DAMS AND STORAGE RESERVOIRS.

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Punjab areas.

The gross area of the Punjab Province is about 87 million acres or 135,000 square miles ; out of which about 62 million acres or 97,000 square miles are British territory, and the remainder belong to Feudatory States. Out of the 62 million acres of British territory 16 million acres (including three million acres of forest) are not available for cultivation ; 29 million acres are cultivated (including fallows) ; and 17 million acres of culturable land are lying waste for want of rainfall or artificial means of irrigation. Thus out of the 46 million acres of culturable British territory, 37 per cent is actually lying waste for want of the moisture necessary to produce crops.

Cultivation and Irrigation.

2. Out of the 29 million acres of cultivated British territory about 10 million acres are irrigated, *viz.*, seven million acres canals and three million acres by wells. The remaining millions of acres are dependent on a precarious and uncertain When the rains fail, there is grave risk of famine, g in heavy expenditure by the Government in relief works.

data.

Out of a gross area of about 42 million acres (including x millions of acres in Feudatory States) commanded by Government Canals, about seven million acres (including alf a million within the said Feudatory States) are irrigaually by these canals. The combined aggregate flow existing perennial canals in the Punjab amounts to ,000 cusecs, flowing continuously on the average the summer ; and about the same volume throughout The combined aggregate flow of all the Inundation about 23,000 cusecs throughout the summer. .., only. So that, roughly speaking, we may say combined aggregate flow of all the Punjab Government verages 35,000 cusecs throughout the year ; and with an area of seven million acres is irrigated annually. from these figures, about half a million of acres in British are irrigated by privately-owned canals flowing in the ason only).

4. Out of the 62 million acres of British territory in th Punjab about 22 million acres lie in districts not commanded by the flow of our irrigating canals. About six million out of these 22 million acres may be commanded by canals built hereafter (and already contemplated), but the remaining 16 million acres lie above the level of the flow of any of our canals, either existing or contemplated.

5. The available perennial stream-flow of every one of our Punjab rivers, except the Indus and its tributary the Beas, has already been fully taken possession of by our canals. The Triple Canal Project, now under execution, will increase the area annually cultivated by about two million acres. The water of the Beas will be delivered mostly to the Feudatory States of Bahawalpur and Bikanir. So that for British territory only the water of the Indus remains unexploited; and this may be applied to the six million acres referred to in the preceding paragraph.

Necessity for extension of irrigation.

6. We are faced thus with the fact that there are 17 million acres of culturable land in British territory lying for want of rainfall or the means of irrigation, and that our 29 millions of acres of land now under cultivation, 19 mil of acres are dependent entirely upon a precarious rainfall the production of crops. These 36 millions (19+17) of of non-irrigated soil may be reduced to, say 30 million the completion of the Triple Canal Project and the Indus Project; but it is easy to see that there will still remain sc more than doubling the canal-irrigated area of the Punjab in territory after all possibilities of utilizing the perennial flow rivers have been exhausted. The only way of increasi irrigated area, after the perennial supplies have been exhaust be that of storing up in reservoirs, for use in the dry sea surplus and enormous floods of these rivers during the fo of the monsoon. The object of the present paper is the possibilities of storage reservoir construction in t

Storage Problems.

7. Practically all streams of any consequence have their sources in the Himalayas. In the interio Himalayan ranges conditions are not as a rule favourable t reservoir construction. The valleys are narrow and pr and the declivities of the streams steep; so a dam of any able height would afford comparatively small storage The declivities of the streams diminish in steepness

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approach their exit from the Himalayas, and the valleys also widen out more frequently; but the rocks here are of later formation than those further up stream, and ideal foundation-soil for high masonry dams is more difficult to meet with. These are the conditions which will usually hamper our projects for the construction of storage reservoirs.

Dam Foundations.

8. Where we have a solid foundation of hard rock practically impervious to water and free from flaws or fissures, the design of a high dam is a comparatively simple matter, but, as I have remarked above, the conditions we are likely to meet with in the Punjab will generally be less satisfactory than we could wish for; and I propose to discuss in this paper the possibilities and methods of dam construction on foundation soil not of the best description. I will consider first the case of dams of masonry or concrete; and with this purpose will review briefly the history of this type of construction up to date.

Early Dams.

9. The earliest examples of high masonry dams, from 60 feet to 160 feet in height, seem to have been built in Spain at the end of the 16th century. We have no record of the theories or principles underlying their design; but they were, roughly, rectangular or trapezoidal in profile, with a base-thickness varying from $\frac{1}{2}$ to $\frac{3}{4}$ of the height and they have proved generally stable and efficient for their purpose up to the present.

The Modern Theory of Dam Design.

10. There was a lull in dam-construction throughout the greater part of the 17th and 18th centuries; but some more dams were built in Spain and in France at the end of the 18th century and early in the 19th, notably the Zola Dam with a base only 42 feet thick and a height of 120 feet.

It was not till the middle of the 19th century, however, that the elements of the modern theory of this branch of science were enunciated by M. de Sazilly, who specified the following conditions as the basis of design:—

1. The pressures sustained by the masonry on its foundation must not exceed a certain safe limit, *viz.*, 5·5 tons per square foot.
2. There must be no possibility of any portion of the masonry sliding on that below, or of the whole wall moving on its foundations.

Sazilly.

11. The profile of dam recommended by M. de Sazilly had a base-thickness approximately equal to its height; *viz.*, a base-thickness of 171 feet for a height of 164 feet.

Sazilly's theory was accepted, with some modification, by M. Delocre, who designed, accordingly, the celebrated Furens Dam, 164 feet high and 161 feet thick at base.

This work was completed in the year 1866; and it has been regarded as "the first dam which has been built in accordance with correct scientific principles." In its design the greatest pressure was limited to six tons per square foot, and as a further precaution it was curved up-stream in plan, so as to resist the pressure of the water by horizontal-arch action as well as by its weight and base-thickness.

Blanc.

12. Under the Sazilly-Delocre design, the vertical component only of the resultant thrust on the dam when holding up water was taken into account when calculating pressures; but in the year 1856 M. Blanc argued that the inclined resultant thrust on the dam exerted a greater intensity of pressure against the plane normal to its own direction than was exerted by its vertical component against the horizontal plane.

Rankine.

13. About the year 1870 Professor Rankine, when advising on the construction of a dam at Bombay, laid down the important principle, which has generally governed most designs for masonry structures ever since then, that no tension must be allowed to occur in the masonry; and that with this view the resultant thrust on each horizontal section of the profile of the dam must fall within its middle third.

He also advised the fixing of a lower limit of pressure at the down-stream face of the dam than at its up-stream face, and for the following reasons:—

"The direction in which the pressure is exerted amongst the particles close to either face of the masonry is necessarily that of a tangent to that face, and unless that face is vertical the pressure found by means of the ordinary formulæ is not the whole pressure, but only its vertical component; and the whole pressure exceeds the vertical pressure in a ratio which becomes the greater

B he greater the batter, or deviation of the face from the vertical. The outer face of the wall has a much greater batter than the inner face ; therefore, in order that the masonry of the outer face may not be more severely strained when the reservoir is full than that of the inner face when the reservoir is empty, the lower limit must be taken for the intensity of the vertical pressure at the outer face than at the inner face."

And the limits of pressure which he imposed were:—

At the down-stream face	...	7	tons per square foot
At the up-stream face	...	9 do.	do.

He designed his profile so as to give the same maximum pressure at a depth of 150 feet, *viz.*, six tons per square foot, as the Furens Dam at that depth; and he reduced the maximum pressures below that depth proportionately to the increased divergence of the face-slopes from the vertical, and he gave logarithmic curves to the faces of his dam in profile.

Bouvier.

14. In 1874 M. Bouvier applied the principles of M. Le Blanc, mentioned before, to the case of the Ternay Dam, giving it a base-thickness equal to 73 per cent of its height up to water-surface. At a depth of 102 feet below water-surface the Ternay Dam is 70 feet thick, the Furens Dam being 74 feet thick at the same depth, and Professor Rankine's type profile 67 feet.

Under M. Bouvier's design for the Ternay Dam the maximum vertical pressures in the masonry were limited to $6\frac{1}{2}$ tons per square foot ; and the maximum pressures in the direction of the resultant thrust to $8\frac{1}{4}$ tons per square foot. The co-efficient of friction necessary to prevent sliding was limited to 0.76

Atcherley.

15. The authorities mentioned in designing profiles for dams, in the plane of ction of the water pressure, confined themselves to the consideration of the intensities of pressures on the horizontal laminæ of that plane of the profile, or on laminæ normal to the direction of the resultant thrust, and this treatment of the subject prevailed unchallenged to the end of the 19th century ; but in the year 1904 Professor Atcherley, of London, raised the question of stresses in vertical laminæ of the plane of the profile, and remarked :—

"We believe that a great number of dams as now designed will be found to have very substantial tension in the tail, and this is, in our opinion, a source of weakness in dam construction which has not been properly considered and allowed for."

Experiments.

16. The latter half of the 19th century was remarkable for the feverish activity and ingenuity displayed by mathematicians in their efforts to solve the general problem of the design of high masonry dams. Only a very few of the numerous theories and calculations propounded by different investigators have been mentioned, *viz.*, those involving the most marked differences of principle.

These mathematical disscussions are interesting because of the scope they have furnished for mental exercise, but with the dawning of the 20th century more reliance has begun to be placed on experimental observation than on mere mathematics. Professor Atcherley based his arguments on the results of experiments in which pressures representing practical working conditions were applied, by means of weights and pulleys, to model dam-profiles made of strips of wood laid respectively vertically or horizontally to represent vertical or horizontal laminæ of the structure.

Sir Benjamin Baker, Past President Inst. C.E., experimented a few years ago with models of dams made of jelly, with horizontal and vertical lines drawn upon the side to show the location of distortion produced by applying pressure.

Messrs. J. S. Wilson and W. Gore, Assoc. M. M. Inst. C. E., experimented in 1905 with weights applied over pulleys to models made of indiarubber ; and still more recently similar experiments have been made with models made of plasticine.

Messrs. Wilson and Gore, as the result of their experiments, corroborated Messrs. Atcherley and Pearson in their assertion of the occurrence of tensile stresses, in the toe of the dam, notwithstanding the latter having been designed with the lines of pressure well within the middle third of the profile ; and they expressed the opinion that in order to eliminate tensile strains it is desirable to give either a greater height of freeboard above water-surface level, or a greater top-thickness, or to make the up-stream face vertical, or to increase the section materially and back the dam with an embankment of earth on the reservoir side; but Sir B. Baker, as the result of his experiments, did not agree entirely with the Atcherely-Pearson conclusions.

Buoyant Forces.

17. Of late years increasing attention has been paid to the question of buoyant forces tending to uplift a dam, resulting from water percolating under pressure from the reservoir through the material of the dam or through its foundation-soil. On this point a great variance of opinion exists within the profession.

Thus if the figure A B C D (figure 1,) be taken to represent the profile of a dam with foundation-base C D and crest A B flush with the water surface A X of the reservoir formed by it, the ultra-conservative wing of the profession, composed of those who aim at the completest safety regardless of cost, contends that the buoyant pressure should be considered as acting over the whole base C D, under a head equal to A C, the full height of the dam ; the diagram of buoyant force being represented by the rectangle A C D Y. This view involves the supposition that somewhere down-stream of the dam there is an impermeable barrier of natural soil which may hold up to the level A Y, or to the F. S. L. of the reservoir, the percolation-flow passing under the foundation of the dam. It is scarcely conceivable that conditions justifying this opinion would ever occur in actual practice.

A more moderate view is that of those who calculate that, assuming the point D to represent the level of free percolation outflow down-stream of the dam, a buoyant force of head A C at up-stream face of the dam, and diminishing to nil at the point D, and represented by the triangle A C D, ought to be reckoned with. Even this view involves the unlikely supposition in the case of a rock-foundation that the soil under the foundation of the dam offers little or no resistance to percolation-flow, and is as permeable to such flow as pure, loose sand.

Other engineers, like Mr. Edward Wegmann, of New York, hold that there is no positive information on the subject of buoyant forces acting on dams ; that high masonry dams should be founded, as a rule, only on hard, solid, comparatively impermeable rock ; and that in most cases, therefore, buoyant pressures will be exerted only on a very small area of the base of a dam, through flaws or seams in the foundation-rock.

Some engineers, again, aim at obviating the occurrence of buoyant pressures by making the up-stream face or skin of the dam water-tight, and draining off all possible percolation by means of horizontal drains or weep-holes in the body of

the dam and at its base. Others even are of opinion that water saturating the material of a dam increases its weight, and hence its stability, and that drains in the body or at the base of a dam are harmful, as encouraging the increase of percolation-flow.

Finally, it is possible that most Punjab canal engineers will agree with the writer of this paper that in the case of a seamy or porous foundation-soil, a cheap and effective way of reducing the intensity of probable buoyant forces acting on the base C D of the dam in Fig. I would be arrived at by laying a comparatively thin but impermeable layer or apron of concrete up-stream of the dam such as E F, whereby the line of maximum possible hydraulic pressures would be set back to the gradient G D striking the face of the dam A C at the point H, and reducing the buoyant pressure area to the triangle H C D as a maximum. In the diagram (Fig. I) I have, for convenience of illustration, assumed the free water surface down-stream of the dam to be level with its base C D, though in practice it will usually be much higher as, for instance, at L M, the stream-bed or valley-bed level, or higher.

Theory and Practice.

18. It is not possible within the limits of this brief paper, to deal in any but the most general and cursory terms with the subject of the design of high masonry dams. Enough has been written, however, to show that no precise canons of design hold the field with the common consent of the profession, and that nearly every salient feature or principle of design is open to dispute and has been disputed in the past.

The elaborate mathematical disquisitions which were so much in vogue and which commanded so much respect in the latter half of the 19th century, seem now to have been honoured more on paper than in practice.

Typical Dams.

19. I have referred above to the Furens Dam. Schuyler, in his treatise, says of it :—

"Among many engineers this famous dam is recognized as a model of correct form, profile, and dimensions, whose outlines conform closely to what are accepted as certainly safe and well-balanced proportions throughout, even though the volume of material may be slightly excessive."

And Wegmann says of it :—

"It is the first masonry dam which has been built in accordance with correct scientific principles."

The Ternay Dam was built as an improvement on the Furens style of design, or at any rate on later and more cautious principles, so we may take these two designs as typical of the French school.

The Vyrnwy Dam in Wales may be taken to be typical of English ideas of sound engineering design, whilst the Tansa Dam at Bombay, which was built presumably in accordance with Professor Rankine's advice, may also be regarded as typical of the English school of engineers.

Similarly the Gileppe Dam, near Brussels, may be regarded as a typical Belgian design.

And amongst the latest examples, typical of American practice, may be mentioned the Roosevelt, Shoshone, and Pathfinder Dams, built since 1905 by the United States Reclamation Service, and the Cross River, Olive Bridge, and Kensico Dams, built or building, for the Municipality of New York.

20. The relative stature, bulk, and solidity of these typical works are compared in the statement furnished below :—

Name of Dam.	Date of Completion.	Height (feet).	Percentage Ratio of Thickness to Height.	
			100 feet below water.	At Foundation.
Almanza	1580	68	...	50
Alicante	1594	140	99	79
Zola	1843	120	37	35
Furens	1866	164	74	98
Ternay	1874	113	70	73
Gileppe	1875	156	160	139
Vyrnwy	1890	136	114	91
Tansa	1891	118	85	85
Pathfinder	1910	210	50	45
Shoshone	1910	305	50	35
Roosevelt	1910	260	56	61
Cross River	1908	155	69	74
Olive Bridge	1912	200	77	95
Kensico	...	250	230	92
Tytam	1887	122	66	63

Effect of Science on Designs.

21. We see that in the early dark ages of dam-construction, Spanish dams were built with a base-thickness from 50 to 80 per cent of their height ; whilst in the case of the Zola Dam in France this ratio was only 35 per cent ; and these dams have proved stable and efficient down to the present day. But with the construction of the Furens Dam in 1866 the scientific era set in, and with it a great increase in bulk of material : the thickness of the Vyrnwy and Gileppe Dams, at a depth of 100 feet below water-surface being actually greater than their height above that depth. The dimensions of the Gileppe Dam in fact register the extreme of extravagance in design ; an extravagance which must mainly be ascribed to undue timidity and lack of perception. But since the dawning of the 20th century a greater degree of confidence seems to have been restored to the leaders of our profession, especially in America, where the United States Government engineers have built some of the highest masonry dams in the world of a slenderness surpassing that of most dams built in the latter half of the 19th century. The Olive Bridge and Kensico Dams, however, of the New York City Waterworks, now under construction, indicate a natural human desire to err on the side of safety in important works close to large cities where the engineers have *carte blanche* as to expenditure.

Vagaries in Design.

22. I have alluded to the influence on practical construction of the mathematical treatment of dam designs in the ...century. The tabular statement (page 57) illustrates my meaning by showing the great differences in thickness of dams at 100 below the water-surface in the case of the Furens, Vyrnwy and Gileppe Dams. The Furens Dam being, to our knowledge stable and efficient enough for all purposes, why was the Vyrnwy Dam made 50 per cent thicker and the Gileppe Dam more than twice as thick ?

In the Furens design buoyant forces were not taken in account, whilst in the Vyrnwy and Gileppe designs they were taken into account. We have no evidence that in either of the latter two cases the foundation-soil was more porous than in the former case ; and if an engineer is at liberty, without obvious reason, to adopt any theory he chooses as to buoyant forces, and to vary his dimensions by cent per cent in consequence, the resultant design would appear to be more a matter of guess-work than

an of precise calculation, and from this point of view the
Economy ver thematics of the design would seem to have more of academic
27. In th n of practical interest. It is as though one were to calculate
like the Vyrnwy limension laboriously to the tenth place of decimals and then
funds are plenti opt the nearest whole number, in which case the obvious reflec-
peace of mind on arises that one need not have laboured the decimal places !
the vicinity o ankine prescribed logarithmic curves for the up-stream and
Rat making assur wn-stream faces of his model profile. He did not attempt to
dopment of th ermine the ratio which the limits of the vertical pressures at
detonomic nece two faces ought to bear to each other by any mathematical
theesigns based iculation as he deemed the data for such calculation to be too
catrength and certain. He designed his profile so as to give the same maxi-
um pressure at a depth of 150 feet below water surface as
Economy he Furens Dam, *viz.*, 5½ tons per square foot.

28. I But the Almanza Dam had, by its existence for 300 years,
tion of deser nown that a pressure of 14 tons per square foot was safe, and
prise, and he had followed that example he would have arrived at a
irrigation in lifferent profile. As a matter of fact, it is now generally recog-
tion of a ch nized that a dam may safely be subjected to pressure of 14 to
he situatio 18 tons per square foot ; and there is good reason to believe
th foundat hat much higher pressures are permissible in good stone masonry
Ihy such cas or concrete.
d ms have
va rious de

Crushing Strength of Masonry.

ste el or co 23. Laboratory experiments with small blocks of
hydraulic stone have shown that most kinds of rock are capable of sustain-
In ing in blocks of those sizes, pressures exceeding 500 tons per
oth. square foot. And there is good reason to believe that larger
etween blocks are capable of bearing *relatively* greater intensities of pre-
irrigsure ; so that a block of stone, say two inches square in plan and
meri of any given height would sustain a pressure 16 times as great
paid (not merely 4 times as great) as a stone of the same quality and
arge height but only one inch square in plan. For, the larger the
ture block for any given height, the greater is the resistance per unit
ams of surface to crushing, owing to greater lateral support to its
component particles.

Masonry built of such stone has less compressive strength
han the stones forming it ; but this reduction of strength is
more than compensated for in the case of large high dams by
the great *bulk* of the masonry. Where, then, is the necessity
for assuming the safe crushing strength of good stone masonry
concrete in such structures to be so low even as 18 tons per
uare foot ?

Material of the Vyrnwy Dam.

24. In the case of the Vyrnwy Dam 9-inch cubes made of the concrete material as actually used in the work showed a crushing strength of 187 tons per square foot at one year's age, and larger blocks of the same material showed a strength of 300 tons per square foot; yet in its design the maximum crushing stress to which the dam is subjected, even when a wind pressure of from 40 to 60lb. per square foot is taken into account, is less than 9 tons per square foot. The necessity for the imposition of so high a factor of safety is not obvious.

The Tytam Dam.

25. The Tytam Dam at Hong Kong presents some features of interest. Its foundation soil is anything but ideal, consisting as it does, largely of decomposed granite, clay, rotten shaley rock, and boulders. In such circumstances, if any, it might have been expected that buoyant forces would act on the dam; and during its construction arrangements for draining its bod and its foundations were actually provided; yet its constructor, M Orange, declared before the Institution of Civil Engineers th the Tytam Dam was far less porous than the Furens Da which latter "leaked like a sieve." The Tytam Dam is of m slender profile than the Furens or Ternay Dams, and of course, more slender than the Vyrnwy or Gileppe Dams; and before Institution of Civil Engineers, Mr. L. Vernon Harcourt poin out that the resultant pressure on it must fall outside the midin third of the base. Yet it has stood the test of the past 25 yest successfully and has shown no weakness, although when it only two years old an extraordinary flood, resulting from a rain of 27 inches within 24 hours, passed over the top of it.

Effect of Assumptions.

26. It comes to this then, that the mathematical inves gations of the stresses in masonry dams, the theories as to pre sure on horizontal or inclined planes, or of tension in vertical horizontal directions up-stream or down-stream, have much influence on the dimensions of the design adopted than assumptions as to forces of buoyancy or as to maxim permissible stresses in the structure. Such assumptions depend largely on guesswork, and vary with the idiosyncras the individual designer in each separate case.

Economy versus peace of mind.

27. In the case of dams built for great and wealthy cities like the Vyrnwy, Gileppe, Olive Bridge, and Kensico Dams, where funds are plentiful and economy is of less consequence than the peace of mind of large populations concentrated on small areas in the vicinity of the dam, it is natural that engineers should aim at making assurance doubly sure at all costs. But in the development of the Western States of America, the pressure of economic necessity has led to more economical and yet rational designs based on financial considerations equally with those of strength and safety.

Economy necessary in India.

28. In the Western States of America, where colonization of desert lands has been carried out mainly by private enterprise, and has been dependent on the storage of water for irrigation in reservoirs, the problem has often been the construction of a cheap dam or of none at all, and the exigencies of the situation have often necessitated the construction of the dam the foundation-soil which falls short of orthodox, scientific, ideals. In such cases necessity has proved the mother of invention, and dams have been built, generally with success and profit, by various devices—of timber, or steel, or of rocks lined with timber, steel or concrete or of reinforced concrete, or of earth and rocks hydraulically sluiced into position.

In India, also, we have to cut our coat according to our cloth. It is a poor country and the ryot's choice generally lies between a cheap coat or none at all. The water-rates leviable for irrigated crops are much lower than those leviable in America, and few reservoir projects in India have hitherto repaid their cost of maintenance in addition to the interest charges on the capital spent on them. Our problem of the future is therefore, how to build cheaper, though efficient, dams in connection with such projects.

American Methods.

29. Of the American methods of dam-construction, those which seem most applicable in India are the hollow-dam of reinforced concrete and the dam of rocks and earth sluiced into position by a current of water.

Reinforced Concrete.

30. In the reinforced-concrete dam the water-face, instead being nearly vertical as in the case of the orthodox types of

solid masonry dam, is usually a decking of reinforced concrete laid at a considerable inclination from the vertical, and supported on the down-stream side by piers or buttresses of masonry or concrete.* By this means the resultant stress can be made to fall either at the middle of the base of the structure or even up-stream of the middle. As the water-thrust is not horizontal, but inclined downward, whilst the dam is hollow between its piers, the pressure intensity on the base and the risk of sliding are relatively slight. There is no risk of overturn and the dam is comparatively cheap. Being hollow, this kind of dam is lighter and can be given a wider base and can be founded on a weaker and more porous foundation soil than a solid masonry dam of the same height.

Hydraulic Fill Method.

31. In the American "hydraulic-fill" type of dam, soil is torn up by a hydraulic jet under high pressure and carried by the resulting stream of water in wooden flumes at a steep gradient to the site of the dam, where it is deposited alluvially in the manner least likely to be displaced later on under water pressure. The stream carries rocks of all sizes up to about a ton in weight, besides sand and clay, and deposits the material in gradation of coarseness so as to form a core of pure clay at the centre of the dam with loam, sand, gravel, stones, and large rocks, in regular gradation from the centre of the dam to its up-stream and down-stream faces.

The soil is excavated and carried and deposited at site in the manner which is generally likely to be cheapest and most efficient.

Although the execution of ordinary earthwork by manual labour is cheaper in India than in most countries, it is not likely that the excavation and carriage of rocks and gravel could be accomplished in that way with anything like the cheapness, rapidity and efficiency of the hydraulic process.

Earthen Dams.

32. Many important reservoir dams of earth have been built in the south of India and in Rajputana even from the earliest ages. These are generally of stiff clay or of loam, in some cases protected on their slopes with loose rock or gritty material. In some dams of comparatively recent construction slips or partial failures have occurred, and Mr. Hill, lately chief Engineer of the Bombay Presidency, as the result of his

special experience, arrived at the conclusion that the profile of earthen dams should be so substantial that the horizontal distance between down-stream toe and reservoir water's edge should be four times the vertical distance between those points. This conclusion can be accepted by us only in the case of dams built almost entirely of stiff clay or of loam deposited by manual labour. In such dams the saturation of the clay by the reservoir has caused it to swell, move, or subside ; and it has often been found necessary to face the dams with heavy porous material and, in particular, to strengthen the downstream toe with a filter-bank of gravel or broken stone so as to prevent the outflow of the bulging core of soft clay. Expert opinion has arrived at the belief that the ideal earthen dam is one with a core of impermeable material (like clay, for instance), and outer zones of more porous and heavier material gradually increasing in coarseness from the core outwards both ways. This is exactly the result effected by the American "hydraulic-fill" process of earthen dam-constructing ; and it is a reasonable view that such dams are stable even when built of slighter profiles than those involved in Mr. Hill's specification of a four-to-one hydraulic gradient.

33. The reinforced-concrete type and the hydraulic-fill type before described are the types of dam that are likely to be most applicable to Punjab reservoir-projects on the score of cheapness, efficiency and adaptability to foundation-soil not of the hardest, impermeable rock ; and it is probable that even solid types of masonry or concrete will meet the requirements of some cases, provided we shake ourselves free of excessive reliance on European precedents, and adopt rational views of the appropriate assumptions as to porosity and safe resistance to crushing of masonry and foundation-soil in the cases concerned.

Examples.

34. Consider a few elements of design. Let A X (Fig. II) represent the water surface of a reservoir flush with the top of a rectangular dam A B C D which supports it. Assume that the height A B of the dam is 300 feet and the base B C 200 feet and that the specific gravity of the masonry is 2·4.

Case I.—Ignoring buoyant forces. The resultant thrust will strike the base at E on the border of its middle third.

The mean intensity of pressure will be 20 tons per square foot, and the maximum at down-stream toe will be 40 tons per square foot.

Case II.—Assume a buoyant force of head 300 feet acting uniformly under the base. The resultant thrust will strike the base at F on the border of its middle half. The mean intensity of pressure will be 11·7 tons per square foot and the maximum will be 35 tons per square foot. Tension will occur at the up-stream toe B, but there will be no risk of the dam sliding on its base, the ratio of horizontal to downward forces being only 0·54.

Case III.—Instead of the assumption in Case II, assume a buoyant force of 300-feet head at point B, diminishing gradually to *nil* at point C. The resultant thrust will strike the base on the border of its middle half at the point F (Fig. II). The mean intensity of pressure will be 15·9 tons per square foot and the maximum will be 48 tons per square foot at C. Tension will occur at the up-stream toe B.

Case IV.—Instead of the rectangle A B C D (Fig. III), assume the dam to be a right-angled triangle A B C, with up-stream face vertical. Ignore buoyant forces. The resultant thrust will strike the base on the border of its middle third. The mean intensity of pressure will be 10 tons per square foot and the maximum 20 tons per square foot.

Case V.—In Case IV add a buoyant force of 300 feet head acting uniformly under the whole base. The resultant thrust would strike the plane of the base at the point G, 70 feet outside the down-stream toe of the dam, and the dam could not stand except by tensile strength.

Case VI.—In Case IV add a buoyant force of 300 feet head acting at B and diminishing gradually to *nil* at C. The resultant thrust would strike the base at H 25 feet inwards from the point C. The mean intensity of pressure would be 5·9 tons per square foot, and the maximum 41 tons per square foot. If the dam could sustain this pressure without crushing it would be safe against overturn ; but it would not be safe against sliding on its base, the horizontal thrust exceeding its buoyant weight.

Case VII.—Instead of the rectangle A B C D (Fig. IV) let us suppose the dam is an isosceles triangle, A K C, 300 feet high and 400 feet base. Ignore buoyant forces. The resultant thrust will be practically at the middle of the base and the mean

DAMS AND STORAGE RESERVOIRS

FIG: I

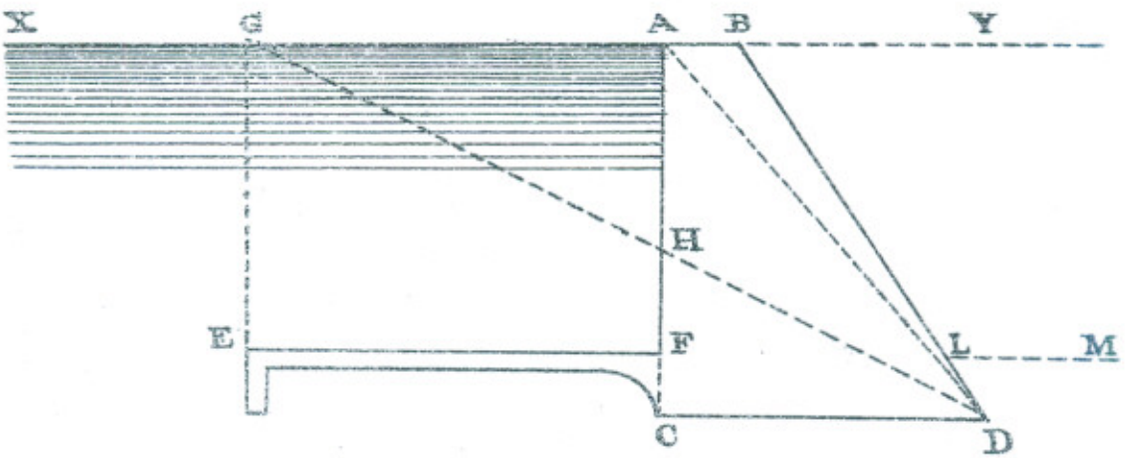


FIG: II

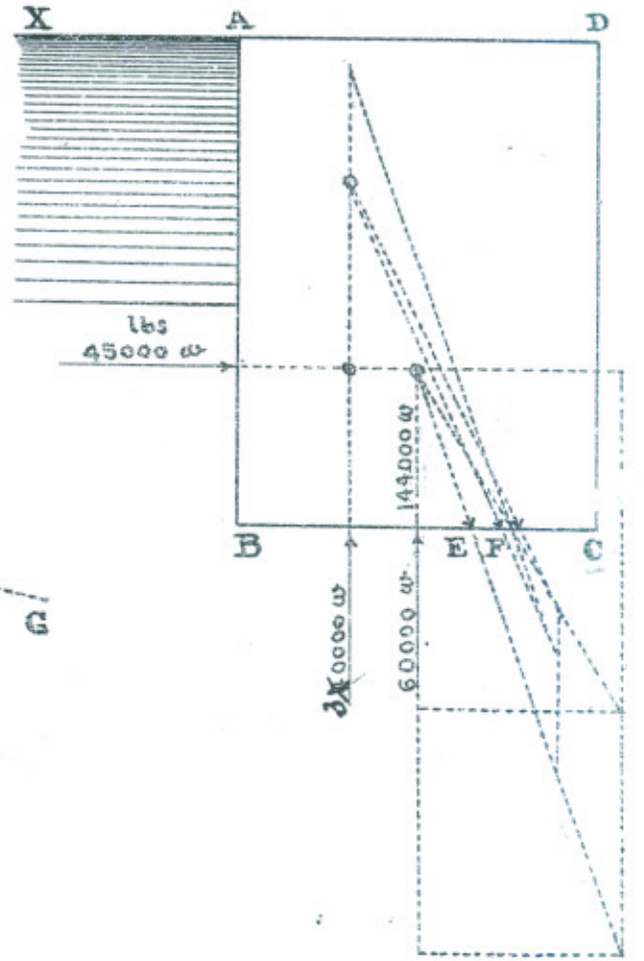


FIG: III

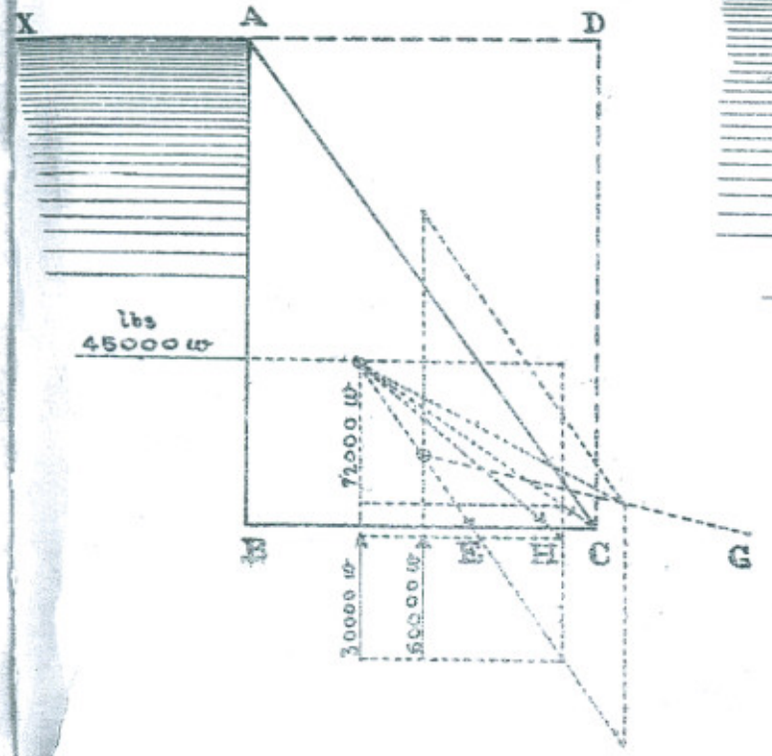


FIG IV

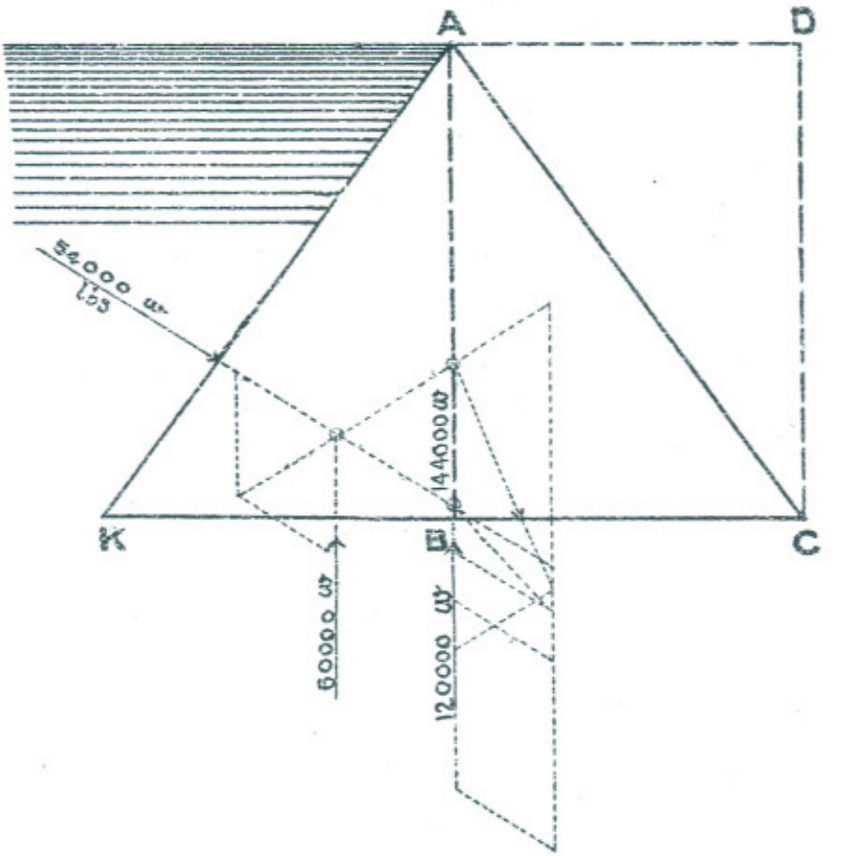
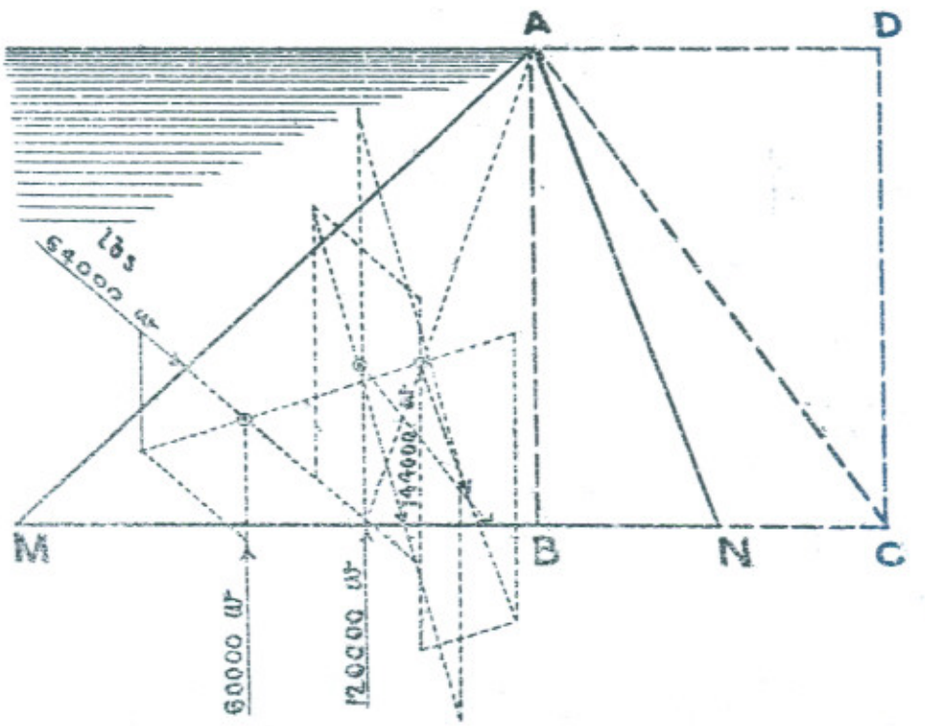

A
D
54000 lbs
144000 lbs
K
B
C
60000 lbs
120000 lbs

FIG V


A
D
lbs
54000 lbs
144000 lbs
M
B
N
C
60000 lbs
120000 lbs

and maximum intensity of pressure will be 12 tons per square foot.

Case VIII.—In Case VII add a buoyant force of 300 feet head acting uniformly under the whole base. The resultant will strike practically the centre of the base, and the mean and maximum pressures will be only 3·6 tons per square foot. There will be great risk of the dam sliding on its base.

Case IX.—In Case VII add a buoyant force of 300 feet head under the up-stream toe K, diminishing gradually to *nil* at the down-stream toe C. The resultant thrust would strike the base 40 feet down-stream of its centre. The mean pressure would be 6·7 tons per square foot and the maximum 10 tons per square foot. There would be no risk of sliding on base.

Case X.—Instead of the rectangle A B C D (Fig. V) let us suppose the dam has a triangular profile, A M N, sloped at 45° up-stream, and with base M N=400 feet, and height, A B=300 feet. Ignore buoyant forces. The resultant will strike the base 25 feet down-stream of its centre. The mean pressure will be 13 tons per square foot and the maximum 16 tons per square foot.

Case XI.—In Case X add a buoyant force of 300 feet head acting under base uniformly throughout. The resultant thrust will strike the base about the border of the middle third. There will be great risk of the dam sliding. The mean pressure will be only 4·74 tons per square foot and the maximum 9·5 tons per square foot.

Case XII.—In Case X add a buoyant force of 300 feet head acting under base at point M and diminishing to *nil* at point N. The resultant thrust will strike the base 70 feet down-stream of its centre. The mean pressure will be 7·8 tons per square foot and the maximum 15·5 tons per square foot, and there will be no risk of sliding on base.

Main Principles of Design.

35. These sketches and rough calculations indicate that where we have a solid practically impermeable rock foundation, a safe and economical profile for a solid masonry dam will approximate to a right-angled triangle with base equal to two-thirds of height and up-stream face vertical.

As a matter of fact we find the latest and best specimens of dams on solid rock that have been built by the Government of the United States of America approximating to this profile.

But with a porous foundation-soil admitting of the possibility of buoyant forces acting under the dam, this profile will not be safe. To resist buoyant forces we need to slope the up-stream face of the dam to a considerable angle with the vertical ; and accordingly, in the design of the Gileppe Dam, where extreme conditions of buoyancy were assumed, we find the up-stream face sloped at one-to-one and the down-stream at one-to-four. Such extreme assumptions as to buoyant forces as were made in the case of the Gileppe Dam necessitate also a great crest thickness for the dam.

Hydraulic Gradients.

36. The practice and experience of Punjab canal engineers will probably lead them to endorse the views expressed in the last part of paragraph 17, as governing the problem of buoyant forces in dam-foundations. ' They have been accustomed to build weirs across the Punjab Rivers with foundations in pure sand, overcoming the forces of buoyancy and " blow-up " by increasing the horizontal path of the sub-soil flow.' The same principle, *mutatis mutandis*, may be applied to cases of foundation-soils of all degrees of porosity and instability, varying from pure, loose sand, to gravel, or boulders, or soft or hard rock.

The firmer and the more impermeable the foundation-soil is, the steeper will be the hydraulic gradient necessary for percolation-flow through it.

37. The example discussed in paragraph 34 indicate that in foundation-soil of soft rock, or gravel or boulders, a masonry dam approximately an equilateral triangle in profile might be stable and efficient. Its cost and weight might be reduced by honeycombing it from the down-stream side by horizontal tunnels at different levels, to a distance of, say three-fourths of the thickness of the dam at each level. A hollow dam of reinforced concrete would involve much the same principles *qua* stability. The hydraulic gradient of percolation-flow through the dam or through its sub-soil must be dealt with as suggested in paragraph 36, and this must be borne in mind also in designing dams of earth or rocks, laid either manually or mechanically or by hydraulic sluicing.

DISCUSSION.

CHAIRMAN.—The next papers for discussion are those by Messrs. Wadley, Blaker and Schönemann. Mr. Wadley has dealt with the formation of land by rivers and torrents in the Panjab, Mr. Blaker with auxiliary reservoirs for irrigation, largely viewed from a geological stand-point, and Mr. Schönemann has mainly considered the historical evolution of the dam. In the absence of Mr. Schönemann, who is on furlough in England, and Mr. Wadley, who has been detained in Peshawar on duty, Mr. Blaker will introduce this group of subjects.

MR. BLAKER in introducing the papers pointed out that they were closely connected in the matter of the rise of spring water level in the Indo-Gangetic alluvium, and in the methods which would probably have to be adopted before long to combat the resulting water-logging on canal-irrigated areas. The method of land formation described in Mr. Wadley's paper was chiefly responsible for the laying down of the alluvium Mr. Blaker described how the alluvium and allied deposits had been successively eroded from the Himalayan mountain areas, and brought down by the rivers and deposited at the foot of the mountains in an enclosed lacustrine basin, the deposits now being many hundreds of feet in depth. He indicated how the weight of the deposits had caused continued subsidence, especially of the northern portion of the deposits, giving rise to the great boundary fault; and how the southern lake barrier had been cut through by the Indus and Ganges with consequent draining of the alluvial surface, below which spring water level (before the present extensive irrigation operations) had reached a state of equilibrium between the then absoption and subsidence factors.

Mr. Blaker then showed the effect of diverting water from the rivers into the canal systems by which drainage water from the Himalayas and plains alike was caused in greater part to be absorbed by the enclosed alluvial basin, the probable extent of outflow thence, and the probable rate of rise in the subsoil to new conditions of equilibrium after the canals of the "Triple Project" had been opened. It was argued that out of every 100 cubic feet of discharge entering the canal systems probably 60 per cent. was absorbed by the soil, out of which 10 per cent. would be a moderate figure for infiltration back into the rivers on existing old canal areas where spring level is high. No appreciable increase of absorption and escape through the

rocky basin floor could be counted on, because of the relatively small increase of pressure head compared with the great heads already existing, and that therefore any further escape from the basin must be by subsoil flow by way of the gaps through the barrier made by the Ganges and Indus, of which, the former only could be counted upon when considering the fill up of the Panjab alluvium ; since, on the Ganges side, irrigation there is filling up that part of the alluvium. It was estimated that within two or three years some 40,000 cubic feet of water per second average throughout the year would be passed on to the Panjab alluvium, and out of this at least 30 per cent. would, under existing conditions, be retained within the basin. This would be sufficient to cause spring water level to rise within the alluvium at a rate of nearly 6 inches per annum *over its whole area*, and this did not take into account the more rapid rise beneath the irrigated areas, while Mr. Blaker believed the rise to be between one and two feet per annum during the earlier stages of irrigation on a given canal area. The rise on the irrigated areas continued until a state of final equilibrium was reached. This was fixed by the drainages and general slope of the country, which limited further rise by cutting off that portion of the subsoil supply, which moves on the surface of the cone * towards the drainages and the deserts, the drainages acting in fact as catch-water drains. For a knowledge of the amount of absorption from canal channels irrigation engineers are at present dependent on the experiments of Kennedy and Ivens on the upper Bari Doab and Ganges Canals respectively, and, which though made independently agree remarkably well. They show that the loss per cent. of the total canal head discharge is distributed as follows :—

Loss from main channels about	...	16	per. cent.
Do. distribution channels	...	6	,,
Do. watercourses	...	22	,,

i.e., altogether between 40 and 50 per cent., so that if all the main channels were lined with an efficiency of 75 per cent., it would save but 12 per cent., *i.e* , the complete lining of all main channels would only reduce absorption from about 44 per cent. to 32 per cent., of the present canal head discharges. There were practical difficulties in the way of lining village watercourses, and to some extent of

* The term cone " is used because subsoil flow from a canal system is synonymous with the conic or deltaic method of land formation explained by Mr. Wadley in his paper."

distributaries, especially on new canals (where the rate of absorption was greatest) and its feasibility could hardly be reckoned on. The lining of main channels would moreover take many years to complete, and in the interim the saturation cones would continue to rise to final equilibrium, move forward, and more and more areas would be subjected to the evils attending high spring water levels. This was the dilemma in which pumping from the subsoil could be of assistance as an immediate remedy.

It seemed to be quite safe to forecast that if supplies for irrigation on such areas were pumped from the subsoil and substituted for direct canal irrigation, that in the case of large tracts the subsoil water level could be lowered from 10 feet and less below the surface, which might be considered "dangerous to well below that figure (15 feet might be considered safe), simply because the gross absorption on and in the vicinity of the area would be materially reduced on account of the—

(*a*) elimination of distributary absorption (distributaries on such areas would no longer be needed) ;

(*b*) elimination of about half the watercourse losses, since they would be smaller, shorter, and would run more continuously giving higher "duties" and less opening and closing of channels.

In this way the lining of distributaries would be dispensed with and half the difficult problem of lining watercourses, which is so high a factor in the total absorption, would be solved.

MR. PURVES said he had only a brief time at his disposal to peruse these very interesting papers. Irrigation Engineers had, for many years, been giving the subject their best thoughts, and the speaker did not feel equal to dealing with either Mr. Wadley's or Mr Schönemann's paper, but would like to offer a few remarks on Mr. Blaker's paper on Auxiliary Reservoirs, and trusted his boldness in speaking with so little preparation would be pardoned. In advocating the cause of reservoirs Mr. Blaker had at page 24, said :— "The careful development of a system of pumping from those tubes on a water-logged area, so controlled as to lower the spring level sufficiently for sanitary purposes, without, at the same time, unduly increasing the lift and consequently the power used, so as to impair their commercial value, appears to be the true solution of the water-logging problem". Mr. Blaker

had also mentioned that about 80 per cent. of the water entering a canal went to no useful purpose, only 20 per cent. being usefully employed in maturing crops, and these figures were generally accepted as nearly correct. Pumping the water to be used locally for irrigation would be practically returning to the soil what was taken from it. Possibly 80 per cent. of the water so pumped would return to the soil, and the result would be that water-logging would be deferred for a time, but eventually would come on all the same. It seemed therefore, that the true solution of the water-logging problem would be found in lining the channels and so preventing the water from getting into the soil. Experiments had been carried on in this direction for years and were now fairly on the way to solve the problem in a satisfactory manner. If the seepage into the soil could be prevented it should be possible to save 50 or 60 per cent. of present losses and pass it down the rivers to be utilized elsewhere. On the prevention of seepage money could be gradually expended, and the results watched, and by proceeding cautiously necessary alterations could be made to ensure the best results. No large sums would be staked at once as in the case of auxiliary reservoirs. The special nature of the formations in which the reservoir dams must, of necessity, be located would cause most engineers to shrink from running the risk of construction, and the speaker was doubtful whether reservoir building was the correct solution for the prevention of water-logging. It was also, at present, doubtful whether the water pumped from saturated soil was suitable for crop maturing. The speaker understood Mr. Blaker to say that in process of time the water-logging would tend to make the whole Indo-Gangetic depression to all intents and purposes a lake, but it would seem that the more the water was spread out over the surface to be evaporated, either from the surface direct, or through the crops matured, the less would pass into the soil. An engineer once drew attention to this point in a local paper, and expressed an opinion that the climatic conditions of the Panjab were changing from this cause. It was difficult to estimate the quantity evaporated, but it must be considerable. It would, therefore, be desirable to spread out the water saved from seepage over as large an area as possible. There were one or two points with regard to Mr. Blaker's design of a dam to which Mr. Purves invited attention. If the culverts at the bottom of the dam were to be used for discharging water, when the reservoir was full, the lining material would not stand the high velocities of

60 or 70 feet per second that must occur. The design moreover did not show the arrangements for passing water for irrigation during the Rabi crop. Again, the roll-away arrangement of passing surplus supplies, with a crest depth of 30 feet, would soon tend to wear down the surface material. The velocities developed would be very high, and there would always be a certain amount of matter in suspension to erode the surface during winter floods. Thin films of water would be less harmful, but such great depths as contemplated appeared to suggest early and repeated renewals of the roll-away surface material.

There also appeared to be considerable difficulties in the way of maintaining, at a certain fixed level, the water ballast in the cavity of the dam, and Mr. Purves would like to know how Mr. Blaker proposed to do this. The speaker regretted he had not, for want of time, been able to consider more fully and do justice to the design of so important a structure.

MR. CARNE pointed out with reference to the proposal for reinforcing the Sirhind Canal from the Beas, that the only tracts that apparently could not be readily served by the Sutlej Valley Project, and which must therefore be provided for by water from the Sirhind Canal, were the Dabwali Tract requiring only 150 cusecs, and Tract A9 in the Sirsa Tehsil for which a supply of 900 cusecs was required. At present the canal could not pass quite enough to satisfy all requirements at the end of the Kharif and beginning of Rabi, as the branches collectively had a greater capacity than the main line, consequently reinforcing the supply by a reservoir did not appear to be of any very practical use. It would require an increase in the capacity of the canal at several works, and the superpassages (particularly the Siswan) did not appear to lend themselves to enlargement. Extension of irrigation on the Sirhind must seemingly be provided for by economizing water, that is by preventing losses by percolation, a matter which was receiving attention. With reference to the geology of the Tarrain, those interested might further be referred to "The Summary of the Geology of India," by Ernest Verdenburg of the Geological Survey of India (2nd edition. Thacker Spink & Co.) wherein the pre-existence of a Mediterranean Sea between the Himalayas and Peninsular India was mentioned (*vide* pages 2, 4, 5, 46, 102, 103, 106, 107). With reference to the closing of the proposed Neila dam sluices in September, Mr. Carne asked the author whether

impounding would not have to begin early in the monsoon season, to ensure catching floods in sufficient number for filling the reservoirs. Regarding the manufacture of Portland cement, the speaker thought that some detail of what was being done would be of special interest to officers on the Sirhind Canal where, as noted above, the saving of water seemed to be essential, and the question of suitable linings was being taken up. Referring to Mr. Blaker's design for a dam, Mr. Carne pointed out that no mention appeared to have been made of the design for the sluice gates of which two sets apparently were proposed ; the use to which even a suitable design of dam, and one over the top as well as through the perforations in the base of which floods could be passed, might be put, would much depend on the sluice gates. Presumably it was considered that the experience gained on the Stoney sluice gates of the Nile Barrage was sufficient to remove from the author's, and perhaps from the minds of others, any apprehension as to such gates being suitable under the conditions to be met with in the case of a masonry dam of this height, located at such a site as Thain on the Ravi, Neila on the Sutlej, or Kooch on the Jumna. There was an interesting paper in the last volume of the Proceedings of the Institution of Civil Engineers on the use of Hydro-electric power for railway work to which the speaker would like to direct attention.

After reading Mr. Blaker's paper, Mr. Carne had written to Mr. Ashford suggesting that the latter should give the Congress some information, etc., as to the *actual*, or if this was not available, the *probable* effect that pumping from tubes would have on the level of the subsoil water, that is to say whether it was expected that tubes, going far into the soil, and only having a strainer at the lower end, would, by reason of the withdrawal of water from a low level, induce a reduction in level of the subsoil water. So far as the speaker knew, in Amritsar there was no regular impervious stratum of any extent of continuity separating the subsoil water caused by irrigation and the percolation from irrigation channels from the deep-seated water-bearing strata fed by natural sources. In the case of a separating impervious stratum, it could be imagined that pumping from the water-bearing soil below that stratum would not affect the level of the subsoil water unless or until a communication were provided through the impervious stratum. Mr. Carne from his own experience on the main line of the Lower Chenab Canal was led to conclude that

the intense water-logging there, was due to percolation from the canal being held up by a thick impervious stratum of clay close to the natural surface, and that it was not due to an actual rise of the natural spring level of the country ; and it was quite conceivable that if long tubes were used under such circumstances that the subsoil water level would not be reduced to any appreciable extent by pumping water from the original water-bearing strata. The speaker understood that Mr. Ashford had been unable to get a sufficient supply by pumping from subsoil charged with water due to irrigation causes, and had to sink tubes so as to tap the lower original water-bearing strata in order to get a sufficient supply.

MR. LYSTER referred to the proposals for the production of Hydro-electric power, where Mr. Blaker had stated that "Power generated at Thain dam, 95 miles from Amritsar, would be available throughout the year." Mr. Lyster understood that all the suitable sites for reservoirs lay on the same geological formation and that on this formation considerable silting must be expected in the reservoirs. In order to deal with this silting it was proposed to allow the reservoirs to remain empty during part of the year. The author says, " actually the reservoirs would be emptied, and scouring begun, by May 1st at latest, and would continue scouring until September 1st. The speaker would like to know what provision would be made for running the Hydro-electric machinery during the four months that the reservoir was empty, and whether such provision was included in the comparative estimates of cost given in Appendix III.

MR. ASHFORD said Mr. Blaker was to be congratulated in having presented a very interesting and important paper to the Congress. Part of his arguments had been based on the success which had attended the speaker's efforts to find a satisfactory method of drawing large volumes of water by pumping from the subsoil. Mr. Carne had also asked some questions on the effect of pumping on the subsoil water. Thinking it would interest the members of the Congress Mr. Ashford had brought to the meeting a length of a tube well strainer pipe similar to that with which discharges up to 2·3 cusecs had been obtained. Experiments were started in 1910, to produce a tube that would exclude sand while admitting water, and which would not be choked by the presence of sand. Without going into details of the successive

steps taken, the speaker would simply point to the sample tube as the result of his labour. It consisted of a foundation tube upon which wire was so bound, that a narrow spiral slit was left between the successive convolution. The wire itself was shaped in such a way that sand could not lodge between the wires, and in actual use it had proved to be free from any tendency to choke. The first tube well was $4\frac{3}{4}$" diameter with 30 feet of strainer coupled to a suction length of plain tube approximately 30 feet long. This tube gave a discharge of about one-third of a cusec, and when pumping had been continued for a week the water table assumed a stable curve in which the maximum depression measured down a tube only about a foot from the tube well was 3 feet, and this tailed off to the normal level at a distance of about 150 feet. The success of this early tube resulted in a decision to experiment with a larger one to get a delivery of at least one cusec. A 10 inch tube, with a strainer portion 95 feet long coupled to a plain suction length at the top 32 feet long was therefore sunk, and many experiments and observations made on this tube well from which a maximum discharge of 2·3 cusecs was obtained. One experiment with a steady discharge just under 2 cusecs (1·93 to be exact) was continued day and night for a week when, as before, the water table was found to have assumed a stable curve. These tube wells when sunk penetrated alternating layers of sand and clay, three clay layers were passed through, and the tube rested at the bottom upon a fourth bed of clay. There was reason to believe that these clay beds were not continuous, but they no doubt extended far enough to ensure that the water flowing towards the tubes moved in a nearly horizontal direction. The curve of the water table gave a key from which a fair approximation of the frictional resistance to the flow of water through the sand could be formed. The quantity of water flowing was known from the discharge, and the area of the sand at different radii, through which the water had to flow, was also known. The curve gave the hydraulic head to force the water through the sand, and thus the frictional resistance could be determined. It was obvious from such curves, that, whereas with a short and small tube giving only one-third of a cusec discharge with a 3 feet depression in the water table, and a much longer and larger tube giving but 6 feet depression with six times the discharge, tubes could be so arranged as to give a maximum or a minimum effect on the water table. By using a number of short tubes a greater effect would be produced on the water surface than if a few long tubes were used to get the same

discharge. Mr. Blaker was looking far forward when he wrote his paper. There would be much resistance to the construction of dams and to the installation of power plants together with the use of electric power so developed for watering the land. The development of power by oil engines and by the use of coal fuel would steadily become more costly. The progress of the world depended upon the production of power for mechanical purposes, and therefore the time was certain to come when the great possibilities for power production, which exist on the Punjab rivers and canals, must be developed, and this could only be successfully done by the building of such dams as those referred to in both Mr Schonemann's and Mr. Blaker's papers.

MR. BENNETT regretted that he had not had time to read the three papers under discussion, but agreed with Mr. Purves that, in regard to water-logging, the best thing to do was to prevent the water getting into the soil. Water proofing the beds of canals was a question that must be taken up in real earnest. The cost would undoubtedly be very great, but it must be faced. The outlay would however in the course of time be recouped by the revenue derived from the water saved. The speaker would not discontinue experiments in pumping, and Mr. Ashford's experiments had proved to be most useful. Outlay, at least heavy outlay, in this direction would be less profitable than in water-proofing the canals. Regarding auxiliary reservoirs, the speaker was of opinion that expenditure on them should be postponed until more had been done towards water-proofing canal beds.

CAPTAIN ANDERSON said that on page 37 the author had referred to the rate of contraction and expansion of steel and concrete. In view of the extraordinarily successful results of the fire tests to which reinforced concrete had been subjected by the British Fire Prevention Committee, the speaker was inclined to think that no fear need be entertained on this account, provided the covering of concrete over the steel was sufficient. After all, concrete was a bad conductor of heat, and no heat could reach the steel embedded in the concrete otherwise than through the latter. As regards the question of railway electrification, it was very doubtful whether the density of traffic on any part of the North-Western Railway would justify it. Taking as a comparison the recent highly successful electrification of the Liverpool-Southport line of the Lancashire and Yorkshire Railway the cost of electrification

of the North-Western Railway would probably not result in any gain. On the portion of Lancashire and Yorkshire line referred to, the train service varied from one at two-minute intervals on exceptional days such as that of the Grand National, to a normal one of ten minutes intervals, and each train carried fully as many passengers as an average train on the North-Western Railway. This was a density of traffic to which the North-Western Railway would take ages to attain if it ever did, while goods trains on the Lancashire and Yorkshire were still worked by steam locomotives. Another difficulty would be the danger of interruption of the supply of current due to damage of the transmission lines by landslips, floods, or earthquakes. Electrification, without some very definite guarantee against this, as well as against the interruptions at the generating station alluded to by a previous speaker, would be impossible. A scheme for driving the Rawalpindi workshops by current supplied from the Baramulla power station in Kashmir, was, the speaker understood, abandoned largely owing to the impossibility of a satisfactory guarantee of the continuity of the supply.

SIRDAR LEHNA SINGH quoted a mixture of clay and *bhoosa* as having been most successfully used to prevent percolation on some watercourses, and also spoke of the importance of reservoirs both towards supplementing the cold weather water supply and providing power. The speaker also advocated the use of reinforced concretes for reservoir dams and suggested that temperature troubles might be overcome by an arrangement for spraying the dams.

Mr. BLAKER, in replying to the various points raised in the discussion, pointed out in reply to Mr. Purves' remarks regarding the design of the dam illustrated in the author's paper, that the preliminary scheme for the Thain reservoir works had been so worked out that it would be unnecessary to discharge through the culverts in the dam, or through the tunnels, at higher velocities than about twenty feet per second. The culverts were only intended to pass scouring floods when the reservoir was empty, and the waterways had been designed of sufficient dimensions to pass maximum floods at this velocity. The passage of Rabi supplies through the dam would be carried out by means of culverts arranged in the flanking bays at different elevations and having balancing cisterns with extra sluices behind, so that water could never flow

through them under greater heads than twenty five feet. The water could easily be cascaded down to river bed level behind the dam. The design of the roll-way was based on the adoption of surfaces formed to the curve of inverted circular arcs of sufficient radius and extent to convert the pressure on the masonry into centrifugal pressure sufficiently low to fall well within safe limits. The kinetic energy of the descending mass would thus be delivered sufficiently into the horizontal direction to reduce the vertical component at departure from the roll-way to well within safe limits, the whole energy of the descending mass of water being finally absorbed by a sufficiently deep and large secondary tank immediately downstream of the dam. The foregoing points were three of the most important ones connected with the design of dams in these situations, on the consideration of which great attention had been bestowed, the form of dam put forward being that which seemed to solve these and other problems most effectually. The system of roll-way adopted had been found in America to be very effective in letting down great masses of water through great heights with but negligible wear and tear on the masonry surfaces. Regarding the maintaining at a fixed level of the water in the water ballast cistern, Mr. Blaker said that it must be understood, that when filling the reservoir was begun, the river discharge would be diverted through the tunnels up-stream of the dam. These tunnels passing through the neck of the loop, the dam being situated at the apex. The dam would thus be isolated, and the sluices could be lowered when dry and tamped at leisure. When the tunnels were closed water would rise over the bar used for diverting water through them, and would pass down to the dam where it would be admitted to the ballast cisterns, when and in what quantities required, by special sluices and culverts taking off from the flanks, and having access to the cisterns along the length of the dam The water level would be raised by this method and could be lowered by the downstream sluices shown in the illustration, when the downstream secondary tank was full, and by separate special flank sluices having connection with the filling culverts when the tank was empty, so as to avoid excessive head of outflow. The proper description of the design of dam was subject matter for a separate paper and the present paper did not pretend to deal with it. In reply to Mr. Carne, Mr. Blaker said that the reservoir supply would only be used in the Rabi and possibly the end of the Kharif, when he believed the available river supply was less

than the Sirhind Canal capacity, and there should be plenty of room for an additional supply then. The extra river supply for extensions in the Kharif would probably only involve a rise of full supply level of a few inches ; the cost of altering the Siswan passage and other works would not be so great as to preclude the passage of the additional supply. That a geological sea existed over the area now occupied by the Indo-Gangetic alluvium and by a large part of the Himalayan area was an established geological fact. The sea is known to have existed in varying extent right through geological times until immediately before the elevation of the Himalayan Mountains. It at one time connected Europe with an Australian Sea.

Mr. Carne's apprehensions regarding the working of the dam sluices against high heads had ,already been disposed of by the author's reply to Mr. Purves's remarks. The design of dam and of the allied works obviated the necessity of working the sluices against pressure heads altogether. In reply to Mr. Lyster, the author pointed out that the hydro-electric power station at Thain could be placed at the downstream end of the tunnels through the neck of the loop, resulting in a fall of 40 feet becoming the minimum head available for power generation by ordinary river discharge when the reservoir was empty. Replying to Messrs. Purves and Bennett's remarks advocating lining of canal channels in preference to pumping, the author emphasized the necessity of finding an immediate remedy for the evils attending high spring water levels on the worst areas. His introductory remarks showed that only about 12 per cent. out of 44 per cent. of the canal head discharge which was absorbed, was likely to be saved by main channel lining, against 17 per cent. entirely eliminated by pumping, plus a large proportion of the subsoil water pumped, all of which would be spread over the ground surface, part would be absorbed by vegetation, part evaporated, and the remainder again absorbed, so that an aggregate equivalent of at least 27 per cent. the permanent absorption could be expected on a given area. The object of the author was not to advocate the substitution of pumping for main channel lining, nor of its wholesale application, but it was rather to focus attention on it or a remedy capable of immediate and expanding application on areas requiring immediate relief. It could be used in conjunction with channel lining, should an efficient form be found. The lining of channels adjacent to such areas could be carried on in addition to pumping as opportunity and occasion required.

PLATE . A .

NEW LOCO SHOPS
LAHORE
ELEVATION OF EAST WALL
SHOWING REINFORCED CONCRETE COLUMNS

SCALE 20 FT = 1 INCH

