

Cockenzie and Longannet power stations: novel features in the design and construction

L. H. BAKER & D. H. McPHERSON

The Chairman, Mr W. A. Fitzherbert

I should like to invite discussion on the Longannet site. Having been in Cornwall and other places recently examining possible sites for power stations, it is indeed a pleasant change to hear about the Longannet site which appears to have all the attributes for a perfect power station location. As far as I can see, it is a level area, with direct cooling from the Firth of Forth, with a conveyor connexion to a coal mine conveniently at hand, and a railway behind the station for transport of loads. Finally, of course, Longannet has excellent foundations on rock. I am sure it would be difficult to find a better site anywhere.

Mr J. S. Oakes, Kennedy & Donkin

I have read with immense interest the account of the novel features in the design and construction of Cockenzie and Longannet. Both of these stations have been built on reclaimed land and both have had their problems.

102. While I have had rather a strong connexion with the former station, I found it particularly interesting to read of the formations of the sea walls, both the permanent site sea wall at Cockenzie and the walls protecting the ash lagoons, with reference to the hearting in each case. Cockenzie used the burnt red blae while Longannet used the unburnt black shale. The red is more pervious than the black and the walls of the earlier station took longer to seal.

103. That fact, oddly enough, was evident in some of the Contractors' storage yards at Cockenzie which were levelled from a black shale bing. During inclement weather such surfaces held the water and had to be covered with a good 8 in. of broken rock which was rolled into the surface. Yet other areas which had been finished in a carpet of red blaes drained well and were most satisfactory for storage areas. I feel that this is a lesson which has been learned on the uses of this waste product. Perhaps the Cockenzie walls would have sealed quicker had black blaes been used.

104. On the wagon discharge house at the coal yard, I think the Authors will agree that construction was slow due to the complexity of the concrete structure demanded by the use of the first 1000 ton payload trains with bottom opening hopper wagons. I am glad to see from § 38 that recognition has been given to the problems of the Contractor, because he certainly had problems.

105. The very heavy reinforcing bars necessary in the narrow but deep rail support beams precluded the use of vibrators in some areas. In placing concrete in the junction between the cross-beams and the walls the operator was literally spooning the concrete between the bars and hand-ramming with a wooden stave to consolidate. The process was slow and the building was late, but fortunately (if fortune can be attributed to a misfortune which occurred later) coal supplies were not required in the station for a further ten months after the programme date.

106. I have referred to a misfortune, and I suppose that no one can mention Cockenzie without bringing to mind that grim and almost unique occurrence of the

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fracture of the first boiler drum while undergoing hydraulic test. No one was injured, but those who suffered shock most were the bricklayers lining the ash hopper, who were deluged by tons of water which fell out of the gaping hole of the drum. There was very little explosion, and although the drum parted, it did not fly apart in great pieces. Once the remainder of the drum had been extensively examined, it was found to be too much damaged and thoughts turned to the question of replacement. There were three possibilities: (a) the boiler front could be dismantled and a new drum lifted up, which would be followed by months of reinstatement of the boiler front; (b) small sections of drum could be lifted into position, and welded in, which might have required turning during welding, and the whole finally stress relieved (this appeared to be a lengthy and doubtful operation); (c) a new drum could be slid into position from the side.

107. Fortunately, if one may use that word again, this was the No. 1 boiler and the gable end of the building was a free end with plenty of space beside it in which to erect a lifting structure. It was decided to use this method, drawing on the experience at a previous Scottish station and using the No. 4 boiler drum, then approaching completion in the boilermaker's works. It fell to the Authors and their structural steelwork designer to develop a structure capable of lifting 165 tons of drum 170 ft into the air and sliding it longitudinally into the boiler house. Such a structure needed ample foundations, two of which fell across the deep drains. These in turn had to be exposed, haunched and strengthened to take the expected load at each leg. The structure itself was slender and designed at great speed using known and available sections of steelwork lying in the stockists' yards.

108. Thus it was possible to have the gantry designed, manufactured and assembled for lifting the replacement drum in the space of only five months. The drum lift itself was delayed one day due to high winds being forecast for the area. However, next day, during quieter weather, the lift was made starting at 4.00 h and watched by the Authors' representatives. The drum itself was entered halfway into the building before the shift retired for the night. The following day the winds blew up again, but from a different direction. Fortunately, however, the drum was safely inside the building.

109. Great credit must be given to all who took part in the operation for the fact that the boiler was ready again for hydraulic tests only 11 months after the catastrophe. A great part of this is due to the efforts of the civil engineers in helping their mechanical brethren in their hour of need and designing the facility which overcame this difficult problem.

110. One more point in which I was interested in reading in the Paper was the 16 000 ft long flume for cooling water discharge at Longannet. I wonder whether the disturbance of nature here will be maintained or whether, in course of time, the river will fill up this channel. Are there any signs yet of its doing so?

Mr D. O. S. Canales, Sir Robert McAlpine and Sons Ltd

I would like to record the great appreciation that the Authors showed of the needs of a main contractor on projects of this size—the big areas required for batching plants, reinforcement yards and all those facilities which go to make the job so much more manageable, from the point of view both of labour relations and of safety on the site, of which we are these days very conscious. The facilities which are given to the main contractor to enable him to do his job are of great importance on major power station projects. Both these sites were tight from the point of view of space. The space was pretty well at a premium, and it was very well thought out.

Mr S. A. Rossiter, Merz and McLellan

My firm's association with the Authors was at Longannet power station and, as elsewhere, we feel that our collaboration with them has been efficient. The following

information which has arisen since the Paper was written confirms the Authors' remarks on the Cockenzie programme.

113. The Longannet sets are 600 MW, the biggest so far commissioned in Britain. They contain certain unconventional features which are not included in the CEGB's 500 MW machines, notably a specified peak rating of 700 MW.

114. The Authors have indicated that the Longannet commissioning programme was between 1969 and 1971. The No. 1 unit boiler drum was lifted in May, 1967; turbine erection began in December, 1967; safety valves were floated in December, 1969; initial steam went to the turbine on 7 January, 1970; initial synchronizing a fortnight later; and a steady loading of 580 MW was achieved, with a peak of 690 MW, on 27 and 28 May, 1970. The last report I have on steaming is that in the week ending 27 November, 1970, the maximum load on No. 1 machine was 615 MW.

115. In No. 2 unit, the boiler drum was lifted in December, 1967; turbine erection began in November, 1968; safety valves were floated in November, 1970; and steam was delivered to the turbine on 8 December, 1970. There were no serious technical setbacks in the construction of this station. This is, in part at least, due to additional specified features as well as to rigorous inspection.

116. In the case of the Cockenzie sea works, the Authors refer to a partly exposed coast, with considerable interference from storms. I wonder whether these storms were seasonal and whether the work was closed down at all during the winter season. Our experience at Blyth and Hartlepool is that the incidence of storms along that coast is not seasonal, and the only encouragement to a contractor to close down on work of this kind during the winter is the shortness of daylight working hours.

117. On the subject of structural steelwork, the Paper indicates that the Longannet boilers have twice the steam capacity of the Cockenzie boilers; their weight is also twice the weight of those at Cockenzie (10 000 tons per boiler compared with 5000 tons per boiler). The span across the boiler opening is about 10% more at Longannet and the steelwork tonnage has come up to the pro rata rate for the tonnage at Cockenzie.

118. The boiler suspension is the only notable difference. Because of limitations of plate thickness, highway load gauges and erection crane capacities, lattice girders were adapted for the Longannet main boiler suspension. One large spine girder and eight subsidiary ones being erected piecemeal, it was possible to build a spine girder with a total weight of 225 tons. Similar erection cranes were used as for Cockenzie.

Mr E. R. Harding, Mott, Hay & Anderson

Having spent two years working at Cockenzie power station, I hope one or two construction details will be of interest.

120. As stated in the Paper the cooling water tunnels were driven in sandstone and shale beds under the estuary and provision was made for compressed air. In order to provide sufficient room at the shaft bottom to manage the skips, the single bulkhead was built about 60 ft into the tunnel. About 60 ft further in a fault was struck running across the tunnel, so steel ribs and lagging were erected to support the ground in this area. Unfortunately a secondary fault was then encountered running parallel with the tunnel so that the left shoulder tended to fall away. We therefore changed to precast concrete segmental lining, a stock of which had been provided, and that was built tight up to the face. There was virtually no damage from blasting and the accuracy of drilling was improved.

121. There were a large number of misfires which were found to be due to the high salinity of the water entering the tunnel breaking down the insulation on the detonator leads. The trouble was largely overcome by using alkethane clad detonators.

122. The tunnels were lined with in situ concrete using a pump set up at the batching plant. The concrete was pumped about 100 ft to the top of the shaft, down the 100 ft shaft and over 600 ft along the tunnel with very little difficulty.

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123. Whilst walking on the embankment to the ash disposal lagoons when an electric storm was in the offing I heard the whistling of the electricity being discharged from steel reinforcing rods which we used to mark the profile. It was a very clear practical demonstration of the way lighting conductors work, which I learnt at school.

124. Finally, I should like the Authors to explain why the layout of the power station is such that the coal has to be conveyed from the coal store on the south, round to the boilers on the north, whilst the cooling water has to be pumped to the turbines which are situated as far from the intake tunnels as possible.

Mr E. W. Hume, The Mitchell Construction Co. (Scotland)

My company was associated with the construction of the sea wall at Cockenzie. My chief memory of the site is the most appalling weather during the summer of 1962.

126. In the Paper, the condition of leakage through the sea wall after closure (Fig. 3) appears to have been somewhat oversimplified. The graph does not indicate that a pumping installation of 10 000 gal/min was installed behind the sea wall after closure at a leakage rate calculated by the graph of 24 000 gal/min. Do the Authors consider that if it had not been for the advent of the 3 day storm condition, the pumping installation would have had to be increased to something more than a leakage rate to establish unidirectional flow through the sea wall?

127. On the Longannet contract, we were concerned with the construction of the intake headworks, the tunnels and the pumphouse. In the tender documents the Consultants made provision for the Contractor to price for dealing with water leakage in excess of 600 gal/min in each tunnel. During construction the average leakage was approximately 300 gal/min, with the exception of the last 50 ft of tunnel at the intake shaft, where a leakage of approximately 1000 gal/min occurred. The assessment of the leakage contained in the tender documents proved remarkably accurate in practice, and it would be of interest to know on what basis the assessment was made.

128. With regard to the increased rate of leakage at the intake end of the tunnel, do the Authors consider that the sheet piling work through the boulder clay for the cofferdam contributed in any way to this condition, or was the leakage attributable to the fact that rock head falls rapidly away from a point 750 ft north of the wall embankment (Fig. 18)?

Mr G. Hargreaves, HM Principal Civil Engineering Inspector of Mines and Quarries

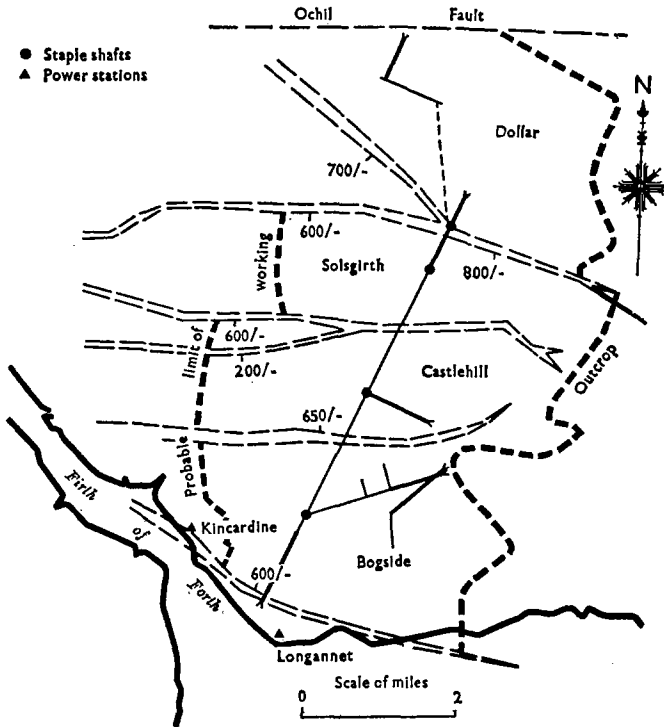
My remarks will be confined to works associated with Longannet. I regret that I cannot fulfil the Chairman's desire to find another site as ideal as the one which I should like to discuss, but I am sure that some of my mining colleagues would be very glad to do so!

130. Longannet power station is a device for turning coal into electrical energy on a fairly large scale. I would like to give a brief description of some of the associated works undertaken by the mining industry.

131. At peak load, this station consumes coal at a rate of about 20 000 tons/day, which would keep the Institution's building going on central heating for several hundred years.

132. Figure 22 shows the Longannet mine complex which serves the station. It is not designed to produce the peak requirement, about 12 000 tons every 24 hours, but the great bulk of the coal requirements of the station. It consists of four mines: Bogside, Castle Hill, Solsgirth and Dollar, all designed to be connected together to a main conveyor road so that the coal produced in all four mines reaches the station bunkers without ever seeing the light of day until it comes out of the ground at Longannet.

133. The four points in the figure marked with large black circles are those at which the coal from various mines reaches the main conveyor belt. Only three of



[Courtesy Mining Engineer

Fig. 22. Mines in Longannet area

them are in commission. The Solsgirth mine has rather unkindly pirated some of the Dollar coal and the Solsgirth bunker has not been constructed, and may not be constructed.

134. At the long wall face (650 ft in length), the shearer works down the face loading the coal directly on to the armoured face conveyor, which is jacked forward after the machine has passed. When the shearer has reached the end of its run, the face is all but ready for the return trip, with the shearer again loading the face conveyor as it works. From the face conveyor the coal is conveyed by two scraper conveyors and fed on to a main 36 in. belt conveyor which travels through the main roadways of the mine to the storage bunker.

135. The 36 in. belt conveyor brings the coal and loads it into the bunker. The staple shaft, or bunker, is about 100 ft deep and 20 ft in diameter with a 2 ft concrete lining. It is directly above the main conveyor road and can feed directly on to the main cable belt conveyor.

136. At this point there are five sets of instruments that measure, respectively, the quantity of coal reaching the bunker, the quantity leaving the bunker, the amount in storage at any one time and, particularly important, the ash content and the water content of the coal at that time. This is particularly important because the boilers can accept coal only up to a specified limit of ash and water content. It is part of the complex operation to ensure that these limits are at all times observed.

137. All the readings from instruments are passed to the control station at

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Longannet, where they are fed into a computer. The computer's job is not only to ensure the quality but also to prevent the belt from being overloaded, observing that each bunker could load it to capacity.

138. I must acknowledge the information and advice which I have received from the Production Director and Production Manager at Longannet.

Mr J. Stevenson, Balfour Beatty & Co. Ltd. Scotland

As Area Manager for Balfour Beatty, the contractors responsible for the construction of the ash ponds at Cockenzie and also the Cockenzie cooling water works, I should like as my first observation to say that sea and tidal works are almost invariably at some risk during some stage of construction and that it is a great tribute to the design which was adopted at the Cockenzie ash pond sea wall that the works were considerably protected by the type of design chosen at most stages of construction.

140. The Authors have given details of tide and pond water levels, and it is interesting to see the substantial decrease through time in pond level fluctuation. However, this still indicates that there is a constant movement of water through, and no doubt also under, the sea wall. The Authors state that elaborate and costly measures to secure an impervious wall were not necessary, but I wonder whether the constant flow does not place the embankment at some risk, particularly ponds 3 and 4, which will remain empty of fly ash for a considerable time. Do not the Authors feel that it would have been advisable at that section of the wall to have driven steel sheet piles through the blaes-filled core trench or to have adopted some other method of obtaining an impervious wall, or do they consider that the diminishing water level fluctuation within the ponds justifies the decision not to adopt such measures?

141. With regard to the Cockenzie cooling water works, the Authors refer to the shaft-sinking there as proceeding without incident. This, I am glad to say, is quite true, but it may be rather an oversimplification. I believe that in this work we developed, in conjunction with our sub-contractor, Rock Fall, a rather novel form of shaft sinking, albeit applicable only to shafts of fairly shallow depth.

142. The land and sea shafts were sited in close proximity to sheet steel piling, and as the shafts had to be sunk in water-bearing rock (fissured sandstone, mudstone and shale) we were naturally anxious, for the security of the work, that it should be carried out fairly quickly and be followed as rapidly as possible by the placing of the permanent concrete lining.

143. The land shafts were approximately 60 ft deep and the sea shafts about 50 ft deep. The excavation diameter was 16 ft. It was considered that drilling could be carried out with sufficient accuracy over the full depth from the surface by using waggon drills fitted with a quite simple plumb bob attachment to check the verticality of drilling and ensure that it did not go off vertical. In this way, holes were drilled at 2 ft centres around the perimeter and a standard pattern was adopted within the shaft area.

144. After drilling, the holes were filled with sand and, prior to charging each round, the sand was simply blown out by water jet to the required level. In this way, after the initial drilling had been carried out, no further drilling was necessary and the accuracy of the perimeter holes ensured a fairly even rock split, with the result that steel rib support was not required. Over the top 15 ft of each shaft blasting was controlled very carefully with vibrations restricted to within 0.003 in. at rock head. With rounds of approximately 2 ft only being pulled over this section, it was very helpful not to have to drill each round separately and we would never have achieved such an accurate profile without the perimeter drilling.

145. Another advantage was that this gave a fairly accurate indication before excavating of the nature of the rock that would be encountered. It would have indicated in advance any major source of water which would then have been grouted from the surface.

146. After excavation, the shafts were concreted with a simple slipform shutter suspended from $\frac{1}{2}$ in. dia. reinforcing rods anchored to screw jacks at the surface and concreting progressed continuously, rising at a rate of about 9 in./h.

Mr C. A. Frettsome, Kier Ltd

I was the Agent on the coal handling plant at Longannet. The Authors have noted the presplitting technique used by the Contractor for rock excavation on this contract. I feel that some further details may be of interest.

148. We formed vertical sides to all the hopper and conveyor tunnel excavations to a maximum depth of 65 ft using a single split. The tolerance achieved on the rock faces was generally ± 3 in. at the bottom, and nowhere did it exceed 6 in. As a result, the walls and slabs could be cast direct against the rock face without any further trimming. This was the first time we had used this technique to that sort of depth, and the benefits were considerable. The saving in volume of rock excavation and subsequent backfill, together with a saving of back shutter on the hopper walls, compensated for the cost of the presplitting. In addition, there were several less obvious advantages, for example, being able to operate heavy constructional plant around the tops of the excavations and within a few feet of the actual work in hand. The Contractors were indeed grateful for the co-operation of the Consultants in permitting the minor modifications to design which were necessary for the technique to be used successfully.

149. The use of small shrinkage gaps has been mentioned by the Authors in relation to the Cockenzie culverts. This form of construction was also used at Longannet in the coal hoppers and tunnels. These gaps caused some problems in construction, both as a result of delays in the concreting sequence and the subsequent difficulty in cleaning out the gaps prior to concreting. The latter problem was, of course, aggravated by our decision to cast the slabs direct against the rock face. Are the Authors still convinced that this form of construction holds any real advantage over the more conventional alternate bay system which was actually used in the Longannet cooling water system? Having had experience of both systems, which would they use in the future?

150. The Authors have also mentioned the use of steel bunkers in the coal hoppers at Longannet. The main plates of these bunkers were curved, and expensive to manufacture and fix. Why did the designers use steel in preference to reinforced concrete construction here?

Mr J. F. C. Gayner, Balfour Beatty & Co. Ltd

I was particularly interested to read in §§ 17-20, references to the cooling water intake and tunnels at Cockenzie, since I was responsible for the design of the intake headworks cofferdam as my first task on joining Balfour Beatty in 1963.

152. The parameters that I was given were broadly three: (a) Larsen 4B piles in high yield steel would be used, fitted with rock driving tips; (b) frames 1 and 2 would be erected first on false-work and the piling pitched around them; (c) most important, the weight of the cofferdam bracing should not exceed the allowance made at the time of tender. The arrangement for bracing shown in the half plan in Fig. 7 was that designed at tender stage but not the configuration ultimately adopted, which is shown in Fig. 23.

153. The tender design calculations were taken from the triangular water pressure diagram, allowing for wave action, and dividing it up between the frames on the basis of all the frames being fixed, when the loads were shown to be 3.7 ton/ft for frame 1 and the remaining five frames varied from 5.7 to 6.1 ton/ft. On the particular calculation sheet was written the statement 'Design all walings for the maximum load of 6 ton/ft run.'

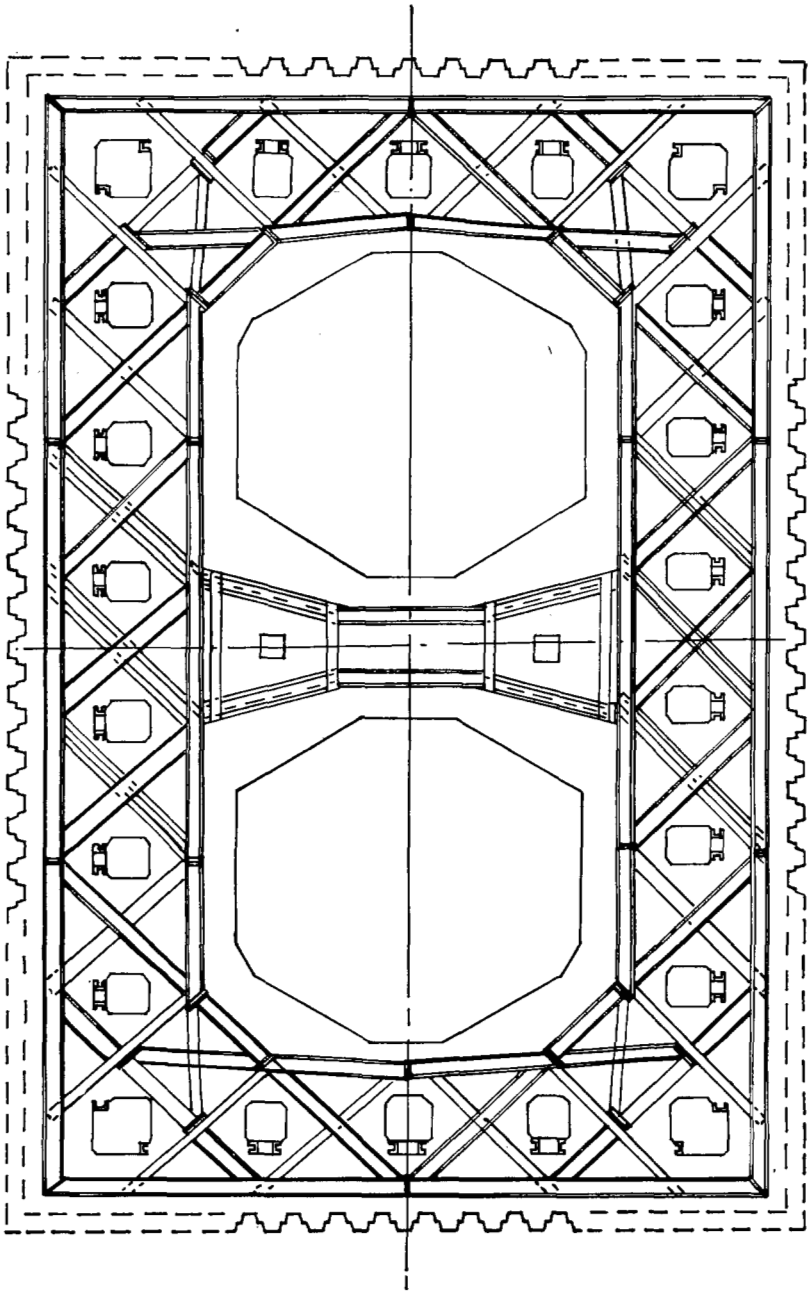


Fig. 23. Cofferdam bracing

154. The truss illustrated in Fig. 7 was 'just stiff' and was analysed by resolution and the triangular forces, and, taking account of the axle load effects, the size of members determined in accordance with the requirements of BS 449 and the tonnage of steel were evaluated. What the designer failed to recognize, however, in the short time available at the time of tender was that frame loads during the transient conditions when the water inside the cofferdam was pumped out for the fixing of the lower frames were considerably at variance with the final condition. The actual loads were:

Frame 1	2.9 ton/ft
Frame 2	6 ton/ft
Frame 3	14.9 ton/ft
Frame 4	11.6 ton/ft
Frame 5	8.5 ton/ft
Frame 6	5.7 ton/ft

These figures were a long way from the 6 ton/ft which had been allowed. Indeed, it was found during the construction phase that the load in frame 2 rose to 9.9 ton/ft. It was, therefore, decided to design the frames for loads as follows: 6 tons, 10 tons, 15 tons, 12 tons, 10 tons and 6 tons.

155. In the detailing of the frames, the configuration shown in Fig. 23 was adopted to make construction of the peripheral columns simpler, the columns fitting neatly inside the spaces. This arrangement was not capable of analysis by conventional Bow's notation and graphical solution because it was statically indeterminate. Therefore, the framing was considered as a double portal on the centre line of the framing, and the bending moment and shear force diagrams were drawn using moment distribution.

156. The diagram was drawn for a load of 1 ton/ft and multiplied for the various ranges that were considered. A chord depth between the two flanges of 8 ft 6 in. was assumed and the moment of resistance of the frames was found and the steel sections chosen accordingly, using rolled steel channels and beams throughout. The compression forces and the members were evaluated from the shear force diagram and the combined bending and compression effects considered for each frame.

157. The analysis of the lattice bracing started basically from knowing the load in the centre strut, which was then resolved into the diagonal members—all very rule of thumb. The several bolted connexions which were necessary to erect the lower frames were designed on the shears or the tensions induced, as the case might be. That this method of analysis was successful is borne out by the fact that the cofferdam bracing did not fail.

158. The total weight of steelwork was 180 tons, which, I am glad to say, showed a slight saving over the tender allowance. It was, however, considered necessary to brace the frames together using 6 in. dia. steel pipe and mild steel angles, a wise precaution in view of the racking effects of the wave action to which the cofferdam was subjected in this exposed part of the coast.

159. I commend the Authors' firm for their guidance and helpful criticism during the design of the cofferdam and particularly the problem of coping with possible overturning.

Mr N. C. Smyth, James Williamson & Partners

I had the interesting experience of being on the site for six years while the operations described by the Authors were carried out.

161. The year 1962 was one of very difficult weather conditions. During three months of that year (April, May and June) there were four times as many winds exceeding 18 mile/h as there had been during the preceding five years. This naturally made conditions very difficult at Cockenzie, with which I was associated. The fact

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that the sea was whipped up by the wind made it impossible at times for divers to try to fix the walings underwater for the sea wall.

162. I should like to refer to a point in connexion with the driving of the Larsen 4B sheet piles for the sea wall. A problem which arose here was the interpretation of the driving records. One could be driving at a rate of perhaps 1 ft in 30 blows, or 1 ft in 3 blows, and driving conditions would suddenly change. The problem was to determine from the driving record graph exactly when one had entered rockhead, and of course the rock varied in its hardness.

163. This variation worried me, particularly when we were carrying out piling operations on the east end of the sea wall where the overburden was only about 3-4 ft. We stripped down and examined the section and found that the pile had penetrated about 4 ft, but at that point the rock was very soft. It was an entirely different matter across the bay, near the whinstone reef.

164. It was necessary to keep a record of each pair of piles, which numbered 1700 pairs. Final levels were based partly on the driving record of the pile and partly on achieving some uniformity in toe level, to avoid one pair of piles standing up too high with respect to the adjacent pair.

165. In § 10 of the paper, the Authors refer to the speed at which construction proceeded. The main foundations contractor started in February 1963, and was ready for steelwork by November of the same year. This meant an extremely rapid build-up in the amount of work and, indeed, in labour strength. The peak labour strength of about 300 was reached in November of that year, and the foundations contract and the steelwork contract were substantially finished eight or nine months ahead of programme. I account for this by the mere fact of the tempo of the work having built up. The problem was to keep the tempo running, and fortunately, this was possible. Sufficient design information was produced to keep the machine running at full speed. The result was a rather earlier finish in substantial completion of the contracts.

166. I should like to refer to the effects of natural silting-up of both the steel sheet pile wall on the main site and the ash pond embankments. The initial leakage was about 24 000 gal/min. Pumping capacity was 10 000 gal/min. The shortfall was made up by ashing along the face of the sea wall, by caulking the clutches and admittedly by the effects of the sea storms. It is interesting to notice that the figure of $\frac{1}{2}$ gal/sq. ft of wetted surface area agrees very well with the figures given by Packshaw.⁶

167. In connexion with the ash pond embankments, the effects of natural silting-up have been shown in the Paper. It had been known from our previous experience at Kincardine power station that silting-up was a great natural phenomenon and use should be made of it. The problem was to know the speed at which this would occur.

168. I am sure that progress was helped by the excellent relations which were established with the contracting organizations. Perhaps it was also helped to some extent by the fact that at one stage, on four of the major contracts, the works manager in each case was an Irishman and the resident civil engineer was an Ulsterman.

Mr R. Y. McNeil, Member, Canada

I would like to congratulate the Authors on presenting a very interesting Paper which comprehensively describes many of the most unusual features of the two power stations. Having worked as a site engineer during the early stages of construction at Cockenzie, there is one feature that was not brought out in the Paper which I feel is worthy of mention.

170. Due to the fact that the toe of large portions of the double-skinned sheet pile sea wall were exposed at low water, it was considered that some form of erosion protection was desirable on the seaward side of the wall. During the investigation for suitable material, which had to be heavy enough to withstand considerable wave action, it became apparent that the anti-tank concrete blocks which had been installed

more or less continuously along the section of the coast east of the site during World War II would provide an economic and practical solution. These blocks, which measured about $4 \times 4 \times 5$ ft, were transported to the site where they were individually placed at the foot of the wall. Some were split prior to placing to achieve the desired profile.

171. At a time when engineers seem to be more frequently criticized as desecrators of the countryside, I feel that this small feature is worthy of note. As well as providing an engineering solution to the problem at hand, the amenities of several miles of the coast were improved by the removal of ugly reminders of wartime defences at no direct cost to the communities that benefited.

Mr B. G. R. Holloway, Partner, Rendel, Palmer & Tritton

This interesting Paper on two power stations can, because of its comprehensive nature, only deal relatively briefly with the many forms of civil engineering involved. I would have liked to have seen more details dealing with design of the steel frame since this part of the Paper is mainly concerned with the problem of replacing the steel drum of No. 1 boiler.

173. It is clear that the factors which can be ignored in the design of steel frames for smaller power stations become more important matters when one is dealing with the larger sets, particularly the present 500–625 MW sets. Amongst these factors is the question of limiting horizontal deflexions at the suspension level of the boilers and the doubtful contribution to the stiffness of the steel frame by cladding, etc.

174. Where simple design with bracing is provided, problems arise with the slope of end connexions of heavily loaded simply supported beams, especially if they are of high tensile steel, and particularly where these members are required to transmit horizontal wind to the braced frames. Additional horizontal forces are due to ambient temperature and local temperature effects arising from the plant.

175. It appears from recent experiments that the wind on the partially erected and partially clad structure is a significant factor and might involve the provision of temporary bracing. It would appear that the suction effect on the turbine house roof when a portal design is used could cause significant variation in the gauge of the crane rails.

176. My experience with the use of HYP steel has involved welding difficulties, rejection due to plate defects seriously affecting the fabrication programme and overall economy.

177. Finally, the need to maintain an economical crane erection rate and bring into use the gantry crane for plant erection as early as possible necessitates the erection of the turbine house in advance of the boiler house, leading to difficulties from their relative stiffnesses.

Mr A. C. Paterson, F. R. Bullen and Partners

I have felt for some time that the civil engineering works involved in power stations are less than adequately recorded in the *Proceedings*. Each power station represents expenditure, on civil engineering works alone, only equalled by civil engineering projects of the first magnitude. Such works have provided opportunities for important innovations and progress in design and construction as witnessed by this Paper.

179. I should like firstly to refer to the cooling water culverts at Cockenzie described in § 11. These were constructed in 20 ft lengths on Lubrithene sheeting, with 4 ft gaps between successive sections which were concreted not fewer than 28 days later. Lubrithene I take to be some form of plastic sheeting with a low coefficient of friction when in contact with concrete. I wonder whether any measurements of the friction were taken or what frictional resistance was assumed.

180. There is a parallel with work on an experimental prestressed concrete road

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at Winthorpe in Nottinghamshire, reported by Kidd and Stott.⁷ There, a particular endeavour was made to reduce the frictional resistance. Under the road slab there was a layer of lean concrete, and particular care was taken to produce a smooth upper surface, a tolerance of $\frac{1}{4}$ in. in 19 ft being achieved. On this was laid a sliding layer of two thicknesses of special polythene with a low friction additive, the whole being between two sheets of building paper. Laboratory tests on the polythene showed a surface co-efficient of friction of 0.1. Despite this, measurements on the slab itself showed an average value of 1, or 10 times the laboratory results. As the tensile stress in the concrete due to the shrinkage may vary directly with the frictional resistance its significance will be apparent.

181. Still on the subject of cooling water culverts, I feel we should be fully alive to the possibilities now being offered by glass reinforced plastics (GRP). In response to recent enquiries I have been making I have been informed by a manufacturer that pipes of this material are now available up to 10 ft in diameter. With, for example, a wall thickness of $\frac{1}{4}$ in. they may be buried up to 20 ft deep and will withstand an internal pressure of 100 lb/sq. in. Apart from the obvious advantages of lightness and ease of handling, resistance can be provided against a range of corrosion conditions. The frictional resistance to flow is also reported to be reduced allowing, for example, a 9 ft dia. GRP pipe to be substituted for a 10 ft dia. reinforced concrete pipe without change in power consumption. The increase in flow velocity that this implies and the smooth surface of the GRP will also be beneficial in reducing mussel growth.

Mr Baker and Mr McPherson

Mr Oakes, who has first-hand knowledge of events at Cockenzie, notes the use of red or burnt blae at this site and unburnt blae at Longannet for sea walls and contrasts their comparative permeability.

183. Longannet did use black unburnt blae very successfully for achieving almost watertight embankments for intertidal ash lagoons, but the circumstances differed from those at Cockenzie. The Longannet embankments were above low water, and in an area where sea action was much less of a problem; the black blae was placed and compacted between a seaward rock embankment and a containment cover layer of rock on the landward side. The enclosed lagoon did not have to be dewatered.

184. At Cockenzie main site the blae was to be placed between sheet piling and round through tie rods. Chemical inertness was desirable and avoidance of self ignition essential. The burnt blae met these requirements, was readily available and initially would have contained enough fines to give adequate impermeability but for the wash-out of these elements by sea action. Layers of black blae were used in the site filling up to a limit decided after consultation with the NCB on the question of self ignition.

185. On the point about the wagon discharge house, it is true that complexity arising from the changing design requirements at upper levels did result in congested sections—on average $2\frac{1}{2}$ cwt of reinforcing steel per cubic yard of concrete. The delay to the building was due to complexity and other factors rather than concrete placing.

186. Mr Oakes raised the question of the possibility of silting in the cooling water outfall channel at Longannet. On completion of construction and with the flume open to tidal flow, silt was deposited to a depth of 2–3 ft. However, when the first generating set became operative, the silt was to some extent washed out to sea. There is little doubt that when the other sets come into operation the rest of the silt will go. Calculations show that in the normal operation of the station deposition of silt is not to be expected in the flume. It is thought that the channel across the foreshore, which is an extension of the flume, will be self-scouring under normal operating conditions.

187. The Authors would like to thank **Mr Canales** for his remarks when mentioning the problems of a main contractor on power station projects and his need for space to plan his operations efficiently. Construction space must of necessity be limited where the main buildings are sited on reclaimed land.

188. **Mr Rossiter** has shared the experience that severe storms at sea are not confined to winter but are more frequent. At Cockenzie marine works were not halted during winter periods, but an estimate of loss of time of 40 days/year, which the tenderers were asked to allow for, proved very close. The specification of the stage by stage construction of the ash lagoon embankments was designed to allow continuous working at all water levels and work was only stopped when spray and wash or high wind made conditions unworkable.

189. **Mr Rossiter** went on to refer to the use of 40 and 60 ton tower cranes. These were most useful tools, but an interesting statistic quoted by the Contractor was that the average load lifted by the 40 ton tower crane was $1\frac{1}{2}$ tons, a surprising fact.

190. **Mr Harding** asks in effect why the station was not reversed through 180° . The answer is twofold, in that the layout adopted was designed to present the best appearance and massing of the buildings, with the turbine hall facing the public road, and secondly to reduce as much as possible the costly heavy cabling from the generator transformers to the 275 kV switchhouse.

191. **Mr Hume** makes the point that the graph (Fig. 3) does not indicate that a pumping installation of 10 000 gal/min was installed after closure with an inflow of 24 000 gal/min, and asks whether the 3 day storm was of material assistance in sealing the wall. The inflow plotted is the maximum inflow, which varies with the head acting.

192. Initially the pumps were not coping with the peak inflow but were able to do so at lower tide levels. The storm occurred at a time of rising tides and it is probable the pumping arrangement would have coped and would have achieved unidirectional flow of water through the sea wall with falling neap tides to come in a few days. In the event the 3 day storm did the job for us. Once unidirectional flow was achieved there was no further difficulty.

193. **Mr Hume** asks how the figure of 600 gal/min came to be specified in the contract documents for Longannet. The Authors can only say that this was based on the expectation of the nature of the rock through which the Longannet tunnels were to be driven and the boulder clay covering of the rock surface in the river bed. The penetration of the boulder clay by piles for the temporary jetty and the cofferdam might be expected to lead to leakage paths. The Authors would think that this was the cause of the greater leakage noticed during the construction of the last 50 ft of the tunnels and at the intake shafts, and was not due to the falling away of rockhead, since that would imply a greater degree of protection from the boulder clay covering (see Fig. 18).

194. The Authors thank **Mr Hargreaves** for his contribution to the discussion which is of much interest to all concerned with Longannet power station.

195. **Mr Stevenson** in a reference to ash lagoons 3 and 4 questions whether they are at some risk. A record of ash pond fluctuation during a 16 ft spring tide in September 1969 showed in pond 3 a 1.50 ft fluctuation of level. A very recent record shows this to have diminished to 1.1 ft and that at pond 4 to have reduced to 0.6 ft of fluctuation over a similar high spring tide. From site observations the indications are that the flow path is below the toe beam and there is no indication of flow at bottom of bank level and no sign of settlement or displacement of armour to indicate wash out of sea bed material. Pond fluctuations continue to fall with time and levels do not fall below about 0.8 ft above mean sea level, which appears to confirm increasing impermeability at low levels. The maximum rate of rise is 0.28 ft/h at high water, when the head differential is of the order of 7 ft.

196. **Mr Stevenson's** comments on shaft sinking are of particular interest and his

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methods were successful in meeting the Authors' concern to avoid any disturbance of the sea bed rock strata near the sea wall since the main building area lies below tide level.

197. **Mr Frettsome** refers to the use of small shrinkage gaps in the Cockenzie culverts and Longannet coal tunnels and asks if the technique would be used again. Much depends upon detailed layout, foundation conditions and programme implications. Both shrinkage gap and alternate bay construction have viable applications. The means of providing shrinkage gaps in large concrete structures is always a problem. It is believed that the 'hit and miss' method as used at Longannet is to be preferred for the culverts, but the form adopted on the coal handling plant foundations is to be preferred for these comparatively massive structures.

198. The reason, at least in part, why steel bunkers were specified at Longannet was because of the difficulties experienced at Cockenzie in the construction of in situ reinforced concrete bunkers (see §§ 38, 104 and 105). The plates used in the steel bunkers at Longannet were curved, as required by the Board, and even had it been considered feasible, it seemed doubtful whether the cost of the construction of reinforced concrete bunkers would have been any less than steel ones.

199. **Mr Gayner's** comments are of considerable interest on the intake cofferdam, which was successfully used with less than desirable cut-off in an area subject to heavy seas but which enabled the structure to be solidly founded on good rock all over its base area.

200. **Mr McNeil** mentions the use of war time anti-tank defence blocks. Some 2500 of these were uplift from beaches, golf courses and fields, many weighing as much as 10 tons, with great benefit to amenity.

201. **Mr Holloway** is thanked for his contribution on certain aspects of the structural steel framework design of modern power stations.

202. The Authors agree that limitation of horizontal deflexion can be an important factor in design. At Cockenzie with smaller 300 MW units the horizontal deflexion was limited by the use of bracing wherever possible. No account was taken of the stiffening effect of cladding.

203. No particular problems were encountered in the design of connexions of simply supported beams, mainly because the design conception practically eliminated any cases of deep, heavily loaded girders being considered as simply supported beams. The only high yield steel girders used were in the boiler suspension steel, where the slope of the ends could be taken into account and where the main girders rested on the column caps. The point raised is an important one, which must be considered in the design of large structures, and in the Authors' view it is unrealistic to assume that deep girders can be considered as simply supported at the ends without making special provisions for the points raised by Mr Holloway.

204. Cockenzie steelwork preceded the recent experiments referred to, but the structure was checked for the effects of wind on the partially completed building. No temporary bracing was required in this case, but it is agreed that this is sometimes necessary.

205. In considering the welding of high yield steel the fabricators were consulted very fully, tests being made where these were thought necessary before finalizing the details. Consequently few or no difficulties were experienced in welding this material. Rejections due to plate defects did appear to affect the fabrication programme at one stage, but this phase soon passed and the overall rate of rejection was not high.

206. Turbine house erection did proceed in advance of boiler house erection, but no difficulties were experienced as a result. The gantry crane was not brought into use before the main steelwork was erected, and the rate of erection achieved was such that the structure was ready for the gantry crane in advance of requirements and of programme.

207. **Mr Paterson** is correct in his assessment of the use of Lubrithene sheeting which is in fact a two-thickness material. No site measurements were taken but

clearly it will always be difficult to attain laboratory results for friction in large-scale field works. Techniques in the use of new materials advance very rapidly, and since power station cooling water systems are generally very much influenced by conditions in individual stations it is right to give consideration to possible new applications of materials in the initial design stages, in collaboration with plant designers and station operators.

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