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Papers 9520, 9528, 9648 and 9569

The Victoria Project, Sri Lanka Project planning, design and construction of Victoria Dam; construction of Victoria Tunnel; Victoria Power-Station

P. A. A. Back, W. T. Mee, K. L. Cole, P. C. Neal, A. J. H. Speirs, P. J. Chapman and B. Creber*

■ **T. D. Pike,** *Overseas Development Administration*

As a civil engineer and member of the ICE, I take great pride in what has been achieved with this project, and I should like to make a contribution from the point of view of the funding agency involved, the Overseas Development Administration of the British Government.

2. The project formed part of an accelerated programme that was decided on by a newly elected government, for whose political credibility it was important that it should show its ability to deliver the goods. The British Government was anxious to support it in those endeavours; therefore, we were given the go-ahead to investigate this project as part of the Mahaweli accelerated programme.

3. As a result, we found ourselves under extreme political pressure and up against time constraints. Our first intention was to investigate this project as an irrigation project with power benefits. That is how it was first presented to us. We had to put together a multidisciplinary team to do that initial rapid investigation. There were a number of 'firsts' for the ODA throughout the programme. For example, we managed to select three firms of specialists independently and then persuade them to work together as a group, enabling the best possible advice to be obtained on all topics. After the initial six-month period, when the first report came in, it was evident that the main justification of the project from the economic point of view was as a power project, so from then on the agricultural benefits tended to fade into the background and we concentrated on the potential power benefits.

4. There existed a considerable shortfall in supply against suppressed demand within Sri Lanka at that time, so there was clearly great potential for the project. We therefore started a crash programme of investigation, and much of what we did was to an extent pushing against the frontiers of knowledge as far as the ODA was concerned. We had never taken on such a large project on a bilateral basis. The government of Sri Lanka and the government of the UK alone funded the project. That in itself constrained us in the way we had to operate.

5. Our first problem was a rapid site investigation, and for the first time ever we had to air-freight our site investigation equipment. This shows the degree of urgency, and such pressure was reflected throughout the design, tendering and construction processes.

6. When the results of the site investigation started to come in, it was evident that the upstream site was of such good quality that an arch dam would be appropriate. That, in turn, affected the economics, and pushed towards the high dam solution. I remember being involved with my economist colleagues in reviewing the appraisals that were coming in at that time from the consultants, and persuading ODA, and subsequently the government of Sri Lanka, to go for the high dam alternative. That subsequently presented us with some problems environmentally. We then had to put together a project that was tied to British procurement, which was rather difficult to do with such a complex project as this. We were limited in the number of manufacturers and contractors who had the capability to take on this sort of work, so we had to be quite careful in the way that we carried out our preselection procedures and in the way that tenders were put together. We had a great deal of difficulty in trying to frame clauses in line with the 'buy British' policy, which gave rise to considerable difficulty for the contractors subsequently. It was a difficult constraint under which to work, particularly for such specialized equipment as the Blondins and the tunnelling equipment. We had to submit waiver mechanisms in order to obtain the permission from the Department of Trade and Industry to 'buy foreign' where required.

7. At that time, the port of Colombo had a fairly low limit for the tonnage of ships that it could accept. Since the British companies seemed interested in supplying only in bulk in ships of their own, which were too large for the port, they declined to bid for the project. In this great British flagship project for a concrete arch dam, the first thing for which we had to

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* *Deceased.*

ask the DTI was a waiver for cement! We overcame that embarrassment subsequently and the mechanism worked reasonably well.

8. One major contribution to the project coming in largely on time and at cost was the bonus system which was built into key milestone dates in the contract. That again was a 'first' for the ODA. However, the ODA's senior management accepted that sort of incentive, which was rather unusual for our administration. And, after the bonus system had been accepted, its worth was proved, as I am sure it was responsible for the way in which this project came ultimately to be one of the most successfully completed programmes that we have had.

9. Soon after the project was completed, there was considerable press coverage in the UK and in Sri Lanka about the displacement of people from the reservoir area, a major environmental concern. This, of course, had been foreseen, and I remember, as part of a mission to Sri Lanka, discussing the problem with the government to explain the implications of going for the high dam. It was agreed at that time that they would accept the penalties involved and would deal with them. Subsequent events showed that the method of dealing with it could have been improved. I suspect that a number of new immigrants had entered the reservoir area, so that the number of people who received compensation exceeded the number who had lived there when we originally moved in. This should have been foreseen.

10. The fact of the matter is that this project has gained such a bad reputation environmentally within my own organization that it is now used in our environmental awareness seminars as a warning of the problems that can be encountered if care is not taken in the way environmental impact assessments are carried out. This has given the project a reputation that is undeserved, and it requires a great deal of hard work by the engineers concerned to persuade people that this is, in fact, an extremely good project that is a credit not only to the UK but to all those who took part in its construction. It has led to a number of other projects for the companies and people concerned. To that extent I consider it to be a major success of British engineering, and I am very pleased to have been associated with it.

W. J. Carlyle, *Binnie & Partners*

The Victoria Dam comes very close to what I would regard as the ultimate elegant dam. It presented a number of difficult problems to which elegant solutions have been found, and it has done a great deal to keep the British flag flying in dam design and construction.

12. The successful completion of the job, with Gibbs as designer and project manager, and a thoroughly experienced and well resourced contractor, was achieved by the planned target date despite some formidable difficulties. This teamwork was the foundation for the highly successful joint venture for the Samanala Wewa Dam which followed, but in a different form, where the contractors put the finance up, as I understand it, and invited Gibbs to be their design consultants.

13. In the planning studies, the catchment development appears not to be optimized because of the restraints on maximum water level in the Victoria reservoir. Nevertheless, the full Victoria Dam and the 5·7 km power tunnel for 210 MW of power and 686 GWh/a of firm energy, seems to be rather expensive, and some details of the economics would be useful. A dam of that size would normally have a significant flood attenuation benefit. This is not available at Victoria because of the upstream constraints. For example, at the Mudiq Dam with exactly the same design flood characteristics as Victoria, but having no constraints on flooding upstream, it was possible to route the whole of the inflow through the reservoir with a 10 m rise, with no fixed gates at all.

14. While counterweighted float-operated crest gates are not new, the Gibb method has brought a new level of non-electronic security. It appears that the step opening the gates is planned for a design storm so that the opening takes place over a mere 640 mm of reservoir rise. Is it a fact that there is no seasonal rule curve within the top 9 m of the reservoir and that there is no provision for operator override in anticipation of any unusual storm in the catchment upstream? Is it possible to persuade the owner not to interfere?

15. The difficulty has to some degree been transferred to the power-operated closure programme. A relatively small flood will give full gate opening. How long does it take to close? How much water from the retention band in the reservoir would be lost during such a closure?

16. With regard to the design for the maximum flood, it would be most helpful to have the rainfill depth duration and area relationships on the basis of which the probable maximum precipitation was determined.

17. Staying with the spillway and the jet plunge pool design, energy dissipation in a comparatively shallow pool is notoriously difficult. In theory, a jet falling through 100 m will attain velocities in excess of 40 m/s. Even the improved crest splitters did not spread the model jet over more than 10 m. It is surprising that the pressures measured were of the order of only 5 m in excess of static head, suggesting that the velocity head was not destroyed by the impact. Of course, fluctuating pressures are difficult to measure. What is the prototype experience?

18. The Paper presents a highly satisfac-

tory picture with modest tensile forces in the basin of the arch. Displacements have followed the predicted loading behaviour. In the Paper the deflexions due to thermal effects are not differentiated. I am reminded of a dam which is unique in that it spends much of its life almost completely empty and subject to very high thermal stresses, with a range of 20°C of the mean ambient summer to winter temperature. The thermal deflexions of the arch are 9 mm in the upstream direction owing to the heating up in the summer, and the deflexion due to a modest rise in reservoir level was of a smaller order than that due to the thermal movement.

19. In the cooling of the arch prior to grouting, it was cooled to a constant 22°C, and it is difficult to achieve this with a high thermal gradient in the arch. In some dams which are exposed to sunshine on one side and not on the other there is up to 10° thermal gradient from one side to the other. Some people attend to this by trying to set up the cooling with a gradient, but it may be that this is totally unnecessary. I imagine that the Authors had the right solution.

20. In Fig. 12 of Paper 9528, a deep gallery is shown below the arch. What was the purpose of this gallery, where was it located and how was it being de-watered, etc.?

D. S. Lawrenson, *Halcrow*
I was the Tunnel Manager for the Joint Venture of Balfour Beatty Construction Ltd and Edmund Nuttall Ltd. Edmund Nuttall Ltd first devised the method of continuous concrete lining in tunnels for the 12 mile long tunnel section of the Haweswater Aqueduct from Accrington to Bury, in the early 1950s. In those days, they achieved up to 1500 ft in a week, with 240 ft of shuttering. Over the years, many tunnels have been lined using this method, but I have often encountered a marked reluctance on the part of certain engineers to allow this process because of one major drawback in the amount of shrinkage that it incurs. Sometimes shrinkage cracks of 6 mm or more can arise, but these generally become filled in during the normal cavity grouting operation.

22. I should like to know the extent of the cracking that eventually appeared in the completed lining prior to flooding? Could the authors confirm that they would be happy to use this type of lining again in the future?

23. The decision to replace the trouble-prone Broyt loaders with Cat 980s was a case of putting money in to chase an escalating loss. In fact, the matter had become so urgent that two jumbo-jets were chartered: two Cat 980s were put in one, and one in the other, together with four additional compressors to boost up the inadequate air supply installed originally for both the tunnel and the dam sections.

W. P. Wright, *Kvaerner Boving Ltd*
Could the Authors say something about the operating experience of the reservoir? We have heard a great deal about the high dam alternative and the ability of the radial gates to hold the reservoir at a high level and yet to be able to discharge the rain as it came. As far as I am aware, the reservoir has not always been kept at that high a level. Perhaps the demand for power has exceeded the rain?

J. L. Beaver, *Halcrow*
Paper 9648 describes the findings of the site investigations in the area of the surge shaft. Serious geological problems were encountered in this area during the downstream drive, leading to the rock fall, the eventual abandonment of the tunnel and the subsequent realignment and the relocation of the surge chamber, which was expensive and very complicated. It is easy to be wise after the event but it would be interesting to have the Authors' comments on how such events might be avoided in the future, and whether any additional investigations were ordered in view of the poor conditions encountered during the main investigations in the period from November 1978 to February 1979.

K. L. Ariyananda, *Consulting Engineer*
As a member of the UNDP/FAO/Irrigation Department team who originally investigated the Victoria power project, I wish to point out that our studies indicated that raising the full supply level of the reservoir above elevation 428 m was not economical. Our computations were based on a simulation of operation of Victoria power-station integrated with other hydro/thermal power-stations in the CEB system. It should be borne in mind that Victoria reservoir inundated some of the most fertile land in Sri Lanka.

B. S. Piper, *Institute of Hydrology*
As a member of the Institute of Hydrology team that carried out the hydrological analysis, I should like to respond to the question by Mr Carlyle concerning the derivation of the spillway design flood.

28. The probable maximum flood (PMF) was used for the design of the spillway. The PMF was calculated by convolution of a unit hydrograph (derived from observed events) with the probable maximum precipitation (PMP). There is a good network of rain-gauges over most of the catchment, with records dating back to the early 1900s. We are indebted to Mr P. Sumenesekere, of the Sri Lanka Meteorological Department, who had carried out an analysis of all the intense storms in the upper Mahaweli catchment that had occurred between 1911 and 1971. From his analysis of 190 events, it was apparent that the maximum daily rain-

falls at many of the rain-gauges in the upper catchment had occurred during the storm of August 1947. A simplified moisture maximization technique was used to estimate the catchment PMP using the data from this storm, which was the largest ever recorded.

P. J. Hughes, *Edmund Nuttall*

I was involved in the drilling and grouting on the Victoria Dam and Tunnel. My point, however, is a historical one, and is based on the great author, Parker. A hundred years ago in Sri Lanka (Ceylon, as it then was) more than 1 m of rain fell in certain areas in one period of 24 hours. This over-topped every existing dam in the country and all were totally destroyed in that terrible storm.

30. Has there been any problem with silting up since impounding? Is there now in existence a national grid to distribute the power throughout Sri Lanka instead of just to Columbo? Have the controversial resettlement schemes proved successful in the long run?

M. J. Kenn, *Independent Consultant (formerly Head of Hydraulics, Imperial College, London)*

With reference to Paper 9520, could Dr Back comment on the effectiveness of the rather novel anti-vortex beams situated over the tunnel intakes? I am surprised that they do not extend beyond the width of the tunnels.

J. G. Eldridge, *Fellow*

I suggest that a table showing the quantities of the principal items of work (such as excavation, concrete, steel, etc.) involved in each main element of the project, together with the local, foreign and equivalent total cost of each main element, would be a valuable addition to these Papers, which provide such a wealth of detailed information about the design and construction of the Victoria Project.

33. Perhaps the Authors would also indicate the reasons for adopting the 92 m deep vertical gate shaft instead of a sloping intake gate, for which the topography at the tunnel intake shown in Fig. 10 of Paper 9648 appears well suited and which, at first sight, would seem to be significantly less expensive.

34. On one further technical matter, an outline of the considerations which led to a 3 m thickness for the spillway apron (the integrity of which is so vitally important, as explained in §§ 41–50 of Paper 9520) would be of great interest, as would a cross-section showing the type and disposition of the waterbars, anchors and drains described in § 49 of the paper. Perhaps the Authors could also say whether measurements made during routine inspections of the apron have so far shown any loss of concrete from the top of the slab, or other defects in the apron.

35. Turning to more general topics, an important factor to be assessed when planning the construction of a large and important project such as Victoria is the performance of local labour, for on this depends the number of relatively expensive expatriates that will have to be employed. The Papers show that the number of Sri Lankan nationals and the number of expatriates employed by the contractor at the peak of the work were as given in Table 1. At the peak of the work, therefore, there were about 20 nationals to every expatriate on the contractor's staff.

36. This rate is surprisingly close to the corresponding ratio for the Mangla Project, built in Pakistan during the 1960s. At the peak of construction, the contractor at Mangla employed 500 expatriates and 13 000 Pakistanis, a ratio of 1 to 26. Doubtless the differences between the nature of the two projects—Victoria Dam was a sophisticated concrete arch dam whereas the dams at Mangla were earthfill—will have influenced the two ratios.

37. The contractor at Mangla devoted much attention to the training of his local workers and also to their feeding, for he realized that to work as hard as he needed them to work, they would need a greater calorie intake than they usually enjoyed. He therefore insisted that all the workforce should eat at canteens where they would have sufficiently nutritious meals, instead of fending for themselves, as had been the custom at earlier construction projects in Pakistan. It would be interesting to learn how the contractor at the Victoria Project approached the training and health care of his local workforce.

38. The performance and dedication of expatriate staff is also crucially important on a lengthy project in a developing country, for a high turnover of expatriate staff can be exceedingly expensive. Perhaps the Authors could provide information about the housing and recreational facilities provided for expatriates and their families and also of the schooling available for children, for education problems can often prevent admirably suitable staff from accepting an overseas appointment.

Table 1. Total numbers of Sri Lankan nationals and expatriates employed by contractor at peak of work

	Sri Lankan nationals	Expatriates
Dam and tunnel	4400	225
Power Station	643	25
Total	5043	250

39. The numbers of expatriate and Sri Lankan staff employed by the Engineer at the Victoria Project would also be of great interest.

40. Finally, I offer my compliments to the Authors in having succeeded so admirably in the purpose expressed in the synopsis of Paper 9648, namely 'Plant and construction methods and problems are expressed in sufficient detail to be of practical benefit in the planning of future projects'.

G. G. T. Masterton, *Babtie Shaw & Morton*

With reference to Paper 9648, the Authors' account of the concrete tunnel lining operation contained many parallels to the lining of the Kielder tunnels.[1–3] Although smaller in cross-section, haul distances for concrete were greater (up to 8 km) and truck mixers were expressly excluded from operating inside the tunnel. As a result, a rail system with passing places had to be persevered with, and derailments, which occurred from time to time, were minimized by regular track maintenance. In the early stages, concrete delivery times of 4 hours were not uncommon, although this soon reduced to 1–2 hours as progress improved. The continuous advancing slope method was also used, and a best weekly lining advance of 460 m was achieved. With a concrete volume per linear metre some 5–6 times that at Kielder, the best weekly advance of 272 m in the Victoria Tunnel (§ 54 of Paper 9648) is quite respectable.

42. In § 57 of Paper 9648, the Authors refer to the occurrence of long inclined flaws along the advancing slope line. This was identified at Kielder and is a result of air being trapped as the concrete flows down the advancing slope without effective vibration. The solution lay in greater attentiveness to the synchronization of formwork mounted vibrators and the advancing slope. If vibrators (particularly in the invert) are mounted too far ahead of the natural slope of the concrete there is a tendency for concrete to be drawn along the invert too quickly. If, when the next load of concrete is discharged through the slick pipe, the vibrators are still too far ahead in the invert, crescent-shaped lenses of honeycombed concrete will be left. At Kielder, poker vibrators were largely dispensed with after the technique of ensuring that form-mounted vibrators were correctly positioned, was fully mastered.

43. In most in-situ tunnel linings, non-structural circumferential cracks develop[4,5] (although if humidity is high, hydration temperatures low and/or partial cement replacement is used,[6] they may be very infrequent). Were any cracks identified in the lining of the Victoria Tunnel and, if so, were they successfully bridged by the surface coating?

C. Pollard, *Engineering Manager Tunnel Construction, TML (formerly Senior Tunnel Agent, MBZ, Dinorwig Power Station Project)*

In response to the concern about explosive consumption expressed in Paper 9648, I would agree that this is on the high side. At Dinorwig, it took a considerable time and effort to convince the shift crews that the fully charged burn cut was unnecessary. Two 'spiral' cuts were used (see Fig. 1) and the cut holes were charged with one primer (80% gelatine), approximately 2·0 linear m of low density explosive (Polar Rockite) and Cordtex to ensure good initiation. This avoided 'freezing' and sympathetic detonation and reduced vibrations, thereby improving the resultant profile. Adoption of this type of cut would reduce the weight of explosive per round in the Victoria Tunnel by over 20 kg.

45. The hole diameters at Dinorwig were generally 32 mm with 28 mm diameter explosive. The wider difference—45–32 mm—at Victoria would have required more explosive and a greater tamping effort to achieve the same length of fully charged hole. This would amount to more than four sticks of explosive per hole. The adoption of a smaller hole at Victoria could have given a further saving of over 80 kg per faced charge.

46. Further reductions could be made by using only one primer in each smooth blast hole and a reduction in the charge of the ring of holes adjacent to the smooth blast holes.

47. These factors would reduce the specific charge to 1·3 kg/m^3 and bring it close to the Dinorwig figures for a similar face area.

48. While Dinorwig was largely excavated in hard slate, the 'spiral' cuts proved successful in the extremely hard gritstones and dolerites. However, great care had to be taken to ensure that all holes in the cut were collared accurately and then drilled parallel to each

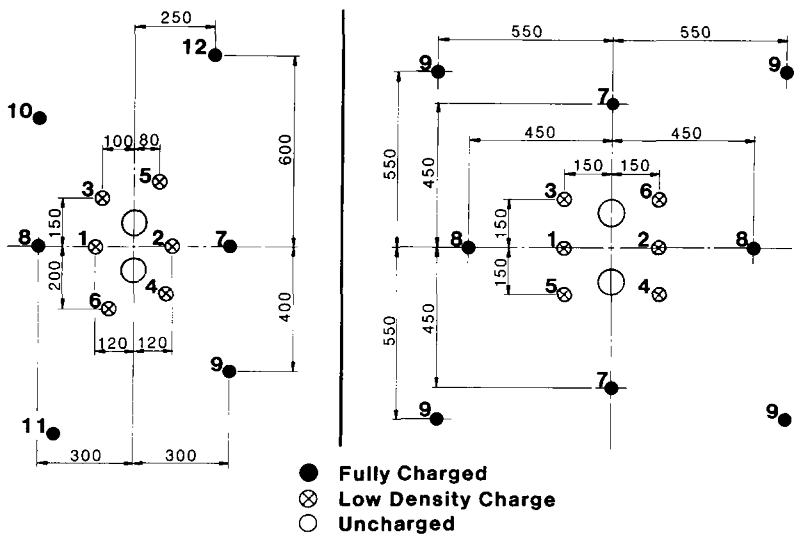


Fig. 1. Cut configurations used in Dinorwig Power Station excavation

other. In all 'failed' rounds, the investigation showed drilling to be at fault.

T. H. Douglas, T. J. M. Paterson and I. D. Isaac, *Mott MacDonald Power*

Paper 9520

Was the correct decision taken to avoid the limestone along the route of the power tunnel (§ 15)?

50. This project involved high risks. A major dam and tunnel had to be constructed to a very tight programme at a site where the site investigation does not appear to have been exhaustive and by contractors who had no recent experience of arch dam construction and limited recent experience of hard rock tunnelling. The choice of a conventional FIDIC contract (§ 73), with bonuses for staged completion against target dates but without penalties for late completion, gave the contractor little incentive to perform once he had missed the opportunity to win the bonuses, whatever the reason for that failure. It appears that some form of target cost contract could have provided a more effective and equitable contract. Such a form of contract might also have provided a framework within which agreement on contractual issues could be reached more readily. With the benefit of hindsight, what form of contract would the Authors recommend for a similar project?

51. Considering the likely lack of incentive for the Contractor to sustain efforts to achieve the work once he overruns the intermediate key dates for bonuses, why were liquidated damages introduced to those particular key dates?

Paper 9528

52. How was simultaneous construction of river bed works and clearance of the abutment foundations upslope carried out in safety (§ 47)?

53. Considering the extent of deep weathering discovered on the left bank (§ 20) and other difficulties encountered elsewhere, what views do the Authors have with regard to the extent of early site investigations carried out for this project?

54. Could the Authors expand on the extent of early exploratory works undertaken (§ 38)? Was consideration given to the possibility of introducing a pre-contract exploratory tunnel drive? Reference is made to introducing additional site investigations as part of the Contract. Why were such investigations not carried out pre-contract and what contractual significance did they have with respect to 'unforeseen conditions' under the terms of this Contract?

55. The Paper refers to 'blast-induced open joints' (§ 79). What particular steps were taken when designing the blasting in this area to minimize and reduce potential blast damage?

Paper 9648

56. Could the Authors provide some information on the design of the power tunnel, in particular the hydraulic parameters of the tunnel and the design parameters for tunnel and shaft rock support and linings?

57. The expertise of a Swiss contractor was utilized on the dam contract. In view of the limited recent experience of UK contractors in driving hard rock tunnels, would such overseas expertise not have proved valuable on the tunnel contract? The contribution of an overseas contractor, Conrad Zschokke, to the success of the Dinorwig pumped storage project, one of the few recent hard rock tunnelling projects in the UK, was very considerable.

58. All concerned with the Victoria Tunnel are to be congratulated on the reduction in the overrun on completion from 15 months to about three months, and, in particular, on achieving high rates of progress with concreting and grouting the tunnel. Could the Authors indicate on a progress chart the rates of progress of the works at the different faces?

59. Victoria Tunnel appears to have had a pattern of progress typical of many tunnel projects, with initial slow progress and major problems, followed by much faster progress towards the end of the project. Another example is the drivage of the Channel Tunnel. Should tunnel planners not take allowance of this pattern in their programming?

60. The contractual arrangements for the construction of a tunnel are of considerable interest (§§ 2 and 25). Could the Authors put figures on the tender price, and final cost, the bonuses offered under the original contract, and the enhanced bonus system and key dates introduced after the tunnel was re-routed? What penalties or liquidated damages were included in the contract and how were these modified at the time of re-routeing?

61. With the benefit of hindsight, should further site investigation have been carried out (§ 4)? Was there any surface expression of the fault encountered at the major rockfall?

62. The bedding of the rock over the initial 3000 m appears to be near horizontal (§ 6). How well annealed were the joints and what difficulties did such geometry of jointing give with respect to rock stability in the tunnel crown and shoulders?

63. Could the Authors indicate the rates of progress of the works at the different faces on a progress chart given in Fig. 2?

64. With reference to Fig. 4, what standard of cleaning was specified for the tunnel invert before concreting?

65. What lengths of probe hole were drilled in the tunnel on a regular basis and how long did this take (§ 9)?

66. While it is appreciated that 'Quarrex' proved to be too powerful an explosive for the

perimeter holes (§ 10), was consideration given to the introduction of intermediate uncharged holes to reduce the specific charge and hence reduce the risk of potential blast damage? What average pulls were achieved with the 4 m round length (§ 11)?

67. The system of the Engineer's agreeing to and measuring the temporary rock support proposed by the Contractor (§ 13) is a simple one but could be prone to misuse. Why was the system adopted in preference to a system of joint nomination of temporary rock support? Did the temporary rock support play no role in the design of the final tunnel support? If so, was this not potentially wasteful?

68. With hindsight, would the Authors have preferred to use a smaller mesh than A393 (§§ 13 and 69) to ensure effective contouring of the mesh reinforcement and reduce the influence of the mesh in creating voids behind the shotcrete?

69. In § 14 there appears to be some confusion between relaxation of the rock and satisfactory perimeter blasting. Relaxation of the rock could surely occur irrespective of the quality of the perimeter blasting.

70. About seven months were spent in tunnelling the rockfall area for the tunnel (§§ 20–24). Could the Authors provide a detailed breakdown of the activities and chainages in the rockfall area during this period and further information on the investigation and geology of the rockfall zone? Was consideration given to a pilot drive through this zone after completion of pre-grouting? What methods of excavation and support were used in the rockfall zone?

71. Do the Authors have any explanation for the ineffectiveness of the grouting in the major rockfall zone? Was the grout washed away or did it travel far from the rockfall zone? What grout mixes and procedures were used in the rockfall zone?

72. The major rockfall has similarities with the rockfall in the diversion tunnel of the Dinorwig pumped storage project in North Wales, but tunnel advance was resumed there with a much shorter delay.

73. The realignment of the tunnel and resiting of the surge chamber referred to in § 24 appeared to arise from unforseen ground conditions referred to in § 15 of Paper 9520. What extent of site investigation would the Authors propose in order to avoid the delays which occur and, with hindsight, would the earlier investigations have been economically justified?

74. It has been reported in the Paper (§ 29) and in the technical press that some of the plant selected did not come up to expectation, in particular the Broyts and four-boomed jumbo. With the benefit of hindsight, what plant would now be selected for an identical project? At what intervals were the passing bays constructed? Was an enlarged passing bay required to turn DJB dumptrucks?

75. With reference to Fig. 8, is there any correlation between rock type and quality and the usage of explosive? Could the Authors indicate the typical rock classifications for the different rock types and how much these varied?

76. In circumstances where a high specific charge was necessary to effect suitable pulls (§ 31), was consideration given to introducing uncharged holes with the likely effects of reducing the specific charge and blast damage?

77. Was the intake structure designed on the basis of hydraulic model testing (§ 32)?

78. In the raise boring of the surge and gateshafts, what was the accuracy of the raise bore pilot hole and how was the weak limestone layer stabilized in gateshift 1 (§ 36)? Did this layer and the solution cavities in the surge shaft cause any problems during enlargement of the shafts (§ 65)?

79. What concrete trials were undertaken to determine the likely thermal shrinkage and temperature effects from large concrete pours (§ 55)?

80. Has the problem of long inclined flaws (§ 57) in the concrete lining placed behind the telescopic shutter been reported elsewhere? How were these flaws made good? What method was used for concreting the steel-lined section of the tunnel?

81. Could the Authors indicate the spectrum of grouting pressures and depths of stages during the consolidation grouting (§ 75)?

82. Do the Authors consider the crown voids (§ 77) between the tunnel lining and the rock can be avoided with the use of a telescopic shutter and slick line concrete placement? By contrast, at Dinorwig, where single shutters and pump placement of concrete were used, such crown voids were almost entirely absent, probably because some overpressure could be applied to the concrete by the pump to fill crown voids.

83. The application of a surface coating to the concrete lining of a hydraulic conduct is an interesting development (§§ 82 and 83). What length of tunnel was coated with each of the surface coating materials and why? Has it been possible to assess the effectiveness and durability of the surface coatings to date?

84. Figures 2 and 9 show the anti-vortex structure adopted at Victoria. Could the Authors describe in more detail the options considered before the somewhat unusual arrangement was determined, and also the relationship between the various flow vectors at different stages of reservoir level.

Paper 9569

85. Could the Author describe briefly the methods adopted for monitoring behaviour of the penstocks (§ 17)?

86. With the realization of potential pressures which might cause flotation (§ 19), was any consideration given to provision of a cut-off wall to minimize the effects?

87. Could the Authors describe the techniques and type of grout used to grout the difficult areas under the stay rings (§ 34)?

88. Pre-splitting open-jointed rock (§ 60) can, in certain instances, cause damage by causing deeper dilated joints. Do the Authors consider that such a concentration of explosive charge might have influenced the opening of the joints referred to? Should this have been the case, would smooth blasting and/or line drilling techniques have alleviated such damage?

S. Xavier, *Department of Transport (formerly PSA Projects/Project Management Office)**

Would the Authors clarify how the 'probable maximum flood' (PMF) of 9510 m^3/s, given in Table 1 of Paper 9520, was calculated?

90. It is interesting to note that aggregates larger than the sizes normally used in building construction had been specified for the dam construction. Sizes such as 150 mm and 75 mm would cause difficulties in carrying out the simple site tests, such as the cone test, to check the consistency of concrete before placing it in the formwork for the dam blocks. The conventional 150 mm cube used to check the 28 day strength would be also a problem! In the civil engineering technology associated with dam construction, how are these problems overcome?

91. Would the Authors comment on whether or not any precautionary testing was carried out on aggregates and cement to keep track of alkali–silica reactions in concrete works.

92. Was any attempt made to use locally produced cement or cement from India nearby in lieu of the long-haul cement from UK sources?

93. Regarding the provisions made for future extensions of the power-house to accommodate another three 70 MW sets, would the Authors clarify if these will merely repeat the 5·15 m dia. steel-lined tunnel and the 3 m dia. trifurcation system, all from the parent 6·2 m dia. concrete-lined tunnel, or are the extension works envisaged much more involved and complex? What would the order of costs be for this second power-house in 1991 prices?

94. The Victoria Project would appear to complement the Polgolla diversion works already completed some time ago. Are the Authors aware of any interesting papers published on the Polgolla scheme?

95. With regard to the Master Plan of 1968 referred to in § 2 of Paper 9520, are the Authors aware of any provisions to extend the North Central Province canal deeper into the Northern Province, as far as the relatively dry Jaffna Peninsula? Are there provisions to supply the dry zone districts such as Mannar and Mullaitivu by lateral canals? All this begs the question of whether or not there is enough water upstream after allowing for power generation. If this is not the case, do the Authors think that the Mahaweli Ganga could meet this shortfall, perhaps at a location nearer to the origin of the Left Bank 2 canal? One solution would appear to be to interconnect the canals in this trans-basin zone. If this is feasible, the water supply required for power generation may not be affected after all. How do the Authors react to this proposal, or do they see other possible solutions to the water supply needs of the Northern Province, including the Jaffna peninsula?

96. An interesting project of this type would have attracted many keen local minds of all ages. How many Sri Lankan civil engineers, mechanical engineers and electrical engineers were engaged by this project, both at graduate training level and at senior level? Were the doors opened to local Sixth Form College students to visit the works and to observe the progress from vantage points of safety?

97. Finally, the Authors are to be congratulated on these high quality papers, which will probably serve as a fine work of reference for keen engineers of various disciplines for a long time to come. To enhance the interest further, the Authors could present another paper devoted exclusively to the art and science of project management which went into the condensing of a 30 year programme to a seven-year 'crash-programme' success project.

Dr Back, Mr Cole, Mr Neal, Mr Speirs, Mr Chapman and Mr Creber

Mr Carlyle queries the restraint on maximum water level. We were restrained because of the need not to flood the Polgolla diversion upstream. The restraint on water levels was a very real one, but the economics were such that the internal rate of return climbed steeply as we increased even by 2 m or 3 m, and made a substantial difference to the net benefit. Mr Pike has already drawn attention to the fact that the project began essentially as an irrigation project, but that the all-importance of power rapidly emerged. Although it seems modest by modern day standards, 210 MW and 686 Gwh/a, in the context of the system into which it had to be placed, represented a 40% addition to the system, and an internal rate of return of about 10·6%. The irrigation benefit added only 0·3% to that. This was the differ-

* The views expressed are solely the contributor's own and do not by any means imply or reflect the views of the DTp.

ence between the two. In fact, we were faced with very little choice but to maximize the power benefits.

99. Mr Carlyle says that there is no flood attenuation benefit and that is true: there is not. The flood attention benefit was not a substantial one because it was known that downstream of Victoria there would eventually be the Randenigala Dam, which was not built at the time, but which has now been constructed. Randenigala provides the flood attenuation for the lower reaches of the river.

100. Mr Carlyle refers to the fact that the step opening of the gates was planned so that the opening should take place over a rise of 640 mm. That is correct. There is no seasonal rule; there was no need for one. We have a provision to override the gate if we have to, but this has never had to be implemented. We anticipate that it is never likely to be used other than for test purposes. So far, it has always responded. It is true that the gate does not operate under full reservoir head all the time; it cannot because it is necessary to use the stored water in the dry season to continue to provide power. What is very important is that we store as much as we can when the water is there, hence the height of the dam. We have had no problems with anyone interfering with the operation. Closure of the gates is a very rapid process; they can operate in a few minutes.

101. A point has been raised concerning spillway energy dissipation. This is extremely successful. The method does not reduce the velocity, except in so far as it scatters the jet. The dissipaters break up the jet and turn it into a shower as opposed to a solid jet. This is a most important benefit. We used transducers placed on the floor of the model to study this effect, and the results were dramatic. Our experience is that the prototype is even better because the aeration effect is greater. In the model (1–40) used to study the problem, the results did not demonstrate the same benefit as with the prototype. However, we were confident that if the model gave us a good result, the prototype would be even better.

102. With regard to the design of the dam, the stress behaviour, cooling and so on, we looked at the worst stress situation we could conceive of, with many different possibilities of stress combinations to give us the worst envelope of stress under any configuration. We looked at a nest of possible configurations of water level and temperature in order to arrive at the most adverse conditions.

103. The drainage gallery referred to is not a deep gallery. We had drainage galleries under the flanks which drained by gravity. It is only the lowest gallery in the arch itself that has to be pumped, as this below the river level. Apart from that, there are no other drainage tunnels.

104. With regard to the tunnel lining, we used the continuous shutter and did not have any particular shrinkage or cracking problems. It was eminently successful. There were indeed, some discrete cracks but with the grouting programme which followed, the cracks could be rendered harmless and not a problem, but it does require a very satisfactory and thorough grouting. I certainly subscribe to the benefits of continuous shuttering.

105. With regard to *Mr Wright*'s point about operating experience and high level operation (§ 24), it is true that the reservoir water is not at high level all the year; it is there only when the flood season has come and the water is stored. Our operating experience has been 100% successful so far, with no reports ever of malfunctioning of the automatic process. We can override if we have to, but so far that has not proved necessary.

106. In response to *Mr Beaver*'s question on exploratory investigations for the tunnel (§ 25), it is inevitable that when one is dealing with a 6 km tunnel, the investigations have to be a sample only. It was extermely difficult to obtain more than about 15 or 16 holes along the whole line of the tunnel. Although we put down a hole in the area where we sited the initial surge shaft and it showed that conditions were satisfactory, we were not able to sample all the way along the tunnel. As a result, we encountered a zone which had not been sampled and this led to great difficulties which have been described. In the circumstances, we ultimately decided that we should make a change. In making that change, we have to introduce a throttle at the base of the surge shaft, otherwise the water would have emerged from the top of the shaft. The throttle enabled us to contain the surge within the available headroom that we then had. To relocate the surge shaft we had to go further down the hill where the available headroom was less. The surge chamber stands 10 m out of the ground which we deemed to be the limit of acceptable visual impact.

107. In reply to *Mr Ariyananda* (§ 26), the subject of resettlement was studied in great detail. We did not carry out an independent assessment of the benefit of the land that was to be lost; we had to rely on figures that were given to us for the assessment. However, our economic analysis certainly took into account the loss of irrigation land, the displacement of people, and all the downside aspects attached to a raising of the water level. We looked at 430 m; we looked at 435 m; we looked at 438 m. We could not go higher than 438 m because that would have meant going above Polgolla, which would have been unacceptable. The analysis demonstrated a clearly rising benefit as we went higher, which was why we ended up where we did. As *Mr Pike* mentions, the decision was made for these reasons, but the issue

was looked at very carefully.

108. In reply to *Mr Hughes*'s question in § 30, there has as yet been no problem with silting up. We anticipate that it will take about 70 years before some 10% of the live storage will be depleted.

109. The national grid is an ongoing development. Indeed, the power that comes from Victoria is distributed very widely over the country, although one could not put a label on any 1 kW/h and say that it ended up in any particular place. However, the grid is extending and is now a 220 kV system, which is extending further to the south. I am not sure how far it has extended to the east and to the north. Certainly it is an ongoing and developing programme.

110. In reply to *Mr Kenn*, the anti-vortex beams are not particularly novel. We have used them in four or five schemes to date and they are very effective. They are effective because they are not solid and have gaps, so that at every water level there is always a lip of intrusion into the possible circulation of water. There has not been any evidence so far that a vortex problem has developed in front of these intakes. I cannot say that I have stood and watched them at every level, but we have never had any reports indicating that this is a problem.

111. We developed this design for the Kariba North power station in the early 1970s and have used it on a number of other projects around the world. It has always been highly successful.

112. The design of these anti-vortex beams has been driven by model studies which have demonstrated their effectiveness. Therefore, we have not found it necessary to extend them outside the width of the tunnel intake itself. Under certain configurations that may prove necessary, but bearing in mind the excavation profile within which the intake is placed, there is a damping effect from the side excavations which assists in preventing vortex action.

113. We believe that at Kariba the circulation of water at the intakes was propagated partly by the configuration of the intakes in relation to the dam. The solution of these anti-vortex beams was tested on the hydraulic model, and the exact positioning of the beams and the configuration were determined from the model. They have been very successful at Kariba.

114. The intake was in a rock channel and the vortex structure was actually supported on the side walls, so there would not have been any vortex from the side.

115. In reply to *Mr Lawrenson*, inclined (0–20° off vertical) circumferential cracks appeared at regular intervals (approximately 20 m) throughout the length of the lining. These tended to be inclined in the direction of the advancing concrete face. Many of the cracks were wide enough to pass grout during the consolidation grouting at 10 bar.

116. The nature of any continuous process makes it more difficult for the Engineer to impose strict control in this case than in the case of a discontinuous process. Once in motion, the momentum of the process carries everything forward. Nevertheless, because of the clear requirements of the specification for follow-up consolidation grouting, this was an acceptable situation, as defects would be made good.

117. In response to *Mr Eldridge*'s request (§ 32) for a table of quantities of the principal items of work, this is given in Table 2. It would not be possible to break down costs in the same way owing to the complex manner in which overheads, bonuses, etc., were allocated.

118. For the intake to the power tunnel, the vertical shaft to house the emergency and control gates was adopted to ensure that, in the closing mode, the chances of jamming due to debris or other reasons were minimized. It was not considered desirable in the long term for the control pistons which actuate the control gate to be mounted below top-water level.

119. The spillway apron design was very much dictated by the results of the hydraulic model tests which showed that there would be, over a small area, a significant peak to the dynamic impact forces impinging on the apron. The anchoring configuration was designed to resist any possible uplift forces generated should the underdrainage become blocked, or high dynamic pressures manage to penetrate to the underside of the apron. The keying of the apron slabs and the water bar arrangements were conventional—and very strict quality control was maintained during construction. A typical cross-section of the apron is shown in Fig. 2. Inspections carried out since commissioning have shown that the apron is performing satisfactorily.

120. With regard to the facilities provided for expatriate staff, a pleasant and attractive village was constructed, with a school, club

Table 2. Quantities of principal items of work

Structure	Item	Quantity
Dam	Excavation	997 000 m^3
	Concrete	678 000 m^3
	Reinforced steel	6 000 ton
Tunnel	Excavation	905 000 m^3
	Concrete	116 000 m^3
	Reinforced steel	2 300 ton
Power Station	Excavation	
	Concrete	
	Reinforced steel	
	Structural steel	300 ton

house, swimming pool, tennis and squash courts and guest quarters for visitors. These amenities were available only after the first year, before which life was certainly more difficult for staff and their families, and, consequently, turnover of staff was greater than was the case later on.

121. The peak expatriate and Sri Lankan staff employed by the Engineer were as follows: expatriates—40; Sri Lankan—85.

122. With reference to *Mr Masterton*'s question in § 43, we would refer him to our reply given in § 115. No specific attempt was made to control hydration temperatures, and cracking was anticipated. The competence and low permeability of the rock surrounding the tunnel made the cracking relatively unimportant. The cracks were bridged by the surface coating at the time of application; but with thermal movement, it is to be expected that these cracks will reform.

123. In reply to *Mr Pollard*'s question in § 45, the 45 mm hole was the smallest diameter, with an interchangeable button bit that could be drilled with the boom and drifter fitted to the jumbo. Holes of 38 mm dia. were drilled for the rockbolts by the jumbo rigs fitted with integral drill rods, but these were not favoured for the shot holes. Only the cut holes were fully tamped.

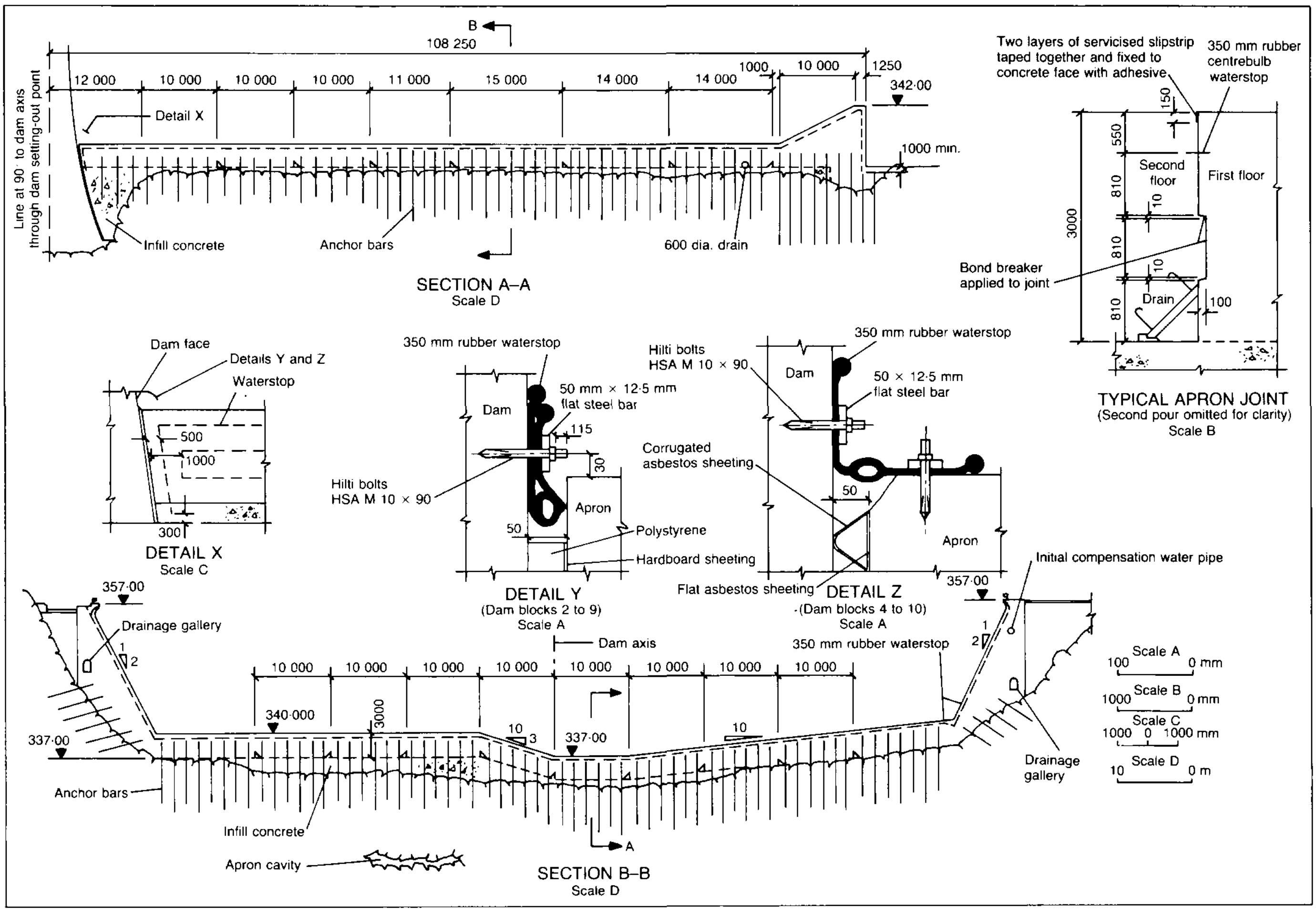
124. In reply to *Mr Douglas et al.*, the major tunnelling problems encountered were close to the interfaces between the gneiss and the limestone. Within the limestones over the limited lengths through which the excavations passed, the conditions were good. It is not possible to say whether or not they would have been as good elsewhere had the tunnels been located mainly within the limestone.

125. Bonus dates were not subject to extension of time—but the application of liquidated damages were, of course, so limited. The application of liquidated damages to intermediate programme dates was generally in relation to access dates for other parties to their work.

126. The reply to the question of how simultaneous construction of river bed works and clearance of the abutment foundations upslope could be carried out in safety is simply by the careful co-ordination of the construction activities.

127. In reply to the questions in §§ 53 and 54, it would clearly be very attractive to designers to spend a great deal of time and money on very detailed site investigations so as to avoid encountering the unknown. However, we have to accept that, at the best, we can normally sample only a very small fraction of the geology of a scheme such as the Victoria Project—with a long tunnel overlain by very

Fig. 2. Apron and training walls: sections and details



difficult terrain, where, in some instances, access proved possible only by helicopter or elephant. In the end, it is a trade-off between a very expensive investigation and a potential delay to the scheme while such investigations are in progress. At Victoria, it was required of us to have the project up and running within six years of our appointment, and this was accomplished. So urgent was the need to get the project started that we were obliged to arrange a negotiated contract for the exploratory works, and the drilling equipment was airfreighted out from the UK. As the construction work progressed, additional local geological investigations continued in order to guide the final design and construction processes.

128. In reply to the question in § 55, all the usual precautions were taken, including experimentation with patterns and weights of charges, to achieve minimum damage to the rock.

129. The answer to the question in § 56 would warrant a paper to itself; the question cannot, therefore, be addressed here.

130. Although no overseas expertise was continuously employed on the contract, the advice of individual specialists was sought on a number of occasions when the excavation progress was poor. This culminated in the deployment over a four-week period of a complete tunnel team from the Italian/Austrian Contractor Prader. The results were not significantly different from those achieved by BBN.

131. With regard to the question in § 58, a progress chart is given in Fig. 3.

132. In reply to the question in § 59, undoubtedly the pattern of progress identified is fairly common and the prudent planner recognizes this. Nevertheless, the shape of the curve of improvement may vary appreciably from project to project.

133. With reference to § 60, the details of tender price, final cost and bonuses are given in Table 3.

134. With regard to the question in § 61, there was no surface expression of the fault encountered which was surprising for such a significant feature. Lineation studies were carried out as part of early geological analyses. It is recognized that cored boreholes are not necessarily an effective means of establishing the lines of faults which may be vertical or subvertical. The determination of the location of these has to rely on aerial photography or seismic investigations. In general, the depth of weathering in tropical regions often renders even these methods of questionable value in disclosing such features. It would have been a matter of chance for further, more intense investigations to have revealed the fault.

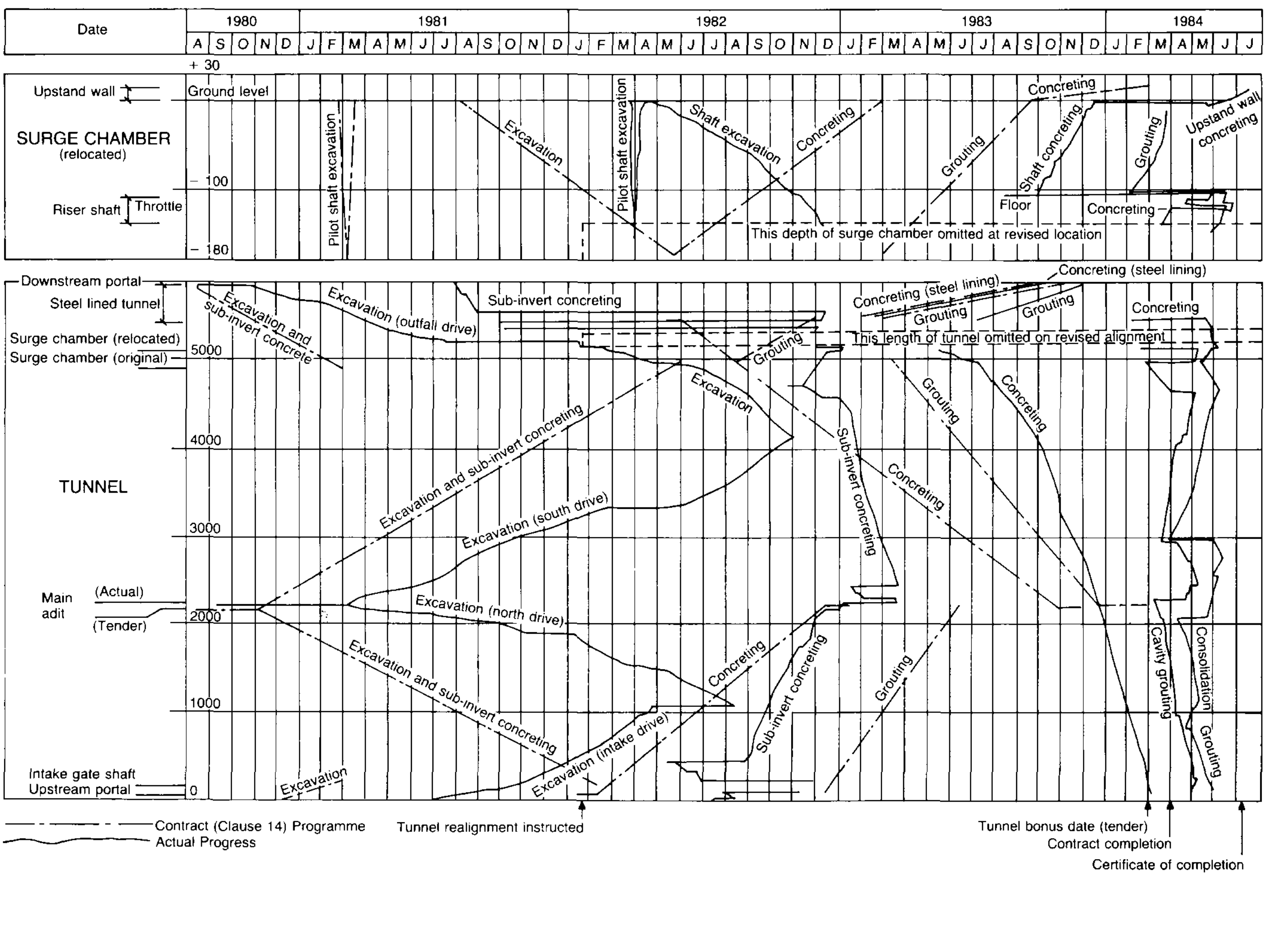


Fig. 3. Time–distance chart: surge chamber and tunnel

135. In reply to the question in § 62, although the lithology shown in Fig. 3 of Paper 9648 suggests distinct lithological boundaries; in practice, these were not clearly defined, and this whole length of tunnel was a mixture of metamorphosed rocks. The rock type shown is that which predominated in the zones concerned. The boundaries between different rock types were well annealed. Few problems of rock stability could be attributed to this situation.

136. The answer to the question in § 63 is also given in Fig. 3.

137. The tunnel invert was required to be cleaned to undisturbed material, i.e. generally sound rock. Compacted tunnel spoil was considered unacceptable for the pressure tunnel.

138. In reply to the question in § 65, once experience was gained, probe holes of 50–60 m length were generally drilled. This was done as an additional activity during the regular drilling cycles. The time for drilling the hole was around 90 min for one boom of the jumbo rig.

139. The introduction of intermediate uncharged holes was not adopted. The average pull (all round lengths) was approx 3·6 m.

140. With reference to the question in § 67, while the system of measurement of rock support could be prone to misuse, it is generally the case that contractors recognize the overriding benefit to themselves of maximum possible progress. This is not served by installation of unnecessary support. The system was adopted to ensure that the Contractor's responsibility for safety was undiluted. The temporary support played no role in the design of the final tunnel lining. This may be regarded as wasteful, but temporary support is often installed in adverse conditions and the quality falls short of the designer's requirements.

141. In reply to the question in § 68, even without the benefit of hindsight, the Authors would like to see a more suitable mesh available in the British Standard. A393 is a 200 mm × 200 mm mesh, with a 6 mm dia. wire. This mesh is considered too stiff to be ideal for shotcrete use. Unfortunately, the overriding demand in the British market is for structural uses other than shotcrete.

142. It is considered that a 200 mm × 200 mm grid is optimal for shotcrete use. A mesh size smaller than this can result in openings smaller than 100 mm × 100 mm in overlap areas. This can readily result in build-up of shotcrete on the wires. A wire diameter of 4 mm is considered heavy enough to suit the structural needs but small enough to permit contouring. No ideal mesh exists in BS 4483: 1963.

143. In reply to the question in § 69, some relaxation of the rock will inevitably occur. However, the degree of this relaxation can be limited if blast induced fracturing of the rock can be limited. It is considered that there is a very definite correlation between the degree of relaxation and the quality of the perimeter blasting.

144. In reply to the question in § 70, the probe hole encountered water on the 23 June 1981, at Ch. 5231. The face was advanced two rounds to maintain a minimum 10 m of rock to the apparent source of the water. Grouting was carried out from this face at Ch. 5224. Initially, two holes were drilled at opposite quarters of the face, and this pattern steadily increased with checkholes until the source of water was driven back. The length of the grouting holes was generally 20 m. By 16 July, the water had been pushed sufficiently far back to allow the face to be advanced by three rounds. This was done and the same pattern of grouting carried out from a face at Ch. 5216. Again, the water was apparently pushed back; and again, a limited advance was permitted. It was during the course of this advance that the face collapsed after the third and last round of the intended advance. No consideration was given to driving a pilot through this zone and, with hindsight, this might have been prudent. However, the plant and equipment available were quite unsuited to this type of exercise, and the mucking in particular would have been problematic.

145. Horizontal exploratory holes were drilled to the left and right of the tunnel line. These were 100 m long and drilled from approximately 30 m back from the face. The holes were inclined outwards from the tunnel line at 30°. These determined the orientation and the width of the fault zone. The material was a mixture of fault breccia and moderately to highly weathered rock within the crushed zone of the fault. The holes indicated that the zone was not less than 50 m wide. The fault behaved as an aquaclude, prohibiting the normal movement of groundwater from the high ground toward the Mahaweli River valley and generating the high pressures which were encountered when the acquaclude was penetrated.

146. Excavation and support methods in the fault zone are described in § 22. A pilot heating was advanced, with mucking carried out by means of an Eimco overloading shovel brought to site for the purpose. The pilot heading was taken forward approximately 30 m, and this was in the course of being widened to a full

Tender price	Rs 645 million
Final cost (the final cost cannot be precisely defined, as the dam and tunnel contracts were the subject of a joint final settlement)	Rs 838 million +396 million for escalation and additional works
Bonus payments paid	s 4 million
Liquidated damages	None were invoked

Table 3. Details of tender price, final cost and bonuses

crown heading when the decision was made to abandon this alignment.

147. With regard to the question in § 71, the ineffectiveness of the grouting in the rockfall zone is believed to be attributable to the impermeable nature of the ground produced by the intense degree of weathering which occurred in the fault zone. Grouting cannot improve the strength of a weak matrix which is virtually ungroutable. However, a large amount of grout was injected, and the ultimate destination of this remains a mystery. It is probable that it was washed away by groundwater movement. Grout mixes started with a thin grout, and were rapidly thickened to a 1:1·6 pure cement grout mix.

148. For an answer to their question in § 73, *Mr Douglas et al.* are referred to § 127.

149. In reply to the question in § 74, the choice of the four-boomed jumbo was to maximize the input of UK industry into a sphere in which they do not have world prominence. With hindsight, the ultimate arrangement of a three-boomed jumbo with access basket and Cat 980 was very satisfactory. The DJB dumptrucks did not require a large passing bay. Passing bays are generally 5 m wide × 5 m high × 8 m deep (normal to tunnel).

150. With regard to § 75, no obvious correlation existed between rock type and quality and usage of explosive. All the rock was metamorphic, including the limestone. The mineral content consisted of quartz, pyroxene, feldspar and mica. Many different options were experimented with in order to reduce the charge, including the use of uncharged holes.

151. In reply to the question in § 77, hydraulic model testing was carried out by the Hydraulics Research Station, Wallingford.

152. In reply to the question in § 78, the accuracy of the raise boring of the shafts was adequate for their intended use as pilot shafts for mucking. Greater accuracy could have been provided at a higher price which would also have entailed a slower rate of pilot hole drilling. The weak limestone layer was stabilized by the installation of a soffit shutter and a precast pipe (for ventilation), and by the concreting of the annual surround.

153. The limestone and solution cavities caused no problems during shaft enlargement. However, the limestone voids did cause a problem during pilot hole drilling because of the loss of flushing water. Drilling had to be interrupted and grouting had to be carried out, involving the removal of the drilling rods.

154. With reference to § 79, no concrete trials were undertaken to determine thermal shrinkage.

155. The problem raised in § 80 appears not to be uncommon judging by reports received from other tunnels where this concreting method has been used.

156. The flaws in the concrete were made good by cutting out the faulty concrete and by filling with shotcrete sprayed into a neatly keyed repair. The effectiveness of this was tested by coring through the completed repair and carrying out compressive and shear tests. The interface was often not discernible and the strengths proved entirely satisfactory.

157. The steel-lined section was concreted in 30 m long fixed length pours, using a concrete pump. The pump and slick line were withdrawn as the concrete advanced in the crown.

158. In reply to the question in § 81, the pressure of consolidation grouting was 10 bars. All holes, mostly 3·5 m in length, but some of 7 m, were grouted in single stages.

159. With reference to the question in § 82 on crown voids, these are, if anything, more prevalent than with fixed length pours. At one isolated location, a void of approximately 5 m^3 was discovered, with a skin of concrete only 50 mm thick over the tunnel crown. This was not apparent at the shutter stripping stage and was discovered only at the time of grouting.

160. In reply to the question in § 83, the intake was treated with 'Hydroseal Extra'. All other works, including the entire length of tunnel, were treated with 'Deepseal 200'. The choice between the different products was entirely commercial. Unfortunately, as the tunnel has never been dewatered, assessment of the efficacy of the treatment has not, as yet, been possible.

161. With regard to § 84, the design of the anti-vortex structure was developed from earlier designs on other projects which had proved successful in suppressing vortices. The design was tested on a hydraulic model which demonstrated that it would successfully inhibit the development of vortices. We did not experiment with other types of anti-vortex device since, from our previous experience, we were satisfied that the design adopted here was efficient and effective.

162. In reply to the question in § 85, as the penstocks were being tested before being encased in concrete at the anchor blocks, it was necessary to make provision for movement of the penstocks along their length and in directions radially outward at each anchor block (bend) position. To present an integral pipeline for testing, all expansion joints were locked to withstand the longitudinal forces.

163. At each bend, temporary supports were arranged to accommodate greased plates to permit combined longitudinal and radial movement. Plumb-bobs were suspended below each bend, and measurements were taken of initial positions before filling and during subsequent stages of pressurizing.

164. The permanent sliding supports between anchor blocks had their side guides removed to allow for radial movements near the

bends. During filling and pressurizing, measurements were taken of movements at all bends and of increases in girth dimensions at points of change in the penstock diameter. Wherever leakages occurred at expansion joints, these were tightened up, and all permanent supports were checked to ensure that no distortion or concrete cracking had occurred. All measurements were analysed and were within predicted safe limits.

165. The upstream trifurcation was also initially pressure tested separately, using 68 strain gauges at 16 locations.

166. In reply to the question in § 86, provision of a cut-off wall was not considered. Such a provision would have been complicated because water levels were designed to be the same on both the upstream and downstream sides of the power station (see § 9 of Paper 9569) and excavation for the walls would have resulted in fracturing of the foundation rock. Any resulting reduction in uplift pressures could not be guaranteed in the long term.

167. The rock foundations were subjected to consolidation grouting after placing of the first-stage concrete.

168. With reference to § 87, filling under the stay-rings and spiral casings is also referred to in § 77 of Paper 9569.

169. The mortar fill consisted of a 1:1:2 water:cement:sand mix. The grout used was initially a 5:1 water:cement mix which was thickened to a maximum of 2:1 water:cement ratio as work proceeded. The grout was mixed outside the building and piped to the inlet of the standpipe feeding the grouting circuit; the discharge into the standpipe being valved. The pressure in the grouting circuit was thus determined by the level of the standpipe which was kept topped up at all times during the grouting operation.

170. In reply to the final question by *Mr Douglas et al.* in § 88, it is considered unlikely that the pre-splitting had any significant effects on the joints, nor that the use of smooth blasting or line drilling techniques would have produced much better results.

171. With reference to *Mr Xavier*'s first question in § 89, Mr Xavier is referred to the contribution from *B. S. Piper* of the Institute of Hydrology (§ 28).

172. In reply to the question in § 90, the use of 150 mm aggregate for concrete in dam construction is normal practice, and testing methods have been developed which are appropriate. For example, 300 mm cubes, and even 450 mm cubes, are used for crushing tests which, in turn, require high-powered cube crushing apparatus and cranage for cube handling.

173. In reply to the question in § 91, a full range of testing was carried out to investigate the suitability of the aggregates and cement for use in the project. The testing included testing for alkali–silica reaction.

174. All cement used in the works was supplied from Mombasa, Kenya. A holding company was set up specifically for the supply of cement to the Victoria project, and later it was also used for cement supply to other Government projects.

175. Cement from Sri Lanka was considered for the tunnel works but was found to be inconsistent in quality, and supply was not dependable. Its gain in strength was slow and it did not comply with the required British Standard.

176. The main reason, however, for using imported cement was the dam, for which a relatively low heat cement of consistent quality was of considerable importance. A thorough review of the local market was made before the decision was taken to import.

177. With regard to § 93, the provisions made in stage 1 for a future extension of 210 MW included the intakes and a short length of tunnel therefrom. The future extension will require a second tunnel, surge shaft and penstocks, all as for the first stage. The work will be relatively straightforward and the cost in real terms should be lower than for stage 1, as access for tunnel driving from an intermediate location is already in place as well as the pre-excavation for the power station itself.

178. In reply to the question in § 94, we are not aware of any published papers on the construction of the Polgolla project.

179. With regard to the question in § 95, we are not aware of any future plans to divert water from the Mahaweli, other than at the existing diversion at Polgolla. All flows through Victoria pass on to Randenigala, Rantembe and on into the transbasin canal. The Madura Oya irrigation scheme carries the water northwards and eastwards, and, in due course, it may extend up to the Jaffna Peninsula.

180. Finally, with reference to § 96, information on the numbers of Sri Lankan engineers is given in Fig. 3 of Paper 9528 and in § 121. We are not able to break the numbers down into more detail.

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