Dinorwig pumped storage scheme

J. A. Baines, V. G. Newman, I. W. Hannah, T. H. Douglas,
W. J. Carlyle, I. L. Jones, D. M. Eaton and G. Zeronian

Dr A. D. M. Penman, *Geotechnical Consultant*

I would like to describe the deflection measurements that I made while I was still with the Geotechnics Division of the Building Research Station; the subject for a paper shortly to appear in *International Water Power and Dam Construction*.

336. The interest of the Building Research Station in Marchlyn Dam stemmed from the fact that it is amongst the first British dams to use an upstream asphaltic membrane. As part of our study of the behaviour of dams, aimed at an improved understanding that can lead to design improvements, it was felt desirable to measure deflections of the membrane under water load, particularly since, as reported by Dr Charles and mentioned in the Papers, laboratory tests had been carried out in our 1 m dia. oedometer on rockfill samples.

337. In preliminary discussions with Mr Carlyle, I had proposed the use of an inclinometer in tubes laid just under the membrane, but understandably this had

Fig. 41. Expected differential movement between concrete and membrane, Marchlyn

Fig. 42. Marchlyn Dam membrane deflection

been turned down because of a risk that trenches made for the tubes could form weaknesses: settlement of the backfill could produce undesirable strains in the membrane. It was therefore decided that measurements could be made on the surface of the membrane. Freed of the constraint of a small access tube and the need to design a special inclinometer to work on the 1 on 2 slope, a trolley was developed to run on rails and carry a standard inclinometer in a vertical position.

338. Because the waterproof membrane consists of one 80 mm thick layer, it was not feasible to put bolts through it for attaching the rails, so sub-frames were used. These were placed on the surface of the completed membrane at the major section of the dam and buried in an additional layer of asphalt by the paver straddling over the line of frames. Tapped holes in the frames were then exposed by coring through the asphalt and fitted with threaded stubs to carry the rails just above the surface of the asphalt. The space round the stubs was filled with hot bitumen and the whole surface covered by seal coat before the rails were attached.

339. The trolley was made with a 2 m gauge length between wheel centres and the rails had stainless steel inserts at 2 m centres. A length of inclinometer tube was built into the centre of the trolley so that it would be vertical when the trolley was on the 1 on 2 slope. A special cable containing a straining wire as well as the conductors was used to lower the trolley from a winch screwed to the floor of a side-door van driven on to the crest. The inclinometer had a half metre gauge length and gave offset readings to 0.1 mm. It was turned round in its tube to overcome zero errors, and four runs were made, two with the inclinometer on each face. By summing these four readings, the offset values of the 2 m long trolley were obtained.

340. Careful consideration has always to be given to the connection of the membrane to the bedrock at the toe of a dam where reservoir water pressures will be greatest. At Marchlyn this was done by a concrete structure, shown by Fig. 12 of the Paper. Differential movement between the rigid concrete and more flexible rockfill can be exacerbated by the difficulty of obtaining perfect compaction immediately adjacent to a structure—a common experience with bridge abutments. Great care had been taken at Marchlyn, with special provision at the junction between membrane and concrete. With the trolley in its lowest position, its lowest wheels rest on rails bolted to the concrete, so that readings taken at its first few positions on its climb up the rails, show the magnitude of the expected differential movement.

341. The results shown by Fig. 41 reveal a pronounced dip just beyond the concrete structure, but its magnitude, after first filling of the reservoir, was only 25 mm. Deflections of the rest of the membrane did not exceed 20 mm and were less than a tenth of those predicted by the Authors and given by them in Fig. 11 of the Paper. These values of predicted deflection due to water loading are shown together with the measured values in Fig. 42.

Mr R. N. Burbridge, CEBG

The Dinorwig pumped storage scheme has been engineered by the Generation, Development and Construction Division of CEBG over the past nine years, and it is now close to being fully commissioned. My personal responsibilities as Director of Projects over the past five years have included the substantial completion of the civil engineering, the installation of the great majority of the plant and setting it to work. The earlier works, from the passing of the Act, were the responsibility of my fellow director, the Director of Station Design, Mr Brian Powell.

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343. Despite being largely successful, with hindsight, and perhaps due to prolonged over-exposure to the detail, I can now see how much better the scheme might have been. I suspect that all engineers capable of learning from their experience would similarly wish to have a chance to improve next time. It is an irony of our joint profession that we are so rarely afforded the opportunity of a next time, particularly in civil engineering.

344. Both time and cost over-runs on the scheme have to be acknowledged. Perhaps all such estimates for major projects need to be optimistic to persuade clients and backers to proceed at all. However, the obverse of that policy must lead to some disillusionment and disappointment when experience inevitably exposes such optimism.

345. Beyond such estimating errors, Dinorwig suffered an immediate setback when a preliminary access tunnel collapsed. The psychological damage caused by such an accident on the engineering clients better versed in the building of conventional power stations on flat fields is readily appreciated. Perhaps the tunnellers did too little to forewarn of the hazards that they usually accept in their business, but the consequence was doubt and prolonged delay in proceeding further.

346. Ironically the rock quality improved dramatically beyond the new portal, making these delays seem over-cautious with the benefit of hindsight. The lesson lies in the need for improved communications between the engineering disciplines.

347. Of major significance to the project was the positive contract management attitude applied to all the contracts involved. A good instance of this attitude was the employment policy. It was written into the contracts from the start that the major part of the site labour force had to be drawn from the local area. In the event 95% of the hourly paid work force were people living within about 50 miles of the site. Because of this, the construction of the station was welcomed by the local people from the outset. As the work proceeded, the project became increasingly well known because of the challenge it was presenting and the achievements being attained. The prestige accorded to the project, and hence that of the working force associated with it, grew speedily and rapidly. Welcomed from the start the project improved in local standing as it progressed. Without doubt this feature made a major contribution to the excellent industrial relations.

348. In their dealings with the site labour force the management acted positively and in a timely manner to prevent the development of any site grievances. This approach too made a significant contribution to the industrial relations created and maintained through the whole life of the project. The CEGB have not been very successful with target cost contracts, mainly because we have usually been established when the full extent of the work is not really known. Both at the Isle of Grain, where we used it at about this time, and also at Dinorwig there was not enough security in the project to make target cost contracts work well. It will be seen from the papers that the contract activity was changed.

349. There were also recommendations in the early part of the 1970s for larger single contractors. They followed NEDO recommendations of the previous decade. It is not certain how that would now be handled but the work could be broken down into slightly smaller packages.

350. The Dinorwig project is now nearly completed with the construction work substantially finished and five machines are currently operating. From experience so far the plant is very flexible in its operation, and the CEGB's original expectations are being more than fulfilled.

351. In due course the Dinorwig pumped storage scheme will prove itself to be

a most satisfactory facility playing a major role in the operation of the CEBG network.

Mr G. Dunton, Frederick Gibberd, Coombes and Partners

As landscape architects, what were we charged to do? The North Wales Hydro Electric Power Act 1973 required the appointment of a landscape consultant, his primary aim being: 'To preserve for the public the natural beauty of the district to preserve flora and fauna and the geological or physiographical features of special scientific interest.'

353. Further, we were to consult with the Countryside Commission and to advise on such matters as the disposal of soil and reinstatement of land and the planting and felling of trees and shrubs. We were to assist the Board in their consultations with the local planning authority, over the situation and design of any dam, aqueduct, building, bridge, wall, embankment, road or fence.

354. I would like to pick out two fundamental issues raised by Sir Frederick Gibberd in his evidence at the Parliamentary Inquiry. First, the site selected was considered the best, being centred on the Dinorwig slate quarry, an enormous monument to 19th century industry, 550 m high and manifestly the work of man. The presence of the quarry made it possible to introduce engineering structures unobtrusively. In a wilder setting where there is no evidence of man they would have been alien.

355. Secondly, the views from Nant Peris and Llyn Padarn looking up the valley pass towards the Snowdon Range are magnificent. No structure or alien form should be introduced to reduce or partially obliterate these views. Sir Frederick was able to make the case for a low dam which finished up as a shallow embankment. The capacity of Llyn Peris could be made adequate by excavating from the banks of slate waste.

356. An early idea was to use both Llyn Peris and Llyn Padarn for the lower reservoirs. The decision to make Llyn Peris the lower reservoir by damming it and increasing its capacity ensured that the visual problem of drawdown was limited to one lake. Drawdown on the shoreline to Llyn Padarn at Llanberis would not have been socially acceptable.

357. I would like to mention two general matters before describing individual sites: the disposal of spoil and planting. It was very fortunate that all the spoil could be disposed of on site for the embankment dam at Afon-y-Bala, the new land form at Nant Peris as well as the roadworks. Planting is a minor element in this particular project, good land form being regarded as the more important factor. At Marchlyn Mawr no planting was proposed. The only areas of mass planting are at Afon-y-Bala. Planting is confined, in the main, to indigenous trees and shrubs and is used to screen views or to focus views and to marry the new works to the valley landscape.

358. Afon-y-Bala, the land between Llyn Peris and Llyn Padarn, was built up by silt, which caused the collapse of the retaining wall, brought down by Afon Hwch, which crosses the narrow channel containing the flow of water between the two lakes. Our scheme in this area was designed to echo as far as possible the existing layout and character of the area.

359. The most economical solution for an embankment dam would have been to introduce a straight minimum length design with steep sides. It would have resulted in a structure which would have crashed into the mountains each side. But in fact the shape of the dam is so remote from the conventional form in

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appearance that it is not seen as a dam at all. The crest of the dam is disguised as a roadside wall and the raised land form of the embankment was designed to accommodate the various structures in a natural way.

360. The contours are complex and the geometry of the road and the diverted river have elaborated the complexity. The gentle gradient of 1 : 5 on the upstream face is also not one normally associated with dam structures.

361. Turning to the southern end of Llyn Peris reservoir we come to Nant Peris and the southern works. The works at Llyn Peris are set in flat pastureland flanked by mountains. Here the valley floor was raised to prevent the flooding of the meadows in a way which blends in remarkably well.

362. The problem posed by Marchlyn Mawr was the landscaping of the dam on its downstream face. From the valley below it could have been visually intrusive. At the Inquiry it was claimed that it might be seen as far away as Anglesey.

363. The problem with a dam in a mountain landscape is that it tends to have a level crest and produces a straight line which is seen in contrast to the silhouette of the mountains beyond on each side of it. We were fortunate (I cannot claim that we were instrumental because I am sure there were engineering considerations!) in having a dam which was curved on plan which produced a hump appearance when seen from the valley below, and that gentle curve of the dam linking the mountains on each side is enormously subtle and helpful to the problem of disguise. In fact it is so unobtrusive that when I have taken visitors to the site I have to point it out to them. It is an amazingly successful piece of landscape design in that respect. When people are shown round the major landscaped area a year or so after completion and they ask, 'What did you actually do?' then you have almost certainly succeeded in your task.

Mr G. B. Murdoch, Merz and McLellan

In §§ 11 and 20 the Authors refer to the immediate reserve criterion and its effect on the design of the hydraulic system. The system itself is described in outline in § 19 and in detail in §§ 106–119: a single high pressure shaft leads from the surge chamber to a six-branch manifold, thence individual steel lined tunnels continue to the inlet valves, the pump-turbines and the draft tube valves then on to three tailrace tunnels, each serving two pump-turbines. Maximum flexibility in operation is achieved by minimizing starting time ($LV/(gH)$) of the unregulated hydraulic system and by keeping physical profiles well below the hydraulic gradient in all conditions of steady or transient operation. Figure 8 shows a virtually ideal profile from the surge chamber to the tailrace. Table 3 gives a L/H ratio of 5 : 8 for the overall scheme but in the context of operational flexibility the ratio of 3 : 1 from the surge chamber to the tailrace is more significant. It remained to ensure water velocities sufficiently low to meet the unusually severe operational demands for starting, stopping and reversing the unregulated water column.

365. In addition to the immediate reserve facility of increasing generated power from 0 to 1320 MW in 10 s a number of other requirements were to be met as follows.

- (a) An integrated power–time criterion for the whole station when supplying immediate reserve
- (b) Rapid changes in power generated over the full range of capacity from one or more sets up to the whole station
- (c) Emergency change from full pump discharge to full generated output in 1.5 minutes.

Fig. 43. Predicted and actual results for a full load rejection on a 225 MW Francis turbine, Argentina

Fig. 44. Predicted and actual results for a full load rejection on a 150 MW set at Foyers

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These are additional to the normal requirement of safely withstanding an instantaneous loss of full station load.

366. Hydraulic transient studies were carried out using a suite of computer programmes which model separately the major components of the high pressure system: conduits, valves, pump-turbines, draft tubes and tailraces. Turbine and pump loading, off-loading, tripping and mode changing were modelled mathematically for all critical operational and emergency conditions. The programmes had earlier been developed and refined by comparison with actual recordings made on conventional and pumped storage stations so a high degree of reliance could be placed on the studies.

367. Figure 43 shows predicted and actual results for a full load rejection on a 225 MW Francis turbine in Argentina. This is a simple installation of penstock and turbine. Full generating load rejection on one of the 150 MW reversible sets at Foyers again shows close correlation between predicted and actual site results (Fig. 44). This is complemented by a pump release as recorded on the same Foyers pump-turbine (Fig. 45).

368. So when the Dinorwig studies were initiated there was in existence a proven suite of programmes on which design decisions could be securely based. Fig. 46 shows predicted head, speed and flow following a full generating load trip on one set and on all six sets. The latter, of course, shows much greater departures from steady running conditions than the former.

369. Figure 47 compares predicted values of heads, flow and speed following tripping of a single set with measurements of the same parameters actually record-

Fig. 45. Pump-release, Foyers pump-turbine

Fig. 46. Predicted head, speed and flow following a full generating load trip

Fig. 47. Predicted values of heads, flow and speed after tripping of a single set compared with recorded values

Fig. 48. Comparison of turbine opening times of 6.25 s and 4.3 s

ed at site. The close correlation found between prediction and observation of the behaviour of a single set gives confidence in the prediction for the six set load trip situation and may perhaps be taken to satisfy the traditional requirement of a full station trip without the need to subject the public supply to more disturbance than the minimum necessary to substantiate regulation guarantees.

370. The integrated power-time criterion was the single factor which had the greatest influence on the sizing of the conduits between the surge chamber and the tail race. Fig. 48 compares turbine opening times of 6.25 s and 4.3 s. The straight line of the energy integral requirement is shown and is, of course, the same in either case. The increase in output with an opening time of 6.25 s does meet the requirement of attaining 1320 MW in 10 s (§ 11 item (a)). However, the energy integral falls below the requirement at about 5 s and does not reach it again until 9.5 s. With the 4.3 s opening time, however, the $\sigma P dt$ line is tangential to the energy integral requirement line at about 6 s but never falls below it. The opportunity to verify this in actual performance is awaited.

Mr F. Shrimpton, George Corderoy & Company

The brief of the quantity surveyors was, in general terms, to prepare the tender bills of quantities; to measure and value the final accounts; to provide costing assistance; and to operate the CEGB's cost planning procedures.

372. It was decided from the outset that some of the tendering risks inherent in underground works should remain with the employer. To put this into effect separate provisions were made for changes in rock classifications, overbreak, temporary rock supports, dealing with underground water flow and the like.

373. Reference was made to the fact that there was some cost over-run. Table 16 of the Paper lists the major contracts and of those final contract sums no less than £90 million was accounted for by inflation over the period of the contracts.

374. The CEGB's procedure for design cost control of the construction works was used for the Dinorwig project. The procedure required that once a basic scheme had been established authority for variations or additional works had to be obtained before inclusion into the design or before an instruction or drawing of

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the change could be issued to the contractor. The procedural documentation considered by the Board's responsible officer described the proposal, included cost valuations and programming implications and was categorized as either essential or desirable. By this method approval to change had to be given before a contractual commitment was entered. The financial effect of these approved changes together with adjustments resulting from design development, admeasurement, contractual claims and inflation were summarized monthly on both a contract and project basis, thereby keeping under constant review the estimated final scheme costs.

Mr R. Bunn, Binnie & Partners

The cross-section of Marchlyn Dam is deceptively simple but, as usual in major construction projects (particularly those of embankment dams) there were many difficult problems to overcome during the construction stage. By far the most difficulty was experienced on the grouting and development of the grout curtain at the upstream toe of the dam which is critical to the successful performance of a deck dam.

376. In the weather conditions encountered it was a formidable task to keep grout lines, waterlines and airlines open during the winter months. I think it is all credit to the contractors that any progress was made in the severe winter months.

377. The technical problems varied along the length of the grout curtain. The problems in the centre section of the curtain are described in Appendix 4 of the Paper. The problem there was that the fissures in the bedrock were filled with clay, sand and rock fragments, and it was not possible to found the gallery below this clay filling or to flush out the clay between adjacent grout holes. The measures which were taken to overcome this are in the Paper and comprised an extra two rows of blanket holes upstream of the gallery. On the left abutment the conditions were completely the opposite. The fissures are open and closely spaced. As a result the ground is highly permeable and grout takes were fairly high.

378. Because of the open nature of the rock the grout curtain was extended beyond the body of the embankment to join with a dolerite dyke which forms an aqueclule along the left hand side of the reservoir. The grouting was monitored by water tests in various stages of the holes and the initial lugeon values were measured in primary grout holes. The most difficult decision was to know when to stop grouting. In the early sections of work it was found that lugeon values of 5 or less in the top 10 m were possible and below that lugeon values less than 10. These could be obtained without excessive drilling in sections other than on the left abutment.

379. In the deeper sections—and the curtain on the left abutment was 120 m deep—50 lugeons were measured in the initial tests in the 80 m to 120 m depth range. At completion drilling was being done for quaternary holes, and the hole spacing was down to about 5 m. Whether any more grouting was to be done was considered very carefully. If more work had been ordered the possibility of decking in April 1979—which was a weather sensitive operation—would have been in jeopardy. A decision was made to terminate the grouting on the left abutment in the knowledge that the toe gallery could be reached again as well as the extension of the grout curtain after impounding, if necessary.

380. The rather high permeability in the curtain in this section is reflected in the downstream groundwater levels and in the seepage flows during the first filling of the reservoir. As the level was increased to 613 m so the water levels down-

stream of the curtain increased until at 633.3, near the top water level, there was about a 30 m rise in groundwater levels immediately downstream of the curtain.

381. The total inflow into the drainage gallery from the whole length of the dam was 35 l/s and of that up to about 5 l/s is the natural drainage flow which was measured before impounding started. Although the drainage flow is twice that estimated, it still is very small compared with the turbine generation flow and it is not proposed to do any further grouting at this time.

Mr A. G. Cook, French Kier, incorporating Charles Brand

During excavation of the machine hall roof it became apparent that, as the plant and electrical layouts evolved, the electrical annexe would have to be lengthened and other plant areas increased appreciably. This presented an opportunity to analyse and adjust the major structural features of the machine hall so that they could be more readily and speedily constructed, bearing in mind all the relative handover dates. The first and most urgent decision was to lengthen the gallery by 25 m. An extension of the east end was chosen because of the favourable geology and because, by opting for a full span extension, it was possible to create a new loading bay fully serviced by the overhead construction cranes.

383. The required reduction in width of the original 50 m level central loading bay to create larger plant areas led to the decision to remove all the rock down to the 34 m level. To re-establish the loading bay with the least disruption, the new floors in this area were constructed of steelwork with precast floors and in situ topping. As it would have been almost impossible to maintain the roof of the pipe gallery through the remaining rock a potential delay to the upstream foundation of this steelwork had to be avoided. An anchored stub wall was designed to take the construction loading, and it was detailed so that it could be built during benching down of the gallery before breaking through to the pre-excavated lower levels.

384. The original position of the main power station sumps entailed deep excavation below the Set 2 and Set 3 locations and the excavation, rock support and structural concrete emplacement would have been in-line activities. With the changes to the central loading bay and the removal of the pipe gallery rock roof, the width of this gallery could readily be enlarged allowing both the sumps to be relocated beneath it. Here they could be excavated and completed, off the critical path.

385. The relatively early demand for full access to the electrical annexe (at the same time as access to the turbines) made it essential to found the annexe at the lowest possible level. By introducing massive steel bridges over the draft tube tunnel (26 m level), the steelwork for the annexe was built in parallel with the main machine foundations instead of starting after they had reached the generator floor (42 m level). The main RC work was carried out during the day and the steelwork subcontractor worked at night to minimize the programme effect of the large increase in steelwork.

386. To obtain access to the new east end loading bay a 300 m long adit was devised and driven, spiralling down 25 m from the high level access tunnels. This bay was indispensable during the civil construction programme because of the temporary removal of the central loading bay. In addition, when plant and electrical installation began at the Set 1/2 end using the re-established central loading bay, MBZ could complete their civil works for Sets 3 to 6 'backing out' of the machine hall with exclusive use of this new bay.

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387. A series of short adits were established from the new spiral adit enabling the lower levels of the machine hall and three other galleries to be excavated more efficiently. These adits also provided valuable permanent access.

388. The initial plan for installing all the heavy turbine parts was to use the two permanent 250 t cranes. These ran on rails which were originally supported on the upstream side by brackets anchored to the rock. As another new access adit on the west end at the 34 m level had been planned to assist the excavation, its possible use as an access for the first two turbine casings was investigated. By lowering and enlarging this access, and utilizing a specially designed temporary gantry, it was possible to install the first two spiral casings without waiting for the main cranes to be erected. Substantial redesign of the permanent works was necessary to achieve the required equivalent plant foundations and temporary access hatches.

389. This method of turbine installation then allowed the main crane rail support to be re-examined and it was established that there was time on the upstream side to use steel columns founded on top of the concrete structures instead of using brackets. The 3500 kN rock anchors for these concrete brackets would have been installed from the appropriate bench (57 m level) on the way down with the excavation. They would then have had to be cast from staging off the 50 m level, further delaying benching beneath this level. The difference between the 'in line' effect of these brackets and the non-critical construction of the columns gave an even greater saving than the alternative turbine erection scheme. The remarkable interaction of these modified features produced significant improvements to the overall programme.

Mr B. G. T. Copeland, James Williamson and Partners

Two topics deserve more than a passing mention: the use of sprayed concrete throughout the tunnels and the underground cavern works, and the use of pulverized fuel ash (PFA) in the concrete of the main works.

391. The contribution of sprayed concrete to the overall safety of the underground works cannot be overstated. Of the five fatalities which occurred during the project only one was underground, and that was in an area which had not been sprayed.

392. Sprayed concrete was used both for the temporary support in tunnels which were later to be concrete lined, and for the permanent support in cavern roofs and access tunnels, for example. Most of the sprayed concrete was applied by a specialist sub-contractor using the dry mix process with remote control spray booms and special mobile mixing units. The remainder was applied by hand held guns fed from standard machines. The quality of the sprayed concrete was heavily dependent upon operator skills, and this quality was monitored in the main by core sampling.

393. In the high pressure shaft the wet mix process was used. In the initial layer of the sprayed concrete which was applied in tunnels and caverns immediately after excavation an accelerator was used, but this was omitted in later structural layers. These permanently exposed surfaces were mesh reinforced and a series of trials in the early part of the excavations established that a small mesh (a 75 mm × 75 mm × 2.5 mm dia. mesh) gave the best results since it could be hammered to conform with the rock surface, and this could be successfully sprayed where it overlapped. The measured rebound was high at times and varied between 19 and 30%.

394. The caverns were of the span of this room and extensive. The vertical upstream face of the machine hall was 180 m long and 50 m deep. Any water that arrived in these surfaces was led down through drains which were incorporated in the sprayed concrete behind the mesh reinforcement. The finished sprayed surface proved to be an ideal indicator of strain and in one of the tunnels between the machine hall and the main inlet valve gallery this was revealed with diagonal cracking as the strain developed between the two caverns as the machine hall excavation was lowered.

395. Turning to the concrete of the works, the engineer had originally specified that PFA should be incorporated in concrete exposed to the soft moorland waters. In the light of the improved performance of PFA and PFA mixes during trials, the contractor decided to incorporate the material in concrete throughout the works. Bleeding of the harsh mixes was virtually eliminated providing the workabilities were controlled, and the pumping performance was also improved. The cohesive nature of the mix enabled a high surface finish to be obtained. The combination of concrete mix and shutter quality and the pumpability of the mix made site supervision straightforward as far as concrete quality control was concerned.

Mr R. D. Harman, CEBG

I would like to make a contribution on the lifetime economics of Dinorwig. The initial economic appraisals in 1972–74 showed it to be the most economic new plant option available at the time. The overall economic comparison of alternative plant options, particularly those which will inherently operate at significantly different load factors, is made normally in the CEBG in terms of net effective cost, which is simply the annual capital plus other fixed charges minus the net annual saving of total CEBG running costs. For comparison of plants of different power outputs, the net effective cost is expressed per kW, i.e. it is in £/kW p.a. Dinorwig, with lower capital cost and longer life than the alternatives, combined with the system operating cost savings derived from reduction of spinning reserve provision from steam plant and from the pumping/generating marginal cost differential, was found in the original study to have a net cost lower than that of new nuclear or fossil fuel generating plant. This was confirmed by studies in 1976.

397. In 1977 the discount rate used in appraisals dropped from the unrealistically high level of 10% (in real terms) to 5%, consistent with the 5% real required rate of return. This had the effect of reducing the capital charges in all appraisals and reducing the apparent net cost for all plants, with the greatest benefit felt by plant of highest specific capital cost, i.e. nuclear. So it altered the general relative standing of different types of plant.

398. In a re-appraisal made in 1978 Dinorwig was still found to be more economic than any other new non-nuclear plant, and also had a net cost lower than that of existing old plant. The system operating cost savings exceeded Dinorwig capital charges, so it was economic irrespective of a need to install more total plant capacity to meet any estimated increase in future demand, and irrespective of any credit for a saving in the net cost of any old plant which might be decommissioned.

399. After construction of any station has begun, the economic appraisal is updated at intervals, and a full appraisal is usually made at about the time a new station is commissioned. Preliminary indications are that the station, with its significant operating cost savings will be in the economic position found in 1978, with a lower net cost than any alternative new plant option other than nuclear.

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400. Cost and savings figures have been published in the CEBG's 'Power News'. Capital cost is £425 m, and operating cost savings are about £50 m per annum. These figures, produced for general appreciation of the cost and worth of the project, have already been questioned and misinterpreted, so some explanations seem to be necessary. The capital cost quoted was the simple sum of actual cost, i.e. the £425 m is not all at the same price level. Costs would be brought to the same price level in a full 'real terms' economic appraisal. The £50 m per annum saving is a saving in the net total operating costs of the CEBG. The corresponding savings figures for future years are currently being re-evaluated and it is the lifetime levelled value of these savings which must be set against capital charges plus other fixed costs for the station, in deriving the net effective cost.

Dr J. A. Charles, Building Research Establishment

The Paper referred to rockfill testing which was carried out at the Building Research Establishment for Marchlyn Dam. A major difficulty with rockfill testing, and what distinguishes it from normal soil mechanics testing, is the size of the material that has to be dealt with. This needs large equipment. At the Building Research Establishment we have large one metre dia. oedometers and tri-axial equipment which will take samples 0.23 m in diameter. Tests on the quarried rock from Marchlyn were carried out in both oedometer and tri-axial equipment. The tests were primarily to establish the differences in geotechnical behaviour between the typical hard slate rockfill and some softer material.

402. The compressibilities measured in the large oedometer could be used in finite element analyses to predict the deformations of the embankment at Marchlyn during construction and during reservoir impounding. Fig. 11 of the Paper shows the movements predicted by such an analysis. The settlements occurring during embankment construction are larger towards the crest, whereas one would expect there to be a minimum value of settlement at the crest of the embankment at the end of construction. However, it would appear that the particular analysis was done in a single stage without modelling the sequence of construction. It is essential to model the stages of construction in the finite element analysis of embankment construction in order to obtain the correct pattern of constructional deformations.

403. The shear strength of the two types of slate rockfill (the harder material and the softer material) was measured in triaxial tests. The softer material gave a much lower shear strength than the harder material. The failure envelopes were markedly curved and the stability analysis should allow for this. The Building Research Establishment has recently examined the effect of curvature of the failure envelope on stability of a rockfill slope (Charles and Soares, 1984).¹²

404. The other property of the rockfill that is of interest is the permeability. The permeability can govern the behaviour of the material. If the permeability is very low then the rockfill really behaves as an earth fill and not a rockfill. Tests were carried out in a 150 mm diameter permeameter on different gradings of the rockfill. There was a very marked change with more than about 20% of the material finer than 0.6 mm. The permeability went down from 10^{-6} m/s to a permeability of less than 10^{-8} m/s. Confirming the specification at Marchlyn for more than 20% to be finer than 0.6 mm.

Mr N. Paine, Binnie & Partners

In § 54 the Authors remark that, with hindsight, fewer caverns would have been better. How wide do they think they could have made the main cavern? Is the

implication that they could have included perhaps the inlet valves and possibly even draft tube valves, increasing the width from 24.5 m as it is at the moment, to 34 m or possibly even 43.2 m?

406. If they had been able to go wider, perhaps even to 40 m, would they have contemplated incorporating the transformer hall? A few major underground power stations have the transformers in the machine hall despite the risks from explosion and fire. Could the authors comment?

407. On a different topic: steel lining of the high pressure tunnels starting 100 m from the inlet valve gallery is short by conventional standards. In § 116 the Authors say that the length was deemed adequate to minimize seepage from the concrete lined tunnels into the caverns. Has the length proved to be adequate, i.e. is seepage very small? What were the ground water pressure fluctuations and the bulk permeability that enabled them to make this major saving in steel linings?

Mr M. W. C. Williams, *North of Scotland Hydro Electric Board*

In the early 1970s the Hydro Board was carrying out investigations for a similar scheme at Loch Lomond. Unfortunately, this scheme has been shelved sine die because of lack of growth in demand for electricity. One of the favourite layouts for Craigroyston was to have the main inlet valves and the transformers in a single cavern alongside the main machines, to cut down on the intersections with the main hall excavation. Had the Authors considered this layout? Because of limitations on the span, if this type of layout is chosen it is necessary to locate the main transformers in between the machines. This makes for a rather long hall and, if back-to-back starting of the pumps is required, leads to complications in the bus-bar layout. Maybe this is what precluded that type of layout.

Mr B. Seddon

At an internal pressure of 590 m Dinorwig has the highest internal pressure of any concrete-lined tunnel now in service. More discussion of the tunnel design philosophy at Dinorwig compared with Ffestiniog would be useful, particularly as grouting of the rocks surrounding the concrete lining was omitted. What are the Authors' views on the upper limit of pressure for concrete-lined tunnels—if 590 m, why not 750 m or 1000 m?

Mr J. C. A. Roseveare, *Fellow*

In § 17 the Authors say that, due to the submergence required, the choice of an underground station was a foregone conclusion. Is that still their view?

411. When I was involved with Ffestiniog, of course the turbine itself was really suspended in mid air so there was concern about the embedment of the turbine spiral casing. How was the spiral casing embedded? At Ffestiniog it was thought necessary to provide a compressible membrane around the spiral casing. There is no mention of that here but it is partially surrounded. What are the Authors' thoughts on this matter which, when we did it 20 years ago, caused considerable controversy.

412. Ffestiniog was the last power station in the UK which was completed within months of its scheduled commissioning. It seems inevitable now that the delays are measured in years. Could the Authors pinpoint what causes the delay? There must be some significant factor which makes a delay of three and a half years on the commissioning of the station. It does the profession no good for there

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to be this delay. It is a British disease and should be borne in mind when any future project like this is being planned.

Mr A. W. Shilston, Consulting Engineer

The existence of consulting quantity surveyors on this project (presumably instructed direct by the CEGB) is referred to, with the utmost brevity, in § 170 and in Appendix 1. Presumably they had a substantial role to perform.

414. At what stage in the evolution of the scheme did they appear? Previously, I have been astonished to hear from a senior partner in a leading firm of consulting quantity surveyors, that the cost planning of new dams could be based on updated priced bills of quantities from completed dam schemes undertaken elsewhere.

415. In actual fact, a forecast of price had been established by an estimating team from the design consultants' organization, headed by a contractor's former estimator. The team visited the proposed dam site, conducted investigations and made enquiries, and duly engaged in a first principles analytical estimating exercise similar to that adopted by a tendering contractor with a heavy civil engineering background.

416. No doubt independent quantity surveyors would have a useful supporting role to play in the fulfilment of a scheme such as that undertaken at Dinorwig but presumably not at the design stage as I believe is increasingly the practice in building design ('cost planning').

417. Mindful that in the execution of the contract works there were *pre* and *post* September 1979 stages (§§ 175 and 176), could the Authors describe the detailed role of the consulting quantity surveyors, as part of the Employer's multi-disciplinary team?

Mr F. W. Coates, Atomic Power Constructions Ltd

Dinorwig has received a great deal of media coverage in the past few years, and it is most important to have in the records of this Institution such authoritative papers by authors who played leading roles in the development and construction of the project.

419. It is just 20 years since the first major pumped storage scheme in Britain, at Ffestiniog, was fully commissioned and it is interesting to note how both machine sizes and the scheme size have increased in that time. The Paper lists a dozen schemes with considerably higher capacities which have been developed throughout the world since then. It is also interesting to note how attitudes to pumped storage have changed over the years. Twenty-five years ago there was a powerful lobby of engineers in the electricity supply industry who were quite convinced that investment in pumped storage was a waste of time and money. Fortunately their views did not prevail and there was a complete turn round of attitude after just one year's operating experience of that first scheme. Attitudes of the general public were quite different too. Powerful and articulate organizations established themselves to defend North Wales against hydro pumped storage developments. In today's climate it is almost incredible to recall that 25 years ago more objections were levelled against a pumped storage scheme at Ffestiniog than against a nuclear power station half-a-dozen miles away at Trawsfydd.

420. Ffestiniog incorporated an above ground power station and was probably the last of the separate pump and turbine layout scheme. Consideration was given at the time to undergrounding the station, but the unit cost of rock excava-

tion coupled with the adoption of separate pumps and turbines which necessitated long machine shafts and deep machine chamber made that alternative not viable. Nevertheless, there were still 1 million tons of rock excavation in the scheme—most of it at the power station.

421. Ffestiniog was justified at a time of capacity need on the system, by comparison with an alternative coal fired station, and no benefit was attributed to spinning reserve capability. In fact, that facility at Ffestiniog was incorporated after initial commissioning with only minor modifications. The project was justified on a 300 MW S.O. capacity and immediately after commissioning was uprated by 20%—again with only very minor modifications, resulting in a construction cost of some £40/kW. An interesting reflection on the cautious approach to the scheme and on the design margins built into mechanical and electrical plant in those days!

422. Inflation has played havoc with comparisons of cost/kW, but a simple multiplication of construction costs indices suggest that £40/kW then is equivalent to some £300 kW at 1983 price levels. Cruachan was slightly cheaper. It is not clear from the Paper what the overall cost/kW S.O. at Dinorwig has turned out to be. It would be interesting to see a figure at 1983 price levels. What prompted a change in capacity from the original 1350 MW preferred to the final 1800 MW chosen? Presumably optimum capacity was to a large extent determined by the unit costs of rock excavation originally estimated. Could the Authors comment on the actual unit costs achieved as described in the Paper in relation to the original estimates?

423. Dinorwig is the culmination of a massive programme of capital investment in North Wales by the CEBG which has extended over the past 25 years and it is pleasing to acknowledge that James Williamson's and Sir Alfred McAlpine have played major roles in most, if not all, of the projects. With two pumped storage schemes and two nuclear power stations, the electricity industry has invested over £2 billion at 1983 price levels.

424. The site labour effort has been of the order of 50 000 man years in the same period—equivalent to a continuous labour force of 2000 men—a massive contribution to the economy of North Wales by any standards. It is gratifying to note that the clauses relating to employment of local labour and also on landscaping treatment procedures adopted for the first time in the Ffestiniog enabling Act were repeated for Dinorwig and proved equally successful. It is clear that the site enjoyed markedly better industrial relations than other large construction sites during the 1970s. There is, of course, a world of difference in the site labour situation on a pumped storage scheme in North Wales and for example a 2000 MW conventional or 1200 MW nuclear station close to our industrial conurbations. The labour mix is so different, for example.

425. I hesitate to draw simple conclusions on anything so complex as industrial relations on large construction sites—but of one thing I am sure—geographical location plays a major part. After reading the Paper I looked back over personal notes and came across a statement I made on press day at Ffestiniog in 1963, to the effect that the site had been particularly free of labour disputes and loss of time due to strikes had been negligible, and that at a time when serious industrial relations difficulties were being encountered elsewhere. Similar statements could have been made for both Trawsfynydd and Wylfa.

426. My conclusion from this is that the projects in North Wales provided in general very well paid jobs in an area badly in need of them, and with a long history of quarrying and slate mining, and all were of such magnitude that they

generated a sense of pride and achievement in the labour force working on them. I am sure Dinorwig benefited from this—as did the others.

427. As a pumped storage enthusiast, it is a little sad to read in the summary that Dinorwig is expected to meet system needs in England and Wales until the turn of the century. Presumably this conclusion must derive in part from the anticipated capital cost per kW of any future scheme coupled with the forecast of the differential costs of pumping and generated energy in 20 years time and the shape of the system load curve. It is a sad fact of life that each pumped storage scheme on the system must make each succeeding scheme just that much more difficult to justify economically. Unless of course the unit costs per kW of installed capacity can be reduced by finding a better site or by simplifying the plant or by deriving improvements from the economies of scale, but after the magnitude of Dinorwig the latter seems hardly possible. Have the Authors any thought about how such reductions in cost might be achieved in the future in the light of their experiences at Dinorwig?

Dr T. G. Carter, *Golder Associates, Toronto, Canada*

I would like to expand on the geotechnical aspects of the project. While working for Binnie and Partners, I was part of the design team involved in the planning and supervision of the site investigations for the dams and reservoirs. The geotechnical design work for the dams brought together a wide spread of expertise encompassing engineering geology, soil mechanics, rock mechanics, hydrogeology, engineering geophysics, and glacial process geomorphology.

429. As a consequence of this diversity some novel approaches were taken during the site investigations. Numerous alternative ideas were evaluated before reaching the designs now constructed. In §§ 67 and 68, for example, the principal reasons for selecting an asphaltic membrane for Marchlyn are explained. However, before this decision, when a clay cored rockfill dam was still considered a viable design, an extensive search for an economic source of clay borrow was conducted northwards from the Snowdonia National Park boundary to the coastal strip. For a variety of reasons, including haul distances and environmental considerations, no suitable source could be located, thus further strengthening the arguments for use of a synthetic membrane.

430. At each site in these investigations and in the foundation investigations at Marchlyn and Peris, the glacial overburden, whether exposed at surface, or in test pits, or recovered from boreholes, was described not only in engineering geological terms but also by means of *glacial fabric diagrams*.¹³ Engineering geological maps were compiled covering each dam site and reservoir basin. In 1973, such maps were forerunners of much of what is now commonplace following wide acceptance of the *QJEG*¹⁴ guidelines on standardized engineering geological mapping methods. Fig. 49 illustrates part of one of these maps and shows the left abutment area of the Marchlyn Dam site.

431. Because of the relatively high permeabilities measured in lugeon tests¹⁵ performed in this area of the dam foundation, it was decided early in the detailed site investigation stage that the dam's grout curtain should extend well into the spur. Mapped groundwater spring locations when considered together with drill-hole piezometric head data suggested that the dolerite dyke cutting through the spur constituted a groundwater barrier. The dyke appeared to act as a weir to water flow moving from the high ground to the south and west (Fig. 1) and thus could potentially form an effective abutment for the required grout curtain. A

Fig. 49. Engineering geology of left abutment area of Marchlyn Dam Site

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decision was therefore made to preserve the hydrogeological integrity of the dyke; and, as a consequence, a programme of magnetometer profiling¹⁶ rather than drilling was utilized to map the dyke's locations between visible outcrops.

432. In addition to acting as a potential leakage pathway, the rock structure of the spur was also of concern from a dam stability viewpoint. For conducting the stability analyses outlined in §§ 79 to 84 of the Paper, careful discontinuity surveys were completed at over 200 surface stations across the spur. Discontinuity data was also determined from azimuthally oriented drillhole core. Five deep drillholes in the spur area were selected for systematic detailed engineering geological logging and photographing. The core was carefully oriented with respect to the dominant bedding present in the quartzitic slates as a means to determining the dip and dip direction of the primary rock mass discontinuity trends. Over 600 individual fracture orientation measurements were completed as indicated on Table 20.

433. This type of joint data collection is usually only undertaken when core is available from holes within which mechanical core orientation (using the Craelius, Christensen-Hügel or other methods) has not been undertaken but from which discontinuity data is required. However, the method is limited in applicability to holes which intersect at a steep angle a consistently oriented datum fracture set, such as, for vertical holes, a 30–60° dipping bedding, cleavage or foliation suite.

434. At Peris, as only 3 m of freeboard was required for any dam structures impounding the lower reservoir, major earth or rockfill embankments were not necessary. Instead, because of limited natural stream inflow and potential leakage through permeable sands and gravel, conservation of water within the reservoir was considered the prime requirement.

435. Investigation of foundation conditions beneath each end of Llyn Peris again brought together a wide spectrum of engineering geological techniques. In addition to surface mapping and photo-interpretation to delineate overburden/bedrock boundaries and glacial fabric trends, several large diameter, fully sampled wells and numerous conventional soil borings were put down through the 90 m deep glacial deposits.

436. To map the horizontal continuity and characteristics of the overburden materials, use was made of cross hole and uphole seismic profiling.¹⁷ Standard Hvorslev methods of permeability testing¹⁸ were supplemented by Theis and

Table 20. Marchlyn Dam rock spur core discontinuity data collection

Borehole	Orientation*	Data collection interval: m	Number of discontinuity measurements	Description and data collection
M44	45° to N315°E	5–41	125	All joints logged in a 1 m section every 5 m along the borehole length
M39	vertical	15–31	25	
M38	vertical	10–38	84	
M 9	vertical	5–100	110	
M37	vertical	4–20	105	All joints logged over intervals indicated
		20–40	136	
		40–60	102	

* Bedding assumed at constant strike of N040°E.

Jacob type analyses of monitored pumping tests.¹⁹ During pumping, impeller flowmeter profiles²⁰ were generated for establishing the hydraulic conductivity of various individual glacial horizons. On the basis of engineering geological cross sections compiled from this data, the diaphragm wall cutoffs were designed not to extend to bedrock at 90 m depth but rather to terminate within low permeability peat or silt layers that occurred beneath the upper 15 m of sands and gravels.

437. As an adjunct to these overburden investigations, detailed stability analyses were undertaken of the slate tips which had encroached onto the glacial lake sediments along the north side of Llyn Peris. After much difficulty penetrating the blocky slate wastes, rotary drillholes were advanced through the base of the tips into the glacial lake deposits. Undisturbed sampling of the soft laminated glacial clays was carried out using sophisticated drilling techniques. Coring was undertaken with a French Mazier retractable-shoe triple-tube core barrel²¹ not previously used in the UK. The quality of core recovered allowed an assessment to be made of the extent of lake floor disturbance by the slate quarry tipping operations (see Fig. 38). High quality undisturbed samples were also obtained for strength testing.

438. This use of the most appropriate site investigation tools available for the particular job typified the approach adopted throughout the fieldwork. However, because of the complexities of the foundation conditions and the limited data that could be collected in even these most detailed investigations, the dam contracts were formulated to allow sufficient flexibility for design modifications during construction if so required. A full-time resident engineering geologist was therefore assigned to the dam engineer's site staff with responsibility for evaluating the accuracy of the site investigation data in the light of as-exposed ground conditions. That few modifications were required to the design by virtue of differing ground conditions is an indication of the thoroughness of the pre-design investigations and a credit to both the Consulting Engineers and the CEGB for sanctioning the use of many sophisticated site investigation techniques, some in their infancy and others largely untried in conventional practice.

Dr L. R. Richards, Golder Associates

The main underground complex of the Dinorwig Power Station is located in the sequence of Lower Cambrian slates beneath the 550 m high Dinorwig Quarries. The slates in this area have been exploited by surface and underground workings on a vast scale for some centuries. There is consequently a great deal of local experience in the working of the material. However, international experience on large underground excavations in slate appears to be very limited. The extensive bibliography on large underground excavations compiled by Hoek and Brown²² does not list any other caverns created entirely in slate.

440. The geotechnical parameters of most relevance to underground excavations are the strength of the intact rock material and the characteristics of the discontinuities in the rock mass. In the case of Dinorwig, the rock material is very strong and the discontinuities are unusually planar and persistent.

441. The powerhouse complex lies close to the axis of an anticline plunging SSW at 30°. Nine different types of slate can be identified in the Lower Cambrian sequence. The engineering properties of the intact rock materials are not significantly different. The underground works are also cut by a network of metadolerite dykes which are generally subvertical and have highly sheared margins (see Fig. 50). The strength of the slate varies considerably depending on the angle between

Fig. 50. Geological plan of powerhouse complex

the direction of the applied load and the cleavage planes. Fig. 51 shows that the material has an anisotropy ratio of over 4 for uniaxial compressive strength. Because of the powerhouse orientation in relation to the cleavage direction, the minimum strength appropriate to the cross sections of the main caverns is over 100 MPa. The strength of the doleritic material is not influenced by the direction of loading and has a mean value of about 125 MPa. Similar conclusions apply to the modulus of elasticity as measured in uniaxial compression tests. However, the anisotropy ratio is closer to 3 for this parameter.

442. The virgin stresses in the rock mass were measured by an overcoring technique. This involves fixing a strain cell in a small diameter pilot hole. Strain readings are taken before and after overcoring which allows the three-dimensional state of stress to be determined (Fig. 52). The vertical stress calculated from the depth of cover is about 9 MPa. The measured major principal stress is about twice this value and has a direction normal to the cleavage planes in the slate. The other principal stresses are slightly less than the overburden stress and somewhat inclined to the vertical.

443. The induced stresses around the major openings were analysed by boundary element and finite element techniques. Fig. 53 shows a boundary element stress analysis of the machine hall. The in situ stresses shown on this figure are those in the cross section of the machine hall; the major principal in situ stress acts approximately normal to this section and parallel to the axis of the cavern. The typical value of the major principal induced stress around the machine hall circumference is 0–20 MPa. This is considerably less than the intact rock strength and indicates that overstressing of the rock material is unlikely.

444. There are discontinuities in the rock mass in the form of very well developed sets of joints and faults as summarized on the stereographic projection in Fig. 54. The most closely spaced joints and faults are parallel to the cleavage planes. Other main joint sets are subvertical and subhorizontal and thus give rise to a blocky structure in the rock mass.

445. The main cavern direction is nearly normal to the main discontinuities and subparallel to the major principal stress. This is a favourable direction.

446. The joints and faults are typically planar and continuous with laboratory-determined friction values of about 30° . The field shear strength is very similar to the laboratory value because of the very smooth nature of the discontinuity surfaces.

447. A major part of the cavern design was concerned with the identification of potential structural release mechanisms. Computer and stereographic techniques were used to identify kinematically feasible blocks and wedges which could fail into the excavation. The closely spaced and persistent nature of the discontinuities was such that rock support requirements were almost entirely dictated by the structural features in the rock mass.

448. Observations of rock movement in response to excavation were carried out during a trial excavation of the main cavern and as an integral part of all main construction work. Despite the jointed nature of the rock, the deformation response observed was very compatible with that predicted by pseudo-elastic calculations. This was considered to be due to the favourable stress state in relation to the joint pattern. The main exceptions to the above were movements in the vicinity of the dyke margins which were up to 5 times those in the slate. The maximum rock movements were typically 25 mm at the sidewalls which is small in comparison with the main cavern dimensions of 24.5 m span and 60 m height.

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Fig. 51. Unconfined compressive strength of slate

Fig. 52. In situ stress measurements

Fig. 53. Boundary element analysis of machine hall

Fig. 54. Stereographic projection of discontinuities in main underground complex

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449. The Dinorwig geology is relatively unusual for large underground excavations in its combination of uniformly strong, highly anisotropic rock material with closely spaced and persistent discontinuities. The engineering difficulties of working in this geological environment were minimized by the favourable orientation of the in situ stresses and the favourable directions of the prevalent sets of discontinuities.

Mr T. J. M. Paterson, *Geostore Ltd*

At Dinorwig the main civils contract went out to tender long before completion of the exploratory tunnels. This put tendering consortia at considerable risk. The result was the refusal to tender on a normal measure and value basis. The momentum of the project was maintained but contractual risk had to be carried by the CEGB. At the other extreme it would have been possible to complete the exploratory tunnels and trial enlargement before tenders were issued. Project momentum would have been reduced but technical and contractual risks would have been much reduced. If Dinorwig were to be repeated how much of the exploratory works would the Authors recommend should be completed before tenders for the civil works were issued?

451. The Paper makes no mention of the rock mechanics team (RMT) formed as part of JWP's site staff. This team combined the efforts of engineers, geologists and technicians, with the support of a site laboratory. The RMT could analyse and solve problems of rock stability, rock support and rock instrumentation on site as they arose. The RMT was set up after the failure of 56/1 portal and was a new concept in the UK. It was one of the successes of Dinorwig.

452. Shafts were sunk at Dinorwig by the main contractor (headgate and surge shafts), by specialist contractor (Thyssen—ventilation shaft) and by specialist subcontractor (Thyssen—HP shaft). Shaft sinking by the specialist contractor was markedly faster and more efficient than shaft sinking by the main contractor.

453. At Dinorwig it proved impossible to drill the pilot hole for the raised bored HP shaft mucking raise to the required accuracy beyond a depth of about 300 m, despite redrilling of the pilot hole to full depth. At greater depths it may be necessary to specify for the pilot hole the use of the drills employed in deep freeze hole drilling. These drills can be deviated to offset any misalignment and can drill deep holes with great accuracy. This technique has been used in West Germany and the USA for raise bored shafts. The HP shaft was sunk as a sub-vertical shaft with the headgear in the orifice shaft and winders in a separate cavern. This allowed simultaneous construction of the surge shaft and the HP shaft below it. The high speed skip winder was required only to supply concrete for shaft lining by kibble. If concrete had been delivered by vertical pipeline, as at Selby, the cost of the shaft could have been reduced.

454. The raise drilled mucking shaft used for the sinking of the HP shaft was blocked when the mucking crew in the HP tunnel below allowed spoil to stack up in the mucking shaft to a depth of some 100 m. Various attempts to clear the blockage failed. Eventually it was agreed to raise a new bypass section of mucking shaft by Alimak raise climber. The use of the Alimak raise climber had not originally been permitted at Dinorwig because of safety problems in previous Alimak usage in the UK. The new mucking shaft was completed very quickly and efficiently with the Alimak by the Austrian subcontractor OSTU.

455. What was the total length of raise drilled by the raiseborer at Dinorwig?

456. The lining of the HP shaft is of interest not only because of the 402 m depth of lining slipformed but also because slipforming was only semi-continuous. Did these planned stoppages at weekends cause any problems in the quality of the shaft lining?

457. In the lining of the major hydraulic tunnels the invert and arch were concreted separately. The invert was concreted both before and after the arch. Do the Authors see any particular advantage in either method? The hydraulic tunnels were concreted using slide-valved ports in the crown of the shutter rather than via slickline. This resulted in very good filling of the crown void. Can the Authors confirm that the take of cavity grout in the crown was almost negligible?

458. What were the final design criteria for contact and fissure grouting in the hydraulic tunnels?

459. In view of the problems of wear and damage of the hydraulic linings found in previous pumped storage schemes in the UK and abroad it would be interesting to have initial feedback from Dinorwig. Has any inspection of tunnel linings after dewatering been carried out to date? If so with what results? Bernold lining was used on a small scale at portals and in the diversion tunnel. It was not very successful. On this limited experience it would appear that if more than, say 100 m of tunnel is to be lined a conventional shutter would be a better alternative.

460. The CEBG's pressure for the inclusion of foreign expertise in the tendering consortia was fully justified by the considerable contribution of the small number of experienced Swiss engineers seconded by C. Zschokke to fill key positions on MBZ's site staff.

461. The management of the civil works at Dinorwig was often Byzantine in its complexity. In particular, changes in both construction methods and design, some major, appeared to be made on an almost daily basis. The presence on site of the CEBG's Project Manager ensured that such decisions could be made quickly.

462. Reference was made in the verbal discussion by a CEBG representative to the fact that Dinorwig ran over budget and over programme. Major design changes, particularly in the layout of the MUC were demanded by the CEBG during construction. If a project of the complexity of Dinorwig is to be built to programme and budget the design must be frozen at an early stage and must remain frozen. Tinkering with the design, however attractive to the client, will inevitably result in delayed completion and increased costs.

463. The CEBG's system of prior agreement by the client, before issue, of costed site instructions was unworkable in practice. In major underground works it is essential to issue instructions on the spot. To wait several days for costing and agreement of an instruction will often result in far greater consequential costs due to delay.

464. The change from a target cost contract to a cost plus contract for the main civil works in September 1979 suggests that the target cost contract was not a success. Was this because the target cost formula was set so that there was a greater incentive for the Contractor to increase the target cost than to decrease the actual cost?

Dr G. P. Sims, Balfour Beatty Engineering Ltd

Three aspects of the design of the steel liners are of particular interest. What criteria were selected by the designers to determine the point at which steel lining was no longer required? Did the designers compare the water pressure within the tunnel with the natural state of stress within the rock? How did they determine the

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safe hydraulic gradient between the end of the steel liner and the surface of the power station cavern? What steel was specified for the linings?

466. The thickness of some parts of the liners was governed by the internal pressure. Did the designers consider reducing the tensile stress in these lengths of liner by imposing a compressive prestress? Such a prestress could have been produced by high pressure grouting between the rock and the liner.

Mr F. G. Johnson, North of Scotland Hydro-Electric Board

The Dinorwig scheme represents a significant advance in pumped storage and its operation will be followed with great interest in coming years.

468. Experience on the Hydro Board's pumped storage schemes at Cruachan and Foyers has shown that the pumped storage regime is very much more onerous than a conventional hydro operating regime and leads to much greater maintenance of both plant and civil engineering structures, with correspondingly longer outage times and higher maintenance costs. The total capacity of Dinorwig of 1800 MW is large and its satisfactory operation is dependent on the plant and structures of the scheme performing satisfactorily. Are the designers of the scheme confident that one high pressure tunnel is satisfactory to serve the six machines? At Cruachan, where only four machines are installed, it has been found that the two penstocks are a valuable feature of the scheme, permitting half the plant to be taken out of service without completely losing the output of the Station.

469. The very rigorous operational requirements for changing power on the station will impose very severe operating requirements on plant and structures. On both the Cruachan and Foyers Schemes maintenance on tunnels and shafts has been at least an order of magnitude greater than on comparable conventional hydro schemes. This is probably due to the bi-directional flow of water which is much more erosive than the uni-directional flow found in conventional hydro schemes, the very numerous operating mode changes which are characteristic of the current operation of pumped storage schemes and which each lead to a pressure cycle and hence stress cycle and, thirdly, the higher velocities which have been adopted in recent pumped storage schemes compared with earlier hydro schemes. The tendency in recent years for system operators to require increasing numbers of mode changes per day and to specify even shorter start-up and shut-down times has compounded maintenance problems for both plant and structures. This was also the practice on the EdF system until recently when these requirements for very short operating times were challenged. After a very rigorous study of requirements, a fundamental revision of requirements took place in which the shortest operating times were specified as approximately five minutes from standstill to full load generation and ten minutes from standstill to full load pump. The very short times now being called for on such schemes as Dinorwig could lead to significant problems with plant and structures and, in the light of the French experience, is it certain that these extremely short periods are necessary?

470. The high pressure concrete tunnel lining will be subjected to heads of over 500 m and there is, apparently, no experience of concrete linings being subjected to these pressures in pumped storage schemes. What criteria did the designers adopt to determine the change from concrete lining to steel lining of the high pressure tunnel and on what operating experience and design data did they base their design? Have any extraordinary measures been taken with the Dinorwig linings compared with conventional hydro tunnel linings?

471. On the Cruachan and Foyers Schemes numerous defects in the concrete

linings of the high and low pressure tunnels were observed within a matter of a year or two of initial operation. These defects were large and quite outwith the experience of any defects encountered with conventional hydro tunnels. Was any special surface treatment or impregnation considered or applied to the concrete lining of the tunnels and shafts and especially the high pressure manifold? The development of low viscosity epoxy type treatments which can penetrate into the surface of the concrete lining is a development which might well ameliorate this problem.

472. In view of the loads and the major changes in flow which occur in the manifold, was consideration given by the designers to constructing this structure in steel?

473. The Authors state, in § 54, that in retrospect and with the experience gained of actual rock conditions the relative merits of a lesser number of caverns seem more attractive. Could the Authors amplify their views on this subject and indicate how they might plan the station if they were embarking on the design of the station today.

474. What form of contract strategy would the CEBG now adopt if they were building a similar project to Dinorwig?

475. It would be very helpful if the equivalent March 1983 costs could be given for the main contracts covered in Table 16. What are the overall costs for the scheme, including plant, and how do these costs compare with the original estimate for the project?

476. In the presentation of the Papers to the meeting I was surprised to learn that the CEBG are basing the life of Dinorwig on approximately 30 years. From the Board's experience, the lives of the main structures of a pumped storage scheme such as Dinorwig, should be very considerably in excess of this period (eg dams considerably more than 100 years, underground caverns more than 100 years, tunnels more than 50 years) provided proper and regular maintenance is undertaken. Why has such a short period been adopted for Dinorwig?

Mr R. Watts, Balfour Beatty

Paragraph 6 refers to the need for pump storage in England and Wales, and the economic penalties for other means of dealing with 'spinning reserve'. Could the Authors give a comparison of the saving produced by the Dinorwig scheme and an estimate of how much more room there is for further pump storage spinning reserve?

478. In the latter part of the Paper, Tables 16, 17, 18 and 19 give a very good idea of the costs of the civil engineering work involved but the costs of the electrical and mechanical work are not given, thus making it very difficult to form a view of the whole of the Dinorwig scheme. Can this omission be rectified?

Authors' Reply

In preparing the Papers the Authors were conscious of the virtual impossibility of adequately recording the many interesting facets of civil engineering of Dinorwig. We are therefore particularly grateful to the twelve contributors to the discussion who have personally been much involved in the Dinorwig project. They both supplemented material in the Paper and were able to highlight many novel and diverse features, ranging through architecture, dam settlement instrumentation, rock mechanics, construction planning, the plant itself and its economic justification.

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480. Since preparing the Papers, more experience has been obtained on three of the civil engineering topics wherein the Authors suggest Dinorwig attempted to advance the methodology of hydroelectric work. Firstly the experimentally based method of determining rock anchorage requirements, §§ 124 and 125 continues to be fully satisfactory. In metamorphic rocks this work has demonstrated the dominant importance of maintaining a coherent rock structure immediately below exposed surfaces and perhaps the relative unimportance of the deeper longer reinforcements.

481. The choice of the single skin asphalt concrete membrane for Marchlyn Dam was novel in its requirement to meet water level movements greatly in excess of those normal in water supply applications. Although still not fully tested, the operational experience to date has included rapid movements of level. The dam has responded in a fully satisfactory manner with leakage rates being readily tolerable.

482. The decision to limit severely the amount of grouting behind the concrete linings of the hydraulic tunnels has similarly given no cause for concern. Leakage rates into the main complex were initially small and are tending to reduce with time.

483. The Authors have a few comments on the contributions of co-workers on the Dinorwig scheme.

484. *Mr A. G. Cook* refers to the decision to remove all the rock below the loading bay in the machine hall as arising from the need to create larger plant area. While this advantage accrued the original reason arose as a result of design development whereby the combination of the excavation of temporary and permanent openings through the rock supporting the loading bay and the rock reinforcement necessary to secure it was shown to be uneconomic compared to simple removal of the remainder. The additional space created under the loading bay was then utilized by the CEBG for improved plant layout and simplified access and maintenance throughout the length of the machine hall at the turbine floor (34 m) and generator floor (42 m) levels.

485. *Dr L. R. Richards* refers to the importance of the strength of the rock material and characteristics of discontinuities in the rock mass. He concludes with a comment on the favourable orientation of the *in situ* stresses. The main point to emerge in the later stage of construction and, in particular, during the formation of numerous openings around the main caverns, was the considerable advantage which arose wherever any residual *in situ* stress remained in the rock. The value of the stress was less important. For example, during the last breakthrough in the machine hall, openings in the upstream wall remained intact even where these were in close proximity due to the vertical component of stress remaining even though the horizontal components were no longer present.

486. *Dr Richards* refers also to the rock movements or deformational response being very compatible with those predicted by pseudo-static calculations. This comment is appropriate only for the very early stages of excavation, including the roof of the machine hall where very small movements were registered (Figs 55 and 56). It did not apply, however, to later stages of the cavern excavation when, for example, movements at about 1/3 of the height of the upstream wall averaged around 25 mm (with a maximum of 48.4 mm)—in comparison the mid height theoretical movement was 6 mm (Figs 57 and 58). This apparent discrepancy arose almost entirely due to near surface loosening (Fig. 59) since, for economy and to permit improved rates of excavation, benching work was permitted to proceed

Fig. 55. Machine hall roof: comparison of theoretical and actual movement

well ahead of the installation of rock tendons.

487. Mr T. J. M. Paterson's statement on the rock mechanics team is useful but does not fully explain their role. The design work was initiated and directed from the Consultant's Head Office and the rock mechanics team had two principal functions.

- (a) To gather geological and geotechnical data as excavation proceeded and to pass the data periodically to Head Office.
- (b) To interpret the design requirements specified from Head Office in the light of exposures as these were formed so that 'on site' decisions could be taken immediately. These interpretations were always transmitted to Head Office for ratification or further directions, where required.

Mr Paterson also comments on the accuracy of drilling the pilot hole prior to raise boring. Various techniques have been used, particularly by oil companies, for adjusting the deviation of deep drill holes. Since the raise boring work was carried out at Dinorwig, there have been further developments also in blind hole drilling with machines available now to drill holes up to 4 m or more in diameter to depths of over 500 m.

488. While it is accepted that the use of 'bernold' steel linings was not wholly successful, the use of this material had one special advantage—it could be set up

Fig. 56. Plan of machine hall roof showing movements (mm)

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fairly quickly and blocked locally against the rock in advance of placing the concrete linings. The introduction of a conventional shutter would have taken longer and could have delayed, therefore, the introduction of early support in broken zones.

489. *Mr N. Paine* asked how wide the main cavern could have been made. With the benefit of the experience gained at Dinorwig, it was considered the spans could be formed in similar rock types of at least 35 m and possibly 45 m or more. After all, there is now a 60 m height (or vertical span) formed in that material. However, at the time the span was chosen it was limited to 25 m not only because it was considered a realistic maximum in relation to past experience, but also because the opening up of roof spans in excess of 25 m would have been a slow and

Fig. 57. Comparison of theoretical and actual upstream wall movements

Fig. 58. Machine hall upstream wall movements (mm)

costly operation which could have delayed the earliest practical completion of the project—our basic task.

490. Mr Paine also asks whether, with a wider span, the transformer hall could have been incorporated in the main cavern. As mentioned by Mr M. W. C. Williams, this is being considered as a potential layout for the Craigroyston scheme in Scotland and the question is asked as to why this layout was not considered at Dinorwig. The two schemes are different in that the upper and lower reservoirs are much closer together at Craigroyston and the banks run parallel for some distance. The potential for second (and possibly third) phase extensions

Fig. 59. Effect of near surface loosening on movements

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favours a longer cavern layout which could accommodate the transformers 'in-line'.

491. The enhanced possibility of explosion and fire raised by *Mr Paine* in the larger cavern would not be a governing factor in layout. Acceptable levels of these hazards could be engineered with confidence whether or not the single or multi-cavern approach was followed. At Waldeck II the transformers have been installed in the same cavern as the machines. This arrangement has normally been for two unit stations with the transformers located at the end of machine hall, but the Authors see no real impediment to the solution being adopted on larger stations.

492. *Mr Williams* raises a similar question on the possibility of incorporating the MI valves in a larger machine hall. At Dinorwig at the time of tender, doubt remained as to whether single or dual valves were to be adopted. As the latter would have involved having a considerably larger MIVG, it was clearly prudent for the MIVs to be in a separate cavern, although this was more a matter of prudence than an inevitability. With single valves, as at Dinorwig, the Authors' again see no impediment to them being incorporated in the single large cavern.

493. *Mr Paine* asks, further, about seepage into the cavern and data on this has been presented to the ISRM Symposium in Cambridge, September, 1984 Fig. 16.²³ *Mr Paine, Dr Sims and Mr Johnson* refer to the length of steel linings adopted. At Cruachan the seepage path from the upstream end of the steel linings to the cavern was 25% of the static head. The confidence gained from the water-tightness achieved there led to a figure of 20% being adopted at Dinorwig. This is not unique, as at least four pumped storage stations in the USA have carried steel economy as far as this or further. Seepage in the MIVG is not negligible, but has not caused concern. The total flow into the MIVG from all sources rose from 20 to 120 l/min during the initial filling period, but has since declined to 75 l/min. Taking a crude notional flow area equal to the upstream wall of the MIVG an apparent bulk permeability of the order of 10.5 cm/s could be deduced, but this would be a misleading figure as at least half of the total seepage is concentrated at the six penstock tunnel 'eyes'.

494. Although as *Mr Seddon* says Dinorwig has the highest internal pressure (590 m) of any concrete-lined tunnel, the containment problem was relatively more severe at Ffestiniog where minimum rock cover was less than 50% of the head compared with two-thirds at Dinorwig. There is no theoretical upper limit of head known to the Authors provided that the minimum stress in the surrounding rock exceeds the maximum internal pressure, and that it is possible in the first place to drive a tunnel through the highly stressed rock. In the Authors' opinion *Mr Seddon's* figure of 1000 m (10 N/mm²) is well within the strength of both concrete and most of the older rock formations.

495. *Mr Roseveare* asks about the choice of underground power station in relation to submergence. The required minimum submergence of the pump-turbine runners of 60 m below the lowest operating level in the lower reservoir of 92 m AOD fixes the crown of the machine hall roof at around 71 m AOD, making either underground or cut-and-cover construction of the machine hall necessary. The latter was considered but ruled out on grounds of cost and the adverse effect on waterway starting time of siting the station close to the lower reservoir. By locating the station underground much nearer the pressure shaft the rapid acceleration of the water needed to achieve the specified loading time of 10 s from spinning reserve could be achieved with feasible shaft and tunnel diameters.

496. *Mr Roseveare* also asks whether it was necessary to provide a compress-

ible membrane round the spiral. The pros and cons of such a membrane were considered, but the rigorous requirements with regard to machine shaft displacement ruled in favour of as rigid encasement in concrete as possible. A further requirement was that the key welds between the stay-ring and spiral should be left exposed to facilitate future NDT checks. The result was a hybrid partial encasement, which presented problems both in respect of stress concentration where the encasement ends and in the potential for fretting action between concrete and steel. To ameliorate these factors, use was made of a rubber membrane 6 mm thick and up to 500 mm wide to give a measure of transition at the edge of encasement.

497. Mr Roseveare questions the apparent inevitability of large delays in power station constructions since Ffestiniog was questioned by Mr Roseveare and, with few exceptions, overall delays and cost overruns had become the norm. It is interesting to record, however, how often the key dates in the civil engineering programme were met or were very nearly met; the main delays arose in the later work. Due to major efforts to improve the early planning and detailed design before construction and due to a marked improvement in the industrial relations climate on CEBG sites, the most recent stations Drax and Heysham 2 are both very close indeed to their original programme forecasts. It would be incorrect to suggest that the Dinorwig programme and performance compares unfavourably with construction times for similar projects elsewhere in the world.

498. In answer to Mr Shilston's question on the introduction of the consulting quantity surveyor, the Authors would refer him to Mr Shrimpton's contribution. Although the consulting quantity surveyor was appointed at a comparatively early stage in the design development of the scheme, they worked closely with James Williamson and Partners in the preparation of scheme estimates as well as bills of quantities and other elements of the contract documentation.

499. Mr Shilston questioned the detailed role of the quantity surveyor in the multi-disciplinary team. Apart from the 'conventional' duties of bill preparation, tender preparation and assessment, financial administration of contracts and the provision of a contract accounting service to the CEBG project team, the consultant was required to implement a cost control system used on all recent CEBG power stations. This system operated throughout the detailed design and construction phases of the scheme and provided a wealth of management information of benefit to Engineer, Consultant and Contractor alike at Dinorwig.

500. Mr Paterson pinpointed a crucial matter in his question on the need for exploratory work to be completed before tender invitation. Due to a series of circumstances falling largely outside the engineers' control it proved impossible to undertake an exploratory tunnel at Dinorwig as had originally been planned. Unfortunately also the single deep drill-hole failed to reach the cavern area. Although the rock quality eventually proved to be good and water conditions in the rock were very favourable, the lack of this data seriously hampered tendering at Dinorwig and, with more difficult rock conditions elsewhere, could have proved disastrous in cost and programme. The Authors recognize that the outcome of the omission at Dinorwig was fortuitous, and are unanimous in recognizing the essential need for such work as a norm on any such major tunnelling project.

501. Mr Paterson's reference to the work of the rock mechanics team at Dinorwig is welcome and appropriate. They proved to be a valuable adjunct to the Engineer's site establishment throughout the main excavations. More detailed comment on their role is made above. The total length of raise bored shafts at Dinorwig amounted to 1113 m. We are grateful for Mr Paterson's contribution on

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the adoption of this technique at Dinorwig.

502. The planned stoppages when slipforming the HP shaft did not cause any problems in the quality of the lining. In addition to the seven planned weekend stoppages there were three due to breakdowns. At the end of a pour and just before the start of the next pour, precautions and preparations recommended by the C&CA were carried out with the result that sound, smooth joints were obtained.

503. Mr Paterson asks, further, about the advantages in either concreting the arch or the invert first when lining the hydraulic tunnels. In leading with the invert concrete, (HP Tunnel/LP Tunnel), the principal benefit was derived from the first class working surfaces obtained, from which the arch formwork could be accurately and quickly set up. Against that there was some difficulty in forming top edges of invert slab when forming radial joints. It was also essential that the invert concrete be protected during subsequent construction from heavy traffic on older concrete and from other equipment on greener concrete.

504. In concreting the arch first, as in the tailrace tunnels, it became readily possible to maintain access through the formwork for transportation of steel linings to draft tubes. Hence programme advantages were gained. It suffered the disadvantage however of requiring extra work in constructing kerbs in advance of the arch form so that bottom edge of form could be secured accurately in place.

505. In summary the Authors suggest that the choice of whether invert is constructed before or after the arch depends entirely on the problems faced on a particular job. The take of cavity grout in the crown of hydraulic tunnel linings was negligible. This was particularly evident when concreting 'uphill' where tunnel gradients were up to 10% or steeper in many instances.

506. The relatively small amount of grouting used in the hydraulic tunnels at Dinorwig represented a major departure from previous practice. It was recognized that metamorphic strata with significant existing compressive stresses in all directions were unlikely to prove seriously permeable. Providing the interspaces immediately behind the concrete lining contained minimal voidage, overall leak tightness could be reasonably anticipated.

507. With the further assurance that additional grouting could have been introduced after initial filling and draining of the hydraulic system, the C&CA and its consultants agreed to limit the general grouting to 'contact' only with maximum grouting pressures limited to about 1 N/mm². Fissure grouting of moderate depths and pressures was only retained at certain areas of known geological deficiency or of system geometrical complexity. This minimal grout approach certainly achieved significant economies in time and money. Its technical adequacy will not be fully proven until there has been an opportunity to inspect the system after some years of service, but initial results give every prospect of permanent success. The only problem to date which could be attributed to deficiencies in grouting was the initially excessive leakage into the adit and the main inlet valve gallery. Happily this leakage has shown a progressive decrease since commissioning.

508. Dewatering of the partly filled HP hydraulic system took place on 22 October 1981 after which a short inspection took place. Dewatering again took place on 13–18 November 1981 after the HP system had been filled for two weeks. Nothing amiss was noted in respect of the concrete linings on either occasion, except the usual build up of small debris (dirt and such washed from walls of tunnels and shafts) in the manifold sump. Very minor spots of mist were noted at

two locations in the paint system of the penstock linings.

509. The tailrace tunnels have been filled and dewatered on several occasions. Again nothing untoward has been noticed. Where water spills on the flat tunnel invert just below the tailgate during the filling of the tunnels, via the bypass pipes, slight erosion of the concrete surface has taken place.

510. 'Tinkering with the design', referred to by *Mr Paterson*, is far from attractive to the client. Indeed on all power station works one of the principal objectives, if not the main one, is to forbid or to limit such modifications to the maximum tolerable extent. Regrettably however it is almost always necessary to allow some modifications late in the construction stage. At Dinorwig layout changes had to be made by the CEGB often for operational reasons. Changes were also made as a result of proposals by the contractor to improve access for construction or to suit his construction techniques. While design changes should normally be avoided, it has to be recognized that occasionally it may be even more time consuming and costly to be excessively rigid when a better solution is available.

511. *Mr Paterson's* comment on costed site instructions being unworkable is strongly disputed. In the Authors' opinion one of the lessons from Dinorwig, particularly after the full scale roof support experiments referred to, is to question the wisdom of rock support decisions always being taken at local level. In fairly consistent strata as at Dinorwig a basic pattern of support should need little or no modification as excavation proceeds. This does not mean, of course, that local conditions should not be checked at every stage of excavation—an essential requirement. In the CEGB philosophy of project management, site instructions which have more than trivial cost implications should not be issued without consideration of the likely sum involved, unless the matter is of safety or other very considerable urgency.

512. The change from a target cost form of contract queried by *Mr Paterson* was introduced because it was considered that time being spent on the target aspect of the contract by members of the technical teams of all parties could by then be more usefully channelled to the execution of the works. Paragraphs 175 and 176 indicate the improvement.

513. It is an unfortunate fact, arising from the basic arithmetic of target cost contracts, that the great bulk of the work may be regarded as indistinguishable from cost plus and, as such, attracts little commercial attention. The residue is small in content, but of great commercial significance to both contractor and client, thereby attracting undue attention from all parties to the somewhat hypothetical matters surrounding the target cost, probably at the expense of the actual happenings and their consequences.

514. Despite these strictures however, it would be quite wrong to suggest that the target cost contract at Dinorwig was a failure. It served a very useful function for several years and was only discarded at an appropriate stage of the work after the most detailed consideration and agreement by all parties.

515. As noted in § 117 of the Paper, carbon manganese steel to BS 1501/213 and 223, grade 32A, LT30 was adopted for the tunnel linings.

516. *Dr Sims* referred to the possibility of inducing pre-compression in the steel linings by high pressure grouting. This was not regarded as a serious option at the design stage. Grouting was, however, important in ensuring that the steel lining in tension transfers a proportion of the internal load through the concrete encasement into the surrounding rock.

517. *Mr Watts* asked a key question on the economic viability of Dinorwig as

spinning reserve. Perhaps it is still too early to give a full answer as the role of the station in the CEGB system will take a few years beyond the current commissioning period to become fully established. Current estimates suggest that the station will yield operating economies of £50 million per annum when fully in service. Dinorwig will provide the spinning reserve necessary in the CEGB system for the next 15 years at least.

518. In reply to *Mr Watt's*, *Mr Johnson's* and *Mr Coates'* queries on the costs of the other engineering components of Dinorwig the following figures represent the mechanical and electrical work costs on the same basis, i.e. mixed £ due to inflation up to a March 1983 price level: mechanical work, £90.8 million; electrical work, £55.5 million and other costs (including all engineering and back-up services), £51.7 million. The total costs of Dinorwig on the same basis is likely to be about £410 million. In specific terms this gives the cost per kW as about £225, based on a nominal 1800 MW, although for accountancy purposes a lower output figure, related to the mean head, of £241.2 per kW has been adopted. It is not readily possible to compare these specific costs with the £40 per kW quoted by *Mr Coates* for Ffestiniog.

519. *Mr Johnson* drew the Authors' attention to the onerous nature of pumped storage water flow, compared with conventional hydro-operation some years ago. It will be of great interest to extend our experience from that of the NSHEB. In theory it is difficult to see why tunnels and shafts should be more subject to erosion from bi-directional than uni-directional flow provided care is taken in the design of bends, bifurcations and splitter walls to ensure freedom from flow separation and cavitation with flow in either direction. A recent survey of conventional hydro and pumped storage stations in the USA²⁴ has shown there that, although plant outage times and maintenance costs are greater for pumped storage, civil engineering outages and maintenance are much the same. Shafts and tunnels are not significant in this respect. This is in line with the CEGB's experience at Ffestiniog. High standards of concreting are, of course, essential for low maintenance costs under all water flow regimes. No special surface treatment has been applied to the Dinorwig concrete linings. Instead reliance has been placed on the specification of high quality concrete and special attention given to the highest standards of workmanship in the formwork, the placing, compacting and the finishing of the concrete.

520. *Mr Johnson* also queried whether the short response times of Dinorwig are still considered necessary. A major economic benefit of Dinorwig is the fuel cost saved by using it to replace thermal plant as spinning reserve for meeting sudden losses of generation elsewhere in the system. Currently, the largest infeed loss covered is 1000 MW but the possibility is foreseen of future nuclear stations in which each reactor would supply two 660 MW generating units. This will increase the spinning requirement to 1320 MW. Studies made of the system response following the instantaneous loss of 1320 MW, with Dinorwig providing the whole of the spinning reserve, show that to contain the initial fall in system frequency within the lower statutory limit of 49.5 Hz, the 1320 MW must be available from Dinorwig within 10 s. Dinorwig has accordingly been designed to be capable of providing a generated output of 1320 MW in 10 s from the spinning in air mode.

521. In the electricity supply systems of continental Europe and North America, where the networks are interconnected by strong tie-lines for purposes of economic energy transfers, individual generation losses are a much smaller proportion of the total capacity than in Britain. It is therefore unnecessary for their

reserve plant to have the fast response of Dinorwig.

522. Mr Johnson's related question on the number of conduits to be provided at Dinorwig was carefully considered early in the design. It was estimated that the provision of dual shafts and tunnels would cost an additional £1.25 million (at 1972 prices), including the capitalized cost of the increased friction losses if the flow velocities were the same as in a single waterway. Alternatively, if the design had been based on no increase in these losses the capital cost would have been £2.5 million higher. On the other hand, station availability was then valued incrementally at only £400 000 for a 1% increase, indicating that an improvement of some 4% would be necessary to justify a second shaft and tunnel. Such an improvement was not considered feasible. It was therefore decided to adhere to a single shaft. So far there has been no cause to regret this either during construction or since commissioning. A technical factor which reinforced this decision was the avoidance of the safety assurance problems associated with working in a dewatered shaft adjacent to a shaft in service.

523. Consideration was given to constructing the manifold in steel but was dismissed on the grounds of economy.

524. Mr Johnson queries the likely form of contract strategy for a similar project today. As a general rule the CEBG has moved away from the large or 'mega-contract' ideas of ten years ago in favour of smaller civil contracts based on the 'natural' sub-divisions of the particular scheme. For a further Dinorwig type of project this approach would almost certainly underlie the strategy, but the absence or abundance of rock quality data, the lack of availability of adequate constructional space in mountainous areas, the current economic climate, local and national industrial relations situations and several lesser factors could all influence that approach.

525. One can only accept and agree with Mr Johnson's reservations on the 30-year amortization period when applied to the massive and durable civil engineering works that comprise hydro and pumped storage schemes. Thirty years is indeed rapidly becoming a short life even for a conventional fossil fired station, wherein the great majority of the buildings and structures offer a minimum longevity of 40 or 50 years.

526. The opportunity to repeat major civil engineering artefacts occurs relatively rarely for any engineer and yet *Mr Coates'* question on how reductions of cost could be achieved in future similar projects has great appeal and is totally relevant to the learned body function of the ICE. Given Dinorwig's quality of rock and water conditions, the use of larger caverns, or even of a single cavern probably offers the greatest potential economies. Improvements in rock support methods, including a fuller appreciation of the actual structural interactions between discrete bolts and tendons and sprayed concrete can reasonably be expected to yield economy without additional hazard being introduced.

527. Large sums of money are inevitably and unavoidably spent on the excavation of hard rock to provide space (and overbreak) for thick concrete linings. Can we be sure that the improved hydraulic flow conditions offered by concrete tunnel linings really justify the extra expenditure? Larger diameter unlined tunnels could in some circumstances be technically acceptable at lower cost.

528. The methods of excavation and in particular the use of tunnel boring machines is a further area of potential improvement and economy. This aspect was considered carefully by the consultants and by potential contractors at the time of tendering but not pursued. Improvements in the performance of these machines

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and especially their availability could lead to more general use in future.

529. It is not the primary purpose of our Paper to reconsider the basic station specification, but obvious economies are offered by any possible concessions in the extremely rapid response times demanded of Dinorwig. In fact this rapid response facility will not need repetition for the foreseeable future on the CEGB system.

530. Contract strategy may well be a worthwhile and contentious area to re-examine for the future. The CEGB's experience generally favours small competitive contracts for major civil engineering works, but other views are held with equal fervour.

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