*Discussion on Paper No. 6984**

Some design and construction features of the Cruachan pumped storage project

by

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Mr C. D. Crosthwaite (Freeman, Fox & Partners) said that Cruachan was owed to the genius of the late James Williamson. The Cruachan scheme depended entirely on the availability of an upper reservoir site which did not appear on ordnance survey maps at all. The discovery of the site was due entirely to James Williamson.

185. The Authors did not speak of anything except the civil side; they had not dealt with the economics or the machine part. It was, however, mentioned that there had been no fewer than 43 bids for alternative vertical reversible pump turbine machines. This was quite astonishing when one realized that only a few years ago there would not have been any at all.

186. Concerning the layout, he would have thought (the Authors did not say so explicitly) that one of the main reasons for preferring B and C to A would have been to reduce the length of the penstocks in favour of an unlined tail race tunnel.

187. With regard to § 30, in view of the difficulties the Authors experienced in driving and lining the inclined shafts, would they now have preferred a scheme with vertical shafts?

188. He endorsed the decision to build a massive buttress dam for this site. This again was owed to Williamson. It was interesting to note the Levy factor being quoted as a design criterion in § 34. He had never been so hostile to the Levy factor as some dam designers appeared to be, believing that the main risk of uplift in concrete dams arose at the lift lines and that the Levy factor had some meaning in this connexion.

189. It had been stated that at the time of design very little was known about stresses in dams subject to rapid rise and fall in water levels. This was perfectly true then but by 1964 quite a bit more was known. Fraser Campbell and Crosthwaite presented a Paper at the 1964 ICOLD Congress dealing with Ffestiniog, where it was established that stresses and strains due to water pressure were very close indeed on rise and fall to those that calculated by classical methods, and he would wait with interest to see whether there was confirmation of this when Cruachan was in full operation.

190. The designers had used very elaborate methods to look after leakage through the contraction slots and followed these up with a drain to take away the leakage. This was good, because leakage at contraction slots was structurally of no great significance but it was very unsightly and could give cause for alarm. Much more important was the leakage from horizontal construction joints. To what extent did the Authors succeed in eliminating leakage from horizontal construction joints and, if so, what did they do about joint preparation?

191. He was surprised at the close pattern of foundation grouting in view of the statement that the fissuring of the rock only went down a short way. Could this not have been opened out quite a bit? On the other hand, the depth of the holes might have been a bit more and the pressures a bit higher. There was general agreement in

* *Proc. Instn civ. Engrs*, 1966, **35** (Nov.) 407–450.

this field; it was just a matter of experience of some other site related to the present one. The engineer responsible for the dam must take a personal interest in grouting the whole time, right from the beginning.

192. Quite a lot had been said about stress moduli and Poisson's ratio, etc. To what extent did the Authors consider that the very marked difference between the stresses which were disclosed by the Swedish tests (three or four times what they were estimated to be from the natural weight of rock above) was due to under-estimation or to the rather natural fact that one could expect to get much higher stresses at the boundary of underground rock caverns? It was not too clear how they used these stresses and made use of the moduli and the Poisson's ratios in their design. He had also noted that the moduli the Authors established seemed to go right out from the range of what was normally regarded as non-elastic rock into the middle of the range of elastic rock, and a Poisson's ratio of 0·13–0·22 seemed to be on the low side for a diorite.

193. He had been very struck by the extraordinary complication of the underground chambers. Without knowing a lot about the relative prices of chamber and tunnel excavation, one wondered whether some simplified scheme with less tunnel and more chamber might not have proved to be cheaper in the end. The ventilation was appalling in 1963 and it was most surprising that nobody, apparently, had suffered under those conditions from the diesel fumes.

194. In § 74 the Authors referred to the Alimak and stated how much faster it would have been with an electric drive. The Swedes had used one, as the Paper explained, and they had every bit as good safety records as we had, so there would seem to be no reason for not following their example.

195. He presumed that the pattern of rock bolting was decided by the engineers as part of their design and not left to the contractor.

196. He had been surprised to learn that the very small amount of bentonite grout was supposed to have assisted in the grouting around the pressure shafts.

197. Finally, had the Authors used pulverized fuel ash in the concrete? It would appear not.

Mr J. C. A. Roseveare (Freeman, Fox & Partners) said that the Paper had created tremendous interest. It was pleasing to see a Paper of good length before the Institution but as always there were things not contained in it that one would like to see. Could it not be considered that a work of this stature should justify a series of Papers, as there were so many things to be said?

199. It was of particular importance because in this country today the field for hydroelectric work now lay almost exclusively in pumped storage. Very high efficiency thermal generation was now possible and there were great reductions in nuclear generation costs, so that a very hard battle was being fought and anyone at present engaged in this sort of work, or likely to be in the future, needed all the ammunition he could get to bring down the cost per kilowatt of hydroelectric work.

200. The Consulting Engineers must know by now roughly what the final estimated cost of the scheme would be and this would be of great interest to everybody. He had first shown Tables 2 and 3 at the 1964 World Power Conference in Lausanne.

201. At the same Lausanne Meeting he had detailed the losses in hydroelectric stations as far as these could be determined from published information. There were many blanks in the information for 'Cruachan' and it would be interesting if the Authors could fill in the missing figures. It would also be interesting to have more information on the design of the conduit system which, in these schemes, accounted for such a big proportion of the total Civil Engineering cost. The conduit losses for Cruachan were stated to be very low, an estimated 1·3%. Was this figure applicable to the final design and what proportion of the total loss occurred on the pressure side of the machines? With the operation of the machines, had it been possible to confirm these figures.

TABLE 2: COST OF PUMPED STORAGE SCHEMES

	MW	£ sterling/kW
Cornwall (U.S.A.) . . .	1800	30·4*
Cruachan (G.B.) . . .	400	36·4*
Vianden (Lux.)	900	41·4
Ffestiniog (G.B.) . . .	360	43·1
Including transmission		* Estimate

TABLE 3: LOSSES IN PUMPED STORAGE STATIONS

Losses %	Cornwall	Cruachan	Vianden	Ffestiniog
Turbine . . .	12·0		7·5	7·6
Pump	12·0		10·8	9·6
Generator . .	2·0		1·8	1·7
Motor . . .	2·0		3·0	1·6
Product . . .	25·6	20·9	21·4	19·6
Conduit . . .	4·5	1·3	1·0	4·0
Other losses*. .	—	2·8	3·0	4·2
Net efficiency. .	—	75·0%	74·6%	72·2%

* Auxiliary power, transformers, loading, etc.

202. Mr Crosthwaite had already mentioned the relative cost of cavern and tunnel excavation costs. What were the Author's thoughts now on the vertical shaft versus sloping shaft controversy following their experience on this scheme?

203. Steel-lined tunnels and shafts were one of the most difficult civil engineering works to construct and so far as he was concerned the fewer of them there were the better. In a more or less horizontal tunnel the rate of progress with steel linings was about one quarter of that with a plain concrete lining. In addition the design was basically inefficient. The job of the steel lining is to keep the water inside the tunnel, but in fact the design criteria is generally buckling due to external water pressure when the tunnel was drained down. The result is a high cost of construction.

204. Construction difficulties increased still further in a sloping shaft at a self-mucking gradient. With modern grouting techniques in concrete shafts and tunnels the limit of working pressure has surely not yet been reached. It would have been possible to arrange the steel-lined section of the tunnel at an easy gradient away from the Power Station and so avoid steel linings in steeply sloping or vertical shafts—a modification of the alternative design 'B' given in the Paper with the power station as in 'C'.

205. With the bulk of the work underground it was clear that contingencies must be allowed for on a larger scale than for equivalent works at the surface. What was the increase in cost of the work due to unforeseen conditions underground? Reading between the lines it was obvious that it was substantial in view of the alterations to the work and the many changes in plan in carrying it out.

206. In the planning stages of the scheme, had the Consultants made any attempt to evaluate the cost of the scheme with surface or sub-surface penstocks and a power

station on the lakeside? The latter would have been a major undertaking with the site conditions and the submergence required for reversible units, but the expensive and time consuming underground works would have been avoided.

207. With regard to reservoir capacity it was clear that this was really a weekly pumped storage scheme and not a daily one. In future no doubt it would be possible to find out how that worked. It was not quite clear how the Authors were firming up the 1200 MW installed capacity of hydro plant in Scotland with the reservoir capacity at Cruachan. With only a day or so's generation at 400 MW this would not amount to very much.

208. The scheme was very fortunate in some ways. There was a good catchment area brought in by back-aqueducts and also a ready-made lower reservoir. The latter no doubt was a great help in keeping down the cost per kW but the cost of lower reservoirs in pumped storage schemes was not one of the major items. At Ffestiniog the lower reservoir cost only 10% of the total civil construction costs. The cost of dealing with the problems involved in other sorts of lower reservoirs such as the sea might exceed the cost of a conventional inland reservoir.

209. The power intake on the front face of the upper dam was obviously beneficial when considering the vortex problem and the head works for the shafts being well downstream of the dam must have simplified construction work at site. The plant arrangements for dam construction were excellent. Cableways for concrete only and derricks for all other handling purposes are by far the best solution of the problem.

210. The main control gate appears to require power for closure as distinct from merely a signal and tripping mechanism for gravity operation. Presumably the supposition is that with stand-by batteries this method is as foolproof as a trip on a gravity system.

Mr H. G. Keefe (Consultant, Sir Alexander Gibb & Partners) said that he had been asked to stand-in for Mr Guthrie Brown, who apologized for being unable to present himself.

212. One of the matters which, not unnaturally, had interested Mr Guthrie Brown as President of ICOLD, were details of the dams, in particular the water sealing arrangements. It was interesting to study the development of these water seals over the years and to see how they differed with time and with various consultants. In the slides shown, the dates were not completion, but what might be called 'vintage' dates, or assumed approximate design dates for the dams. The heights given were above natural ground and not above foundation level (in accordance with the World Register of Dams conventions).

213. Fig. 25 showed firstly, Loch Sloy Dam, the forerunner of the very well-known group of massive arch dams built in this country since the war. In the case of Sloy there were three simple keys sealed by bitumen fillings.

214. The next date, Errochty, was a diamond head buttress type dam. There was a fundamental difference in that in the case of Sloy there were two water paths, since the concrete in between had to fill a contraction gap, whereas free-standing buttress dams such as Errochty were structurally isolated with faces abutting but independent of each other. In Errochty the upstream waterstop was a copper strip which varied from 12 in. at the top to 24 in. at the bottom of the dam; in the centre was a 6 in. dia. bitumen core with a $1\frac{1}{2}$ in. steam reheating pipe; downstream was a Z-shaped copper strip, the main purpose of which was to prevent bitumen from flowing out on the downstream face of the dam.

215. The Glen Shira Dam (1949) was a roundhead buttress type, with heads 50 ft wide. This, however, had a contraction gap plug, similar to Loch Sloy, hence two lines of waterstops. There was a copper strip upstream and a bitumen core downstream, but nothing beyond that.

216. Next came Lednock Dam (1952) in Scotland, another diamond head buttress type. The head was 50 ft wide. (In the case of Errochty the head was only 40 ft

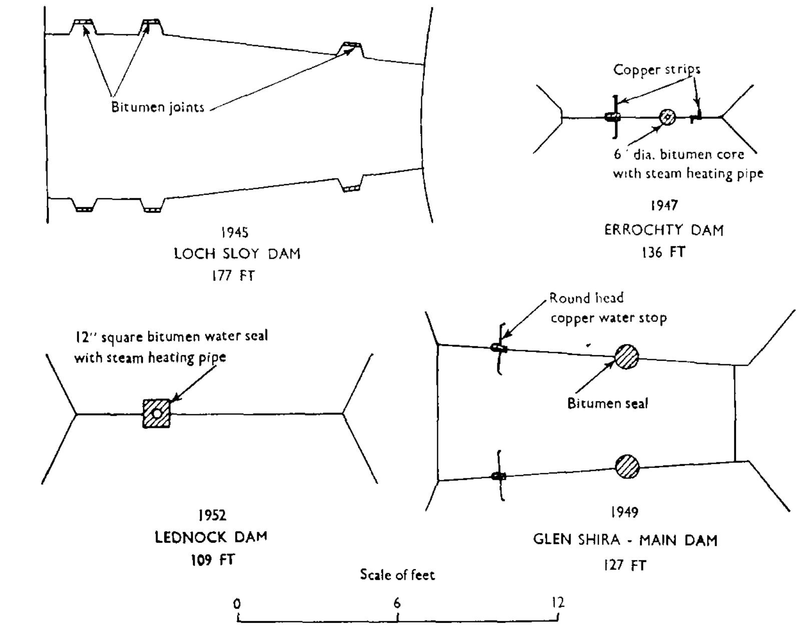

FIG. 25: COMPARISONS OF DAMS

wide.) At Lednock there was just one 12 in. sq. bitumen core with $\frac{3}{4}$ in. dia. U-shaped steam pipes for reheating.

217. Fig. 26 showed firstly Stwlan Dam (1958), a massive buttress type with contraction gap. This had a PVC 'dog-bone' waterstop upstream, bitumen strip filler in the centre and a recessed key downstream.

218. Cruachan Dam (1958) with its contraction gap, had a 12 in. PVC waterstop upstream, a centre 9 in. dia. bitumen core with Pyrotenax reheating cables, and a 6 in. downstream PVC waterstop followed by a 6 in. dia. drain beyond that. A marked change since its predecessor Loch Sloy 13 years before.

219. Lamaload Dam (1959) at Macclesfield was a roundhead buttress type, with 50 ft wide heads. The waterseals were, from upstream a 12 in. rubber 'dog-bone' waterstop, 9 in. rubber/bitumen and reheating pipes in the centre, and downstream a 3 in. dia. drain and a 9 in. rubber waterstop.

220. The next, Roseires (1960) in the Sudan, successfully opened a few days previous to the meeting, was a teehead buttress type. This had a 12 in. waterstop upstream, a bitumen core in the centre, with concentric oil reheating pipes, and with a 4 in. PVC waterstop downstream.

221. This was followed by the Farahnaz Pahlavi Dam in Iran (1961) (Fig. 27), a diamond head buttress type dam. Here there was a slight difference. There was a 14 in. rubber waterstop upstream, and in the centre a Guttaterna filler 1 in. wide with a 4 in. dia. bulb/core, and downstream there were two $8\frac{1}{2}$ in. wide PVC grout stops, the space in between being grouted up solid to prevent earthquake chatter.

222. The last in the group was the Clywedog Dam in Wales (1963), again a diamond head buttress type, with 12 in. rubber waterstop upstream, 9 in. $\times$ $4\frac{1}{2}$ in. bitumen core with oil reheating pipes in the centre, a $\frac{1}{2}$ in. sheet of Kork-pak, and downstream a 6 in. dia. drain and a 9 in. rubber waterstop.

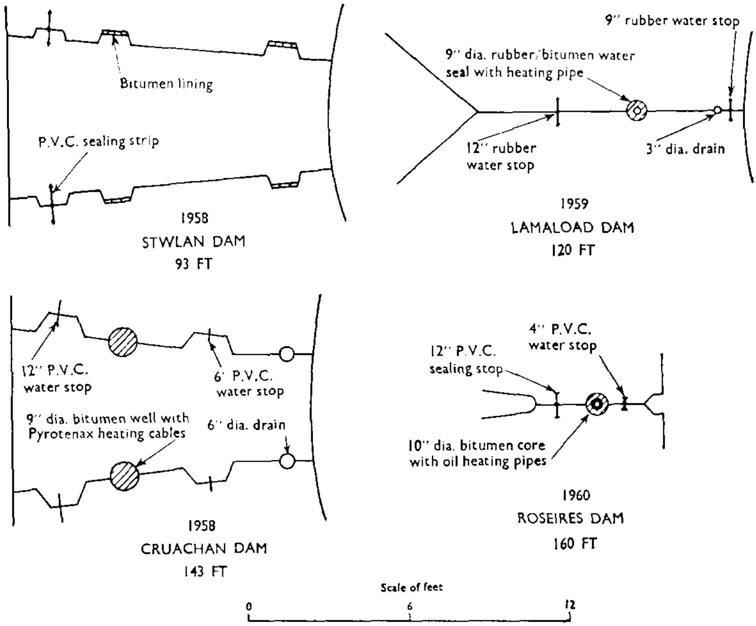

FIG. 26: COMPARISONS OF DAMS

223. There have been wide variations to the system first adopted for Errochty in 1947, but the basic principle would not seem to have been improved. A stop at the waterface, then most important a plastic bitumen plug, capable of subsequent reheating to prevent leakage, finally a downstream stop to prevent the heated bitumen staining the downstream face. This system is simple, and foolproof and for some 20 years, in the case of Errochty, has shown itself to be effective.

Mr L. H. Dickerson (North of Scotland Hydro-Electric Board) said that having been intimately associated with the Cruachan scheme since the construction work started, he would like to compliment the Authors on submitting a Paper which gave such a wealth of detail on a job with so many complex problems.

225. Referring to the question of layout, would the Authors, as a result of constructional experience, still favour the continuous self-mucking slope between dam and machine hall or would they consider the combination of the vertical and horizontal high pressure tunnel more attractive? The results of the alternative tender prices would naturally have a marked bearing on the decision, but one would have to bear in mind the limitations of the Alimak referred to in § 74. Other factors were the cost of the subinverts for the steel pipes and the restrictions imposed on handling all the pipes through the power station access tunnel. Assuming that the financial balance was still in favour of the long inclined shaft, then the provision of additional access to the bifurcation would doubtless have been attractive and from experience gained the cost of the added length of access tunnel might well have been justified. Some further comment on this from the Authors would be helpful.

226. He regretted that there had been no space for reference to the setting out methods used for the sloping shafts. This must have been an extremely difficult task,

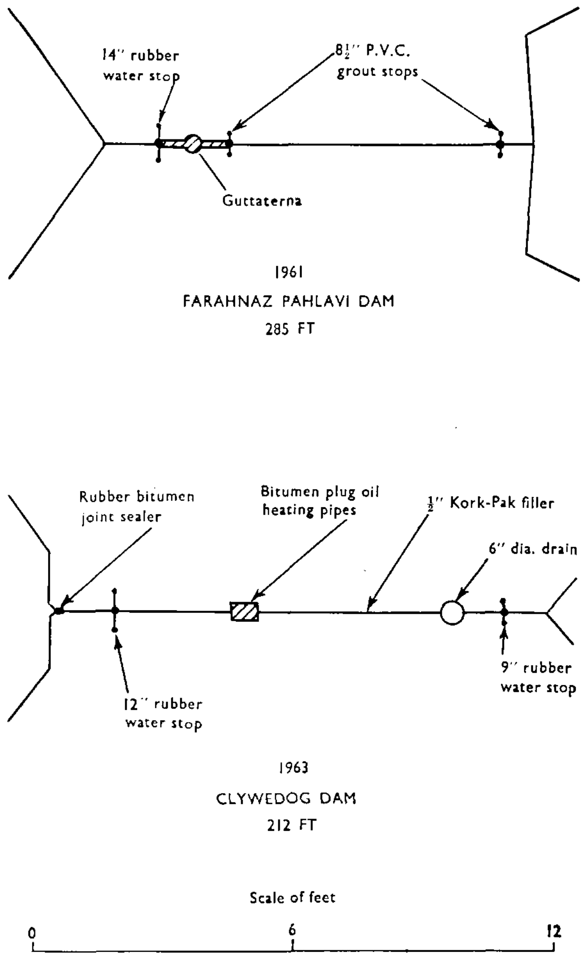

FIG. 27: COMPARISONS OF DAMS

particularly when one had not only to get round the bifurcation but also to junction with shafts being sunk from ground level. Was it possible to have a little more information on this, in particular the accuracy to which the survey work was carried out?

227. With regard to the layout of the power station, it was not unusual for a loading bay to be placed centrally between the machines. This reduced considerably the amount of craneage, and the Authors could perhaps comment on this.

228. Reference was made to the use of the 7½ ton temporary crane, which was provided at a later stage at the Board's request, and supported on the main steel columns. Having acknowledged the usefulness of this crane, would the Authors have any ideas for a different means of support, so that if installed at an earlier stage in the work it could be used for other operations such as erection of steelwork.

229. The arch roof with a span of 75 ft was much larger than in other Scottish underground stations, where the arches were continuous mass concrete and pressure grouted. The ribbed construction adopted at Cruachan no doubt involved considerable added expense with scribing shuttering to rock on each side of every arch rib. Accepting the need to divide the arch into panels, could this not have been achieved by much narrower standard slots exposing only small areas of rock to permit drainage?

230. Reference was made to the various measurements of the physical properties of the rock in the cavern. Was it possible to give some idea of how the results from different methods compared? Could the Authors confirm that the figures obtained influenced the decision to concrete the 'eyes' of various openings before proceeding with more excavation?

231. On the question of concreting of the pressure shafts, it would be useful for the record if some information could be given on the mix, and what limits were placed on the heat rise during hydration. One of the most difficult problems for maintenance painting, etc., was the access down the sloping shafts, and the Authors might care to comment on the Australian practice, where permanent guides had been provided in the shafts for inspection and maintenance trolleys. The present ideas on which the engineers were working involved wheeled trolleys which had to be steered round the bifurcation.

232. The records of the strain gauge measurements at the dam were extremely useful, although it would be agreed that some of the instruments left a certain amount to be desired.

233. The brief reference in § 23 to the diversion aqueduct system hardly did justice, due to space limitations, to the detailed design and construction involved in this important section of the scheme. All the 74 intakes had been designed for high efficiency coupled with a minimum of attendance in this high rugged catchment, where in many cases the only access was through the actual diversion tunnels. When this was necessary the controlling intake gates were closed by remote control from the entry point, and personnel travelled in by Land Rover.

Mr C. M. Roberts (Sir William Halcrow & Partners) said that his first point concerned the storage capacity chosen for the upper reservoir. In § 16 this was stated to be 7·5 million units. To serve the pumped storage installation alone, if it were on a daily basis, presumably all that would be necessary would be a capacity equivalent to about 4 hours' full output of the plant, as at Ffestiniog, or 1·6 million units. Mr Roseveare, however, had drawn attention to the fact that the basis of design was a weekly and not a daily one. This meant that his (Mr Roberts') arithmetic was a little off the mark but he would quote these figures so that the Authors could put them right, including the correction to a weekly basis. On the basis of daily storage, the remaining 5·9 million units of storage would be available to regulate the direct and diverted run-off into the upper reservoir. Presumably in this case it was not considered economical to provide extra storage to firm up the output of existing hydro-electric power stations as mentioned in § 8.

235. In § 23 it was stated that the aqueduct system yielded 36 million units during a year of average rainfall. Presumably this was discounting the losses due to aqueduct capacity.

236. Taking the diverted run-off only and ignoring the small direct run-off, the 5·9 million units of storage represented just over 16% of the average annual flow into the upper reservoir. This was considerably smaller than the usual capacity, which amounted to approximately 30% in most normal hydro-electric schemes.

237. Could the Authors say whether the height of the dam was in this case selected on the basis of economics of storage capacity in relation to the utilization of the catchment?

238. With regard to the measurement facilities embodied in the dam, the Authors stated in § 53 that the movements indicated by direct measurements, in other words, by inverted plumb-bobs, collimated points and precise levelling, showed a general pattern of temperature and shrinkage effects with insignificant deflexions attributable to water loads. This seemed to be the conclusion to be drawn also from the strain gauge measurements, although from these the Authors stated in § 54 that they were able to derive information as to the time at which the concrete mass commenced to act in a homogeneous manner. It would be interesting to know the nature of this information as it was not immediately apparent in Fig. 7 showing these strain gauge measurements.

239. Mr Roberts then compared the strain measurements with those obtained at Monar arch dam, where a range of measurement facilities was embodied in many ways similar to that used by the Authors. Fig. 28 showed the plotting of the readings over the last four years. There was no external loading at all during the major part of 1962; however, the behaviour of the strain gauges was very erratic indeed, and this covered the period of setting and hardening of the concrete. The influence of temperature variations from summer to winter had to be taken into account.

240. The other point of interest was that as soon as the water load was built up the dam deflected upstream at the crest, which was quite common to any type of dam.

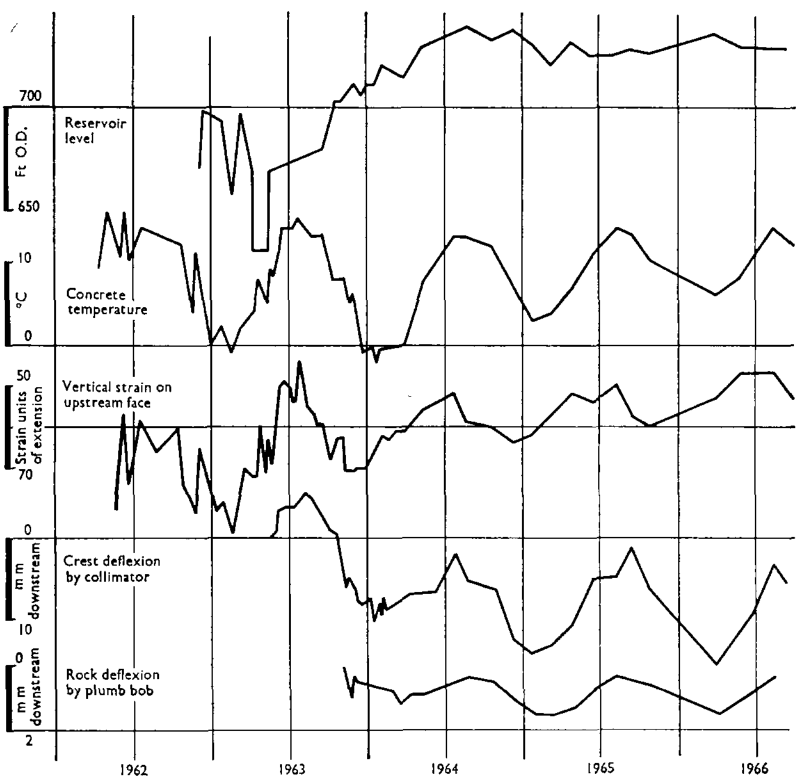

FIG. 28: READINGS OBTAINED OVER A FOUR-YEAR PERIOD AT MONAR

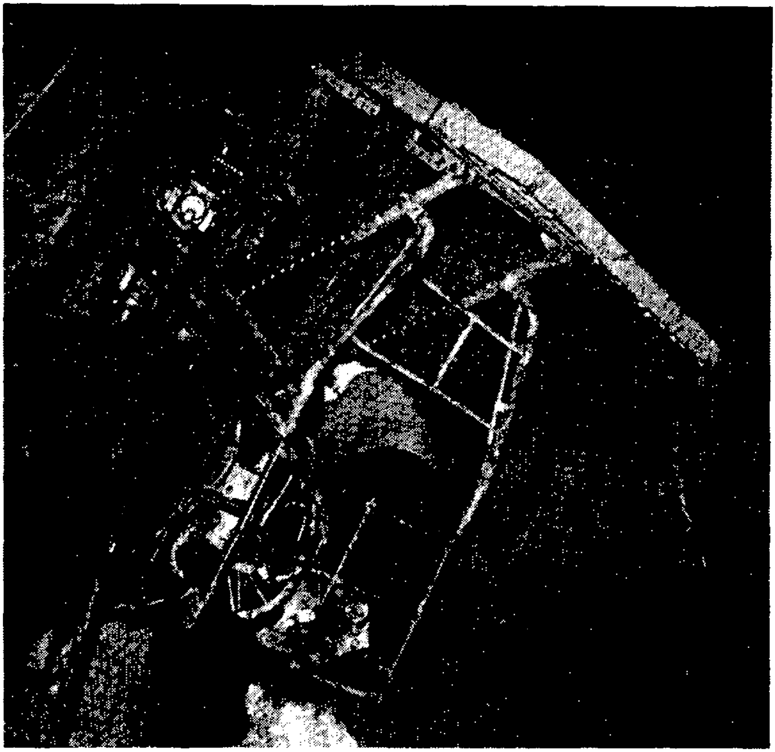

FIG. 29: STANDARD ALIMAK PLANT

After that the deflexions followed the seasonal variation of temperature. Unfortunately, there was no inference to be drawn from the water loading as this had been fairly steady, but the readings confirmed what the Authors had found.

241. Would the Authors say whether in their opinion these results indicated that high stresses could be locked into the concrete created by temperatures and shrinkage during the hardening period when inevitably some parts of the mass had hardened while others were still plastic and yielding? The tests on creep at present being undertaken by Professor Zienkiewicz at Swansea University would help to solve this problem.

Mr J. E. W. Hill (Edmund Nuttall, Sons & Co. (London) Ltd) referred to the Alimak shaft raising equipment and the shaft raising problems at Cruachan. Figs 29 and 30 showed the standard type of air-driven Alimak machine which was in common use before Cruachan and which had been used quite extensively in mining and hydro-electric work but had a limitation in that, due to the weight and lengths of air hose, it was limited to about 100 m in height. To do shafts higher than that it was found necessary to drive them in stages.

243. In order to get over the difficulties associated with staging, diesel driven equipment was chosen. The ventilation problems associated with diesel engines gave rise to some concern and a 5 in. dia. pipe was inserted into the guide rail, the

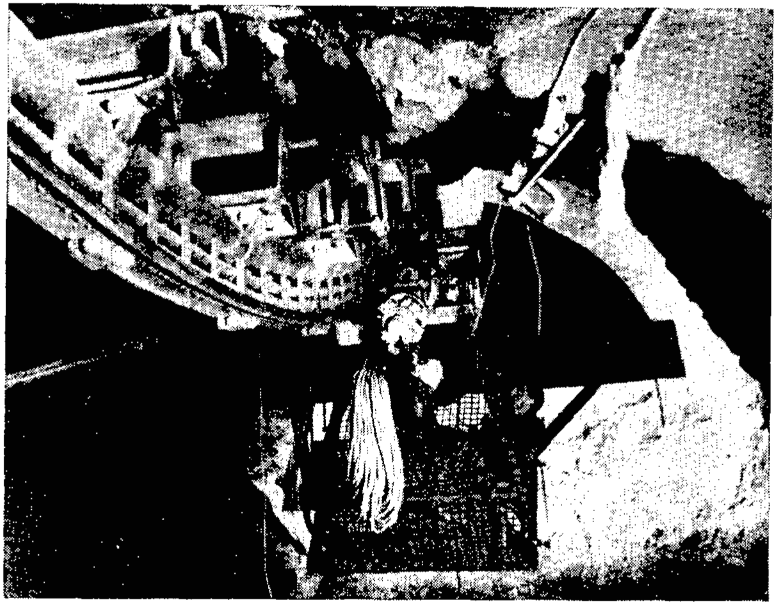

FIG. 30: THE ALIMAK'S PERSONNEL CAGE

proposal being to ventilate through that pipe and so get rid of diesel fumes. In fact, as had been remarked, the diesel equipment was very smoky and caused very great difficulties both in setting out and driving the shafts. They had therefore gone to Messrs Alimak who had developed for them a much more robust type of air-driven machine which was eventually used for driving the Penstocks. These were done in the manner indicated in Fig. 31. With the new type of machine it was possible to raise shafts up to about 750 ft, and they had proceeded in the manner shown here going up approximately 500 ft and making a staging point just above the bifurcation, from where another machine went up a further 700 ft to complete the junction. This method of working proved extremely successful.

244. The diesel driven service cab continued to be used throughout the contract and worked very well.

245. The vertical shaft was driven approximately 800 ft high with diesel gear and it was done quite quickly, taking 228 working days.

246. There had been some remarks on survey. He did not wish to steal the Authors' thunder, but throughout all the shaft work visibility was a very great problem. In the penstocks the line was carried forward from steps cut in the foot wall where the surveyor could stand with his gauging instrument while the Alimak ran back down to the back station to take the back site and then travelled forward again over his head to the forward station. All the accurate measurements in the shafts were done with a geodimeter and this was found to be a very useful aid to the survey. The junctions all fell within a few inches.

Mr B. T. Seddon (Binnie & Partners) said that he would like to compare certain aspects of the work at Cruachan with similar work within his own experience and to ask the Authors for more information on certain points of detail arising from their Paper.

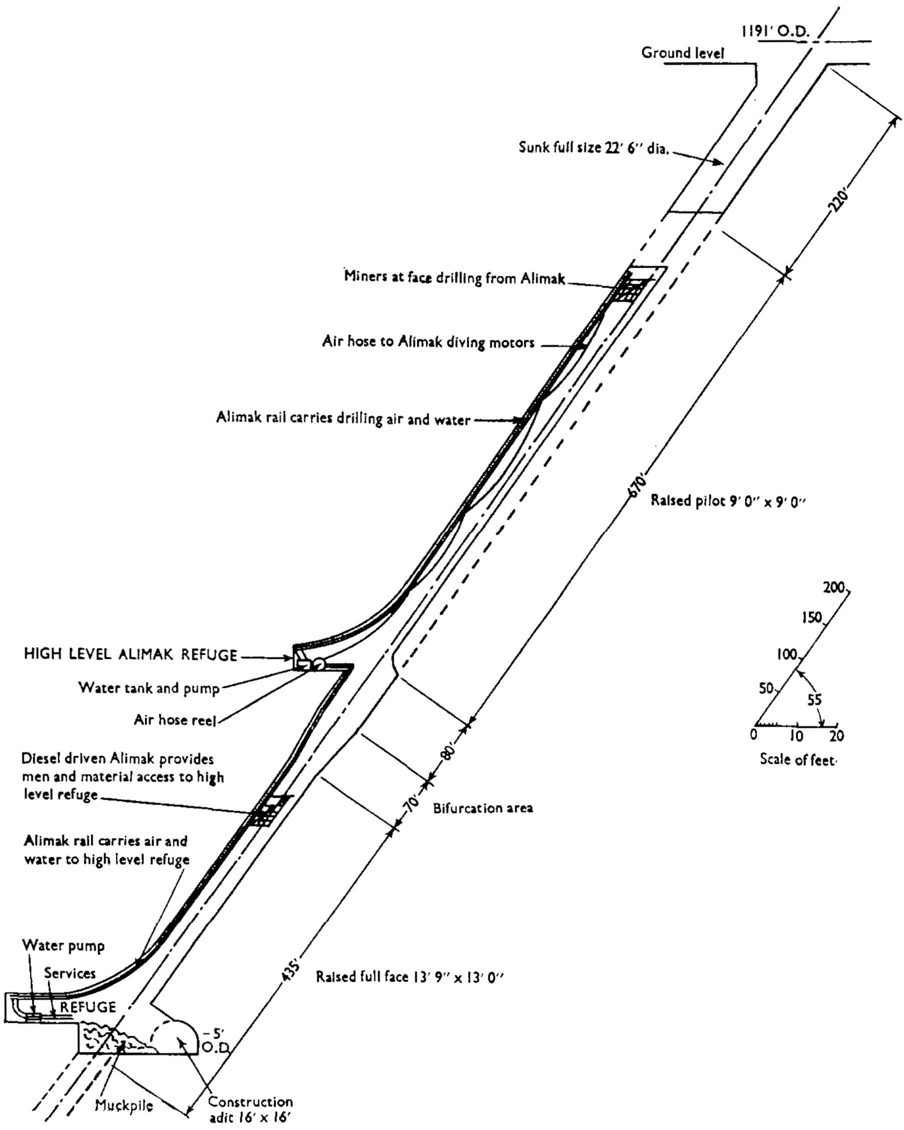

FIG. 31: THE AIR-DRIVEN ALIMAK PLANT SYSTEM

248. At the Woh underground power station for the Batang Padang scheme in Malaya, the power station would house three 50 MW Francis turbines operating under a gross head of about 1350 ft.

249. Fig. 32 showed the cross section of the Woh power station as indicated on the contract drawings.

250. The span of the main cavern was about 70 ft and overall height about 90 ft as compared with 75 ft and 125 ft respectively at Cruachan. The rock cover was about 900 ft as compared with 1100 ft at Cruachan, and in order to avoid the high local stress concentrations which often arose in vertical walls below arch springings it was decided to adopt elliptical excavated sections for the main underground caverns as far as possible. The rock was a coarse grained granite and had a modulus of elasticity of about $4{\cdot}5 \times 10^6$ lb/sq. in. as determined by plate loading tests. Before the orientation of the power station was finally settled, rock investigations were carried out in galleries driven from the inner end of the tailrace tunnel, which was excavated under a preliminary contract. Rock stresses were measured by similar methods to those adopted at Cruachan and the stress ellipsoid was shown in Fig. 31. The maximum principal stress at Woh was about 1·8 times the stress due to the weight of rock above as compared with vertical and horizontal compressions at Cruachan equal to about three times the weight of rock.

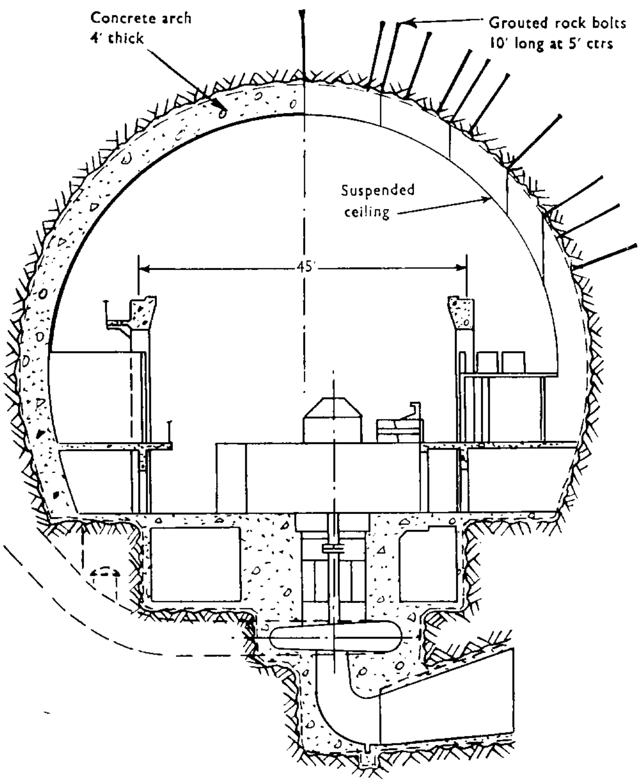

FIG. 32: CROSS SECTION OF WOH POWER STATION

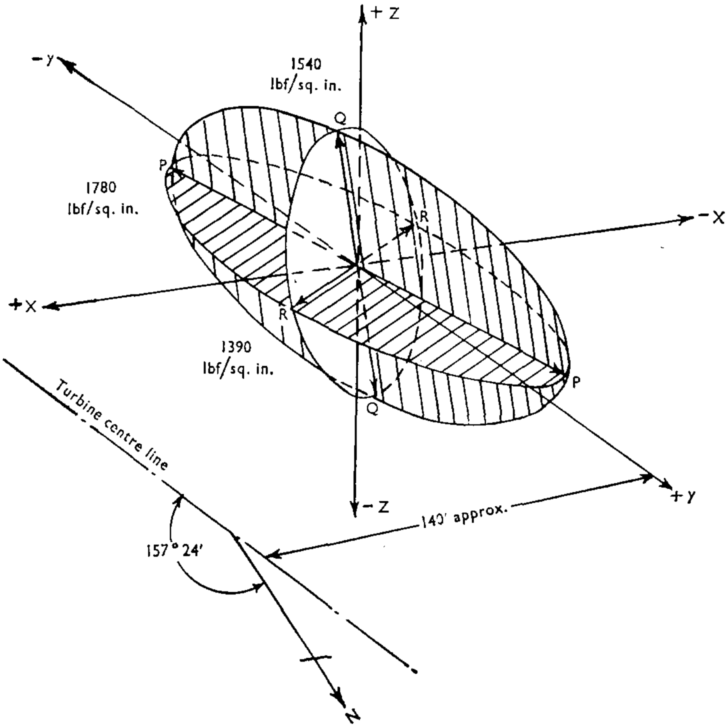

FIG. 33: THE STRESS ELLIPSOID AT WOH

251. The contract document made provision for either a concrete arch roof or for grouted rock bolts; wire mesh and gunite, depending on the rock conditions revealed on excavation of the upper part of the cavern. In the event a concrete arch was not necessary.

252. The 1 in. dia. high tensile steel grouted rock bolts installed in the main Woh cavern were generally 10 ft long on a 5 ft grid and tightened to 250 lb ft as compared with 15 ft long on a 7 ft 6 in. grid and tightened to 150 lb ft at Cruachan. The main difference between the bolts appeared to be the $\frac{3}{16}$ in. dia. central deaeration hole provided at Woh. Grout injection continued until grout passed down this hole, which was then sealed. As at Cruachan, grout leakage was initially high but this was largely overcome by sealing round the anchorage plate with cement/sika paste after final tensioning.

253. The Authors mentioned the use of 'smooth blasting'. In what areas of the power station was this technique used? Was it successful in achieving its purpose and how was it measured and paid for?

254. The steel tunnel linings for Zone B were designed so that the normal working stress was not exceeded under 20% water hammer. How was the normal stress defined in relationship to the yield and the tensile strength?

255. As the Authors indicated, there was often an advantage in adopting high tensile steel where the thickness of steel lining was governed by internal pressure rather than external pressure. This point was brought out in Table 4, which compared steels used on recent projects. Savings in weight could approach 40% as compared

TABLE 4: STEEL TUNNEL LININGS
(Comparison of steel used on recent projects)

Type of steel	Tensile strength *lb/sq. in.*	Yield stress *lb/sq. in.*	Elong.[N] minimum %	Impact properties *ft/lb at 0°C*	Relative thickness *in.*	Project
B.S. 2762 ND1, grade B	63 000*	34 000*	20	20	1·0	Cruachan
A.S.T.M. 212, firebox grade B . .	70 000	38 000*	22	30 (10°F)	0·90	Manglia Wye branches
Colville's Coltuf 32	72 000*	45 000	24	50	0·81	Ffestiniog. Cruachan
Vöest Aldur 58	82 500*	55 500	22	35	0·71	Batang Padang
U.S.S. T1A	115 000*	100 000	16	20	0·51	Manglia tunnel linings

* Governing stress for determination of thickness.

Notes—
1. Relative thickness is based on assumed design stress equal to 60% yield or 35% tensile strength whichever is least.
2. No support assumed from rock.

with conventional mild steel but the cost per ton of plate was higher and greater welding skill was necessary. In the case of the high tensile steels, the required factor of safety on the tensile strength might determine the allowable working stress rather than the factor of safety on the yield.

256. With regard to the spiral casings, the Authors stated that grouting was carried out using neat cement. Were the casings concreted under internal pressure or was some form of elastic filler placed between steel and concrete?

257. What was the minimum thickness of concrete in the tailrace tunnel lining? This tunnel drained towards the bulkhead gate at the outfall on Loch Awe. Was any form of pumping chamber provided for dewatering the tunnel?

258. Was there provision for entering the shafts for maintenance and, if so, what form did the provision take?

259. The operating conditions necessitated a very comprehensive study of surge and water hammer conditions which was probably unique as far as British experience was concerned. This subject would in his view merit a paper on its own.

Mr V. G. Newman (Central Electricity Generating Board) said that the Cruachan scheme marked an important second stage in the development of major pumped storage projects in this country, following the completion of the Ffestiniog scheme five years ago. The intervening period had seen a quite remarkable advance in the development of the high-head reversible pump turbine and it was for this feature that Cruachan was particularly notable. There seemed little doubt that the reversible Francis pump turbine showed decisive overall advantages in most medium head applications, compared with alternative types of machine, and one would anticipate that in future it would not be necessary to obtain tenders for such a wide variety of alternative plant types and arrangements as was necessary at the planning stage of Cruachan.

261. From the civil engineering aspect, the adoption of concrete-lined pressure tunnels to withstand a head of 1000 ft was also a notable step forward and was to be welcomed, since the trend towards even larger unit capacities with larger diameter tunnels would greatly add to the difficulties of providing steel linings and might make their cost prohibitive in the larger sizes. Cruachan was by far the largest underground power station so far built in Britain and in this respect was one of the forerunners of a now general tendency to locate water conduits and power caverns below ground, not merely for reasons of public amenity but also from technical and economic considerations.

262. Turning to some details of the Paper, it appeared that additional reservoir storage had been provided for the purpose of firming up the output of existing hydro stations in dry years. It would be interesting to know how much of the total storage capacity was accounted for in this way. Was any further margin added for seasonal regulation of run-off? Considering operation in a normal year when presumably operation for firming up purposes would not be necessary, could the Authors state the annual load factor of generation for which the station was designed and indicate the expected daily and weekly operating regimes?

263. The dam was provided with a conventional overflow spillway for protection against flood inflows from the catchment areas and against over-pumping. The 3800 cusecs capacity of this spillway he took to be the discharge of all four machines when pumping against maximum head. This figure represented a fairly considerable volume of water, about 50% greater than the average flow of the Thames at Teddington. He felt certain that the possibility of this discharge cascading down the hillside was only contemplated as an exceedingly remote possibility, and it would be interesting to know what safety measures would normally take effect to ensure that spilling at full pumping discharge did not occur.

264. Reference to a working gradient of 1:8 in certain tunnels was made in § 71.

Did the vehicles bringing in heavy generating plant to the caverns have to negotiate a gradient as steep as this?

265. Grouting was dealt with in § 112 and it seemed that the spiral casings of the pump turbines were embedded solidly in concrete without the interposition of any layer of resilient material. What were the considerations behind this decision? It was noted that the velocity through the lower outfall during pumping had been limited to 1 ft/s for the protection of smolts. He assumed that this velocity was fairly uniformly distributed over the screen area. On the other hand, during generation, the discharge stream could not be expected to expand over the entire 110° arc of the forebay and he would expect it to be concentrated over the centre panels of the screens, giving rise to velocities well above 1 ft/s. No doubt the hydraulics of the outfall were model-tested and it would be interesting to know the maximum velocities of discharge into Loch Awe which could occur in the generating phase.

Mr J. Paton (Babtie, Shaw and Molton) said that in reading this interesting Paper it was intriguing to compare Cruachan Dam with Sloy Dam, of the same type, completed in 1950, and Cruachan underground station with, say, Ceannacroc power station, the first of the North of Scotland Hydro-Electric Board's tunnelled underground stations, commissioned in 1956.

267. Sloy was the first of the massive buttress dams of the design favoured by the late James Williamson—a design which had been progressively improved in detail over the succeeding years. As an earlier Speaker had pointed out, it was interesting to note the change which had taken place in a matter of 15 years, wherein the simple checked joint of Sloy Dam had given way to the more sophisticated joint of Cruachan, incorporating two PVC water stops, a bitumen sealing core with re-heating equipment and a downstream drain on the back slope. It would be helpful if Mr Young, who had been associated with both dams, would comment on the performance of these joints at Sloy and Cruachan.

268. There was only one aspect to which he would refer in the time available, namely, the grouting. In § 47 the Authors stated that 'it was evident that the opening of joints in the bedrock was due to blasting and was confined to the surface as injection at depth proved very stubborn', and in § 49 they said that 'few areas with open joints were found and perhaps in retrospect a less intense curtain would have sufficed'.

269. He mentioned this as he had had a very similar experience with the foundations of Finglas Dam, Perthshire, commissioned in 1965. At that site water pressure tests in boreholes sunk into the schist bedrock indicated that below the level of 15 ft the rock was very impermeable (generally less than one-tenth of a lugeon). Above 15 ft, due to fissuring and possibly disturbance by excavations, higher permeabilities up to 1 lugeon or even more were found. With these results established, the scheme of grouting adopted at Finglas was a dense but shallow curtain with holes as close as 2 ft 6 in. centres extending to a depth of 15–20 ft below rockhead, and below this, at 5 ft centres, to depths of 30–40 ft. Below the depth of 15 ft the acceptance of grout was very small indeed.

270. To enable these experiences to be compared in a quantitative manner, and in the light of the comments in § 49, it would be helpful if the Authors could indicate what the water testing of the bedrock at Cruachan Dam had shown, and also the criterion adopted as an acceptable standard of permeability.

271. Turning to the Cruachan underground works, it was obvious that by reason of their sheer scale and complexity, coupled with exceptional site difficulties, as at the outfall to Loch Awe, the design and construction of these works involved a great deal of original thinking on the part of the engineers and contractors. It was to be hoped that the valuable experience gained would be used in developing the new pumped storage schemes which were presumably required in conjunction with the large thermal base load plants planned for the future.

Mr A. A. Fulton (formerly of North of Scotland Hydro-Electric Board) said that he was very pleased to have the opportunity to say something about Mr James Williamson. Mr Williamson did not live to see the final form of Cruachan but would have been very cheered to know that such a large measure of pumped storage was embodied at this site, for, apart from his great flair for spotting schemes, he was a great advocate of lower load factor projects, and was frequently very disheartened by the poor reception his ideas received in some quarters.

273. Many speakers had asked whether the Authors would have done the same things again. In dealing with the possible repetition of the use of the Alimak equipment, he doubted if the conditions would be the same again, for the reason that if Cruachan were done today there would probably be only one and not two pressure shafts, and this would require an excavation performance much in excess of the Alimak plant used at Cruachan.

274. When it came to deciding the size and number of pressure shafts, the comment made in § 26 of the Paper was very pertinent. It emphasized the important part played by the 'acceptable outage in a supply system'. In his experience, there was a tendency to be too much influenced by the position at the time a scheme was initiated. With electricity loads doubling in less than ten years it was easy to see how quickly the problem of acceptable outage solved itself.

275. In the case of pressure shafts, the principle query left in his mind was whether there was any prospect of eliminating steel linings. He said this because, although the Authors did not say so, he was sure they would agree that the problems of placing and welding these linings, not to mention manufacturing, delivering and storing them at site, were among the most intractable met with on the job. In the light of that, the advantage of having no steel linings was so obvious that it was not surprising great efforts had been made in recent years to learn more about this possibility. Perhaps Mr Young could say how much help he received on this subject from the rock testing done at Cruachan.

276. Mr Fulton said that he had already referred to the headaches provided by the steel pressure linings but, as everyone knew, civil engineering works on the Cruachan scale seldom got away with only one crisis-creating problem. Cruachan was no exception. The start was not altogether auspicious because, as mentioned in § 69, it was delayed for several months as the access tunnel was not available when it should have been. It might be, however, that the accelerated programme which this and other circumstances made necessary paid an unexpected dividend, in that the introduction of extra working points and adits helped greatly in coping with the severe fault conditions experienced when excavating the main hall.

277. At that early stage the excavation problems created by the crush band overrode all others. In due time these were succeeded by piling difficulties at the outfall site and then the slips and falls experienced in excavating the pressure shafts. At these times of crisis it was all too easy to become depressed about progress, whereas in fact everyone who had any responsibility for the situation rose wonderfully to the occasion and fully deserved all the encomiums paid to them by the Authors in § 179. These tributes would, Mr Fulton hoped, also help to make up for the absence in § 72 of any expression of sympathy to those who had to work underground when the ventilation conditions were so bad.

Mr W. E. Blackmore (Sir William Halcrow & Partners) referred to the steel pipe in the high pressure shaft. In what was referred to as Zone C (§ 122) the design criterion was the external ground pressure whereas in Zone B (§ 121) it was the internal pressure. His own experience was that, for these high heads, the external pressure was the dominant factor; it would have been otherwise if there had been external anchors welded to the pipes, but there was no mention of these having been used at Cruachan.

279. The main control gate (§ 41 and Fig. 3) had a lower edge with an almost

symmetrical bevel. It was said that 'The V-shaped lower edge which seals against the sill ensures that the unbalanced forces are at a minimum and in consequence that least power is needed when the opening operation starts.' Mr Blackmore would have expected that this object would have been better achieved by a bevel entirely on the upstream side with a vertical (face) downstream. This was also the shape that helped best to avoid vibration.

280. With regard to the intakes (§ 23), Mr Dickerson had pointed out, in the discussion, that they were self-cleaning. It would be very helpful to know what was the type of intake which produced this self-cleaning property. As there were 74 of them obviously a good deal of thought must have been given to the design, and presumably a considerable amount of experience had now been gained on their operation. It would be helpful to have further comments on this.

Mr V. H. Collingridge (Director, Mowlem (Civil Engineering) Ltd) said that not long ago Mr Falkiner had reminded him that the Authors had made use of the gravel pit that John Mowlem & Co. Ltd (who had been close runners-up) had been able to locate while tendering for some of the previous jobs on the Cruachan scheme. It might be of some interest to refer to the way in which this gravel pit was located.

282. An adequate supply of suitably graded sand was seldom readily obtainable in the Highlands, although there was plenty of rock about, as everyone knew. The location of a suitable supply of gravel could very readily effect an appreciable saving on other sources of coarse aggregate supply such as processed tunnel spoil, or, indeed, a much larger saving as against quarry-won material if the haul were considerable, as in this particular case.

283. With this in mind, when tendering for the various sections of the Loch Awe hydro-electric scheme, they had gone all out to locate suitable deposits of natural aggregate, to get an edge on their rivals. The investigation was put into the hands of Mr K. Earley of Soil Mechanics Ltd to locate and report on the available resources in the very limited time available during the tender period. The available resources were, firstly, tunnel spoil; secondly, river terraces; thirdly, shore deposits; and, fourthly, raised beaches.

284. In considering the tunnel spoil aspect, an early assessment of suitable and unsuitable materials had to be made, and it was not very easy to do until one got inside the tunnel. Working in ground associated with large-scale faultings, the nature of the rock could change from suitable granites to unsuitable phyllite with the advance of a tunnel face; sorting the spoil became a necessity and there could well be a large fluctuation in the volume of suitable material to the crushing plant.

285. With regard to river terraces, some good deposits of rich gravel were evident in the flood plain of the River Awe. One was close to the power station site at Inverawe; another lay immediately upstream of the Bridge of Awe at Crunachy. Both of these deposits were not much above existing river level and it would have been unwise to assume greater depth than 6 ft, most of which would involve excavation below water level. There appeared to be a high proportion of boulders in these deposits which would either involve considerable wastage or secondary processing by crusher.

286. There were some shore deposits of sand visible at the mouths of the Nant and Awe rivers. These were difficult of access and were mainly resorted sand of shallow thickness and underlying common grazing land.

287. Just north of Taynuilt a raised beach existed which seemed to indicate from exposures the presence of material closely akin to ready-mixed all-in aggregate. Another raised beach deposit was noted north of Inverawe House. After further superficial investigation it was decided to prove the Taynuilt deposit for suitability and reserve. Eight resistivity soundings were made indicating depth of gravel varying between 16 ft and 30 ft, followed by three pits for sampling purposes. These were so encouraging that three boreholes were put down a week before the tender was to go

in, which proved bedrock at 20 ft. Calculation showed that this depth would provide an adequate reserve of over 250 000 cu. yds of usable gravel below a sandy overburden 3 ft thick; generally, the gravel was rather over-rich in sand, with a tendency for the coarser factor to fade out towards the sea.

288. It would be interesting to know whether this investigation and the findings were borne out in practice. He had not been in that part of Scotland since the time of tendering. Perhaps the Authors would give this information in their reply.

Mr J. K. Hunter (Sir Alexander Gibb & Partners) referred first of all to the pressure shafts. The Authors had stated that both vertical and incline pressure shafts were considered and that the decision was left until tender prices had been received for both alternatives. This implied that the choice of the incline shaft was made on a comparison of costs, but it would seem that in the event, as other speakers had already mentioned, the construction of the incline shafts gave rise to a good deal of trouble and anxiety, so it seemed relevant to enquire whether the actual cost did not accord very well with the prices on which the decision was taken, and whether, in the light of subsequent experience at Cruachan, and bearing in mind the very great body of experience which had been obtained in other parts of the world in the last few years in the construction of these incline shafts, if the choice were made today it would be any different.

290. Over the past 20 years very great advances had been achieved in the construction of hard rock tunnels and these advances had led to very significant reductions in real costs and, of course, to profound changes in the overall planning of hydro-electric schemes. In recent years there had been some improvement in the methods of the shaft construction, especially in the mining industry, but in spite of these advances it did appear to be the case that the costs of driving shafts, whether vertical or incline, were a good deal less certain in their outcome both as regards time and cost. This uncertainty led to a difficulty, in that it became a little difficult perhaps to choose between a shaft or a tunnel where these respresented possible alternatives, since the engineers' estimate of the two alternatives might not be equally reliable.

291. The Authors had referred to the difficulties which arose in the use of the Alimak shaft raising equipment, and this had been mentioned also by a number of previous speakers. It seemed a little surprising that these difficulties should have arisen in the light of recent Norwegian experience, where incline pressure shafts up to 16 ft in dia. had been driven full face. It might well be, as the Authors implied, that the particular equipment employed at Cruachan was used beyond its real capacity, and that the shafts which had been constructed in Norway during the last few years had used more advanced design. However that might be, it would appear that in Norway at least the system of shaft construction using the Alimak climber had now become well established and was accepted as a reliable and safe method of construction, and had virtually superseded other older methods. The same was true in Sweden, too. There appeared to be no consistency among engineers in regard to the choice of vertical or sloping shafts for hydro-electric development. A survey of recent hydro schemes carried out during the last 15 years all over the world suggested that the use of vertical pressure shafts was mainly confined to Sweden, the country which virtually originated the underground power station, while the greater majority of other projects carried out in the rest of Europe and North and South America employed sloping shafts.

The following contributions were received in writing

Dr W. M. MacGregor (Director, George Wimpey & Co. Ltd) wrote that the Paper contained a wealth of information which would repay careful study by engineers interested in heavy civil engineering construction.

293. It was noted in § 17 that the plant manufacturers imposed a limit of 100 MW per machine where heads of 1200 ft were concerned, which was consistent with the

conclusions reached after studying the civil engineering design of the high pressure penstocks where welding at the lower end became the critical factor. Coltuf 32 steel was used in preference to steel to B.S. 968 in zones A and B and steel in accordance with B.S. 2762 was used for zone C. What were the thicknesses in each of these zones and what special precautions and electrodes were used in the welding?

294. In view of experience gained at Cruachan would the Authors be prepared to use greater thicknesses of steel in future designs.

295. Measurement of strains and other movements in dams was of special interest to the International Commission on Large Dams. In the report on reservoir safety recently published by the Institution reference was made to the need for proper instrumentation which should be installed to record settlement and movement of dams. At Cruachan there has been a certain amount of pioneer work and the failure of the polarized material by lime leached from the concrete was a warning to others interested in this field of research. What amendments would the Authors suggest to avoid similar happenings in the future?

296. In § 52 mention was also made of a stress value of 870 lb/sq. in. which was mostly attributable to shrinkage of the concrete. This seemed a high value on an unloaded structure and further information on this would be useful. The strain gauge measurements on Buttress No. 7 were rather confusing and he wondered if further data were available for the year 1966, which would show more clearly the effect of water load.

297. Very interesting data had been recorded in § 95 on the loss of tension in rock bolts as excavation took place in the machine hall. Presumably this was due to slippage of the anchors rather than release of stresses in the rock mass.

298. Some data on the amount of overbreak in the tailrace, tunnels and access shafts would be useful. In this connexion on the 3¼ mile long tunnel (approximately) leading from the Awe barrage to the Inverawe power station near Loch Etive (Fig. 1) the overbreak in the arch and sides averaged 13 in., which was higher than expected considering that ladder drilling and smooth blasting techniques were used. This tunnel had an equivalent diameter of 23 ft 6 in. and the minimum thickness of concrete of 11 in. was specified in areas of rock of 4 sq. ft and over.

Mr W. D. Short (H.M. Senior Construction Inspector of Factories) wrote that he wished to comment on construction site accidents and their prevention—which was not mentioned, except indirectly in §§ 72 and 74 which dealt with ventilation problems and accidental firing of detonators. He emphasized that what he said was not directed at firms or individuals concerned with the Contract but at the construction industry as a whole. If the Authors would allow use of the words in the first paragraph of their Paper, his points were put forward 'in the hope that the experience gained will be of benefit in future schemes'.

300. Work on all hydro-electric contracts involved hazards of many types which unfortunately often resulted in accidents to the men employed. But over the years many of these hazards had, by repetition, become well known and it should therefore be possible to eliminate some and mitigate others to a degree which would eventually reduce the number and severity of the accidents which were such a burden on the construction industry; a burden which was a combination of human suffering, of the loss of trained men and of a monetary sum amounting to many millions of pounds.

301. The Writer visited Cruachan several times and discussed some of the accidents with those concerned as well as studying the accident figures as a whole. A brief record of these accidents with suggestions for the prevention of some was as follows.

302. During the five to six years' work on this contract, and on the allied one at Inverawe, 448 accidents were reported to H.M. Factory Inspectorate; there were ten fatalities and one of the worst single accident incidents on any hydro-electric scheme in this country when one man was killed, eleven were disabled and much material

damage done. The 448 accidents can be classified into groups which follow much the same pattern as those in the tables relating to the construction industry in the Annual Reports of H.M. Chief Inspector of Factories.

303. At the Cruachan site 78 of the accidents, with two fatalities, were due to falls from working places of various types and 133, with four fatalities, due to falling material striking workmen. Two of the men who were killed fell some 500 ft from the drilling platform of the Alimak climber—the machine which was described in §§ 73 and 74. This useful and ingenious equipment solved some of the problems in shaft work but, although comparatively new in this country, a number of the hazards likely to arise in its use should have been foreseen and could have been eliminated or reduced. For example, the working platform was small, often not level, slippery, cluttered with loose gear and inevitably there was a fairly large clearance between its edges and the walls of the shaft. But when first put into use there were no guard rails or other methods of preventing men from falling from the platform. Guard rails were fitted later and safety belts issued but the platform could have been partly enclosed and if discipline could have been enforced, the men should have been made to wear safety belts continuously.

304. A static drilling platform collapsed whilst being moved into position and a man was killed. This platform was made of welded steel frames 16 ft high × 10 ft long, which were heavy and difficult to handle where overhead lifting gear was impracticable, and were unstable until braced together. The frames could have been divided into smaller, more easily handled sections and designed so as to be stable as a single unit. A man was also killed when another drilling platform collapsed when *one* horizontal cross rail was accidently knocked out of position.

305. Several other accidents were due to falls from unfenced platforms or through holes with no edge fencing or covers.

306. There were three cases of severe injury when flexible compressed air lines broke away suddenly from a connexion because makeshift connectors had been used —no spares apparently being available at the site.

307. One man was killed and another injured by a sudden movement of sand in the bin of a batching plant into which they had been sent in order to trim and move packed material. Safety ropes were provided but there were no safety belts or harnesses or boatswains' chairs and there was no supervision to see that this work, which was well known to be very hazardous, was done safely.

308. Work in shafts, whether vertical or sloping, was difficult and dangerous but in three separate accidents hazards which could have been removed or reduced were, in fact, greatly increased by the method of doing the work or by the way the equipment was used or installed. In one of the sloping shafts a man was killed because the clearance between the top of a skip used for access and some fixed steel work in the shaft was only 2 in. After the accident this clearance was considerably increased and the top of the skip was protected with a mesh screen which kept riders out of danger.

309. In another fatal case, during the erection of a steel access ladder in the shaft, four sections broke away from the hoisting gear when an attempt was made to pull two of the sections into position before the other two had been bolted to the shaft walls. Contributing causes were a failure of the signalling system to the haulage winch driver and the attempt to move too great a length of the ladder at one time.

310. The third shaft accident was the disastrous one mentioned earlier, where one man was killed and eleven injured. This shaft is shown in Figs 10 and 20 of the Paper and the placing of the concrete lining is described in §§ 109–110. At the time of the accident, concreting was being done about 950 ft below the top of the shaft, the concrete being fed from there down several 6 in. dia. steel pipes built up from 20 ft long sections with victualic-type joints. Each section of pipe was supposed to be secured to the shaft walls by steel clamps and bolts but in fact few such connexions had been made and at other points the pipes were tied with wire to temporary steel framing. When a blockage of concrete occurred near the bottom of one of the

pipes it was filled to the top before any effective signals reached the man in charge of loading. Shortly after this the choked and heavily loaded pipe broke away from its 'fixings' and practically the whole length fell 900 ft down the shaft. The failure to bolt the pipe properly to the shaft walls had been noticed some time before the accident but the instructions which were given to correct this had not been carried out.

311. There were 68 machinery accidents, many of which could have been prevented if guards for the dangerous parts had been devised and kept in position.

312. Transport accounted for 52 accidents and one fatality. This class of accident would continue unless—and allowance was here made for difficult working conditions—better standards of track, signalling and lighting were achieved and unless there were improvements in the design of locomotives and other vehicles, particularly as regards controls and the sight lines of the drivers.

313. These accidents were not peculiar to the Cruachan contract but have their counterparts on other sites, sometimes on many occasions. There was even a case with similar features to the disastrous one described in §§ 305 and 311. The continued repetition of accidents of a similar type was distressing when some at least could have been prevented if use had been made of the unfortunate experiences.

314. Mr Short repeated the suggestion which he had made in discussions on other Papers on construction work: that the Institution should encourage the inclusion of a section on accidents and their prevention in every Paper of this type. In this way experience gained could be of benefit to other engineers and to the construction industry as a whole. The Institution had shown its appreciation of the problems of accident prevention by the award of a Fellowship for the study of safety in civil engineering to Mr G. R. Brueton and by allocating time for presentation of Papers on construction failures and the organization of safety on construction sites, on 25 April, 1967.

Mr E. M. Wilson (University of Washington, Seattle) wrote that the Authors were to be congratulated on an excellent Paper which added greatly to knowledge of Cruachan. In particular the constructional methods and details were excellently described. He had only one question, arising from § 23, which was almost an aside from the main theme.

316. When a pumped storage scheme was being built it seemed only sensible to use it to generate extra units from natural run-off, since the expensive parts, the dam, tunnel and power station were there already. However these units contributed nothing to capacity for peak load operation as this was controlled by the installed capacity, which was presumably chosen after study of the interconnected system. Accordingly the value of the extra units was 4/3 times the fuel cost of the most efficient station available for pumping, i.e. 4/3 times the 'incremental energy cost'. In the case of Cruachan this value must be about 0·4 d/kWh. For 36 million units the annual value was 60 000. Allowing 7½% interest and normal amortization, this indicated a capital value of about 600 000. More than this and the economic justification for the aquaduct system would seem to disappear. Would the Authors say what the collection works described cost? They seemed an excellent bargain at this price.

317. If this approach was not correct, could the Authors say what contribution to firm capacity was made by the natural run-off, or, if the incremental energy cost was underestimated, could they give the correct value?

Mr G. S. Ghanekar (Executive Engineer, Koyna Hydro-Electric Project, India) wrote to enquire whether the alternative multiple arch type of dam was considered. If this type had been considered in the early stages, what made the engineers decide eventually upon a buttress type dam? Was it not more economical to use multiple arches of longer span with a high grade concrete capable of resisting higher stresses, such as at Grandval, France, or Manicuagan, in Canada?

319. The Writer would like to know if any pre-cooling or post-cooling of the concrete was undertaken at Cruachan, and, as an aside from the main issue, the

general British practice in this respect in the construction of buttress dams. How were the joints sealed into the buttresses and the buttress heads?

320. What formulae were used for working out the thickness of the steel lining, especially in zone C (Fig. 20) in the inclined pressure shafts? Was the recent formula developed by Montel[2-4] (considered most rational by many engineers and used, along with Amstutz's formula, in verification of the pressure shaft design for Vianden pumped storage scheme in Luxembourg) used in this regard? If this was the case, what was the design gap between the concrete, rock, steel and concrete, at Cruachan?

321. When the protective paint was chosen, was the behaviour of it against the hardness and PH value of water taken into account, and what type of paint was chosen? Was there a chance to test prototype behaviour of the paint after the pressure shafts had been in commission for a few days, and if so, with what result?

322. What types of drill bits and rods were used in the excavation of the machine hall? What was the life of the drills in terms of feet drilled; and at what interval was sharpening of drill bits done?

Mr W. Wright (formerly of James Williamson & Partners; now Senior Lecturer in Surveying, Enfield College of Technology) wrote that during the oral discussion reference was made to the use of a geodimeter in the tunnel work. Would the Authors please provide more details of this work, with a comment on its accuracy compared to taping.

324. Would the Authors explain the precautions taken to prevent large boulders rolling down the steep hillside.

Mr A. I. B. Moffat (Lecturer, University of Newcastle upon Tyne) wrote that the development of pumped storage in the United Kingdom generating system had been undertaken rather belatedly, and then perhaps with some reluctance. Certainly Britain was not plentifully endowed with sites as suitable for a pumped storage project as Cruachan was. Possible locations, even in England, did however offer a similarly high head/length ratio and were near to base-load generating stations and to areas of high peak demand. Proposals to develop such sites might, however, founder on the debatable issue of amenity. On this subject of amenity, tribute ought to be paid to the achievements of the North of Scotland Hydro-Electric Board both on the Awe Project and elsewhere.

326. The primary considerations governing the selection of new generating capacity were not only those of amenity, but also those of economics, politics, and technical merit. Until recently those factors had been applied principally to the choice of nuclear or fossil-fuel sources of energy for thermal plant. Much less consideration had been given to the advantages which would accrue from the introduction of a considerable pumped storage component to the generating system.

327. Some merits of pumped storage had been outlined by the Authors in § 4. Development of pumped storage potential would, however, add to the flexibility of our predominantly thermal generating plant. This flexibility applied not only to the peak-lopping function of pumped storage, but also to an emergency which required that one of the large new 500 MW or 660 MW steam sets be taken off load quickly at a time of low overall system load. Older low-efficiency steam plant was expensive to maintain warmed through, or on spinning reserve, and was slower in coming on load from this state in order to meet an emergency than was hydro plant. Furthermore, if the emergency outage on the steam plant arose when the pumped storage scheme was pumping, the system variation which could be induced was approximately twice the installed capacity of the pumped storage plant. Thus schemes such as Cruachan, or the proposed Bideford Bay or Loch Sloy developments, served a dual emergency function both as a conveniently rejected pumping load and a rapid means of augmenting the generating capacity to meet demand.

328. On Cruachan Project itself, there were a few points regarding which further information from the Authors would be of interest:

(*a*) What was the capital cost of the Cruachan section of the Awe Project, analysed into its main constituent parts and in terms of (i) civil works and (ii) other works on each of those parts? What interest charges totalled during the construction period, and what interest charge was applicable over the assumed life of the Project?

(*b*) Was 'uplift' at any section on the dam considered, as § 34 suggests, only in terms of the stability of the dam above that section? If not, the concrete was presumably considered, bearing in mind the time element, as a material of low but certainly finite permeability. Would the Authors therefore care to indicate what consideration was given to the effect of this 'pore pressure' on the internal stresses of the dam, and of how the effect on the state of stress was assessed?

(*c*) What assumptions were made with regard to both magnitude and distribution of uplift under the dam, both within the open jointed diorite, and at the concrete/diorite interface?

329. A similar programme of instrumentation to that at Cruachan should be carried out on all new concrete dams. Much useful information could be gained for an outlay on instrumentation and recording of data over a long period of, say, 0·5% of the capital cost of the dam. For the smaller client, such as a water authority, University Departments of Civil Engineering could assist in the planning and execution of such a programme. The information gained would considerably extend our knowledge of conditions of stress, temperature, etc. existing within dams and be an investment for future construction.

The Authors, replying to Mr Crosthwaite, said that the benefits of short high pressure penstocks were not in doubt, but an unlined tailrace tunnel (driven almost wholly in the somewhat friable phyllite) would have introduced risks of carrying broken rock into the pump whose water passages were very restricted. The contraction joints at Cruachan Dam had proved very effective and the downstream face was almost blemish-free. The horizontal joints too had turned out to be very good, but here no special measures were introduced other than careful scabbling, cleaning and grouting.

331. Mr Crosthwaite's suggested alternative treatment for drilling and grouting at Cruachan Dam (i.e. wider spaced deeper holes, higher pressures) was interesting. Faced with a similar problem at Finglas Dam Mr Paton seemed to have closed the centres of the holes and adopted somewhat shallower depths of hole. It was doubtful therefore if there is general agreement in this field.

332. At Tumut hydro-electric project in Australia and Woh project in Malaya (Mr Seddon) horizontal rock stresses were found, as at Cruachan, to be appreciably in excess of those that could be accounted for by overlying rock weight. Hast and Nilsson in their Paper to the 1964 ICOLD Congress expressed the view that, probably over most of the world, the earth's crust was in a state of very great horizontal compressive stress and that it was reasonable to suppose that these forces might be responsible for many unexplained geological and geophysical phenomena. Perhaps this gave the explanation.

333. The underground works were complicated but some of the complications derived from the numerous temporary access galleries provided. The Authors were not quite clear what Mr Crosthwaite meant by 'more chamber and less tunnel' but they were definitely of the opinion that large deep chambers should be kept to a minimum.

334. The reluctance to use too much bentonite was explained in the Paper.

335. In § 194, in connexion with the Alimaks, Mr Crosthwaite referred to the use of electric drive successfully and safely in Sweden and remarked that 'they had every

bit as good safety records as we had'. This was perfectly true; but in Sweden work of this nature would be carried out by a carefully picked crew of trained and experienced miners, and this was the overriding reason for their better statistics. At Cruachan one pair of Swedish crews worked for seven months in the raises and incurred no reportable accident. In the British crews there would always be some more or less inexperienced men. To anyone who had first-hand knowledge of the effects of accidental initiation of a round of explosive the use of electric power was a thing to be avoided if possible at anything less than 150 ft from the face.

336. The general pattern of rock bolting was decided by the Engineers, but the Contractor was obliged to introduce any rock bolting required in the interest of safety. Pulverized fuel ash was not used in any concrete—in fact the tender selected for the dam showed the use of this material to be more expensive.

337. The Authors agreed with Mr Roseveare that the stature of Cruachan merits further publications—for example, hydraulic surge, the design of the conduit system and perhaps the design of the aqueduct system—all of which are discussed too summarily. Space was an overriding problem.

338. The Authors had insufficient cost information at their disposal to complete Table 2 provided. Work on civil mechanical and electric contracts continued at the time of writing and no doubt this information would be published when final costs were known.

339. Table 3 referred to losses and the following information would provide the comparison with other stations:

Losses %	*Cruachan*
(*a*) Turbine	9·9
(*b*) Pump	9·1
(*c*) Generator	1·9
(*d*) Motor	1·6
Product	20·9
(*e*) Conduit	1·7
(*f*) Other losses	2·4
Net efficiency (excluding transmission)	75·0

340. Of the total conduit losses 70% occurred on the pressure side under maximum flow conditions.

341. In making this comparison it was important to remember that the figures quoted were not the best machine efficiencies but a weighted mean having regard to mean head (both pumping and generating) and to the periods during which the station was operating at other than full load.

342. The subject of steel linings vertical or horizontal shafts was dealt with in general terms later. The 'easy gradient' suggested by Mr Roseveare for the high pressure penstocks was attractive and was seriously considered but in the circumstances was not feasible because of the decision made (§ 125) to limit the head to which the concrete lining was subjected to 1000 ft. Experience at Cruachan indicated that this head could be increased if the rock was good and well grouted. If this was done, the gradient of the steel linings could be flattened and access to the bifurcation became very much cheaper.

343. In the planning stages no serious consideration was given to a surface or subsurface station. Apart from the loss of amenity and the difficulties of a restricted site, the necessity of carrying the excavations to a depth of about 180 ft below the adjacent loch in heavily bouldered moraine or phyllite together with the very long high pressure system virtually ruled out this arrangement. It was very doubtful if time of construction would have been saved.

344. No storage allowance was made at Cruachan for firming up conventional hydro-electric stations in dry years although in practice some small benefit in this

direction might accrue. In the Paper, § 8 was intended to convey that this matter merited some thought if a pumped storage project was in the planning stages.

345. The main control gate when tripped would close by gravity under conditions of full flows and only required pressure for opening. However, hydraulic connexions were incorporated in the pressure cylinder so that top (or closing) pressures could easily be arranged if necessary.

346. Mr Guthrie Brown's study (presented by Mr Keefe) of the development of water seals over the years had obviously involved a good deal of work and the Authors thanked him for this useful record. The effectiveness of the simple keys of Sloy Dam varied considerably over the period covering summer expansion and winter contraction of the dam. Cruachan seals were much better but, of course, cost a good deal more money. When dampness or minor water runs appeared on vertical joints of dams they had usually little structural significance and the question was largely one of outlay to improve appearance.

347. Mr Dickerson and several other contributors raised the question of whether inclined shafts or vertical shafts and horizontal tunnels should be adopted. The attraction of the inclined shaft was not only the shorter length but also the rapidly diminishing penstock and tunnel pressures that were associated with it. At Cruachan both the inclined or alternatively the horizontal/vertical arrangement were priced by tenderers and the selected tender showed not a marginal difference but something like 19% in favour of the inclined shaft, which was adopted. Inferior rock conditions, lack of robustness of the Alimaks, failure of full face driving methods made this shaft work much more difficult than anticipated, and coupled with the fact that any operation was manifestly more complicated in an inclined shaft, the Authors believed that at Cruachan the more conventional and well-tried vertical and horizontal arrangement might have been better. For the future they do not necessarily hold this view and were very interested to hear from Mr Hunter that the inclined arrangement was finding much favour on the Continent and in North and South America. There was no general solution to this problem—each case should be judged on its merits.

348. To provide additional access to the bifurcation required a 16 ft × 16 ft tunnel whose minimum length (1:6 gradient) would have been about 2500 ft. As the total length of the two high pressure concrete shafts was about 2200 ft this was a heavy oncost. In addition, miners were in short supply at that time because of intensive tunnel work elsewhere. Had it been practical substantial advantages would have accrued both on the concrete lined and the steel lined shafts. If a head on the concrete linings greater than 1000 ft had been acceptable the level of the bifurcation could have been lowered and a very much shorter gallery would have been possible.

349. In § 226 Mr Dickerson referred to the difficulty of setting out in the shafts. Mr Hill described later in the discussion the principle adopted for setting lines and levels forwards. Whilst satisfactory survey results (within 4 in. for direction) were obtained, the actual driving was at one point between 7–8 ft off line. The geometry of the bifurcations was very difficult to reproduce satisfactorily and simplifications of this would have been helpful.

350. Much thought was given to whether a central or end loading bay should be adopted. One dominating argument in selecting an end loading bay was the possibility of Cruachan being extended at a future date when the present loading bay would be central on the extended station.

351. Replying briefly to Mr Dickerson's remaining points: steelwork erection would have been considerably simplified if the temporary crane had been available for this purpose, but the following were the disadvantages:

(*a*) The rail and girder arrangements would need to be incorporated in the roof arches at the time the latter were being constructed. Subsequent blasting in the cavern would undoubtedly have created damage.

(*b*) To provide block-to-block clearance the span would have been 50% greater than the arrangement adopted.

(c) After steelwork erection had been completed the temporary crane (operating above the main crane) would have been of little further use and would probably have to have been re-erected in the position selected. A monorail system similar to the above was considered but abandoned because of its inaccessibility and the dangers from blasting.

352. The shuttering to rock on each side of the roof arch rib was not scribed to rock. The arch soffit shutter carried prefabricated side shutters and these were made up quickly to the rock by the use of steel mesh hyrib panels. The size of the space between the roof sections (2 ft) was designed to give reasonable access for the striking of the end shutters.

353. Bearing in mind the diversity of the quality of the rock at Cruachan the various methods of measuring physical properties gave reasonable agreement. Seismic methods for finding the Modulus of Elasticity yielded substantially higher results than other methods.

354. Mr Roberts's question on storage was answered by the fact that Cruachan storage was designed on a weekly basis and the reservoir was replenished both by night and at week ends which permitted much more than four hours' full load generating output by day.

355. Little or no storage was kept in reserve for regulating the diverted and natural run-off. In times of heavy peak demand the reservoir would normally be well filled by Sunday night after week-end pumping. The generation consumption Monday to Friday exceeded the volume restored by night pumping in this period so that the reservoir level fell and adequate freeboard existed for accommodating aqueduct and direct catchment inflow. Moreover, one set generating on full load (1250 cusecs) could absorb the aqueduct and direct catchment run-off in times of very heavy rainfall.

356. The abbreviated reference in the Paper to 'the time the concrete mass commenced to act in a homogeneous manner' (§ 54) referred to the fact that at Cruachan during the first three weeks of observation on the vibrating wire gauges the readings on the parallel and identical gauges were most erratic, but after that they started to act with much more unison.

357. The Authors found it difficult to answer Mr Roberts's last question as to whether high stresses existed on Monar arch dam. Strain observations during plastic conditions did not necessarily imply stress and one difficulty lay in determining when (if ever) plastic conditions cease.

358. Mr Hill carried heavy responsibilities indeed as Agent during the civil engineering operations of Cruachan power station and his first-hand account of Alimak experiences, coming as it did from the man who had to do the job, was most valuable.

359. Some of the information from Woh underground station in Malaya as described by Mr Seddon was interestingly similar to some of the problems at Cruachan. Of particular interest was the fact that at Woh no concrete roof arch was necessary in the machine hall.

360. Smooth blasting was used extensively in trimming the bare rock walls of the power station as nearly vertically as possible so that the arch springers were not undermined by overbreak. As would be seen from Fig. 34 the final effect is very pleasing. The operation of smooth blasting the last 2 ft of excavation adjacent to the walls was specified, scheduled and priced.

361. The following stresses were used for the Coltuf 32 steel of the high pressure penstocks:

$$\frac{\text{Working}}{\text{Yield}} = 0{\cdot}5$$

$$\frac{\text{Working}}{\text{Ultimate}} = 0{\cdot}3$$

FIG. 34: SMOOTH BLASTING USED IN TRIMMING BASE ROCK WALLS IN THE POWER STATION

362. Mr Seddon's remaining questions were answered briefly as follows:

(a) The spiral casings were concreted without internal pressure and without an elastic filler between the concrete and steel.

(b) The minimum thickness of the tailrace concrete lining was 6½ in. at rock points. No areas of more than 4 sq. ft were permitted inside the 10½ in. nominal lining.

(c) The tailrace tunnel can be dewatered from a pump chamber immediately behind the outfall gate (Fig. 24). The tailrace tunnel slopes down to this sump.

(d) The inclined shafts and penstocks could be inspected from and maintained by a specially designed carriage lowered by winch from the bend near the dam, into which was incorporated a 6 ft × 6 ft access door.

363. The contribution made to the discussion by Mr Short was useful, and should focus the attention of engineers and clients on the safety aspect of their work.

364. Apart from the disastrous affair mentioned in § 304, many of the accidents were caused by lack of experience and training in the men employed. The employment offered by the construction industry was casual by tradition, and in times of full employment there was less inducement for men to stay in the industry and to become skilled and aware. Again, as far as civil engineering construction was concerned there was no continuity. For instance, by the time the next large rock tunnelling work was to be done in Great Britain the men who worked at Cruachan would be scattered far and wide, and the men available would be to a great extent inexperienced. Training where it was possible would be a great help in accident prevention in the industry as a whole, but it was difficult to see how training in the skills required in tunnelling could be taught anywhere but in a tunnel.

365. The overall reportable accident frequency rate was 7·14.

366. The Authors thanked Mr Newman for his comparisons between Ffestiniog and Cruachan. His points about seasonal storage, regulation of run-off and firming up had been answered elsewhere, and it would be desirable to have more operational experience before giving specific answers to the questions of load factor and regime. It was anticipated that the overall annual generating load factor would be about 11% and the weekly regime would follow the broad pattern described earlier.

367. The Authors agreed that full designed discharge over the spillway of Cruachan Dam was something that was most undesirable, but it was created at very little extra cost and only to be invoked in the very remote emergency of overpumping to a full reservoir in flood condition of inflow.

368. The maximum gradient that heavy plant vehicles had to negotiate was 1:19. Reference to gradients of 1:8 were only applicable to temporary roadways for civil engineering construction traffic.

369. The spiral casings were bedded solidly in concrete and shrinkage spaces lightly grouted. Interposing a flexible material between the casing and the concrete could, in the Authors' view, encourage steady vibrations which solid concrete surrounds would damp out.

370. After several alterations, the hydraulic model of the outfall on Loch Awe gave a very even distribution of pumping velocities. As Mr Newman infers, it was more difficult to arrange equally good distribution of velocity during generating flows, but the maximum generating velocity (very local) was of the order of 2 ft/s. During generation there was no risk of drawing smolts into the intake.

371. Mr Paton's grouting procedure at Finglas Dam seemed very logical. At Cruachan Dam the relative permeability of the top 20 ft of grout hole and the length at depths between 40–60 ft was given below:

	Depth of grout hole	
	0–20 ft	40–60 ft
Approximate number of holes where no initial water test pressure could be recorded . . .	66%	16%
Average percolation of those holes where pressure could be built up	0·9 lugeon*	0·15 lugeon

372. After final grouting the acceptable percolation was 0·034 lugeon.

373. The Authors shared Mr Fulton's view that designers must not be over-influenced by conditions as they exist at the initiation of a project but must look forward at least to the time of first commissioning and preferably further.

374. The erection and welding of the steel linings at Cruachan introduced constant problems and with attendant fall-back in progress. On the low pressure side of the machines steel linings adjacent to the draft tubes and the surge chamber were necessary but intermediately greater use could probably have been made of well grouted concrete lining provided that there were no risks of water percolation into the transformer gallery cable shaft and machine hall. The erection of the high pressure penstocks would have been simplified if a flatter gradient had been possible.

375. To omit steel linings was a desirable objective, and this had been carried out successfully by prestressed concrete pipes on schemes of lower head.

376. In the Paper, § 120 made reference to the possibility of water hammer at Cruachan giving rise to twice the maximum static head of the reservoir. During commissioning tests upsurge pressures of 1650 ft have been recorded at the lowest high pressure pipe, and prestressed pipes would have to be designed with working

* Where water test pressures were less than the 10 kg/sq. cm defining a lugeon, the observed percolation has been increased in the ratio 10 kg/sq. cm : actual pressure.

stresses for this condition. In addition, they would have to withstand the longitudinal tension of about 1800 tons from the valve load.

377. It would be necessary therefore to arrange the prestressed pipes to be in a straight line co-axial with the spiral casing entry pipe and taken by tunnel into massive rock to an extent that would ensure sufficient anchorage and, on the assumption that sufficient rock strength existed, to be able to continue upwards thereafter with thick in situ tunnel linings capable of taking a maximum head of about 1650 ft.

378. If the weight of the pipes so designed was restricted to, say, 20 tons the length of each pipe would be about 4 ft 6 in. Ducts would be incorporated in the pipe wall through which longitudinal tensioning could be carried out after finally aligning the pipes.

379. Estimates show that this arrangement might be somewhat cheaper than steel linings and further savings would be achieved in the protective treatment, but careful consideration would have to be given to ensuring that the numerous joints were completely watertight. The Authors believed that this subject merited some experimental work.

380. Mr Blackmore's first point is answered by reference to the design assumption described in § 122 whereby in the vicinity of the machine hall and lower galleries little external water pressure could build up because of the relief offered by these cavities.

381. His suggestion that a full width bevel on the bottom of the main control gate would have required less power on opening was quite correct if only the reservoir pressure was considered.

382. However, the gate was designed that if inadvertently lowered during pumping it could not close fully. By adopting the bevel as shown in Fig. 3 some uplift was available to arrest the descent, whereas the full width bevel mentioned would preclude this uplift and in fact lead to downpull.

383. The V-shaped bottom of the gate was therefore a compromise between the two conditions.

384. The majority of the intakes were either screenless or incorporated a screen that was steeply inclined and over which most of the potential blocking material would pass.

385. A cut-off wall, whose lower edge was always submerged in the impounded water, was provided in front of all screenless intakes and the floating material thus trapped was carried over the spillways during floods.

386. One major intake in a stream where boulders and gravel were liable to be troublesome was designed to be self-scouring using the principle of causing the scour gate to open when the gravel accumulated to such an extent that water was by-passed to a tank counterbalancing the gate.

387. The Authors much appreciated Mr Hunter's very interesting and pertinent contribution on the subject of shafts to which earlier reference had been made.

388. In answer to Dr MacGregor's various questions on the high pressure penstocks:

Zone	*A*	*B*	*C*
Diameter	8 ft	8 ft	9 ft
Pipe thickness	$1\frac{3}{4}$ in.–$1\frac{5}{8}$ in.	$1\frac{5}{8}$ in.–$1\frac{1}{2}$ in.	$1\frac{1}{2}$ in.–$1\frac{1}{4}$ in.
Material	Coltuf 32	Coltuf 32	B.S. 2762 N.D.1
Electrodes	Babcock A_2	Babcock A_2	Fontrex 35

389. The electrodes were baked for a minimum of two hours and were not cooled down before welding. Other precautions taken included:

(*a*) electrical preheating equipment designed to keep the welds hot from start to completion of the weld;

(*b*) backing straps to the circumferential welds to ensure proper alignment and minimize water entry;

(c) all welds were ground flush and subjected to 100% ultrasonic inspection and, where necessary, radiography.

390. On the question of using greater thicknesses of plate if circumstances required it, the Authors considered this might be possible provided the plate ends were free of all blemishes and the greatest precautions observed.

391. Dr MacGregor expressed the view that the stress of 870 lb/sq. in. in Cruachan Dam brought out by the glass cylinder method seems high for an unloaded condition. Dr Williams of Sheffield University who advised on the incorporation of these gauges had confirmed that notwithstanding the deterioration of the polaroid material the gauges have accurate readings and in his view a stress of this magnitude existed. This was not doubted but the Authors suggested that the presence of the glass cylinders (not subject to shrinkage) in a mass of shrinking concrete may set up these local stresses which would not otherwise obtain.

392. In clarification of the strain gauge measurements in Fig. 7 the upper line in both diagrams represented the change in the length of the gauge which was built into a pocket in the dam and therefore not influenced by external loading of the concrete such as weight of concrete (as the dam grew in height) and water loads. The lower line represented the change in length of an exactly similar gauge built into the mass of the dam and therefore subjected to the effects of external loading, temperature, plastic deformation and all other influences. By difference it was hoped to deduce the effect of external loads but the differences between the two lines were too large to be wholly attributed to those, and it was evident that the results needed to be corrected for complex temperature effects.

393. Nevertheless the trends were of interest. For example, in the period until about the end of 1964 while the weight of the buttress was steadily increasing but no water loads had been applied the two curves for the upstream gauges were diverging, indicating increasing compression, whereas the curves of the downstream gauges widened only very slightly.

394. In early 1965 when reservoir filling commenced the opposite happened, indicating reduced compression upstream and increased compression downstream.

395. The tailrace tunnel overbreak was 11 in.

396. Mr Wilson was not wholly correct in assuming that the run-off from the collecting aqueducts contributed nothing to the capacity for peak-load operation, although the contribution was not very significant.

397. It appeared that Mr Wilson assumed pumping units to be available at 0·3*d* per unit. As the thermal stations with best efficiency were correctly being used on base load duties, the stations available for pumping operated somewhat less efficiently. At the time of writing the Authors believed that the unit cost of night pumping was approximately 0·5*d*, which converted to a generating unit brings out a figure of 0·67*d*. This was an average cost over the period of pumping and at the time when pumping commences or ceases the pumping costs were even more.

398. The justification for collecting water by aqueducts could not be made against the generation costs resulting from the normal pumping cycle but against pumping at other periods of the day when pumping costs could rise very sharply and be completely uneconomic.

399. In the case of Cruachan it must also be borne in mind that some of the water collected by aqueduct for Cruachan augmented the output of Interawe Station.

400. The following were brief answers to Mr Ghanekar's questions:

(a) Previous studies showed that, in Scotland, the intricate shuttering and reinforced concrete of multiple arch dams showed no economy when compared with buttress dams.

(b) No cooling measures other than continuous water spraying were used in Cruachan Dam. Flake ice cooling was used in the thick concrete linings of the pressure shafts.

(*c*) The method of sealing buttress joints was shown in Fig. 4.

(*d*) The design of the steel linings in Zone C was based on formulae similar to those devised by Amstutz. Due regard was paid to the shrinkage gap between the steel and concrete. Careful control of diameters and curvature of the pipes was exercised during fabrication.

(*e*) P.H. value of Cruachan water was about 6·8. An epoxy paint was used on shot blasted steel and inspections to date show this to be satisfactory.

401. Mr Ghanekar refers in § 323 to the types of drill bits used. About 80% of the drilling on the whole of the excavations was carried out with rods supplied by Messrs Holman Bros. These were tipped with tungsten carbide. The majority were 7 ft long with 33 mm dia. chisel bits. Holman Silver III drilling machines were used. Altogether the footage drilled was over 250 miles.

402. Mr Wright asked for information on geodimetric survey. The Geodimeter was first used at Cruachan on the main hillside survey for measuring the base line, which would have proved virtually impossible by banding. It was used also for checking the main underground survey which had previously been done by conventional methods. It proved very successful in checking the setting-out in the sloping shafts, the original setting-out being done with forward and back stations being under 1000 ft apart (on the slope). This would give some cause for inaccuracy in a shaft with a driving length of some 1200 ft.

403. As it was necessary to know, very accurately, the plan distance and hence the level of the face, the Geodimeter was used to check stations every 400 ft apart. This was a comparatively easy operation compared to banding this length, which would have proved very impractical because of access up the shaft. The accuracy of the Geodimeter was $\pm\frac{1}{4}$ in. in its entire range, which of course is much better than can be obtained by other methods.

404. Boulders and rock fragmentation during the construction of the access road to the dam proved very troublesome on account of the steep slope of the hillside along which the road was constructed. It was imperative that no boulders or blasted rock rolled across the main railway line and trunk road at the bottom of the slope, and to ensure this temporary barriers consisting of heavy pickets and sleepers were constructed on the hillside immediately below the area in which excavating machinery worked, and all rock areas were covered with heavy steel netting prior to blasting.

405. The Authors had read Mr Moffat's interesting contribution and agreed wholeheartedly that not enough consideration had been given to the advantages which accrue from the introduction of pumped storage in the generating system. In this connexion the merits of pumped storage were clearly brought out by Mr E. J. K. Chapman, B.Sc., M.I.C.E., in his article in *Water Power*, April 1963.

406. The emergency function of pumped storage, whereby it could assist the system during pumping to an extent equal to twice the installed capacity, was a further asset that perhaps did not get due recognition.

407. The Authors regret that as they did not know the overall costs of Cruachan they were unable to answer Mr Moffat's first question.

408. At Cruachan Dam uplift was only considered in relation to stability.

409. Pore pressure was not taken into account in the stress analysis of the dam. Pore pressure would exist to some extent within the 3 ft 6 in. rich concrete forming the upstream face, but it was very conjectural to what extent they built up throughout the remainder of the dam because of:

(*a*) the relative impermeability offered by the upstream face concrete, and

(*b*) the relatively high permeability of the hearting concrete and the very short leakage paths to atmosphere inherent in a buttress dam.

410. If pore pressures were taken into account it seemed equally important to recognize the offsetting effects of the dilatations associated with wet concrete.

411. It was assumed that the area upon which uplift would act was the product of

half the length of the buttress section and the minimum thickness of the dam. Uplift pressure was taken to vary linearly from full reservoir pressure at the upstream face to zero at the line of minimum thickness. The assumptions at the junction of the concrete to diorite were no different from those at other sections of the dam.

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