Discussion

The Author introduced the Paper with the aid of a series of lantern slides.

Mr A. A. Fulton (General Manager, North of Scotland Hydro-Electric Board) said that it would have been gathered from the Paper and from the Author's description that the individual hydro-electric schemes in Scotland were of an interesting variety and on a substantial scale. On the civil engineering side alone, work to the value of £84 million had already been carried out since the war. Associated with that there had been expenditure on plant of £10 million, on transmission of £11 million, and on distribution of £30 million. Almost all of that work had been done and supervised by consulting engineers, so that, although split up into units and spread over several consultants, the field in Scotland for engineers and contractors was something approaching in value the much-publicized High Aswan Dam project.

It was a good thing that details of such work as was embraced by the Shira scheme should be recorded in the Proceedings. Much of the interesting work of a hydro-electric scheme was covered over or otherwise rendered inaccessible by inundation and flooding, and so, unless it was fully written up, some valuable lessons were bound to be lost.

The Shira scheme was not "just another hydro-electric scheme". It had an individuality of its own. It was a two-stage scheme, and, as the Author had explained, that was inevitable. It meant, however, that its operation would require more co-ordination and attention than would have been the case if it had been an orthodox conventional single-stage station. It had been made even more complicated by having pumped storage associated with the upper stage. It was the first real example of that in the United Kingdom. From its nature, its use would probably be quite limited. It was not intended to be anything other than experimental, and it would have its greatest value in a wet year.

The Author had explained that Shira was another good example of the extent to which diversion from adjacent catchments had been practised in schemes where heads were high and water was valuable. The Author had quoted the figure of 21.5 sq. miles of catchment area utilized as compared with only 5.2 sq. miles of natural catchment, which represented a proportion of about 24%. That compared with just over 20% for the Sloy scheme.

Like Sloy, Shira had a relatively low load factor, and in a year of average rainfall it would only have water to run at full load for about 2,000 hours. It would operate in association with the Sloy station, with which it had, as he had explained, many features in common. Its main generating set would be treated more or less as an extra unit in the Sloy station and was, in fact, for all practical purposes almost a duplicate of the Sloy machines.

The dams and the sloping pressure shaft, to which the Paper devoted particular attention, were the highlights of the scheme. Whilst it was always interesting to have new problems to face, Mr Fulton doubted very much if, in solving amenity problems by putting a pipeline underground and in cutting down cement requirements by using a buttress dam, the new problems of making up one's mind how much external pressure one had to allow on the lining of the pressure shaft or where all the carpenters for shuttering the dam were to come from, were not just as troublesome.

The wide variety of the work covered was impressive. There were three varieties of dam, and if that was not enough, there were side-stream intakes, free-flow tunnels, pressure tunnels, pressure shafts, collecting aqueducts (both open and closed), a substantial tail-race canal, and an underground power station. Those had given opportunities for experience and training for the many young engineers who had had the good fortune to work on the scheme.

The works would stand as a permanent monument to the care and diligence which he knew had been employed by the Author and by all those who had been associated with him. They had done a good job.

Mr J. A. Banks (Partner, Messrs Babbie, Shaw and Morton, Consulting Engineers) commented on the merits of the earth dam for the lower reservoir. Puddle-clay cores commonly associated with earth dams, although perhaps less frequently so in recent designs, were an inevitable source of weakness on account of the low shear value of the puddle clay and also the difficulty of achieving adequate compaction of the embankment materials adjacent to the core. Those objections had been overcome in the case of the Glen Shira project by adopting a reinforced concrete core, and the thin flexible section of the core wall ensured that positive advantage could be taken of the passive resistance of the compacted earth work. To provide flexibility without cracking, it had been necessary to put a heavy proportion of steel in the core. The reinforcement amounted to fully 3% by volume as compared with the volume of the concrete. The combination of bars and fabric had been found, by tests carried out on slabs, to give the most favourable arrangement of reinforcement for minimum cracking with maximum permissible deflexion.

The importance of avoiding any plane of weakness was the greater in the case of the Glen Shira project because of the rapid draw-down in water level. In a normal water-supply reservoir the rate of draw-down might rarely exceed a few inches per day, whereas the lower Glen Shira reservoir could be emptied in the course of a few days. Had a puddle-core wall been incorporated in the embankment, then the slopes, particularly upstream, would have had to be much flatter to lengthen the path against shear failure, with a consequent corresponding increase in the volume in the embankment, and that would in any case have been difficult because the site conditions limited the permissible base width of the dam. The concrete core had preserved the structural strength of the dam with economic advantage.

Preliminary designs indicated that an earth dam would be cheaper, showing a saving in cost of about 20% as compared with a concrete gravity dam, but it should be remembered that, in practice, an apparently substantial saving in one section of a project might not be fully realized when the works as a whole were taken into account. The Author had pointed out that, whilst the embankment had been raised at the rate of about 12 ft/month in reasonable weather conditions, progress had dropped to less than 2 ft/month when the weather was adverse. If, as so often happened, the material readily available on a site for the construction of an earth embankment had a very low or negligible frictional value, so that its strength was radically affected by change in the moisture content, then the rate of progress in constructing the embankment might be so slow as to affect progress with the works as a whole. The economic advantage of the earth dam was not materially affected in the case of the Glen Shira project, but it was obvious that the possibilities of delay in construction should be kept in mind in preliminary analyses of what might be the most economic form of dam in a particular case.

With regard to the grouting of the foundations, the quantity of cement required to provide an impervious cut-off on the Glen Shira site was surprisingly, almost astonishingly, small. In the lower dam, where the formation comprised phyllite with occasional bands of igneous rock, the quantity of cement injected amounted to 1.2 lb/sq. ft of curtain area. At the main dam, where there was a complex rock structure, ranging from soft phyllite and quartzite to limestone, the quantity of cement required was even less, averaging 0.67 lb/sq. ft of the grout curtain.

He had compared those figures with other works where the geological conditions were different, and he had found that in one instance, with a cut-off in sandstone, the quantity of cement injected amounted to 57.3 lb/sq. ft, and in another instance, where the formation consisted of volcanic ash with bands of somewhat shattered whinstone, 36.2 lb/sq. ft of cement were required. Those comparative figures indicated the uncertainty of forecasting in advance what provision it was necessary to make by way of cementation to provide a watertight cut-off. Both the extent of drilling and the quantity of cement varied over a wide range. In the two comparative instances given, primary, secondary, and tertiary holes with 5-ft spacing were required, whereas on the Shira site only at a few local points was drilling closer than 10 ft found to be necessary.

Mr J. Guthrie Brown (Partner, Sir Alexander Gibb and Partners, Consulting Engineers) confined his remarks to that part of the Paper dealing with the buttress dam.

As in the case of the Author, his firm had for some time been engaged in the design of several of the major hydro-electric works being carried out in the North of Scotland. As far back as 1944, his firm had had to give consideration to the best form of design for the Errochty dam (part of the Tummel-Garry project) which was to be founded on rock at least as bad as that at Shira. Mr Banks had stated that the amount of grout absorbed in the cut-off curtain at Glen Shira had been 0.67 lb/sq. ft. At Errochty the grout absorbed in the cut-off curtain was 8 lb/sq. ft.—12 times that at Shira.

The foundations for the Errochty dam consisted mainly of heavily shattered mica-schist, quartz-schist and hornblende-schists with veins of soft sandy clay at intervals at one end of the dam. For those reasons, a maximum foundation stress of $7\frac{1}{2}$ tons/sq. ft had been chosen as compared with 10 tons/sq. ft for Shira.

Investigation of the different types of dams possible for such a site had led him to the same conclusion as the Author's, namely, that a buttress dam was the most suitable and economical, connecting with a gravity dam at each end as the height of the dam reduced. The ratios of relative economy at which he had arrived were almost identical with those quoted by the Author, namely, rock fill 1.4, gravity 1.2, and buttress 1.

The Errochty dam, like the Shira dam, was 130 ft high, but it was slightly shorter, necessitating the use of twenty-eight buttresses only compared with thirty-seven at Shira. Both dams, it was interesting to note, were being constructed by the same firm of contractors. Errochty dam, however, was the earlier in design and was more advanced in construction.

One major difference was that the spacing of the Errochty buttresses was 40 ft compared with 50 ft at Shira, and they were diamond-headed buttresses instead of the round-head type. He did not agree with the Author's reasons for adopting the round-head type and was still satisfied with the choice of the diamond-headed type. In the case of two other buttress dams at present under construction, one in Eire and the other in Scotland, both had adopted diamond-headed types of buttresses similar to the Errochty design. Therefore, if he had sinned, he seemed to be in good company.

One point of criticism on the Shira dam was the wedge-shaped closing section, about 5 ft wide, between buttresses. Whilst that might have certain advantages, it had the serious disadvantage of doubling up the number of water seals and possible leakage paths. At Errochty, the buttress heads abutted closely together, providing greater structural isolation for individual buttresses as compared with the Shira type. In the case of Errochty, a restriction of 2 months had been placed on concreting against existing buttresses so as to minimize the risk of too great a contraction opening. That had not held up construction to any appreciable extent, and it had been entirely satisfactory.

After full consideration, it had been decided that no inspection shaft should be provided in the centre of the Errochty buttress heads. The shaft had advantages during the cooling stage and for inspection, but it was considered by some to have objections from the point of view of artificially steepening the pore-pressure gradients. Since both Errochty and Shira dams were designed for a very high uplift, the shaft would not seem justified solely as a means of easing uplift conditions.

The Author had not made any reference to deflexion measurements during the filling of the reservoir, for which fairly elaborate provision had been made at Errochty. It would be interesting to know whether any such arrangements were contemplated.

Mr Guthrie Brown then showed a series of slides illustrating the points he had made regarding the comparisons between the Errochty and Shira dams.

Dr Charles Jaeger (Consulting Engineer, English Electric Co. Ltd, Rugby) said that the Clachan power station of the Glen Shira hydro-electric project was the first underground power station built in Scotland. It was noteworthy that the designers of the project had shown such confidence in that very new design as to adopt it straightaway for a 56,000-h.p. station. Because of the success of that type of construction, it had been

used for the design of other power stations belonging to the North of Scotland Hydro-Electric Board and also for many projects overseas where British consultants and manufacturers were involved.

A project such as that at Glen Shira required great versatility from the designer. Two dams of modern and difficult design, a large surge tank, a steel-lined pressure shaft, an underground power house, and a huge pump were some of the most interesting features of the scheme. Each of those items required special knowledge and great skill, and the design had to be backed by extensive investigations.

One particular example would serve to show the type of information which was required to finalize one detail of the design.

The tail-race tunnel was a free-flow tunnel. It was not desirable for such a tunnel to be suddenly put under pressure unless the tunnel was specially designed for such changes in the type of flow. When the turbine gates were opened and the discharge rapidly increased from no-load conditions to full-load conditions, a transitory wave was created in the canal caused by a sudden change in the rate of discharge. The height of such a wave could be predicted.

When the turbine closed and the relief valve opened simultaneously, the discharge did not vary, but the momentum of the flow suddenly increased widely. What was the effect of that change of momentum? The designers of the Glen Shira project thought that problem should be investigated on a model.

Fig. 14a gave the general arrangement of the power house, the tail-race tunnel, and the tail-race canal. Further details showing the turbine draft tube and the relief valve were shown in Fig. 16, Plate 2.

Fig. 22 showed the general arrangement of the tests carried out in a model canal at Rugby. The pumps A, the pressure equalizer C, and the three-way cock H directed the flow towards the model draft-tube J or towards the relief valve K. L represented the tail-race tunnel and M the tail-race canal.

The canal model had been made to scale and the roughness had been calculated in accordance with the scale.

Fig. 23 showed the main test results. Curve (a) represented the wave in the tail-race tunnel and the tail-race canal when the turbine was opening from 0 to 630 cusecs in 20 sec. Curve (b) represented the case with the turbine closing with the relief valve opening in 2 sec, the flow being 630 cusecs. In that case the discharge was constant, but the momentum was increased. It could be seen that the transitory wave height created in that case was far greater than that created by the ordinary condition when the turbine was opening. Finally, there was the emergency condition when the relief valve suddenly opened in 2 sec. In that instance there was the highest possible wave in the canal. Fig. 23 showed the first level for the tail-race tunnel soffit, and the revised design based on the tests.

The examples which Dr Jaeger had shown were taken from a Paper by Adler⁹ which illustrated the type of detailed research work on which the design as a whole was based.

Mr P. L. Blackstone (Head of Hydro-Electric Department, Messrs Merz and McLellan, Consulting Engineers) said that the single machine at Clachan power station had, as the Author had mentioned, the highest designed output of any on the Board's system, being 40,000 kW. It was not physically the largest machine; the turbine itself was actually the same as the turbines at Loch Sloy, but with a different runner design to suit the slightly larger head. The size of the runner at Clachan (5 ft dia.) was almost the same as that at Sron Mor, which produced 5,000 kW. In other machines on the Board's system there were runners of about 10 ft dia. It was a conventional vertical Francis turbo-generator.

The plant at Sron Mor included an induction generator of 5,000 kW output—one of the

9. G. F. W. Adler, "Model Tests on Clachan Underground Power Station." English Electric Journal, vol. 11, No. 4, June 1950.

largest in the world. The induction generator had certain disadvantages. It caused a fairly large surge on the electrical system when it was switched on, and it had, of course, to be run on an inter-connected system because it had to be supplied with its magnetizing current from synchronous machines. Therefore, it was applicable only to an inter-connected system, but provided that there were not too many in the system, it had some advantages.

The turbine required no governor because there was no close speed control required on an induction generator. It was much simpler, and probably in the long run the maintenance of the machine would be less than with a conventional synchronous machine.

The difference in cost between an induction generator and a synchronous generator for Sron Mor had been closely investigated. It had seemed that there would not be much difference for the capacity involved. For smaller capacities the induction generator appeared to be cheaper than the synchronous generator, but that advantage was reversed when the machine became much larger in capacity than that in question. It was difficult to say where the dividing line should be because designs had not been developed for very large induction generators.

The original idea had been to have the Sron Mor station at the foot of the dam; in that case a synchronous machine would have been quite suitable, but later it had been decided to move it down-stream—as would be seen, the pipeline was about 700 ft long—and turbine regulation would have been difficult if a synchronous generator had been adopted. For that reason, among others, the decision had been made in favour of the induction generator. It was quite a novel feature of the station, but the Board was now introducing some others of a smaller size on its system.

Sron Mor station was entirely automatically controlled—a feature which is particularly desirable for stations built in intractable country, as that area was in winter. There would, of course, be local manual control in emergency.

On the remote control panel at Clachan there were five controls for the Sron Mor plant. There was a button which would start the turbine, a button to start the pump, a stop button, a button to control the output of the machine (which was done by electric drive of the guide vanes of the turbine), and a switch to open and close the generator breaker.

Pressing the turbine start button automatically started the auxiliary plant. It caused the turbine spiral casing to fill with water, and at the same time it emptied the pump casing of water by means of compressed air. When those operations had proved themselves electrically, the operator could then open the turbine guide vanes by means of the output control and run the machine up to synchronous speed. He could then close the machine on to the system and with the output control load it up to whatever load he wanted. It was all just as simple as that.

If the operator wanted to pump, all he had to do was to press the button marked "pump start." That set in motion a different sequence of events. It started the turbine first in order to get the machine rotating at synchronous speed before it was switched on to the system, so reducing switching surge; the pump was primed by means of an injector; the operator closed the generator on to the system; and the pump valve then automatically opened. The whole operation was done by the pressing of one button.

The control was by direct wire and the alarms and indications were taken back to Clachan power station by supervisory equipment working over pilot wires carried on the inter-connecting 33-kV power line between the two stations.

Fig. 10, Plate 2, showed that the machine incorporated a flywheel. It was one of the disadvantages of the induction generator that it had little flywheel effect. The flywheel was required not for the regulation of the turbine but for the pump. If the motor was driving the pump and the supply was tripped because of a fault developing, the water column which was going up the pipe would stop and then reverse and drive the pump as a turbine. That might lead to serious waterhammer effects.

The machine had a rigid coupling between the generator and the pump. It was common practice with pump storage plants on the Continent to have either a mechanical or a hydraulic coupling so that the pump could be disconnected from the machine very quickly

and so avoid losses which would otherwise occur when generating by driving with the pump either in water or in air. It was not considered necessary at Sron Mor to incur the expense of having such a coupling, but provision had been made so that if it was anticipated that the pump would not be used for a long period, the rigid coupling could be disconnected by putting jacks on to the flywheel and moving the shaft of the pump slightly longitudinally. By that means, a saving of about 1% was made in the losses which would occur when driving the pump.

Dr L. F. Cooling (Head of the Soil Mechanics Division, Building Research Station, D.S.I.R.) referred to the pore-pressure observations on the earth embankment of the lower reservoir. As stated by the Author, the dam was likely to be exposed to rigorous draw-down conditions, and, from the point of view of stability analysis, rapid draw-down was likely to be critical, and so it was decided to install pore-pressure points to enable observations to be taken both during construction and after the reservoir had been put in service.

The Building Research Station had been able to help in supplying details of suitable apparatus for the purpose and details of techniques, and in return the Author had provided them with copies of the results of his observations. They were particularly glad to have those results because very little data was available, particularly factual data, about draw-down pore pressures. So far as he knew, only a little work had been carried out and that was by the Bureau of Reclamation in the United States.

The pore-pressure measurements taken by the Author had been quite interesting. They had fluctuated with the reservoir level but with a lag depending on the position of the point relative to the distance from the free surface. Dr Cooling gathered, and hoped, that there would be more opportunities for taking further observations of the draw-down conditions. It was particularly important to assess how much of the time-lag was due to the equipment which was being used. He hoped that such measurements would be made and that particular care would be taken to see that the pore-pressure points were de-aired so that they were as incompressible as possible.

Already a brief analysis had indicated that the safe working rule given by Bishop in a Paper in *Géotechnique*¹⁰ (i.e., taking the pore-pressure coefficient $B = 1$) fitted the case fairly well. Dr Cooling hoped that in due course the Author would prepare a Paper, perhaps for *Géotechnique*, in which his valuable observations, together with soil tests and analysis, would be put on record.

Mr W. E. Blackmore (Senior Assistant, Sir William Halcrow and Partners, Consulting Engineers) referred to the design for the major stream intake shown in Fig. 21. He assumed that the pond formed upstream of the structure would ultimately fill with gravel and that it would be necessary for it to be mechanically or manually scoured.

His firm had also been developing an intake designed to reject the flow in spates when it carried gravel, so that the conduit would take only comparatively clean water. They had, however, sought to go a little further than the Author's design and to eliminate altogether the need for any human intervention to clear the accumulated gravel at the intake. It was done by a type of intake called a scoop intake, shown diagrammatically in Fig. 24. The principle was that at low flows the water entered the intake and met an accumulated body of water in the "bowl" part of the intake. That body stopped the flow from going further and diverted it into the conduit over intake weirs on each side and past baffles which stopped floating debris. As the flow increased, it reached a stage where it had sufficient energy to sweep away the body of water in the "bowl"; thereafter it traversed the whole intake as a shooting flow which discharged over the outlet lip and escape weirs and missed the intake weirs; during that stage, therefore, no water entered the conduit. At the same time it swept away any gravel which might have accumulated in the "bowl" or floating debris stopped by the baffle during the period when the flow was

10. A. W. Bishop, "The use of pore-pressure coefficients in practice." *Géotechnique*, vol. 4, p. 148 (Dec. 1954).

only small. When the flow again decreased, a point was reached when it no longer had enough energy to maintain the shooting flow and the conditions reverted to the original state; the "bowl" filled and the water passed over the intake weirs again.

There was scope for hydraulic refinements, mainly directed at narrowing the gap between the value of the flow which started the shooting flow and its value when it ceased to shoot. One of the refinements was to raise the outlet lip of the structure a little above the level of the side escape weirs with the effect that the increasing shooting flow, when it started to push the body of water out of the "bowl", more or less swept it sideways over the escape weirs until it established shooting flow over the outlet lip. Another refinement was slightly to dish the outlet leg of the intake so as to concentrate the flow into a jet which could almost wedge its way through the body of water instead of having to push it out frontally.

An intake of that type, constructed about a year previously, had worked very well although it did not have the hydraulic refinements first mentioned. There was no sign of any gravel having entered the conduit and there had been no need for any human attention to scour out the "bowl" of the intake.

Mr P. L. Critchell (Technical Service Advisory Officer, Expandite Ltd) referred to the methods used for sealing joints. He appreciated that bitumen plugs such as those described in the Paper were frequently used. There were, however, certain incompatibles. For example, to fill a small-diameter cavity completely with a sealing material which was poured hot, it was necessary for the material to be very fluid indeed to penetrate and fill the cavity satisfactorily. If, however, the material was of such low viscosity at that pouring temperature it was highly probable that subsequently, owing to normal atmospheric conditions, it would tend to flow through the small crack which would inevitably develop upon the shrinkage of the adjacent sections of concrete. As already indicated, therefore, another material such as copper strip was necessary to retain the thermoplastic material in position.

The incorporation of 5-ft-wide shrinkage gaps, as already pointed out, doubled the number of points where cracks might develop, and therefore doubled the number of joints required.

Referring to the remedial treatment described by Mr Guthrie Brown, Mr Critchell stated that once the adhesion of a bituminous material had been broken by water, no amount of heat would enable that material to adhere to wet concrete.

The work of the Bureau of Reclamation had already been mentioned and the Bureau had in fact developed flexible water-stops made of rubber, which were capable of accommodating movement and so would obviate the necessity of using 5-ft wide closing sections.

The rubber water-stops developed for the reclamation work in America had been in service since 1935, which, it was agreed, was still not a long period considering their application to dams. They had, however, accommodated during that period movements of up to $5\frac{1}{2}$ in. in shear and it would appear, therefore, that joints could be spaced at quite wide intervals, larger amounts of movement being accommodated by the water-stops without fracture. The water-stops did not rely on adhesion to the concrete, but on movement which could be predicted, i.e., movement due to shrinkage. They were shaped in a dumb-bell cross-section so that if shrinkage of one or other unit occurred, the flexible rubber at the end bulb of the dumb-bell was tightened against the concrete, giving a ball-valve effect.

It might be argued that in the case of copper large widths of diaphragm could be used, which would enable dense waterproof concrete to be obtained over a large area of the water-stop. It had been shown, however, that shorter widths could be used satisfactorily if reasonable care was taken in compacting the concrete.

Had the Author considered the incorporation of water-stops which would accommodate movement and did not rely on a bituminous sealing material, which, as Mr Critchell had indicated, if satisfactory in one respect would necessarily be unsatisfactory in another?

* * **Mr P. O. Wolf** (Lecturer in Fluid Mechanics and Hydraulic Engineering, Department of Civil Engineering, Imperial College) observed that during several visits to Glen Shira in the past 6 years he had been greatly impressed by the ingenuity displayed by the Author in dealing with difficulties which, with the application of conventional methods, would have found less economical and aesthetically less satisfactory solutions.

The reference to his former chief, the late Mr James Williamson, was greatly appreciated, for Mr Williamson had taken a keen interest in the development of the Shira catchment, which in its comprehensiveness resembled that of Mr Williamson's own Loch Sloy area. Mr Wolf felt that the choice of a round-head buttress of slenderer proportions, for the main Shira dam, would not have been made by Mr Williamson, who had favoured the slightly less economical but sturdier massive-buttress type, in the construction of which less skilled concreters could be employed. However, once the choice had been made, the detailed design would doubtless have received Mr Williamson's approval. That applied particularly to the 5-ft closing wedge between buttress heads: their advantage had been repeatedly stressed in Mr Williamson's publications, and Mr Wolf had found at another large dam in Scotland that the time interval specified for the construction of adjacent blocks (cast without a closing gap) imposed on the contractors difficulties and expense in the provision of extra shuttering and the limitations on the use of cranes, which would express themselves in somewhat higher unit rates. From the point of view of design, the advantage of the closing gap was that each buttress head could be allowed to shrink fully before the wedge in the gap was cast. If that closure took place after a period of cold weather, the vertical joints on each side of the wedge were not likely to open, either owing to temperature effects or because of the shrinkage of the narrow wedge. That applied particularly if it was accepted that the filling of the reservoir would, by the saturation of the buttress head, cause the concrete of the upstream face to expand.

With regard to the choice of sealing strip for the vertical construction and contraction joints, Mr Wolf appreciated that copper sheet suitably shaped had much to commend it, but site experience of the difficulties of bronze welding or brazing successive lengths of rigid sealing strip in the prevailing high winds led him to favour continuous strips of special rubber or "plastic" material, unrolled as construction proceeded, and combined with a bitumen which would just remain plastic at ordinary temperatures.

Mr Oscar Elsdon (Senior Engineer, Sir Alexander Gibb and Partners, Consulting Engineers) was interested in the adoption, for the embankment dam, of a thin reinforced concrete diaphragm hinged at the base. It was a possible disadvantage of a wall of that type that the leakage path through horizontal and vertical construction joints was very short. The Author had referred to the careful preparation of the construction joints. What precautions had been taken; had water-stops been introduced in addition to the customary scabbling?

Mr Elsdon noted that a considerable portion of the sand used for the concrete consisted of rock crushings. That practice could give rise to a great deal of trouble owing to the presence of rock-flour in excessive quantities. For that reason some engineers would not permit the use of crushed sand in water-retaining structures. Presumably the results at Glen Shira had been satisfactory, and it would be interesting to know what measures had been taken to wash the sand and to remove excessive quantities of flour.

Mr J. H. Thornton (Resident Engineer, Upper Moriston Works) said that to obtain fairly uniform pressure distribution under the base of a concrete dam an inclined buttress was the obvious choice. Present-day shortage of materials, however, led one to wonder if a rockfill dam might be better. He noticed that at the full height of the dam a rockfill structure was more expensive than a buttress type. What type of rockfill dam had been examined?

* * This and the following contributions were submitted in writing after the closure of the oral discussion.—**SEC.**

When comparing with a conventional mass-concrete gravity dam had the Author included for forming the 6-ft-wide contraction slots similar to those adopted in the lower dam? They doubled the quantity of contraction-joint shuttering and Mr Thornton doubted if the benefits merited the additional expense.

The maximum output of the concrete plant had been less than one-third of the rated output, and Mr Thornton asked if the limitation had been imposed by the rate of placing the shuttering or by the cableways. As the higher portions of the dam were being concreted had no limitation been imposed by being able to use only one cableway; or had the cableways been placed in such a manner as to reduce that limitation?

The area adopted for the bearing tests appeared small and could not give information about the deeper-lying strata. Had measurements been obtained of the deformation of the foundations under the load of the dam structure, and if so did it differ at any place from the value indicated by the tests?

Would not a rockfill structure have been more suitable for the lower dam in view of the high rainfall? It would have avoided the delays which occurred when placing soft material. In general, an earthfill dam should not be constructed in areas of high rainfall unless there were also long dry periods. It was not clear from the Paper whether the rate of draw-down had been limited to 5-ft/day because of the possibility of pore pressure becoming too high, or if that was the maximum rate of draw-down so far experienced.

From the drawings it appeared that the station itself would be more expensive than one built at ground level, since the walls and reinforced concrete roof were fairly substantial. To offset that, the cost of the tail-race and access tunnels had to be less than the reduction achieved in the cost of the pressure tunnel. It seemed, therefore, that for "tail" development stations generally the maximum economy was achieved by adopting the shortest possible pressure tunnel. The limit was probably reached when the free-flow conditions could not always be maintained in the tail-race tunnel and surge systems had to be used both upstream and downstream of the station. A short pressure tunnel could result in substantial saving in quantity of steel, weight of alternator, and the need for turbine relief valves.

Mr Thornton noted that the shaft at Shira was dry and self-supporting and that the weight of rock overburden was sufficient to balance the water pressure. In the design of such a shaft there would always be some doubt as to the point at which steel linings should be adopted. Steel linings had some disadvantages—they were expensive, steel was not always easy to obtain, the steel deteriorated in spite of protective coatings, and there was always the problem of external water. They were, however, watertight and should therefore be used where the surrounding rock would deteriorate if wetted. A concrete lining on the other hand was cheaper. It decayed no faster than the rest of the tunnel and it could withstand external hydraulic load. The rock and concrete could both withstand the external hydraulic load, otherwise no allowance could be made for it in the design of the composite shaft; there seemed to be little reason to use steel provided that:

- (1) extremely high heads were not being dealt with;
- (2) the rock cover was adequate;
- (3) the station was more than about 100 ft away.

With careful grouting some degree of precompression could be applied to a concrete lining to help negate the hydraulic tension and so reduce cracking.

Could the Author give the difference in overall cost and time to form flanged and welded joints? Mr Thornton believed that the advantage lay with the welded joint which could be individually tested and photographed with gamma-rays on site. Welding had been used in modern high-head sections on the Continent and in Canada but there was a limit to the plate thickness which could be jointed. Although flanges help to stiffen the pipe against buckling they would impede the proper placing of concrete around the pipe.

It was not clear from the Paper whether the grout holes perforated only the steel or if they penetrated to rock. If the latter, then some of the grout would be expected to fill fissures in the rock. The clearances found between the steel pipe and the concrete

filling appeared to be too small to be grouted; what had been done about it? Recently Mr Thornton had found similar clearances between the steel of the top casing of a draft-tube liner and the surrounding concrete. Two holes had been drilled within the same hollow-sounding area and it was only after maintaining water pressure on one hole for some hours that signs of dampness had appeared at the other. That seemed to indicate that any attempt to inject grout would be futile, so the holes had been plugged. At the time, Mr Thornton had considered the heat of hydration to be the cause of the trouble and he had noticed that hollow areas appeared at points where small lugs had been welded to the draft-tube liner, indicating some distortion.

The Author, in reply, expressed his gratitude for the reception given to the Paper and his personal thanks to Mr Fulton and others for their kind remarks. As Mr Fulton had indicated, the development and execution of the scheme involved a wide variety of problems and it had been a very real pleasure to be engaged on such a job. Mr Fulton's observations were of special interest since he had been closely associated with the impressive development and construction of the large-scale works undertaken by the North of Scotland Hydro-Electric Board and was thus in a position to draw some very useful comparisons and weigh up the merits of the designs submitted for projected works.

The Author expressed his thanks to Mr Banks for bringing out further points of interest relating to the choice of the reinforced concrete core wall for the earth embankment of the lower dam and for directing attention to the low intensity of grouting applicable to the schists and phyllites occurring at the sites of the Shira dams.

Mr Guthrie Brown had referred to the site of the Errochty dam and compared the foundation conditions at Errochty and Shira. The Author had had the opportunity of examining the foundation rock and conditions at both sites and did not consider there was much to choose between them. It should not be concluded that the Shira foundation rock was more favourable because it absorbed a much smaller quantity of cement in grouting. In the Author's experience the quantity of grout absorbed was not a reliable criterion since large quantities of cement were frequently required when forming a grout curtain in massive igneous rocks such as sound granite, the joints in those masses being generally well defined, whereas in the soft phyllites and mica schists the rock strata comprised a series of thin pliable laminae which the grout did not readily penetrate. If one were making a catalogue of the unfavourable features of the foundation conditions at the Shira main dam, there was the soft phyllite dipping in a downstream direction interspersed with narrow intrusions of harder rock, a region of pot-holed limestone, and a badly faulted zone which crossed the site near the spillway. The figure of 10 tons/sq. ft quoted by Mr Guthrie Brown for the Shira main dam referred to the maximum principal stress in the foundation rock. The maximum vertical pressure on the foundation rock was $6\frac{1}{2}$ tons/sq. ft, which was considered to be a safe figure for the conditions.

Considering that both dams were of about the same height and had to be designed for unfavourable rock conditions which were understood to be somewhat similar, it was not surprising that the two firms of consultants appointed by the Board, working quite independently, had arrived at designs of the same general form. The detailed design of the Shira dam had been undertaken in 1947 and tenders for the whole of the civil engineering works of the project had been invited in February 1948. The tender documents for the Errochty works were issued several months later and, as explained by Mr Guthrie Brown, the construction of the Errochty dam had been carried out ahead of the Shira main dam, the reason being that the former structure was situated alongside a public highway, whereas the works access road constructed to reach the site of the Shira dam extended to a length of 9 miles.

Mr Guthrie Brown had sought to compare the relative merits of the diamond-headed form, as at Errochty, with the round-head type adopted at Shira. The Author was of opinion that there was no case for departing from the round-head form, which was the basic structural conception, unless it could be established that the substitution of a series of straight faces, as in the diamond-head buttress, was more economical. The curved

shuttering for the upstream face of the Shira round-heads had not been in any way complicated and the panels had been raised stage by stage without difficulty (see Figs 6 and 7). There had been no awkward mitres to form as in the diamond-headed type and analysis of the actual prices quoted had disclosed that the unit rate for the curved shuttering was about 87% of the unit rate for the straight shutters at the sides of the buttresses, where mitres had to be formed. Having seen the shuttering schemes for both dams which, as explained by Mr Guthrie Brown, were being built by the same firm of contractors, the Author was satisfied that the round-head design was the more suitable for the conditions at Shira and would not hesitate to repeat it under similar circumstances.

The inspection shafts referred to in the Paper had been of considerable assistance during the construction stage and served as a permanent means of access to the foundation drains. The provision of the shafts had not added to the cost of the dam since the cost of the shuttering had been offset by a saving of concrete.

Mr Guthrie Brown had criticized the block and gap construction adopted at Shira and had compared it with the alternate block method adopted at Errochty, stating that the latter procedure provided greater structural isolation of the individual buttresses. Block and gap construction had been adopted for a number of dams as a means of reducing secondary stress effects and joint contraction caused by shrinkage. As stated in the Paper, the Shira design had been developed with due regard to the variable nature of the foundation rock to provide the maximum possible degree of structural isolation. Although the number of joints was thereby doubled, the main problem to be dealt with in the design of the joints, namely shrinkage, could be largely dismissed when the block and gap method was adopted. Having observed the manner in which the contractor had been able to raise the buttresses in groups, thus minimizing the handling of shutters from one block to another, the Author was satisfied that the block and gap method offered material advantages which were reflected in the unit rates quoted for the work, and he was of opinion that those advantages more than outweighed the disadvantages to which Mr Guthrie Brown had alluded.

Mr Wolf, who supported the Author's contentions relating to the advantage of block and gap construction, had rightly directed attention to the difficulties which arose on those exposed sites when using copper sealing strips. The erection and maintenance of those sealing strips admittedly required very thorough supervision on the part of the inspection staff and in that respect the rubber or plastic type of sealing strip was preferable.

Mr Critchell had also commented on the relative merits of rubber water-stops and copper bitumen for sealing the joints of a dam. Rubber water-stops had not been in general use in Britain when the Shira works were issued to tender. Soon after they were introduced commercially the Author had a series of tests put in hand in an endeavour to obtain reliable data regarding the probable life of rubber seals subjected to those working conditions. The tests had necessarily taken some time to carry out in an attempt to simulate long-term conditions and in the interval a final decision had to be taken regarding the joints of the Shira dam, the use of copper-bitumen seals being confirmed. On completion of the tests referred to, the Author had been satisfied that rubber water-stops had an application to such works and offered advantages in certain respects. He had adopted them for another dam which was at an advanced stage of construction. One disadvantage he had encountered in the use of rubber water-stops had been that of ensuring thorough compaction of the concrete against the rubber. The plastic material referred to by Mr Wolf was rather better in that respect and he was of opinion that there was scope for the production of heavier sections of that material for structures such as dams, the sections commonly produced being suitable for use in tanks and other such fluid-retaining structures.

The problems to which Mr Critchell had alluded relating to the use of bitumen seals as illustrated for the Shira dam had not arisen because means had been adopted to obviate those difficulties. The bitumen used was not of a very fluid type, as suggested by Mr Critchell, but was of a stiff consistency. When the work at each section of the dam was sufficiently advanced to enable the filling of the 5-ft-wide gaps to commence the open

sides of the diamond-shaped recesses were shuttered to form a container for the bitumen and hot liquid bitumen was filled in to a height of approximately 5 ft around a long tubular-shaped immersion heater centred within the recess. On stripping the shutters supporting the outer faces of the column of bitumen the concrete of the closing gap was raised one lift and when that was set the immersion heaters were switched on. Those heaters were of a rating sufficient to remelt the bitumen which penetrated into the pores of the recently formed abutting faces of concrete. The heaters were then withdrawn by a carefully developed technique to eliminate blow-holes or cavities and were then positioned ready for proceeding with the next lift. Before adopting those arrangements full-scale tests had been carried out with precast concrete blocks, so arranged that the joint faces could be separated for examination. When the immersion heaters were omitted there had been occasional evidence of blow-hole formation but the tests incorporating the heaters had given very satisfactory results, confirming that the methods developed should provide an efficient seal.

Contrary to Mr Critchell's suggestion, the upstream copper strip provided at the Shira dam had not been introduced to retain the bitumen in position but was the first barrier against leakage with the bitumen forming a second independent seal downstream of the copper.

Provision had been made for measuring deflexions during the filling of the reservoir and for recording the relative movement between adjacent sections of the dam at the closing gaps. It was agreed that the bearing areas of the rock loading tests referred to in the Paper were of very limited extent and efforts were being made to correlate the results obtained with measurement of the deformation of the foundation rock under the loading of the dam structure.

The Author was indebted to Mr Blackstone for contributing further information on the mechanical plant with particular reference to the Sron Mor generating and pumping set, of which brief particulars had been included in the Paper.

Dr Jaeger had rightly emphasized the value of model investigation on works of that type and the Author was grateful for the assistance he had received from Dr Jaeger and others who had undertaken those investigations, since the results had been most helpful in finalizing the design of certain sections of the work. The expenditure incurred on the model experiments had been saved many times over in economies achieved in the design.

From the design point of view the most complex problems to be resolved had been those relating to the effect of sudden drawdown on the stability of the earth embankment of the lower dam. That was a relatively small structure, but analysis by the several theories propounded had confirmed that investigation by means of pore-pressure measurement would be necessary to assess the order of the stability factor for a given rate of drawdown. Dr Cooling had expressed the hope that there would be further opportunities of recording the pore pressures under drawdown conditions and the Author was confident that with Mr Fulton's helpful co-operation further data would be obtained as operating conditions permitted. On obtaining a comprehensive series of records it was intended to submit the details for publication.

Mr Thornton had enquired if a rock-fill structure would not have been more suitable for that section of the lower dam. Rock-fill construction would have been more convenient but also more expensive since the filling would all have had to be quarried. The earth embankment was a relatively small element of a large scheme and, although there had been delays in its construction, to which reference had been made in the Paper, those had not affected the overall completion of the project and commissioning of the main generating plant. The drawdown of 5 ft per day was the maximum rate so far experienced and the records of pore pressure obtained to date indicated that for the range investigated the dam was well within the desired factor of safety.

Mr Blackmore, in referring to the design of the major stream intakes, had pointed out that the pool formed upstream of the structure would tend to fill with gravel. As explained in the Paper, the designs had been developed with due regard to the tendency for debris to accumulate at any barriers formed across those watercourses and the intakes should

still function in a satisfactory manner even if the gravel accumulation built up to spillway level.

Mr Blackmore's suggestion for a self-cleansing type of intake appeared to suffer from the disadvantage that the flow to the aqueduct was cut off entirely during flood periods. For a high-head scheme such as Shira that would represent a considerable loss of revenue since short intermittent spates were a frequent occurrence, being a significant element of the annual run-off. The Author's firm had also developed a type of intake arranged to deal with the trash passing downstream during spates. The design had not been considered suitable for general application to the Shira scheme and the Author had not included a reference to it in the Paper. In place of the usual weir a concrete sill was formed at the downstream end of a natural pool in the stream and the water was diverted to a chamber at the side of the pool constructed with the necessary trash baffles and self-regulating inlet well as for the large stream intakes shown in Fig. 21. The proportions of the pool and sill were so arranged that any debris which might accumulate would not restrict the entry to the aqueduct and would be swept downstream during flood periods.

Mr Elsdon had enquired regarding the joints of the reinforced concrete core of the earth embankment for the lower dam. Copper sealing strips had been provided in the vertical joints and the horizontal joints had been thoroughly scabbled and bonded with a layer of stiff grout. The application of bituminous emulsion to the outer surfaces of the wall, as referred to in the Paper, had been provided essentially as a protection against deterioration of the concrete and consequently of the reinforcement, since the wall was buried and inaccessible for inspection or maintenance. The emulsion coatings also served as a useful means of sealing any deficiencies there might be in the joints of the core.

The rock sand used in the concrete was all washed to remove fine dust and rock flour and complied with B.S. grading. The site quarry plant incorporated "Sandor" equipment and was operated to produce the maximum quantity of sand obtainable by economic working. Tests carried out proved that a high proportion of the rock sand could be used in the mix without detriment to the workability of the concrete, and for a given cement content and water/cement ratio an increase in the proportion of the rock sand raised the compressive strength.

Mr Thornton had commented on the choice of an inclined buttress type for the main dam and had suggested that there might have been some advantage by adopting rock-fill construction. The alternative design prepared for rock fill had been in accordance with normal practice and incorporated a flexible reinforced concrete membrane on the upstream face. Any deviation in matters of detail would not have altered significantly the cost relation given in the Paper, since the rock fill would have had to be quarried in any event and there were deep deposits of peat on the site which would have had to be excavated and backfilled with rock fill. In Britain the relative prices of concrete and rock fill were such that rock-fill construction was seldom more economic unless a considerable proportion of the material was available from excavations necessary for other works. The position was reversed in many parts of the world where skilled labour was in short supply. In some cases owing to climatic conditions rock fill was favoured because the work could proceed over a longer construction season.

The Author had been associated with another Scottish hydro-electric scheme for which contractors tendering had been invited to quote alternative prices for concrete gravity and rock-fill construction. The dam referred to had a maximum height of about 75 ft and the site conditions were favourable for rock-fill construction but the tenders for that design had been fully 33½% higher than those submitted for concrete gravity construction. The concrete design was generally similar in form to that adopted for the spillway section of the Shira lower dam including 6-ft-wide construction gaps. The Author had given his views on the relative merits of block and gap construction in his reply to Mr Guthrie Brown's contribution.

The rated output of 100 cu. yd/hour given for the main batching plant compared with a maximum of 70 cu. yd/hour for the capacity of the cableways. The batching plant was used for supplying all other concrete works in the vicinity of the main dam and the

margin was provided to cover the peak requirements having regard to those other works. In comparing the weekly output with the hourly ratings of the mixing and concrete handling plant, allowance had to be made for weather conditions and the effects of working 12-hour shifts including night working, also the progress with the erection of shutters and other operations for which the cableways were used on occasion. The cableways had been arranged so that both machines could be used when concreting was proceeding at the higher portions of the dam, but there was a moderate reduction in the load carried since the cables had to be tensioned to provide the requisite clearance when passing loaded skips over the partially completed sections of the dam.

As stated in the Paper, the decision to provide an underground station at Clachan had been taken following comparison of the tender prices for alternative surface and underground layouts. Mr Thornton had suggested that the walls of the underground station were fairly substantial. Those walls comprised panels of common brickwork, 14 in. thick, and the cost had been very much less than would have been required for a surface station since the latter would have been faced with masonry to conform with amenity requirements. In that case, the adoption of a "tail" development with a short pressure tunnel had been ruled out since it was desired to divert the tributary streams along the route of the main tunnel into the pressure system.

As Mr Thornton had stated, there was always some doubt regarding the point at which steel linings could be dispensed with in pressure tunnels and shafts. The rock conditions at Shira were such that steel lining had been considered essential for the greater part of the length and a change to concrete lining, involving the provision of special shutters for the upper section of the sloping shaft, had not been favoured. When first efforts had been made on some schemes to pre-compress concrete linings by grouting, the results were likely to be doubtful when dealing with rock which varied in quality from point to point. The grout holes at the Shira pressure shaft had extended through the steel and concrete lining to the rock, and the main purpose of the grouting had been to ensure that there would be no open fissures or cavities around the encasing concrete. An effort had been made to inject grout into the clearances around the steel lining and that had confirmed that the clearances were of a very minor order which would close up on filling the shaft by elastic deformation of the steel lining. The flanged joints had not impeded the placing of concrete around the pipes as had been suggested by Mr Thornton. The steep slope of the shaft had been helpful when placing the surrounding concrete and immersion vibrators had been used with good effect.

Welding might have been adopted more extensively, but as indicated in the Paper the Author had not been satisfied that dependence should be put on overhead welding for the high pressures involved. The position was different where the welded joints had been introduced since the welder's position relative to the weld was much improved on the slope. For the thicker plates site welding had been considered impracticable and for the thinner plates the welded joints had been less expensive. The flanges had been manufactured in pairs and matched at the pipemaker's works. There had been no difficulty in bringing them together at the site and the flanged joints had generally taken no longer to make than the welded joints. Had the welding been extended to thicker plates the time for making those joints would have been greater than for the flanged joints.

Mr Thornton had listed certain criteria relating to the conditions under which steel lining could be dispensed with. The Author did not favour the general acceptance of such broad assumptions and considered that each case should be examined with due regard to the circumstances, particularly the nature and condition of the surrounding rock. In designing a pressure shaft or pipeline for works of that magnitude and head the designer must have particular regard to the risks involved and the serious consequences which would arise should a failure occur on such an important element of a scheme; a form of conduit which might be appropriate on a small-scale low-pressure installation would be unsuitable for works of much greater head and scale.