ACKNOWLEDGEMENTS

In conclusion the Authors gladly acknowledge the work done by, among many others, the Board of Engineers; Messrs D. A. North, J. G. Gräbe, and R. T. Shaw of the Cementation Company (Africa) (Pty), Ltd; Messrs C. Grundy and P. I. Parker of the parent Company; and Messrs D. C. MacKellar, J. G. Welsh, B. D. Kark, and D. Lipsett of the City Engineer's Department, all of whom participated in various stages and degrees in the execution of the project.

REFERENCES

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3. L. A. Mackenzie, "Flood Peak Probabilities in Certain Rivers of the Union of South Africa." Professional Paper No. 17. Irrigation Dept, Union of S.A., 1951.
4. K. B. Keener, "Uplift Pressures in Concrete Dams." Proc. Amer. Soc. Civ. Engrs, June 1950. Separate No. 25.
5. "Design Criteria for Concrete Gravity and Arch Dams." U.S. Bureau of Reclamation, Nov. 1950.

The Paper, which was received on the 8th June, 1955, is accompanied by twenty-six photographs and five sheets of drawings, from some of which the half-tone page plates, folding Plates 1 and 2, and the Figures in the text have been prepared.

CORRESPONDENCE on the foregoing Paper should be forwarded to reach the Institution before the 15th January, 1956. Contributions should not exceed about 1,200 words.—SEC.

Discussion

Mr Morris introduced the Paper with the aid of a film showing the operations described in the Paper.

The Chairman said that when Mr Lloyd Davies had described the dam (reference 1, above) there had been criticism of the estimate of flood discharge, but Mr Morris had provided for 17,000 cusecs, which was about $2\frac{1}{2}$ times the normal maximum flood according to the Floods Committee Report. Quite rightly, he was on the side of safety.

Those who might have to contemplate similar work, or even the construction of a dam from the very beginning on the same principle, would find the Paper extremely useful. The Chairman said that his firm were concerned with a project for heightening an important dam in the Midlands by about 45 ft, about 25% of the present height above the foundation. In the case of Steenbras, the heightening was 6 to 7%, which was much more modest.

It was hoped in the Midlands to have a trial run with a single borehole right through the dam and 120 to 130 ft into the underlying rock, to determine the rock conditions, which were something like those described by the Authors. Then they wanted to satisfy themselves about the arrangements for stressing. The Author had given much food for thought

FIG. 10.—CABLE BRUSH ENTERING A HOLE

FIG. 11.—VIBRATING A CABLE AFTER HOMING IN

FIG. 12.—CABLE BEING TENSIONED; COMPLETED CABLE IN BACKGROUND

FIG. 13.—PLACING "COLGROUT" ON LEFT FLANK. COARSE AGGREGATE IN PLACE

PLATE I
STEENBRAS DAM:
RAISING AND STRENGTHENING

THE RAISING AND STRENGTHENING OF THE STEENBRAS DAM

FIG. 3.—PROGRAMME OF WORKS

and his information together with the experience gained in the Midlands would be very useful. If the trial borehole and cable were successful, there would be 200 such boreholes and cables. The job was on a much larger scale than that described by the Authors.

The method described in the Paper had been adopted on the Continent and in North Africa, and a dam had also been designed from the start on the basis of pre-stressing.

Mr R. C. S. Walters (Partner in the firm of Herbert Lapworth Partners, Consulting Engineers) agreed that the Authors had made flood provision $2\frac{1}{2}$ times that envisaged by the Floods Committee. The Floods Committee had in mind a normal flood, which had actually been recorded, and possibly the Author intended his flood to be a catastrophic flood, twice that envisaged by the Institution Committee. That being so, it struck him as interesting that the floods in South Africa were comparable with those in the United Kingdom. Floods in America for similar upland areas might also be similar to the Institution Floods Committee's standards.

He was particularly interested in the provocative remarks in the concluding paragraph of the Paper dealing with the incorporation of siphons or prestressing in a new dam. There was a difference between adding something to an existing dam and incorporating an extra height of dam in a new design; when impounding the equivalent or balancing about 70% of the average run-off, it might not normally be economical to go any further. But if about 1,500 million gallons could be added to the storage for an expenditure of £100,000 on an existing dam, it seemed extremely good going. He had recently been associated with a problem of putting a siphon on an existing dam to cope with about 4,000 cusecs for an expenditure of £20,000, to increase the storage by about 200 million gallons. It was well worth while.

But it had to be ensured that the siphons were *safe* and that the prestressing was *permanent*. The Authors had suggested that they had rejected the idea of siphons because of the difficulties. Mr Walters had visited France to see a comparable siphon (for the same quantity—4,000 cusecs) in operation in a hydro-electric station where the turbines could be shut off for testing the siphon visually.

According to pp. 30 and 31, the Authors were nervous of the siphon because of wave action. Mr Walters had avoided that by making a series of sixteen siphons at different levels. The criterion was that the catastrophic flood level in the area could not be increased. The priming and depriming difficulties referred by the Authors could be overcome where necessary.

Les Cheurfas dam in Algeria had been raised in 1935 on the Coyne system. The total tension in the cables was 1,000 tons. In 3 years that had deteriorated by 40 tons, in the next three years by 4 tons and in the next 3 years by 1 ton. Apparently at Steenbras there was now no means of measuring the tension in the cables, and it may be that Mr Coyne did not deem it necessary to measure it and to go on taking the tension stresses, especially since Les Cheurfas dam, still standing after 20 years, had proved to be perfectly safe. Presumably the cables at Les Cheurfas had been put in holes and grouted up. Had the Authors any means of observing any loss of tension in the cables?

Another Coyne variation Mr Walters had seen* in France was at the Castillon dam. There had been trouble with the abutments and the rock had been reinforced with pre-stressed cables let into the rock. It seemed that prestressing was a safe process and although it was not popular with some Canadian engineers, the French were apparently quite confident about it.

Mr C. M. Roberts (Partner in the firm of Sir William Halcrow & Partners, Consulting Engineers) said that the narrow crest of the dam was one of the chief drawbacks of the method when applied to raising and, in addition to making the work difficult to execute, set a practical limit to the amount of raising. The Chairman had spoken of a 45-ft heightening elsewhere and the Authors had stated that investigations had been made

* Thanks to the kindness of M. A. Decelle, Electricité de France, Marseilles.

into raising the Steenbras dam between 3 and 30 ft. Were the heights approaching 30 ft to be accomplished by the same method as that described in the Paper? Possibly some of the investigations were on the basis of extending the cables up through the raised portion of the dam.

The arched part of the dam was not prestressed; was the arching effect alone the criterion there, or had the Author taken into account arching and gravity in the original portion of the dam and arching and cantilever for the raised portion?

The Authors had referred to the low pH-value of the water, which was surely very aggressive. Apparently a piece of exposed steel did not show signs of deterioration, but it would be interesting to hear a little about the effects of the water on the concrete itself. Classes of concrete were mentioned in the diagrams; but what were the cement content and the nature of the cement in the concrete between the water face and the cables? Was there any protective treatment of the water face?

Mullardoch dam in Scotland had been partly raised by 20 ft in 1951, as described in a recent Paper^a presented to the Institution. The method used for strengthening had been to thicken the original section. If the saving by using cables was as great as the Authors had said on p. 47, it might be asked why the method had not been applied at Mullardoch. The reason was very simple: only some of the blocks of the Mullardoch dam had had to be raised during construction of the remainder. Various blocks surrounding those being raised were being built to the full required height and the foundations had been prepared for the final height of the dam throughout. The fact that certain blocks had been finished to a lower section was purely a matter of temporary economy, but financial relief had materialized before the completion of construction and it had been possible to raise the comparatively few blocks which had been built to the lower section. Ultimately those lower blocks were required to conform to the same profile as the full-sized ones to meet spillway requirements and there had been no question of adopting a thinner section in the case of the Mullardoch dam.

Mr A. J. Harris (Consulting Engineer) said that the method which the Authors had used with such signal success for this dam had first been developed by Coyne about 20 years previously, and although the techniques of prestressing had developed considerably since then, he thought it true to say that Coyne's cable had not developed very much. There might be a very good reason for that; it worked very well and in all likelihood the cost of the cables was not very great in comparison with the cost of the remaining works.

Nevertheless, certain features of the cable invited comment. The stressing cables were of laid wires and passed over quite a sharp curve over the head of the jacking device, which did not seem very judicious, since the state of stress, which was high to start with, would be considerably complicated by the additional curvature.

Again, one of the advantages of using prestressing as distinct from reinforcing, in a dam such as the Steenbras, was that it was possible to ensure a very high degree of protection of the cable. Had the Authors considered impregnating the cables with bitumen prior to dropping them down the holes, which would provide a very high degree of protection? The creation of bond was of no value, because if the dam ever reached the point where it cracked, the operation had failed and variations in the reserve of strength were a matter of indifference. Bitumen coating would permit the engineers to keep their cables under observation over a longer period; a cable without any protection at all had a very high rate of corrosion, but the use of bitumen would enable the engineers to keep it open, possibly indefinitely.

The Authors had given very little information about concrete strength in either the old or the new dam, and it would also have been interesting to see the distribution of stresses. Stabilizing by prestressing of the type described usually provided a very much more favourable distribution of stresses and frequently a smaller maximum stress, and it would be interesting to have some information on that.

^a C. M. Roberts, "Special Features of the Affric Hydro-Electric Scheme (Scotland)." Proc. Instn Civ. Engrs, vol. 2, p. 520 (Sept. 1953).

What technique had been used for blocking-up the jacking heads when the cable had been extended?

Most of his comments seemed to have been "Why did not the Authors do this?" It was sufficient to say, however, that the responsibility taken by an engineer in making a dam of such a magnitude with a technique which, while not novel, was bold, was very considerable, and the Authors had every reason to be proud of the result.

Mr Jack Widdowson (of Civil Engineering Department, The Cementation Company Ltd, Contractors) observed that it had apparently been the Authors' original intention to do most of the drilling by percussion methods but that intention had to be abandoned and quite a proportion of the drilling was carried out by diamond methods. Even with both methods of drilling they had had considerable difficulty in maintaining at all times the necessary degree of straightness and verticality tolerances.

He compared the drilling operations at Steenbras with a project where the drilling operations were basically the same—a dam on the Coyne principle, involving the principles of strengthening but not of raising. He referred to the Tansa dam, in India, which was one of the most important features of the Bombay Municipality Water Undertaking. At Tansa the dam was approximately 9,000 ft long and the strengthening operations involved the drilling of 2,400 holes along the top of the dam and the spillway. Each of those holes was carrying a 70-ton cable and was of 2½-in. dia. The average depth was 100 ft and the maximum depth was about 180 ft. Drilling, in the core of the dam, was through rubble masonry of Dekkan Trap basalt in lime-cement mortar. The underlying rock below the dam was also Dekkan Trap basalt. On the whole of the project, all the 2½-in.-dia. holes had been drilled by percussion methods, with no diamond drilling whatever. In the early stages of the drilling operations and after the first few holes had been carried out, tests were made for straightness and verticality, and at the first few holes there was every indication of a high rate of straightness and verticality being achieved.

The method of testing was very simple: a small electric battery torch, suspended on a piece of string, was lowered down the hole, and so long as the light could be seen in the 2½-in. hole, to all intents and purposes it was straight. If they could reflect the movement of the piece of string at the top of the hole by the movement of the torch, as indicated by the light moving across the hole relatively the same way, there was an indication of free suspension; and this in turn, the depth of the torch at the time being known, provided the possibility of calculating the extent of the verticality.

Great care was taken, in starting the hole, to get it going truly vertically, and straightness was secured by precisely the same methods as at Steenbras, namely by a follower with a diameter ¼-in. less than that of the bit, at the end of the drill rod.

Overall, it had been found that about 99% of the holes tested were straight, whilst many of the holes in which the lighted torch had been suspended 80–90 ft down had indicated that the torch was still freely suspended at that depth, giving an indication of verticality. In view of the consistently satisfactory results achieved in the early stages, the testing of one hole in 20 was laid down as routine. Almost a quarter of a million feet of 2½-in.-dia. hole had been drilled satisfactorily by percussion methods.

It might therefore be logically stated that although there had been difficulties in drilling by percussive methods at Steenbras, that had not been the case at Tansa, where a very large footage of drilling had been carried out.

Mr Herbert Addison noted that when the authorities had first considered the heightening of the dam, they had found that trunk mains and filtration plant were "fortunately" already in existence, which could take the full augmented supply of water. Was that due to good fortune or to good management?

He had had the privilege soon after the war to be shown some of the Cape Town undertakings and he had seen signs of great resourcefulness. As the Author had said, there was indeed a problem by 1950, but in the meantime measures had been taken, if not to augment the total amount of water collected in the catchment, at least to augment the total amount delivered under peak conditions into Cape Town. That was particularly

important, because at that time—in the early stages of the war—convoys to the East and Far East had been routed round the Cape and they had made heavy demands on water at Cape Town. An emergency solution had been found by installing a booster plant in the existing main from Steenbras to the Cape Town storage reservoir, by which temporary supplies could be put through the pipe at a rate a good deal higher than by gravity alone.

Very soon afterwards, when conditions permitted, a new main had been put in, parallel to the existing main, and the filtration plant had also been installed.

It would further increase the value of the Paper then if the Authors could give a short summary of the various measures taken between the first heightening of the Steenbras dam and the second heightening described in the Paper. Moreover, it would show an interesting parallel with conditions in England in somewhat similar conditions: for example in the Manchester Water Works, where the supply had been increased by successive stages of boosting until finally a new source had been brought into operation; that was the Haweswater source, which was comparable with the new South African reservoir now approaching completion mentioned by the Authors.

Mr C. B. Barlow (Chief Engineer's Department, Metropolitan Water Board) observed that the original dam had a capacity of 600 million gallons and with a maximum height of 62 ft. In 1917, raising the wall 40 ft. had increased the storage to nearly 6,000 million gallons. Another $6\frac{1}{2}$ ft had now been added and it would be interesting to know the new capacity of the reservoir.

When the first dam had been built, as a matter of extreme urgency at the time, it had been positioned higher up the Valley than necessary in order to leave the most favourable position free for a larger dam in the future. Presumably the possible use of such a site had now been abandoned, since reference was made on p. 31 to the possibility of a further major raising of the dam. Exactly how was it proposed that that should be done? Would the width of the wall be increased? What further increase in capacity was possible? Mr Lloyd-Davies, in a Paper⁷ presented to the Institution in 1923, stated that the ultimate capacity of the basin was about 6,000 million gallons, but that had already been exceeded.

Turning to the strength of the dam, Mr Barlow said that the original dam was arched in plan, except for the short end portions, but it had been designed as a gravity section and he understood that an additional factor of safety had been attained thereby in order to guard against abnormal floods. Presumably that had not been achieved when raising it in 1927. In that connexion, it was interesting to note from Fig. 1, Plate 1, that the arched section had not required any post-stressing. He observed that the concentration of the vertical cables inserted in the wall was maximum at the junction of the flank walls with the arched portion of the dam, where the cables terminated abruptly, and it would be interesting to know whether any cracking had occurred in those regions, where, it would appear, considerable shear stress must have been induced.

It was stated on p. 26 that the calculations for the stability of the dam had as their basis an uplift ranging from two-thirds hydrostatic pressure at the upstream face to zero at the downstream face. On p. 33, the figure of two-thirds appeared to have been reduced to 0.6. It was also stated that when the holes for the cables had been drilled, with a few exceptions they had been watertight until 10–30 ft into the rock. It therefore appeared that the uplift was much less than assumed, and it would be interesting to know whether, in the light of that discovery, the dam before the present raising would have been stable under the conditions of the synthetic flood envisaged.

Had any account been taken in the calculations of the compressive stresses due to the thrust from the arched section of the dam being transmitted to the flank walls? In the discussion in 1932⁸ Professor Snape had suggested that the dam could well have been calculated as an arched dam.

⁷ D. E. Lloyd-Davies, "The Works for the Augmentation of the Supply of Water to the City of Cape Town, South Africa." Min. Proc. Instn Civ. Engrs, vol. 216 (1922–23, Part II), p. 338.

⁸ See p. 51 of reference 1.

It was stated on p. 24 that "the storage, supply, and distribution of water in Cape Town are the responsibilities of the City Engineer's Department." Was there still an officer officially known as "Water Engineer"? Cape Town had a population of half-a-million comparable to that of Leeds or the South Essex Water Company, with a daily supply of more than 25 million gallons. Was there any particular reason why Cape Town had not yet fallen into line with the majority of the larger water undertakings, which were looked after by water engineers employed whole-time on water supply? Generally, only smaller undertakings were the responsibility of the Council Surveyor and Engineer, who was also responsible for many other aspects of local government. In that connexion, Mr Delwyn Davies, who was known to most present and who had written at length on the organization and management of water undertakings, had stated in a recent Paper that modern water supply dictated the need for a specialist operational force. No longer, Mr Davies had said, could they expect any engineer to be expert in water, sewerage, highways, and town planning.

Mr A. C. Buck (a Senior Assistant Engineer, Messrs Binnie, Deacon & Gourley, Consulting Engineers) said the Paper brought to mind a regrettable aspect of British civil engineering as it was today and had been for 50 years—the apparent inability to conceive, develop, and exploit new ideas of major importance. He was thinking, for example, of prestressed concrete as developed by French engineers. Nothing of comparable magnitude had been done in the United Kingdom in the past 50 years.

The Paper described another example of French virtuosity. Mr Coyne had not only conceived it; he had worked it out to a practical end and brought it to fruition. His courage and tenacity of purpose were to be admired even more than his originality, for it was a very considerable undertaking to start such a project for the first time. The heightening of the Steenbras dam was also a bold undertaking, but it had been very successful and had achieved its end in a peculiarly fitting manner.

Now that Engineers were becoming accustomed to the use of high stresses in hard-drawn steel wire, a major point which arose was that of corrosion. He thought the $\frac{1}{8}$ -in. cover in the cables was sufficient for steel wire and provided permanent protection; but all the boreholes were not straight. Had the Author any qualms about cables touching the sides of the holes? The concrete and the rock underneath were, of course, grouted, so perhaps it was a point of little significance.

Some failures had occurred when loading the cable because of insufficient depth of grout in the hole. Could not some device have been provided for directly measuring the level of the grout? Apparently the load on the cable had been measured by both jacks and the elongation of the cable. The jacks might have an error of about 5%. Which of the readings had been taken in preference? What tolerances had been considered for the thickness of the packing pieces? When the packing pieces were put on the pedestal irregularities had to be controlled within fairly close limits.

In view of the difficulties experienced with the drilling, what was the Author's present opinion of using larger cables and fewer holes?

What protection, if any, had been given to the cables during transport to the site to prevent corrosion?

Turning to the cable heads, Mr Buck said a full-scale test was always of particular interest. Could the Authors give some details of the reinforcement in the heads and how the heads had failed when tested to destruction?

Mr Coyne's first essay in cable-anchored dams had been 20–25 years ago, and in that case he had assumed that the load from the rock which anchored the end of the cable was in the form of an inverted cone with a half-angle of 45°. When turning to the pertinent paragraph on p. 34, Mr Buck had hoped to see that there had been some development meanwhile and that there had been a different approach, but apparently Mr Coyne had kept to his 45°.

Reference had been made to a dam in Scotland which had been designed from the beginning on the same cable-anchored dam principle. Mr Banks, who had designed that

dam, had performed a very interesting experiment. He had excavated a shaft 18 ft into the rock, placed four jacks in the bottom, concreted the hole, and pumped oil into the jacks, taking readings at intervals until the jacks had burst at a load of 4,400 tons; the rock had given no evidence of failure. That compared with about 480 tons or so on Mr Coyne's theory, so it appeared that in some instances, at all events, Mr Coyne's approach was extremely conservative.

There were other means of approaching the important problem of how deep to take the cable. There were two theoretical solutions in existence, one a two-dimensional solution by Malan and the other a three-dimensional solution by Mindlin. The equations were cumbersome and not easy to use. Rock was not like the theoretical material on which mathematicians based their assumptions; it did not possess the same elastic properties in all directions; it was fissured and often stratified. It had to be remembered that rock was precompressed by its own weight. The horizontal planes were in compression and presumably so were vertical planes. If things were arranged so that the cable load was not increased beyond a point at which the tensile stresses of the horizontal planes did not exceed the compressive stresses superimposed by its weight, the problem might be approached with more confidence. It had been applied to Mr Banks's example, and assuming a conveniently placed horizontal fissure at the centre of the cable it was estimated that a load of approximately 4,000 tons would just begin to open up the fissure. Beyond that it was reasonable to expect the rate of movement of the anchor to increase with the load. If Mr Banks's results were examined, it would be seen that that sort of thing did in fact happen. Mr Banks had plotted the curve of movement of the anchor against load; there were several kinks in it, but the biggest kink, which seemed significant, was a load of about 1,400 tons.

Before Mr Banks's figures could be applied to the more theoretical reasoning, a correction had to be made. In that case the jacks exerted a force downwards and a force upwards and the tensile stress at the anchor was doubled. That meant that Mr Banks's load had to be doubled, obtaining 2,800 tons as the load where fissures began to separate at the anchor. Some sort of rough agreement exists, therefore, with the calculated 4,000 tons. Those figures were for a free surface at the top of the rock and not for a foundation load caused by the presence of a dam.

To investigate the extreme lower limit of the load capacity of an anchor, consideration had been given to the replacement of rock by sand; that might be assumed as equivalent to a very bad case of fissured rock. For forces applied vertically upwards on the anchor there was no spread of the load in sand when it was loose; the anchor would pull straight out. Where the sand was thoroughly compacted, the angle of spread was approximately half of the angle of 45° in rock assumed by Mr. Coyne.

Mr J. M. L. Bogle asked two questions on points which had not been raised in the discussion. The first concerned over-flows. In Fig. 2(a), the Authors had shown a raising which apparently meant a vertical drop on the spillway of about 15 ft. In cases of a major raising, such as the 45 ft which the Chairman had mentioned, would not that introduce problems which would lead to undesirable conditions of impact and surge? Would it not be possible to raise the crest by cantilevering on the upstream side?

The second question concerned the tension when the reservoir was empty. Would any tension be developed on the downstream face and, if so, would not the extent of it be the limiting factor in raising? It seemed that while the dam was full the loading of the cables equalized and helped the general stress distribution. If it were emptied, obviously there would be a reverse effect with tension on the downstream face. What was the result of the Authors' investigation on that point?

Mr J. S. Shipway (an Assistant Civil Engineer, Maunsell, Posford & Pavry, Consulting Engineers) referred to the question of the centre part of the dam functioning as an arch, and to the statement made on p. 34: "This assumption was supported by observations in the inspection gallery, since construction joints which intersect the gallery are

completely closed and watertight in the central part of the dam, whereas those on either flank show appreciable opening and in some cases slight leakage."

Presumably if the central part of the dam was acting as an arch then the thrust would have to be transferred through the flanks and thence to the abutments. In that case the joints in the flanks should also be closed, and not showing "appreciable opening."

The position of the inspection gallery was not shown on the cross-section and might prove illuminating. If it was located towards the top of the structure and on the downstream side, then it might be that the upper parts of the flanks were tending to act as horizontal beams between the centre section and the abutments, causing some tension on the downstream face. That would cause the joints to open.

Alternatively, the thrust from the arch might be resisted by the immediately adjacent sections of the flanks, leaving the remainder free to contract. The rock levels shown on the elevation, Fig. 1, would seem to confirm that.

The Authors' reply to the discussion, together with the correspondence on the foregoing Paper will be printed in a later number of the Proceedings.—SEC.