know if the leakage had further increased in 1893, and whether the water in the wells of the neighbouring villages had been raised since the construction of the reservoir. The loss by percolation was not surprising, as the puddle-trench did not appear to have been carried deep enough—not as deep as the beds of the two rivers crossed by the embankment. The average of five years, observations showed the discharge of the Surya river to have been only 18·4 per cent. of the rainfall, which alone showed how porous the subsoil must be.

It had been assumed by the Author that the evaporation from the reservoir would be 72 inches annually. As the general depth of the reservoir was only 96 inches, there would be a large deduction from the storage on that account. The shallow nature of the reservoir would be favourable to the growth of vegetable matter, and as the water would be tepid, the Author had wisely introduced a system of purification and aëration, as well as works of filtration.

Mr. J T. Fanning. Mr. J. T. Fanning, referring to the behaviour of the siphon pipes of the Tansa works, as regards expansion and contraction, had observed that large cylindrical plate-iron conduits, resting horizontally on trestles, when used to convey water from a canal on one side to the canal on the other side of a valley, expanded and contracted daily with the variations of temperature, so far as to require expansion joints at each end of the tube. In the construction of some American water-power works, he had used wrought-iron tubes to convey water under pressure. At Spokane Falls, Washington, two wrought-iron penstocks of 7 feet diameter and 500 feet in length were used to convey a portion of the water-supply to the turbine wheels. These tubes had an inclination of 1 in 12, and conveyed water to turbine-wheels working under 70 feet head. There was daily expansion and contraction of the pipes during construction. At Great Falls, on the Missouri river, Montana, there was first laid a plate-iron tube of 9 feet diameter and, approximately, 350 feet in length. This conveyed water to turbines working under 40 feet head. During its construction daily expansion and contraction was observed.

Mr. J. T. Fanning.

At Austin in Texas, on the Colorado river, eight pipes of 9 feet diameter and 290 feet in length, with branches of 7 feet diameter to the turbines, were laid to convey water to wheels in the power-house to work under 60 feet head, and similar daily expansion and contraction was observed.

Recently, in the development of the water-power at Great Falls in Montana, a wrought-iron tube of 10·5 feet diameter and 420 feet in length had been constructed for conveying water to two turbines of 900 HP. each, working under 40 feet head. In this case the building of the tube in place was commenced at the masonry of the inlet valve-house, and was carried forward from this point a distance of 350 feet, with an inclination of 1 in 100, toward the power-house. This tube was exposed during construction to all changes of temperature. The movement at the free end of the 350-foot section, before it was connected to the lower section, was on some days found to be fully 2 inches, or 0·000476 of its length, longitudinally, between midnight and midday. The differences in temperature of the upper portion of this tube, exposed to the sun, was probably not less than 70° Fahr. between midnight and midday. The weight of this piece was about 70 tons, and its cross-section was about 100 square inches. It was built first upon blocks of timber without bearings upon the earth, until it was riveted and tested under pressure. If the coefficient of friction of the plate-iron upon the bearings of wood was as much as 0·60, then the stresses of tension and compression in the tube near its fixed end may have been approximately 42 tons, or 0·42 ton per square inch of section, in addition to the stress required to draw the 70 tons weight 2 inches up the incline in, say, eight hours. These tensile and compressive stresses were acting on the masonry of the valve-house in which the higher end of the metal cylinder was anchored fast. These considerations led to the bedding and covering of the pipe at the earliest convenient time. Anticipating some expansion and contraction in the power-house, where the branches from the main pipe led to the turbines, he had deemed it prudent to use an expansion joint near the turbine branches. Since this tube had been completed, bedded, and filled with water, no movement due to expansion or contraction had been observed. Experience with the other tubes at Great Falls and Spokane also

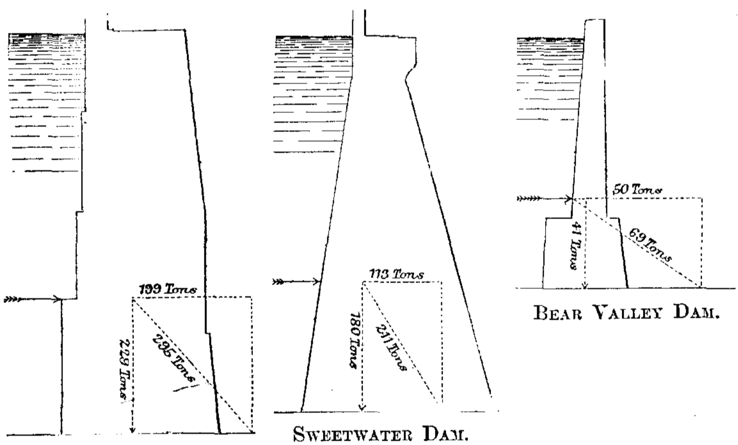
Mr. J. T. Fanning. indicated that there was no apparent daily movement due to expansion or contraction after the tubes had been bedded, covered, and filled with water. The friction between the iron and the earth, when the tube contained its load of water, resisted the longitudinal movement and forced the fibres of the iron to yield all along the pipe to the stresses induced by changes of temperature. If, in the exposed Tansa cast-iron pipes, the expansion in each 12-foot length was approximately 0·0000062 of the length for every degree Fahrenheit of change of temperature, as suggested by experiments, and the daily range of temperature at any time reached 70°, as seemed probable, then there was a tendency to extension equal to approximately $\frac{1}{16}$ inch in each lineal foot, with probably a compressive stress of $2\frac{1}{3}$ tons per square inch of section of the pipe-wall in those pipes which lay horizontally, increased in the case of pipes on the inclines. The stresses of complex expansion and contraction had to be resisted by the pipe-joints as well as by the metal of the castings, and the effects upon the joints must be very severe, and perhaps occasionally cumulative by the combined movement of several 12-foot lengths of pipe. He took additional interest in the behaviour of these exposed pipes since observing that the drawing for the pipe-joints was in form and dimensions identical with a design published by him about fifteen years ago for pipes of that diameter.

Mr. George Farren. Mr. GEORGE FARREN remarked that, in dealing with the subject of masonry dams, the principles involved in the construction of dams arched in plan ought to occupy an important place. In certain circumstances and under peculiar conditions, the arched form might be introduced as a valuable adjunct to dam building. There were only three dams of considerable magnitude which were incompetent to resist by their gravitational stability the thrust of the waters impounded by them, and which therefore depended for stability upon their arched form. There were the Zola dam, *Fig. 18* (constructed by the father of the well-known novelist), in Provence, and the Sweetwater and Bear Valley dams in California, *Figs. 19* and *20*. The resultants of the stresses, reservoir full, in the Sweetwater dam fell well in the outer third of the section, showing that tensile stress must occur at the inner toe; while in the Zola and Bear Valley dams the secants exceeded their bases altogether, indicating that the arched structure was operating as an arch. Some years ago he had visited the Furens dam, passing through the Gouffre d'Enfer, and had also inspected the Chartrain and Roanne dams. The Furens, and, indeed, most of the French

Mr. George Farren.

dams, were arched in plan, though the curvature added nothing to their stability, as the direction of the lines of the secants would show. The completed dams visited by him did not appear to be quite water-tight, but in the new Roanne dam, which was probably now about finished, the inner face was being rendered with a mixture of cement and sand in equal parts, laid on about $1\frac{1}{4}$ inch in thickness. As soon as it had set, which occupied about five hours, the water was allowed to rise and cover it, and this treatment appeared to be satisfactory. It would be observed that the work was on the inner face, where it was important to ensure water-tightness in such structures. He had been informed by

Fig. 18. *Fig. 19.* *Fig. 20.*



ZOLA DAM. SWEETWATER DAM. BEAR VALLEY DAM.

Scale, 1 Inch = 50 Feet.

French engineers that the Zola dam had done its work well up to the present time, and that they were perfectly satisfied with it. The Bear Valley dam was startling in its boldness, and the stresses occurring in it were very high. He would not care to reside in any town situated immediately below it. It had been reported that it was proposed to erect a stronger and higher dam somewhat below it, and so to relieve the strain in the existing wall, and that seemed a very prudent step to take. In cases where there were no houses below it, such a light structure might be allowed, and the complete and sudden ruin which would attend its failure might be contemplated without alarm; for

Mr. George Farren. example, in a mining district, where the dam was carefully watched by those interested in it. But he thought it would be imprudent to build such a wall above a town or populous district, whose inhabitants could not hurriedly take measures to secure their safety in the event of an accident. He would here compare the Furens, the Zola, the Sweetwater and the Bear Valley dams, in respect of their relative cost, with similar sections below the water-level, cut off from the Furens dam. This comparison would be made in tons weight per middle section for a length of one foot, and the cost might average about £1 per ton.

	Feet deep.	Tons.		Tons.
Actual weight of the Zola dam, say	120	229	Furens	370
Actual weight of the Sweetwater dam . . . . . . ,,	98	180	,,	264
Actual weight of the Bear Valley dam . . . . . ,,	62	41	,,	126

showing the relative savings in tons weight which would be gained by the arched form of construction over the gravity type. The savings in money would, however, not be anything like so much, on account of the necessary dressing of the masonry in the arched dams.

A totally different method of comparison might have been adopted by reducing these extraordinary dams to the linear dimensions of the Furens dam, increasing their heights and thickness in the same relation. The objection to that course was that it was uncertain how their respective designers would have modified the sections for such increased height. With a water-pressure of 330 tons, maintaining in each dam the existing condition as to the incidence of the resultant stress, reservoir full, the weights of the several sections per lineal foot of dam were, for the Furens, 740 tons; for the Zola, 513 tons; for the Sweetwater, 500 tons; and for the Bear Valley, 270 tons. He would point out that by reducing all four dams to the same dimension as regarded height, whilst preserving their relative form, the lines showing the direction of the stresses and their resultants would not be disturbed, though of course the stresses would be very different from those in the original forms, as the stress due to the water-pressure was the same in all four. The resultants of the stresses would be, for the Furens dam, 830 tons; for the Zola dam, 552 tons; for the Sweetwater dam, 626 tons; and for the Bear Valley dam, 463 tons.

It was worthy of notice that two of the engineers of the proposed Quaker bridge dam were both opposed to arched dams, and had recommended a straight gravity dam; one of them had even worked out the problem of the Quaker bridge dam—providing that the stress should not exceed 16 tons per square foot—and had shown that the sectional area of an arched dam about 277 feet high would be more than double that of a gravity dam. On the other hand, other American engineers had strongly objected to a straight dam at Quaker bridge, and recommended a dam curved to a 1,200 feet radius, which, however, was practically a gravity dam, their principal reason being that in case of the yielding of the masonry in a straight gravity dam, "the curved form given to it might prove of advantage," and that "the curved form better accommodates itself to changes of volume due to changes of temperature." Mr. George Farren.

Mr. C. F. FINDLAY mentioned that, on the question of the elasticity of masonry and concrete in relation to the theory of dams, it might not be superfluous to state that even if it were possible to use absolutely homogeneous material, and the modulus of elasticity of that material were known with the utmost precision, engineers would still be at an immeasurable distance from any rigorous determination of the distribution of stress in a body of irregular figure, such as a masonry dam. Mr. C. F Findlay.

There were two assumptions universally made in the theory of the subject, and almost universally passed over in silence (as by Professor Kreuter) so that they were liable to be mistaken for physical truths. These assumptions were: (1) That all the internal stresses acted in planes normal to the face of the dam; and (2) That in any such plane, if a horizontal section of the dam were taken, the resultant pressure was distributed across the section as a uniformly varying pressure. The sanction for these assumptions was to be found in comparing the deductions made from them with the results of study of actual structures, and not in any scientific theory of elastic solids. Professor Kreuter was in no sense responsible for the hypotheses with which his Paper commenced. He had simply taken them as he had found them, representing the accepted basis of current practice, and had limited his Paper to showing how the form of dam to which they logically give rise could best be arrived at.

The Paper introduced a new, more complete and more direct method, but a few points might be noticed on which it appeared to him that the Author had not done justice to the value and completeness of his own proposals. On p. 66 it was said: "the simplest among appropriate cross-sections . . . is a right-angled triangle with vertical inner face." Now a vertical face was no simpler than a battered face, but, fortunately, there might be found a better

Mr. C. F. Findlay. reason for the verticality of the inner face, viz., that the vertical inner face gave the smallest triangular cross-section consistent with the conditions assumed, when the masonry (as in almost all practical cases) had at least double the specific weight of water. If masonry were used of a less specific weight, maximum economy would be realised in giving a batter to the wall of 1 in $\frac{2 - \rho}{\sqrt{4 + \rho^2}}$, where ρ was the ratio of the specific weights of masonry and water. The only reason why it was not permissible to give to the inner face an equal batter inwards (or overhang) with masonry of ordinary weight, was that the line of pressures, reservoir empty, would then fall outside the middle third. In this connection, it might be suggested, whether, where there was no provision for emptying the reservoir, the case of the reservoir empty need be considered? In the Tansa dam, no doubt this had been fully weighed, but, where a reservoir could only be emptied by the destruction of the dam, it would seem advisable to obtain any economy to be realised by neglecting such a condition.

Next, as to the superstructure, it was not essential to the Author's theory to assume a rectangular block of masonry "equivalent" to the actual superstructure; and in practice a block "equivalent" in his sense would not coincide in height and width at once with the dimensions already fixed by the actual superstructure. It would have been better, therefore, to assume merely a certain weight of superstructure with a line of action not necessarily coinciding with the middle of the width. In the same connection, it seemed a superfluous assumption to take the top-water level as the level of the top of the dam. It might be appreciably higher or lower, and the theory in no way required such an assumption. Next, coming to portion No. (2), an assumption was made "convenient . . . and having the further advantage of simplifying the calculations," that the height of the trapezoidal portion should be equal to that of the superstructure. This resulted in disfiguring the inner face of the cross-section with a re-entering angle. No engineer would accept such a form, and it would have been better to have assumed a vertical inner face for this portion, and to have let the height of it follow as a consequence, instead of being arbitrarily assumed. The resulting height would have been greater than that in the figure, but quite unobjectionable.

Next, attention might be called to a remark, p. 71, that "if the slope of the outer face is to be allowed for in the limiting pressure (as is done by Rankine)," etc. This point seemed to him much too important to be dismissed as a matter of taste. Judging by the fact that the batter of the face was ignored in the succeeding section of the investigation, it seemed that Professor Kreuter did not consider Rankine's argument conclusive, in which opinion many others would probably join; but the fact of a strong opinion existing on the other side on so important a question demanded that reasons should be assigned for whatever course was adopted. Lastly, as to the portion No. (4) which entailed the most serious part of the investigation and related to the heaviest part of the dam, the Author said: "an investigation which it is needless to reproduce here shows that," etc. Now, as the course which the Author had adopted was only one of several which were open, and as other writers had adopted a different course, he could not concur in the opinion that it was needless to reproduce the investigation in question. Its general nature should, at any rate, have been sketched so that others might be able to find for themselves the same justification which the Author had found for following that course. What the Author had done was to make the pressure at the inner face, reservoir empty, constantly equal to the maximum limit, and that at the outer face zero. Then (vertical pressures only being considered) the pressure at the outer face with the reservoir full would be always less than the maximum limit, and that at the inner face would always have a positive value. He believed that the conditions laid down would be more economically complied with by making the pressure at the inner face, reservoir empty, and that at the outer face, reservoir full, both equal to the maximum, and keeping the two lines of pressure always equidistant from the centre of the dam. The equations of condition for this arrangement were not simple but were soluble. For the purpose of design, however, he would prefer to use a graphic method throughout. While the analytical method was invaluable in deciding upon the process and in giving a general view of its results, it was inadequate to the complete determination of the cross-section, as might be seen in the treatment of portions (3) and (4); and the whole of the analytical expressions could be translated into graphic methods, which, at any rate, as a check on the calculations, should in practice be applied. Mr. C. F. Findlay.

Professor PH. FORCHHEIMER, of Aachen, remarked that whilst in designing dams the weight of the masonry and water were considered, the uplift and the influence of varying temperature did not appear as elements of the calculation. When water penetrated through masonry it naturally exercised a tendency to lift the superincumbent mass, which was especially manifested at the Prof. Ph. Forchheimer.

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mortar joints. It might be assumed that water flowed in approximately horizontal lines through the masonry; and, as its velocity was constant, it suffered a uniform loss of pressure. This pressure, multiplied by the volume of the interstices per unit volume of the mortar, which might amount to one-fourth, would give the uplift. The safest plan was to make the inner face of the dam as water-tight as possible. In a dam lately built at Remscheid, Westphalia, by Professor Intze of Aachen, the inner face was plastered, and then rendered over with two coats of asphalt. Greater security might be obtained by draining a wall so constructed close to the inner face. Of even more importance than uplift in dams was the movement due to variation of temperature, especially in countries subject to climatic extremes. It had been observed by Professor Intze that, at the middle of the crest of the dam alluded to, which was 82 feet in height, a backwards and forwards movement amounting to $1\frac{1}{16}$ inch occurred during the filling and emptying of the reservoir, and that the movement due to temperature was almost as great as this. The latter was due, less to the temperature of the air, than to direct solar radiation. The crest of this dam was 460 feet long, and was arched with a radius of 420 feet. One side of it was exposed to the sun longer than the other; and the more exposed part moved to and fro $\frac{7}{8}$ inch, in the course of the year, whilst the other part moved only $\frac{1}{8}$ inch the crest expanding $\frac{1}{9,000}$ of its length, or $\frac{5}{8}$ inch. In arched dams such movements did no harm, but in straight dams these phenomena were objectionable. As dams were usually built during the warmer seasons of the year, the masonry had a tendency to contract in the colder weather. In a curved dam this can take place by movement of the structure without cracking, but not in a straight dam. Adopting Adie's coefficients of linear expansion, a reduction of 10° Centigrade in temperature would produce contraction to the extent of $\frac{1}{10,000}$ of the length of the wall. The data concerning the modulus of elasticity of stone was scanty and unreliable; but it appeared generally to be between 1,400,000 and 2,800,000 lbs. per square inch. That of mortar, according to Hartig,[1] was higher. If the temperature of masonry was lowered 10° Centigrade, and it was not free to contract, tension amounting to between 140 lbs. and 280 lbs. per square inch was set up, which was greater than the mortar would stand. When a wall became warmer than it was at the time of its construction it formed a compressed beam; but, owing to the considerable

[1] Civilingenieur, 1893, p. 472.

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length of dams in relation to their thickness, and the high compressive strength of building materials, there was seldom danger of fracture from expansion. That a straight, or almost straight wall, incurred considerable danger of fracture was shown by practical experience. The dams of Habra, Grands-Cheurfas, and Sig in Algiers had broken, and in that of Hamiz a tear had occurred during the first filling. The Habra dam broke in December, and the Grands-Cheurfas and Sig dams gave way in the month of February. The Beetaloo[1] dam in Australia had also developed a crack $\frac{1}{8}$ inch wide in the middle of the winter, without any apparent cause. The Mouche dam, Haute Marne, a structure 1,346 feet long and about 100 feet high, exhibited clearly the dangers attending straight dams. In the winter of 1890–91, when the temperature varied between $-10°$ Centigrade and $-20°$ Centigrade, and the water-surface was 10 feet 8 inches below the normal level, seven vertical cracks appeared in the dam, situated at uniform distances of about 160 feet apart. They were widest at the top, and died out about 37 feet below the normal water-level. Their aggregate breadth was $2\frac{7}{8}$ inch. The cracks gradually closed as the temperature rose, and by the end of February, 1891, four of them had completely vanished, whilst the others had perceptibly contracted.[2] In other buildings, also, contraction during cold weather caused cracks. A long quay-wall[3] at Bremen, for instance, developed cracks due to low temperature, which opened between $\frac{1}{6}$ inch and $\frac{1}{3}$ inch in winter and closed to fine hair cracks during summer.

Prof. Frühling.

Prof. FRÜHLING, of Dresden, noticed that the Author of the Paper on the Tansa Waterworks attributed the bursting of several 48-inch pipes mainly to an insufficiency of air-valves; but many hydraulic engineers would look for the cause of failure in the bedding of the pipes upon the rock.

In the Baroda Waterworks there was no provision of a third high-level inlet to take the upper portion of the water from the collecting channel. Such an arrangement was desirable, especially as, during the rainy months, this upper supply had held a minimum quantity of solid matter in suspension. The circumstance of the omission of a puddle-wall in the embankment appeared, in spite of the good quality of the materials employed, to have been the primary cause of the percolation of water to the toe of the outer slope of the embankment.

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[1] Minutes of Proceedings Inst. C.E., vol. cxiii. p. 158.

[2] "Reservoirs in the South of France." By Marius Bouvier. Fifth International Congress on Inland Navigation, Paris, 1892.

[3] Zeitschrift des Architekten-und Ingenieur-Vereinszu Hannover, 1889, p. 444.

Prof. Frühling. In the case of the Jeypore Waterworks, percolation was to be expected from the nature of the material of which the dam was constructed, and the impracticability of introducing a puddle-wall. It might, however, be questioned whether, failing the insertion of a puddle-wall, it would not be better to construct the flatter slope on the outer side. In this case illustrated in *Fig. 21* the greater portion of the dam was dry, in contrast to the case of *Fig. 22*, whilst the leakage was less owing to the smaller inclination. In regard to the plan of the service-reservoir, it might be remarked that, as a consequence of the inlet and outlet being placed opposite one another, the water would not, generally speaking, circulate sufficiently freely through the reservoir, whereby the freshness of the water was lost the development of bacteria was favoured.

Fig. 21.

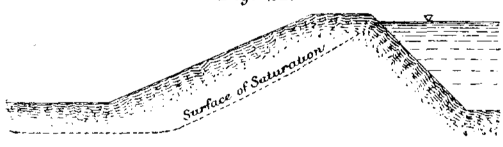
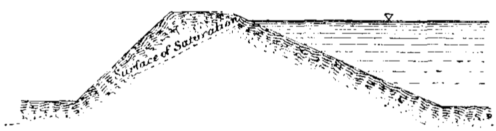


Fig. 22.



Mr. J. H. E. Hart. Mr. J. H. E. HART presumed that the constructors of the Baroda and Jeypore reservoirs had, to a certain extent, followed the Bombay irrigation standards for earthen dams, introduced by General Fife, R.E. The peculiar feature of his practice was the rejection of the puddle-wall, which was typical of the English system. By depositing the material in thin layers, saturated with water, and carefully consolidated, he, in fact, made the entire dam a mass of puddled earth. A serious mistake had been made in not reaching a clay bottom with the first puddle-trench of the Baroda embankment for the 200 feet between the rivers. It was stated that it had been reached with the second trench all through. But why not with the first? for the difficulties must have been similar in both cases. The second trench was, in his opinion, quite useless; for it was quite unconnected with either the mass of the dam or the other puddled trench. A somewhat indiscriminate use of the terms "sand" and "earth" had been adopted by the Author of

Mr. J. H. E. Hart.

the third Paper, but the dam described ought to have had something in the shape of a puddle-wall up the middle, for it appeared that it was little more than a filter. The way in which the water falling on the surface of the embankment was disposed of would not answer with good stiff material. The guttering of the slopes was prevented in the irrigation dams by a thick outside casing of mixed black soil, and decomposed trap and gravel. The questions of the discharge from catchment-areas, and the evaporation from reservoir-surfaces, had engaged the attention of Indian engineers, as well as those of other countries. In the nature of things no definite answer could be given to these questions, because each catchment must necessarily have its own coefficients. Surprise might well have been expressed at the large estimated discharge of the Tansa waste-weir—25,000 cubic feet per second—but the maximum discharge provided for over the waste-weirs of certain reservoirs in the Deccan amounted to 200,000 cubic feet per second in two instances. At Lake Fife, with a catchment area of 196 square miles and an estimated discharge of $\frac{1}{2}$ inch per hour, the waste-weir was calculated to discharge 62,000 cubic feet per second; and at Bhatghur, with $\frac{5}{8}$ inch discharge per hour from 128 square miles, and the discharge provided for was 57,000 cubic feet per second. He might here direct attention to the large masonry and concrete dam at Bhatghur, which formed the impounding reservoir of the Nira Canal, designed by Mr. J. E. Whiting, M. Inst. C.E., and now complete, or nearly so. Its height was 127 feet above the lowest foundations, and 103 feet above the river-bed. It was 3,020 feet long, with 810 additional feet of waste-weirs. The upper part was built of concrete faced with masonry, the lower part being solid rubble; and it carried a roadway 11 feet wide along the top. The section of the dam was designed by Mr. Whiting, and all the calculations were worked out by Mr. A. Hill, Assoc. M. Inst. C.E., who was afterwards Mr. Clerke's assistant on the Tansa works. The curve adopted for the face was a catenary; but the wall was actually built in a series of batters. The three primary conditions of the design were, 1st, the intensity of the vertical pressure was nowhere to exceed 120 lbs. per square inch; 2ndly, the resultant pressures were to fall within the middle third of the section; and, 3rdly, the average weight of the material was to be assumed at 160 lbs. per cubic foot. The use of concrete in the section was only permitted where the pressure was calculated not to exceed 60 lbs. on a square inch, which gave a factor of safety of between 6 and 7. With reference to the movement of dams under the pressure of water, Mr. J. H. E. Hart. he had found, after prolonged and careful observations with theodolites, the supposed deflection of the Kurruckwasla dam—Lake Fife—was a myth. Indeed, at times, it seemed to move in a direction contrary to the pressure. The observed anomalies were considered to have depended on the temperature and on refraction. Finding that instrumental observations were unreliable, he had arranged with Mr. Whiting, when building the dam at Bhatghur, to leave a vertical pipe in the masonry through which a plumb-line could be dropped, so that the movements, if any, could be directly measured.

Col. J. O. Hasted. Col. J. O. Hasted, late R.E., supposed that, as the three reservoirs which had been described are accomplished facts, and were fulfilling the duty for which they were intended, they must be considered successful examples of their class, but it was rather startling to find that in two of them the embankments had been formed simply of the adjacent soil thrown in a heap on the ground.

In the case of the Jeypore reservoir the embankment was of sand, without any core-wall, resting on sand and mud, and, when full, the water would stand 40 feet deep against the embankment. It seemed to be a bold undertaking, and he noticed that it had been considered desirable to take precautions in case of leakage appearing at the toe of the outer slope of the embankment.

He had, during a long service in the Irrigation Department in Madras, been obliged to make embankments of sand on a smaller scale, and well remembered one instance, after the cyclone of 1864 at Masulipatam, when it was his duty to excavate a channel from a navigable canal some 3 miles from the town to bring in fresh water, all the wells having been ruined. That channel had to cross hollows where the water was held up by embankments, and the only material available was sand. It was almost a matter of life and death, so, working night and day, the water was brought in; but the sand embankments caused endless trouble. They were continually giving way, and for some time had to be watched day and night, work-people being kept ready to stop breaches in them until the river-water, which was highly charged with silt, had in effect puddled the inner slopes.

That sand might be compressed by weight until it became capable of withstanding considerable lateral pressure, he was well aware. The great weir across the Kistna river at Bezwadah was a masonry wall about $\frac{3}{4}$ mile long and 14 feet high, resting on wells 6 feet deep sunk in the sandy bed of the river. On the down-stream side of the wall there was an enormous talus or apron, over 200 feet in width, formed of blocks of rough stone, its surface being sloped at about 1 in 20. At one time, owing to some particular set of the river, the sandy bed was eroded above the weir, until there was a hole 30 to 40 feet deep, and below the apron there was another hole 90 feet deep; and as the flood subsided there was a difference of level between the water above and below the weir of 10 to 20 feet, but the weir never moved, the weight of the stone having compressed the sand so as to render it quite firm. Col. J. O. Hasted.

In the case of the Baroda reservoir the embankment had no puddle-wall, as the material employed in its construction was of good quality, which, he supposed, meant soil with some admixture of clay, but there was a puddle-wall in the ground below.

No doubt the old reservoirs constructed by the natives of India, called in the Madras Presidency, where they numbered some 39,000, tanks, were made by simply throwing up the earth to form a bank; but those banks frequently gave way, and in some of the larger ones, that had stood for many years, he had known great trouble arise from leakage. The toe of the outer slope was apt to become saturated, and the slope, and possibly more of the embankment, sat down. The only remedy was to find where the water came from, and to lead it away in rough stone drains to a distance before filling in again with earth.

In reservoirs with earthen embankments there was always risk of leakage, and therefore a good puddle-wall, carried well down through the subsoil, was desirable. In India it had sometimes been the practice to form the puddle-wall along the face of the front slope under the revetment. This, however, was not a good arrangement in a tropical climate, as when the water fell in the reservoir, the puddle was liable to dry and crack. In some cases, when the puddle-wall rested on soil of such a nature that it was possible water might creep through, it was desirable to lay rough stone drains to convey the water to a distance in rear of the embankment.

He would also, with reference to the earthen embankment reservoirs, remark on the long front slopes employed—in one case 3 to 1, and in the other case 4 to 1. He preferred a steeper slope in front with a long slope in rear for this reason, if the water-spread was considerable the waves formed by wind so easily ran up a flat slope. A case in point was that of the Red Hills reservoir, which supplied Madras with water, and was breached in a cyclone in 1884. The front slope there was 1 to 1, and the revetment was formed of fairly smooth blocks of stone. When the

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gale was blowing at an angle of about 25° to the embankment, the water ran along the face, rising more than 9 feet, and overtopped the parapet-wall. When the reservoir was reconstructed he had provided for the lower layer of stone in the vertical parapet-wall being projected as a string-course purposely to break waves running up the front slope.

With regard to the disposal of the rainfall which fell on the Jeypore embankment, the water which fell on the top was allowed to soak into the bank, which thus became a huge sponge. In a dry climate very likely this was all right, but he could not recommend such an arrangement in any locality near the coast in India, where very heavy rainfall sometimes occurred; when the sponge became surcharged the result would be disastrous. When the rear slope of the embankment was well grassed over, the rain-water might safely run off the surface, if it was conducted away from the toe of the slope, but until it became so grassed over it would be advisable to form channels to carry off the rainfall.

The material of which the Tansa dam had been constructed was uncoursed rubble masonry set in kunkur limestone mortar. He would say the smaller the stone the better. With the very considerable pressures in the dam, anything which tended to uneven settlement was to be avoided, and the mass should be as homogeneous as possible. With this view he had advocated the use of concrete in the construction of the dam, 155 feet high, at the Periyar Reservoir, in South India, which was now in progress.

Mr. Clemens Herschel.

Mr. CLEMENS HERSCHEL desired to comment on two points suggested by the interesting group of Papers under discussion. One was the substitution of riveted, or lap or butt-welded, or seamless drawn, or rolled steel pipes for cast-iron pipes; the other was on the method of keeping the records of the flow off a catchment area. As regards the former, it had been demonstrated by practice in the States west of the Rocky Mountains, in the United States, that cast-iron could not compete with wrought-iron or with steel pipes. This had been the fact for the past thirty years, unless in exceptional cases. Scarcely any main water-pipes were now laid, or had been laid for thirty years past, on the Pacific Slope, other than some form of wrought-iron or steel pipes. This had been due to lack of coal and of iron ore in those States, rendering necessary the importation of iron and steel; and the weight of cast-iron pipe compared with that of steel pipe of equal capacity and strength, had put the former out of the competition. It might be thought that in India, similarly situated as regards importation of cast-iron pipe, which was worth £5 15*s.* 8*d.* to £5 19*s.* 11*d.* per ton, delivered on the wharf, that material would

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be equally out of the race. With another such case of a main to be laid on a stretch of country reaching 50 miles into the interior, no small economy could be effected by the employment of steel pipes. The cost of the $17\frac{1}{2}$ miles of siphons did not appear, but the pipe on the wharf was about $15·25 per lineal foot, assuming an average of the three prices stated; and experience with riveted pipe led him to say that such siphons could be laid and completed for less than the cast-iron pipe cost on the wharf. Clearly, India should offer a great field for riveted or other steel pipes. Steel pipe was a source of economy, not only in exceptional places like California, Nevada, and the East Indies, but also nearer home. The same was true in the eastern states of the United States, and in England. Only the diameter of the pipe appeared, in these places, to limit the field within which either kind of pipe might prevail over the other. Thus it has been established in the States last mentioned, at prices which have ruled for the past two or three years, that steel pipe was more economical than cast-iron pipe down to about 36 inches or 34 inches in diameter. The engineers of Rochester, N.Y., who had laid riveted 24-inch and 36-inch iron pipes in 1874, were now laying a 36-inch riveted steel conduit, some 20 miles long. The supply of Newark, N.J. (and at the same time that of the towns of Montclair and West Orange), was conveyed by a 48-inch and 36-inch riveted steel conduit, 26 miles long, laid in 1889–92.[1] In the manufacture of lap-welded steel and iron pipes, and of butt-welded (electrically (?)) seamless drawn or seamless rolled steel pipe, improvements in manufacture, or reduction in the price of the product, were constantly taking place. Thus lap-welded pipes in 20-foot lengths, and 24 inches in diameter, were already in the market. Tubes of 4 feet diameter, butt-welded, were in use for high-pressure steam-boilers, though still too expensive for water-pipe. And it would, no doubt, only require an order for 10,000 tons to start the manufacture of some of the other forms of pipe that had been named. There was a fair prospect that, before long, steel tubes of any desired strength and diameter, 25 to 30 feet long, might be purchased in any desired quantity, at a price that would make them the most desirable class of pipe for conveying water long distances. It was not unlikely that seamless rolled steel tubes might be furnished at little, if any, over the price of steel plates.

As regarded the second point—a uniform method of keeping the records of the discharge from a catchment-area—English engineers

[1] Minutes of Proceedings Inst. C.E., vol. cxiv. p. 418.

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had great opportunities in the course of their practice, extending into all corners of the earth. Those opportunities appeared however, to have been in this respect neglected. The necessities of construction and lack of control of works after their completion, to some extent explained the paucity of records of stream-discharge. It seemed, however, that more might be done and recorded, if a uniform system of keeping such records were adopted. No gaugings of the discharge of the Tansa river accompanied the first Paper, the yearly discharge for five years was given in the second Paper, and the third Paper contained the mere statement that only one-sixtieth part of the rainfall on the catchment-area had been shown to reach the reservoir. The weight usually given to irrelevant records of rainfall in engineering Papers of this sort, afforded perhaps a clue to the reason why the important subject of stream-flow had been so neglected, and a consideration of the greater ease with which rainfall records might be arrived at, compared with the measurement of daily stream-flow, would probably solve the question. Civil engineers were under a moral obligation to present in engineering Papers, useful facts to their professional brethren, and if either stream-flow or rainfall was to be omitted, they should surely omit the rainfall, which might be safely relegated to the weather reports, or might be studied in connection with other classes of engineering Papers, such as, for example, treat of sewerage-works. What engineers wanted for water-supply purposes was the daily stream-flow, measured over a long series of years; and no harm would accrue if the matter of rainfall were ignored in such works entirely. It had, indeed, but a mildly scientific interest, and its ratio to the discharge of a stream, possessed only the interest that appertained to matters of scientific curiosity. It was of no service in the design of works for water-supply, or of irrigation works, which a knowledge of the stream-flow alone would not better render. As for the relation of the rainfall during a particular month to the stream-flow during that month, it presented an effect coupled with only one of the causes that produced it, such as necessarily gave results palpably absurd. Work of this kind brought the art of the civil engineer into disrepute with other practical men, and it should be abandoned for that reason, if for no other. To demonstrate the unscientific, untruthful and utterly useless character of such relations between rainfall in any one month, and stream-flow in the same month, it was only necessary to look at, say, the Minutes of Proceedings Inst. C.E., vol. cxiii, p. 321, where it was stated that a certain river in India discharged during the rainless season about ten times as much rain as fell on its drainage-area, due to the river being fed

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by rain that had fallen months previously. Discharges of 150 to 250 per cent. above the rainfall were of annual recurrence in the body of such tables during the winter months in cold climates (due to the streams discharging melted snow and ice which had fallen months previously). Again, such relations had a great range in the same place from month to month, and during the same month in different years. Years having the same rainfall yielded different portions of it, according to the sequence and the distribution of the rainfall throughout the year. Knowledge of the rainfall during any one year, and of the portion of it yielded by the river, was therefore no valuable guide as to the stream-flow during any other year, or even during the same calendar month in any other year. That was true on one river, and even on the same portion of the same river. It was also true for different rivers in different countries, and even for different portions of the same river in any one country. Again, to arrive at these relative yields, it was first necessary to know the discharge of a river. But inasmuch as the discharge of the river was the thing sought, why not rest content when it was found? An analysis of the flow of any stream, and of its proportional yield of rainfall, might be useful some day when studied in conjunction with such data for other rivers, and as teaching the effects of steepness of slope, and of quality of the soil, on the yield of streams; but such refinements applied to the remote future, while to-day, the actual yield and discharge of rivers in general formed information much needed by engineers, whenever an impounding reservoir or other water-supply work had to be constructed. With these ideas in mind, the writer desired to suggest for general adoption among civil engineers, a plan of reporting the discharge from catchment-areas, somewhat as follows:

For areas of catchment-surface and of reservoir surface: square miles.
For discharges of rivers or streams during one month or shorter period: cubic feet per second.
For specific discharges of drainage areas: cubic feet per second per square mile.
For volumes contained in the reservoir: cubic feet.
For rainfall: inches in height.
For yield of drainage-area, when it was to be represented in bulk, as for a year or month: inches in height (considered as covering the whole area on which the rain fell); or else, in cubic feet per second, on the average.

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Experience had taught him the propriety and convenience of the suggested units for use by civil engineers. They were drawn up for the use of English-speaking engineers only. It must commend itself to them that some uniformity of action should be adopted, so as to make convenient, and ensure the use, by one engineer, of the work of another. So long as such arithmetical vagaries as reporting the discharge of rivers in "tons per 24 hours," annual rainfall on a catchment-area in gallons, discharge in cubic feet per second per 1,000 acres, and so forth, were indulged in, the information afforded was more apt to be passed by than to be treasured up and used. As to the data to be reported, he would suggest:

(*a*) The cubic feet per second flowing in the river, or which would flow in it were none withheld in the reservoir, daily or as often as could be readily arrived at. This might be called the "natural discharge" of the river under consideration.

(*b*) The drainage-area in square miles above the point of gauging the river; the character of the drainage-area as regards slopes, and the quality of the soil.

(*c*) The available volume of the storage-reservoirs or reservoirs, *i.e.*, between top water and extreme low water, in cubic feet.

(*d*) The area of the water-surface of the reservoir or reservoirs, in square miles.

(*e*) The rainfall in inches, monthly, or daily.

(*f*) Natural discharge of the river, monthly or annually, to be expressed in inches of rainfall on the catchment-area; or to be given as an average in cubic feet per second per square mile of catchment-area.[1]

Mr. A. Hill.

Mr. A. Hill stated that in India catchment-areas varied much in regard to the amount and certainty of rainfall. In all cases it was desirable that the assured annual supply should not be largely in excess of the quantity required, as, otherwise, the reservoir was liable to silt up. The Tansa reservoir was most favourably situated in this respect. At the Bhatgarh reservoir for the Nira Canal, where there was a masonry dam more than 4,000 feet long and impounding a depth of 100 feet of water, the total volume of which exceeded 4,600,000,000 cubic feet, the ordinary annual supply was some six times the volume of the reservoir. To minimize

[1] A set of Tables constructed on the lines indicated, exhibiting the characteristics of the discharge from the drainage-area of the Pequannock river in the State of New Jersey, U.S.A., was furnished by Mr. Herschel in illustration of his remarks, but space did not admit of its reproduction in the Minutes of Proceedings. They may be consulted in the Library.—Sec. Inst. C.E.

silting of the reservoir, a series of fifteen sluices, each 8 feet high and 4 feet wide was constructed near the bottom of the dam. The area of these sluices was sufficient to pass all ordinary floods under a head of less than 30 feet, and as the full depth of the reservoir was 82 feet, the greater portion of it, when these sluices were open, was empty and could not be silted up. The flood-water also remained but a short time in the reservoir, and did not deposit so much silt as they would during a slow passage through the reservoir when full. The sluices were closed in ample time to fill the reservoir, and the river water was then comparatively clear and free from silt, grass and vegetation having grown on the catchment-area during the rains. Such a system of sluices could of course only be adopted where the supply was regular and assured, like that from the western ghauts of India during the monsoon. For all sluices, whatever gearing were adopted for raising the sluice-gate, it was important that it should show the position of the gate; in practice, a simple nut and collar with holding-down bolts at the top of the rod, the top length of rod being screwed to fit the nut, was the best arrangement. The gearing should be set so that the top of the rod was flush with the top of the nut when the shutter was closed, and the position of the shutter was then always known, for if 6 inches of the lifting-screw were visible the shutter was raised 6 inches. With this gearing also the screws could be oiled and cleaned. At the Bhatgarh dam the same uncertainty of rock foundations was experienced as at Tansa; at one place a sheet of sound rock suddenly stopped and a drop of 30 feet had to be excavated to obtain good foundations. This was unexpected, for the trial-pits here were not more than 150 feet apart and showed the same kind of rock at about the same level. The drop in the foundations necessitated a change in the position of the waste-weir. Again, in the breadth of the dam at the lower parts, in one place a fissure some 15 feet deep and 20 feet wide occurred; and what was most expansive of all it was found that apparently sound rock in the river-bed was a slab of rock not bearing evenly on the rock below. This slab varied between 8 feet and 30 feet in thickness, and it was deemed advisable to remove it all for about 250 feet in length and more than 70 feet in breadth. No accurate estimate of the cost of a dam could be framed until the foundations were laid. In constructing a masonry-dam it was very convenient to have a sluice at the bottom of the structure, through which water could be let out for washing sand, for bathing, and for the many other requirements of a large work. The sluice-way could be readily plugged if desired when the work

Mr. A. Hill.

Mr. A. Hill. was finished. At Bhatgarh, such a sluice was built at the level of the bed of the river; and at 16 feet from the inner face the sluice-way was reduced from 6 feet to 4 feet in breadth. It was afterwards built up under a head of 10 feet of water; the leakage through the planks which closed the sluice being collected into a 4-inch pipe fitted with a valve. This 16-foot plug of masonry was now subjected to a head of 100 feet of water and was quite watertight. The sand was washed in shallow masonry pans 20 feet by 10 feet by 1 foot deep, with a stream of water running through them; and sand with 15 per cent. of silt in it, could be perfectly cleaned by that means. The water-supply was practically unlimited, and from 12 to 15 cubic feet of water was used per cubic foot of sand washed. Sweating or percolation of moisture occurred in all masonry-dams in the Deccan. The trap stone was very dense, and that used at Bhatgarh and Vir weighed 183 lbs. per cubic foot. Nevertheless water could penetrate the stone, and when quarrying blocks in the lower parts of the quarry which had been under water during the rains, water was found in the small cavities common in trap rock when the stones were split under the hammer. In the hot weather the rock did not contain water if it had been exposed some time. In the Paper on the "Designs of Masonry-Dams," the Author had stated, "that the shearing-stresses acting parallel to the layers of the wall were not allowed for." These stresses had also been neglected in calculating the maximum intensity of pressure—in other words, the horizontal thrust of the water was neglected. It had frequently been shown that this course led to serious under-estimating of the pressures. It had been repeatedly pointed out that the whole resultant of the weight of masonry and water-pressure combined must be considered. Again, in calculating the intensities of pressure, horizontal planes or bases had been considered by Prof. Kreuter. He desired to suggest that such a course was not correct, and that a plane at right angles to the neutral axis of the cross-section of the dam should be considered, in other words, the profile of the dam must be considered; and, as Rankine had pointed out, the intensities of pressure were in some way affected by the shape of the cross-section or profile. In selecting the final profiles for a dam, facilities of setting-out and construction should be considered, and it should also be remembered that the upper parts of the dam were the longest, the lower parts being confined to the gorge in the river; a little excess of section in the lower portion was therefore less wasteful than equal excess in the upper portion of the dam. The easiest profile to construct was an arc of a circle, and the most difficult was the long straight batter. In a long straight batter, if the masons made a mistake, or a template were wrongly set, the error could not be rectified without causing a bulge in the face; on the other hand, if the profile were an arc of a circle, a mistake could be promptly remedied by introducing a chord or a tangent as required. Mr. A. Hill.

Mr. W. R. Hutton.

Mr. W. R. HUTTON, of New York, thought that the profile of the Tansa dam had been based upon the true principles which should govern such designs, with ample margin for safety. The construction also, in masonry of uniform character throughout, and the care taken that no voids should be left in the mass, left nothing to be desired, if the latter condition, most difficult of enforcement with ordinary workmen, had been successfully carried into effect. The expense of excavating the firm but fissured rock down to the solid trap, was justified by the experience at the Bouzey dam (Eastern Canal of France), a portion of which was displaced by sliding forwards on its base—a movement rendered possible, it had been stated, by the reduction of its effective weight by the up-lift of the water in the seamy rock on which the dam stood. It had been justly observed by the Author that the character of the mortar to be used in such works was of the first importance. The experiments of Mr. Bouvier, quoted in the Paper, upon the compression of cubes of mortar, confined on all sides in blocks of wood, did not seem to reproduce the conditions of actual practice. Those of Mr. Gillmore upon short prisms of different heights afforded a better guide in estimating the resistance of the mortar in a wall. In the latter experiments the shorter prisms showed a considerable excess of resistance to compression over cubes of the same material and composition. It should be observed that the greatest pressures were always at one or other face of the masonry, where the mortar was unconfined on one side. The percolations through the Tansa dam were so far of little importance. In this respect it was more fortunate or better built than many other high masonry dams. A calcareous efflorescence on the lower part of the outer face, due to percolation, was frequently found. The greatest care in construction, raking out the joints and filling them, and plastering the upper face of the dam with Portland cement, did not always prevent the occurrence of such leakage. Mr. Bouvier seemed to have feared a slow solution and washing away of the lime in the mortar, particularly in primitive formations, whose waters being very pure had great dissolving-power. A large amount of hydraulic lime in the mortar of impounding-works afforded a guarantee of their stability.

Mr. W. R. Hutton.

But it was none the less necessary to devise means to prevent further impoverishment by the slow action of percolation. The use of asphalt in this connection had received attention. Asphalt coatings on the slopes of the embankments of some of the French reservoirs had failed. Recent lining with similar material of reservoirs in Colorado and California, U.S., had so far been successful. In building the brick conduit in the High Bridge of the original Croton Waterworks, General George S. Greene had required one course of bricks to be heated and dipped half their width in hot coal-tar, and laid in that material. That conduit had never leaked. The head of water on it was, however, small. The observations made in the Paper upon the discharge of the aqueduct indicated the necessity of most careful description of the surfaces of channels. Bazin's coefficient for his third category was applied by Kutter to walls of dry rubble, and not to a conduit of which one-third the wetted perimeter was of plastered concrete and the remainder of well-pointed rubble.

Prof. Kovatsch.

Professor KOVATSCH, of Graz, pointed out that, thousands of years ago, Hindoos, Chinese, Egyptians, and other great peoples, had, instead of expending their powers in river improvements or canalisation, constructed immense impounding reservoirs for irrigation and other purposes. Many recent examples had afforded instances of the necessity for clearly determining the principles and details of construction of such works, and the disastrous results due to false economy. It was only by the progress of scientific investigation and study that accurate methods of construction appropriate to the requirements of the case, had superseded the tentative and haphazard works of earlier times. The Papers were replete with practical information and valuable data; while Professor Kreuter's exposition of the design of masonry dams completed the consideration of the subject alike from the analytical standpoint.

Mr. Graham R. Lynn.

Mr. GRAHAM R. LYNN remarked that the Author of the Paper on the Baroda Waterworks had noted that, as the general material used in the embankment was of good quality, no puddle-wall was constructed in it. Looking to the leakage that had taken place, it seemed, however, a pity that that precaution was not adopted; as the best of the material used had 37½ per cent. of sand in it, and some of the samples taken from the waste-water-course had proved on analysis to have contained as much as 66 per cent. of sand. Lately, analysis had shown the albumenoid ammonia in the lake to have increased much above that given by the Author, and had also proved that there was only a slight diminution of that impurity after the water had passed through the purifiers and filters.

Mr. Graham R. Lynn.

That had led to an investigation of the purification-works. The Anderson purifiers, which were each originally charged with $1\frac{1}{2}$ ton of cast-iron borings and had each been daily replenished with 20 lbs. per day, were found to be empty and entirely devoid of that material. They had no doubt been inoperative by reason of the light borings having been carried through the cylinders by the flow of the water. Steps had since been taken to replace the borings by punchings to avoid the recurrence of such a condition in future.

Mr. A. T. Mackenzie.

Mr. A. T. MACKENZIE said that the largest dam made in England of late years was that of Vyrnwy, which the Author of the first Paper had diplomatically described as "magnificent." If the Corporation of Liverpool was satisfied with the extravagant proportions of that dam, no one else need complain, so long as the design was not set up as a standard. The theory of dams was simple and advanced enough. What was wanted to reduce their proportions was faith—in the theory and in the material employed in constructing them. Referring to details, it was true that in the matter of foundations, trial-pits give no satisfactory indications, and, unless the bare rock lay naturally exposed to view, it was impossible to estimate accurately the character and cost of the foundations. The original rock exposed to atmospheric influences gradually disintegrated, and above the bed-rock there was sure to be a layer of boulders with seams that might descend 10 feet to 60 feet. That has been so in the case of the Periyar dam. Whether in the case of the Tansa dam it was essential, particularly on the higher levels, to entirely remove those boulders was another question. At any rate, if they were in the middle, or towards the rear of the dam, it would probably have been sufficient to rake out the joints as far as it was possible to reach, subsequently filling in the seam with mortar. At the Periyar dam, broken stone for concrete was being carried by coolies from a shoot to the work, a distance of over 100 yards, for less than 1*d.* per ton, and under the circumstances of the work no machinery could compete with that. The Tansa dam was said to be practically dry, but a dam of mortar only would probably be that. The Periyar dam was being built with a front rubble masonry wall of varying thickness, never less than 6 feet and sometimes as much as 20 feet thick and a rear wall of similar construction, the heart being of concrete. The concrete laid in 6-inch layers and tested in a block of 1 cubic yard unsupported at the sides, leaked between two layers first under a pressure of 75 lbs. per square inch. In 1-cubic-foot blocks, unsupported at the sides, it did not crush under a load of

Mr. A. T. Mackenzie. 12 tons. The rubble masonry was built of the largest manageable stones, averaging perhaps $1\frac{1}{4}$ cubic foot, and had stood in the course of the work such trials by nature that it had not been thought necessary to incur the expense of testing it.

The question of passing the flood-water was of the greatest interest. The floods at Periyar were occasionally enormous, the maximum known being 120,000 cubic feet per second, so that the matter was more serious even than at Tansa. There was no chance of building the foundations on dry ground, and a year-and-a-half was spent before a stone of the dam was permanently laid in the river-bed. A diversion had been made round the flank which was intended to discharge 2,000 cubic feet per second; but it was always breaking into the foundations, and floods of any magnitude had of course to pass over the entire work. On the whole, hardly any damage had been done to the dam, but enough had been learnt to show the enormous harm that could be done by eddies if they were once formed. The lake already formed comprised a large area, and small floods were absorbed before topping the work; though occasionally a weir left lower on one flank of the dam was submerged for a few hours. The ordinary discharge of the river had always been passed round the flank by vents; which of course entailed a protecting wall and a joint. There was also a guiding wall, a continuation of which formed a channel conveying water to a turbine to drive the stone-breaking and mortar-mixing machinery.

It would be useful to learn the experience of engineers in obtaining sand by mechanical means. The sand at Periyar was dredged from the river-bed, but as the water became deeper the process became more difficult and costly. The earth which covered the syenite of the hills was really disintegrated rock containing a large percentage of silica, which could easily be separated by washing. But there always occurred a considerable portion of alumina which did not dissolve in water for a long while, though it yielded fairly easily to pressure. To get sand from this earth, it had to be washed, then passed through stamps or grinding-mills, and finally washed again. The sand in the river-bed was often covered with a layer of fine silt, and contained vegetable matter and a good many pebbles, though when screened it was of fair quality. Tailings from the stone-breakers had been tried, but, besides being limited in quantity, they contained too much fine dust, too many small bits of stone, and a perceptible quantity of alumina.

Mr. R. Moore. Mr. ROBERT MOORE, of St. Louis, U.S., mentioned that the wisdom of the precaution taken by the Author in carrying the foundation of the Tansa dam below all veins of soft material in the rock, had been strongly emphasised by a recent event in Texas, when the failure of an important masonry dam, over 60 feet high, during the last summer, had been caused by neglect of that precaution. He had visited the work shortly after the fracture occurred. The water had found its way through soft seams in the foundation, and had exerted almost incredible destructive force. Heavy masses of masonry had been undermined and thick ledges of overlying rock had been completely broken up. In considering the schemes for the water-supply of Bombay and Baroda as set forth in the Papers, nothing struck an American engineer more forcibly than the small provision of water per head of population. For Baroda 25 gallons per head per day had been considered an ample supply, whilst it had been estimated that 50 gallons per head per day were more than would ever be required for Bombay. To Americans, accustomed to a consumption of 75 to 100 gallons per head per day, those figures indicated differences in the scale of living which it would be interesting to investigate in detail. But whatever the cause, the simplification of the problem of water-supply to Indian cities produced thereby were most important. Mr. R. Moore.

Lieut.-General JOHN MULLINS, late R.E., observed that, including additional water, received during the continuance of the rainy season, the quantity of water made available by the Tansa works was, in round numbers, 3,400,000,000 cubic feet, and on that basis the cost of providing storage amounted to about £70·59, or Rs.1,129 per million cubic feet—a rate much in excess of that financially admissible for storage for irrigation. The Tansa site presented favourable conditions as regards water-supply, reliable foundations for the dam, abundance of stone for its construction, and facilities for the disposal of surplus water. It necessitated, however, a great length of dam; and the alignment of its axis, though doubtless determined by the circumstances, was not of the most desirable form, being concave towards the water. He had had occasion to consider the subject of formulas for approximately determining the proportions of masonry dams for reservoirs, and had concluded that a modification of Sir Guilford Molesworth's formula for deducing the width of the base, exclusive of the part appertaining to the inner face batter would be desirable. The modified formula was— Lieut.-General John Mullins.

$$y = b + \sqrt{\frac{0 \cdot 05\,(x-b)^3}{\lambda + 0 \cdot 05\,(x-b)}},$$

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in which x = the depth in feet of any horizontal plane below top of dam;

y = offset in feet, from a vertical line through the inner top edge of the dam, to its outer face at any depth x;

b = breadth in feet of the top of dam;

λ = the limit of pressure in tons per square foot.

The minimum limit of b was 7·50 feet, and for a less top breadth than 12 feet a somewhat different form was given to the equation, viz.:—

$$y = b + (12 - b) + \sqrt{\frac{0\cdot 05\,(x - 12)^3}{\lambda + 0\cdot 05\,(x - 12)}}.$$

For the base of the inner face batter, the formula was

$$z = \left(\frac{0\cdot 09\,x}{\lambda}\right)^4$$

in cases in which the reservoir, as at the Tansa site, was not liable to be emptied, and became

$$z = \left(\frac{0\cdot 0925\,x}{\lambda}\right)^4$$

where that condition did not obtain. It might be interesting to compare the dimensions given by these formulas with those adopted in the design for the Tansa dam, at a few depths (x) below the top.

Values of x.	Tansa Dam.[1]			Mullins' Formulas.		
	y.	z.	$y + z$.	y.	z.	$y + z$.
Feet.	Feet.	Feet.	Feet.	Feet.	Feet.	Feet.
50	27·50	..	27·50	29·31	0·15	29·46
60	33·00	..	33·00	35·94	0·30	36·24
90	56·00	0·60	56·60	58·13	1·56	59·69
110	74·00	1·80	75·80	74·23	3·47	77·70
135	96·50	3·30	99·80	95·32	7·89	103·21

The limiting pressures on which the section of the Tansa dam was designed were not given in the Paper, but the maximum pressure

[1] *Vide* Fig. 4, Plate 1.

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shown in Fig. 4 was 125·22 lbs. on the square inch at the toe of the outer face, which was equivalent to 8·05 tons on the square foot. The formula was based on a limiting pressure of 8 tons per square foot, with masonry or concrete weighing 140 lbs. to the cubic foot. The Tansa design was based on the supposition that the masonry would weigh 150 lbs. per cubic foot, and its actual weight was about 155·8 lbs. The value of λ in the formula had therefore been taken at 7·25 tons. It would be seen on comparing the figures given in the Table with the profile (Fig. 4) that Gen. Mullins' formulas gave increased breadth just where the calculated lines of pressure approach most closely to, or fell without, the boundaries of the middle third of the section. The outer face of the dam below 85 feet from the top was apparently a straight line tangential to the curve of the upper part of the face at that point, which curve had been struck with a radius of 160 feet. It might perhaps be concluded, both from the figures given by the formula for y and from the calculated pressures at the face, that the profile of this face was not quite as conformable as possible to the conditions to be satisfied; but, on the other hand, for practical purposes it was no doubt sufficiently accurate.

As the water in the reservoir was not likely to fall at any time even to the level of the lowest sluice, which was only 45 feet below the top of the dam at the full height of 135 feet, Fig. 4, the exact position of the line of pressure, reservoir empty, with reference to the middle third of the section, was not of practical importance. That line of pressure, with the water at its lowest level, would certainly fall well within the middle third.

The catchment-area of the Tansa reservoir was of medium size. The maximum rainfall recorded (Appendix A) was 6·79 inches on the 19th July, 1889. It would be necessary to know the rainfall of each of several consecutive hours during exceptionally heavy storms to enable a conclusive judgment to be formed as to what maximum duty the waste-weir might be called upon to perform.

Taking into account the moderating effect of the reservoir, it might be considered that, starting with the water-level with the crest of the waste-weir, the latter would suffice to dispose of a discharge of 1·69 inch from the whole catchment-area in one (the first) hour, and 0·75 inch per hour afterwards. Assuming that experience at the Nagpur reservoir might be taken to indicate that the duration of discharge would be about 1·70 times that of the duration of rainfall, the 1·69 inch would represent a rainfall at the rate of 2·87 inches per hour. The coefficient 0·70 used for

Lieut.-General John Mullins. estimating the depth corresponding to a given discharge passing over the waste-weir, seemed somewhat high, taking into account the width of the crest, which was apparently about $15\frac{1}{2}$ feet. The coefficients of weir-discharges varied with reference to both the depth passing over them and the width of the crests. The information at present available was insufficient for the exact determination of the coefficient proper to any given conditions, but in this case it seemed probable that, for the discharge of 25,000 cubic feet per second, the depth on the crest would approximate to 3 feet instead of 2·5 feet as estimated by the Author. The adjusted coefficient for depth alone was 0·624, and for depth of water and width of crest 0·56. It was easy to ascertain coefficients correctly for very small depths, but the measurement of the large discharges due to considerable depths was difficult to deal with experimentally. Over-estimating the depth required in a case like that in question was an error on the safe side.

The arrangements made for the disposal of surplus water during the building of the dam—a matter of considerable anxiety to engineers in charge of work of this kind—had proved to be entirely satisfactory, and the Author was to be congratulated thereon. A depth of 6 feet of water passing over a dam upwards of 50 feet high was a severe trial, but the form of the outer face was one which greatly reduced shock, and gradually converted nearly vertical to nearly horizontal direction of current. The account of the works for conveying the stored water to Bombay was full of interest, and would be of high value to engineers who might have similar work to perform. The handling and laying of large pipes weighing 3 to 4 tons, and of an aggregate weight of 48,000 tons, required, in a country where suitable appliances were not obtainable, much forethought, and the fact that only one length of pipe failed in the $17\frac{1}{2}$ miles, above Ghat Kopar, showed that the manufacture and testing of the pipes, and the subsequent arrangements for handling and laying them had been efficiently carried out.

Referring to the Baroda Waterworks, he observed that the annual quantity of water to be delivered was $182\frac{1}{2}$ millions of cubic feet. To fill the reservoir, which had a storage capacity of 1,287 millions of cubic feet, a discharge of 15·3 inches or 0·39 of the average annual rainfall from the catchment area would be required, which was more than could be relied on. From the data given in the Paper it appeared that the discharge in the years 1885 to 1889 was in each case less than the capacity of the reservoir. From the diagram, Fig. 3, it would appear that in 1891 also the reservoir did not nearly fill, as the water-level in October, at the end of the rainy season, was about 4 feet below the weir-crest. In that year, however, the quantity of water stored was, after allowing for evaporation, sufficient for the supply of Baroda. It was to be hoped that the surplus remaining in the reservoir from good years would make up for the deficiency of other years; otherwise he feared that the reservoir would disappoint the expectations of the designer and of the Government of Baroda. The particulars given of the reservoir embankment showed that the site was unfavourable for the construction of a large impounding-reservoir. About 4,000 feet in length of the dam had to be based on black soil, which was not a satisfactory foundation, as it was liable to considerable changes of condition. It cracked to great depths from the surface in dry weather, and the cracks were sometimes of formidable dimensions. When dry it was very friable, and when moist it was a very soft material. There were, however, many considerable reservoirs which had existed for longer or shorter periods, the embankments of which were composed of and founded upon soil of this description. The manner in which the embankment was formed had shown successful results. At one part, where apparently clay was not found even at a depth of 40 feet, some troublesome and, it might be, dangerous leakage occurred, which had been treated by the precautionary measures adopted, with, it might be hoped, satisfactory results. The waste-weir, the length of which is 800 feet, was probably of ample discharging capacity. Its circumstances were such as to make it difficult to estimate definitely what the discharge would be, but approximately 9,400 cubic feet per second might be taken. The Author had put the discharge at 3,370 cubic feet per second; but that could hardly be correct when the water in the reservoir rose to 212 R. L., or 4 feet above the crest of the weir, which was the intended top-water level. The total cost of the works was Rs.34,40,000, or say £215,000. The outlay, therefore, was £1 15*s.* 10*d.* per head of the population, which was a moderate expenditure if a reliable water-supply had been secured.

Lieut.-General John Mullins.

The Jeypore Waterworks afforded an interesting example of a well-planned attempt to construct a reservoir under very unfavourable circumstances. The soil on which and with which the impounding embankment had to be thrown up was of so sandy a character that impermeability could not be secured, and the problem was therefore to devise arrangements which would prevent leakage at the place where its consequences might be serious, viz., at the junction of the embankment with the masonry of the

Lieut.-General John Mullins. sluice culvert, and to guard against leakage under and through the embankment, which would lead to the gradual removal of the sub-soil or of the material of the dam.

The reservoir had apparently not filled since its completion in September, 1885, and the embankment had not therefore been subjected to the full head of water; but in that year a depth of 31½ feet of water was retained, or only 9½ feet less than the full depth. Had a water-tight foundation been obtainable, a puddle-wall would have rendered the superstructure impermeable, and then the sand which formed the embankment would have been in most respects a suitable material for the greater part of the dam, though something less liable to disturbance by wind and erosion by rain would still have been desirable as a covering for the slopes, and would have facilitated their protection by the growth of grass.

In this case moderate, or even considerable, leakage did not actually involve loss of water, because such leakage was intercepted by the weir at the pumping-station 750 feet below the reservoir. The reservoir scheme was confessedly one to make the most of a very scanty water-supply. The discharge from the surface of the land was, however, considerably supplemented by springs, with the result that the water in the reservoir was maintained nearly at the level attained at the end of the rainy season for several months; and, altogether, it seemed probable that the water stored would secure a fairly adequate supply to the city during the months when, otherwise, it would have fallen much below requirements.

The arrangements for the disposal of surplus water were not stated in detail, but the channel provided was situated at a safe distance from the dam, and no doubt its requisite capacity of discharge had been carefully considered. Small as was the ordinary discharge from the catchment-area, it might be assumed with certainty that sooner or later one of those exceptional rainstorms, which occasioned so much damage to public works of all kinds, would occur, and then the value of adequate provision for the discharge of flood-water would be appreciated. Probably the means of disposing of 3,600 cubic feet per second, with a depth of 3 feet or 4 feet of water at the crest of the outlet, would be found not an excessive allowance.

Mr. Reynolds. Mr. PLAYFORD REYNOLDS, referring to the Baroda Waterworks, doubted whether the second puddle-trench had any beneficial effect; it was certain that a considerable leak existed under the embankment. He believed that this leak was due to the existence of porous strata of earth under the embankment. It must be remembered that the puddle-trench in no place reached rock nor even true clay, and a leak of greater or less extent was a certainty. At the same time it was expected that the leak would cease in time, owing to the large amount of silt carried by the river and deposited in the reservoir. He took exception to the measure recommended by Mr. Whiting, who had been called in during Mr. Reynolds' absence in England, if it was correctly described in the Paper. The danger of founding on black soil was that, becoming saturated with water, it might slip or be displaced by the weight of the superincumbent embankment, and the best preventive measure against an accident of that kind was the thorough drainage of the black soil. Mr. Reynolds.

Professor HENRY ROBINSON thought that a Paper on "The Design and Stability of Masonry Dams,"[1] by Mr. W. B. Coventry, might with advantage be studied in connection with the valuable communication of Professor Kreuter. Attention might also be directed to the clear treatment of the same subject in the recently-published book on "Waterworks Engineering," by Messrs. Tudsbery Turner and Brightmore. He had had to advise in regard to the design of masonry dams, and, on more than one occasion, to combat the view that increased strength was gained by augmenting the thickness of the dam beyond the limits ascertained by the recognised methods of determining the profiles. The question of the discharging-capacity of culverts was raised by the Paper on the Tansa works, and he would emphasize the fact that calculations based on the older formulas were erroneous. The results of his investigations of that matter were set forth at length in his work on "Hydraulic Power." Prof. Henry Robinson.

Mr. T. FRAME THOMSON submitted a sketch, *Fig. 23*, showing comparative sections, calculated with the same data of (1) the Tansa dam; (2) an equivalent dam designed by the method described by Professor Kreuter; and (3) an equivalent dam designed by the method given by Messrs. Tudsbery Turner and Brightmore in their recent work on "Waterworks Engineering." As regards economy of material for the particular height of dam in question, viz., 135 feet, Nos. (1) and (3) exceeded No. (2) in area to the extent of $2\frac{1}{2}$ per cent. and $5\frac{1}{2}$ per cent. respectively. The diminished section of No. (2), compared with No. (3), was chiefly due to the equalising portion with an overhanging inner face, introduced by Professor Kreuter, shown hatched in the *Fig.* Mr. Thomson.

[1] Minutes of Proceedings Inst. C.E., vol. lxxxv. p. 281.

N 2

Mr. Thomson. The following formula facilitated the plotting of portion No. (3) in Professor Kreuter's method:—

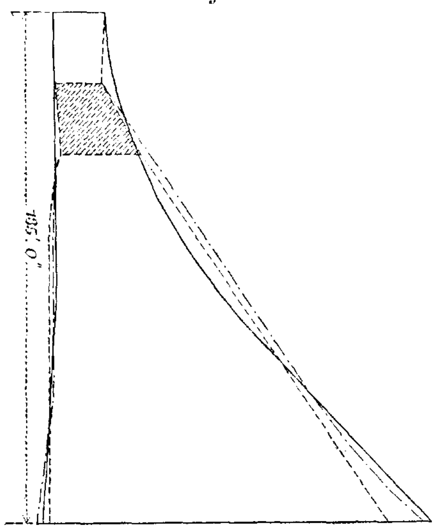
Let a_n be the area of the nth section.

A_n ,, total area down to and including the nth section.

g_n ,, horizontal distance between the inside toe of the $(n - 1)$th section and the centre of gravity of the nth section, plotted with the inner edge vertical under the inner toe of the $(n - 1)$th section.

G_n be the distance (one-third of the base) from the inside toe of the nth section at which the line of pressure cuts the base of that section.

Fig. 23.



Scale 1 Inch = 50 feet.

Tansa dam shown thus . . . . ———

Equivalent dam, designed by the method of Kreuter, thus . . . - - - - -

Equivalent dam, designed by Tudsbery Turner and Brightmore, thus —.—.—

p_n be the distance through which the base of the nth section must be moved beyond the vertical through the inner toe of the part above it, in order that the line of pressures (reservoir empty) should pass through it at one-third of its length from the face.

t_n be the thickness of the base of the nth section.

$$\text{Then } p_n = \frac{\frac{t_n}{3} - A_{n-1}G_{n-1} - a_n g_n}{A_{n-1}}.$$

Mr. Thomson.

The formula embodied two approximations to the result, and was sufficiently exact for practical use.

Mr. Tomlinson.

Mr. S. Tomlinson thought the following brief description of the works recently completed for the supply of Ahmedabad might be appropriately mentioned in connection with the water-supply of cities in Western India. Ahmedabad was situated 300 miles north of Bombay, near the River Sabarmati, and had a population, according to the census of 1891, of 148,412. The supply was drawn from closed wells sunk in the bed of the River Sabarmati. The four wells were built of brickwork, being 25 feet in diameter, and sunk to depths varying between 8 and 13 feet below summer water-level—their coverings being about 4 feet below the surface of the river-bed. The water was collected from them and conveyed to the jack well under the pumps by 24-inch cast-iron pipes running from the two nearest wells. The pump-well was situated on the banks of the river, and was sunk to a depth of 10 feet. There were two pumps and two engines. The pumps were vertical, with cylinders 28 inches in diameter and plungers 20 inches in diameter. The engines were compound surface-condensing, with 12-inch and 23-inch cylinders of 36-inch stroke. The steam-pressure, generated in Lancashire boilers, varied between 90 lbs. and 100 lbs. per square inch. The delivery-main from the pumps to the elevated tank was of cast-iron, 27 inches in diameter. The suction varied, but might be taken as about 18 feet; the lift was 75 feet. The fuel used was wood, of which about 400 lbs. per hour was consumed at a cost of 4 to 5 pies per 1,000 gallons pumped (192 pies = Rs. 1). There was a high-level tank, which contained 160,000 gallons, supported on a brickwork tower. The tank was of steel, 18 feet deep and 42 feet in diameter, the bottom being 40 feet above the floor-level. It was provided with 27-inch inlet- and outlet-pipes. The outlet-main to the city was about $2\frac{1}{2}$ miles in length. The works were opened in June 1891; the total cost had been Rs. 7,76,800. The scheme was designed to afford a supply of 10 gallons per head per day to the population. The water was supplied for ordinary domestic purposes at a charge of $6\frac{1}{4}$ per cent. on the assessed value of the premises, with connections, $3\frac{1}{8}$ per cent. on those premises without connections, and for trade purposes by meter at 8 annas per 1,000 gallons. The net cost to the Municipality, allowing for interest and sinking fund, was about $2\frac{1}{2}$ annas per 1,000 gallons.

On the completion of the Tansa works and the retirement from municipal service of the Author of the Paper, Mr. Tomlinson had taken charge of them, and had much pleasure in stating that the

Mr. Tomlinson. experience of the past year had been of a most satisfactory character. No accident calling for comment had occurred in any portion of the works, and the water had only been shut off once for the annual inspection of the interior of the conduit and for trifling alterations and repairs to sluices, &c. Referring to the rainfall of 1893, the lake overflowed on the 20th June after 26·63 inches of rain had fallen. As the draw-off from the lake was in 1892–93 almost as great as the one line of pipes now laid could carry, that fact showed clearly how small a proportion of the rainfall was utilized.

The highest water-level in 1893 had been 1·75 feet over the waste-weir. The rainfall registered at the dam on five days preceding that record being—

	Inches.
17th June	3·68
18th "	8·53
19th "	7·09
20th "	5·12
21st "	6·22
22nd "	2·24
	32·88

The highest level registered was on the 22nd June at 8 A.M. (406·75.)

The outlet arrangements were simple and efficient, but it was not clear why the sluice-shutters should not have been on the outside of the dam, where the rods would have been exposed and could have been painted or renewed readily.

Observations had been made daily of the temperature of the water in the Tansa lake, and at the upper and lower ends of the siphons.

The particulars might be taken as typical of any 48-inch pipe fully exposed to a tropical sun and discharging about 20,000,000 gallons daily; 8° F. had appeared to be the maximum rise of temperature from the upper to the lower end of a siphon about $11\frac{1}{2}$ miles long. At the north end, 9 A.M., and at the south end, 3 P.M., gave the greatest differences, as might have been expected. It might be noticed that the minimum temperature registered was 73° F. at 6 A.M. on the 1st February, and the maximum temperature was 95° F. at 3 P.M. on the 15th May, 88° F. being the maximum 6 A.M. reading. Those readings accorded with others taken at the Vehar and Tulsi lakes, and at the service-reservoirs in Bombay.

The Tansa water was distributed unfiltered, and the monthly analyses had indicated: Mr. Tomlinson.

		Free Ammonia.	Albumenoid Ammonia.			Free Ammonia.	Albumenoid Ammonia.
		Parts per Million.	Parts per Million.			Parts per Million.	Parts per Million.
1892.	8th April	0·06	0·12	1892.	1st Nov.	0·005	0·05
„	30th „	0·05	0·12	„	2nd Dec.	0·01	0·06
„	1st June	0·01	0·06	1893.	5th Jan.	..	0·04
„	4th July	0·05	0·09	„	21st Aug.	0·04	0·05
„	1st Aug.	0·05	0·08	„	3rd Oct.	0·01	0·06
„	1st Sept.	0·005	0·04	„	6th Nov.	0·005	0·10
„	3rd Oct.	0·005	0·04	„	4th Dec.	0·03	0·07

Mr. E. Wegmann, Jun., considered that the formulas proposed by Professor Kreuter for determining the sections of dams were simple compared with those of Sazilly, Delocre and Pelletreau,[1] and were more general than those given by Rankine[2] for a profile-type bounded by logarithmic curves. He did not think, however, that Professor Kreuter's method satisfied all the practical requirements of the case. Formula No. 1 was simple, though not new, as it was used in 1884 in the calculations for the proposed Quaker Bridge dam. Formula No. 2, which was also very simple, was obtained by making the inner face of the wall overhang the lower portion. As this arrangement saved but a small amount of masonry, and produced some tension in the dam, it was not to be recommended. In all the profile-types which had thus far been proposed, the inner face of the dam was either made vertical near the top, or was slightly battered. If the condition were introduced that the inner face of the wall was to be vertical until a joint was reached where the line of pressure, reservoir empty, was situated one-third of the breadth from the inner face, or where the limiting pressure was reached (as the case might be), Professor Kreuter's formula No. 2 could not be applied. An equation to suit that case could easily be found, but it would contain two unknown quantities (viz., the depth below the top of the dam and the length of the joint), and could, therefore, only be solved by trial. Formulas Nos. 3 and 4 enabled the thickness of the dam to be calculated for that part in which every horizontal joint was trisected by the two extreme lines of pressure (*i.e.*, reservoir full and reservoir Mr. Wegmann, Jun.

[1] See the "Annales des Ponts et Chaussées" for 1853, 1866, 1876, and 1877.

[2] See "Miscellaneous Scientific Papers of Professor Rankine," London, 1881, or the *Engineer*, January 5th, 1872.

Mr. Wegmann, Jun.

empty). The auxiliary expression given in connection with formula No. 4 was not satisfactory as it could not be directly integrated. The problem could be better solved by means of equation 6 of the formula used in designing the Quaker Bridge dam, given hereafter. Professor Kreuter's formula No. 5 would give the dam sufficient thickness to keep the maxima pressures within the fixed limits of safety, but would not determine the minimum area of profile which fulfilled the requirements. An unnecessary amount of masonry in a dam involved, not only a waste of material but also increased pressures in the structure. Whilst formula No. 5 kept the maxima pressures at the up-stream face at the given limit, it did not do so with the maxima pressures at the down-stream face; and consequently made the profile larger than was necessary. By keeping the line of resultant pressures, reservoir empty, on the up-stream limit of the centre third of the profile (a condition for which no necessity was evident), the thickness of the dam was made to increase very rapidly. Professor Kreuter had stated that for a certain depth, formula No. 5 would make the lines of pressure, reservoir full and empty, fall in the same half of the profile. That would lead to an extravagant thickness for a dam of considerable height.

Rankine had recommended that a lower limit of vertical pressure should be assumed for the down-stream than for the up-stream face, in order to compensate for the increased pressures in an inclined direction which occurred when the reservoir was full; and this was probably a safe course. If it were desired to comply with the condition usually adopted by writers on the subject of masonry dams of keeping the maxima pressures at both faces of portion No. 3 exactly at the limit of safety, formula No. 5 could not be applied. With reference to the question of including or omitting the vertical component of the water-pressure in the calculations, he would recommend the latter course for the following reasons: The first writers on the subject of masonry dams—Sazilly and Delocre—assumed limits as low as 4 to 6 tons per square foot for the maxima pressures. With this assumption considerable slope had to be given to the inner face of a dam, and the vertical component of the water-pressure became consequently a factor which could not be omitted in the calculations. Since those writers had designed their profile-types, the tendency in constructing dams of masonry had been steadily towards higher pressures. Rankine adopted a limit of about 8 tons per square foot for the maxima pressures at the outer face and 10 tons at the inner face. With these data, the inner face of a dam became almost vertical for a considerable depth below the top, and the error (in the direction of safety) resulting from omitting to consider the vertical component of the water-pressure was trifling. For a depth of 160 feet the error in the maxima pressures at the outer face would be only about 5 per cent. with the data assumed by Rankine. By including the vertical component of the water-pressure the calculations became very complicated, and, after all, the actual maxima pressures were not ascertained, and were probably under-estimated, as the usual formulas by which they were calculated did not take into consideration the elasticity of the masonry. Rankine had recommended that the maxima pressures at the down-stream face should be diminished as the slope of the wall increased in order to make allowance for the inclined direction of the stresses at the face of the wall. As in the present state of knowledge, the law of this diminution of pressure was not known, Mr. Wegmann advocated reducing the pressures at the outer face by omitting the vertical component of the water-pressure in the calculation. Mr. Wegmann, Jun.

The problem of finding the profile of minimum area for a masonry dam was exceedingly complex, as the different portions of the wall were not determined by the same conditions. In the upper courses the positions of the lines of pressure, reservoir full and empty, had to be considered; while in the lower courses the limit of the maxima pressures was the controlling condition. In making calculations for the proposed Quaker Bridge dam (a reservoir wall across the Croton Valley, which was to have a height of about 260 feet from the foundation to the top), he had carefully studied all formulas for determining profiles for dams of which he could find a record, and had endeavoured to apply them to the problem, without much success. The methods of Sazilly, Delocre, and Pelletreau, which were lengthy and complicated, could not be used with high limits of pressure, as they did not confine the lines of pressure to the centre third of the profile. The logarithmic curves proposed by Rankine gave only an approximation to the profile required by the conditions he assumed. Krantz, Crugnola, and Harlacher had published good profile-types, but had given no formulas to vary them for different data. Sir Guilford Molesworth had given an empirical formula for determining the profile of a dam, and Mr. W. B. Coventry had proposed a method [1] consisting partly of trial calculations and partly of the use of equations.

All the formulas proposed for determining the profile of a dam

[1] Minutes of Proceedings Inst. C.E., vol. lxxxv. p. 281.

Mr. Wegmann, Jun. (including those of Professor Kreuter) involved, more or less, trial calculations. With the complicated conditions of the problem it was not likely that anyone would succeed in finding exact general formulas for determining the profile of minimum area. While it was difficult to find at once a practical profile fulfilling the given conditions, it was comparatively easy to determine the exact thickness of the dam, at regular intervals, commencing at the top. If the intervals were taken sufficiently small (say, 10 feet), and the vertical component of the water-pressure were omitted, a correct theoretical profile of a dam could be calculated by the equations which were used for the Quaker Bridge dam, fully set forth in a work written by him a few years ago.[1] He had determined the first profiles for the Quaker Bridge Dam, to fulfil the conditions laid down by the Chief Engineer, by a simple but rather laborious system of trial calculation, consisting in commencing at the top of the dam, and finding the required thickness at regular intervals by trial, verifying the conditions by taking moments about a vertical axis. He had subsequently improved this rather primitive system by finding correct equations. The profile found by those equations would have polygonal faces, but by taking the depths of the several layers considered sufficiently small the profile of minimum area could be approached as closely as desired.

Having found the profile which contained practically the minimum area which fulfilled the given conditions, it only remained to simplify its outlines by substituting a few straight lines or curves for the angles of the polygonal faces, in order to obtain a practical design. Small changes in the outlines for this purpose had no appreciable effect. The profile obtained in this manner might not fulfil the given conditions exactly, but it would certainly do so practically; and as the distribution of pressures in masonry was unknown, it seemed to be a useless mathematical refinement to insist on greater accuracy than that which resulted from the method referred to. The reservoir wall, known now as the New Croton dam, was being constructed at a place $1\frac{1}{8}$ mile further up-stream than its original location. Its maximum height above the foundation would be about 240 feet. The storage-reservoir for New York, which it would form in the Croton Valley, would contain about 30,000,000,000 United States gallons.

Mr. Whiting. Mr. J. E. Whiting, referring to the form of the masonry conduits and lined tunnels of the Tansa works, thought it manifest that the

[1] "The Design and Construction of Masonry Dams." New York, 1888.

lower corners should have been rounded off, the extra cost of a slightly modified section would not have been great, and its advantage as regards cleanliness would have been considerable. Nothing had been stated in this Paper as to the quality of the water from Tansa; that it was better than the water received from Vehar and Tulsi appeared from the following analyses: Mr. Whiting.

Analysis taken 6th November, 1893, of water drawn from the Tansa main at the outskirts of the city: free ammonia, 0·005 part per million; albumenoid, 0·10 part per million. Tulsi water gave free ammonia from 0·03 to 0·04 part per million; albumenoid, 0·12 to 0·14 part per million. Vehar water gave free ammonia 0·03 to 0·04 part per million; albumenoid, 0·12 to 0·15 part per million. An analysis of Tansa water taken from the main on 8th June, 1893, showed, total solids, 9·10 grains per gallon; chlorine, 0·98 grain per gallon; free ammonia, 0·03 part per million; albumenoid, 0·09 part per million.

When considering the splendid work described, it seemed a pity that either distinct provision had not been made for the utilization of the whole storage hereafter, or that the dam had not been designed of less thickness. It appeared that the lined tunnels and conduits would carry about double what could pass through the one cast-iron main already laid. The aqueducts across the Bassein creek and elsewhere were constructed to carry a second pipe, and that Bombay would ere long require the second main was probable. But, if the double storage should be required, how would the extra water be got out of the reservoir? The present outlets, would apparently not permit that; fresh tunnels could perhaps be formed, and the conduits be doubled in width. Probably new cylinders would have to be sunk for extra girders across the creeks; or could the present iron-work be strengthened and the new pipes be carried on the top of the present ones? All those matters might be arranged, but new outlets through the dam would be troublesome to make. As regarded the bursting of some of the pipes, it was to be regretted that the Author had not given his reasons for inclining to the opinion that the want of air-valves in that portion of the main was the cause of failure. Air in a pipe could not of itself increase the pressure; on the other hand, if any hydraulic ram occurred in a pipe, the presence of air would modify that and reduce its bad effects.

Mr. Clerke. Mr. CLERKE, in reply to the correspondence, observed that the Tansa pipe-joint, though closely following that published by Mr. Fanning some years ago, was not identical with it. The dimension *m n*, on the drawing of the Tansa pipe, exceeded the cor-

Mr. Clerke. responding dimension of Mr. Fanning's drawing by $\frac{1}{3}$ inch, giving greater substance in the shoulder of the socket and adding 80 lbs. to the weight of the pipe. The form and dimensions of the spigot of the Tansa pipe differed from those of Mr. Fanning's. With reference to Lieut.-Gen. Mullins' remarks on the cost of storage at Tansa, he must observe that the necessity for drawing off at a high level to command Bombay left a much greater quantity of water unutilized than would be the case in a similar work constructed for irrigation purposes. In the latter case, the cost of the storage at Tansa would be about Rs.705 per million cubic feet. The information given by Mr. S. Tomlinson in reference to the maximum flood of 1893 and the rainfall immediately preceding it, emphasized Mr. Clerke's remarks in his reply to the discussion, in which he had pointed out that the rainfall at the dam site could not be regarded as an index of the general rainfall on the catchment-area. He had to express his gratification at the interesting and able remarks of both a practical and theoretical nature on the general subject of masonry dams, which the discussion and correspondence on the group of papers had elicited from the engineers who had taken part therein.