

Design aspects of the Strathfarrar and Kilmorack hydro-electric scheme

by

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Mr L. H. Dickerson (Chief Civil Engineer, North of Scotland Hydro-Electric Board) congratulated the Authors for their valuable contribution to the records of the Institution. He said there were, in the scheme, several features not previously met with in other developments of the Hydro-Electric Board, and these were well described in the Paper.

74. Mr Dickerson said he would confine his remarks to a discussion of the historical background of the present scheme and a comparison with the earlier development proposed by the Grampian Electricity Supply Company in 1929. This proposal was rejected by a House of Lords committee after a protracted inquiry, and it was not until 1957 that the Hydro-Electric Board put forward their scheme for development of Strathfarrar and Lower Strathglass, so as to complete the utilization of the River Beaully basin. The inquiry in this case decided in favour of the development, and work was started early in 1959. The earlier Grampian proposal was shown in Fig. 32 and two important differences would be noted. The first was that it was then proposed to develop the fall through the Aigas/Kilmorack gorge (Power Station 4) as a single stage, with about 120 ft head, compared with the two-stage development which was actually built, giving a combined head of about 112 ft. The merits of the two-stage scheme were particularly evident now that the works had been finished. In the single-stage scheme, the water level at Kilmorack was raised by about 100 ft, which meant not only the virtual disappearance of the whole of both the upper and lower gorges which were important scenic and geological features, but also entailed a good deal of flooding in the wider portion of the valley with an extensive diversion of the public road. In its present state, even with the increased water levels, a large part of both the gorges was still preserved and with the restricted flooding at the upper end of the Kilmorack Reservoir no road diversion was needed. The fact that in the present two-stage development all the mechanical equipment, sluice gates, turbines and alternators were of an exactly similar pattern in both dams reduced the disadvantage of having two installations instead of one. In addition, the Fishery problems on a 60 ft dam were easier than on one twice the height.

75. The second difference was that in Strathfarrar the original scheme proposed three stations to develop a maximum gross head of 497 ft compared with the present two-stage scheme with a maximum gross head of 562 ft. Of the total difference in level of 707 ft between the top water level in Monar and the tail water at Kilmorack the present scheme developed 674 ft or 95% through four stations, whereas the earlier scheme only provided for development of 617 ft or 87½%.

76. The total storages available in the present scheme (i.e. 5193×10^6 cu. ft) were roughly 25% greater than those originally proposed, the major part of this increase

* *Proc. Instn civ. Engrs*, 1965, 30 (March) 449-487.

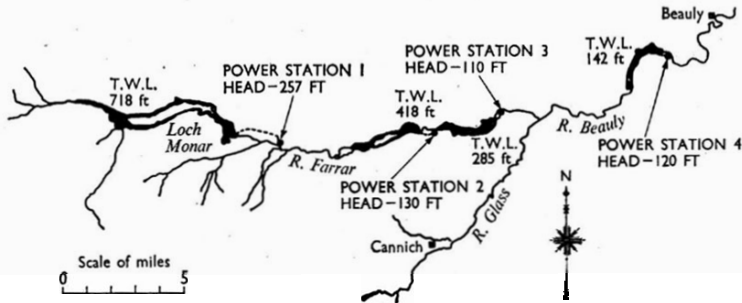


FIG. 32

being caused by the larger storage in Loch Monar, which more than compensated for the reduced storage in Loch Beannachran.

77. Originally the upper stage was developed with a tunnel about $2\frac{1}{4}$ miles long, while the intermediate and lower stages had tunnels or aqueducts each of about $\frac{1}{2}$ mile in length, a total for the three stages of about $3\frac{1}{4}$ miles compared with $9\frac{1}{2}$ miles total on the present scheme. Mr Dickerson felt that this considerable difference was only made practicable by the reduction in tunnelling costs brought about by the great advance in techniques over the last 30 years.

78. It should also be noted that the earlier scheme did not augment the flow to the power stations in Glen Strathfarrar by diversion from downstream tributaries. The present scheme had the Misgeach and the Deanie aqueducts to augment the flow to the top power station and obtain the benefit of the storage provided in Loch Monar.

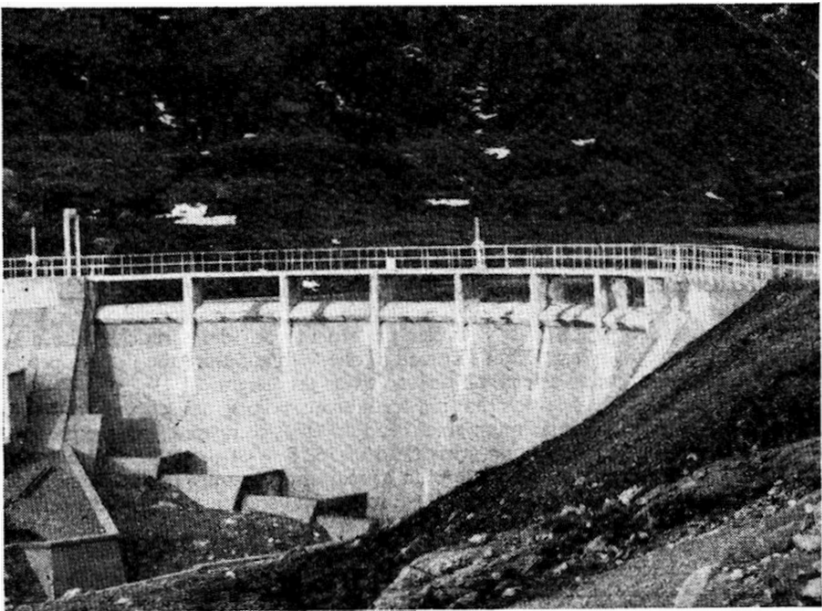


FIG. 33

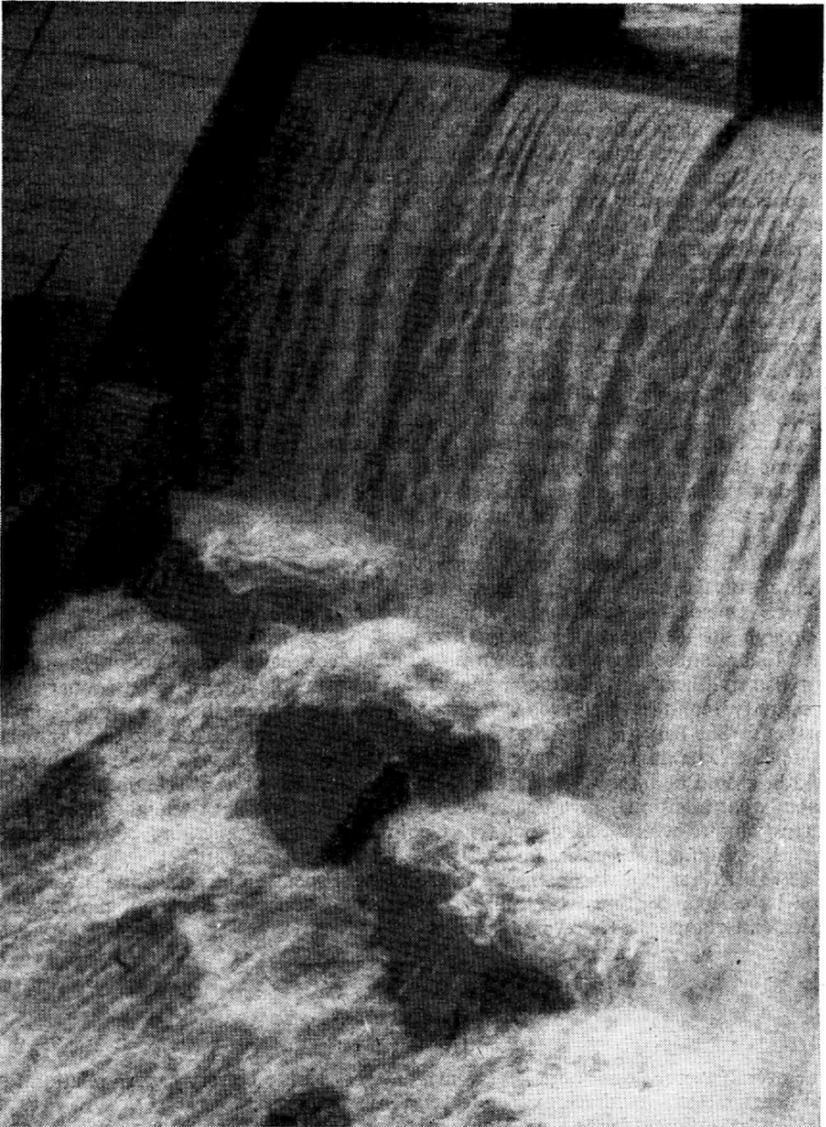


FIG. 34

Again, both of these diversions involved a good deal of tunnelling which might have been the reason for their omission, although other schemes planned by the Grampian Company at about the same time did provide for extensive diversion works.

79. Referring to the details of Monar dam given in the Paper, Mr Dickerson was interested to note the Authors' remarks about the need to wait for the coldest period of the year before grouting the contraction joint, and he confirmed that, should the

concreting programme not be sufficiently advanced to allow this grouting to be done at that time, as much as a year of water storage could be lost, although in other respects the dam was virtually complete at an earlier stage. Could the Authors suggest any way in which this risk could be overcome, possibly by thickening the lower portions of the dam slightly to extend the cantilever effect, so that more water could be stored before the final grouting had been done?

80. The design of the spillway arrangements at Monar with the tilting crest gates was a bold one, and he knew of no other arch dam where this principle had been used. It apparently involved complicated stresses in the upper arch ring. In discharging such large quantities of water to fall free from the crest, the quality of the underlying rock and concrete in the spillway channel and the downstream apron had to be good. In a small arch dam 50 ft high (Fig. 33), completed in 1963 on the Island of Harris and designed by the Hydro-Electric Board's staff, the rocks downstream were of doubtful nature and it was considered essential to provide some way of catching the water and dissipating as much as possible of its energy as it fell from the free overfall spillway. As a result of model experiments it was found that this could be effectively done with a series of small stilling pools around the foot of the arch (Fig. 34).

81. The design theory used at this dam at Chliostair was similar to that adopted at Monar but some additional simplifying assumptions were made. The Hydro-Electric Board proposed to investigate further theoretical stresses using an analysis based on the thin-shell theory developed by Dr S. R. Davies at Edinburgh University, and Mr Dickerson asked if the Authors had any similar ideas for the Monar dam.

Mr E. J. K. Chapman (Messrs James Williamson & Partners) congratulated the Authors on the features of the scheme which had been selected for inclusion in the Paper. One of the main themes running through the Paper was the importance of thorough site investigation before design work was started. The investigations for the Monar dam seemed to have been very thorough as they included not only borings but also trial headings into the abutments. However, the experience at Monar was a salutary reminder that even after thorough investigation the unexpected could happen. The Paper implied that if the presence of the faulted dyke had been known in advance, the dam would have been sited further downstream and Mr Chapman said he would be interested to know if this was so. It would also be interesting to know what additional investigations the Authors would suggest might have been carried out to avoid this difficulty. Could they suggest anything beyond more extensive drilling both upstream and downstream of the dam area?

83. Mr Chapman was interested in the amount spent on planning and investigation by various continental authorities as a proportion of the total of the project costs, and quoted the following figures:

France	2½–3%
Norway	1–2½%
Portugal	1½–2½%
Sweden	2–4%.

The overall average was about 2½% which did not seem to be very high in relation to the benefits that should accrue from careful investigation.

84. A considerable section of the Paper was devoted to the strain measurements at Monar dam, but the results had been difficult to interpret. Recently, extensive strain observations were made at one of the buttresses of the Stwlan dam which forms the upper reservoir of the Ffestiniog pumped storage scheme. Being a pumped storage reservoir it appeared to offer a unique opportunity for observing strains caused by a complete emptying in a matter of a few hours thereby eliminating several factors, other than loading, which might cause strains when long-term observations were taken. However, these results also proved difficult to interpret as the following sets of observations might illustrate. It was found that surface strains of the same

order of magnitude were measured on two occasions when caused by two quite different factors. On an overcast day of extremely even temperature the strains caused by a complete emptying of water were similar in magnitude to those caused on another day when there was no change in water level, but the sun shone brightly and caused a marked change in temperature. This was not inconsistent with the conclusion of § 36 of the Paper that the observed deflexions of the crest of Monar dam had been mainly influenced by temperature changes.

85. One of the unusual features of the Strathfarrar and Kilmorack scheme as a whole arose from the adoption of gated spillways throughout, to the complete exclusion of any free-flowing spill weirs. Gates would seem to be a natural choice for the two lower dams at Aigas and Kilmorack which contained power stations but not for the Monar dam. It was not clear why the choice of tilting gates was made for Monar. Mr Chapman asked if there was some overriding reason why such a small flood rise was permitted, or if there was an economic case for maintaining a higher operating level than would have been possible by the use of the cheaper spill weirs. Allowing for the narrower piers that could have been used with the spill weir, for no extra length, a flood rise of about 3 ft 6 in. would have sufficed to pass a 'normal maximum' flood and this rise would only increase to about 5 ft for 'catastrophic' floods. Mr Chapman had some misgivings about the use of the tilting gates at Monar although he was glad to learn from Mr Dickerson that they had operated satisfactorily.

Mr D. A. Harries (Mitchell Construction, Kinnear Moodie Group Ltd) said that after reading § 5 he was a little concerned that perhaps this dam would not have been constructed if the full extent of the geological conditions had been known when the arch dam was chosen in preference to the gravity dam. Perhaps the Authors would confirm that his fears in this respect were unfounded. He felt that the Authors had been too harsh on arch dams when they suggested that a greater allowance for contingencies should be made for this type of structure. One of the risks in all civil engineering work was the geology of a particular site. It was fairly certain that the cost of site investigation for an arch dam would be higher than that required for a gravity dam, but the element of the contingency sum which was included for the geological risk, could surely be reduced which would make arch dams more acceptable to the Authors. Referring to § 5, Mr Harries wondered if the need for close observation of an arch dam was really much more of a burden to the promoter than the routine inspection of any dam, and he asked the Authors to provide some information on this point.

87. With respect to the instrumentation at the Monar dam, Mr Harries wished to ask the Authors about the purpose of the strain gauges which were embedded in the dam concrete. In § 29 it was stated that the gauges would not provide sufficient information to permit any direct comparison between theoretical stresses due to the water level as measured by the model, and those obtained by theoretical calculation. From the results given in Figs 8 and 9 it was difficult to correlate the different stress values obtained by the two methods. Mr Harries wondered if the Authors considered that additional strain gauges would have been worthwhile in order to check the two sets of figures, because an opportunity for research might have been missed by not having them.

88. In § 39 the Authors lamented the fact that the organization of construction work was becoming less flexible and less able to cope with the unexpected. Mr Harries wanted to reassure the Authors that contractors were always able to cope with the unexpected; the lack of flexibility was in their rates for doing the unexpected if existing Bill rates are used to pay for it! The construction industry today probably had more competition than ever before, and need for detailed planning which should come into the tender if the contractor hoped to be at all competitive under present conditions, should be emphasized. Critical path analysis enabled a computer to be used at the time of tender to spread and balance plant on large contracts so that it

could be used under optimum conditions. Such an analysis was bound to reduce the tender but if, as mentioned by the Authors, the rhythm of construction was disturbed unduly there was a much greater effect upon the overall economy of the work than was the case if such detailed planning and refinements were not applied to the Bill rate. The Authors had emphasized that the client could have the advantages of keen tendering, when it was associated with accurate site investigation to reduce the likelihood of large variations on the actual job due to changed ground conditions. Mr Harries thought that the risk of the penalty referred to in § 39 was increased when optimum planning conditions were applied to tenders for projects where ground information was too scanty to make forward planning anything more than an optimistic guess.

Mr R. H. Cuthbertson (Messrs R. H. Cuthbertson & Associates) said that he wanted to comment on two or three aspects of construction which the Authors had emphasized. It appeared from the remarks in § 5 that there was little scope in Gt Britain for the deployment of arch dams, and this more from an economic point of view than from the already obvious lack of topographically suitable sites. The relatively small margin of costs in favour of an arch rather than a gravity dam which the Authors indicated, could well be absorbed by the additional cost of extended site exploration, of excavation of foundations before designing the concrete arch, and of rock fortification.

90. The elimination of free-discharge spill weirs from all the dams implied that the scheme was an interesting departure from normal practice, especially as three different basic types of spillway gates were incorporated. Mr Cuthbertson wondered whether, in the case of the Monar dam, the concentration of water load at the crest of the dam, which arose from the type of gate adopted, imposed any significant design difficulty. Like Mr Chapman, he also had some misgivings on the design of the gate at Monar and asked the Authors for information concerning its origin, and records of experience they had had before it was adopted. It had certain similarities to a floating drum gate in operation, but was obviously more economical in construction and material. He wondered what maintenance problems were envisaged. As far as could be seen from Fig. 24, there did not appear to be any access from the operating well into the chamber to give access to part of the underside of the gate.

91. The Authors' remarks on the reliability of operation of electrically and hydraulically operated gates were reassuring but Mr Cuthbertson wanted to know what criterion was adopted in considering the consequences of failure to operate. In the case of Monar it would appear to entail the overtopping of the roadway across the dam.

92. On the question of design floods he said that, while in the case of Monar and Beannachran the ratio of normal maximum to catastrophic was normal, in the case of Aigas and Kilmorack the ratio was low, viz. 1 to 1.33. Was this because the associated generating stations were on a 'run of the river' basis?

93. The Authors' appreciation of the need for accurate advance knowledge of ground conditions would be endorsed by all those concerned with the construction of dams. The relatively small scale of many engineering projects of this nature in Gt Britain in relation to the potential production to be achieved by modern techniques tends to enhance the penalties which can arise from the interruption of the 'rhythm' of construction to which the Authors refer.

Mr J. Paton (Partner, Messrs Babbie, Shaw & Morton) said that § 5 and § 39 would seem to indicate that there were relatively few opportunities for large dams of the arch type in Gt Britain. In that case, and having regard to the cubic content of Monar dam, the margin of 9% between the tenders for dams of the arch and the gravity types was indeed slender. It was customary, as had been mentioned, to allow contingency margins of about 10% for civil engineering works where there was a

degree of uncertainty as to the extent of foundation requirements. As mentioned in the Paper, it seemed logical that a larger contingency margin should be adopted in the case of an arch dam which had more exacting requirements for foundation and abutments.

95. In § 5 the suggestion was made that it was desirable to excavate the foundations of the dam before designing the concrete arch, and in § 39 reference was made to 'the rhythm of construction' and to the seriousness of disturbing this rhythm. Mr Paton found some difficulty in reconciling these two statements. If it was the intention to require the contractor to complete excavation towards the end of one construction season, and then wait for a winter period while the arch design was finalized, there was likely to be a problem in maintaining continuity of construction through to the start of the next construction season. It would be helpful if the Authors could expand on this theme and explain how, in the circumstances, 'the rhythm of construction' was to be maintained. Mr Paton suggested that there could be an alternative such as designing to less precise limits and avoiding this problem which seemed to involve postponing the final detailed design until excavations had been carried out.

Mr C. C. Marshall (Sir Alexander Gibb & Partners) remarked that it would be most helpful if the Authors could give more information on the design of the gates, on their manufacture, how the gates had in fact worked, and whether there had been a sizable flood since they had been in operation. He was most interested to note the use of drum gates. His firm had experience of drum gates at Pitlochry and Clunie which had on the whole been most satisfactory. However, an interesting condition had been observed at Clunie where, as in the case of Aigas and Kilmorack, the approach to the dam was comparatively narrow, with the result that when the drum gates started to function water levels dropped, causing an appreciable approach velocity and resulting in a tendency to hunt. He wondered if this effect had been noticed in the project under discussion.

97. These works in Scotland had been helpful to all concerned with work overseas. Apart from the electrical equipment, gates formed a significant proportion of expenditure and he felt it was somewhat sad to see so much gate construction being done by foreign and not British manufacturers. He wondered whether the Authors could comment as to why this was so.

Mr J. Coward (Generation Engineer, Conon/Beaully Group, North of Scotland Hydro-Electric Board) said that before discussing the Paper he wanted to mention the arrangements for passing fish. Aigas and Kilmorack each passed about 10 000 fish per annum and the design had been very successful. This was because the positioning of the fishpass in the centre of the dam between the two machines, and the shape of the plan of the dam, encouraged fish to move along the dam towards the fishpass entrance. The principle of the Borland fish lock had been entirely successful, despite some minor difficulties with the sluice gates at the upper end of the lock. He felt that the only satisfactory method of operation for these sluices was through rising screws, driving the gate in both lifting and lowering movements. This method was much more positive in a gate of this sort than gravity lowering, when a gate tended to stick if it was not heavy enough to overcome frictional forces. At Aigas and Kilmorack, because of restricted headroom, the gates at the lower end of the fish lock were operated by non-rising rotating screws; these had proved quite successful although the restriction against operating on unbalanced heads in excess of 20 ft had been a handicap.

99. At Deanie station, where the head could vary by at least 10%, a single Deriaz machine would be attractive. Could the Authors say if a suitable design was available?

100. The Paper concisely described the ingenious and successful method of using

a common oil system for bearing lubrication and governor operation which simplified the problem of runner blade operation. Mr Coward thought, however, that the main advantage of this arrangement was that it obviated the need for an oil distributor head at the top of the shaft. There was no advantage at all in avoiding a hollow shaft and indeed the existing Culligran machine had a hollow shaft.

101. It was stated in Table 3 that the sector gates at Beannachran and the drum gates at Aigas and Kilmorack dealt with daily variations in level. Mr Coward thought this was not strictly correct because generally these gates were intended to pass small floods and thereby maintain a reasonably constant reservoir level. The radial gates at Beannachran and also at Aigas and Kilmorack were provided basically for discharge of medium and high floods. Under normal flow conditions there was little variation in the headpond levels at Aigas and Kilmorack stations which could each pass about 5400 cusec at full load. This flow gave a fair margin for runoff from the incremental catchment area, in addition to the discharge from the stations further upstream (i.e. Culligran and Fasnakyle) which together passed a maximum of 3500 cusec.

102. The electrical arrangements and control of hydroelectric schemes in many places in the past had on the whole been based on the practice in a predominantly thermal system. The first real departure from this was the introduction of centres controlling not only plant but also the associated transmission system. The Paper recorded the culmination of this concept in the section dealing with remote control. This was an excellent example of a major complex group where all the control was centralized.

103. An earlier development with the same electrical requirements as Aigas and Kilmorack would either have had two transformers, two 132 kV oil circuit breakers and two transmission lines or, alternatively, with circuit breakers on the generator side, two generator oil circuit breakers, two transformer oil circuit breakers, and again two transformers and two transmission lines. This had been reduced to two generator voltage circuit breakers, one transformer and one transmission line, and the performance of the plant as part of the system had not suffered at all by this simplification.

104. The automatic pre-set load control equipment had been very satisfactory. The static equipment controlled the output of the turbine by acting on the conventional mechanical governor.

Mr D. M. Hamilton (Partner, Messrs Crouch & Hogg) said that Mr Cuthbertson had drawn attention to the relationship of catastrophic flood figures to the corresponding normal maximum floods. So far as Aigas and Kilmorack were concerned this would appear to arise from the amount of reservoir capacity available above those two stations, but further information on the means by which the effective catastrophic flood outflow was obtained would be of interest. Mr Hamilton believed that no allowance was made, or no reliance completely placed upon, the passage of water under flood conditions through the machines.

106. Regarding the decision to use flood gates without spillways the Authors stated that in their world-wide investigation of 800 gates they had come across no case of a failure to function when they were required. He was not sure whether this related to drum or tilting gates, but on the assumption that it related to flood gates as such it appeared to be a very remarkable record. In so far as electrical and mechanical equipment were relied upon for the discharge of floods he felt that some free overspill capacity was desirable.

107. As far as the site investigations were concerned, in § 42 it was stated that very careful initial investigations had been carried out and that on the basis of these investigations the choice of the arch dam had been felt to be a sound one. In the Paper by Henkle *et al.*⁸ it was stated that the rock was extensively exposed on the north side of the valley. Initial site investigations made in 1951 and 1952 were confined to two comparatively shallow borings on the south side together with geological appreciation based on geological survey. This work confirmed the feasibility

of the site but the rock was observed to be rather heavily jointed. The next paragraph of that paper stated that, soon after the contractor had been appointed, two headings were driven one on each side of the valley beneath the abutment; the rock in these headings was found to be sound. In view of the brevity with which Dr Henkle and his associates described these preliminaries it might be of value for the present Authors to give more details of the further investigations which presumably were carried out between 1952 and the exposure of rock in the excavations, which occasioned the calling in of Professor Skempton and his colleagues.

108. Professor Skempton, Dr Henkle, and others carried out investigations, particularly of the lamproschist dyke and established certain results from experiments in the laboratory, no doubt taken from samples of borings. The Authors of the present Paper went on to say that the foundations were grouted to reduce the shear resistance along the joints in the rock, to fill open joints to increase the overall crushing strength of the foundations, and to form a grout curtain against seepage beneath the dam. However, while the Authors gave details of the natural conditions found and the grouting measures taken, there appeared to have been no follow-up in the form of rock cores or at least these were not mentioned, particularly in the vicinity of the lamproschist dyke; these would have given an indication of the extent to which the shear resistance in the joints of the rock had been improved or the overall crushing strength of the foundations increased.

109. Regarding the improvement of watertightness, the description that was given of the lamproschist dyke suggested that little grout would be taken and therefore it would presumably remain reasonably watertight like other clay fissures in the vicinity. Mr Hamilton wondered if the Authors would give more details of measures which might ensure that clay fissures, whether small or large, could be treated to ensure that, not only were they watertight for many years but also they gave a satisfactory shear resistance or an improved shear resistance from their original state.

110. In so far as cost was concerned, it would be helpful if details could be given of the extent of the cost of the investigations and also of the relative cost of the dam as constructed compared to a gravity dam which would have been constructed under similar circumstances.

Mr D. F. Campbell (Partner, James Williamson & Partners) said that when the opportunity arose to install some instruments in the Cruachan pumped storage dam gauges manufactured in Italy, similar to those installed in the Monar dam, were chosen; as a precaution against breakdowns parallel duplicate gauges were installed in the directions of anticipated principal stresses, at the upstream and downstream faces of the selected buttress. It was of interest to note that the upstream parallel gauges gave, as expected, almost identical strain readings but within a few hours of placing the readings of the downstream pair of parallel gauges diverged, although both gauges indicated the usual tendency towards rapid contraction during the setting of the concrete. One of the gauges showed a deflexion of -200 microstrains from zero, the other a deflexion of -500 , which presented the anomaly of two similar gauges similarly stressed, with an indicated strain difference of 300 microstrains. This demonstrated that during the very early life of concrete the absolute value of strains measured could not be taken as having anything to do with stresses in the dam.

112. After diverging so much, the parallel gauges began to act in consort after about 20 months, and thereafter the difference in readings varied between 300 and 330 microstrains. This led to the conclusion that perhaps the strains in concrete were not transferred to these gauges with the accuracy which the manufacturers claim for the instruments.

113. The advantage in instrumenting pumped storage dams was that with the rapid fluctuation of water loads many of the variables could be eliminated. For example, dilatation, shrinkage, creep, elasticity and Poisson's ratio would be reason-

ably constant throughout the experiment. By choosing the right weather conditions for the experiment temperature variations could also be small enough to be negligible.

114. In Gt Britain there seemed to be two disadvantages in interpreting the strains of embedded gauges. The first was the lack of an official research organization to correlate all experiments and to ensure adequate instrumentation in dams; the second was the weather. It would be noted that the no-stress gauge reading given in Fig. 7 of the Paper indicated that in general the concrete after setting was expanding rather than contracting. This was also typical of the Cruachan result and it was significant that in both cases the rate of expansion was greatest during the winter months. This expansion was characteristic of a water-cured concrete. Dr F. M. Lea¹⁰ stated that hydration of cement particles ceased when humidity in the materials dropped below about 80%. Lower humidities were a rare occurrence in the Scottish West Highland winter atmosphere. Only in the afternoons for a few short periods during the summer months could an atmospheric humidity of between 60% and 80% be expected. Thus we seemed to be dealing with water-cured concretes which expanded in winter, shrank during the few air-drying months of the year, and returned to a lesser degree of expansion the following winter, since after air-drying, saturation was only partially effective in restoring dilatation. This would also mean that the degree of expansion and shrinkage of a particular concrete was a function of the season of the year in which the concrete was poured. Altogether this property of concrete was very much more difficult to assess in the British climate than in that of many other countries.

115. Much work had been done on the quantitative analysis of strain-meter results; perhaps the fullest explanations of procedure emanated from Portugal. Two references were given in the Paper^{5,6} and it was interesting to note that in the six years that had elapsed between the appearance of these two papers additional terms had been added to the quantitative analysis to allow for non-recoverable strains. Also humidity measurements seemed to be taken at several Portuguese dams. It might be that when humidity terms were added, the coefficients of influence of the various equations would become physically intelligible; certainly at some time this ought to happen when enough influences were considered in analysis.

116. The Authors were to be congratulated upon adding some knowledge to a subject in which so much research was still to be done. Although the results might be very difficult to interpret it was really by the behaviour of the finished dam rather than by the deflexions of models or the results of theoretical analysis that the safety of the structure would ultimately be known.

Mr P. L. Aitken (Chief Hydraulic Engineer, North of Scotland Hydro-Electric Board) endorsed Mr Roberts' remarks that the scheme was a very good one. It would produce electricity at a load factor of about 30% and storage was good, therefore electricity could be produced when the system most needed it. This was being done, notwithstanding the difficulties which the Authors and the Contractors had overcome, at a price which was below the average cost of generation in Gt Britain.

118. He added that it was now difficult to obtain authority to develop the hydro-electric potential of Scotland further. Nevertheless, in many countries in Europe more hydro-electric plant was being added to existing hydro-electric schemes to increase the amount of flexible low-load factor plant in the electrical systems of these countries and supplement the operation of the large thermal and nuclear power stations which were being built.

The following contributions were received in writing:

Dr W. H. R. Nimmo (Consultant) wrote that the average annual runoff (Table 1) represented from 86% of the rainfall for the Monar catchment to 83% for Kilmorack. In the experience of the writer in the sub-tropical region of Queensland, such percentages of runoff would be expected only during a major flood. The hydrological

efficiency of the Beauly River basin must result from a favourable combination of the following factors: a large rainfall, well distributed through the year; moderately high altitude; and comparatively high latitude. The evapo-transpiration loss was only 12–14 in. per year. Temperature was not given in the Paper but, assuming that the figures in Table I referred to the natural conditions prior to the filling of the reservoirs, the losses were consistent with an average annual temperature of 45°F at sea level and as low as 40°F for the Monar catchment.

Mr R. J. T. Casinader (Binnie & Partners, Kuala Lumpur, Malaya) wrote that he had read the sections of the Paper dealing with the Monar arch dam with considerable interest, since he had been associated with the design of this structure as a member of the staff of one of the Authors. The details given in the Paper of the foundation difficulties encountered during construction and their treatment as well as the measurements of the behaviour of the dam served to emphasize that the most interesting and difficult part of the engineering of an arch dam was not always the structural design.

121. Reference was made in § 8 to the use of a scientific method to determine the effects of different shapes of dam. The design was 'free' in the sense that no limitations on shape were arbitrarily imposed such as the upstream face having to be vertical and cylindrical. An examination of published literature showed that the number of shapes of arch dam adopted in the past had only been exceeded by the number of different types of analysis proposed by various writers. There was very little guidance in the literature on how to derive the best shape of dam for any shape of valley. One suggestion was to make a trial design based on 'independent arch' theory and to improve this progressively by more accurate methods of analysis. When a trial design based on independent arch methods was attempted for the Monar dam the thickness estimated for the arch near the base of the dam was discouragingly large. This was chiefly due to the fact that with a fairly high chord/height ratio the central angles near the base had to be quite small to ensure that vertical sections would be stable during construction. On independent arch theory the water load near the base was taken entirely by arch action and with small central angles the thickness had to be large.

122. It soon became apparent that the design had to be conceived from the start as a 'monolithic' structure. To facilitate this the concept of an interconnected system of arches and cantilevers was adopted. In accordance with this design concept, and as stated in § 9, the simplified arch-cantilever method attributed to Max Ritter and published by Jaeger was selected for analysis of the various designs, the later designs being checked by model tests. The Max Ritter method could be, and had been, criticized on a number of counts but it had the great merits of (a) being relatively quick to use and (b) preserving in the designer's mind a physical concept of how various shapes of dam would behave. A full 'trial-load' arch-cantilever analysis would undoubtedly have been more accurate but the time taken for each analysis would not have justified its use in developing the design. With other methods such as relaxation analysis the designer had no physical concept or 'feel' of how design modifications would affect the behaviour of the dam.

123. The arch-cantilever concept used for the Monar dam resulted in progressive modifications of the design culminating in the doubly curved design which was finally adopted. In order that reasonably large central angles for the horizontal (arches) near the base of the dam could be obtained, it appeared at first that the crown vertical section should lean downstream whereas the abutment vertical sections should lean upstream. However, the arch-cantilever concept had indicated that it was essential to develop as much dead load compression as possible at the heel of the central 'cantilevers' and this led logically to the type of vertical profile shown in Fig. 9. The lack of strength in the lower 'arches' would not matter too much because the degree of fixity of the 'cantilever' at this level meant the load was transferred to the higher 'arches' which, because of their larger curvature, were quite strong. So that this

effect could be increased, the thickness at the base of the central 'cantilever' was reduced to a minimum consistent with stability, watertightness and shear strength, and the upper 'arches' were strengthened to take the increased load. The arch-cantilever concept had also been very useful in determining what would happen if a crack developed at the base of the central cantilevers due to the high tensile stresses which were shown by the theory to exist there. As stated in § 33 the calculations had shown an acceptable result; the cracking of the 'cantilever' base would throw additional load on to the higher arches which were of adequate strength to take it and the crack would be arrested quickly.

124. The writer feels, therefore, that the arch-cantilever concept together with the Max Ritter method were very useful tools in developing the design of the dam. He asked if the Authors would adopt these methods again under similar circumstances.

125. Figs 8 and 9 show that there were considerable differences in the stress patterns given by the theoretical method and by model tests. Although the order of maximum compressive stresses given by the two methods was comparable especially in the centre of the upstream face, the tensile stresses given by the theoretical method, and in particular those at the base of the central cantilevers, were higher than those given by the model. Would the Authors please comment on the possibility that some relief of tension by cracking had already taken place in the model at the time strains were measured? It was sometimes suggested that model tests gave an accurate representation of the stresses in the prototype dam. The writer would suggest that test results from monolithic models should be treated with as much caution as those from arch-cantilever analyses. The prototype dam, being intersected by potential planes of weakness at vertical contraction joints and horizontal lift joints, was far from being monolithic in tension. Furthermore, the effect of the sequence of construction on gravity stresses and the time of grouting the contraction joints on the water load stresses could have a great bearing on the prototype stresses and were difficult to assess from tests on monolithic models.

126. The ultimate answer to the arguments regarding the best design method to use could only be given by measurements on prototype dams. It was hoped therefore that despite the difficulties mentioned in § 27 the Authors or their client will in due course publish an interpretation of the measurements on Monar dam.

127. In § 5 the Authors suggested that it would be desirable to excavate the foundations before designing an arch dam. This would appear to be a very stringent requirement and it would break the 'rhythm' of construction referred to in § 39. Would the Authors agree that an intensive site investigation programme might be designed to give the required information both to determine the proper foundation line for the dam and to assess the extent of rock fortification required? It would be very useful if the Authors would suggest, as a result of their experience at Monar, an adequate site investigation programme for similar dams.

128. The writer would be grateful if the Authors would amplify their comments on the grouting of the contraction joints by giving the relative grout intakes during primary and secondary grouting and by stating the observed deflexions of the blocks during grouting and on what basis these were restricted by limiting grout pressures.

The Authors, in reply, said that Mr Dickerson had given some reasons why the Aigas and Kilmorack section had been developed in two stages instead of the one stage planned in the earlier Grampian proposals. There were some reasons other than those he mentioned, such as the Board's anxiety to interfere as little as possible with good agricultural land.

130. Mr Coward had referred in § 98 to the arrangements made for fish in the design of the Aigas and Kilmorack dams. In fact the necessity for making provision for the passage of migratory fish had a strong influence on the layout of these dams, which incorporated the power stations with their large turbine intakes and ancillary works, the floodgates and culverts, as well as the fishlocks.

131. The maximum width of the river at the two sites was 300 ft and 420 ft respectively and, with a flood of 40 000 cusec to be discharged, the natural difficulties were increased by the stringent measures needed to encourage the passage of fish. The final layout of the dams had evolved from many preliminary designs and discussions with the fishery advisers of the North of Scotland Hydro-Electric Board and resulted from the following premises.

- (a) Both the upstream and downstream faces of the dam should taper horizontally as far as possible uniformly across the river, forming a symmetrical lead to the fishpass located in the apex of the angle so formed.
- (b) The intakes to the machines should be disposed symmetrically on either side of the fishlock.
- (c) The flood gates should also be disposed symmetrically on either side of the fishlock.

132. These measures were of undoubted benefit to the fish, but the resulting layout (Fig. 26(a) and (b)) was not the most economical in space and materials; it had complicated the construction and made it necessary to have a bridge over one of the spillways to obtain access to the power station; also flotsam was funnelled along the upstream faces of the dams, introducing a screening problem.

133. At both these sites, temporary fishpasses had to be formed upstream of the dams during construction. Further upstream, just above the tailrace from the Culligran power station, pools and channels had to be cut in the Culligran Falls and a better passage was provided for the fish to enable them to cope with the restricted flow. This was a good example of a permanent fishpass in a natural setting (Fig. 35).



FIG. 35: CULLIGRAN FALLS FISHPASS

134. Mr Dickerson had mentioned (§ 79) a very real short term disadvantage of the thin arch dam in that it was not structurally complete until the contraction joints had been finally grouted at the minimum operating concrete temperature. This in effect restricted the impounded water level to the level below which the dam could be considered to act as a gravity section. There seemed to be no way around this restriction, since the joints must be grouted up at the coldest period of the year irrespective of the operator's requirements for storing water because they might not be sufficiently open at higher temperatures to allow the penetration of grout. The difficulty could be overcome by leaving narrow gaps between the main blocks to be filled with concrete when most of the shrinkage of the main blocks had taken place and in that case the design should allow for the temperature and shrinkage strains expected after grouting. It might be noted that the problem is accentuated for a small dam because the overall time available for grouting in stages is less than in the case of a high dam. The grouting had been done at Monar dam when the temperature of the concrete was about 32°F. Efforts to grout at 2F° below freezing were unsuccessful because the grout froze in the joint.

135. Mr Casinader requested further information on the actual grouting as carried out. The cement used in primary grouting of contraction joints amounted to 213 cwt and, in secondary grouting, to 106 cwt in 24 bays. In addition 17 bays were grouted once only, taking a further 177 cwt. The observed deflexions during grouting of a block did not exceed 1.5 mm. The grout pressures were restricted to limit the theoretical tensile stresses which would be developed assuming that the joints, even though grouted, would not resist shear. The grout pressure at the top of bays being grouted was therefore limited to 25 lb/sq. in. and was reduced to 10 lb/sq. in. at the top of the critical bay near the top of the central block. During grouting strains and joint openings were monitored to confirm that no excessive forces were being built up.

136. To attempt to obtain an increase in cantilever strength by thickening the base, as suggested by Mr Dickerson in § 79, was merely to seek an acceptable compromise between a true gravity section and an arch—an arch gravity—of which a number had been built. In that case there could be some tolerance in the season in which the joints might be grouted. However, thickening of the base of the cantilevers of an arch dam caused rigidity which might be undesirable if it increased the vertical component of the bending moment at the base excessively and reduced the contribution which was required of the arches higher up.

137. The arch dam at Chliostair in Harris illustrated by Mr Dickerson (Fig. 33) had the distinction of being the first structural arch dam built in Britain and he was to be congratulated on the excellent job which he and his staff had made of it. When the Authors had examined it recently there had been absolutely no sign of seepage through the lift joints, a standard which was not often attained with comparatively thin concrete walls. This had been attributed to the successful use of PVC water stops in each lift joint. The rock on that site was not very strong and the measures adopted to eliminate erosion by water spilling over the crest were certainly justified.

138. In reply to Mr Dickerson's query in § 81, further design calculations for Monar dam were not contemplated unless called for in the interests of research. The work of Dr Davies at Edinburgh had not been published when the calculations were made for Monar dam, but in the general sense it was hoped that the work of the Committee on the Design of Arch Dams would indicate the most promising design methods in given cases.

139. Mr Cuthbertson had asked in § 90 whether the load on the crest of Monar dam arising from the crest gates had imposed any significant design difficulty. The Authors said that no such difficulty had been experienced; a stiff crest section had been avoided by leaving the contraction joints ungrouted above 727.5 ft O.D. and it was therefore assumed that the loads above this imposed by the gates were transferred to the lower sections by cantilever action.

140. The development of the design of Monar dam had been due to a large extent to the excellent work carried out by Mr Casinader, and his comments were particularly welcome. His question on whether the Max Ritter method would be adopted again in similar circumstances was, however, somewhat rhetorical. The Authors considered that it was a very suitable method in the case of Monar dam, having regard to the size of the dam and the period when it was designed. However, design techniques had developed in recent years, and circumstances would never be similar again. The Committee of the Institution of Civil Engineers on the Design of Arch Dams had attempted a comprehensive assessment of design methods and it was hoped that its final report would give valuable guidance on this difficult question.

141. The possibility suggested by Mr Casinader in § 125 that tensions in the model arch may have been relieved by cracking could not be proved, but it was considered by the Authors to provide one explanation of divergencies between the stress patterns obtained by model and by theory. Due to the loading plates on the upstream face of the model, fine cracks could easily have passed unnoticed, and this aspect of model investigations deserved careful consideration.

142. Many questions revolved around what might be called the 'uncertainty factor' of a site, i.e. the extent to which it could be reduced by site exploration particularly in relation to arch dams and the reasonable allowances which should be made for contingencies in the estimates. The site investigation for any structure founded on rock had two main objects—to determine the position of rock head, and to determine the natural characteristics of the rock to a depth influenced by the structure (including the range of variation of these properties). The basic cross section of a concrete gravity dam was not affected by the position of sound rock head in a vertical plane, and this was also the case for a buttress dam. These structures merely altered uniformly in horizontal upstream-downstream dimension as the depth varied. They consisted in fact of independent blocks which did not rely on support from adjacent blocks. The horizontal arch dam was quite different and any unexpected variation in depth or in symmetry of foundations or in the strength of the foundation rock could quickly affect the shape and thickness of the whole arch. This accounted for the strong preference expressed in the Paper for the whole of the foundation of an arch dam being excavated before any concrete was placed.

143. Mr Paton, Mr Cuthbertson and Mr Casinader asked what effect such a stringent condition would have on the 'rhythm' of construction. Obviously, the 'rhythm' would be slower and the overall time for construction longer. The Authors considered that was one of the factors to be taken into account when an arch dam was selected as opposed to any other type of dam. The normal type of site investigation was not sufficient to establish the design of an arch before excavation was carried out, or even to establish whether an arch would be the most economical choice.

144. In the light of these observations, the Authors felt that at the stage when the only information available concerning a site comprised the surface indications (sample borings to a depth at least equal to the height of the dam and geophysical surveys), a greater allowance should still be made for contingencies when the estimated cost of a thin arch dam was compared with, for instance, a gravity structure where the normal 10% allowance would be considered reasonable.

145. Mr Chapman's query as to what further investigations, other than more extensive drilling upstream and downstream, should have been carried out at Monar dam site highlighted the whole problem of site investigations, particularly on a hydro-electric scheme which, by its very nature, was more intimately concerned with the topography and geology of the country than most constructional projects. It might be that, in time, geophysical methods would be able to solve all the problems but until then it was necessary to rely on borings and to refuse to take anything for granted. If this attitude was applied at all stages while the layout was taking shape on the drawing board, then it would always be guided by the existing borehole results and, if there were any loose ends or doubts, a further series of borings or other investigations

should be undertaken in an effort to ensure that these dubious points are not going to affect the layout adversely. In most cases this could mean delay before starting work, but, if that procedure was not feasible, the possible consequences in additional cost should be accepted as being worth the risk.

146. However, nothing but luck, and a constant awareness of possible trouble as was, in fact, the case, would have found the dyke at Monar since it had proved impossible to detect such intrusions except by surface features or by good fortune in the location of boreholes. This dyke was a good example of how cunningly concealed such a feature could be. In fact, had it not been necessary to construct a new road on each side of the dam above Top Water Level it was possible that this important intrusion would never have been found at all. That it had been found was due to the steady routine monitoring of excavations by geologists of the Geological Survey coupled with a fine piece of field geology by a member of the Resident Engineer's staff. Such wideawake surveillance was obviously essential for important dams during construction.

147. Mr Hamilton had wanted to know what further investigations were carried out at Monar dam between 1952, when the initial borings had been put down, and the exposure of rock in the excavations in 1961, which had occasioned the calling in of Professor Skempton and his colleagues. It was not, of course, the exposure of the rock which led to Professor Skempton's investigations, but the discovery of the dyke. When the dam was being planned in the late 1950's there had been no doubts about the foundations because the original investigations had shown the rock to be extremely strong. Consideration had been given to further exploration of the foundations prior to construction, but it had been decided that it would be preferable to go ahead with the excavation to disclose the surface conditions and to adjust the depth of the foundations as might be necessary. Provision had therefore been made in the contract for the exploratory headings to be driven into the two abutments, for the rock to be exposed in the excavations as quickly as possible, and for special measures should a fault be uncovered.

148. As regards Mr Hamilton's query as to what action was taken to test the effectiveness of the foundation grouting, the borings put down for investigation of the lamprophyre and referred to in § 44 had also enabled the grouting to be observed. Some very good cores had been obtained which, on inspection, had showed that the grouting had been satisfactory.

149. The majority of intrusions which had been found in Strathfarrar were of essentially weak and impermanent materials, the ferro-magnesium silicates found in lamprophyres were a prime example. They constituted a plane of weakness where major differential rock movements had generally taken place. These movements had led to decomposition of the minerals into clays. Mr Hamilton had asked for information regarding techniques employed to ensure that clay fissures had been made watertight. The technique employed at Monar, to replace a clay in a nearly vertical seam by grout had consisted of drilling a large number of vertical holes at about 12 in. centres into the seam and flushing out with water until the return water was completely clear. Each hole had then been grouted. Systematic recording and plotting of the drilling and grouting data in association with regular tests under water pressure had enabled the effectiveness of the grouting to be assessed. Obviously there were other techniques for grouting clay seams employed by various specialist contractors. At Chliostair dam, referred to by Mr Dickerson, the clay seams were almost horizontal and had been washed out and grouted effectively in a similar manner to those at Monar.

150. Mr Chapman and Mr Harries questioned whether, if the location of the dyke had been known in advance, Monar dam would have been built further downstream or whether an arch would have been chosen in preference to a gravity dam in that case. The problem was to build a dam near the outlet of Loch Monar and, while the chosen site would have been avoided had the presence of the dyke been

known, to have gone either upstream or downstream would have meant the loss of the valley shoulder on the left bank, thus precluding an arch dam; there would therefore have been no choice. On the other hand, if circumstances had forced the adoption of the present site in spite of the dyke, the same foundation difficulties would have existed whatever the type of dam and the additional expense which arose of grouting, culvert extensions, upstream cofferdam, etc., would also have remained. There was no reason, therefore, to assume that a gravity dam would have been any cheaper and it was possible that an arch dam would still have been chosen.

151. Mr Hamilton had raised some questions on costs in § 110. The total cost of the site investigations at Monar dam had been just over £40 000. It was not possible to compare the relative cost of the dam as constructed with a gravity dam which would have been constructed under similar circumstances, since it was impossible to estimate the cost of the latter with sufficient accuracy.

152. The data which had been collected from the gauges in the dam since the publication of the Paper showed that the behaviour of the dam had continued to develop in a consistent manner, and there had been no significant opening of the horizontal lift joint at the upstream heel of the dam referred to in § 23. If a crack had been going to develop, this was the joint where it would have occurred, since it was the point of maximum tension according to both the model tests and the analyses. The results from gauges isolated from load indicated that, in some cases, considerable stresses were built into the concrete while it was still young, and the Authors felt that a great deal of research was still required to discover the manner in which stresses were established due to internal or external restraints on the concrete during the critical period, in this case about three months after placing, when concrete was placed in the lifts above the gauge. The evidence obtained at Monar seemed to confirm that such stresses might be of the same order as the working stresses for which concrete was conventionally designed. Fig. 36 was an extension of Fig. 7 and showed the vertical and horizontal strain measurements in the south abutment at level 664.0 ft O.D. Unexpectedly high compressive stresses, which varied in extent in different locations, appeared to have been built up near the upstream face during the first year in the manner referred to above.

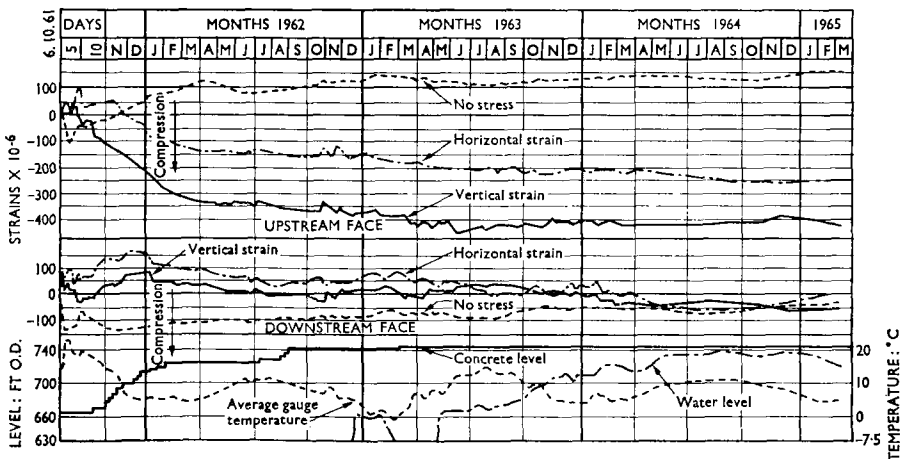


FIG. 36: VERTICAL AND HORIZONTAL STRAIN MEASUREMENTS AT LEVEL 664 FT O.D. NEAR THE SOUTH ABUTMENT

153. Mr Harries had asked whether additional strain gauges would have been worthwhile and wished to know the purpose of the strain gauges embedded in the concrete. The Authors considered that it would have been of value to have the additional strain gauges mentioned in § 32. In order to calculate stresses from the measured strains it was also necessary to measure the elastic and time dependent properties of the concrete by long term laboratory tests. It was hoped that the concrete as used in Monar dam could be tested in this way, in order that as much knowledge as possible could be gained from the observations. Nevertheless it should be appreciated that the financing of investigations posed a problem for what was after all only a relatively small dam in international terms. Recently it had become possible to pursue these investigations through the interest of the Civil Engineering Research Association.

154. Mr Campbell's brief account of strain measurements at other sites was very interesting, but the Authors did not agree with his conclusion that the absolute value of strains measured could not be taken as having anything to do with the stresses in the dam. In the present state of knowledge, if there was no definite evidence to the contrary, it seemed safer to accept the results of apparently reliable strain gauges rather than to assume that the concrete under observation was behaving differently from the indications given by the gauges. A great deal more information was required, however, about the coefficient of thermal expansion and the modulus of deformation of concrete at early ages. This was most difficult to obtain because of the number of variables involved, such as shrinkage, moisture content and temperature.

155. It was correct that the no-stress gauges in Fig. 7 generally showed expansions after 10 days (see § 114). Unfortunately the no-stress gauge itself was an imperfect representation of conditions in an unloaded mass, since it was perforce artificially divided from the surrounding structure except on one side, and the moisture content of its encasing concrete could be influenced by the division. The statement in § 114 that the degree of expansion and shrinkage of a particular concrete was a function of the season of the year in which the concrete was placed was undoubtedly true, and helped to explain why a predictable pattern could not be found in the observations of the six no-stress gauges, as mentioned in § 32.

156. Fig. 37 was an extension of Fig. 10 showing joint movement on either side of Block No. 8. An opening of a few thousandths of an inch had been noticeable during the winter of 1964/65, particularly at the upper level, despite high reservoir levels. Fig. 38 was an extension of Fig. 13 and showed the observed movement of the crest at the crown. The overall movement had been 0.8 in. (20 mm) at 750.0 ft O.D. of which about half was attributable to water load from a reservoir level 7 ft below the designed level (746.0 ft O.D.) and the other half to a temperature change of about 22°F (12°C). This would be consistent with an overall modulus of deformation (made up of creep, elasticity, etc., since they cannot be separated) for both rock and concrete of about 3 000 000 lb/sq. in.

157. Fig. 39 was an extension of Fig. 14 and showed horizontal movements of surface rock measured by the inverted plumb bobs. Movements had not exceeded 0.08 in. or 2 mm. The movements of the surface rock and the reservoir level could be compared, but did not conform superficially. However, movements did generally conform with the crest deflexions, resulting from temperature changes as well as water load, as shown in Fig. 38. These two figures showed quite good agreement in trends. The results shown in Fig. 39 had been obtained from the two centre plumb bobs, the one in Block 6 being anchored below the lamproschist dyke; results from the other two plumb bobs at the abutments had not been available long enough to show any definite trends. In Fig. 14 there were pronounced movements or apparent movements. There was no obvious explanation for these and it might, perhaps, have been due to human error in taking the reading or to a change of reader.

158. The Authors wished to endorse Mr Campbell's statement that one should

look to the behaviour of the finished dam, despite the difficulties, in order that the safety of the structure could ultimately be known. They also appreciated Mr Casinader's wish that an interpretation of the measurements on Monar dam might be published, and trusted that this would be possible in time. Meanwhile it was hoped that the full results of observations would be made available by arrangements which were at present under discussion.

159. Several questions had been raised on design floods and methods to deal with them. Mr Cuthbertson and Mr Hamilton had queried the ratio of normal maximum to catastrophic floods at Aigas and Kilmorack. While the Authors' normal practice was to take that ratio at 1 to 2, the effect of attenuation due to one, or several, reservoirs in series, all full at the inception of a flood, would be to slightly increase this ratio. However, at Aigas and Kilmorack dams, which were preceded by four reservoirs, account had been taken of probabilities which would decrease this ratio. This was because the improbability of all four reservoirs being simultaneously full would have had a greater effect when combined with the small probability of a catastrophic flood than with the greater probability of a normal maximum flood.

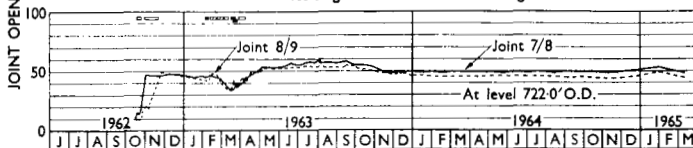
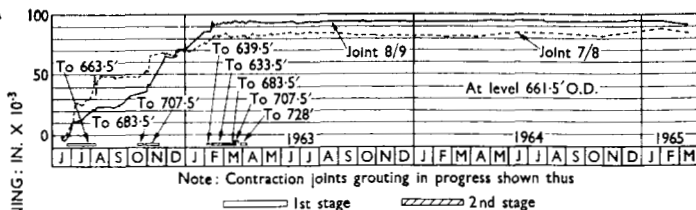


FIG. 37: JOINT MOVEMENT ON EITHER SIDE OF BLOCK 8

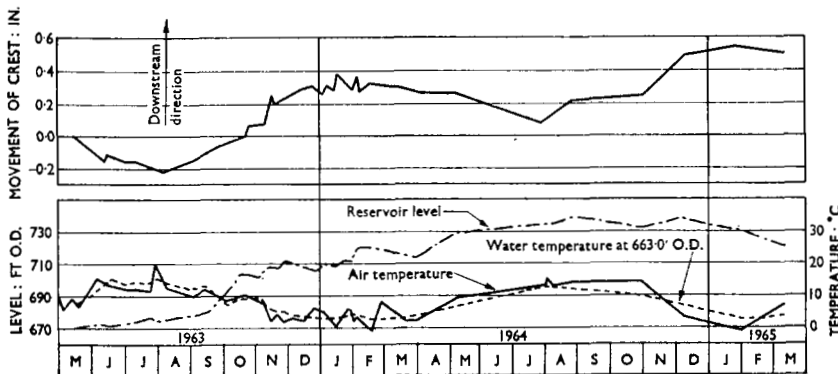


FIG. 38: OBSERVED DEFLEXIONS OF THE CREST

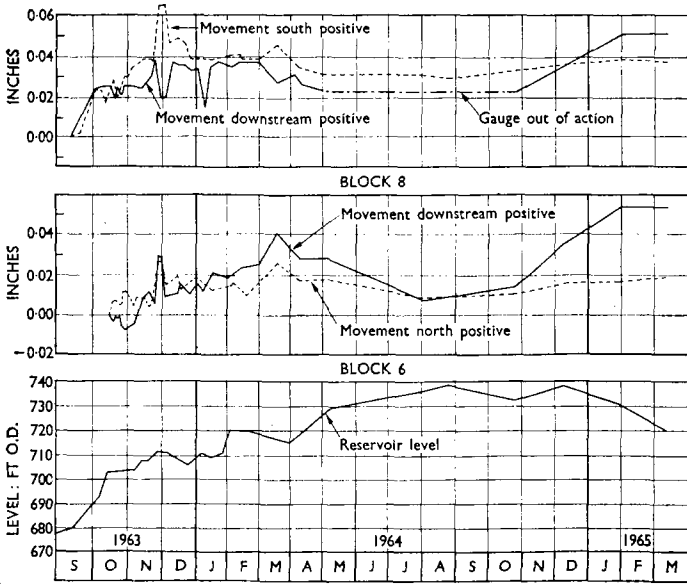


FIG. 39: HORIZONTAL MOVEMENTS OF SURFACE ROCK MEASURED BY INVERTED PLUMB BOBS

The reasoning had only applied qualitatively, not being governed by any calculation, and gave the ratio of 1 to 1.3.

160. Mr Chapman had queried the reason for the very small flood rise in Monar reservoir which necessitated the use of gates. A limiting level had been dictated by the necessity of avoiding the flooding of a property at the upstream end of Loch Monar and it had been in the interests of the North of Scotland Hydro-Electric Board to have an operating level, or T.W.L., which would be below this limiting level by as small a margin as possible; hence the adoption of a gated spillway. As Mr Chapman had said, a fixed spillweir would have caused a flood rise only about 3 ft higher but that extra 3 ft of head and storage had made the gates worthwhile. Generally, however, it is preferable to have a free spillweir, if only to take a part of the flood. It was particularly desirable, although it had not been possible at Beannachran, Aigas or Kilmorack dams, to provide free spillweir capacity as an adjunct to electrical or mechanical gates, as suggested by Mr Hamilton in § 106, who was correct in assuming that no reliance was placed on passing part of the flood through the machines.

161. Mr Hamilton also referred to the Authors' world-wide investigation on gates. The investigation did indeed deal with flood gates, or rather spillway gates, and not only drum or tilting gates but also other types of water operated and electrically powered gates. The Authors' precise statement on the result of the investigation was that there had been no report of failure of an automatic device. In fact it was precisely in order to find out the reliability of automation and to find out how much faith the designers had in it, that the investigation was conducted. Of the total 806 gates covered by the replies received, 634 were non-automatic, 80 were automatic-electrically powered and 92 were automatic-water powered. In the first category 1.6% were said to be unsatisfactory, in the second category 59% and in the third 21%.

'Unsatisfactory' in the latter two categories did not necessarily mean failure to operate and, in fact, no failures had been reported. In most cases, however, it meant that, having installed automated equipment, the operators did not have sufficient confidence in it to allow it to work fully automatically.

162. The enquiry had been inconclusive on the relative reliability of automatic-electric power and automatic-water power. However, the Authors felt that in a remote installation which might be dormant for a long period, the latter was more reliable than the former. This opinion arose from the fact that, with water operation, an initial failure of the mechanism to work when the water level was rising caused the actuating force to go on increasing until it overcame the impediment; a dead electrical circuit, on the other hand, remained dead however great the force acting on the switch and, if the site was remote, it was difficult to ensure that attendants would be alerted in time to intervene and correct the fault. Furthermore, the remoteness had made the electricity supply itself less than fully reliable. For these reasons water-powered gates had been chosen for the most remote site, Monar dam, and a mixture of water-powered and electrically-powered gates for the less remote dams.

163. Mr Cuthbertson had asked what criterion had been adopted at each dam to cope with a failure of the gates. No measures had been taken to dispose of floods in an orderly fashion in that eventuality; this meant that the flood waters would pass over the top of the gates, over the top of the non-spillweir part of the dam and might swamp items of machinery not intended to be submerged. The higher level reached in the reservoir as a result might also flood properties intended to be kept out of the flooded area. All these effects would result in material damage and inconvenience, the risk of which was sufficiently remote to be acceptable. What could not be accepted, however, was the risk of destruction to the dam itself and for this reason all calculations of the structural stability of the dams had been made on the assumption that the gates had failed to open.

164. Having chosen water-powered gates for Monar dam, the types of gate within this category had to be considered and tilting gates were chosen. This choice had been questioned by Mr Chapman and Mr Cuthbertson. Manufacturers had been asked to offer the type they thought best suited to the conditions. The principal factor in the choice of tilting gates, compared with, say, drum gates, had been the necessity to reduce to a minimum the size of chambers and other features as the gates had to be accommodated in the comparatively narrow crest of an arch dam. The Authors had had no previous experience of this type of gate but they had been satisfied by enquiries from people who had. In answer to Mr Cuthbertson's question in § 90 access was available into the chamber behind the lower part of the gate leaf through a hatch in the gate leaf itself.

165. Mr Marshall had also asked about the gates on all the dams. In accordance with their usual practice the Authors had specified the required function of the gates and the design criteria, and left the suppliers to propose details of type and manufacture; in checking the design, materials and workmanship put forward, they had also been concerned with ensuring that the cost of the associated civil engineering works was kept to a minimum. There had not been a sizeable flood since the gates had been in operation but from tests of operation at normal water levels and by simulation of flood levels in the float wells to test the automatic devices, the Authors were satisfied that the gates functioned satisfactorily. Fig. 40 showed the centre gates spilling under test. The nap of water was just clearing the valve house at the bottom. Mr Marshall had also raised the point that gates placed in a narrow channel tended to hunt, but no such tendency had been observed at Aigas and Kilmorack. The conditions, however, were thought to be very close to critical, and experiments were currently in progress to determine the exact channel surge effect and to check that the gate opening sequence at present in force was stable in all circumstances.

166. Mr Marshall had expressed concern that so much gate construction had been done by foreign manufacturers and he had wondered why this was so. Perhaps



FIG. 40: CENTRE GATES SPILLING AT MONAR DAM

the answer lay in his own remark concerning work overseas; in order to expand work overseas British firms had to accept foreign competition at home. The offers from foreign firms had been of equal technical merit to British offers and had been accepted purely on the basis of cost; the offers had provided for about two thirds of the manufacture to be carried out in Scotland.

167. Mr Coward had not agreed that the sector and drum gates in the three lower dams had been intended to deal with daily variations in level, but had thought that they had been intended to pass small floods and maintain a reasonably constant level. The Authors had stated the assumptions made for the original design of the gates. However, it was gratifying to note that the machine operation was capable of dealing with normal daily variations of flow and that the sector and drum gates only operated on the development of minor floods. As mentioned by Mr Coward in § 98 the operation of the lower fishlock gate had been restricted to unbalanced heads of 20 ft or less and this was for the supposed comfort of the fish. The switch for the gate was governed by pressure control.

168. Mr Nimmo had made a very good estimate of the average annual temperature; in fact it was 45°F at Monar. The Authors agreed that this temperature, coupled with the other factors mentioned by Mr Nimmo, led to the small evapo-transpiration loss and high proportion of runoff.

169. Mr Coward had asked if a single Deriaz type of turbine could have been adopted for the Deanie Power Station. The average static head was about 350 ft and the variation 10%. Whilst this, and an ability to accept considerable variation in draw-off, brought to mind the appropriate characteristics of the Kaplan or Deriaz

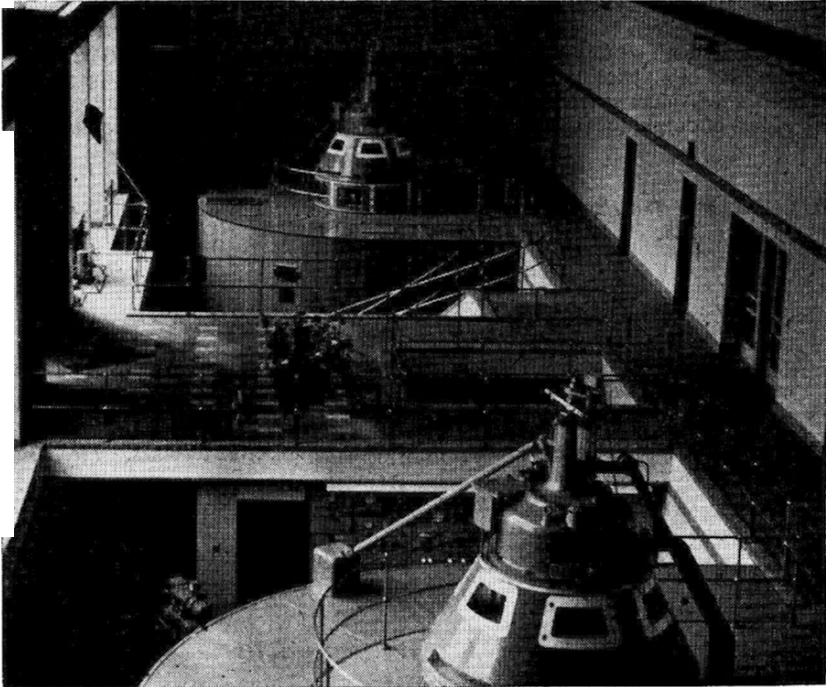


FIG. 41: INTERIOR OF KILMORACK POWER STATION

designs, the head was considerably in excess of that at which the only Deriaz machines in existence in 1959 were operating. The best solution at that time had therefore been to adopt a Francis type of turbine. Two machines had been installed both of which could be satisfactorily operated down to half the full load in order to provide operating flexibility and to cater for the required variation in discharge. The specific speed of the Deanie machines is 50, i.e. towards the upper end of the speed range for Francis turbines at this head.

170. The Authors wished to add that for the lower head station at Culligran it had originally been intended to install a single 22 MW machine with a Kaplan turbine. This had subsequently been changed to a Deriaz turbine at no extra cost and with marginally greater efficiency. With a design net head of 180 ft this was probably, at the time, the highest head for which a Deriaz turbine had been built. The Board's decision to install the Deriaz turbine had been in accordance with its desire to assist British manufacturers in trying out a new design of turbine whenever appropriate.

171. Mr Coward had been quite correct when he referred in § 100 to the advantage of using a common oil system for bearing lubrication and governor operation. The type of oil distributor head which had been dispensed with at Culligran could be seen in Fig. 41, which was a general view of the Kaplan machines at Kilmorack Power Station.

172. Mr Coward had also referred to the effectiveness of centralized control of the whole Strathfarrar Project including water flows and storage, the power generated and its transmission into the main electrical network. This co-ordinated control was centralized on a Strathfarrar machines control desk (see Fig. 31) together with hydraulic and electrical indications panels.

173. It had been reassuring to note in § 103 that the simplified transmission arrangements both on this scheme and on others in the North of Scotland had been fully justified in practice by their reliability since commissioning. With the greater number of individual generating stations it was possible to accept the slightly greater risk of loss of output for the benefit of minimum cost of transmission equipment.

REFERENCE

10. LEA F. M. *The chemistry of cement and concrete*. Edward Arnold, London, 1956, p. 225.

Corrigenda

Fig. 7 and § 30: For 'north' abutment read 'south' abutment.

Fig. 10 and § 34: For 'central block' read 'Block 8'.

Fig. 13: Negative readings indicate deflexions upstream and positive readings indicate deflexions downstream.

Page 486, reference 9 should read as follows:

9. P. DERIAZ. Comparative study of Kaplan and Deriaz turbines. *Electrica Review*, Nov. 1959.