The embankment dam

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The Geoffrey Binnie Lecture

P. A. BACK, Sir Alexander Gibb and Partners

Before I turn to the topic of my lecture it would be appropriate to say a few words about the man whose name we especially remember today - Geoffrey Binnie.

I consider it a great privilege to be invited to give this - the first Geoffrey Binnie Lecture and to recall briefly just a few aspects of his distinguished career and the considerable contribution he made both as an individual and also through his illustrious firm to the craft of dam building - and indeed to the profession of Civil Engineering.

Geoffrey Binnie was born in 1908 and died in 1989. He came from a long line of distinguished engineers - both his father and grandfather being eminent in the profession and who together founded the firm of Binnie & Partners in 1902. Geoffrey was at Charterhouse and then Trinity College Cambridge. He was born partially deaf and the need to overcome this disability which might have adversely affected his career perhaps gave him that extra determination to succeed. He proved himself a most able young engineer and after varied experience in Switzerland and Hong Kong and the U.K. he was appointed a Partner in his firm in 1939.

On the outbreak of war he volunteered for the Royal Engineers and served in North Africa and the Middle East from 1940 - 1945. At the end of the war he returned to the U.K. to find his firm reduced to no more than a handful of staff. From that low point Geoffrey was at the forefront of the reconstruction of his firm which under his leadership went from strength to strength and was to achieve an international reputation for excellence in the field of dam engineering.

Two projects in particular stand out:

- The Dokan Dam in Iraq and Mangla Dam in Pakistan

The Dokan Dam marked a watershed in the development of new computational methods for the analysis of arch dams.

Mangla Dam was at the time (1957) the single largest project undertaken by a consulting engineer. An organisation was set up with Binnie & Partners as Project Consultants and with Geoffrey Binnie as Project Partner. This is not the place to go into any detail on the Mangla project, but it is worthy of note that the project was carried through to a most successful conclusion and much of the credit for this was undoubtedly due to Geoffrey Binnie and the wise and firm, but always courteous leadership that he provided.

Today we honour his memory and the traditions of engineering excellence which marked his work.

I have chosen as a title for my lecture:

THE ULTIMATE DAM

No doubt this title will be considered the ultimate in cheek.

How can there be such a thing as the ultimate dam? Surely every dam is a once-off structure designed to meet the very specific needs of a particular site.

Very true - but perhaps only half true.

We all know, I am sure, that we each bring to the design of a dam our own particular preconceptions - our own idiosyncrasies - our own foibles and prejudices. We say that we always consider all options - but find strange to say that every project we are involved in tends to have a certain type of dam. And if another designer were involved it would almost certainly be a different type of dam.

I say that we delude ourselves if we believe that any one of us is truly impartial.

> Terzagi, Cassagrande, Penman - Embankment dams
> Andre coyne, Serafim - Arch dams
> Barry Cooke - Rockfill dams with upstream concrete face
> Schreder, Dunston, Hollingsworth - Roller compacted concrete

I certainly have to confess my own partiality!

I have been known to wax lyrical about the sublime beauty of a thin arch dam compared with the splodge of an embankment dam. Now there's prejudice for you.

To parody the words of Dryden in Absalom & Achitophel:

"For dams they've built of every size
that engineers and damsmiths could devise.
But each to his favoured type
will ever return - no matter what the
site."

If we could just for a few moments put aside our prejudices and look at the whole spectrum of dam engineering - where amongst the huge variety of choices may future dams tend to go? What should be the hallmarks of a dam designed for the 21st Century and hopefully be still there - like Buck Rodgers in the 25th Century.

I have selected 10 criteria which in my view would be desirable for the ultimate dam. Not every dam could meet all 10 - but the more that can be met - the safer and longer the expected life of the dam. The ultimate dam would, by definition, meet all the criteria.

I am well aware that in putting forward these criteria I am also setting out my own prejudices. I am sure that many of you would produce a different list. There is scope here for much argument - and fun.

But when I have produced my list I would then finally examine what type of dam these criteria drives us toward - and whether that type should then be the initial starting point for each new site we encounter - and we back away step by step only when other considerations force us to do so.

So here are my criteria:

1) Materials of construction are very durable.

2) Flood routing is independent of power or operator.

3) The dam will not be destroyed by overtopping.

4) There is redundancy in the structural behaviour of the dam.

5) The structure can accommodate reasonable settlements and deformations.

6) The structure is resistant to internal erosion.

7) The structure is capable of withstanding substantial seismic shock.

8) The structure should be highly resistant to destruction by acts of sabotage or bombs.

9) The reservoir behind the dam can be drawn down to at least half height and preferably emptied within a few weeks.

10) The dam satisfies the generation test.

There is no special significance in the order - all 10 criteria are important - but clearly there are many dams which can only satisfy some.

You may be puzzled by my 10th criteria - the generation test. More about that later - but that is one criteria which every dam built in the future should satisfy.

So a few words about the list.

1) <u>Materials of construction are very durable</u>

This, as indeed many of the criteria I have listed, almost goes without saying. But it is of course of quite fundamental importance. If we get this wrong, there is almost no other alternative than to abandon the dam and start again. Witness what we are having to do at Maentwrog in North Wales. That dam is suffering from Alkali Aggregate Reaction and is having to be replaced after only 60 years - 60 years should be as nothing in the life of a dam.

When I am asked how long should a dam last I always say its life should be indefinite

- it should be seen in terms of geological time. And in geology 100,000 years is recent.

A few years ago I was asked to report on a dam that had failed in Sri Lanka with grievous loss of life. The dam was in fact 1400 years old and the part that led to failure was the sluiceway, built by the Royal Engineers some 115 years ago. Derek Knight and I were amazed to see what those early Sri Lankan engineers had achieved. They built better than they knew and they built for keeps. Short termism is a modern scourge.

2) <u>Flood routing is independent of power or an operator</u>

It is a well known statistic that failure to correctly route a flood past a dam has been perhaps the single most common cause of dam failure.

The reasons for maloperation of the spillway gates are almost invariably for one of two reasons:

(1) The power supply has failed, making it impossible to open the gates - or

(2) The operator has not responded in time for an incoming flood.

The dam is then overwhelmed by the flood.

Clearly the safest way to deal with this problem is to have no gates at all - but a simple ungated overspill. But this can be very costly in terms of lost storage due to the allowance that has then to be made for the flood surcharge.

One way round this problem which we used on the Victoria Dam in Sri Lanka is an automatic gate which opens without the need for power or an operator and responds exactly to the needs for optimum flood routing. And the method is entirely mechanical. There is no dependence on sophisticated computer hardware or electronics.

3) **The dam will not be destroyed by overtopping**

This follows straight on from the previous criteria - but goes still further.

Spillway design is linked to statistical or other projections of what might happen in the future. And it is very easy to get the projections wrong. Take for instance the Manchu II dam in India. In the space of 20 years the projections and then modified projections were proved wrong and in the end resulted in a five fold increase in the design flood with of course the attendant dramatic change in the design of spillways.

In the middle of building Kariba we experienced what till then was the 1000 year flood which, of course, was then no longer the 1000 year flood. We had to add 50% to the capacity of the spillways.

All this emphasises the need for a certain humility in our attitude to design floods and surely demonstrates the benefit of constructing a dam which, even if we get our flood projections wrong, will not fail if the worst happens and the dam is overtopped.

Does this mean that embankment dams would never be able to satisfy the criteria. Not at all- but it puts pressure on embankment engineers to come up with a solution to this problem. The answer may lie in suitable geomembrane fabrics - or torpedo netting type reinforcement or some other device which protects the downstream face from being washed out by overtopping. That is the challenge.

4) **There is redundancy in the structural behaviour of the dam**

Redundancy is a very good thing. With all our cleverness we are still quite capable of getting it wrong - of not visualising correctly the possible mode of failure - or failing to take account of a particular loading configuration which might arise in the life of the structure - such as a severe earthquake or deterioration of foundation conditions. Redundancy enables the loads to be carried in more than one way and can significantly enhance the safety of the structure.

A typical example is a concrete gravity dam. Almost invariably these are built with a straight axis and therefore sliding resistance on the foundations is the only mechanism preventing failure. Introduce a gentle curve and immediately other forces would be brought into play to help carry the load if sliding resistance proved inadequate. Why do we <u>ever</u> build a gravity dam with a straight axis?

If I am a designer I will always insist on a curved gravity dam unless there are other overriding considerations.

In considering redundancy, however, we must be very careful not to delude ourselves as to the overall safety. The secondary resisting mechanism frequently only comes into play when the first mechanism has failed. We cannot add the two mechanisms together as they will in fact be overcome one by one. It is like tearing along the dotted line.

5) **The structure can accommodate reasonable settlements and deformations**

Perhaps the single factor in which our uncertainties are greatest is in the foundation of the dam - which is as much a part of the structure as the edifice above the foundations.

We inevitably have to make assumptions as to how the foundation will behave - and our assumptions are more likely to be inaccurate there than anywhere else. It is therefore very desirable that the dam can accommodate deformations without suffering unduly. Or that if the foundations yield, other mechanisms come into play which transfer the load safely elsewhere - and the dam's safety is not jeopardised.

One type of dam which is particularly good at redistributing stress is the thin arch dam but there is of course a limit to how much stress re-distribution can be tolerated.

The Kariba dam is a case in point. At this dam the North bank and up to half of the South bank the foundations of the dam were on excellent rock. However, the upper half of the South bank was composed of quartzite with thick bands of clay. The quartzite itself was good and at first we tried to jet out the clay but without much success. Our concern was that the quartzite and clay would deform under load relatively easily and the loads would therefore be reflected downwards to the hard relatively unyielding gneiss which might then become overstressed.

Paradoxically, the dam might then fail at its strongest point. So it was that we decided to construct four massive underground buttresses to carry the thrust through the quartzite and into the underlying gneiss. Of course, we then had to completely ignore any contribution from the quartzite in carrying the loads from the dam.

6) **The structure is resistant to internal erosion**

If a dam is to have an indefinite life all the material that goes to make it should be in a state of stable equilibrium.

Internal erosion, however slow and whether it be as a result of chemical action or of physical movement of particles represents an ultimate threat to the security of the dam. We must satisfy ourselves therefore that such phenomenon as piping, or dispersivity, or leaching or AAR or any other phenomenon which is changing the internal condition of the structure will not happen. For concrete dams, as our knowledge of the chemistry of aggregates and cements grows, so too are we coming to realise the considerable problems we can face in completely satisfying this criterion.

7) <u>**The structure is capable of withstanding substantial seismic shock**</u>

This may seem obvious, but past practice in dam design has frequently treated seismicity with scant respect.

Now that we have much more sophisticated computational methods to hand we know that the old approach of treating the seismic load as a pseudo static phenomenon is very misleading and can underestimate the real effects of an earthquake very significantly.

For example, in the case of an arch dam we analysed - using first a dynamic approach and then a pseudo static method for the same peak acceleration, the dynamic method gave a threefold higher maximum stress in a completely different part of the structure.

In considering what seismic effects to design against, one naturally tends to look at past history for guidance - in much the same way as we do for floods. There is a growing view in some quarters that such an approach may be misleading. And that indeed some of the greatest shocks that are waiting to happen is where previous seismic activity has been low.

I understand that in the USA where traditionally all the emphasis has been on the Pacific Seaboard, some seismologists believe that the Eastern Seaboard could be in for even greater trouble.

I have just returned from a visit with Derek Knight to the Philippines where they had a Magnitude 7.7 earthquake in July close to two of their major dams. We were asked to inspect the dams and advise on remedial works.

It is of interest to note that on one of the dams with an upstream concrete face, not as the water barrier, but in place of rip rap protection, the concrete face was crushed in one area and pulled apart in another. Undoubtedly, had the concrete face been the water barrier, the dam would have been breached.

The construction of the concrete face was not exactly as it would have been for use as a water barrier, and reinforcing steel did not pass through the joints in the slab - but the degree of damage to this concrete certainly raises questons in my mind as to the seismic performance of dams which depend on such concrete faces as the main water barrier.

8) <u>**The structure should be highly resistant to destruction by acts of sabotage or bombs**</u>

It is a sad commentary on the age in which we live that such a criteria has to be included. But included it should be because undoubtedly a dam that is vulnerable to easy destruction could become the target of terrorists - witness the agonies that the security people are having over the Channel Tunnel.

The dam type which obviously is most vulnerable to sabotage is the buttress or multiple arch. In some countries such dams are no longer allowed to be constructed. And if they already exist the space between the arches have been infilled. However, it may well be that it is still an economic form of construction even allowing for such infilling.

9) <u>**The reservoir behind the dam can be drawn down to at least half height and preferably emptied within a few weeks**</u>

I am surprised that, as far as I know, no country has actually legislated such a requirement and that is has been left to the designer - or the owner as to whether or not there is any significant drawdown facility.

When one considers the regulations which control the nuclear industry it is surprising that dam safety is still generally treated so perfunctorily. It surely must be a matter of normal safety and common sense, that once impounding begins behind a dam, it is possible to reverse the process if a problem arises. Otherwise all control is lost and we can become helpless spectators to a possible catastrophe.

It is perhaps because we have not had this requirement imposed upon us that we have had to be so conservative in our designs to ensure that when uncontrolled impounding takes place nothing will go wrong. We actually might save money by spending more at the outset on ample drawdown facilities.

10) <u>**The dam satisfies the generation test**</u>

This criterion has nothing to do with the basic design or safety of the dam - but should be applied to determine whether the dam should be built at all.

We dam designers and builders do not enjoy a very good press. We are frequently thought of as despoilers of nature bringing destruction to flora and fauna

- threatening the habitat of man and beast alike

- upsetting the delicate balance of nature and generally causing a net disbenefit to the community

Sometimes these strictures are true - and sometimes patently false. But we do ourselves and our profession enormous harm if we do not bring a truly critical eye to our activities and learn to differentiate between the good and the bad scheme and to resolutely oppose the bad. I am deeply troubled by those who in their enthusiasm sweep aside all criticism as if it was always uninformed, and blindly support dam building as if it was always a good thing.

Now in this day and age whether a scheme goes ahead or not is often decided by Accountants and Bankers applying their economic criteria of cost benefit analyses and internal rates of return to assess the merits of the project. But I believe such decisions are far too important to be left to Accountants and Bankers. Their horizon is limited to something like 30 years. Nothing that happens beyond that time has any real significance for them. But we are building structures to last for hundreds if not thousands of years.

And so I believe we must introduce another test to be satisfied that what we are doing is good - and I have called this the generation test.

This is an approach I first put forward two years ago at a Seminar in the United States on the subject of River Basin Development in Africa.

Consider two development options - one with a 12% internal rate of return - the other with 6%.

In 30 years time the 12% option may have created a desert - by say reducing the water table catastrophically.

The 6% option, on the contrary, may have created a thriving community. Yet by strict economic criteria, the choice is made to go for the 12% option - because it satisfies the repayment criteria.

Such a limited and clearly inadequate basis for guiding decision making will no longer be acceptable. Evaluation of a project must look beyond the immediate economic goals - it must look to succeeding generations.

Planners must metaphorically be made to stand before their children 30 years on and tell them "this is what we have done". Here is this desert or here is this fertile land - for you" Only if this generation test is passed, should a scheme be considered worthy of support.

Finally, with these 10 criteria before us what will the ultimate dam be like. We must of course assume that the 10th criteria has already been satisfied.

My ultimate dam would be built with relatively low grade RCC in the form of an arch gravity structure without joints, with sloping upstream and downstream faces and with an impermeable membrane on the upstream face, possibly created by a clay filled geosynthetic mattress which is protected by sand and random rockfill. I would take no special measures to make the RCC impermeable, indeed I would wish it to be relatively porous to avoid uplift and ensure that the phreatic surface is close to tailwater level.

Such a dam would be designed for overtopping and could accommodate reasonable settlement without affecting its ability to store water. It would be highly resistant to internal erosion or earthquake damage or sabotage and would of course have to include low level outlets to permit drawdown. The dam could be overtopped safely during construction so the cost of diversion would be minimised. Whether or not gates are provided at the crest or a simple overspill is adopted is a matter of choice depending on the required storage volume - but if gates are used they should be capable of operation without power or a controller.

Mr. Chairman, I am aware that much of what I have said is highly controversial and I dare say if 100 of us were asked to define our ultimate dam, there would be 100 different answers. For what it is worth I have given you mine but I don't expect that I will ever have the opportunity to build it. Life is not like that and with dams the evolution of methodology is, probably with good reason, very very slow.

1. Design and performance of the forty mile Coulee East dam on a soft clay foundation

B. G. CHIN, D. M. DAVISON, E. K. KLOHN, R. P. BENSON, and J. W. CAMPBELL, Klohn Leonoff, Calgary, Canada

The Forty Mile Coulee East Dam is a 28 m high earthfill embankment founded on up to 60 m of highly plastic, soft clays. Large foundation movements were measured during construction. Moreover, high pore pressures exceeding predictions by up to 35% were recorded, despite the benefits of a test fill and the (then) conservative assumptions used in design. This paper describes the construction performance of the East Dam and highlights some of the key trends of instrument readings. The results of finite element analyses to match the field behaviour are also shown, which demonstrated that undrained yielding of the clay can result in horizontal stress increases much larger than those assumed in a linear elastic model. Neglect of this behaviour will lead to unconservative estimates of the pore pressure response in the field.

INTRODUCTION

1. As part of continuing efforts to alleviate problems of cyclic drought patterns, the Alberta (Canada) government commissioned construction of the Forty Mile Coulee Project to supply additional water and to improve water delivery times to the farmlands in southern Alberta (Fig. 1). The project includes two 28 m high earthdams (East and West Dams); a 10 km 86 000 000 m^3 reservoir; a spillway; an inlet chute; a 750 m long inverted syphon; and a 20 m^3/sec pumpstation (Fig. 2). Construction was staged over a two year period from 1986 to 1987, and the reservoir was filled to full supply level in 1989. The total cost of the project was $55 million.

2. Foundations at the damsites comprise soft lacustrine clays up to 60 m and 35 m thick at the East and West Dams, respectively. During construction, the West Dam performed generally as expected. However, the East Dam performance was poorer than anticipated, requiring design modifications to enlarge the toe berms for construction prior to raising the embankment to its ultimate height. Vertical and horizontal movements in the foundation were up to 1.6 m and 0.3 m respectively at the end of construction, and have continued slowly after construction (2.5 m and 0.5 m to May 1990). Moreover, unusually high pore pressures exceeding predictions by up to 35%, coupled with minimal or no dissipation, raised concerns with respect to stability during reservoir operations. Finite element studies were undertaken to investigate

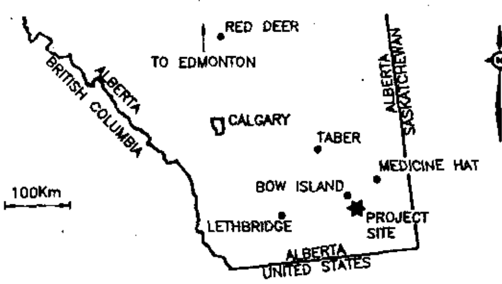

FIGURE 1 – LOCATION PLAN

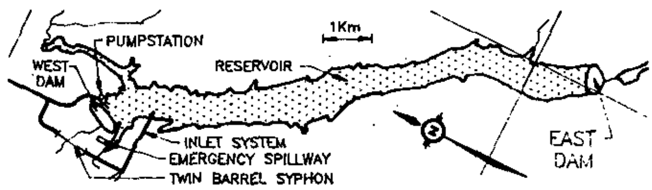

FIGURE 2 – FORTY MILE COULEE PROJECT SITE PLAN

the key factors controlling field behaviour, and to evaluate future performance. Based on the results of this study, and because the downstream implications of a dam breach are minimal, the reservoir was allowed to be filled in a controlled manner. Completed to full supply level in mid-1989, the performance of the East Dam has been satisfactory to date.

3. This paper describes the performance of the East Dam foundation during construction, highlights some of the key trends of instrument readings, and presents typical results of the finite element studies. In particular, pore pressures predicted by finite element analyses are shown to provide a good match to field response, and are compared to original estimates to highlight the inaccurate (and unconservative) predictions obtained from linear elastic models for deformable foundations. Space limitations preclude a thorough discussion of all data and analytical studies carried out. Nevertheless, sufficient detail is presented to illustrate the importance of the observational approach to the design and construction of large embankments on soft clays.

FOUNDATION CONDITIONS

4. Formation of prairie coulees typically resulted from ice marginal channels during the last glaciation, which eroded through the glacial drift into bedrock. Deposition began as the major flows subsided, with soils derived primarily from adjacent bedrock and eroded upland glacial drift. Coalescing slopewash colluvium formed along the coulee walls below bedrock outcrops, while lacustrine clays and silts

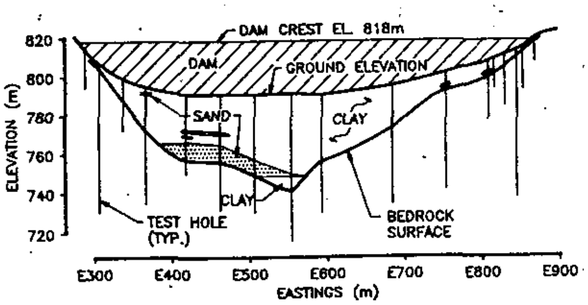

FIGURE 3 – STRATIGRAPHIC PROFILE ACROSS VALLEY

deposited behind either fans spreading across the coulee or ice dams from local glacial advances. The Forty Mile Coulee was downcut by up to 90 m, then infilled with up to 60 m of sediments. These sediments comprise highly plastic lacustrine clays near mid-valley, interfingering with sandy colluvial soils toward either side of the coulee. On the south side of the coulee, at the East Dam, a thick, beached sand deposit underlies the clays above a near horizontal shelf in the bedrock surface (Fig. 3).

5. Liquid and plastic limits of the clays range from 50% to over 100% and 18% to 25% respectively. Natural moisture contents vary from 20% to 35% and liquidity indices from 0.1 to 0.3. Clays of higher plasticity (liquid limit 90% to over 100%) are more predominant and much thicker at mid-valley. Towards the coulee walls, the clays are less plastic (liquid limit 50%±) and contain more frequent sand layers as a result of interfingering with the colluvial soils.

6. Consolidation tests and pocket penetrometer readings, indicated that the clay had been overconsolidated to a depth of about 18 m. The primary process of overconsolidation at this site is believed to have been by desiccation. A typical soil log at mid-valley, and the inferred site stress history are shown on Fig. 4.

EMBANKMENT SECTION

7. The effective stress method was used for design of the dam. Initial predictions of pore pressures were based on Henkel's[1] pore pressure equation, using pore pressure parameters estimated from Law and Bozozuk[2] and total stress

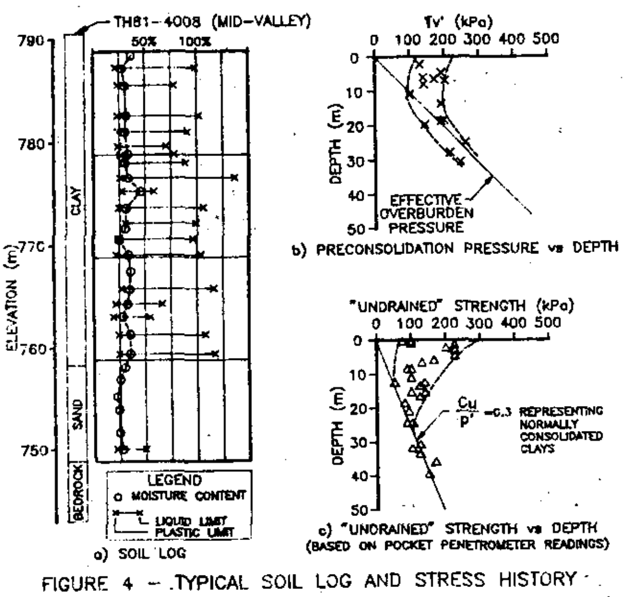

FIGURE 4 – TYPICAL SOIL LOG AND STRESS HISTORY

changes from linear elastic solutions[3]. Measurements beneath a 12 m test fill constructed in 1982 (Fig. 5) showed good agreement with the above predictions, and therefore the same method was applied for design of the main dams. Dissipation parameters were derived from the test fill data, but modified to account for possibly slower dissipation beneath the much wider base of the East Dam.

8. Despite the benefits of a test fill, and as a safeguard against unfavourable pore pressure behaviour, provisions were included in the contract documents to construct toe berms if required. The initial design of berms was based on the conservative assumptions of no dissipation using undrained pore pressures. Furthermore, the observational method[4] was selected as an appropriate approach to design and construction. Therefore, the necessary instrumentation was included to permit ongoing monitoring and evaluation during construction.

9. The final design tendered for construction had 8H:1V slopes, and included an upstream impervious clay zone, a downstream random zone, a vertical chimney drain/filter, a drainage blanket/finger drain system along the downstream base, and toe berms (Fig. 5). The wide plastic upstream zone and the vertical chimney drain/filter were included to provide protection against cracking due to the large deformations anticipated.

10. As a result of the observational method, higher than expected pore pressures were able to be identified during construction, and allowed design changes to be made part way through construction to enlarge the toe berms. These berms were constructed prior to raising the embankment to its full height. The as-constructed geometry at the end of construction is shown and compared with the original design on Figs. 5 and 6.

FOUNDATION PERFORMANCE DURING CONSTRUCTION

General

11. The instrumentation program included 136 pneumatic piezometers in the foundation and embankment fill; 17 pneumatic settlement sensors; 6 overflow manometer settlement gauges; 3 Sondex settlement gauges; and 6 telescoping type inclinometers socketed into bedrock, which had settlement rings attached to measure incremental settlements in the foundation. Some of these had been installed for the test fill. In addition, 66 surface monuments (of which 25 were abandoned as a result of berm construction) and 8 standpipes downstream of the dam were installed. Typical layouts of the instrument types are shown on Fig. 5. Locations of the instrumented sections are shown in plan on Fig. 6.

12. Because of space restrictions, only selected data relevant to the foundation behaviour are presented. For reference to the following discussions, typical time plots of pore pressure responses, settlements and horizontal movements are shown on Fig. 7.

Pore Pressure Response

13. All piezometers exhibited a steady rate of rise during construction, reaching a well defined break in the response curve after each

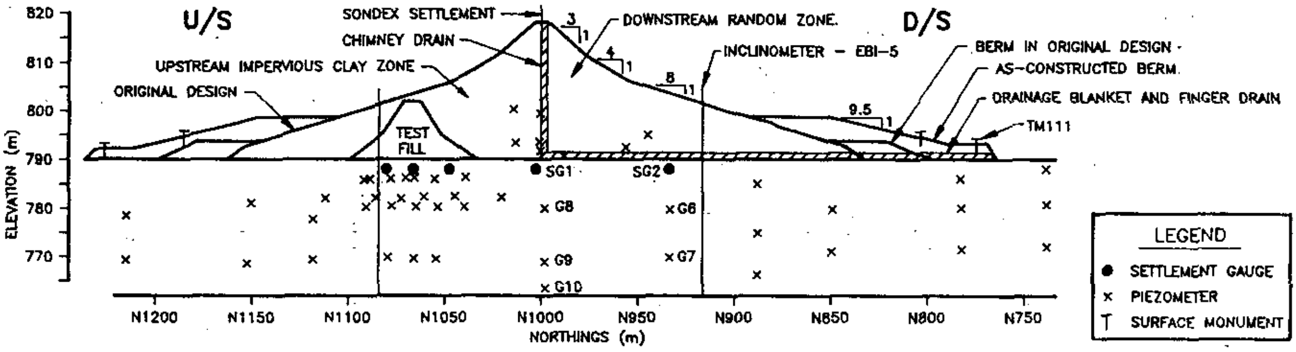

FIGURE 5 — TYPICAL DAM SECTION AND INSTRUMENTS

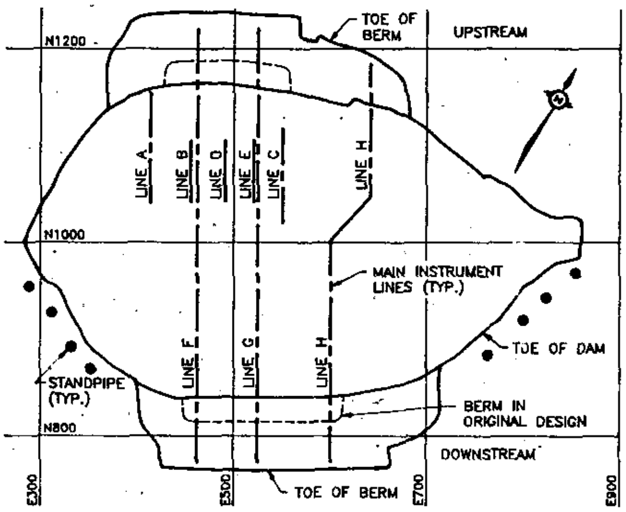

FIGURE 6. — EAST DAM KEY PLAN

construction stage (Fig. 7). Piezometers near the toes also exhibited a decrease in the rate of rise, as construction moved further away from the piezometers. Some piezometers continued to rise slowly after construction, with most piezometers near mid-valley showing minimal or no dissipation to date (May 1990).

14. The maximum piezometric surface measured at the end of construction, together with original design projections, are shown on Fig. 8. The field $\bar{B}$ values (defined as the ratio of the excess pore pressures to the total vertical stress) varied from about 0.5 near the toes up to about 1.2 near the centreline.

15. Further insight into the pore pressure behaviour can be gained from Fig. 9 which shows the piezometer data as a function of embankment load. For reference, a 45° line is included to represent the case where the excess pore pressures are equal to loading. The following points are highlighted:

- Piezometers G6 and G8 installed at 10 m depth showed an initial low response to loading, followed an increasing, almost linear response paralleling the 45° line.
- Piezometers G7, G9 and G10 installed at or below 20 m depths exhibited a high response immediately at the start of construction, nearly matching the 45° line. They deviate away from this line at higher dam heights, however, this may be due to the "stress bulb effect".
- Piezometers G8, G9 and G10 located directly beneath centreline exhibited an increase in response steeper than the 45° line above the 300 kPa stress level.

16. The first two observations generally confirm the site stress history established by laboratory tests. That is, the upper parts of the clay formation behaved initially as an overconsolidated soil, followed by an undrained, normally consolidated response. The stresses at which this change in behaviour occurred were 65 kPa and 105 kPa, which are within the range of preconsolidation pressures measured in the laboratory. The deeper clays, however, behaved as a normally consolidated soil from the beginning of construction.

17. The third observation implies that during the final stages of construction, the incremental pore pressure increases were greater than the incremental increase in vertical stress. This behaviour is believed to be attributed to local yielding of the clay, as a result of being critically stressed.

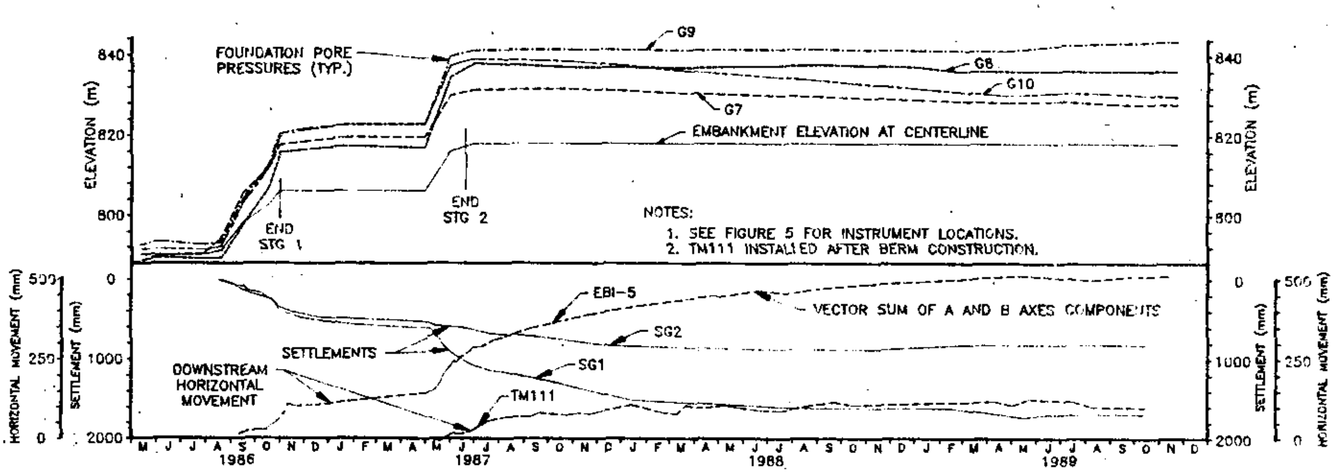

FIGURE 7 — TIME PLOT OF TYPICAL INSTRUMENTATION READINGS

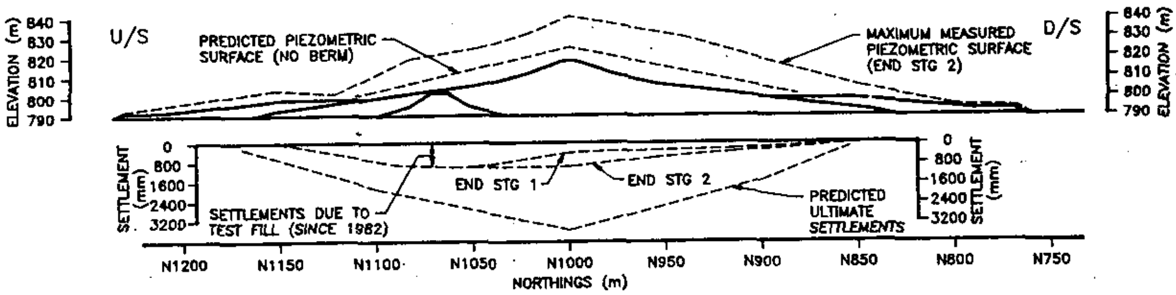

FIGURE 8 – PIEZOMETRIC SURFACE AND SETTLEMENT PROFILE AT MID-VALLEY

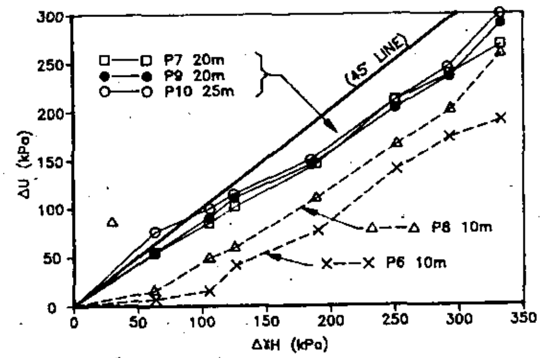

FIGURE 9 – PORE PRESSURE VS EMBANKMENT STRESS

Vertical Deformations

18. Foundation settlements exhibited an immediate response during construction, followed by time-delayed consolidation after construction (Fig. 7). Profiles of the measured settlements for various time periods, and the predicted ultimate values are shown on Fig. 8.

19. Fig. 10 shows the immediate settlements (i.e. during construction periods only) as a function of embankment load. By considering only the immediate settlements, the slope of the response curve can be viewed as indicative of an average undrained deformation modulus of the entire foundation. As shown, the trend is similar to that observed for the shallow piezometers within the overconsolidated portions of the foundation. Thus, beyond a critical stress level, an initial low deformation response is followed by a large deformation response, and the average foundation stiffness was successively decreased. This can be attributed to a progressively greater proportion of the upper zones behaving as a normally consolidated soil as the critical stress levels were reached. The entire foundation is speculated to have reached a normally consolidated state at 200 kPa, which is consistent with the maximum preconsolidation pressure measured in the laboratory.

Horizontal Movements

20. Typical horizontal movements during construction are shown on Fig. 7. Analysis of horizontal deformations is extremely complex, and indeed is less understood in geotechnique compared to settlement behaviour. From a construction perspective, it must be ascertained if the movements reflect normal behaviour, or there is an impending failure. Horizontal deformation near the toe is a sensitive indicator of instability. In this respect, toe movements at the East Dam were nominal during construction and have since ceased, confirming the embankment's stability. However, lateral displacements in inclinometers at midslope were found to be near the upper bound of values previously experienced for stable embankments on similar soils in the Canadian prairies, and are continuing (although at a decreasing rate). The depth of movements also appeared to coincide with the critical slip surface determined by limit equilibrium analysis. Thus, a detailed review was undertaken in an attempt to better understand the mechanics of the movements, and to confirm the foundation stability.

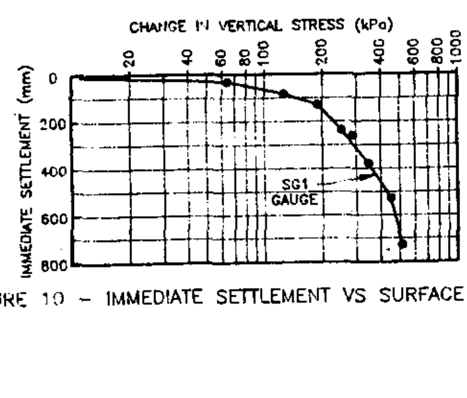

FIGURE 10 – IMMEDIATE SETTLEMENT VS SURFACE LOAD

21. The review was carried out in two parts. Firstly, other case histories were reviewed to provide a comparison with the movements at the East Dam. The results suggested that the construction movements were not unusual, given the deformation characteristics and thicknesses of the foundation materials involved. Secondly, the characteristics of the shear strains were analyzed with respect to the vertical strains in the foundation. It was thought that an analysis of this type would help determine if the movements were occurring under drained or undrained conditions. Movements in inclinometer EBI-5 are illustrated on Fig. 11. Part a) shows the cumulative movements versus depth for separate time periods. Part b) compares the shear strains to the vertical strains determined from settlement rings attached to the casing. Part c) shows the time history of the two strain types at the depth of maximum movement. The following key points are highlighted:

- The major movements occur over a 6 m to 7 m thick zone, located at about 25 m depth. Above this zone, the shear strains in the clays are minimal.
- The pattern of vertical strains is an approximate mirror image of pore pressure isochrones, consistent with double drainage in one-dimensional consolidation theory. Local "peaks" in the vertical strain distribution are likely indicative of intermediate drainage boundaries. It is noted that vertical strains within the middle of the clay formation are also minimal.
- Maximum shear strain is located near the base

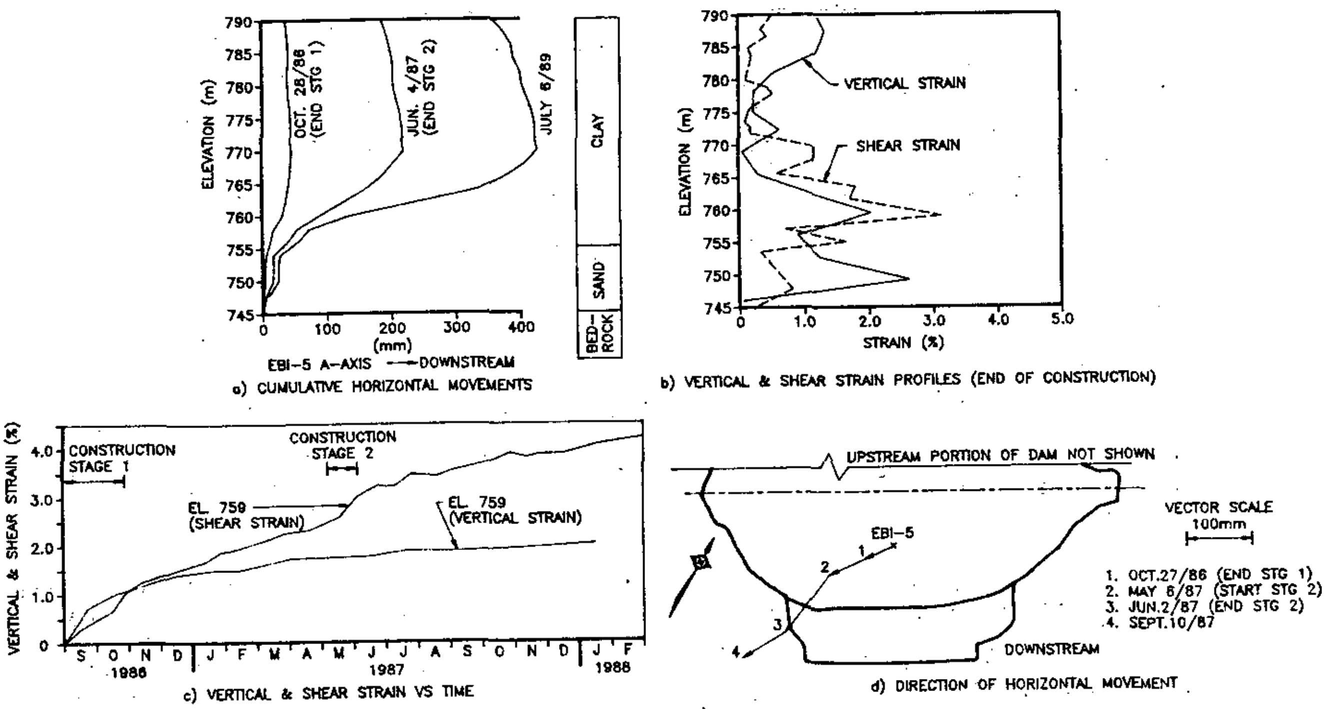

FIGURE 11 – INCLINOMETER DEFORMATIONS – EBI–5

of the clay, immediately above a thick sand layer. Significant vertical strains are also recorded at this depth, which suggests that the shear strains can be attributed to the development of consolidation.

Fig. 11 c) shows that both the vertical strains and shear strains increased in a similar manner during stage 1 construction, followed by a time dependent increase over the shutdown period. During stage 2, however, the shear strains are further increased, accompanied by very little additional vertical strain. This change in behaviour suggests that the early shear strains were related to consolidation, while the shear strains in the final stages of construction likely occurred under undrained conditions. A review of the construction history indicated that the transition from "partially-drained" to "undrained" behaviour occurred when the embankment was about 3 m to 4 m below ultimate height.

22. A different perspective of the deformation behaviour was obtained through a review of the directions of movements (Fig. 11 d). The results further illustrate the influence of the deep sand layer on the horizontal movements. As the sand is located on the south side of the coulee (Fig. 3), the major movements tended toward the sand where, as shown by the analysis of vertical strains in the foundation, greater consolidation is occurring. During stage 2 construction, the movements apparently swung more normal to the dam axis. Thus, the direction of movements appears to be consistent with the interpreted "undrained" behaviour of the shear strains near the end of construction. Following construction, however, the movements have swung back toward the sand layer, suggesting a progressively greater influence from consolidation. It is noted that the inclinometers located further north and away from the influence of the sand consistently showed movements perpendicular to the dam axis, as would be normally expected.

23. The above observations indicated that any unfavourable (undrained) shear strains in the foundation occurred near completion of the embankment, when the toe berms were already in place. The fact that minimal movements were recorded at the toe confirmed that the berms were successful in mitigating any potential for instability that may have existed. Of interest, the shear strains have continued at a steadily decreasing rate after construction, and there is presently no evidence of unstable behaviour in the foundation.

FINITE ELEMENT STUDY

General

24. The instrument data have confirmed the complexity of soft clay behaviour beneath embankments. To develop a basis for evaluation of future performance and to clarify the various aspects of clay behaviour, finite element studies were undertaken to match the observed performance during construction.

25. Stress-deformation analysis for the East Dam was carried out using FEADAM 84[5], which simulates the non-linear and stress dependent stress-strain properties of a soil by a hyperbolic function. Both the embankment fill and foundation clays are quite ductile and are well-suited for analysis with the hyperbolic model. Derivation of the hyperbolic parameters was based on consolidated-undrained triaxial tests and oedometer consolidation tests, supplemented with data from the technical literature. Parameters were assigned to different soil layers to account for over-consolidation and/or partially-drained, drained or undrained behaviour, based on actual behaviour inferred from the instrumentation data. Fig. 12 shows the finite element grid used for analysis of the dam at mid-valley. Construction was simulated by applying the dam in successive lift increments.

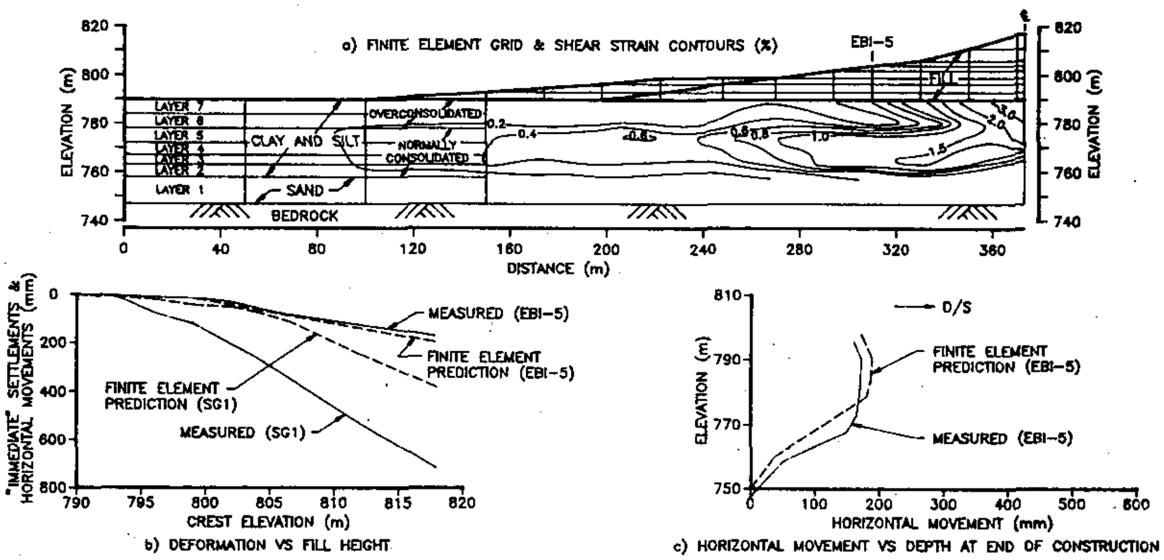

FIGURE 12 – STRESS DEFORMATION ANALYSIS

26. FEADAM 84 does not account for pore pressures and it was necessary to apply the construction lift increments as a single stage. This approach was considered reasonable in view of the minimal pore pressure dissipation observed during winter shutdown. For comparison to the computed deformations, however, only the portions of the movements which occurred during construction were used. Some fine-tuning of the model was necessary in order to achieve a satisfactory match to field performance.

Comparison of Deformations

27. Fig. 12 shows the key results for comparison. Part a) superimposes on the finite element grid, the contours of the computed shear strains in the foundation. Direct comparison of the strain magnitudes may not be applicable because of the coarseness of the finite element grid compared to the depth interval measured by the inclinometers. However, the overall pattern of the contours indicates the largest strains occurred beneath the central portion of the dam decreasing towards the toe. This is in agreement with the observed behaviour, where the inclinometer movements at mid slope were greater than those at the toe. Moreover, the contours reflected a preferential zone of movement at about 20 to 25 m depth, consistent with the movement zone recorded in inclinometer EBI-5. Parts b) and c) compare the computed horizontal movements at various embankment heights, and the cumulative movements with depth at the end of construction, to those measured in the field. Again, it is evident that a satisfactory match was achieved.

28. Comparison of vertical deformations (Fig. 12 b) showed that the measured settlements were slightly higher than those computed. This can be expected, however, due to the locally greater consolidation at intermediate drainage boundaries, which was not accounted for in the analyses. Nevertheless, the general trend of computed settlements during dam raising is consistent with that observed.

Foundation Stresses and Pore Pressures

29. Fig. 13 compares the stress increases computed by finite element analyses to those computed from linear elastic solutions. The comparison is made for three piezometer locations (10, 20 and 25 m depths) beneath centreline. As shown, the vertical stresses from both methods compare very well in each case. The horizontal stresses at the shallow 10 m depth are also in agreement. However, there are significant differences between the horizontal stresses at the 20 m and 25 m depths, where the finite element results are up to 100% greater. Of interest, the major zones of horizontal movements also occurred at these depths.

30. Fig. 14 shows the recalculated undrained pore pressures using the stresses from FEADAM 84 in Henkel's pore pressure equation, for the two deep piezometers beneath centreline (G9 and G10). Also shown are the measured pore pressures and

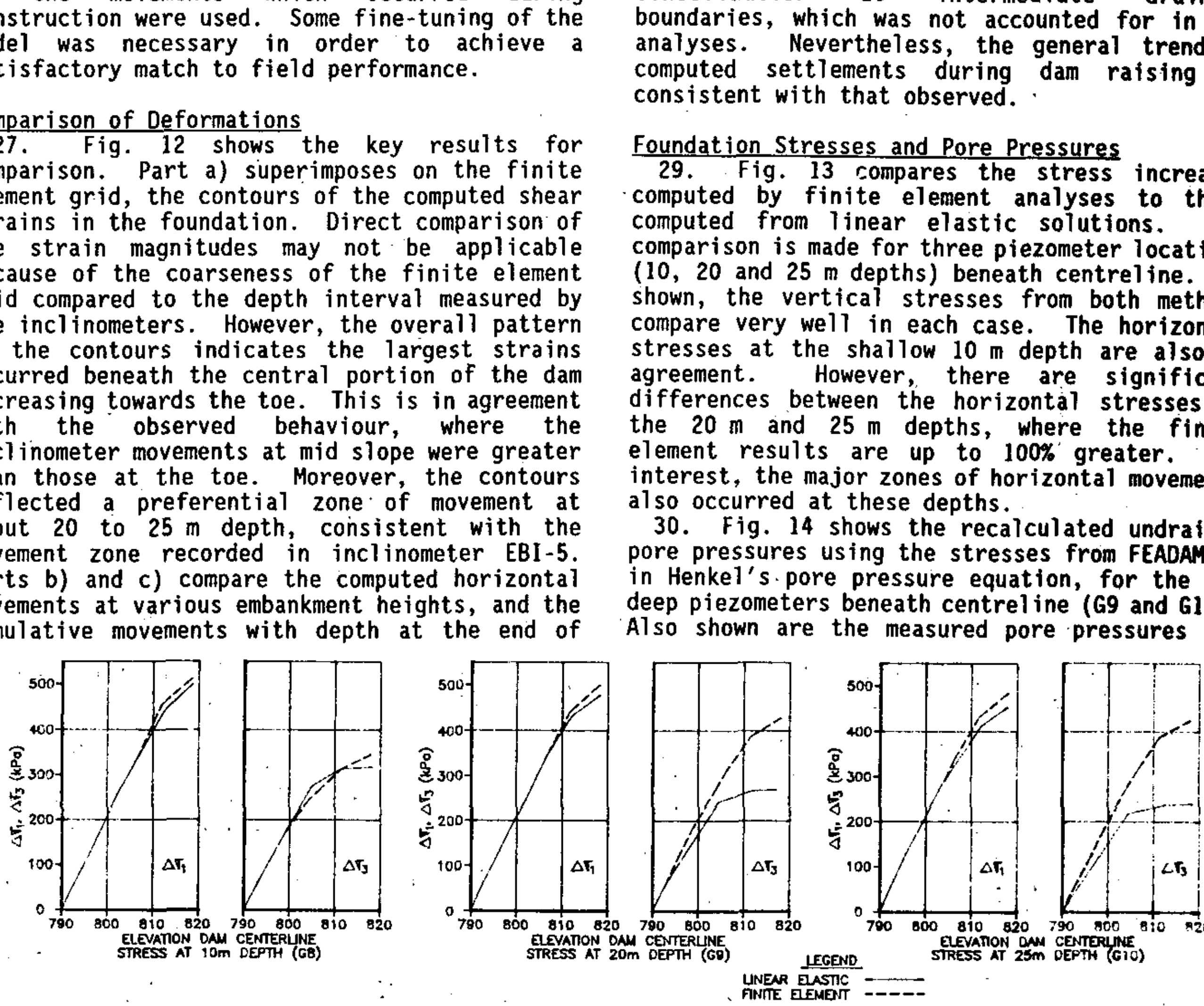

FIGURE 13 – COMPARISON OF FINITE ELEMENT AND LINEAR ELASTIC STRESSES

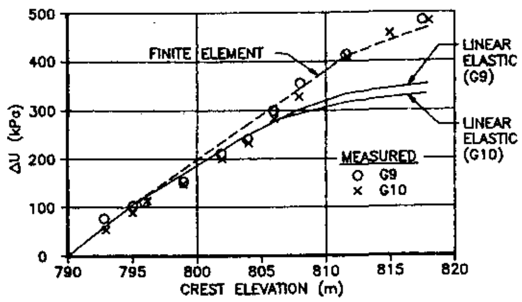

FIGURE 14 – COMPARISON OF PORE PRESSURE PREDICTIONS

those predicted by linear elastic methods for comparison. As shown, the pore pressures computed from linear elastic solutions significantly underestimated the actual pore pressures at the end of construction. However, the finite element solutions not only correctly predicted the final pore pressures, but also the rate of pore pressure generation during construction. Although not shown, the finite element model also provided a satisfactory history match of pore pressure generation for the remaining piezometers along the dam section analyzed.

CONCLUSIONS AND COMMENTS

31. The East Dam case history has provided an excellent illustration of the value of the observational approach to the design and construction of large embankments on soft clay foundations. Successful completion of the project within budget and on schedule may not have been achieved without adopting this approach.

32. The performance of the East Dam has confirmed the complexity of soft clay behaviour beneath embankments, and re-affirmed the need to account for the stress history and yield behaviour of the clays as well as the anticipated types of loading, in prediction methods. Despite this complexity, the present study has shown that it is possible, with the advent of modern computers and advanced analytical tools, to accurately forecast settlements, horizontal movements and pore pressures concurrently.

33. One of the most important aspects of the finite element studies was to demonstrate that undrained yielding of the clay can result in horizontal stress increases which are much larger than the corresponding stresses for clays deforming in a linearly elastic manner. Neglect of this behaviour can lead to an unconservative estimate of pore pressures, as experienced at the East Dam.

34. The finite element studies presented in this paper were implemented because of concerns for embankment stability during operations, due to minimal dissipation of pore pressures and the slow rate of increase in the factor of safety after construction. The calibrated finite element model was subsequently used to evaluate and predict the performance during reservoir filling. Because of space limitations, it was not possible to include the data collected during the filling periods in this paper. However, the observed behaviour essentially conformed to the finite element predictions and since reaching full supply level in mid-1989, the embankment has performed satisfactorily. Monitoring of the embankment is continuing on a reduced basis.

ACKNOWLEDGEMENTS

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2. The application of new techniques in the design of two high dams in South West China

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The paper describes two high dams located in the southwest region of China. The author puts emphasis on presentation of the design of the Lubuge rockfill dam with a saprolite core and the design of the Tianshengqiao I concrete face rockfill dam, which reflect the modern design level of dams in China.

I. LUBUGE ROCKFILL DAM WITH A SAPROLITE CORE

The Lubuge hydropower project is located on the Huangni river, a tributary of the Nan Pan river between Yunnan Province and Guizhou Province in the southwest of China. It was designed by the Kunming Hydroelectric Investigation and Design Institute, Ministry of Energy and Ministry of Water Resources. The headworks and the powerhouse complexes of the project are being constructed by a Chinese construction force, the Fourteenth Construction Bureau (FCB). The Taisei Corporation was awarded the contract to build the hydraulic system.

1. Introduction

The reservoir formed by the dam has a total storage capacity of 1.11×10^8 m^3, which is capable of seasonal regulation only. The total installed capacity of the project is 600 MW and its mean annual output is 28.5×10^8 KWH.

The construction of the project started at the beginning of 1983. The first two generating units were put into commissioning in December, 1988 and September, 1989 respectively. The other two generating units are scheduled to be put into commissioning by the end of 1990.

The project comprises three major structure groups, namely, the headworks complex, the hydraulic system and the powerhouse complex.

A dam and three spillway structures are the main components of the headworks complex. The dam is of rockfill type with an impervious saprolite core, 103.8 m high and 217 m long at its crest. A parapet wall, 1.2 m high, is provided along the dam crest. The surface spillway is constructed on the left abutment. The left bank spillway tunnel has its upper portion under pressure, while its lower portion, which coincide with the diversion tunnel, is of free flow. The right bank spillway tunnel is used for river diversion during dam construction, flood releasing, flushing and emptying of the reservoir. These spillway structures form a flood releasing system at high, intermediate and low levels to provide a total discharge capacity of 10,092 m^3/s (P.M.F.); The hydraulic system, located on the left bank, consists of a power intake, a power tunnel, a differential surge shaft, two penstocks, two bifurcations, a power house, an auxiliary power plant, a main transformer and switchgear room and a tailrace gate chamber, and four tailrace tunnels. The power tunnel is 8.0 m in diameter and 9,387 m in length. The surge shaft has an upper pool. The two penstocks have a diameter of 4.6 m each and the two bifurcations get into the power house with an inclination. The main power house and the auxiliary power plant, the main transformer and switchgear room and the tailrace gate chamber are parallel to each other. The main power house accumulates four turbo-generator units with an installed capacity of 150 MW each.

For the layout of the headworks complex, see Fig. 1.

2. Embankment material of the dam

A. For the cross section of the dam, see Fig. 2.

B. Impervious core material of the dam: At the initial stage of the construction, the impervious core material of the dam was previously planned to obtain from a borrow area, 13.7 Km away from the dam site, where a large amount of slope wash and residual soil of weathered dolomite is deposited. Its natural moisture content is higher than the optimum moisture content by 8.6-10% and the content of medium-sized clay particles (less than 0.005 mm) in the soil accounts for 60%. It often rains and is wet in the project area and there are only 65 sunny days in a year, which would bring about difficulties to construction of

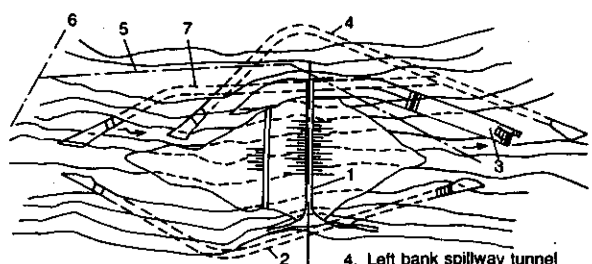

Fig. 1. A layout of the headworks complex

of the dam. So it is necessary to mix 40% sand

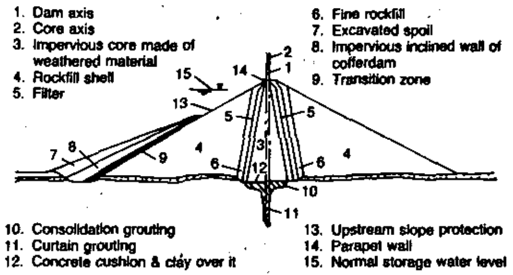

Fig. 2. A cross section of the Lubuge earth & rockfill dam

and gravels into the soil in order to improve the properties of the material and the construction conditions. Thus it can be seen that this kind of soil material needs certain treatment, which would not only be costly, but also take too much time. Furthermore, the hauling distance is long. It is not economical to use such soil as core material. After careful investigation and systematic tests done both in laboratory and at field a nearby borrow area of residual soil of weathered sandstone and shale was established as acceptable core material. It is only about 3 Km away from the damsite and has following properties:

a. The soil material used for the core includes Quaternary saprolite, 2-3 m in thickness, soil of completely weathered sandstore and shale, 3-5 m in thickness, fragments of completely weathered sandstone and shale, 2-4 m in thickness, and blocks of completely weathered sandstone and shale, 1-3 m in thickness.

b. The soil material is mineralogically composed of illite and kaolinite. The chemical analysis has shown that it contains more free silicon, aluminium and iron, and SiO_2/Al_2O_3 is 1.33-2.

c. The natural moisture content and the void ratio of the weathered blocks of the material are 20-41% and over 40% respectively, which are much higher than those of the unweathered blocks. Meanwhile their compressive strength is lower than 3.0 MPa under their moisture content of more than 10%. It indicates that the rocks are highly weathered and subject to be broken under pressure.

d. The test results have shown that the dispersion of the soil is zero. It is considered as a non-dispersive clay.

e. There is no much difference between the natural moisture content and the optimum moisture content of the soil, so no special treatment is needed. The average content of clay particles of the material is 24%. It can be well compacted by roller and convenient for the embankment construction.

f. The soil has a permeability coefficient of $i\times10^{-7}$ cm/s, a effective strength of the internal friction angle, 32^{o} and cohesion, 0.034 MPa, a compressibility coefficient of 0.014-0.027 $cm^2/10N$, which can satisfy the technical requirements for the embankment construction.

It can be seen from the above that the above-mentioned soil material is characterized by low permeability, high shear strength, low compressibility and good compactibility, and it is suitable for construction of the embankment. It is considered an ideal material for the dam embankment.

However, attention should be paid to the following problems during construction of the dam embankment:

a. Control of the gravel content in the course of compaction: The content of gravels shall be within the limit of 30-50% after compaction. The compaction test has proven that the average content of gravels is 65.7% before compaction, while it is 38.3% after compaction. The breaking ratio is 40%.

b. The soil material must be after compaction homogeneous, no seepage passage to creat due to concentration of gravels is allowed. The inspection pits excavated after the field compaction tests after mixing the materials during borrowing have shown that the filling layers are well bonded and the gravels are surrounded by fine-grained materials, and no holes were found in the compacted layers.

c. In order to increase the breaking ratio of the weathered materials, it is advisable to employ heavy roller wet compaction.

C. Filter materials:

Although the above-mentioned soil material has significant advantages, it has poor plasticity, low tensile strength (0.003-0.0078 MPa) and is liable to disintegration, which will result in cracking in the dam core. Of course, it is not easy to prevent the core of the dam from cracking, but we have to protect the core material against erosion and to make the core material be capable of self-sealing. So proper filters are required to protect the core material in case the core is cracked.

The core of the Lubuge dam is provided with two filters each on both sides to protect the core material against erosion and to transit both strength and deformation of the core. The filter material was made from crushed dolomite and limestone. A part of screened river-bed materials was also used for the filters.

A core protection test against cracking by using filters was carried out. For the test, the simulated crack in the core is 2 mm wide, 7 cm long and 15 cm deep, the max. hydraulic gradient is 100, and five gradations of filter materials were adopted. The test results have shown that the seepage discharge for each test is stable and the finer the filter material is, the less the seepage discharge is. The seepage discharge decreased with time. It indicates that the cracks are gradually self-sealed. The Pervious material of D20 less than 2.5 mm will meet the requirements for the filters.

D. Rockfill material:

Material excavated from the surface spillway and tunnels were used for rockfill. Some dolomite and limestone rocks were also quarried near the damsite. The grain size gradation of the rockfill material was determined according to the results of the calculations and the blasting tests. The material excavated from the tunnels is of fine grain size, with a content of grain particles of less than 5 mm in size accounting for about 30%, from which test samples were taken for testing. The tests reveal that its permeability coefficient is $i\times10^{-2}$ cm/s and its shear strength, the internal friction angle is 40-42°. Both of them can meet the technical specifications. Therefore, the content of the fine rockfill material with a grain size of less than 5 mm should not exceed 30%, while that of less than 0.1 mm should not be more than 5%.

After being compacted the dry density of the rockfill material should be 20.6 KN/m^3 and void ratio, 25%.

3. Analysis and calculation of the dam

Three-dimensional and two-dimensional stress strain analysis were conducted for the dam. The results of the calculations have revealed:

A. In the middle of the core, is 60% of the soil pressure applied at that point and there is an arching effect, which decreased to a certain extent after impounding.

B. is of tension in the places where both abutments are close to the river bank slopes. This indicates that cracks might occur in those places.

C. During impounding of the reservoir, the stress level (shear stress/shear strength) of the core is 0.5, which is considered safe, However, the stress level of the dam shell increases and the safety is lowered.

Seen as a whole, the designed dam cross section and the dam materials selected are suitable for such a dam. Local cracks might occur, as long as the filter material could be properly prepared and its thickness and grain size gradation could meet the required technical requirements, the dam would be safe.

4. Foundation treatment of the dam

The dam core is founded on moderately weathered rocks and a reinforced concrete cushion, 0.5-1.0 m thick, has been installed between the bedrock surface and the core of the dam in order to protect the core bottom against concentrated seepage and contact erosion. Blanket grouting was performed under the cushion to keep the foundation complete and integral. The grouting holes, 5 m deep each, were arranged in rows, 3 m apart. Under the dam foundation a grouting curtain has been provided in three rows. The maximum depth of the grouting hole in the middle one is 89 m and they are spaced at 2 m, while the depth of the holes in the two side ones, which are spaced at 3 m and 2 m from the middle one on both sides, ranges from 12 m to 15 m. In order to prevent seepage around the dam, grouting curtain extends 121 m in the left bank and 93 m in the right bank respectively. All the grouting was coducted from ground surface or grouting galleries.

5. Construction of the dam

A. 5.7 m^3 WA600 loaders, 20 t HD205 and 15 t Mitsubish dump trucks were used for borrowing and transporting the weathered materials for the core construction. Bulldozers or levelers were employed for leveling. The layer is 25 cm thick. Compaction was carried out by SPF-84 vibrating tampers, with 12 passes. Before spreading soil materials, bulldozers were used to roughen the surfaces so as to provide a good contact between two layers.

B. Filter materials: After dumping materials over the dam by dump trucks, it would be leveled with a bulldozer. The layer is 50 cm thick. A BW217D vibrating smooth roller was used for compaction, with 2 passes under static pressure.

C. Rockfill materials: CAT769C 32 t trucks were used to transport rockfill material from the stock pile areas or the quarry site to the dam, and bulldozers or hydraulic backhoes were employed for leveling material. The layer is 80-100 cm thick. Compaction was done by BW217D vibrating smooth roller, with 8 passes after watering.

6. Instrumentation

In order to monitor a safe operation of the dam and to verify the design as well as the construction quality of the dam, a number of deformation gauges, seepage pressure meters, earth pressure cells and pore water pressure cells have been installed in the dam embankment. Because the dam has not yet been under operation for a long time, the water level of the reservoir has never reached the elevation as high as the normal storage water level, the observation results obtained are quite limited. However, the analysis of the observation data collected so far has shown that the dam functions normally and all the parameters prove to be up to the design specifications.

II. THE TIANSHENGQIAO I CONCRETE FACED ROCKFILL DAM

The Tianshengqiao I hydropower project, the uppermost one of the cascade hydropower developments on the Hongshui river, is located on the Nan Pan river along the border between Guizhou Province and the Guangxi Zhuang Autonomous Region. This hydropower project is designed by Kunming Hydroelectric Investigation and Design Institute under Ministry of Energy and Ministry of Water Resources. The preliminary design of the project has already been examined and approved by the state. The tender documents are being prepared at present.

1. Introduction

The Tianshengqiao I hydropower project is the uppermost one of the hydropower developments on the Hongshui river. After completion of the dam, the water reservoir formed will have a storage capacity of 102.57×10^8 m^3, which is capable of a long-term regulation. The total installed capacity of the project will be

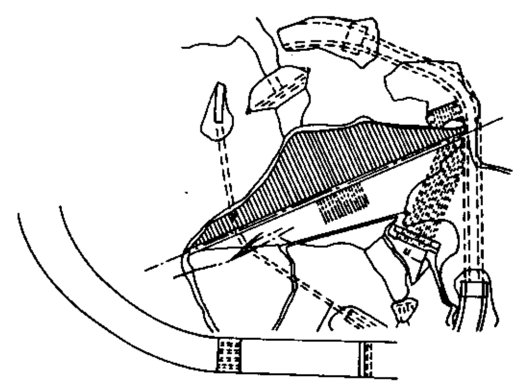

Fig. 3. A layout of the Tianshengqiao I project complex

1,200 MW and its mean annual output will be 52×10^8 KWH. In addition, the total capacity of the three constructed hydropower stations, which are located downstream of the Tianshengqiao I hydropower project will be increased by 640 MW, and their firm capacity and annual output will be increased by 880 MW and 41×10^8 KWH resptly. Therefore the construction of the Tianshengqiao I hydropower project will bring about great benefit.

The Tianshengqiao I hydropower project comprises the following major structures:

. A concrete faced rockfill dam, 178 m high and 1,137 m long at its crest;

. A surface spillway on the right bank, designed for 1,000 years frequency flood with a corresponding discharge of 15,282 m^3/s, checked with P.M.F. with a corresponding discharge of 21,750 m^3/s;

. An intermediate outlet on the right bank, which will be used for emptying the reservoir and for river diversion during construction of the project;

. A hydraulic system on the left bank, consisting of four power tunnels and four penstocks to convey water to four turbo-generators;

. A surface power house to accommodate four turbo-generators with an installed capacity of 300 MW each; and

. Two diversion tunnels in the left bank. During the first year after river closure, flood will overtop the dam, during the second year, the two diversion tunnels will meet the criterion for 300 years frequency flood. After that the flood protection criteria will be raised year by year.

It will take 7.5 years to put the first generating unit into commissioning from commencement of construction of the project. The total construction period of the project will be 9 years. For the layout of the dam complex, see Fig.3.

2. Design of the dam cross section

A. For the cross section of the dam, see Fig.4.

B. Zoning of the dam: Zoning of the dam is specified as follows:

. Zone I - impervious material, upstream of the concrete face;

. Zone II - cushion material, downstream of the concrete face; and

. Zone III - rockfill

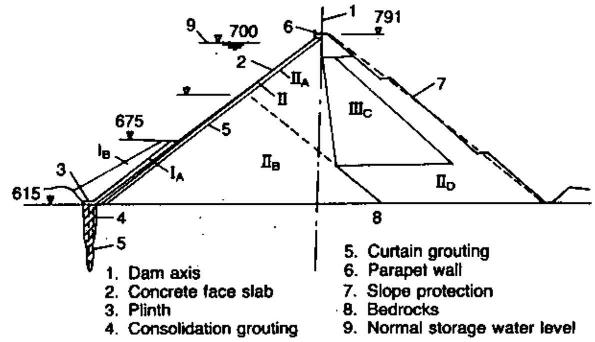

Fig. 4. A cross section of the Tianshengqiao I concrete face rockfill dam

a. Zone I: The Tianshengqiao I concrete face rockfill dam is the highest one of this kind in the world, there will be an impervious earth membrane adjacent to the bottom, upstream of the concrete face. Its height will be about one third of that of the dam embankment. The earth membrane is divided into two subzones, namely, IA and IB. The former will be provided on purpose to protect the concrete face against seepage or to seal cracks and open joints in case it cracks, and it will be made of sandy loam or silty clay or clay and directly placed against the concrete face and plinth, while the latter will be used to stabilize the former as a counter weight, and excavated spoil can be used for this subzone, which does not have special requirement for selection of material.

b. Zone II: It is a cushion under the concrete face. Provision of Zone II is to provide a reliable and smooth foundation surface to support the concrete face, to uniformly transit water pressure to the rockfill as well as to retain water before placing of the concrete face slab. The cushion will be protected with a bituminous emulsion coating on its surface. When the concrete face cracks, soil particles in Subzone IA and Zone II will not be washed away by water, thus reducing leakage of the dam. For these reasons, the material in Zone II shall be well graded and its permeability coefficient shall be 10^{-3}-10^{-4} cm/s so that it can not only function as a filter to Subzone IA, but also be capable of protecting against seepage. The width of the cushion will be determined according to the requirements for construction. Its thickness will be 3 m and can be increased at its bottom.

c. Zone III: It is a rockfill zone, which is divided into four subzones, i.e. IIIA, IIIB, IIIC and IIID. Zone III is so arranged that the compressibility and the permeability of the rockfill in all the subzones shall gradually be increased towards downstream.

Subzone IIIA: As a transition zone between Subzone IIIB and Zone II, it will prevent particles in Zone II from being taken into voids in the rockfill. So its grain size gradation shall meet the requirement for filtering. Subzone IIIA, 5 m thick, will be made of screened materials obtained from Subzone IIIB.

Subzone IIIB: It is a main rockfill zone, which will resist the water pressure applied over the dam. The magnitude of its deformation will have direct influence on deformation of of the concrete face. So it shall be well graded and drained, and also require a high compression modulus. Hard rocks excavated from the spillway or from the quarry site will be used for this zone.

Subzone IIIC: Materials excavated from the structures on the left bank and from the intermediate outlet on the right bank will be placed on the downstream side of the dam axis above the downstream water level to form this zone.

Subzone IIID: It is the most downstream part of the rockfill embankment. Its deformation has no much influence on the concrete face. It has no strict requirement for its thickness and grain size gradation. During construction of the dam, excessive size stones will be pushed to the downstream slope of the dam by bulldozers and trimmed manually to form a slope revetment, nice in appearance.

Materials in Different Zones

Zone	Material	Thick.of a layer (m)	Compaction
IA	riverbed alluvium silty & sandy soil or clay from the borrow	0.3	ordinary compaction roller
IB	random materials	0.3	ditto
II	well-graded crushed sand & gravels	0.3	vibrating roller, 4 passes horizontally, 6 passes from bottom to top over slope under static pressure
IIIA	screened well-graded coarse-grained material	0.3	vibrating roller, 4 passes
IIIB	well-graded limestone excavation from the spillway	0.9	ditto
IIIC	sandstone and mudstone or limestone excavation from the tunnels	0.9	ditto
IIID	limestone excavation from the spillway	1.8	ditto

For the gradation curves of the materials for different zones, see Fig.5.

3. Concrete face slab and plinth

The concrete face is the major impervious structure of the dam. It must be very reliable and meet the requirements for both strength and impermeability.

The thickness of the concrete face is 0.3 m on its top and then gradually increases from top to bottom as per the following formular:

$$T = 0.3 + 0.0035H$$

where T is the thickness of the concrete face and H is the vertical height below the dam crest.

The concrete face will be divided into 71 blocks along the length of the dam axis, 16 m wide each. In order to reduce the joints caused by change of temperature and let them develop in a uniform way, a layer of steel reinforcement arranged in two directions will be installed in the concrete face, with a rate of reinforcement of 0.4% in each direction. The concrete face will be concreted in two stages. The first stage concrete wil not be placed until the dam embankment has been filled as high as 112 m. The second stage concrete will be placed after the rockfill has been raised to the crest of the dam.

The width of the plinth will be one fifteenth

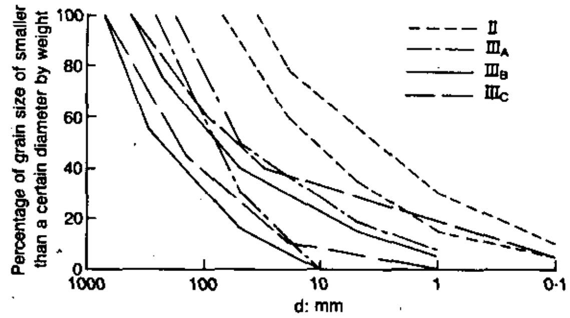

Fig. 5 Grain size gradation curves of the Tianshengqiao I dam materials

of the water head. The plinth will have three different widths, namely, 10 m, 8 m and 6 m, with their corresponding thickness of 1.0 m, 0.8 m and 0.6 m respectively. The plinth, which will be provided with two-directional reinforcement in its upper part, with a rate of reinforcement of 0.3%, will be founded over the moderately weathered rock formations, connected with its foundation with dia. 30 mm dowels, which will be driven into the foundation 3 m deep, spaced at 1.5 m. Beneath the plinth consolidation grouting to be arranged in rows, 2 m apart, will be performed, 15 m deep, with holes spaced at 2 m. In addition, a row of curtain grouting, 80 m deep, is to be installed.

200# concrete, with a permeability requirement of S8 will be used for both concrete face and plinth.

4. Joints

In order to simplify the arrangement of the concrete face, only peripheral joints and expansion joints will be provided for the Tianshengqiao I dam.

A. Peripheral joints: The peripheral joints will be installed between the plinth and the concrete face, which will be founded over the bedrocks and the rockfill respectively. The water load and the self-weight of the rockfill will result in different displacements of the peripheral joints. It is expected that the displacement of the peripheral joints will have significant influence on permeability of the dam. Having consulted the details of the peripheral joints in the Foz Do Areia dam, the Tianshengqiao I dam will also be installed with three kinds of water stops, i.e. copper water stops at the bottom of the concrete face, plastic water stops in the middle and bituminous mastic on the top, to be covered with rubber strips, which will be fixed by bolts into the concrete, so that the expansion joints could be filled by mastic in case they are opened. In the mastic, chloroprene rubber tubes will be embedded. In the joints there will be bitumen-coated wooden boards to prevent the concrete from breaking, when the concrete face is under compression.

B. Expansion joints: The expansion joints refers to the joints between two blocks of the concrete face. After impounding of the reservoir, the rockfill will be somewhat displaced and most of the concrete face will also be under pressure, only on both abutments the concrete face will result in tension due to compression of the rocks on both banks. Based on the analyses and calculations as well as the experience from other projects, 27 open joints, i.e. 17 in the right abutment and 10 in the left abutment, will be provided. Compressible joints will be installed in the middle. The compressible joints will only be provided with copper strip water stops, with a bituminous coat over the joint surface, while the open joints shall be filled with bituminous mastic on their tops in addition to the water stops.

5. Parapet wall

Based on the max. flood water level and the height of waves, a vertical parapet wall, 4.7 m high, i.e. 1.0 m above the dam crest, will be founded on the rockfill, with its upstream face connected to the concrete and with water stops inside. Provision of the parapet wall will make it possible to reduce volume of the upstream rockfill and to increase the width required for sliding formworks.

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3. The use of low grade rockfill at Roadford Dam

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Roadford Dam impounds a reservoir for South West Water Services Limited which will improve water supplies to various areas of Devon. The dam is constructed of low-grade rockfill and an asphaltic concrete membrane on its upstream face provides the waterproofing element. Site investigations with an exploratory quarry and trial embankments enabled the characteristics of the rockfill material to be examined. Minerals in the rocks may cause degradation of the fill material within the dam embankment. The results of testing and the possibility of degradation were taken into account when designing the embankment. The paper presents the results of the investigations and describes the design and construction of the embankment.

INTRODUCTION

1. Roadford Dam forms a 37000 Ml reservoir to alleviate water supply deficiencies in Plymouth, South West & North Devon. Impounding commenced in October 1989 and by Spring 1990 the water level had risen to within 6.5m of TWL where the volume stored is almost 60% of the total capacity.

2. The dam is an embankment, 41m high and 430m long and is formed of 1,000,000m^3 of low-grade rock fill obtained from a borrow quarry within the reservoir basin, about 600m upstream of the dam.

3. Waterproofing is by means of an asphaltic concrete membrane laid on the upstream face and connected to a cut off and inspection gallery structure at the upstream toe which surmounts a grout curtain up to 40m deep.

4. The overflow consists of a bellmouth spillway tower discharging into a reinforced concrete culvert constructed in the foundation of the embankment. An integral culvert provides access to the draw-off tower where pipework and valves allow water to be drawn off from three levels.

5. A comprehensive range of geotechnical instruments are built into the embankment to monitor its performance. These include piezometers in the foundation and embankment, vertical settlement and horizontal extensometers, pressure gauges above structures and electrolevels to measure settlement of the asphalt membrane.

INVESTIGATIONS

6. Site investigations were undertaken in 1975 and 1977 which confirmed that the site was suitable for the construction of a dam and enabled preliminary designs to be prepared for presentation at a Public Inquiry. In fact, the Public Inquiry was re-opened on a number of occasions between 1978 and 1983 to consider various topics before the water Order was finally authorised.

7. A more detailed investigation commenced late in 1983 which included trial pits, boreholes and seismic surveys together with the excavation of a trial quarry and the construction of trial embankments. Assessments of the physical properties of the fill were made by both laboratory and in situ tests and the chemical properties were also investigated.

8. When the exploratory quarry was opened the material was classified into three types relating their depth below ground level and weathering characteristics. Laboratory tests comprised index property determinations, particle size analyses, unconfined compression tests on cored rock samples, rock soundness tests, compaction tests to determine optimum moisture content, permeability testing, direct shear in a 300mm x 300mm shear box, assessment of shear strength by triaxial compression apparatus, consolidation testing by Rowe cell and X-ray diffraction. In situ tests included density by sand and gravel replacement methods. Permeability was measured by soakaway and well permeameter tests, both analysed by methods described in the USBR Earth Manual.

9. The geology at the dam site and borrow quarry consists of sedimentary rocks of the Crackington Formation, part of the Culm Measures of the upper Carboniferous age. The rocks comprise rapidly alternating mudstones, siltstones and sandstones which have been considerably folded and even overfolded. Faulting has also occurred and shear deformation is common, giving a closely fractured structure. Near the surface weathering is extensive in the mudstones to give a considerable depth of clay type material, but above the sandstone there is a comparatively thin layer of gravel and cobbles.

10. A review of the results indicated:-
i) that the most weathered material would not be suitable to form a core within the embankment.
ii) an embankment could be constructed of the less weathered materials obtained from depths greater than about 4m below original ground level.

11. In addition to standard tests carried out on small size samples, large scale tests were carried out at the Building Research Establishment to determine the properties of larger samples of rockfill. These included investigations of compressibility in 152mm and 1m diameter oedometers (see Fig. 1), strength by drained triaxial compression tests in 230mm diameter apparatus (see Fig. 2) and the influence of fines on permeability using a 150mm diameter permeameter.

CHEMICAL DEGRADATION

12. Early in the investigations it was noted that the rockfill, in particular the mudstone fraction, contained minerals which are prone to degradation by chemical weathering. The principal mineral components of quartz and silicates are cemented into a solid matrix by iron sulphide (pyrite), iron bearing carbonate (siderite) organic carbon and occasionally calcite. The most vulnerable of these is the iron sulphide. Mudstone samples from the borrow quarry were found to have a mean pyrite content of almost 1% by weight.

13. Weathering of the sulphides in an oxidising environment will produce acidity and increased sulphate content. The acidic conditions may attack and alter clay minerals and leach out elements. Ultimately the strength of the rock may be affected to such an extent that calcareous mudstone may be reduced to a clay or calcareous sandstone reduced to sand.

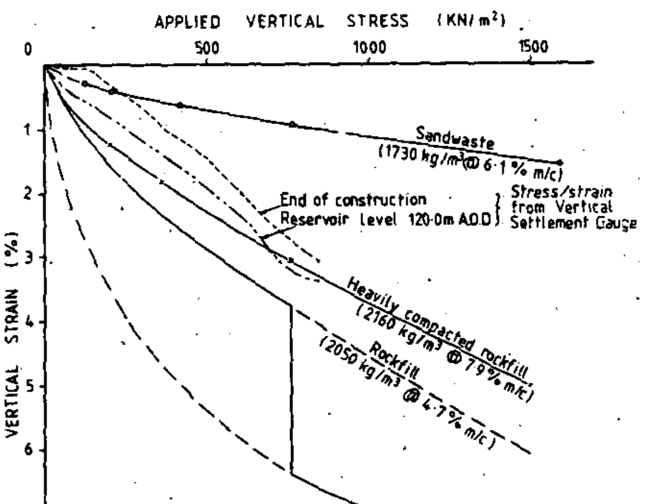


Fig 1 Large oedometer test results

14. The rate at which these processes proceed is the subject of much debate. Estimates ranging from a geological time scale, apart from the few metres near the surface, to only a few years, if conditions within the embankment permit vigorous activity of various sulphur reducing bacteria. The evidence from local conditions and from embankments formed from similar materials elsewhere indicates that degradation is likely to proceed at a rate between these two extremes. Degradation may affect the properties of the dam fill material and make the water draining from the dam unsuitable for discharge to the unpolluted River Wolf.

15. Analyses and tests have shown that it is mainly the fresher mudstone rocks within the dam fill material which are most vulnerable to long continued degradation, the more weathered rocks having a considerably lower potential for such degradation. As the completely weathered residual soils overlying the site represent the climax of the weathering process they will have similar properties to the rockfill in the dam embankment if it degrades completely. The results of tests on the most weathered material gives a measure of these properties. These indicate, perhaps surprisingly, that the material is at least as strong as the rockfill obtained from greater depths in the borrow quarry.

EMBANKMENT DESIGN

16. The stability of the dam has been assessed for a number of design conditions including end of construction, in-service and rapid draw-down. The effect of earthquake forces of 0.05 g in addition to the first two cases was also considered. The analyses were carried out using the GEOCOMP-SLIPSYST program for both circular and non-circular failures. In undertaking the analyses the influence of a variety of possible configurations of planes of weakness in the foundation were considered. A number of different face slopes were examined within the range 1:2.0 to 1:2.5. Satisfactory factors

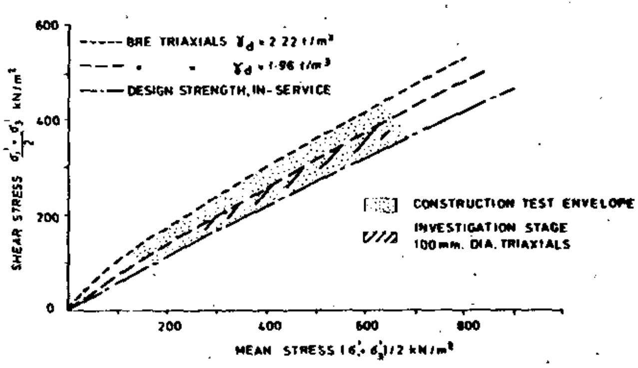


Fig 2 Drained triaxial test results

of safety were found for both upstream and downstream faces having slopes of 1:2.25 and these are the slopes to which the embankment has been constructed, except at the extreme downstream toe where it slackens to 1:2.5.

17. Roadford is considered to be a homogeneous dam. The strength parameters used in the design reflect the properties appropriate to the material type, the confining pressure within the embankment and its state of degradation as shown in Table 1. At the end of construction the fill material was considered to be undegraded with a shear strength depending on the confining pressure within the body of the dam. Under the in-service condition it is assumed that the fill material is saturated and degraded.

Table 1 Design shear strengths

Design Case	Ø' deg	c' kN/m^2	Confining Pressure kN/m^2
End of Construction	40	0	0 - 150
	32	31	> 150
In service	35	0	0 - 230
	29.5	31	> 230

18. The permeability of the rockfill in the embankment at the end of construction was expected to lie within the range of 1 x 10^{-5} to 1 x 10^{-7} m/s. No constructional pore pressures were anticipated.

EMBANKMENT CONSTRUCTION

Programme

19. The £16.1m contract for the dam was let to Alfred McAlpine Construction Ltd. in February 1987. The construction period was 3½ years with the proviso that the dam be sufficiently far advanced for impounding to commence before the winter of 1989. The priority during the first season was the construction of the overflow culvert as it was to be used for the temporary diversion of the River Wolf. Work also proceeded on the cut-off and grout curtain and other structures. Eventually over 50,000 m^3 of concrete was placed from the site plant.

20. Exactly a year after work began on site the river diversion was achieved in March 1988 which allowed the embankment foundations to be prepared and the drainage layer placed in the valley bottom. Embankment construction commenced in June and 1,000,000m^3 of rockfill material was placed in a 5 month period. Also during this season the 30m high valve tower and the 15m high parallel sided section of the overflow tower were constructed by slipforming techniques during a 10 day continuous working period.

21. After the completion of the embankment at the end of October the filters and drainage layers were placed on the upstream face during the winter period. Dry and warm weather allowed the 25,000m^2 of site mixed asphaltic concrete membrane to be placed in an eight week period during the Spring of 1989.

22. By the end of July 1989, three months ahead of target, the dam was ready for impounding. However at that time the flow in the river was below the 9 Ml/d compensation water requirement and it was not until the drought was broken on 20th October that impounding was able to commence. During the intervening period most of the outstanding work was finished and the completion certificate was issued at the same time as impounding, nine months ahead of contract completion date.

Prevention of Pollution

23. As a result of commitments made to the downstream riparian owners, the contract stipulated that no pollution of the River Wolf should occur as the result of dam construction activities. To achieve this, the contractor constructed a series of settling lagoons into which all water from excavations and construction operation was directed.

Constructional Aspects

24. The specification required that material to be used for the embankment would be derived from rock as defined in Table 10 of BS 5930 as "Fresh", "Slightly Weathered" or "Moderately Weathered". The weathering grade was to be applied to the rock mass, based on observation of about 100m^2 of face, and not to the individual layers within the mass.

25. To ensure that the embankment would not have areas of uncontrolled ranges of permeability, two alternative approaches were adopted (i) material with different permeability properties should be placed in different locations within the dam or (ii) blending should be carried out so that a uniform material could be placed at any location. If option (i) were adopted it was desirable to expose the full range of strata in the quarry at the earliest opportunity so that both the more and less weathered material could be worked simultaneously and directed to the most appropriate part of the

embankment. However it was always accepted that whatever method was adopted the more weathered material from the top of the quarry would form the lower part of the dam embankment.

26. When construction commenced the contractor proposed that he would select materials from the quarry and direct them to the appropriate parts of the embankment. Excavation commenced from benches following the hillside contours which resulted in some difficulty in identifying the top of suitable material and in a preponderance of weathered strata. The use of the BS 5930 definitions also gave rise to some discussion regarding the meaning of 'soil' within the moderately weathered description. In addition the weather susceptibility of the weathered material gave rise to concern that fill with properties poorer than had been assumed in the design may be incorporated in the structure.

Operation of the Borrow Quarry

27. Excavation was initially carried out by face shovel loaders, assisted by a ripper, but these became less effective as the less weathered and harder rocks were exposed. To keep up output an increasing number of more powerful rippers were introduced and the material pushed towards the loaders. This method of working produced a well blended and consistent material and very little direction to different locations in the embankment was necessary. At the peak of operations there were Komatsu 455, 355 and 155 and Caterpillar D9 rippers with two attendant Komatsu 155 blades. The loading was done by two Caterpillar 245 face shovels and Caterpillar 245 and 235 and Ackerman backacters.

28. Although the material appeared consistent, regular estimates were made of the rock types within the blend because of the effect a large imbalance in the proportion of either mudstone or sandstone may have had on permeability. These examinations revealed some significant variations but overall the ratio of sandstone to mudstone was 50:50. The sandstone strength was measured by point load tests which showed that over 75% was 'very' or 'extremely' strong with strengths in excess of 100 MN/m^2.

29. As the quarry developed, a large and extremely hard outcrop of sandstone was revealed in the centre of the area. Excavation of this material would have been very difficult and would have yielded material significantly different in character to the fill that had already been produced. It was therefore decided on both economic and technical grounds to work around the outcrop and extend the quarry to the east.

30. From the quarry workings, samples were tested regularly for specific gravity, moisture content and grading. The fill was generally drier than expected with the overall average moisture content being only 4.4% with the range between 0.6% and 13.4%. After ripping and loading the maximum size of rocks was rarely greater than 0.5m but no gradings of the full range of particle sizes was undertaken. From the grading of the fraction below 106mm it was concluded that the fill had a slight excess of fines which would give maximum stability and strength with minimum compressibility.

31. On completion, the embankment fill will be protected by the upstream membrane, the crest road and the grassing and drainage of the downstream face. This configuration will reduce ingress of air and water thus reducing the rate of chemical degradation by the oxidation of pyrites.

32. Regular samples from the fill indicated an average sulphur content of 0.53%, although one test on fresh mudstone gave a value of 2.46%. Tests carried out using BS 1377 Test 11(A) showed the fill to be consistently acidic with the pH in the range between 4.33 and 6.60. X-ray diffraction and X-ray fluorescence tests confirmed that minerals susceptible to significant decay were only present in small quantities.

Embankment Compaction

33. After commencement of the contract, but prior to embankment construction, further trials were carried out to investigate the effect of differing layer thicknesses, numbers of passes and types of roller. The trials were carried out on a 'more weathered' material (predominantly sandstone) and a 'less weathered' material (predominantly mudstone). The trial showed that the permeability was at a slightly higher value than had been assumed, also that a layer of fine material was formed after compaction of each layer and the addition of water caused this surface to become slurried.

34. Compaction of the rockfill was to a method specification which required a 450 mm finished thickness layer to be compacted by 8 passes of a roller. Should it be necessary to change the number of passes or the layer thickness a formula was provided to enable any increase or decrease in cost to be calculated. To achieve the required standard the contractor provided three smooth drum vibratory rollers, an ABG MAW 173 of 4800 kg/m and two Bomag BW 10s of 5350 kg/m which were towed by Caterpillar D6 bulldozers.

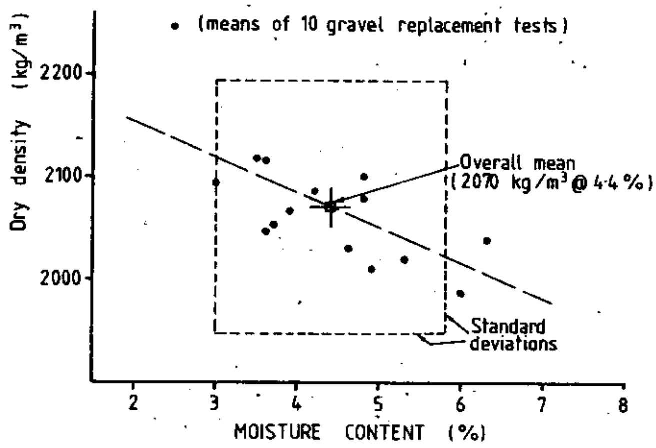
35. Haulage from the quarry was undertaken by a fleet of 12-15 Caterpillar D35 articulated dump trucks. The fill was tipped about 3m back from the advancing face of the layer and bulldozed forward over the edge to assist the distribution of fines within the mix. It was noted that the tracks of the machines caused an additional breakdown of the rock which further increased the fines content. Prior to placing a new layer, the surface was scarified using a multi-tine ripper to break up any 'crust' formed by rolling.

Testing of Embankment Rockfill

36. During embankment construction holes, 800mm cube, were excavated in the fill to obtain the density by gravel replacement methods. Initially the results indicated lower densities and higher air voids then might have been expected and the number of roller passes was increased from 8 to 12. This produced only about 1% improvement in density and trials were carried out to establish the point of 'refusal'. Levelling of layers of rock after each pass of the roller showed that very little compaction occurred after the 8th pass; beyond this the material started to breakdown and a 'bow wave' sometimes appeared ahead of the roller. It was concluded that 8 passes was sufficient to achieve a rockfill compacted virtually to refusal.

37. There were 149 test holes excavated, giving a rate of one hole per 6650m^3 of fill. The mean bulk density was 2159 kg/m^3 with a range between 2675 and 1820 kg/m^3 giving a dry density of 2070 kg/m^3 at an average moisture content of 4.4%. Contrary to what might have been expected it was found that the higher densities occurred with lower moisture content as is illustrated in Figure 3.

38. It was expected that the material would be highly weather susceptible but only on a few occasions was it necessary to stop placing during or after periods of heavy rain. However the weather caused material churned up by the wheels and tracks of construction plant to become a 'sticky slurry'. It was thought that this could represent an extreme condition such as may result if severe degradation of the rockfill occurred. A shear box test gave particular confidence with results ϕ' of 29° (peak) and of 25.5° (residual) for material with a moisture content of 18.5%.



EMBANKMENT ROCKFILL

Fig 3 Dry density/moisture content relationship

39. Over the period of construction routine samples of the fill were taken for triaxial tests and large shear box tests. These tests were carried out on the fraction of material below 20 mm and below 37.5 mm and therefore underestimated the strength of the actual material in the embankment. In general the results from the triaxial tests approximated closely to those obtained in the initial investigations but those from the shear box were lower by about 5%. It was concluded that the reason for this was that the particle size of 37.5mm was too great for the 100mm thick specimens resulting in unrepresentative density and compaction.

40. On completion of density measurements, the 800mm cube holes were filled with water and a falling head permeability test carried out. The permeability was calculated from the soakaway formula given in C I R I A report No.113, modified to allow for the gravel in the hole and for differing hydraulic gradients in the sides and base of the hole. The average permeability obtained was 1.5×10^{-4} m/s with the range between 1.8×10^{-3} m/s and 2.8×10^{-7} m/s. Thus the fill is within the range assumed in the design. In the event of severe damage to the membrane, most of the water will drain through the fill to the underdrainage layer. There will be only a modest downstream flow through the fill material and the embankment will remain stable under these conditions.

Collapse Settlement

41. There was some concern that the placing of comparatively dry mudstone within the fill could result in collapse settlement. This occurs when the point contact between individual particles crush and compress due

to weakening of the rock by wetting. In addition dry rockfill has a strength contribution from pore water suction which disappears when it becomes wet. To investigate the susceptibility of the rock to collapse compression, tests were carried out with the Building Research Establishment's 1m diameter oedometer. Samples of 20 mm down fill were heavily compacted and inundated while subject to a vertical stress. The results indicated that collapse compression should not be a major problem because a high standard of compaction was being achieved on site. Also in spite of the low moisture content of the fill it was considered that the addition of water during placing would not greatly reduce the collapse settlement unless sufficient was added to almost fully saturate the rockfill.

Sandwaste

42. China clay sand waste was used downstream of the concrete cut off to minimise differential settlement under the asphalt concrete membrane. This was the same material that had been used for the construction of Colliford Dam (ref. 1) where it was found that settlements were minimal. The properties of the sand were confirmed by a test in the BRE 1m diameter oedometer where it was found to be one third the compressibility of the rockfill. (See Fig. 1) As this test was at a density below that actually achieved in the field, the compressibility should be an underestimate of its true value.

43. A total of about 40,000m^3 of sand was delivered by road from Parsons Park Pit and compacted in 250mm layers using a towed vibrating roller of 3400 kg/m width. Initially 12 passes were used, based on plant trials, but this was increased to 16 passes after the densities showed a high variability. The routine sand replacement tests gave a final mean density of 1842 kg/m^3 at an average moisture content of 6.7%. This was about 2% less than the mean of 1880 kg/m^3 achieved at Colliford, probably because of different compaction plant operating in comparatively small areas.

Finite Element Analysis

44. A number of finite element analyses were carried out by Professor P R Vaughan to confirm the likely behaviour of the embankment at the interfaces between the gallery, sandwaste and rockfill. The properties assumed for the materials were those obtained from the site investigations and used in the design calculations. In practice those actually achieved during construction were generally found to be superior. The analyses confirmed that there were no zones of high stress which could lead to local and perhaps progressive failure and also that stress levels would be reduced following impounding. The incorporation of the sand waste zone has a major influence in reducing differential movements adjacent to the cut off. In fact the displacements between the embankment fill and the sandwaste were shown to be greater than at the interface with the cut-off structure.

45. The analyses also provided predictions of the likely settlement at the locations of the various instruments installed in the embankment. During construction the settlement measured in the embankment correlated closely with the finite element analysis. (see Fig 4) During a period of heavy rain in February/March 1989 some evidence of collapse compression was noted consistent with water percolating downwards through the fill. The observations have shown the compressibility of the fill to be better than the oedometer tests on a densely compacted fill (see Fig. 1)

46. Horizontal movements indicating a spread of about 80 mm were recorded within the embankment by the end of construction. This was about half that predicted by the finite element analysis. Surveys of the downstream face of the dam have confirmed the movements indicated by deformation instrumentation installed in the dam.

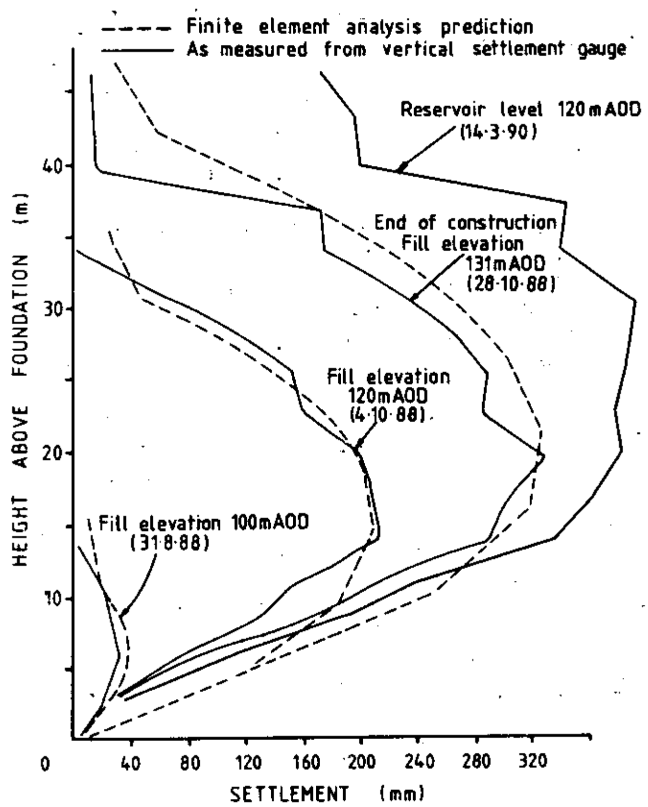


Fig 4 Settlements on embankment centre-line

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4. A perspective of the art of the embankment dam in South West Asia

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Synopsis

Binnie and Partners with their associated firms in Malaysia and Hong Kong have been responsible for a large number of embankment dams in range of height up to 70 m formed of residual soil on weathered rock foundation. Some 20 such dams have been completed in the last decade and have been put into service with no significant problem arising either during the construction or the first filling phase. As a result of this rather satisfactory situation, little or nothing has been published on the techniques that have evolved into a standard form of practice. At present there are 5 major dams in the final stages of detail design or under construction in Malaysia alone, the highest of these being the Upper Muar dam in Negeri Sembilan with a height of 55 m. The particular aspect of interest to dam engineers is the behaviour of the residual soils when placed in humid tropical conditions, usually at moisture contents well in excess of optimum. The difficulty of prediction of construction pore pressures because of the nature of the tropically weathered residual soil and the possibility that post-placement chemical bonding of the clay particles may effect the prototype pore pressure response. The evolution of the design has resulted from the observation of well instrumented dams over the decade. Working practices relating to the inclusion of drainage blankets, the allowable rate of increase in height of the fill have evolved on the same basis. The foundation conditions for most of these dams consist of rock weathered to various states and the appropriate treatment of the foundation to avoid excavation of large amounts of in-place residual soil is fundamental to the economic development of the dam sites. This matter will be dealt with fully in the paper, being a combination of cutoff construction and grouting the different zones of weathered rock in place.

INTRODUCTION

1. The residual soul derived from tropical weathering of the rock mass in situ provides a source of excellent material for the construction of embankment dams. By far the most common soil is the product of weathered granite but the extrusive volcanics such as the rhyolites found in Hong Kong, and sedimentary will also yield soil albeit more variable in mechanical properties.

2. The process of weathering and the erosion of the product by rapidly downcutting immature streams results in sites having fresh rock exposed in the stream bed and depths of weathering increasing in the abutments. Such a profile is ideal for the earth embankment dam which can accommodate the variable deformability of the foundation.

3. The strength characteristics of the soil permit the use of conventional central core dams having relatively economical external slopes, typically on 3.5 upstream and 1 on 3 downstream. Thus the dam will have only 45% more fill volume than a typical rockfill dam with central core and the unit cost of the fill will be significantly less.

4. Other favourable features are the proximity of the borrow pits to the dam, the usual absence of deep alluvium at such sites and the proximity of fresh rock in or near the thalweg. This permits economical arrangements of draw-off and diversion culverts or tunnels.

5. There is usually a range of weathering which provides by selection suitable fill for the zones of the earth dam; more clayey soils for the core and more sandy soils for the outer shells. The sand and gravel materials required for filters and drainage are not normally available from river bed alluvium and are commonly obtained by crushing quarried rock on site.

CLASSIFICATION OF WEATHERING

6. There are various schemes of weathering classification which describe the weathering of the rock mass in relation to the deterioration of the rock crystals and the effect of weathering on the discontinuities and boundaries of the rock mass. The state of weathering is usually subdivided in grades from fresh rock (Grade I) to fully decomposed rock termed residual soil (Grade VI) and follow loosely upon the work of Moy (1955).

7. Table 1 is drawn from BS 5930 1981 with an additional description column drawn from Binnie and Partners' standard specification which gives some added practical aids to identification.

8. In large cuts in weathered granite it may be possible to identify the whole range of weathering from the surface downwards to fresh rock. It may be equally common to go straight from Grade V to Grade I in certain

Table 1. Scale of weathering grades of rock mass

Term	Description	Grade	B&P Specification
Fresh	No visible sign of rock material weathering; perhaps slight discoloration on major discontinuity surfaces	I	Strong, hard rock; cannot be scored with a hand knife; and felspars fresh
Slightly weathered	Discoloration indicates weathering of rock material and discontinuity surfaces. All the rock material may be discoloured by weathering	II	Strength approaches that of fresh rock; sharp edges cannot be pared but may be scored, with difficulty, with a hand knife; ground mass of originally dark coloured volcanic rocks bleached to reddish brown or pale grey and plagioclase felspars may be decomposed
Moderately weathered	Less than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a continuous framework or as corestones	III	Pieces the size of NX drill core cannot be broken by hand; sharp edges of fine grained rocks can be pared and the rock scored with a hand knife; bleached to pale brown or white and felspars considerably decomposed
Highly weathered	More than half of the rock material is decomposed or disintegrated to a soil. Fresh or discoloured rock is present either as a discontinuous framework or as corestones	IV	Pieces the size of NX drill core can be broken and crumbled in the hands, fine grained rocks can easily be pared with a hand knife; does not disintegrate when soaked in water and can sometimes be recovered as cores by careful diamond drilling, using water circulation, but is often lost; bleached or stained yellowish reddish-brown
Completely weathered	All rock material is decomposed and/or disintegrated to soil. The original mass structure is still largely intact	V	Still has a recognisable rock texture; the original felspars are decomposed to clay minerals; it will disintegrate when immersed in water and often cannot be recovered as cores by normal diamond drilling methods. Pale or yellowish reddish-brown clayey-silt
Residual+ soil	All rock material is converted to soil. The mass structure and material fabric are destroyed. There is a large change in volume, but the soil has not been significantly transported	VI	Completely decomposed by weathering in situ and with rock texture destroyed; red or yellow clayey-silt containing over 20% clay-size fraction; the sand content will depend on the original proportion of quartz-phenocrysts

+ Residual soil is used throughout the paper as a term to describe the excavated fill from other lower grades.

Table 2. Soil Properties

Dam	Zone	Average values: Proctor compaction Dry density g/cm^3	Optimum m.c. %	In-situ Dry density g/cm^3	m.c. %	Proportion of fine material	Design values: c' kPa	φ'	k m/s	c_v m^2/year	LL %	PL %
Ayer Itam	Core	1.64	21	1.58	22	< 0.002mm 34%	21.5 (21.5)	30° (30°-35°)	1×10^{-9}	7.8 (2.2-8.9)	71	37
	Shoulders	1.77	15.5	1.72	16	< 0.06mm 16-35%	9.6 (0-40.7)	35° (32°-43°)	5×10^{-8}	22.3 (11-670)	51	28
	Foundation	-	-	1.59	20	-	9.6 (4.8-28.7)	35° (30°-39°)	c.1×10^{-7}	-	-	-
Shek Pik	Core	1.75	16	1.68	18.5)	< 0.002mm	24.0	31°	c.2×10^{-9}	22.3	38	26
	Shoulders	1.81	15	1.73	16.5)	6-48%			-	(22.3)	33	22
Lower Shing Mun	Core	1.72	18.5	1.64	21	< 0.002mm 13-28%	0 (26.3)	30° (33°)	c.1×10^{-8})	(c.130)	57	32
	Shoulders	1.73	18	1.67	17.5	< 0.002mm 2-22%	0 (30.6)	35° (36°)	c. 1×10^{-7})		43	32
Jor	Core	1.75	17	1.71	19	< 0.002mm 22%	12 (24)	31° (32°)	7×10^{-10}	84	65	34
	Shoulders	1.75	16	1.70	18.5	< 0.002mm 11%	14 (19)	31° (33°)	6×10^{-9}	22.3	43	33
Seletar	Core (colluvium)			1.84			19	29°		4.2		
	Shoulders			1.84			14	31°		11		
	Foundation			1.76-1.84			14-19	28°-30°		46		
Plover Cove Main dam	Core (placed)	-	-	1.70	18.5	< 0.002mm 13%	0	30	1×10^{-6}	11	31	22
	Shoulders (dumped)	-	-	1.78	16.3	< 0.002mm 7%	0	35	1×10^{-6}	11	31	22
Upper Peirce												
Langat	Core Shoulders	1.54	24								62	33
Semenyih	Core					< 0.06mm 28-73%	0	29				
	Shoulders					< 0.06mm 7-61%	0	32.5				
	Foundation					-	0	32.5				

* Values in parentheses are measurement averages on fill samples or field tests

circumstances and to find all grades of weathering juxtaposed for no obvious reason.

SOIL IN PLACE

Foundations for embankment dams

9. Where granitic rocks form the foundation of the dam the weathering profile will usually show considerable variations in depth with the greatest depth of weathering on the abutments where the embankment loading is least.

10. In very general terms undisturbed residual soil has relatively high shear strength and low compressibility, thus the foundations are not generally a problem for the design but careful exploration is needed to determine variations which may lead to locally higher construction pore pressures in the foundation and differential settlement of the embankment.

11. Foundation exploration by drillholes can be difficult. It is vital to distinguish between weathered rock in place and material which has been displaced by slope instability. Displace granite core boulders up to 7.8 m diameter have been recorded at one dam site. Bedrock in place must be proved by good core recovery in Grade I to II rock to a depth of at least 10 m from the surface of the rock.

12. The foundation exploration in the weathered zone is made using shell and auger soft ground boring equipment and obtaining U100 samples. It is difficult to make satisfactory core recover even with triple tube core barrels in Grave V and Grade VI weathered granite. The boring is stopped when penetration cannot be achieved without chiselling and the hole is continued with rotary diamond drilling equipment in H or N size. Even then it is difficult to achieve a satisfactory coring result in Grade III rock, although this is the zone which is critical for foundation permeability.

13. The use of geophysical exploration to infill detail between trial boreholes and drillholes has been disappointing in its application to weathered rock profiles, probably because the seismic velocity are too severe.

Soil properties

14. The range of mechanical properties for residual soils varies between the Grade V to VI clay residual soil and the Grade III sandy soil. In the former case there will have been significant loss of the constituents of the parent rock and the residue will be a relatively dense clay soil with a plasticity index in excess of 20. In the case of the Grade III soil there will be a relatively higher strength and stiffness, consolidation being rapid due to the residual fissuring probably resulting from the joint system in the original rock mass. In either case the residual soil behaves as a normally consolidated clay but chemical bonding may explain strengths generally higher than that of normally consolidated sandy silty clays.

15. It is difficult to give typical values of foundation strengths or settlement characteristics because of the varied weathering profile and the difficulty of obtaining representative samples of adequate size to represent the mass behaviour of the soil.

16. Typical soul properties are given in Table 2. Shear strength parameters adopted for design are typically $\phi' = 32°$ $c' = 12$ kPa. For consolidation calculations c_v values vary between 20 and 150 m^2/year and a usual value adopted as a conservative design value is 50 m^2/year. Measured settlements of the foundation under dams up to 60 m in height have been under 200 mm. The foundation consolidation is relatively rapid. A check would normally be run with $c' = 0$ for deep surfaces.

17. Construction pore pressures are not normally a problem. For design purposes r_u values of 0.2 are normally assumed but these are rarely reached in practice. Most dams show little or no post construction foundation settlement.

Reduction of permeability - Cutoff or grouting

18. As recently as 1965 the 57 m high Lower Shing Mun dam, Hong Kong, was provided with a 3 row grout curtain in fresh rock in the river bed and a 2 m wide concrete filled cutoff trench in the residual soil in the abutments. This cutoff trench which extended to a depth of 36 m at the abutments provided an ideal positive cutoff, particularly in the transition zone between the more clay weathered rock (Grade IV) and the fresh rock (Grade I to II). It would now be regarded as prohibitively expensive and would not normally be employed.

19. Although the permeability of most residual soil is low, of the order of k 10^{-5} m/s, there are zones of higher permeability particularly in the transition between the Grade III and the fresh rock Grade I or II. As the foundation is often a weak point in the dam, it must be treated to improve its resistance to erosion and reduce the seepage losses by a combination of grouting and proper drainage provisions in the downstream area of the dam.

20. Thus there are two problems in the design of an appropriate grouting scheme. To treat the fresh rock by reduction of the fracture permeability requiring a difficult technique. No scheme for foundation grouting can be considered complete at the design stage, the detail of the geological anomalies will be revealed as the drilling and grouting progresses and the site work must therefore be controlled by those responsible for the design.

21. The most critical area of any foundation grouting is the shallow contact area immediately below the core. At shallow depth the rock fissures are open, the possibility to develop high grouting pressures is least and on completion the hydraulic gradient is at its greatest. Where the foundation of the core area is predominantly rock and is highly fractured, it is desirable to place a concrete grout cap or sprayed concrete layer on the core area. This performs two essential functions in

addition to providing a good working surface for the blanket grouting operations:

(i) By preventing surface leakage it allows the development of reasonable grouting pressures in the near-surface fissures. Even a sprayed concrete layer 6 cm thick can resist the uplift developed by the grout in relatively small fissures.

(ii) It isolates the erodible base of the core from the high velocity flow which may develop in isolated ungrouted near-surface fissures.

22. Where the rock surface is massive but with occasional fractures or fault zones the surface can be treated by slush grouting and occasional dental concrete. Where the core area is on weathered rock (Grade III) it is impossible to treat the surface in this manner. Nonetheless the core contact area will be a critical zone for permeability. In such cases a minimum of 6 m of core fill is placed over the surface before grout injection of the core contact area and the fresh rock below.

23. The treatment of this shallow zone requires multiple rows, usually 3 or 5 rows of grout holes with a nominal spacing of 3 m. In the abutments where the height of dam is reduced and the core width narrowed grouting can be reduced to 3 rows and the extension above into the abutments beyond the end of the dam is usually continued as a single row curtain.

24. Most rock foundations are treated by cement grouting at depth. This usually takes the form of a 3 row grout curtain. The depth of the grout curtain should be determined by the rock conditions at depth. It is often required that the Lugeon values should be less than 5 and that the profile of rock permeability should be determined by cored investigation holes taken to a depth at least equal to the height of the dam. If Lugeon values are as low as 5 or below, there may be little or no grout take and if the dam can be designed to eliminate risk of damage due to seepage there may be a case for eliminating the grout curtain altogether. However a grout curtain is an effective tool for detailed examination of rock foundation and it should not be omitted without good reason. Normal cement grout can be employed to grout rock fissures down to a width of less the 0.5 mm, the lower limit is usually taken to be about 0.2 mm

25. In formulating an appropriate grout for sealing fine fissures a considerable difference of opinion has arisen amongst various experts. On the one hand there is the need to ensure penetration of fine fissures. This is governed by formulating a grout with sufficiently low viscosity but which will not have too high a percentage of bleed, ie separation of the water from the grout in suspension. On the other hand, the grout must be stable in the fissures with good durability.

26. Cement grout is a suspension of finely ground particles of cement in water. In fine fractures the cement particles will tend not to enter the fracture and the larger particles will act as filters for the smaller particles. It has been suggested that the fracture size should be three times the diameter of the maximum particle size in the cement grout. For most cements this would lead to a minimum fracture width of around 0.3 mm.

27. It is important to formulate a cement grout which is adequately durable. Clearly, thick grouts with a water/cement ratio of 2:1 or 3:1 will have a low proportion of bleed water and will form strong stable grouts.

28. In the past, very much thinner grouts were used down to water/cement ratios of 12:1 or even thinner for good penetration of fine fissures but in certain cases that have been quoted recently, grout curtains have seriously deteriorated and it is now commonly accepted that thin grouts may not be stable.

29. To minimise the percentage of bleed water from the grout it has been the practice for some years, particularly in Europe, to add bentonite to the mix. This in its turn has some disadvantages in increasing the viscosity of the mix and thereby reducing its penetration of the fissures.

30. It would now seem that good practice is to go for a relatively thick grout mix less that 5:1 water/cement ratio and in certain cases to add bentonite equivalent to 2.5% of the cement by weight to improve the ability of the mix to keep the cement in suspension.

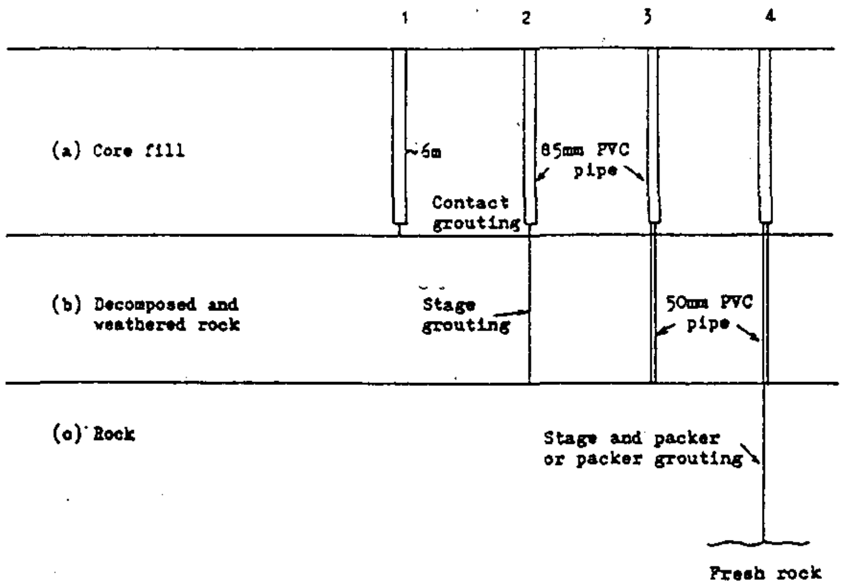
31. The grouting of the Grade III weathered rock beneath the layer of core fill in place is usually performed as stage grouting with a packer set in an 85 mm PVC pipe taken through the fill and grouted into the surface of the weathered rock. Stages of this process are illustrated in the accompanying diagram (Figure 1.).

32. The grout mix is progressively thickened from 5:1 to 1:1 injecting 2-3 batches at each step and then proceeding to the next thicker mix until refusal is achieved. Batch volumes are given in Table 3 - Grout mixes.

33. In conclusion, it can be stated that a properly constructed grout curtain must be based on the determination of insitu permeability and the detailed probing of the geological situation as the grout injection programme proceeds. The depth of the grout curtain should be determined by the permeability conditions rather than a fixed depth. The grout curtain should be composed of at least three rows of grout holes to create a significant thickness of grout curtain.

<u>Grouting pressure</u>

34. Grout pressures are controlled to prevent hydraulic fracture or uplift. In the grouting of the overburden or weathered rock, pressure at the injection gauge is controlled at 6 kN/m^2 per metre depth. Taking account of the grout column to the stage being injected, this ensures that the total grout pressure does not exceed 85% of the nominal overburden pressure at this point. In certain circumstances the minor principal stress can be less than this pressure and it is possible that



1. Drill and install 85 mm p.v.c. pipes in (a) drill into (b) and grout contact of (a) and (b).
2. Drill to rock and stage grout (b).
3. Drill and install 50 mm p.v.c. pipe in (b).
4. Drill into (c) in 3m length, stage and packer grouting in 3m length to fresh rock, or packer grouting in 3m length from fresh rock when rock is good.

Where core fill in direct contact with rock procedure (2) and (3) were omitted.

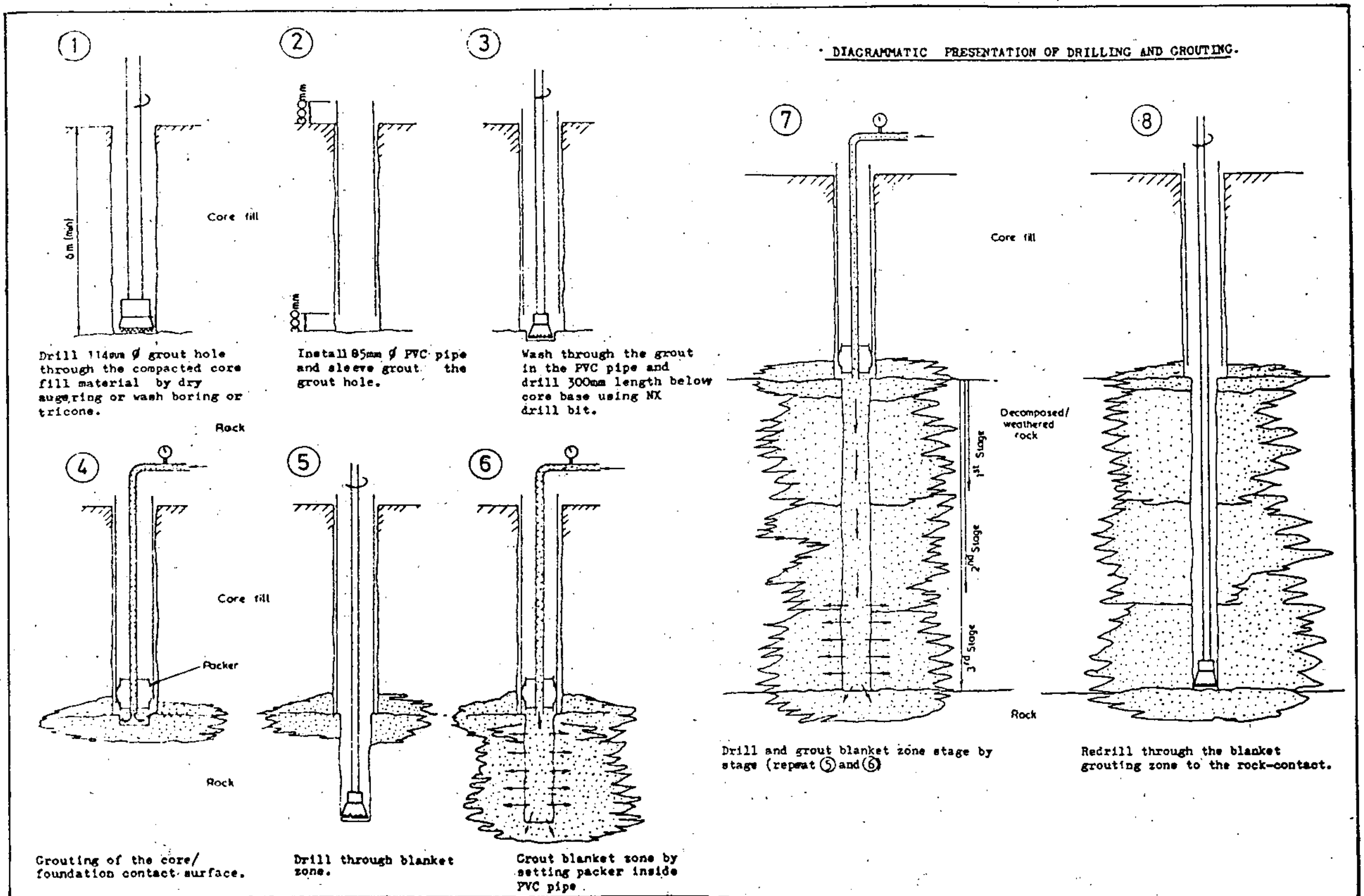
FIGURE 1 - SUMMARY OF DRILLING AND GROUTING

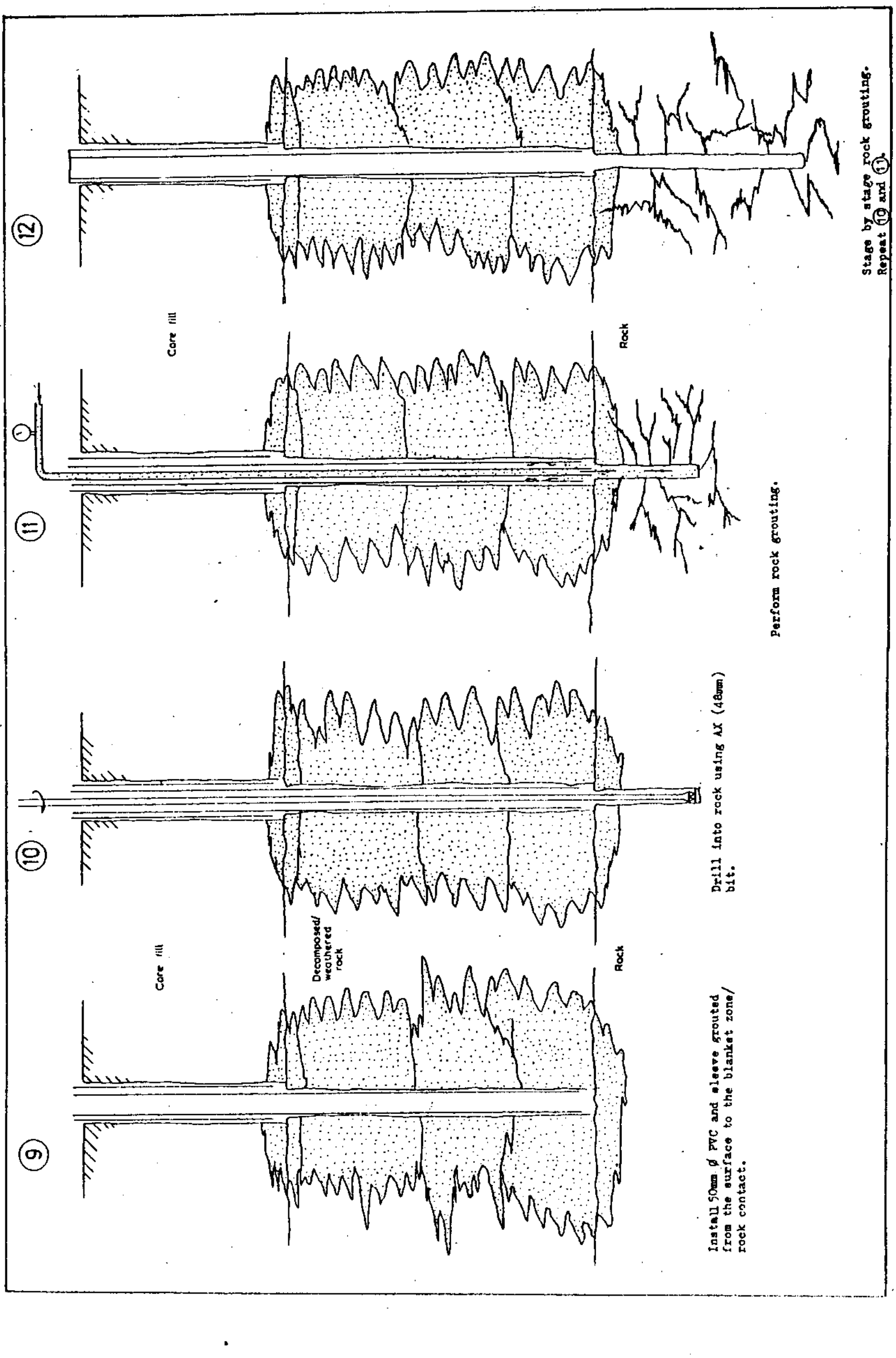
Table 3. Grout Mixes

Water/Cement ratio by weight	5% Bentonite * solution in hydrated form (Litre)	Water (Litre)	Cement (Kg)	Sand (Kg) x	Remark
8:1	12.5	187.5	25	-	
6:1	12.5	137.5	25	-	
4:1	25	175	50	-	One to three batches of each mix to be used for injection depending on grout take, starting with the thinnest mix until refusal. +
3:1	25	125	50	-	
2:1	25	75	50	-	
1:1	25	25	50	-	
2:1	25	75	50	2.5	
2:1	25	75	50	5.0	
2:1	25	75	50	10	
2:1	25	75	50	25	
2:1	25	75	50	50	

* Dry weight of bentonite equivalent to 2.5% by weight of cement.
x sand 5 to 100% by weight of cement.

\+ Defined as grout take of less than 1 litre/min/3m length.


DIAGRAMMATIC PRESENTATION OF DRILLING AND GROUTING.
1
6 m (min)
Core fill
Drill 114mm ∅ grout hole through the compacted core fill material by dry augering or wash boring or tricone.
Rock
2
300mm
300mm
Install 85mm ∅ PVC pipe and sleeve grout the grout hole.
3
Wash through the grout in the PVC pipe and drill 300mm length below core base using NX drill bit.
4
Core fill
Packer
Rock
Grouting of the core/ foundation contact surface.
5
Drill through blanket zone.
6
Grout blanket zone by setting packer inside PVC pipe.
7
Core fill
Decomposed/ weathered rock
1st Stage
2nd Stage
3rd Stage
Rock
Drill and grout blanket zone stage by stage (repeat 5 and 6)
8
Redrill through the blanket grouting zone to the rock-contact.


9
10
11
12
Core fill
Decomposed/ weathered rock
Rock
Core fill
Rock
Install 50mm ø PVC and sleeve grouted from the surface to the blanket zone/ rock contact.
Drill into rock using AX (48mm) bit.
Perform rock grouting.
Stage by stage rock grouting. Repeat 10 and 11.

vertically orientated hydraulic fracture can occur.

35. For fissure grouting in rock a gauge pressure of 14 kN/m^2 per metre depth is regarded as an acceptable pressure (See Figure 1).

Stability in excavation

36. Weathered rock including Grade V and VI residual soil, is remarkably stable in the short term and this permits the use of very steep slopes in temporary cuts required for the construction of the permanent works. It is not unusual to have permanent cut slopes with a batter of 1 on 1 to a height of 30 m when forming tunnel portals and spillways but these must be properly drained and provided with a protective membrane to prevent surface erosion and softening by infiltration of rain. Other stabilising measures include horizontal drainage pipes and rock anchors.

37. Such considerations are particularly important in designing the diversion works for residual soil dams. The depth of weathering often inhibits the use of tunnelling for stream diversion because there is a high cost in the preparation of the tunnel portals for tunnelling lengths usually less than 200 m. Because the fresh rock is often at or near stream bed level in the centre of the dam, it is usually more economical to adopt a cut and cover culvert construction for the diversion works. This can usually be placed on rock Grade I or II and the risk of differential settlement or seepage past the culvert can be limited.

38. Such an arrangement was made for the Seletar dam in Singapore when the invert of the tunnel was designed to accommodate 150 mm of differential settlement. In the event, the settlement was a fraction of this.

FILL MATERIAL

Remoulded soil characteristics

39. The residual souls derived from tropical weathering are close to ideal for dam construction.

40. The fill is usually easy to dig from within the reservoir area near to the dam site, by shovel or scraper, although occasional ribs of rock and isolated core boulders may be encountered. After transportation to the embankment there is usually no difficulty in discharging the fill from the earth mover and spreading it in layers of the required thickness. Equipment to break up lumps is not usually necessary although harrowing to aid moisture content control is effective.

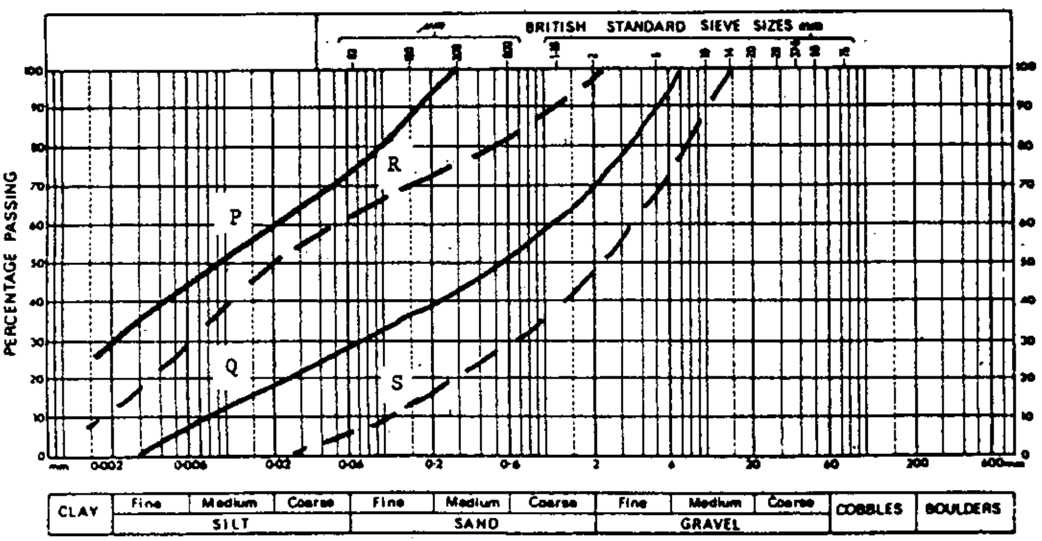
41. The natural moisture content can generally be adjusted without difficulty and after delay due to rainfall, filling can usually be recommenced without delay since moisture loss is rapid in dry weather.

42. Customary characteristics of the fill are outlined below.

Grading

43. The particle size distribution of a residual soil is typically very broad, usually having from a trace to 30% clay and often including all intermediate fractions up to about medium gravel sizes. The percentage of material finer than sand (i.e. clay and silt) may range from less than 10% to about 80% depending on degree of decomposion. (Figure 2).

44. Since the weathering process progresses from the ground surfaces downward, the finer grained (more decomposed) material tends to be found as an upper layer lying over the coarser grained soil, though ribs of more resistant rock or zones where weathering has progressed laterally from faulted or fissured rock may



Semenyih Dam: Core envelope P-Q; Shoulder envelope R-S.

Figure 2. Particle size distribution

result in marked variation in depth of decomposition.

Atterberg limits

45. The wide range of soil gradings to be found in decomposed rock useful for embankment fill are reflected in the results of liquid and plastic limit tests.

46. Liquid limits of between 40% and 60% are common, with plastic limits of between 30% and 40%. However, liquid limits may range on any site from below 30% to over 100% and plastic limits may be between "non-plastic" and 40%.

Natural moisture contents

47. At most sites we have found that the natural moisture content of the fill in the borrow areas has been close to, or a little below the optimum moisture and some watering has been necessary to raise the moisture content to the specified values, including replacing the moisture lost in evaporation during excavation, transporting, placing and spreading it on the embankment. At some sites it has had to be acknowledged that much of the available fill will be too wet to allow 95% of the Proctor maximum density to be reached unless the moisture content is reduced. It has, from study of the Proctor compaction test curves, been possible to specify a range of moisture contents and to require the fill to be compacted to say 98% of the Proctor test density at the placement moisture content. By this means satisfactory fills have been compacted into place under practicable controls.

48. It is observed that residual soils are capable of being handled and compacted often at higher moisture contents with respect to the Proctor optimum than specified. This can be tolerated if the stability due to the pore pressure rise is not critical and the subsequent consolidation settlement is tolerable.

Strength

49. The shear strengths of residual soils are generally appreciably higher than their particle size distribution and in particular their clay content would suggest to the soil mechanics engineer familiar with soil deposits of transported materials in temperate latitudes.

50. As an example, recent tests on a residual soil derived from basaltic lavas showed it to have 33% to 40% clay sized particles and 40% to 44% silt sized particles. Liquid and plastic limits were about 62% and 25% respectively. Given these figures an effective strength angle of shearing resistance ϕ' in say the range of 20° to 25° might be guessed for the material based on experience only of sedimentary soil deposits originating from unweathered rock.

51. However high quality consolidated drained triaxial compression tests with pore pressure measurement on remoulded specimens showed its shear strength to be $\phi' = 34°$ and $c' = 0$ in effective stress terms.

52. The values for ϕ' for tropically weathered granitic and volcanic residual soils generally lie in the range 27° to 37°. For preliminary design of embankments, before test results can be obtained it is usually conservative to adopt shear strength parameters: $\phi' = 30°$, $c' = 0$. There will usually be sufficient of the coarser higher strength fill int he dam cross-section to counter the effects on stability of any finer material of lower strength *(below $\phi' = 30$)* material that is used and which is likely to be directed into a core zone.

Compressibility and permeability

53. The remoulded residual souls in the compacted state are generally of low compressibility. Consolidation is rapid and post construction settlement is very low.

54. Compacted core fills of clayey residual soil can usually be expected to have a permeability of the order of 1×10^{-9} m/s, which is perhaps 2 orders lower than is necessary to keep seepage losses through cores of conventional design to insignificant rates.

55. Even in Plover Cove dam, Hong Kong, where decomposed rock was placed in the core underwater without compaction, Guildford and Chan (1969) report than large-scale borehole tests indicated average permeability (k value) of 1×10^{-6} m/s which corresponded to the design value.

DAM DESIGN

General

56. In general, the embankment dam design has developed to allow the most economical use of the locally available residual soil distributed so that the more clayey material is available for the core and the more coarse grained residual soil is as a sandy clay is used in the shoulders of the dam.

57. The proportions of core width to height are generally selected to have a ratio of 0.5. This allows a core having low hydraulic gradients and a good base width on the core contact.

58. The dissipation of pore pressure during construction is sufficient to maintain a satisfactory margin of safety against slide failures or excessive deformation during construction.

59. With the generally high value of c_v of the order of 100 m^2/year or more, dissipation of construction pore pressure is relatively rapid. End of construction r_u (U/ h) will normally be less than 0.6 (Ayer Itam dam) but more often is significantly lower than this, 0.27 (Lower Shing Mun dam). Some residual pore pressure *in the core has an advantage in* ensuring that the core can respond in a deformable manner to differential stresses in the embankment during construction.

60. The outer shell of the dam is commonly provided with drainage blankets for rapid dissipation of construction pore pressure. Spacing of blankets relates to the consolidation rate of the fill and the permitted rate of raising the embankment. A

spacing of 10 m is a usual value.

61. The pore pressure response in the core of a typical residual soil dam, is affected by the relative stiffness of the outer shell and the resultant redistribution of the vertical stresses to less than the theoretical overburden stress.

Drainage provisions

62. The drainage provisions are complementary to the good treatment of the dam foundations. Foundation relief wells may in certain circumstances be required and a base drainage mattress is essential and must be properly detailed to take care of foundation irregularities. Inadequate construction of the base drain, chimney drains or filters has been a significant cause of dam malfunction in the past.

63. It is now generally accepted that the best all-round filter is medium sand graded with a D_{50} size 2 mm and a D_{10} size 0.2 mm. Such a filter will be appropriate for any type of fine grained soil core. It should be of adequate width to allow for any conceivable displacement. A least width of 3 m will give and adequate margin for intermixing with the adjacent zones during compaction without impairing the integrity of the filter. The control of the filter during construction is as important as that of the core for the security of the dam. The base drainage is controlled by a filtered drain to take care of foundation seepage.

Surface protection - Upstream slope

64. The wind strength is low in Peninsula Malaysia although very high in Hong Kong.

65. Riprap is the usual and most economical form of protection for embankment dams and there have been few reports of any damage to the revetments. Maximum wind and wave predictions are based on Saville's method taking into account the fetch increase appropriate to long narrow valleys.

66. The significant wave height for Upper Muar dam is 0.5 m with small fetch of < 1 km.

67. In Hong Kong the higher wind speeds and larger reservoirs have resulted in maximum wave heights of up to 2.3 m.

68. Riprap design is rarely a critical factor because of the abundance of granite rock of suitable dimension. Minimum requirements follow CIRIA report No 61 which provides criteria for the grading of riprap related to the slope angle and the significant wave height. For "no damage" criteria in the 50 year event, the D_{50} riprap must be equal or greater than Hs the significant wave height derived from the 50 year short period wind event. For Upper Muar riprap was 0.8 m thick on a graded rock base. D_{50} of riprap was 150 kg.

Surface protection - Downstream slope

69. In the high tropical rainfall areas protection of the exposed downstream slope of residual soil is important. Frequent berm drains and substantial mitre drains are essential as is careful establishment and maintenance of grass slope cover.

CONSTRUCTION CONTROL

70. To achieve the objective of producing the most economical embankment from the local materials, the specification for construction control is written to minimise restriction of the flow of embankment material from the borrow pits, and to accommodate both the variation in weathering profile within the borrow pit and the normal tropical humid climate, particularly in Peninsula Malaysia.

Selection

71. The first consideration is to obtain sufficient core material from the Grade V to Grade VI weathering profile. Such material generally has 25% at least passing 0.06 mm (silt size) and should generally have a plasticity index in excess of 20%. Such material exists in the borrow pit at or about Proctor optimum moisture content. It is generally necessary to increase the water content of the fill to between 2% and 5% above Proctor optimum.

72. The core fill is generally placed and compacted to produce a layer thickness of 200 mm and compacted to achieve 95% Proctor maximum dry density or 100% Proctor dry density at the placement moisture content.

73. The material for the shoulders of the dam can be from the remaining weathered material until progressive increase in the hardness of the rock prevents its use as a normal fill material.

74. The moisture content control in the shoulder fill is generally -2% to +3% with same density control as the core.

75. The rate of placement is controlled at a general rate of 3 m in a four week period but the placement of fill must be modified or additional drainage blankets added if the pore pressure response exceeds design values. A wide range of compaction equipment is suitable for residual soil fill.

76. It is normal to specify the adequacy of the plant by the following criteria:

(i) The total output of compaction equipment at the Site shall be at least 4000 cubic metres per day of material measured when compacted to the specified in situ dry density.

(ii) One heavy-duty disc harrow shall be provided.

(iii) Rubber tyred rollers shall each weigh a minimum of 400 kN and shall each have a minimum of 4 wheels located abreast. They shall be capable of exerting average ground surface pressures on a plane firm surface greater than 550 kN per square metre. They shall be designed so each wheel will carry equal loads when traversing uneven ground.

(iv) Vibrating rollers shall have a static weight of not less than 100 kN and of not less than 33 kN per metre width of the roller. The frequency of vibration of the rollers shall be adjustable from 20 to 30 cycles per second so that compaction can be carried out at or above the critical frequency of each type of fill material.

77. Adequate watering equipment is essential to maintain fill moisture content and it is normal to specify at least three 4 m^3 capacity self propelled rubber tyred bowsers having a capacity of 330 l/minute.

MONITORING THE BEHAVIOUR OF THE DAM

78. When planning the instrumentation of embankment dam it is important to limit the array of instruments to those necessary to observe the construction behaviour of the embankment and its long term performance under reservoir full conditions. The instruments that are installed should be designed to give answers to specific questions that are posed by the designers and owners of the dam.

79. During the construction of residual soil embankment dams it is essential to measure pore pressure developed in the core and the shoulder and, if it is compressible, the foundation. It is our practice to use hydraulic piezometers with low air entry tips for this function.

80. No attempt will be made in this short paper to define standards or types of installation.

81. To monitor the consolidation of the fill during placement and the deformation of the foundation, a limited number of vertical settlement gauge tubes should be installed. Although many of our dams have employed the USBR type of mechanical cross-arm settlement gauge, we now routinely use the magnetic variety. It has to be accepted that any vertical settlement gauge tube will cause a considerable problem when compacting fill round the tube. This cannot be avoided but great care must be taken to supervise the installation.

82. Finally, as construction proceeds surface monuments must be installed to monitor final settlement and deformation. It is important to install the control stations before the commencement of construction and install monuments as soon as berms are developed on the downstream slope particularly.

83. The piezometric readout equipment must be installed in a robust, preferably buried, chamber under controlled temperature and humidity conditions beyond the limits of the embankment. If the whole installation is properly installed and serviced during the construction period when there are highly qualified staff available on site, then it is reasonable to expect a very long life for such instruments so that the long term behaviour of the embankment can be monitored.

84. Dam safety requirements demand that there are periodic, preferably annual, surveillance reports prepared for the dam from the date of completion and first impounding, particularly for the first five years on an annual basis.

85. An instrumentation system should be designed to ensure that the readings can easily be made by technically qualified staff and are meaningful to the owner's operation and maintenance staff.

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6. Instrumentation of the Mrica dam

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Synopsis
Mrica Main Dam is situated on the Serayu River in Central Java where it provides storage for a 180MW power station. The crest of the Main Dam is 800m long and its maximum height above the foundation is 110m. The types of instrument, their installation and operation, for monitoring the behaviour of the main dam, appurtenant structures and excavated slopes are described, together with procedures for collecting the readings and processing the data. Some aspects of the behaviour of the main dam during impounding are discussed and compared with predicted behaviour.

INTRODUCTION

1. Mrica Hydro-Electric Power Project was commissioned by the Indonesian electricity authority, Perusahaan Umum Listrik Negara (PLN). The dam is situated on the Serayu River in Central Java at an altitude of about 200m where it provides storage for a 180MW power station.

2. The geology of the area is characterised by Recent and Pleistocene deposits overlying Tertiary basement volcanics, the latter comprising andesitic and basaltic lavas and tuff agglomerates. Overlying the weathered crust of the basement volcanics, the Pleistocene deposits consist of tuff sandstones and agglomerates up to 80m thick and are of lacustrine and alluvial origin. These Pleistocene deposits have weathered to clays up to 35m thick. Recent deposition has yielded a mantle up to 11m thick of gravels and boulders under a clay surface.

3. The river at the damsite runs in a deep, steep-sided gorge and the crest of the main (rockfill) dam, at el. 235m, is 800m long at a maximum height above the foundation of 110m. Subsidiary high level embankment dams, up to 15m high and some 6km long, raise the reservoir rim along each flank.

4. The power house sits in a deep excavation immediately downstream of the main dam on the left bank, while the spillway occupies a similar excavation on the right bank (see Fig 1).

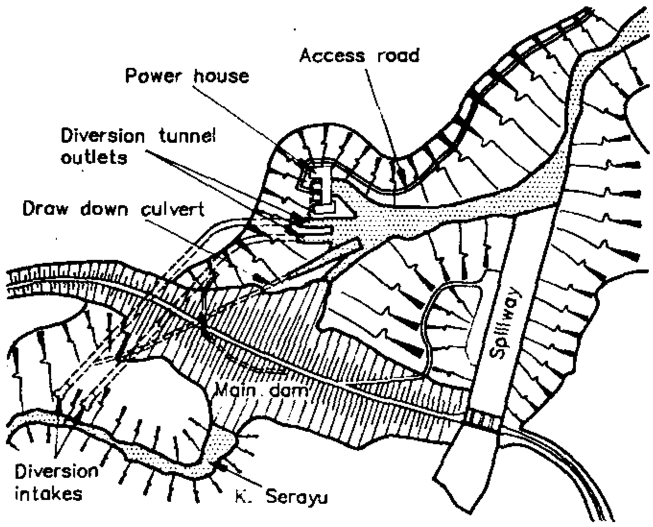

Fig. 1. General Layout

5. Halloysitic, residual clay for the high level embankment dams and the core of the main dam was obtained from borrow areas within the impounded area and immediately downstream of the dams. The core of the main dam is supported upstream and downstream by rockfill shells formed mainly from quarried andesite but including a zone of "rockfill soil" in the downstream shell to utilise selected material arising from the excavations for the spillway and power house. Transitional filters upstream and downstream of the core were formed using crushed rock from the same andesite quarry.

6. The main dam foundation in the river gorge was excavated to sound rock over the width of the core. On the higher, flatter ground above the river gorge the dam is founded on clay and constructed to flatter slopes. A grout curtain some 40m deep provides a cut-off beneath the dam foundation across the river gorge section (see Fig 2).

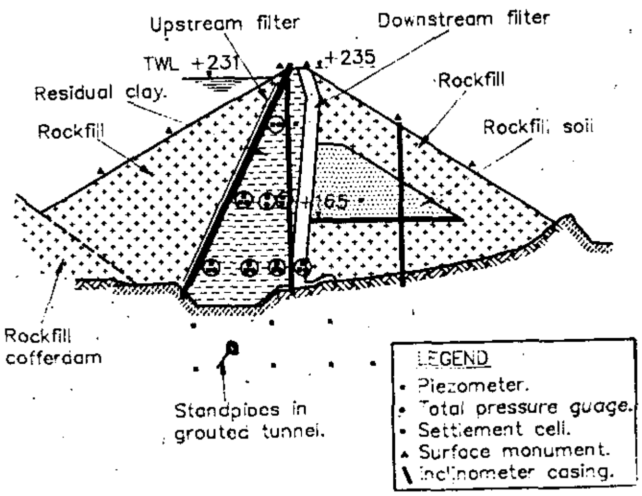

Fig. 2. Cross sections of main dam showing dam construction and arrangements of instruments.

7. The Project was designed by SWECO AB of Stockholm in association with Engineering and Power Development Consultants of Sidcup and *Wiratman & Associates of Jakarta*.

8. Construction took place during 1983 and 1989 with reservoir impounding during May to September 1988. The contractors were a consortium of Shanska, *Balfour Beatty* and Asea.

9. Construction supervision was carried out by PLN with advice and assistance from the Supervising Engineers, Sir William Halcrow and Partners in association with Beca Worley International of Auckland and P.T. Citaconas of Jakarta.

PURPOSE OF INSTRUMENTATION

10. Instrumentation for the main dam was installed for the following three reasons:

- during construction, to provide information that could be used in checking the validity of some of the design parameters adopted
- during impounding of the reservoir, to check actual behaviour against that predicted and to give warning of any serious deviations
- during operation, to monitor the long term performance of the dam and confirm its satisfactory functioning

11. The frequency and form of measurement and reporting were tailored to suit the requirements of each of the phases described above and procedures and computer software developed accordingly. Local staff were trained during the construction phase in measuring the instruments and reducing the data for regular prompt reporting, using the computer programs developed. These staff have been transferred to the operations branch of PLN and will continue to be responsible for the long term monitoring of the behaviour of the Main Dam and appurtenant structures.

LAYOUT OF INSTRUMENTATION

12. Instruments were installed at four sections along the main dam and included piezometers, total pressure gauges, settlement cells and inclinometers. Surface monuments and accelerographs were provided along the crest of the dam. A typical arrangement of instruments is shown in Fig 2.

13. Tubes from the remote reading hydraulic and pneumatic instruments were led to terminal panels in three instrument houses on the downstream slope of the dam where readings were measured manually using portable digital readout units.

14. Pneumatic piezometers, standpipes and survey monuments were also installed along the high level dams, beneath the foundation of the spillway and in the cutslopes around the spillway and power station.

15. A trial embankment, which was incorporated into the left bank high level dam, included pneumatic and hydraulic piezometers and readings on these continued throughout the construction and impounding period.

INSTALLATION AND PERFORMANCE OF INSTRUMENTS

16. The instruments were installed under the supervision of the Supplier's technician who visited site for each episode of installation in the main dam and just before impounding began. Damage to equipment by construction activity and instrument failures did occur and experience in this is described in the following paragraphs. It confirms the need to install redundant instruments where measurements are crucial and provide an adequate stock of spare instruments.

17. Instruments in the main dam were installed in clusters, as shown in Fig 2. Each cluster consisted of piezometers, pressure gauges and settlement cells so as to permit calculation of effective stresses and ru values.

Table 1. Summary of instruments installed

Location	Pneumatic Piezometer	Hydraulic Piezometer	Standpipe Piezometer	Total Pressure	Settlement Cell	Inclinometer	Surface Monument
Main Dam	72	11	-	34	11	5	48
High Level Dams	18	-	1	-	-	-	8
Trial Embankment	20	5	-	-	-	-	-
Spillway	59	-	25	-	-	-	35
Power Station Area	-	-	28	-	-	-	23
TOTAL	169	16	54	34	11	5	114

Piezometers

18. Piezometers in the main dam foundation were installed in boreholes with their tips surrounded by sand sealed on top with a bentonite cement plug. Those in the core were placed in intimate contact with the clay by forming a hole for the tip with a dummy probe. Three types of piezometers were used:

19. **Pneumatic piezometers** were chosen by the Designer to be the principal instrument for measuring pore pressures, mainly for reasons of ease and convenience of use. These instruments were not particularly sturdy, especially in the dam foundation where the failure rate exceeded 50%. Of the 72 pneumatic piezometers installed in the main dam, only 46 were still in operation at project completion. Possible reasons for this relatively high failure rate include constriction in the tubes due to large settlements; silt in the tubes following repair after construction damage; poor reading technique on instruments with long tubing lengths; and equipment malfunction.

20. **Hydraulic piezometers** were installed in the main dam embankment as back-up instruments for the pneumatic piezometers. These instruments have been robust and reliable. Of the 11 hydraulic piezometers installed in the main dam, 8 were still in operation at the completion of the project.

21. **Standpipe iezometers** consisting of Casagrande-type porous pots and standpipes were installed in boreholes to monitor longer term changes in the groundwater levels, particularly around the tunnels, along the downstream toe of the main and high level dams and on cutslopes in the downstream area.

Total Pressure Gauges

22. Pressure gauges of the oil filled, hydraulic-pneumatic type were installed in trenches in the main dam embankment. They were subsequently backfilled and compacted by hand.

23. At the lower levels of the dam, the gauges were placed in rosettes of three, one vertical, one inclined at 45°, and one horizontal, the faces of each being parallel to the dam axis. At the highest level of instrumentation (el 210m) on the instrumented sections at each abutment, a fourth gauge, perpendicular to the dam exis, was added in order to measure longitudinal stresses.

24. Of the 34 pressure gauges installed in the main dam, 30 were still in operation at the end of the period of maintenance and little difficulty has been experienced with them. Unfortunately, two of the gauges no longer functioning are those that were installed perpendicular to the axis of the dam at each abutment.

Settlement Cells

25. Pneumatic settlement cells were installed in a manner similar to that for pressure gauges. Good results, consistent with those from the plate magnets on the inclinometers, were obtained until the readout unit malfunctioned, unfortunately during the impounding period, and had to be returned to UK for repair. Substituting the read-out unit for the pneumatic piezometers gave wildly erratic readings and a consequent gap in data during the long repair time.

Inclinometers

26. Inclinometers were installed at two sections of the main dam, in the clay core (tw), in the upstream filter zone (one) and in the downstream shell (two). That in the upstream filter was inclined parallel to the upstream face of the core.

27. Three-metre lengths of plastic inclinometer casing were built into the embankment and extended as filling proceeded using telescopic couplings to allow compression along the casing. A biaxial torpedo was supplied and readings were stored in a cartridge readout unit and transferred directly into the site computer.

28. Ring magnets placed over the casings and embedded in the fill material enabled settlements to be measured using a reed switch probe.

29. Difficulties were experienced in maintaining the alignment and orientation of the inclinometers during construction. The tubes in the core and upstream filter zone were worst affected and when the embankment level had reached el 187m the torpedo was unable to pass below approximately el 170m and 142m respectively in these tubes. A tolerance of 50m radius on installed alignment was imposed and measurement of tube twist by survey was introduced.

30. In April 1988 a twistmeter was brought to site and a series of twist readings were taken on each inclinometer, but the results showed poor repeatability and it was concluded that the twistmeter measurements could not be relied upon for any of the inclinometers. Consequently the inclinometer deflections themselves are of very limited use. Worst affected is the inclined inclinometer where an incorrect figure for the orientation of the tube will clearly produce very large errors in apparent displacements.

31. With the exception of one tube (blocked at about el 200m) all the casings coul be traversed by the probe for the plate magnets and useful information on settlements within the embankment was obtained.

32. The reason for the excessive twist in the inclinometer casing was not established but it is possible that storage arrangements on site were a contributory factor.

Seismic Accelerographs

33. Seven strong motion accelerographs were supplied but completion of the dam crest after the start of impounding meant that those on the main dam could not be installed before impounding started. Several problems with the instruments themselves, such as trace alginment, time code generator and battery failure, were also encountered. As a result very little information was obtained during the impounding period although two tremors did occur.

34. By project completion all of the seven units were installed and working correctly. Mains power supply was introduced where possible and a spare battery system where not.

Surface Monuments

35. Surface monuments were surveyed using EDM tachymetric equipment with readings stored in a data logger which was downloaded directly into the site computer. In the absence of reliable data from the inclinometers these measurements provided the only information on lateral movements of the dam. The only problem experienced was occasional damage due to construction activity and, as with the accelerographs, completion of the dam crest after the start of impounding meant that some of the monuments were not available during the early part of the impounding period.

Seepage Measurement

36. Seepage measurement at the toe of the main dam in the river valley was by means of a vee-notch weir fitted with an automatic level recorder.

37. The readings proved to be rather variable, being directly influenced by rainfall and, apparently, by the flow from relief wells under the downstream shell. However, the maximum flow with the reservoir full has not exceeded 30 l/min which is considered very satisfactory.

38. Seepage into the grouting tunnel and construction adits was collected in sumps and pumped out. This seepage was measured by flowmeters on the discharge pipelines.

DATA COLLECTION AND PROCESSING

39. Data collection was carried out by PLN laboratory and survey departments under the guidance of the Supervising Engineer. With the large number of instruments involved, this required a full time team. The technicians in this team participated in the supervision of installation and were given instruction on reading techniques by the Supplier's technician during his visits to site.

40. Each day's results were delivered to the computer section for processing and plotting using the site computers. Initially, the micro-computers were two Commodore machines and the necessary programs were written in Basic by the Supervising Engineer's staff. Later, an IBM-compatible micro was acquired and a spreadsheet program was used on it for some of the instruments. The computer section was required to work a double shift to keep pace with the supply of data.

Monitoring during construction

41. All instrument readings and survey measurements were made weekly while seepage measurements were made daily. Results were plotted and reported weekly with copies distributed to the PLN Site Project Manager, Site Supervising Engineer and the Designer's site representative. Copies were forwarded to the Supervising Engineer's head office monthly.

42. Readings from piezometers in the main and high level dams were plotted against time to show piezometric elevation. Embankment level and r_u values (pore pressure ratios) were also shown on the same plot.

43. Total pressures (in kPa) measured by the pressure cells in each cluster were plotted against time. Also shown on these graphs were pore pressure, r_u value and theoretical overburden pressure for the cluster.

44. Settlements measured by the settlement cells were plotted against time. Those measured by the ring magnets on inclinometers were plotted against elevation, settlements on four consecutive dates being plotted on each graph.

45. Data from survey of surface monuments were presented as changes in easting, northing and elevation and plotted against time.

Monitoring during impounding

46. During impounding, weekly instrument readings and survey measurements continued as before, but selected piezometers (in the main dam foundation, power station area, tunnels and beneath the spillway) were read daily along with the seepage measurements. Daily inspections were also made by PLN engineers and inspectors, with advice from the supervising Engineer. The inspections encompassed the downstream toe and slopes of the main and high level dams, cutslopes around the power station and spillway, and the valley sides immediately downstream of the main dam.

47. The daily inspections were recorded on forms, purpose made by the Supervising Engineer, and the daily measurements were summarised on a single sheet that was distributed daily. The measurements were also plotted daily by hand and these graphs were available for inspection in the PLN instrumentation office.

48. The weekly computerised plots were modified as necessary to provide information appropriate to what was required during impounding. Plotting of ru values was discontinued and reservoir level was included in place of embankment elevation. A new datum was used for measurement of settlement. Typical plots of piezometrikc level are shown on Fig 3.

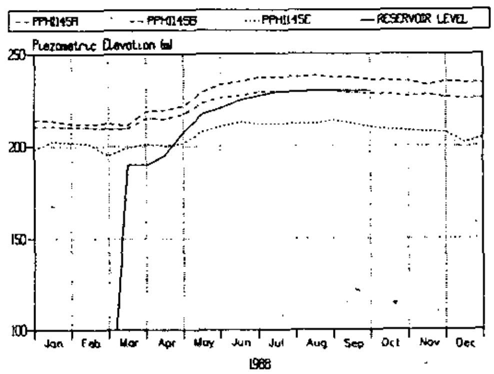

Fig. 3. Main Dam - Typical Piezometric Levels at elevation 145m

Longterm monitoring

49. Maximum intervals between readings of instruments have been recommended by the designer for checking the longterm performance of the dam and appurtenant structures. Measurements of flow and seepage are stipulated to be at weekly intervals and of surface monuments at 3-monthly intervals. All other instruments are to be read monthly.

50. The measurements are to be plotted in the same manner as was adopted during impounding. As stated earlier, the instrumentation team has been transferred to the operational branch of PLN at Mrica and will continue to carry out the measurements and plot the results. "Guard" values for each of the instruments have been supplied by the Designer and a review by Engineering Staff will ensue automatically if they are exceeded.

BEHAVIOUR OF MAIN DAM

51. The instrumentation has confirmed that the dam has generally behaved within acceptable limits and as predicted by the Designer. Only particular aspects will be described herein, notably the deformation of the dam and the stresses in the core following impounding.

52. The Designer carried out finite element analyses of the main dam as a part of the design process, with the purpose of assessing stresses in the core and the risk of hydraulic fracture. Since it had not been possible to test the shoulder materials ultimately used to derive deformation moduli, ranges of values were assumed and three cases were analysed. The analyses showed that the design was reasonable and gave predicted stresses in the core and deformations in the shoulders that it was hoped would be verified by the instruments in the course of impounding.

53. This aim was partly confounded by the failure of the inclinometers to provide reliable results of actual deformations and the malfunctioning of the settlement cell readout unit during impounding. It was particularly disappointing that deformations in the upstream shoulder were not caputred during impounding. However, settlements from the pneumatic settlement cells and the plate magnets on the inclinometers were reliably measured during construction and were used to review predicted settlements thereafter, which were partly verified by measurement of the surface monuments; and the date from the surviving piezometer, pressure gauge and settlement cell clusters were used to compare measured stresses in the core with those predicted. The results of these latter comparisons may be summarised as follows, together with the aid of Figs 4 and 5:

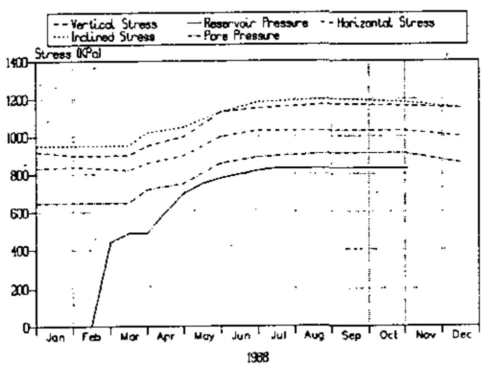

Fig. 4. Stresses at elevation 145m on Section 1

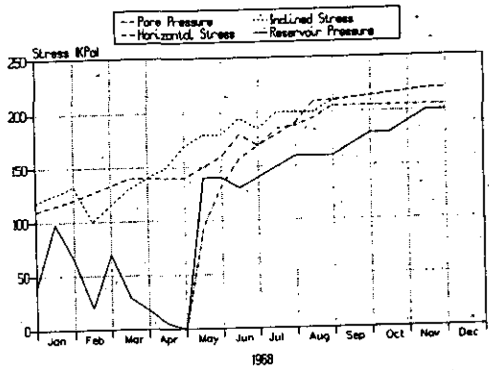

Fig. 5. Stress at Elevation 210m on Section IV

- total principal stresses measured in the core show good agreement with those calculated for one of the finite element cases analysed

- effective shear stresses mobilised are higher and predicted, possibly partly because the finite element analyses tended to underestimate pore pressure

- there is no rick of hydraulic fracture in the lower levels of the core as, additionally, the high pore pressures there will tend to rule this out, as noted by Sherrard (Ref 1)

- at high levels in the core and particularly at the abutments (where there are instrumented sections 2 and 4), there are indications of low effective stresses with a consequent risk of hydraulic fracture. It is unfortunate that most of the casualties amongst the pressure gauges occurred at high elevation and at the abutments.

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Discussion

J.A. CHARLES (B.R.E.)
In a session concerned with the State of the Art of the Embankment Dam it may be of interest to have a brief account of a recent NATO Advanced Study Institute (ASI) on "Advances in Rockfill Structures". The ASI was held at, and organised by, LNEC Lisbon. There were 70 participants and lectures came from Australia, Brazil, France, Germany, Japan, Portugal, Spain and U.K. Although other types of rockfill structure were considered, the ASI was principally concerned with rockfill dams. Some of the issues which emerged during the two weeks of the ASI are as follows:

1. Collapse compression When saturated for the first time rockfill may reduce in volume. Poorly compacted rockfill with a low moisture content is most vulnerable. It can be questioned whether heavy compaction is sufficient to prevent the occurrence of this phenomenon or whether in many cases it is also necessary to water the rockfill. Collapse compression is probably the least well understood facet of rockfill behaviour, yet in some situations it is the most important.

2. Index properties of parent material It would be helpful if these properties (e.g. particle strength) could be used to predict fill behaviour. They are simple and cheap to determine whereas tests on the fill require large expensive samples. However it is questionable whether there is much correlation between index properties and fill behaviour; the density of the fill tends to have a dominating influence on fill performance and obscures the effect of factors such as particle strength.

3. Theoretical soil models These continue to attract the attention of soil analysts. It does not appear that an increase in sophistication leads to any better prediction of constructional behaviour of a rockfill dam that the simple BRE one dimensional compression approach. Reservoir impounding is much more complex; there would be collapse compression in the upstream fill and this is difficult to model.

4. Importance of details For rockfill dams with thin membrane (concrete, asphaltic or geomembrane) details of joints etc. can be of major significance. For example with concrete faced rockfill dams, problems are often associated with the perimetric joint.

5. Low grade rockfill There is increasing interest in the use of low grade rockfill. This raises the question of when does rockfill become so low grade that it is effectively earthfill.

6. Seismic behaviour There is also growing interest in seismic behaviour, although heavily compacted rockfill seems to behave very well. The lecture on this topic was illustrated with many photographs of liquefaction failures, but they were all of sands, none were of rockfill.

It is intended that the lectures will be produced as a book in the NATO ASI series.

P. TEDD (B.R.E.)
Referring to Paper 3 and in response to the Chairman's request to have discussion on instrumentation, I would like to describe a novel instrumentation system that was installed at Roadford dam. An important feature of the design and construction of the dam is the junction of the asphaltic membrane and the concrete cut-off structure at the upstream toe of the dam. Relatively incompressible sandwaste has been placed close to the cut-off structure (see Fig. 1) to limit the movement of the membrane close to the cut-off. The Building Research Establishment (BRE) was requested by Babtie Shaw and Morton, consulting engineers to South West Water to design and install an instrumentation system that would measure the deflection of the upstream asphaltic membrane close to the concrete cut-off structure. The instrumentation made use of the E-L (electro-level) system developed at BRE during the last 20 years (ref. 1).

The E-L is a gravity sensing electrolytic transducer that provides an output voltage proportional to the tilt angle. It consists of a small glass sealed tube, partially filled with an electrolytic fluid and with metal electrodes in contact with the electrolyte. Those used at Roadford were about 30mm long and 6mm diameter. Their range is approximately $\pm$

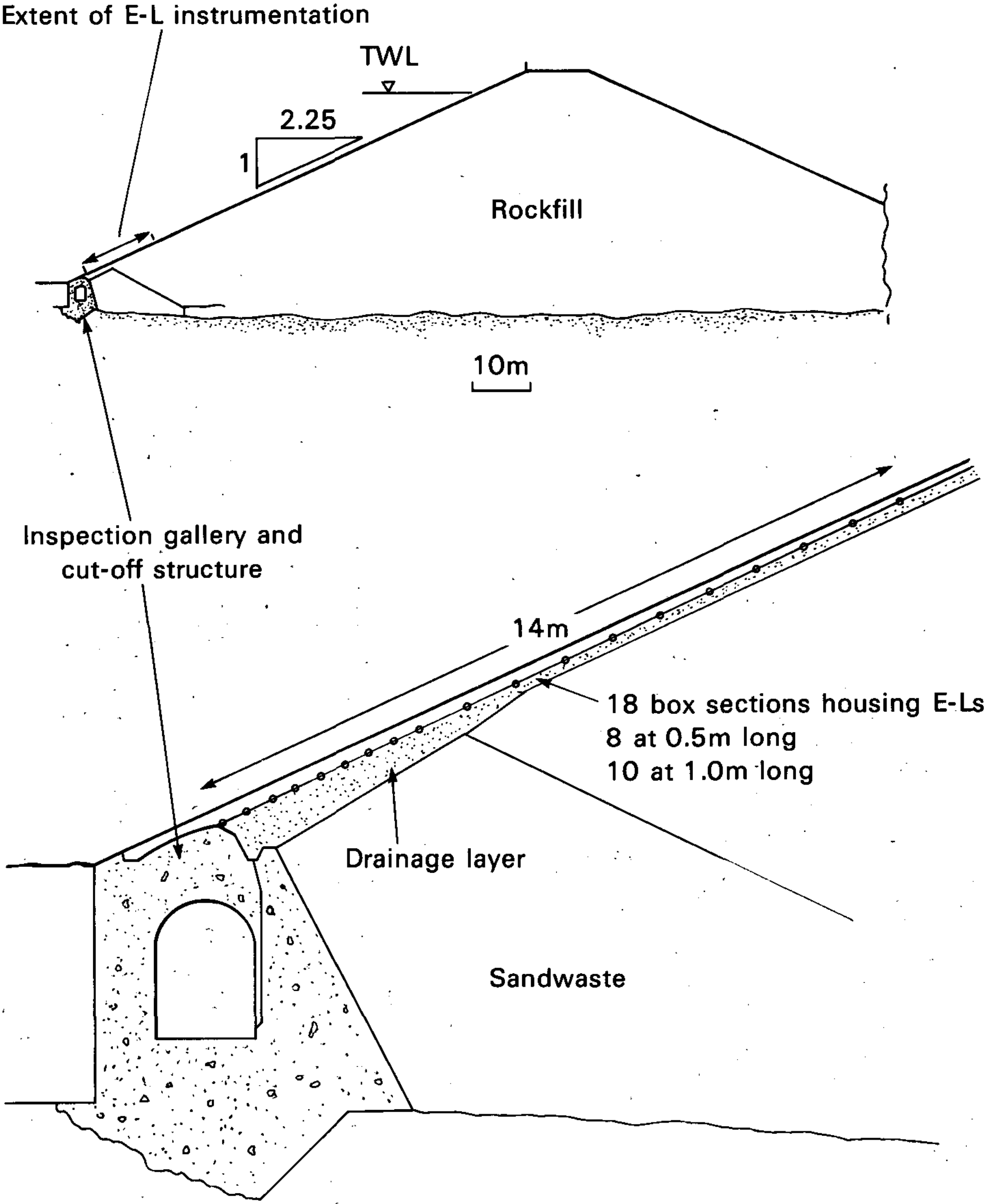

Fig. 1 Section of Roadford dam showing extent and location of electro-level instrumentation

3^{o} (+ 52mm displacement over a metre length) and the long term accuracy is better than 50 seconds of arc (0.25mm displacement over a metre length).

Electro-levels were fixed to the inside of a series of stainless steel box sections, 50mm by 100mm (see Fig. 2 and 3) joined together by a sliding pin joint. Up to 18 sections were joined together extending some 14m up the slope from the cut-off structure (see Fig. 1). They were placed on the upstream slope immediately beneath the asphaltic membrane with the lowest box section being fixed to the cut-off structure. Cables from the instruments were taken into the inspection gallery and read with a portable readout. The E-L measures the angular rotation of each box section in the vertical plane such that displacements perpendicular to the line of the box section and therefore the upstream slope of the dam can be obtained. Summation of the displacements from each box section gives a displacement profile of the membrane relative to the cut-off structure.

The E-L system installed at Roadford survived the placing and rolling of the asphaltic membrane, and has provided satisfactory results during the partial impounding of the reservoir. Full details of the system installed at Roadford will be presented in Ref. 2.

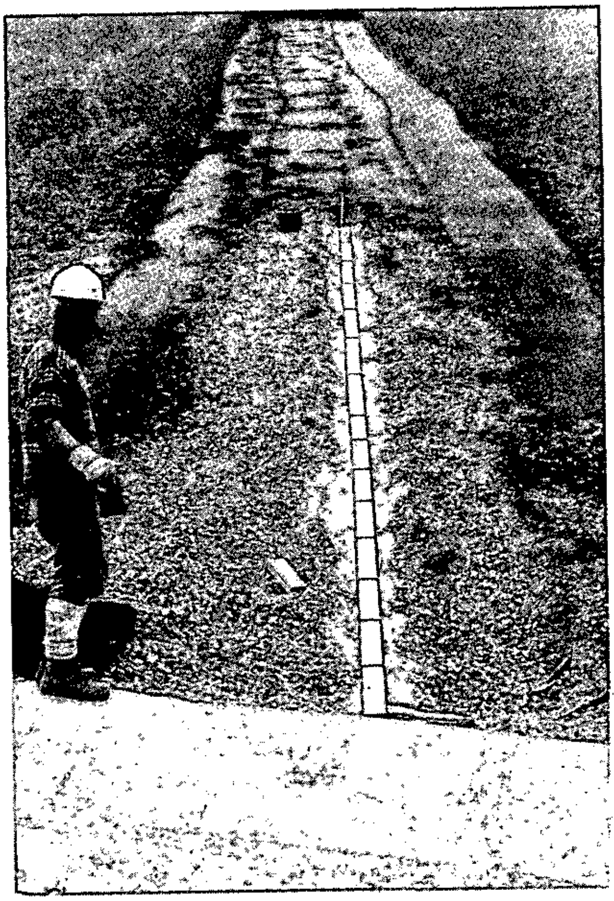

Fig. 3 Completed length of box sections bedded in place

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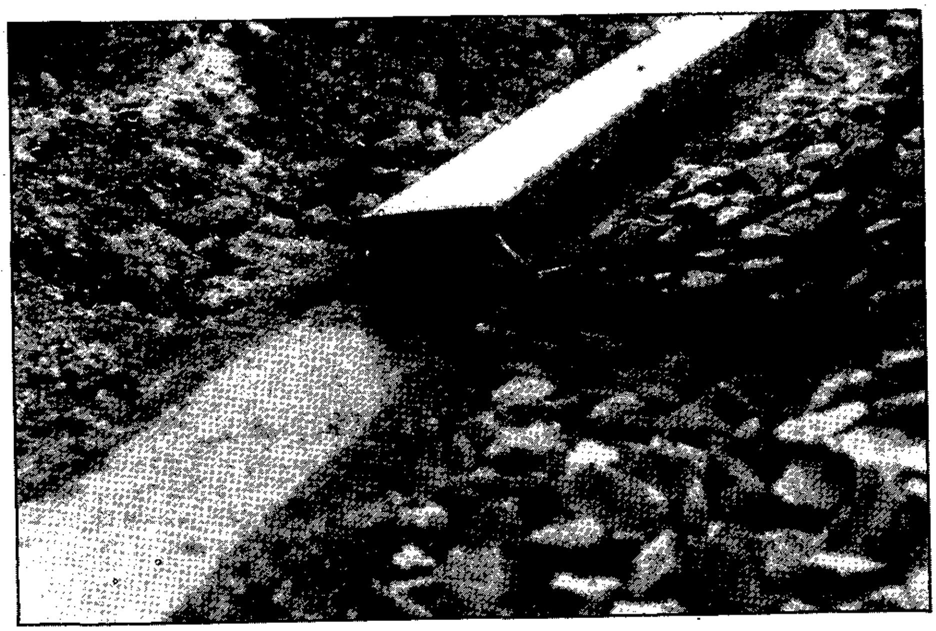
Fig. 2 Box sections bedded in place during installation



S.G. TOMBS (Binnie & Partners)
The Authors of Paper 3 referred to "collapse settlement". A dramatic example of 'collapse settlement' of rockfill occurred at Cogswell Dam in California. The dam was constructed from granite rockfill placed in 7.6m lifts. The rockfill was mainly Class A specified as "a well graded mixture, 40% of which to vary in weight from quarry chips to 1000 pounds, 30% from 1000 to 3000 pounds and the remaining 30% from 3000 to 14000 pounds and the mixture was not to contain more than 3% of its total weight in quarry dust and the maximum dimension of any piece was not to exceed 3 times its minimum dimension. All rock was to be sound, hard, durable, angular quarried rock weighing not less than 160 pounds per cubic foot and to be unaffected by air and moisture and of such toughness as to withstand dumping without undue shattering or breakdown and to have a minimum compressive strength of 5000 pounds per square inch."[1] During placing a layer of much finer rockfill was produced on the surface of each lift by the action of caterpillar tracks. This layer was scarified prior to placing the next lift. No sluicing was carried out because of a lack of water. Part of the dam was completed to the full height of 85m by the beginning of December 1933. On the last day of December a major storm swept in from the Pacific Ocean and by noon on 1st January 1934 about 380mm of rain had fallen on the dam. The crest at station 4 + 80[2] settled by 1.8m overnight and by 13 June 1934 the total settlement recorded (since the beginning of December 1933) had increased to 4.1m. Subsequently large quantities of water were added to the rockfill and by 8 August 1935 total settlement had increased to 5.3m.

In the late 1960s we carried out a number of tests on samples of rockfill in oedometers and compared the results with tests on samples of chert. The rockfill materials were silurian cleaved mudstone similar to that subsequently used for the shoulders of Brianne Dam and granitic gneiss from Venemo Dam. Samples were prepared dry and compacted to a dense state. Loading was applied and the initial settlement was measured. The samples were then flooded by allowing water to enter through porous media at the base of the oedometer. In the case of the silurian cleaved mudstone and the granitic gneiss the settlement increased immediately by about 95% and 13% respectively of the initial settlement. The settlement of the wet silurian cleaved mudstone was therefore nearly double that of the dry material for the same applied loading. The contact points in an angular rockfill will deform and fracture until the product of contact area and rock strength equal the contact load. Wetting of the rock surface reduces the strength of the rock by a varying amount depending on rock type. Therefore after wetting further fracturing takes place at the contact faces so that the product of wet strength and new contact area again equals applied load. Wetting of the chert produced little additional settlement. However where chert particles were crushed under load failure was by shattering producing razor sharp needles which could not be handled.

It should be noted that when the silurian cleaved mudstone and granitic gneiss samples were removed from the oedometer and dried and the tests repeated exactly the same results were obtained showing that the dry strength is only temporarily reduced while the material is wet. This phenomenon of temporary reduction of strength in certain environments applies to other engineering materials and the property has been used to advantage in some machining operations.

In order to assess the minimum amount of water needed when placing and compacting rockfill, so as to achieve the maximum immediate settlement, oedometer tests may be carried out on a series of appropriately compacted samples at increasing moisture contents until no additional settlement occurs at greater moisture content.

References:

1. Baumann, P., Rockfill dams: Cogswell and San Gabriel dams. Trans ASCE Vol 125 1960 Part 2 p29ff.

2. Spielman, J.V., Discussion. Trans ASCE Vol 125 1960 Part 2 p60.

J.M. McKENNA
I would like first of all to praise Dr. Chin and his co-authors for publishing a most valuable paper. Case histories are always of interest but one where the pre-construction design assumptions were found to need significant alteration during construction, is of particular interest.

Secondly, the paper concerns the Coulee East dam. However, 10 km to the west is the Coulee West dam with the same height (28m). There the foundations "performed generally as expected". It appears from the text that the foundation conditions at both dam sites were the same except that the West dam foundation clays were thinner being up to 35m thick compared with the East dam foundation clays which were up to 60m thick. I would ask the authors to comment on the reasons why they got it right for the West dam foundations but did not get it right for the East dam foundations.

Thirdly, the pre-construction pore pressure calculations using "linear elastic" stress changes gave pore pressures up to 35% lower than those measured in the field. However, the pore pressure calculations using "finite element" calculated stress changes, done during (or after?) construction gave pore pressures which are in remarkable agreement with the field measurements. Would the authors confirm therefore that Henkel's and Law & Bozozuk's equations give the correct predictions as long as the stress changes are correctly predicted? Would they use the same methods for their next dam on similar foundations?

Fourthly, the parameters used in the finite element analyses were derived from laboratory test data supplemented with data from the technical literature and that to get deformation agreement "some fine-tuning of the model was necessary". It would be of interest to the profession if the authors would tabulate;

1. the original parameters derived from the laboratory data and

2. the final parameters that were used

so that it will be possible to assess the extent to which the pre-construction laboratory data had to be corrected in order to obtain the observed field performance.

Finally, it is my experience from my involvement on international dam review boards that a finite element or finite difference method analysis is now standard design practice on all major dams to be built on thick soft clay foundations. Deformation and pore pressure predictions are essential elements of modern dam design methods.

References

McKenna J.M. (1989). Properties of core materials, the downstream filter and design. Clay Barriers for Embankment Dams, 63-72. Institution of Civil Engineers. Thomas Telford, London.

T.A. JOHNSTON (Babtie Shaw & Morton)
The integration of the design and construction processes, as described in Paper 1, has occurred on dams in the United Kingdom. Kielder Dam is one example of the cross section being modified in response to construction pore pressures. It is interesting to learn that in Canada this approach is called "the observational method".

A potential draw-back of the observational method is that the rapid construction possible on modern earth-moving projects is at odds with the time constraints imposed by the need to read instruments, collate results, assess their significance and initiate changes on site. How did Dr. Chin and his colleagues resolve the potential conflict between the designer's need for time to observe construction and the builder's wish to construct the dam as quickly as possible.

The effective stress method was used in the design of the dam. Could Dr. Chin provide some information on the factors of safety adopted in the initial design and for the modified cross-section. It would be particularly interesting to know the calculated factor of safety at the time the designer decided to increase the size of the toe berms.

References

Paper 8575 (a) The Kielder Headworks - Proc ICE Part 1, 1982, 72, May.
Paper 8703 (b) Geotechnical Aspects of the Kielder Dam - Proc ICE Part 1, 1983, 74, Nov

A. CAMPBELL (Laboratory Manager, SABCON)

Comments on "Instrumentation of MRICA Dam" Paper 6
by Burton and Ferguson :

<u>Clause 16</u> G I Technical only visited to commission instrument houses, i.e. 3 visits plus one other before impounding. SABCON Laboratory installed all the instruments, prior to those visits.

<u>Clause 19</u> Silt in tubes is considered highly unlikely. All breaks in line were reconnected after removing approx. 3 metres of tubing on either side of break. Lines were checked before jointing by flushing line with Nitrogen Gas.

Breakages/malfunctions most probably caused by long tubing lengths, and differential settlement between core, filter and rockfill. Also, tubing for Pneumatic Piezometers was only 3mm ID.

<u>See</u>: "Field Instrumentation - Accuracy performance, Automation and Procurement" by S.D. Wilson from International Symposium on Field Measurement in Geomechanics. Zurich September 5 - 8 1983.

<u>Clause 25</u> Readout units were unreliable, not really suitable for climate and initially were of digital type. At least 1 No. unit for piezometers was replaced with ordinary Bourdon Gauge type.

<u>Clause 29-32</u> Pipe was stored horizontally on floor of container (don't think it was a contributory factor). Extrusion of Aluminium casing may cause a spiral of 1 degree/3m length. Extruded and machine grooved plastic casing (as used at MRICA), has been known to spiral 18 degree in a 24m length. Exposure to hot sunlight can also cause spiral to develop on occasion. Up to 3m of pipe (when new connection made) would be sticking out above fill and becomes impractical to shade.

There is often the discussion on durability properties comparing aluminium relative to plastic casing and we believe plastic was adopted for this reason. However, from a contractor's viewpoint we consider that a better alignment can be maintained in utilising an aluminium casing.

<u>Clause 39</u> SABCON had the only laboratory on site, which was freely available for use by PLN.

7. The safety of tailings dams and lagoons in Britain

A. D. PENMAN, Harpenden, UK, and J. A. CHARLES, BRE, Watford, UK

Tailings arise from many mining, quarrying and industrial processes. The need for tailings dams and lagoons has grown with increased output of tailings and restriction of discharge into rivers. There are now a large number of tailings lagoons in Britain, with over 1500 associated with mines and quarries. Although the hazards posed by tailings dams can be similar to those from dams which impound water, tailings dams and lagoons are not subject to reservoir safety legislation. Many tailings dams come within the scope of the Mines and Quarries (Tips) Act of 1969, but not all do so. There have been a number of incidents affecting safety and the hazard posed by the sudden release of tailings is examined in the context of the safety of British tailings dams.

INTRODUCTION

1. Large quantities of fine grained waste materials arise in many industrial, mining and dredging activities. The term "tailings" derives from their production at the tail-end of processing plants as waste. Because tailings originate in a wet process, it is usually cheapest to transport them from the plant hydraulically, allowing them to flow by gravity in open channels and flumes, or to pump them through pipelines. The most common form of storage is in lagoons retained by embankments. The cheapest form of construction material for the embankments is usually the tailings themselves.

2. When volumes of tailings were not large, they were discharged into streams and rivers or directly into the sea. Small amounts of waste from the chemical industry and dredgings from canals have been stored in lagoons since the 1920's. The main need for tailings lagoons has arisen with the increased output of tailings and restriction of discharge into rivers resulting from the various Clean Water Acts passed during the 1950's. Information is presented in Table 1 about major producers of tailings in Britain, but it should be recognised that there are many other processes and industries which produce similar types of waste materials.

Table 1. Major producers of tailings in UK

Source	Material	Examples
Coal mining	Fine discard from washery	Lagoons retained by dams of coarse discard; Taylor, 1984 (ref. 1)
Thermal power stations	Pulverised fuel ash	Gale Common lagoon, 154 ha, 51 m high; Haws et al, 1990 (ref. 2)
China clay mining	Micaceous residues and other fines	Kernick Dam, 90 m high when complete; Illsley et al,1976 (ref. 3)
Sand and gravel pits	Silt and clay waste from screening plant	Low bunds surrounding worked out areas
Salt based chemical industry	Hot liquids and sludges	Lagoons retained by dams up to 17 m high of lime stabilised boiler ash etc
Dredging	Clay, silt and fine sand	Lagoons adjacent to Manchester Ship Canal

NUMBERS AND SIZES OF TAILINGS DAMS

3. There is no central register which includes all British tailings dams and therefore there is no accurate knowledge of the total number of tailings dams and very limited information about sizes, shapes, design, foundation conditions, construction methods and operation of lagoons. In a preliminary survey of the safety of tailings dams and lagoons in Britain (ref.4), three approaches were adopted to obtain more information.

4. Inspectorate of Mines and Quarries. Under the Mines and Quarries (Tips) Act 1969 (ref. 5) and the subsequent Regulations, 1971 (ref. 6), notification of all tips connected with mines and quarries must be made to the inspectorate, including tailings dams and their lagoons. This information has been used by the inspectorate to compile a National Tips Register which was first completed in 1975. The report of the Chief Inspector of Mines and Quarries for 1975 refers to this and gives the following information. At the end of the year, there were 2244 tips of solid refuse and 1478 tailings lagoons. By the end of 1976, the number of tips had decreased to 2136, but the number of lagoons had increased to 1547. The decrease in number of tips was attributed mainly to the merging of groups of tips into single, large tips for inspecting and

reporting purposes. More recent figures, released in July 1987, have shown that there are now almost the same number of lagoons as in 1976, ie 1550 made up of 770 active classified lagoons, 479 active unclassified, 215 closed classified and 86 closed unclassified lagoons. The definition of "classified" is given later. An approach to HM Principal Inspector of Mines and Quarries in March 1987 confirmed that while he could identify those tips classed as "liquid", he was unable to help with heights and slopes, etc.

5. British Section of International Commission on Large Dams. When the World Register of Tailings Dams (ref. 7) was published, only 7 tailings dams were recorded for the whole of Britain. A request was made to all members of British Section of the International Commission on Large Dams for information on tailings dams for which they had details which they could release, to provide additions to the British entry in the World Register. This request brought in a further 10 examples, most of which were already known through published literature.

6. Planning Authorities. A request for information was made to 72 Planning Authorities. It was hoped that since all major lagoons require planning permission, these authorities would be able to supply details of tailing dams. Replies have shown that Planning Authorities are unable to give details of height and shape of tailings dams, but many have been able to locate the positions of lagoons and it is clear that there are many lagoons in addition to those coming within the control of the Mines and Quarries (Tips) Act 1969. Within the county of Cheshire, during the year 1979/80, waste from the heavy chemical industry amounted to 21.53×10^6 tonnes, from the Ship Canal 2.4×10^6 tonnes, and from power stations 0.66×10^6 tonnes: most of this waste went to lagoons. A total of 19.32×10^6 tonnes was deposited in lagoons in five districts within the county. None of these lagoons had to be reported to the Mines and Quarries Inspectorate.

LEGISLATION AND INSPECTION

7. The failure of a colliery spoil tip at Aberfan in 1966 led to new legislation to control waste heaps and tailings lagoons. At the time of the enquiry by the Tribunal into the cause of the Aberfan disaster, there appeared to be few regulations controlling tips and lagoons in any part of the world. In Britain, the Inspectorate of Mines and Quarries had no responsibility for tip safety. The report of the geotechnical investigation into the causes of the Aberfan failure (ref. 8) recommended that tips should be regarded as engineering structures to be designed by civil engineers and that a Tip Safety Committee should be set up to advise government. The Tribunal of Inquiry endorsed these recommendations with the following results:

(a) a Tip Safety Committee was set up and advised government on the drafting of legislation,

(b) the Mines and Quarries Inspectorate was strengthened by the creation of a civil engineering branch.

Mines and Quarries (Tips) Act 1969

8. A Mines and Quarries (Tips) Bill was drafted to supplement the existing Mines and Quarries Act, 1954, and became the Mines and Quarries (Tips) Act 1969 (ref.5). It was described as an Act to make further provision in relation to tips associated with mines and quarries; to prevent disused tips constituting a danger to members of the public; and for purposes connected with these matters.

9. In the Act, the expression "tip" means an accumulation or deposit of refuse from a mine or quarry (whether in a solid state or in solution or suspension) other than an accumulation or deposit situated underground, and where any wall or other structure retains or confines a tip then, whether or not that wall or structure is itself composed of refuse, it shall be deemed to form part of the tip for the purposes of the Act.

10. The operation of the Act was laid down by Statutory Instrument No. 1377 (1971): the Mines and Quarries (Tips) Regulations (ref.6), which came into effect on 1 October 1971. It gave a clearer separation between a tip of coarse discard and a tailings lagoon by stating that a "classified tip" was:

(a) a tip consisting of refuse accumulated or deposited wholly or mainly in a solid state and not in solution or suspension and

(i) covering a superficial area of land exceeding 10 000 m^2, or

(ii) of height exceeding 15m, or

(iii) standing on ground at a slope exceeding 1 on 12.

(b) a tip consisting of refuse accumulated or deposited wholly or mainly in a solution or suspension and

(i) with lagoon level more than 4m above the level of any part of the neighbouring land within 50m of its perimeter, or

(ii) with lagoon volume exceeding 10 000 m^3

11. The tips were further separated by the Act, into "active classified tips", i.e. those being added to during working of the mine or quarry, and "closed classified tips" meaning those no longer used for disposal of waste. An inspector may exempt any tip from the application of any of the provisions of the regulations, if he is satisfied that they would be inappropriate. Tipping operations have to be supervised by a competent person, who has to report any defects in a book, also the person responsible for the tip is required to record action taken to remedy the defect in a book.

12. Before construction of a tip can begin, notice must be given to the inspector for the district, who must be satisfied that the condition of the foundation soil and proposed method of construction are adequate to ensure safety. There is no special provision for tailings dams, other than that a tailings dam and its lagoon are regarded as a classified tip if the lagoon is to be more than 4m above adjoining land or exceed 10 000 m^3.

Technical guidance and inspection

13. Following the formation of the civil engineering branch of the Inspectorate, more than a hundred inspectors received special training in soil mechanics and the factors

affecting tip stability. The National Coal Board prepared a "Technical Handbook on Spoil Heaps and Lagoons" and "Codes and Rules for Tips" (ref. 9). Following publication of the Mines and Quarries (Tips) Act 1969, they prepared a production department instruction entitled "Management of Tips". The main emphasis in all these documents related to coarse discard tips: there was little reference to lagoons and no special instruction concerning the inherent danger of tailings dams and the fact that failure is likely to lead to a flow of liquefied tailings over a considerable distance. By 1971 the China Clay Association had produced an excellent handbook on the disposal of waste materials. The principles of design for tips and lagoons were explained in practical terms for those persons who have to carry out the day-to-day operations. The British Quarrying and Slag Federation have issued a useful booklet which will assist quarry owners to prepare the necessary documentation.

Reservoirs Act 1975

14. Water retaining dams are controlled by the Reservoirs Act 1975 (ref. 10) which applies to reservoirs which are capable of holding more than 25 000 m^3 of water above the natural level of any part of the land adjoining the reservoir. Important features of this reservoir safety legislation include:

a) Local authorities are designated as enforcement authorities with specific duties which include keeping a register of reservoirs and demanding full documentation from owners and engineers for dams in their area.

b) Construction and inspection of dams can only be undertaken by qualified civil engineers who are members of specially constituted panels set up by the Secretary of State for the Environment.

c) Owners are required to appoint a named supervising engineer from the appropriate panel to keep the reservoir under continual supervision between inspections.

15. The Reservoirs Act 1975 specifies that "reservoir" means a reservoir for water as such and accordingly does not include a mine or quarry lagoon, which is a tip within the meaning of the Mines and Quarries (Tips) Act 1969. Despite this, some owners of tailings lagoons have had them designed and their construction supervised by a panel engineer. Thus the specialist knowledge and expertise of civil engineers authorised to design and supervise the construction of water retaining dams has been utilised in the design and construction of some tailings dams.

16. The requirements of the the Mines and Quarries (Tips) Act 1969 can be compared with the requirements of the Reservoirs Act 1975. The former only requires 30 days notice to be given to the district inspector prior to commencement of tipping. Method of construction must be specified by tipping rules made by the mine manager or quarry owner, but there is no stipulation for an overall designer for the tip. Prior to commencement of tipping (or construction of a tailings dam) the owner must obtain a report from a competent person on the method of carrying out the intended tipping operation and on every other matter likely to affect the security of the tip, eg site conditions, the total volume to be tipped. The Regulations 1971 require every active tip to be supervised by a competent person. There is no indication as to how the competence of the persons is to be assessed.

FAILURES AND SERIOUS EVENTS

17. In other parts of the world, failures of tailings dams have caused major loss of life. The failure at Buffalo Creek in West Virginia on 26 February 1972, resulted in 125 deaths. The failure at Stava in northern Italy on 19 July 1989 resulted in 269 deaths when two 20 m high tailings dams collapsed and the resulting 250 000 m^3 mudflow engulfed Stava and destroyed part of Tesero. Fortunately no incidents of such severity have occurred in Britain, but there have been a number of uncontrolled releases of tailings. Initially lack of experience in the construction and management of tailings lagoons, when they were first required in Britain, resulted in some failures.

18. In South Wales, at Ty Mawr colliery, a lagoon was formed on the hillside above the colliery, retained by a bund of coarse discard. Sufficient consideration had not been given to the water balance and the lagoon overtopped the bund in December 1961, sending a small flow of tailings down the hillside, which reached the aerial ropeway that carried the waste from the colliery. A second lagoon was constructed on the other side of the ropeway, so that any release would not harm it, but on 25 March 1965, the bank failed, releasing a flow of liquefied tailings which reached the colliery car park, damaging some cars, and almost reaching the shafts. The seriousness of this incident was appreciated, and the Coal Board requested all their colliery managers in South Wales area to check on their tipping arrangements, particularly in relation to tailings, and report. The Aberfan slip, which occurred during the morning of 21 October 1966, was not directly connected with tailings disposal. In this disaster 114 school children were killed when one of the tips failed, causing a flow slide which engulfed their school.

19. Another incident, similar to those at Ty Mawr, occurred on 24 March 1966 at Williamthorpe colliery. A slurry pond which had been built into the Old Dirt Tip collapsed, sending a flow of tailings over an adjacent road which was covered to a depth of 3m and remained closed for 10 days. At Stoney Middleton, in Derbyshire, the retaining dam of a settling pond burst and there was a damage to property and roads, but fortunately there was no loss of life. This failure occurred on 8 February 1968.

20. The annual reports of HM Chief Inspector of Mines and Quarries have shown that dangerous occurrences related to tips and lagoons associated with mines and quarries have continued since they began to be reported following Aberfan. Although the numbers reported under the provisions of the Mines and Quarries (Tips) Act 1969 have fluctuated from year to year, they have shown no consistent downward trend, which might have been expected to follow the Regulations controlling tip and lagoon construction. Over 60 dangerous occurrences involving unstable or potentially

unstable lagoons were reported between 1968 and 1986.

21. Concern about lagoons was expressed by the Chief Inspector in his report for 1968 when he said that it was evident that lagoon bund stability was a matter requiring special attention. He pointed out that difficulties have resulted from inadequate compaction of the earthwork which allowed seepage, saturation and in due course, failure. Occasionally additional material was placed directly on top of a saturated bund, only to become water-logged and so take part in an even larger slip.

22. A large retaining bund of a 4 hectare tailings lagoon at a fluorspar processing plant had been progressively heightened without increasing the width of the base, in order to increase the capacity of the lagoon. Following heavy rains, which reduced the freeboard at the crest of the bund, strong winds caused waves to go over at one point, eventually causing a breach 6m wide and 2m deep. More than 14 000 m^3 of liquid flowed through a nearby village, some parts of which were flooded to a depth of 1m. It was evident that the lagoon had not been properly designed for the capacity it was ultimately required to hold and that, in particular, the upper stages were not constructed on sound engineering principles.

23. A major sand tip failure was caused by tipping sand residues over an old mica tailings lagoon. The initial slip triggered a flow slide which caused the sand to run some 200m across open country. It was an example of the dangerous instability which can occur when tips are not constructed on sound principles.

24. The 10 lagoon retaining bund failures reported for the period 1969/70 were almost all due to excessive seepage caused by inadequate cross section, or failure to incorporate adequate internal drainage in the bund or allowing an excessive depth of free water to stand over the settled tailings. A major breach at one bund was caused by running tailings into the lagoon at such a rate that it flowed over the crest.

25. In 1971 a lagoon bank collapsed when the silty sand foundation became saturated and the bank moved forward towards a freshly excavated area. Seepage had been apparent on the surface over a period and should have served as a warning.

26. One of the three lagoons reported in 1972 had been overfilled and the warning was given by the Chief Inspector that lagoons formed on coal mine refuse tips, particularly those at high level, are undesirable because seepage paths through the colliery refuse are difficult to predict with accuracy. The failure of the other two was attributed to excessive seepage through the bunds. It was pointed out that lagoon dams are often built with readily available but unsuitable material which does not have the required engineering properties.

27. One of the lagoons which failed during 1973 discharged tailings on to a public road and at another lagoon, which had been brought back into use after a period of inactivity, an extensive fissure appeared along the crest of the lagoon bank accompanied by movement of the toe. Part of the bank and its foundation consisted of clayey material and no provision had been made for the drainage of seepage water, although this is important in lagoon banks made of impervious refuse.

28. Coal recovery from an old colliery refuse tip was using a washery process discharging tailings into a lagoon formed in the refuse. During 1974 the lagoon overflowed, causing rapid erosion of the bank and releasing 20 m^3 of water and slurry. A potentially more dangerous situation arose at a sand and gravel quarry where a 6m length of lagoon bund collapsed adjacent to a draw-off pipe and released 9000 m^3 of tailings. This flowed into a series of lower lagoons causing overflows on to adjacent land. This originated in a piping failure and it was said that the onset of piping had been concealed by silt which had built up around the draw-off.

29. Two lagoons gave trouble in 1975. A bank separating two sections of a large lagoon fissured along its centre-line, thereby making it insecure. The bank had not been maintained at its designed height and slope, which resulted in overloading of the foundations. The bank of another lagoon in a sand quarry collapsed, resulting in tailings cascading through a series of lagoons, through the quarry and on to a public road.

30. The very dry summer of 1976 was thought to have contributed to the failure of one lagoon dam due to clay shrinkage. This released about 7000 m^3 of tailings. When the rains came, they were very heavy and were blamed for the failure of a tailings dam at a sandstone quarry. The tailings flowed 500m and flooded the quarry drying plant, an occupied cottage and a minor road.

31. At another lagoon, the rockfill dam failed due to piping through the clay seal on the inner face of the dam. About 7000 m^3 of water flooded the quarry floor and submerged a diesel excavator. At another site, refuse was being placed over an old lagoon when it slipped due, it was thought, to build up of pore pressure under the increasing weight of material.

32. A fatal accident occurred in 1985 at an active spoil tip and lagoon where the reshaping of the site required the location of the tipping operations to be changed. An articulated dump truck driver, employed by earth moving contractors, was drowned when he reversed the truck towards uncompacted ground at the edge of the lagoon. A slip occurred over a length of about 24m and about 2500 tonnes of recently placed colliery discard disappeared into the lagoon together with the dump truck.

33. In another area where landscaping was being undertaken, some lagoons had been infilled during freezing weather. A dam failed, releasing 3000 m^3 of highly viscous tailings which flowed into a second lagoon. This displaced wetter material blocked the drainage culverts and consequently diverted through a tunnel on to adjacent land. Failure was attributed to tipping on to deeply frozen slurry which subsequently thawed.

34. At a quarry, a disused lagoon of solidified slurry had, over the years, been overtipped with the refuse. As a final layer of soil-forming material 1 to 1.5m thick was being placed, the tailings dam failed. It was said that this activity was not covered by the Mines and Quarries Act.

HAZARD EVALUATION

35. Tailings have a water content high enough to permit flow through flumes or pipelines. In a lagoon the particles settle into a meta-stable structure, developing an effective stress under self-weight with corresponding shear strength. When the surface of a lagoon is allowed to dry by evaporation, the pore water suctions developed in the surface layers may produce a surface crust, which may be strong enough to carry the weight of people walking on it and low ground-pressure earthmoving machines. Disturbance of the surface structure can destroy the bond between particles, throwing all self-weight on the pore water and reducing shear strength to zero. Such a phenomenon occurs locally when a machine breaks through a dried surface, producing quick conditions, allowing the machine to sink uncontrollably into the mass of tailings.

36. If the retaining embankment is breached, tailings will be released from the lagoon. Due to their high moisture content, the tailings may liquefy and flow. The storage of hydraulically placed fine grained waste materials above the level of the surrounding ground thus forms a potential hazard to people and property at the lower level.

Hazard analysis and risk assessment

37. The magnitude of the hazard is related to the height of the dam above the surrounding ground, the volume of tailings in the lagoon, the distance of the threatened area from the lagoon and the flow properties of the tailings. Hazard analysis involves not only an evaluation of the magnitude of the hazard, but also an assessment of the probability of failure of the tailings dam and the consequences of such a failure. The hazard may be assessed subjectively or some method of calculation may be adopted. A study of the use of probabilistic risk assessment for a water retaining earth embankment dam has been described by Parr and Cullen, 1988 (ref.11). Despite the difficulties in establishing the probability of many of the base events on which the analysis rested, Parr and Cullen recommended continuing development of the approach.

Comparison of tailings lagoons with tips

38. The hazard posed by tailings dams and lagoons can be compared with those due to tips of coarse waste materials in a solid condition. The Aberfan tip disaster has already been described. In July 1987 the 150 m Little John tip, near the village of Roche in Cornwall, collapsed (ref. 12). Debris flowed 100m down a country lane and an eyewitness said that the failure occurred very quickly. Failure of a tailings dam could release a much larger quantity of tailings from a lagoon than would usually be involved in a tip failure.

Comparison of tailings lagoons with water reservoirs

39. Parr and Cullen, 1988 (ref. 11) recommended that in line with practice in other industries which offer major hazards, the water industry should consider developing emergency plans for dam failure and a scheme for reporting "near misses" and making this available to panel engineers. If this approach were to be adopted for water retaining structures, it may be questioned whether it should also be considered for tailings dams.

40. Failure of a water retaining dam may cause an escape of water that represents the release of a considerable amount of energy. Damage may be done by the force of the water flow as well as the damage caused by submergence. Tailings are more dense than water and can therefore exert a greater pressure on obstacles to their flow. Tailings may push over walls, crush cars and wreck services. The tailings released from the lagoon at Ty Mawr colliery in March 1965 reached the colliery car park where it caused damage to several cars and almost reached the shafts. It is probable that a release of the same volume of water, even if it had flowed through the car park, would have caused relatively little damage because it would not have risen to the same depth as tailings and would have exerted less force. The water would drain away and the incident might not be considered worthy of report. The tailings, on the other hand, remained as a more permanent reminder of the escape which had occurred.

41. The power of tailings flow was demonstrated after the collapse of a tailings dam on 11 November 1974 at the Bafokeng platinum mine in the western Transvaal. "The heavy fluid literally cut away half the winder house and carried with it heavy steel girders, brick buildings, vehicles, reservoirs and heavy stores. Although erosion of the original ground was minimal, all surface installations in the path of the flood were destroyed and a large quantity of slimes entered the shaft, trapping workmen underground and dragging the shaft equipment to the bottom" (ref. 13).

Dam break analysis

42. Estimates of flood damage following the failure of water retaining dams are increasingly being based on dam break analysis. Such analyses permit assessment of damage caused not only by inundation but also by the velocity of flowing water. These calculations can be used as part of a hazard analysis or in the development of emergency planning procedures. The major uncertainty relates to the extent of the breach and the speed with which it develops. A full examination of the hazard presented by a tailings dam can only be made if the potential flow of the tailings released from the lagoon is studied. A realistic assessment of damage would require knowledge of the volume of tailings which would be released, the distance the tailings would flow and the velocity of flow. Application of dam break analysis to tailings dams requires a better understanding of the characteristics of the liquefaction and flow of tailings.

Seismic risk

43. In many parts of the world, earthquakes have caused uncontrolled release of tailings (refs. 14 and 15). Although Britain is considered to lie in a relatively non-seismic zone, Britain is not free of earthquakes. Out of more than 2000 recorded earthquakes which

have occurred during the last 700 years, the maximum had a magnitude of M = 5.5 (Richter scale). Dams constructed of tailings by the upstream method, with steep downstream slopes, are particularly susceptible to earthquake shock. In general, in Britain, no allowance for earthquake has been made during the design of tailings dams and it would be of value to identify those most likely to be affected. It would be of benefit to measure the dynamic characteristics of tailings dams when they have reached various heights in order to check design assumptions. Assessment of in situ properties and dam behaviour by controlled excitation could assist safe design. Little is known about the degree of damping and the low bulk compressibility of the saturated material and interaction with the sensitive structure developed within the lagoon mass may lead to unexpected results.

CONCLUSIONS

44. There are a large number of tailings lagoons in Britain. Over 1500 are associated with mines and quarries alone. They form an inexpensive method of waste disposal for a wide variety of industries and there is no reason to expect that the volume of waste materials disposed by such methods will decrease.

45. Storage of tailings is usually above existing ground level and requires the construction of a tailings dam. Such storage presents potential hazards to life and property which are similar to those occasioned by dams which impound water reservoirs.

46. Incidents affecting the stability of tailings dams and involving the release of tailings continue to occur in Britain. Overseas, there have been failures involving major loss of life and property.

47. Although the hazards posed by tailings dams are similar to those from dams which impound water reservoirs, tailings dams are not subject to reservoir safety legislation. Many tailings dams come within the scope of the Mines and Quarries (Tips) Act of 1969 but not all do so. The Mines and Quarries (Tips) legislation does not provide publicly available registers of tailings dams and does not clearly define how the competence of a person appointed to build or inspect tailings dams is to be assessed.

48. The continuing number of dangerous occurrences involving unstable tailings dams and the consequent uncontrolled release of tailings emphasise that it is essential that the principles of geotechnical engineering are applied in the design, construction and inspection of all tailings dams irrespective of size and whether or not they are associated with mines and quarries.

49. The high standard of safety established for embankment dams which impound water reservoirs owes much to detailed studies of the behaviour of particular dams and the open discussion of problems and hazards among civil and geotechnical engineers. More could be done to study the performance of British tailings dams and lagoons and to evaluate the hazards to which they are subject.

The work described in this paper forms part of the research programme of the Building Research Establishment and is published by permission of the Director.

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8. Tailings dams of the copper mining plant Elatzite after eight years of operation

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SYNOPSIS

The paper gives a detailed description of the structure and the method of execution of the tailings dam at the Elatzite Copper Mining Plant, built after the downstream construction method by hydrocycloning. The tailings dam was designed to reach an ultimate height of 145 m, and in June 1989 it was 95 m high. An analysis is made of the design solution and of the alterations during the construction period of the structure and of the physical properties of the deposited cycloned sand and slime. Lessons and conclusions are drawn on the basis of 8 years of operation, concerning the design, construction and operation of this type of tailings dams.

DESCRIPTION OF THE TAILINGS DAM

The tailings dam of the Elatzite Copper Mining Plant is being constructed after the downstream method by hydrocycloning. It is located about 70 km east of Sofia in the upper part of the hilly catchment area of a small ravine (Fig.1) at an altitude of 600 to 700 m. The area of the tailings dam at the final stage is to be 268 ha, and of the catchment area-330 ha. The average water discharge of the natural runoff in the ravine is 15 l/s. The tailings dam is located in an area of seismicity with a predicted 1000 year return acceleration of 10% of gravity.

The plant has a maximum annual tailings output of 10 mill.t. The tailings dam is designed to use to the maximum the capacity of this site to store the tailings with an ultimate volume of 130 mill.m^3. The outflowing tailings have a mean diameter of 0.068 mm, and the quantity of the particles smaller than 0.074mm (200 mesh) is 70% on the average.

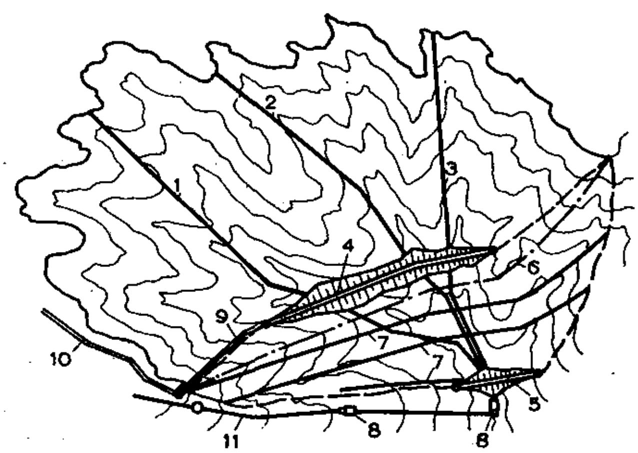

Fig. 1

1,2,3 COLLECTOR
4 UPPER MAIN DAM
5 LOWER MAIN DAM
6 CREST OF FINAL HEIGHT
7 DRAINAGES
8 PUMP STATION
9 SLURRY PIPELINE
10 SLURRY TROVGHS
11 WATER PIPELINE

With a short, but wide reservoir site and this particle size distribution of the tailings, most suitable and economical in comparison with the several alternatives discussed, proved to be the downstream construction method by hydrocycloning. In order to be able to make maximum use of the capacity of this site the tailings dam has to have a final height of 130 m above stream bed at the downstream toe, with a possibility to raise it to 145 m. The projected cross section of the tailings dam is shown in Fig.2.

The upstream starter dam was erected of heavily weathered moscovite-biotite gneiss borrowed from a quarry in the reservoir site, since the open-cast mine was too far off to allow the use of pit waste rock material. The height of the starter dam was determined on the basis of the expected relation of cycloned sand to slime, presuming a 1:3.5 downstream slope of the sand body (by analogy with the Brenda tailings dam in Canada, ref.1) and a 1:2.0 upstream slope. The percentage of the cycloned sand during the first 5 years was expected to be 50% of the total tailings quantity and further on 30%. The 1:1.5 upstream slope of the starter dam was screened with clay up to half its height.

The downstream toe dam is 30 m high and is filled of fresh granite as a permeable body, allowing draining of the sand. Two drains, each 10 m wide, are laid along the entire length of the tailings dam under the sand part between the two dams parallel to their axes. They are designed, just as the toe dam, to drain mainly the processing water carrying the cycloned sand over the temporary downstream slope off the sand part.

The slurry is discharged from the plant with

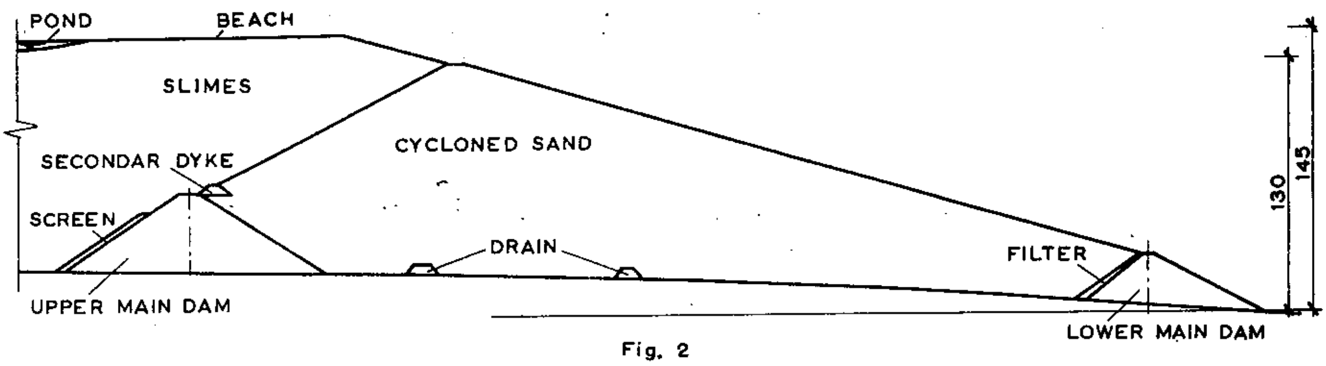

Fig. 2

density of 1.16 to 1.19 t/m^3, which means that one weight unit of tailings corresponds to 3 to 3.5 weight units of water. The specific gravity of the tailings is 2.70 g/cm^3. The slurry flows freely from the plant along the right bank of the tailings dam in concrete troughs, and only in one steep section - in two pipelines. From the high right bank to the starter dam it flows down under pressure in two pipelines of 500 mm diameter with inside basalt coating. The distributing steel pipeline along the starter dam crest, which feeds the hydrocyclones, in 1989 had the same length as the starter dam, but at the final stage it is to be 2000 m long. The work proceeded consecutively in sections along the pipeline. At the start two sections were set up, and in 1988 they were already four. The diameter of the pipeline in the first section and the first half of the second one is 700 mm, then it becomes 630 mm. At the beginning (in 1981) 75 hydrocyclones were installed on the starter dam crest. In the eighth year of operation (1989) 130 hydrocyclones covered the length of 1600 m. The diameter of the hydrocyclones is 500 mm tapering under an angle of 20°, the diameter of the sand nozzle being 46 mm. They are coated with rubber lining. At full capacity of the plant up to 35 hydrocyclones operate simultaneously. The average water pressure of the cyclones was projected to be 2 bars.

The secondary dams are deaigned to be 5 to 6 m high, of cycloned sand. The distribution pipeline is moved on each secondary dam, and the hydrocyclones are raised at each 1.5 m (Fig.3). First the second part of the pipeline is raised

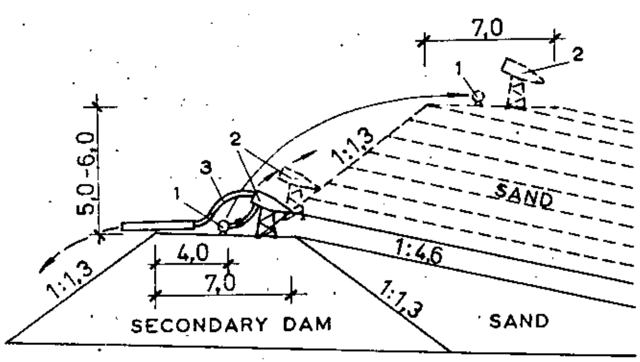

Fig. 3

1 DISTRIBUTION PIPELINE
2 HYDROCYCLONE
3 SLIME HOSE

and then the first one.

In the final 15 m of the tailings dam height the relation of the necessary sand with respect to the slime will be higher to the extent that it will be possible not to cyclone any more, but to spigot the entire quantity of the tailings after the upstream construction method. The balance will have begun to be possitive several years before that. In order that the sand part does not overtake inadmissibly the slime part, cycloning is to be stopped periodically with spigotting continuing with the entire quantity of slurry. Thus a layer of coarser material is to be deposited in the slime part, which will drain the slime and accelerate its consolidation. As a result stability will be improved.

Stand-pipe piezometers of PVC pipes dia.63 mm are provided only in the sand part to monitor the phreatic surface.

After settlement of the hard particles the clear water is discharged over three slope collectors located in the ground folds, into the opposite bank of the pond (Fig.1). They consist of concrete troughs and are covered with concrete slabs, thus raising the spillway sill and controlling the pond level.

Under the starter dam, the sand part and the toe dam, the flow continues into steel pipes dia.800 mm in reinforced concrete casing. These discharge into the reservoir of the recirculating pump section, located downstream of the toe dam. At their end they are provided with slide valves to stop the flow when necessary (for concreting the slabs or in case of emergency). The seepage water caught in the drains is also discharged into the pump station reservoir.

Water is discharged always from the spillway most distant from the site of slime discharge.

OPERATION OF THE TAILINGS DAM

The tailings dam was put into operation at the beginning of 1981. Changes in the structure and the technology had to be introduced during the operation since not everything could be implemented as planned.

The plant was commissioned in stages. The first year it begun to work with 50% of its capacity, in the second - with 75%, and only in the third - with 100%. During the first 2.5 years the plant functioned under different and varying conditions, which hampered the work of the hydrocyclones. Besides that, experience had to be acquired during the first year. The expected separation of 50% as coarse fraction could not be achieved. During the first year the hydrocyclones separated only 42% of the

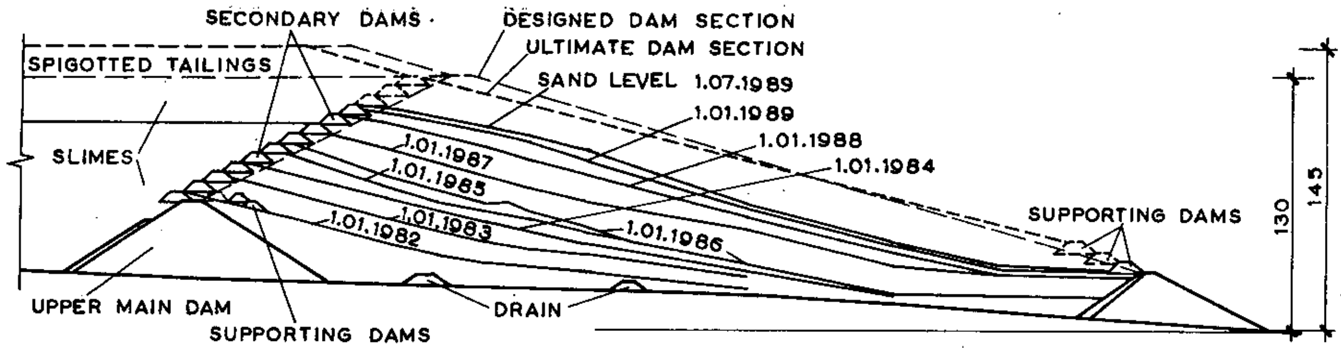

Fig. 4

tailings as coarse fraction. Since they did not operate regularly, the separated cycloned sand amounted to only 35% of the entire tailings quantity. The remaining 65% were discharged upstream of the starter dam as the slime. Besides that the cycloned sand was deposited with a flatter slope. Fig.4 shows the slopes of the sand deposited in the course of the years. In the upper part the slopes are about 1:3. while in the lower part they reach up to 1:10. Figure 4 reveals that the sand part was not raised uniformly every year, although the annual tailings quantities were almost equal. This was so for each specific profile, since operation proceeded consecutively on separate sections, and the changing of the sections did not coincide with the sequence of the years. In order to secure that the rise of the sand part overtook the slime, it became necessary to use rockfill for the 6 m high first secondary dam. It was laid on the starter dam and on the slime since the sand part had remained lower. For the same reason the second and the third secondary dams were built of cycloned tailings only up to a height of 3-4 m, while their upper parts, 2 m high, were made of rockfill borrowed from a quarry.

In the second section the cycloned sand part lagged more behind because of the topographic conditions and for organizational reasons. Therefore it was necessary there to construct two suppprting dykes of borrowed rockfill,3 to 3.5 m high, laid on the temporary sand slope,at a distance of 45 m from the starter dam (Fig.4). With their help the level of the sand part was temporarily raised.

During the first year the cyclones separated 42% of the total tailings quantity in order to maintain the sand part higher than the slime. During the following years the percentage of separation by the hydrocyclones was gradually reduced, in the sixth year being 36%, and in the eighth - 32%. The actually separated sand in the first year was 35%, in the sixth - 31.7% and in the eighth - 28%. During the first year there exhisted the danger of the sand remaining under the slime level. For that reason the secondary dam had to be made of borrowed rockfill. Gradually the balance was improved, during the second year the sand body overtook the slime by 2.30 m and in the sixth year the sand was already higher by 8 m. In order not to increase the level of the sand above the slime by more than 8 m, which endangered the upstream slope stability of the sand part, from the seventh year onwards spigotting was conducted with the entire tailings quantity over the slime. This direct spigotting with the entire tailings was increased every year.

Since the 1:3.5 downstream sand slope could not be acheived, it was decided to build upstream of the toe dam 3 secondary rockfill dams each 5 m high (Fig.4).

The following tests and measurements were carried out on the tailings dam. The average diameter of the sand particles ranged from 0.20 to 0.26 mm, the particles smaller than 0.1 mm being 25 to 35% and the coefficient of uniformity varying at about 6. The slime particles had an average diameter of about 0.04 mm.

The pressure in the first operating hydrocyclone was maintained within 2.0 to 2.3 bars. The pressure in the last hydrocyclone was 1.6 to 1.9 bars. The first hydrocyclone separated sand of an average diameter of 0.25 to 0.26 mm, and the last - 0.20 to 0.23 mm.

The hydrocycloned sand flew out of the cyclones as slurry with high density; one sand weight unit of sand corresponding to one weight unit of water. With this density and with the high permeability of the sand, the water was quickly drained away and no substantial segregation of the sand occured. This is evident in Fig.5. The slime was segregated along towards the pond, since after the first year a non-submerged beach was always maintained, which in 1986 was 60 m long, and in 1989 - 150 to 200 m. The decrease of the average slime diameter towards and in the pond at the end of 1986 is shown in Fig.5.

The cycloned sand was deposited with a low density. The samples taken from the surface of the sand slope indicated an average bulk density of about $\rho = 1.4$ g/cm^3. The void ratio was about e = 0.95 to 1.05 and the average relative density $I_d = 0.41$ at a bulk density of $\rho_{max} = 1.81$ g/cm^3 and $\rho_{min} = 1.21$ g/cm^3. The samples taken from the sand surface had an angle of internal friction of $\varphi' = 26^o$ to 32^o, cohesion $c' = 0.1$ to 0.2×10^5Pa and permeability k = a x 10^{-3} to a x 10^{-4} cm/s. The compression tests give a void ratio of e = 0.9 and permeability of k = a x 10^{-5} cm/s for a vertical load of 5 x 10^5Pa, and e = 0.7 and k = a x 10^{-6} cm/s for a vertical load of 14 x 10^5Pa. Therefore it could be assumed that in depth the bulk density and the relative density are higher.

The slime had a permeability of k = a x 10^{-5} cm/s at the beginning of the beach, and further inside the beach k = a x 10^{-6} to a x 10^{-7} cm/s,

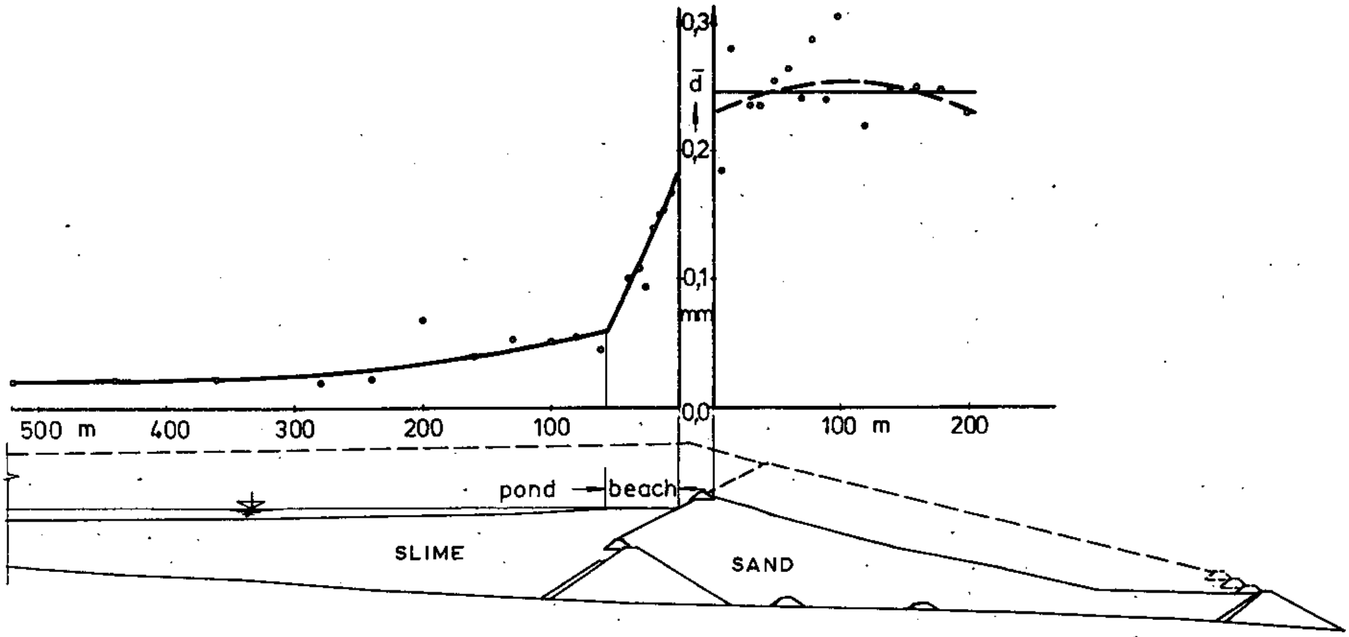

Fig. 5

respectively angle of internal friction $\varphi' = 22^{\circ}$ to 15°, and cohesion $c = 0.2 \times 10^5$ to $0.35 \times 10^5 Pa$.

The cycloned sand settled mainly in the upper part, while in the lower part it remained considerably below the ultimate dam section. For that reason after the seventh year the nozles were adjusted to a somewhat wider opening, to alow the sand to flow out with more water, so as to be conveyed by gravity into the lower sand part. This leads inevitably to an encrease of the fine fractions in the cycloned tailings. However this compromise was necessary to shape the required profile of the tailings dam.

The stand-pipe piezometers indicated a water level not higher than 1 to 2 m above the ground. This phreatic surface was maintained mainly by the water flowing with the sand from the cyclones. The phreatic surface in the section which was being cycloned was by 2 to 3 m higher. This low phreatic surface ensured the seismic stability of the tailings dam.

Since the upper layers of the sand part creeped downwards over the downstream slope, the piezometers tilted and got broken. This made it necessary to replace them with new steel pipes, which however also tilted.

The drains under the sand part functioned well. They lowered the phreatic surface and conveyed a total discharge of up to 80 l/s of drained water.

In the third year of operation a permanent rotation sprinkler system of a range 45 m was set up to reduce dusting from the temporary downstream slope. The system was laid on this same temporary downstream slope. In the process of building up of the sand body, the horizontal feeding pipes were deformed and broken due to settlement of the deeper sand layers, and the vertical pipes - due to creeping of the sand. Rotation sprinkling line proved to be ineffective during continuous strong wind. The system was changed to a semipermanent one with sprinkler laterals shaping vertical water screens. The purpose of the screen was to catch the dust and simultaneously to moisten the slope. However it also proved to be inadequate because of the large area of the dusting surface - 63 ha. Now a new method for chemical and chemical-biological stabilization of the remporary downstream slope is being developed.

CONCLUSION

The downstream method of tailings dam construction by hydrocycloning is highly reliable and economical. In this case it was the most appropriate. Inspite of its merits, it raised the problem, that in the initial stages the sand fraction was not sufficient to support the slime. Dusting was more intensive and the desired final downstream slope was difficult to shape.

Not everything could be predicted in the design. In the course of operation changes had to be introduced consisting in building additional supporting dykes, changing the method of shaping the ultimate downstream slope, as well as its draining and stabilization to resist the effect of water and wind, both during operation and after completion of the tailings dam.

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9. Waste retention embankments on soft clay

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SYNOPSIS

To evaluate the preliminary design for embankments on soft clay which was based on parameters from insitu and laboratory tests a trial embankment was constructed and monitored. The results of these trials enabled a re-evaluation of the design with considerable savings on fill material.

INTRODUCTION

1. The Tioxide Group plc is a leading manufacturer of titanium dioxide, a white pigment used in paint, rubber, plastics, textiles, toothpaste and many other products. The Company is preparing to open a new plant at an industrial site on the east coast of Malaysia in Terengganu state.

2. The major waste by-product of the manufacturing process is gypsum and iron hydroxide initially produced as a slurry. This slurry is to be dewatered by a series of hydraulic presses producing a 'cake' of gypsum and iron hydroxide, and a generally clear filtrate of water which contains mainly gypsum (calcium sulphate) in solution.

3. Before approval for the construction of the factory could be given the entire process was subject to a strict environmental impact study. An outline scheme proposed for the handling of the gypsum and wastewater was approved by the Malaysian Department of the Environment. This scheme provided for the containment of the solid gypsum within a controlled landfill area, retained by impermeable embankments. The wastewater is to be directed through a series of lagoons to enable monitoring of the water and to allow any remaining suspended solids to settle before final discharge to the environment. Because of the high rainfall in the area (3000mm/annum) the lagoons were also designed to receive run-off from the landfill site which would contain some resuspend gypsum material.

THE SITE

4. The factory site is located just north of the town of Chukai in Terengganu State (see Fig. 1). After visits and investigations at several potential storage areas the wastewater treatment plant was chosen to be sited in the area adjacent to the factory (Fig. 2) because it was known to be underlain by clays. This would provide an impermeable base to the lagoons and landfill area, avoiding the need to install an artificial and expensive impermeable lining. This area was however flat, low lying and swampy with an approximate elevation of 3.0 m above mean sea level and subject to flooding. Drainage from the swamp westwards was via a recently excavated channel leading to the small Sungai Ruang and the main Kemaman estuary.

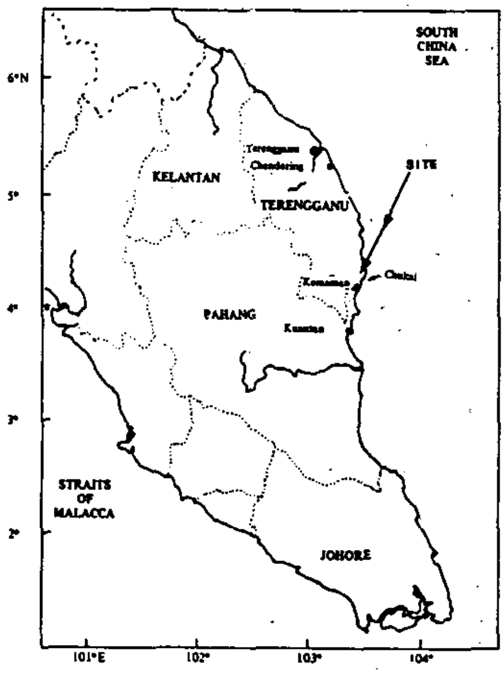

Fig.1 - Site location

Geological setting

5. The factory site was known to be underlain by recent alluvial deposits. The flat swampy area was surrounded by forested hills consisting of Lower Carboniferous metasediments (phyllites, slates, quartzites and schists) with intruded granites of Upper Carboniferous/Lower Permian age. These rocks also form the bedrock underlying the alluvial deposits. The alluvial deposits are predominently soft black clays (believed to have been deposited within the last

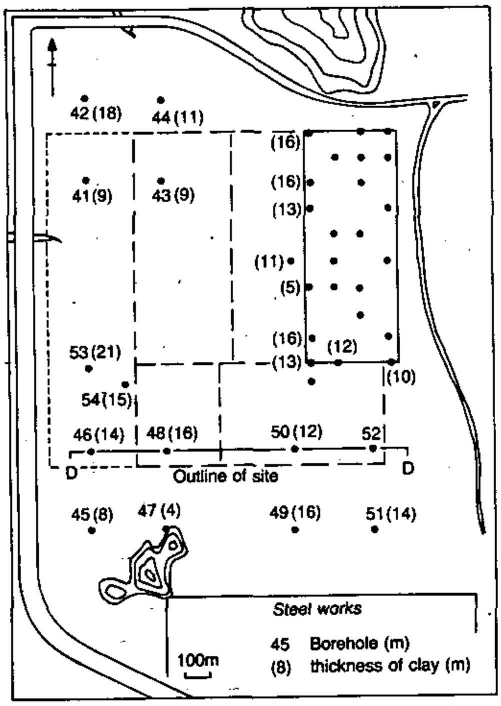

Fig.2 - Waste water treatment site

10000 years), underlain by sands and stiffer clays representing an older deposit.

6. These soft clays are of marine origin and are widespread throughout the Far East but are particularly common in the Mekong Delta (South Vietnam and Cambodia), Chao Phraya delta of Thailand and on the coastal plains of peninsula Malaysia, Sumatra and Java. They can reach a maximum thickness of 30 m at the coastline but thin inland.

7. They have been studied in recent years particularly by Cox[1] among others, mostly in relation to the construction of highway embankments.

Site Investigation

8. A detailed site investigation was implemented in stages. Initially boring was carried out to assess the thickness of alluvial materials with sampling and testing to evaluate soil properties. Use was also made of a large volume of data from an earlier investigation at the actual factory site which included boring and Dutch Cone testing. Second stage investigations included dynamic probing aimed purely at confirming the continuity of the soft marine clay across the site. In the final investigation stage, 4 boreholes were put down at the location of the trial embankment (see below). In 2 of the holes continuous 100 mm diameter piston sampling was carried out through the soft clay and in the remaining 2 holes field vane test at 0.3 m intervals using the Geonor penetration vane apparatus. A simplified section of the site is shown in Fig. 3.

Marine Clay Properties

9. At the proposed waste retention and lagoon site the general soil profile consisted of 8 to 20 m of very soft to soft greenish grey silty clay with traces of sea shell and fine sand. The upper one to two metres generally consisted of a dark brown peaty clay or occasionally a clayey peat with decayed vegetation, roots and wood fragments.

10. Typical soil profiles and index properties are shown in Fig. 4 together with the results of the corrected in-situ vane shear tests.

11. The clays are of high to extremely high plasticity with high liquidity indices. Particle size distributions indicate between 20 and 50% of clay size material and 5 to 15% fine sand. The in-situ vane shear strengths were corrected using the method of Bjerrum[2]. The corrected vane strengths are generally consistent for the top 7 m at about 10 kN/m^2 below which they increased linearly with depth with a Cu/Po' ratio of about 0.35 to 0.40. The shear strength profile indicates a weathered crust with a depth of *weathering of about* 5 to 6 m similar to the profiles for recent clays in Bangkok[2] but deeper than might be expected for clays adjacent to the coastline.

12. The results of oedometer tests and consolidation properties of the soil are shown in Fig. 5.

13. The pre-consolidation pressures (or critical pressure) appear to increase with depth although there is considerable scatter of points with the over consolidation ratio varying from 0.3 to 4.5 The lower values undoubtably reflect sample disturbance.

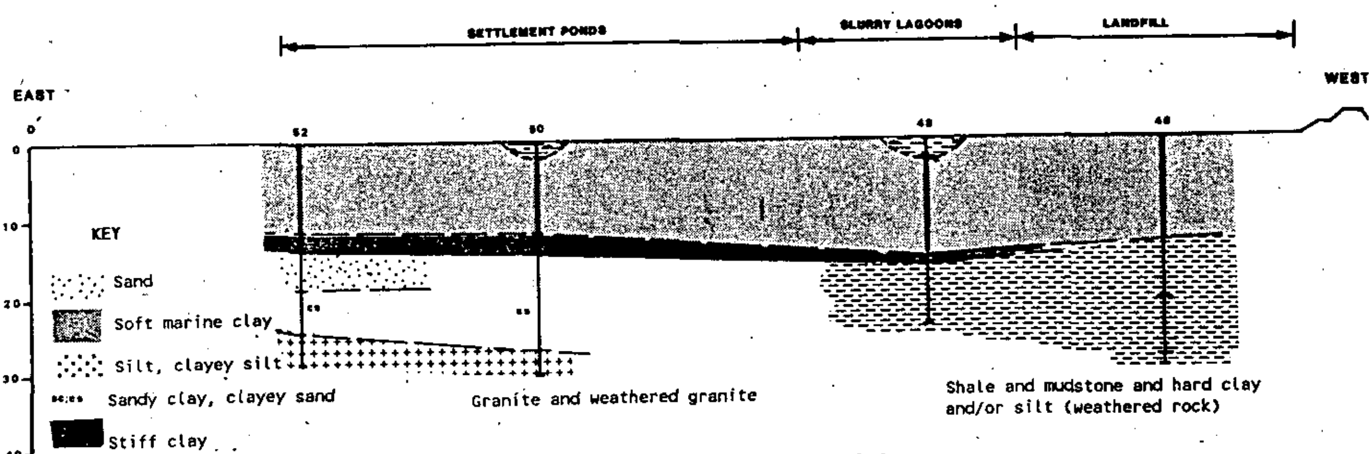

Fig.3 - Simplified geological profile

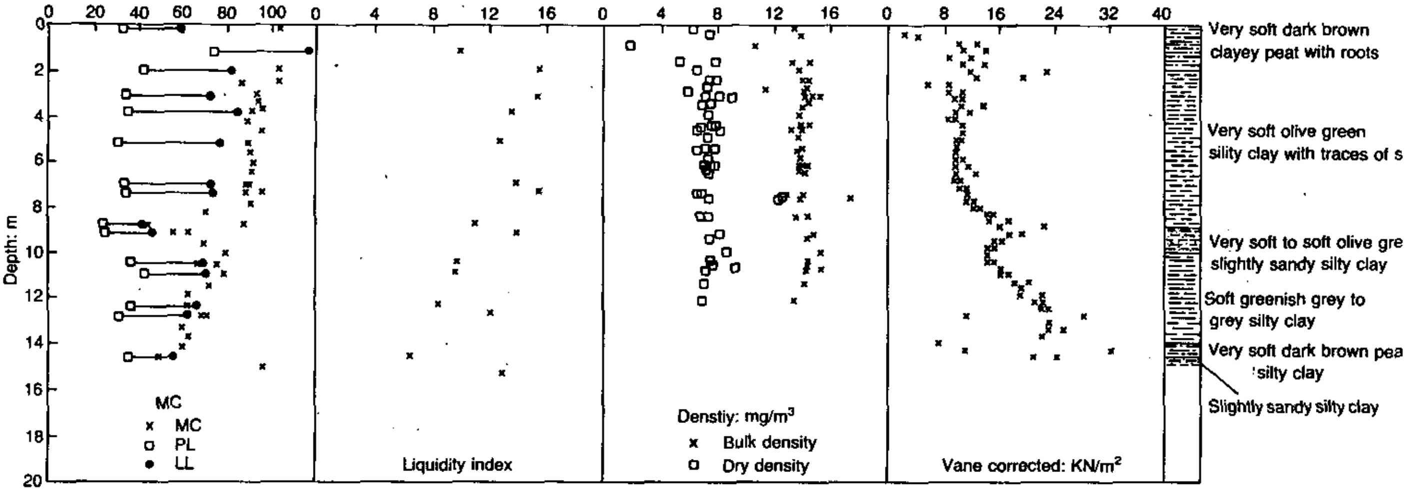

Fig.4 - Index properties

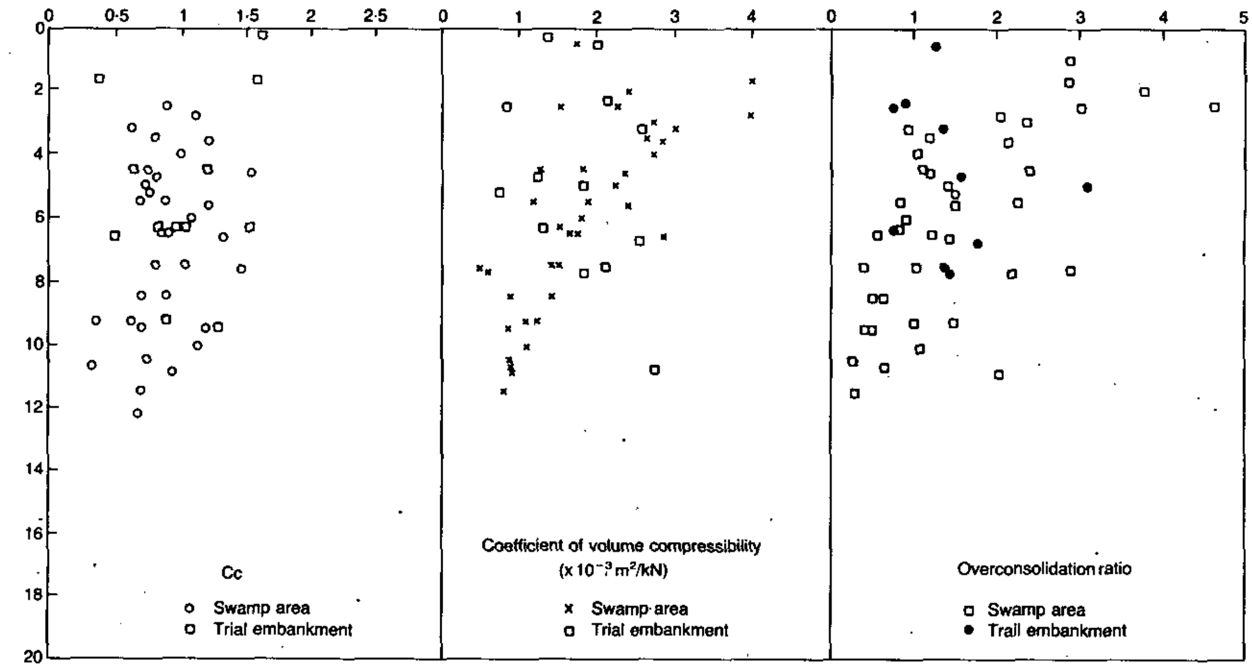

Fig.5 - Consolidation properties

PRELIMINARY DESIGN

14. The scheme called for embankments for the lagoons with a minimum crest height of 2.0 m and for the embankments around the proposed gypsum landfill a height of 2.5 m. Behind the latter embankments the gypsum is to be stored in compacted layers rising to a height of about 12 m. The purpose of these retaining bunds was to contain run-off from the gypsum landfill and direct flow to the settlement and monitoring lagoons. The embankments are to be constructed with completely to highly weathered granite and metasediments with a central core zone taken down below root zone. A drainage blanket 0.5 m thick is to be provided under the shoulders of the embankment.

15. The stability of the embankment at various stages of design was analysed using Sarma's[3] method for both circular and non circular failure surfaces. The strength profile adopted for the analysis is shown in Fig. 6. At the time of the analysis not all soils data was available and strength points showed some scatter. The strength profile adopted was generally conservative.

16. The following assumptions were used in the analysis:

1) The marine clay layer was taken to be 16 m thick.
2) Soil profile as Fig. 6.
3) Embankment to be built in stages due to soft sub-soil conditions, stage 1 to 2.0 m followed by a 'rest' period of 1.75 years with subsequent raising to 2.5 m.
4) Foundation was assumed to improve with time due to consolidation.
5) Minimum factor of safety adopted to be 1.40.

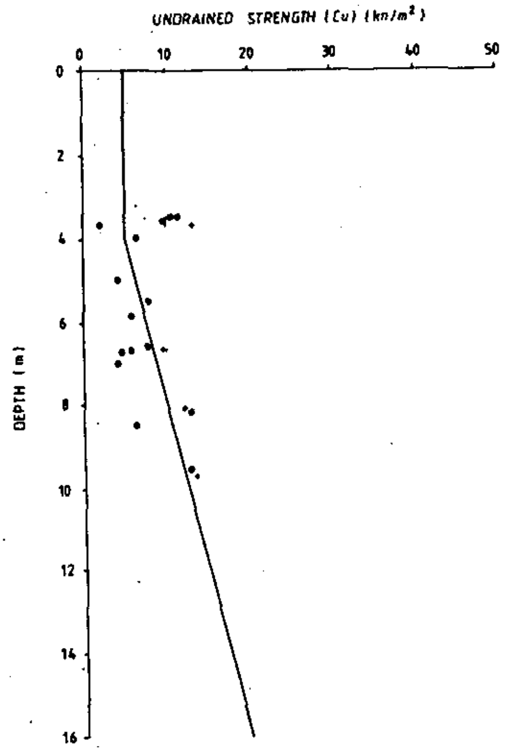

Fig.6 - Design Strength profile

17. The embankment profiles adopted for preliminary design are shown in Fig. 7. This can be compared with profiles of the surrounding highway, an example from Bangkok, and the subsequent trial embankment dimensions.

18. The total final settlement profile of the embankment due to consolidation of an underlying 16 m thick layer of soft clay was calculated to be 0.70 m under the crest of 2 m high bank. The rate of settlement was extremely difficult to calculate since assumptions were based on limited data. The coefficient of consolidation (Cv) was based on a few field permeability test results and oedometer tests. A lower than average value of field permeability was used in the calculation of Cv to allow for the reduction in permeability, especially in the upper parts of the marine clay, due to the increase in effective vertical stress and subsequent decrease in void ratio. It was therefore expected that the initial short term settlement would be faster than calculated. At this preliminary design stage the effect of overconsolidation of the crust or weathered zone was not taken into account. Using a value of Cv of $2m^2/yr$. it was estimated that the degree of consolidation of the marine clay after 2 years would be 27%.

TRIAL EMBANKMENT CONSTRUCTION

19. Stability analysis showed that the minimum factor of safety was very sensitive to the shear strength values assumed for the underlying marine clays. This was particularly true at shallow depths where the critical slip surfaces are located. It was also felt that the strength values adopted were conservative and may underestimate the in-situ material strength of the clays. In addition due to sample disturbance there was not great confidence in the degree and rate of settlement calculated for the embankment profile. It was therefore decided to confirm design assumptions by constructing and monitoring a trial embankment.

20. The trial embankment was located close to the proposed area for the settlement lagoons. Four boreholes were put down at the site for detailed in-situ testing and sampling as described above. The average ground elevation was approximately 3.2 m above mean sea level.

Site preparation

21. Clearing and cutting of the site was done manually and trees hand felled. Roots were left in the ground providing an undisturbed foundation for the embankment.

Instrumentation

22. The following instruments were installed in the foundations of the embankment to monitor the behaviour of the foundation during and after construction of the embankment:

- 12 pneumatic piezometers (P)
- 4 surface settlement plates (SP)
- 4 deep settlement plates (DP)
- 20 deformation markers (DM)
- 10 theodolite stations (TS)

23. The layout and sections were shown in Figs. 8 to 10

24. The pneumatic piezometers used were suitable for low and medium pressure range accurate to ± 0.2 m head. Prior to installation they were soaked and tested for leaks on site under a pressure of 20 m head. They were installed by lowering into boreholes using a PVC placing tube and pushed in for the final 300 mm to the required tip level. The boreholes were subsequently backfilled with 3:1 bentonite/cement grout. The tubing from the 12 piezometers was laid in a trench and terminated at a terminal panel and read-out gauge housed in timber building 15 m from the embankment.

25. Surface settlement plates were constructed of 50 mm dia galvanized steel pipe welded to 600 mm x 600 mm steel plate (5 mm thick). Each length of pipe is 1 m long with screwed couplings for extension and is provided with a 100 mm diameter

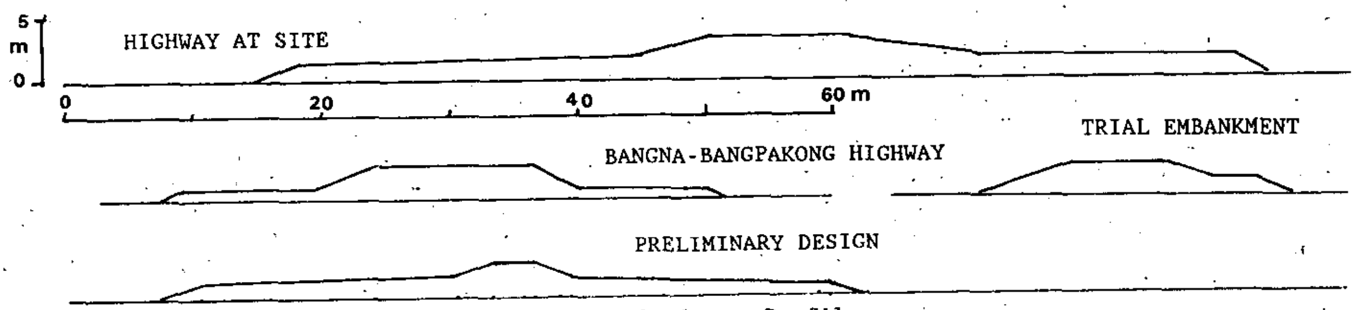

Fig.7 - Embankment Profiles

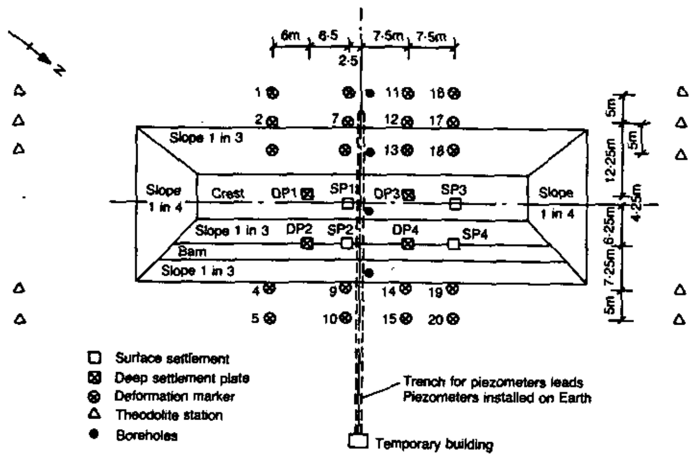

Fig.8 - Trial embankment plan

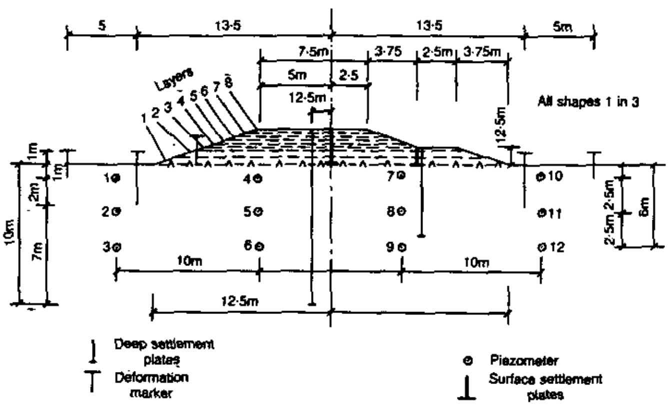

Fig.9 - Trial embankment section

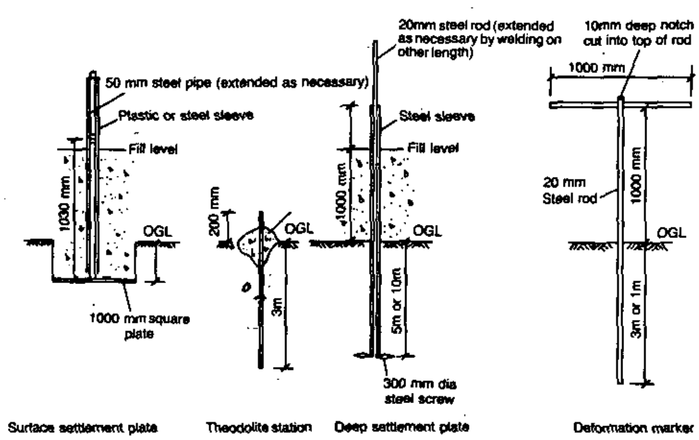

Fig.10 - Instruments

steel sleeve. Deep settlement plates were constructed of 25 mm diameter high tensile steel rods with a 300 mm diameter steel screw at the base (Fig.10). Monitoring of settlement was by precise levelling and referenced to two temporary bench marks outside the trial embankment area.

26. Deformation markers were driven beyond the toe of the embankment and on the embankment slope to measure lateral deformation. Measurement of the lateral movement/displacement of the markers was by precise sighting with a theodolite referenced to stations outside the embankment area.

Fill Material

27. The fill material was well graded very gravelly sand (predominently coarse sand and fine gravel) with less than 5% fines. During construction in-situ density tests were carried on each fill layer. Bulk density ranged from about 1.64 Mg/m^3 to 1.96 Mg/m^3 with a mean value of 1.80 Mg/m^3. Moisture content was about 5%.

Construction of Embankment

28. The embankment was to be constructed in layers. The first two layers were 0.5 m thick and the remaining 6 layers each of 0.25 m bringing the embankment to its design height of 2.5 m. Due to time constraints it was not possible to allow long periods of time for monitoring between each layer and on average only 3-4 days was available between placing layers. The embankment was brought to its final height in 35 days.

29. Fill was brought onto the embankment by 6-wheeler trucks along an access road made up of 0.5 m thick sand. Fill was tipped at the edge of the embankment and was spread out using a back pusher and bulldozer of self weight 1 tonne and 4 tonnes respectively.

MONITORING OF TRIAL EMBANKMENT

30. Generally the reading of all instruments was carried out on a daily basis. With the exception of piezometer P4 and deep settlement plate DP3 all instruments functioned satisfactorily. Piezometer P4 was later replaced with a new set of piezometer and leads. Unfortunately between the placing of the 4th and 7th layer the piezometer read out unit malfunctioned and a number of readings were lost.

Piezometric levels

31. Piezometric levels are shown plotted as excess pore pressure in Figs. 11 and 12. Piezometers P5 and P6 placed under the zone of thickest fill responded during the placing of the second layer (0.5 m - 1.0 m) with a maximum excess pore pressure of 2.5 m - 2.7 m. Piezometer P4 (1 m below original ground level) responded but due to apparent rapid dissipation of pore pressure maximum excess head obtained was 0.5 m. A similar effect was noted for piezometers P7-P9 (Fig.11). The piezometers placed beyond the toe of the fill (P1-P3 and P10-P12) showed negligible response.

Settlement

32. The settlements of the surface (SP1 - SP4) and deep plates (DP1 - DP4) are shown in Fig. 13 As with the piezometers negligible response was noted until the placing of the second layer commenced. Maximum settlements, by day 64 under the thickest fill section was 0.58 m and 0.64 m representing about 23 to 26% settlement. At a depth of 10 m below the bank settlement was about 5%.

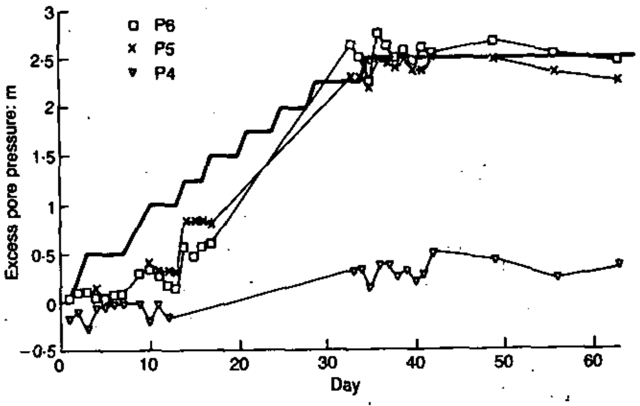

Fig.11 - Piezometric levels

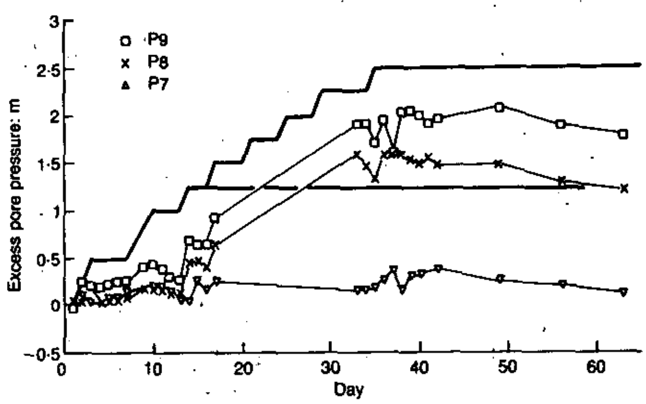

Fig.12 - Piezometric levels

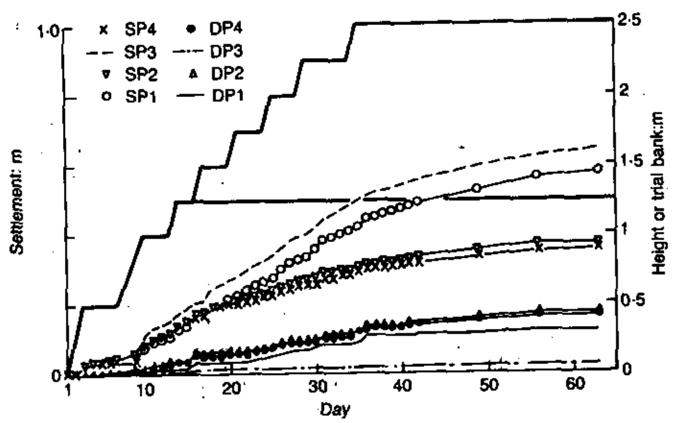

Fig.13 - Embankment settlements

<u>Deformation</u>

33. Movements of the deformation markers are shown in Figs. 14 to 18 Measurements were taken on movement of the top of the marker and base. During the early filling stages there was some tilting of the markers but after the filling of the third layer movements were generally uniform.

34. Deformation was surprisingly small at the toe of the main slope (DM 2, 7, 12 and 17) with initial movement towards the bank after placing the third layer (10-30 mm) followed by deformation away from the toe. The line of markers 5 m away from the toe (DM1, 6, 11 and 16) showed very little movement after an initial displacement of 20-30 mm after placement of the first layer except D11 which showed a sudden movement during the placing of the third layer (maximum lateral displacement of 80 mm).

35. Deformation markers at the toe of the lower

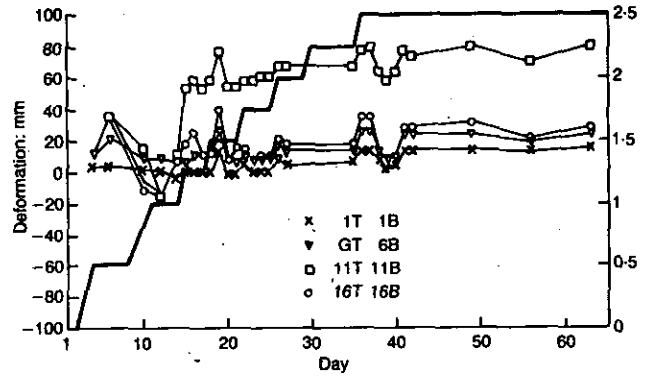

Fig.14 - Embankment deformation

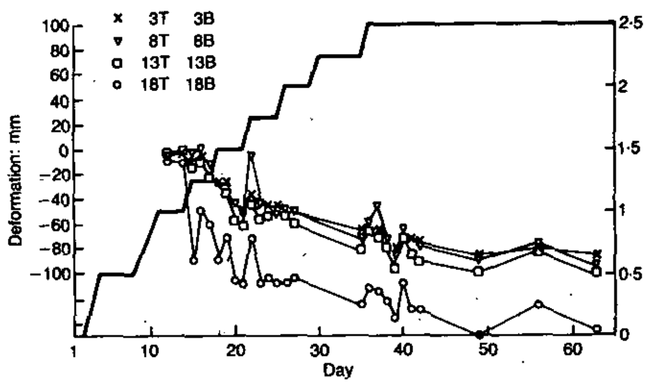

Fig.15 - Embankment deformation

slope all showed a positive movement away from the bank with a maximum displacement ranging from 36 to 78 mm. Five metres from the toe displacements were negligible after placement of the first layer.

DESIGN REVIEW

36. The preliminary embankment design was drawn up on the basis of the shear strength profile shown in Fig. 6. This resulted in very flat slopes. The trial embankment was constructed to very much steeper slopes, 1 on 3 one side and 1

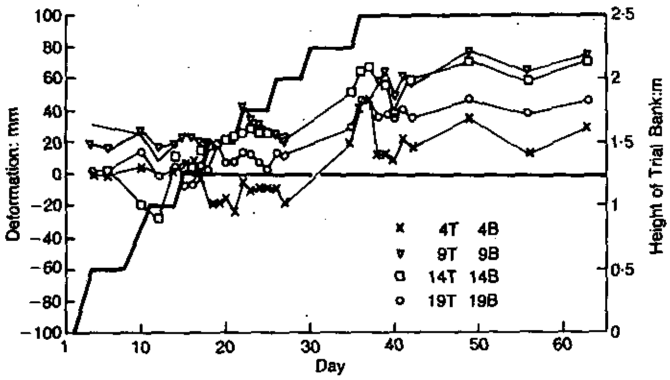

Fig.16 - Embankment deformation

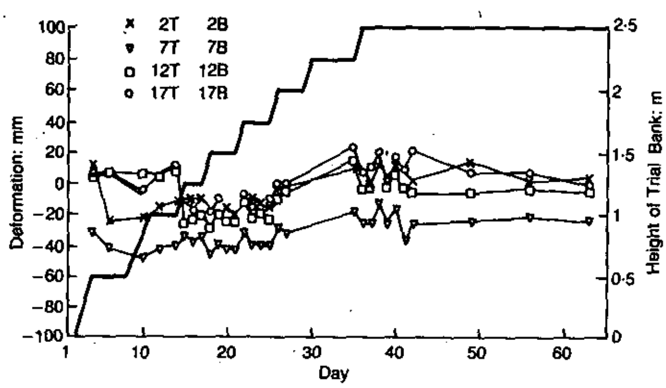

Fig.17 - Embankment deformation

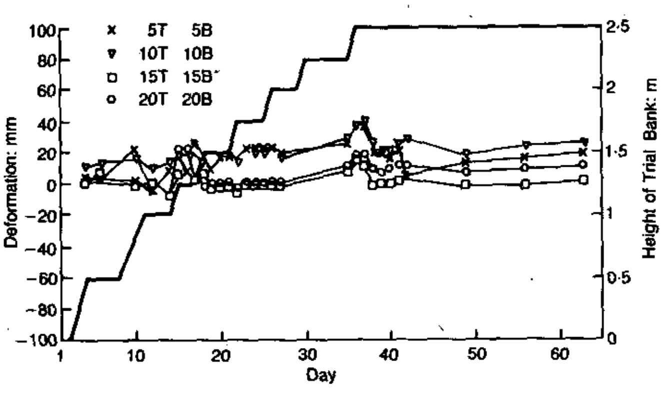

Fig.18 - Embankment deformation

on 3 with a 2.5 m berm on the other. It was therefore expected that failure would be reached before the final height of 2.5 m was obtained.

37. In the event the embankment did not fail so it could therefore be assumed that the factor of safety against shear failure was greater than unity. The margin of safety was, of course, not known. A back analysis showed that a shear strength profile developed from the vane tests done on the site gave a factor of safety of just over one. However, the embankment did not show signs of distress in the form of excessive horizontal deformation that would be associated with embankments built on soft clay and close to failure.

38. Previous experience[4] with trial embankments on soft clays demonstrated that a factor of safety of at least 1.4 was necessary for acceptable performance. Below this margin, delayed failure was possible. It was therefore considered likely that the trial embankment had a similar margin of safety. For this to be the case, the strength of the foundation clay would have to show an increase in strength over the measured vane strengths. Confirmation of this was found in the piezometer readings. Piezometer P4, at a depth of 1 m, and under the highest part of the embankment, recorded a piezometric level equivalent to a pore pressure ratio (r_u) of 0.21 when the embankment was completed. Since the theoretical (r_u) without consolidation would have been expected to approach 0.7 or 0.8, it was clear that considerable consolidation had occurred. A similar behaviour was observed at piezometer P7, the other piezometer at shallow depth under the embankment. The piezometers at a depth of 3.5 m also show some consolidation but non was apparent at the piezometers 6 m deep. This is consistent with the length of the respective drainage paths.

39. It may be noted the consolidation described above showed as a maximum settlement of the foundation of about 600 mm.

40. The above conclusions were applied to the preliminary embankment design by increasing the strength adopted for the top 1 m thick layer of the foundation from 5kN/m^2 to 7kN/m^2 to reflect the increase in effective stress in this layer. Strengths below this layer were not changed. This modification enabled the widths of the berms to be decreased by 5 m with a saving of about 50,000m^3 of fill , justifying and demonstrating the value of trial embankments.

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10. Tailings deposition predictive computer modelling

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SYNOPSIS

The construction and infilling of tailings depositories are dynamic processes. Operational efficiency depends on the form, method and control of impoundment. Accurate prediction of infilling allows designers to assess: life of depository; reservoir area; spillway requirements; rate of wall construction required; and effect of different deposition modes. Development, testing and application of the WLPU tailings deposition model (TADAM) is described. The program is based on over fifty years' experience of tailings dams around the world and is designed to maximise flexibility and minimise data manipulation. Studies at two mines illustrate the macro and micro scale modelling potential of the program.

INTRODUCTION

1. The confining wall for a tailings depository is a unique structure, in that it is usually built from the same material that it is designed to impound. Finely processed mine waste is mixed into a slurry and hydraulically transported to an overland storage site. Discharge of material takes place from discrete locations around the depository perimeter. Material flows away from the point of discharge and is deposited. The rate of filling and structural development of the dam wall is thus controlled by the operation of the discharge points. Tragic failures of tailings structures in recent years have focused attention on their safe design and management.

2. Tailings impoundments are under continual construction throughout the period of their service lives, typically 30 or 40 years or more. Impoundment is therefore a dynamic process and the efficiency of deposition will be influenced by the form that the impoundment takes. Prediction of the life of a tailings depository can be of great importance to the economic viability of a mining project. Too short a life of depository will necessitate future major capital expenditure to provide additional storage. Too long a life of depository implies inefficient use of capital during the cash intensive pre-production phase.

3. Control of the tailings dam pond size and location, as well as the freeboard between pond surface and dam wall are essential elements of depository management. Appropriate filling patterns must be specified by the engineer responsible for technical management of the tailings structure. In reality, any localised encroachment of the pond will be dealt with by the mine operators, who will open or close the appropriate valves, or move the appropriate discharge points, based upon their individual experience. This is, however, a short term solution; long term mis-management of filling could result in inefficient usage of available storage, or even structural instability. The efficient use of the available storage is of major importance to the overall mining operation, and therefore longer term waste disposal management approaches need to be adopted.

4. The authors have developed a software package called "TADAM" to model the hydraulic disposal of tailings within overland storage basins. This program provides a method of predicting long and short term disposal effects and is an efficient and cost effective tool for the design of tailings dams.

DESCRIPTION OF TADAM PROGRAM

5. TADAM uses an incremental generation procedure to generate successive stages in the development of a tailings depository. The program is written in FORTRAN and runs on a VAX 8200 Computer. Run times are normally one to two minutes.

Input data

6. TADAM utilises a topographical database containing elevations of the "original" ground surface on a rectangular grid, typically at 25, 50 or 100 metre spacings. This database is produced by digitising the results of topographical surveys and is stored as a binary file named LOWFILE.

7. The position of any confining wall is stored in a separate binary file named WALLFILE. This minimises the amount of data manipulation involved in modification of the wall location.

8. The extent of the area available for tailings deposition is defined in a third binary file, called VALIDFILE.

Output files

9. Deposition (or dredging) generates a new tailings surface which is stored in a new binary file called TOPFILE. This file is interchangeable with LOWFILE and will probably be used as the new base surface for future runs of the program.

10. ZONEFILE contains the zones of deposition for each deposition point. The main output listing is stored in the ANSWERFILE, this includes a summary of the volumes and tonnages of deposition classified by deposition point and full listings of LOWFILE and TOPFILE. The output listing is cumbersome and is rarely referred to; a summary of total tonnage and volume of deposition is displayed on the screen at the end of each run and screen/hard copy graphics facilities are available.

Parameter file

11. Operation of the program is controlled by a simple parameter file which contains the minimum possible amount of data and therefore allows an experienced operator to run the program, assimilate the results and edit the input data ready for the next run within a cycle time that should not exceed five minutes. The parameter file contains: wall and pond elevations; density and slopes of tailings (dredged, subaqueously deposited and subaerially deposited); and names of the input and output binary data files.

Slope of deposited tailings

12. The behaviour of mine tailings during hydraulic deposition has been discussed by Blight and Bentel (ref. 1) and other authors. The slope of the tailings beach is influenced by many factors including: specific gravity; coefficient of uniformity; moisture content of delivered tailings; depth of phreatic surface; rate of discharge per unit length of embankment; rates of rainfall and evaporation; and even wind speed. Blight and Bentel compare dimensionless beach profiles for four different types of tailings, these indicate that the profile of copper tailings beaches is only slightly concave and can be approximated as linear. Diamond, platinum and gold tailings produce increasingly concave non-linear beach profiles. WLPU's database of tailings slopes, accumulated from over fifty years of experience, supports these findings.

13. TADAM incorporates flexible and sophisticated facilities for the specification of tailing beach slopes. At the conceptual design stage, linear approximations are often used, typically 1:200 or 1:100 for sub-aerial deposition, 1:30 for subaqueous deposition and 1:15 for dredged slopes. Detailed design and project monitoring normally require the use of non-linear profiles, these may be based on laboratory trials or surveys of existing deposition or may be selected from an extensive database of beach profiles that have been used on similar assignments. TADAM's slope database comprises surface elevations and tailings densities at 50 m increments from the point of deposition, intermediate points are interpolated using cubic curve fitting.

Modus Operandi

14. In the first iteration, TADAM examines each deposition point in turn by evaluating adjacent grid points to identify file sectors, with that deposition point as origin, where deposition is possible. The grid points defining the edges of these sector(s) are entered into the deposition table for the next iteration as potential deposition points. The cumulative distance from the real original deposition point is also entered into the table; this distance accurately simulates the real flow path and is the key to the successful implementation of the program. At the end of the iteration the deposition table is examined and points from which no deposition occurred are eliminated. Each subsequent iteration examines the points one grid step further out.

15. The elevation of the deposited tailings at each grid point is determined by referring the distance from the deposition point to the stored beach profile. Deposition can continue, in a straight line, until the elevation of the deposited tailings would be below the ground surface or below the pond level. Grid points where these conditions occur are identified as "hillpoints" or "waterpoints" respectively and these points become virtual deposition points for further deposition.

16. Hillpoints allow the tailings deposition to follow the topography of the impoundment and are probably the most significant improvement over the various "straight line" deposition models.

17. Waterpoints mark the transition from subaerial to subaqueous deposition. They are unlikely to lie exactly on the edge of the pond and the program makes an adjustment to allow for this lying between the grid points.

18. Volumes of deposition are calculated by re-examining the valid area and multiplying the difference between the new tailings surface and the original ground surface at each grid point by the grid spacings. Volumes are apportioned to the real deposition points using ZONEFILE.

TESTING OF THE TADAM PROGRAM

19. Durrant (ref. 2) has described an earlier program, written by the senior author, called "DUMPS" (ref. 3) and his own research work using the Medusa Geographical Information System. The Medusa Geographical Information System is a relatively expensive package which is unlikely to be economically viable for modelling tailings deposition.

20. Durrant has described a series of tests, designed to demonstrate the limitations of DUMPS and the advantages of a more sophisticated modelling system. His five test cases have been repeated using TADAM, the results for a single deposition point with a re-entrant corner are presented in Figure 1. These tests show that DUMPS (and any other simple deposition model) is unable to model hills and re-entrant corners with acceptable accuracy. TADAM can model these features and is a practical alternative to the Medusa Geographical Information System.

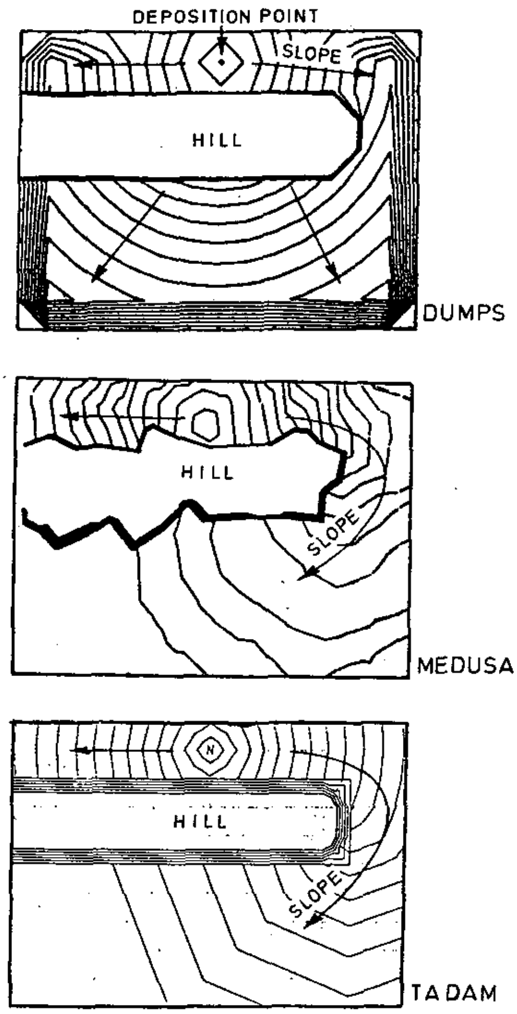

Fig. 1 Contour plots of DUMPS, Medusa and TADAM tailings deposition models for discharge around a re-entrant corner.

APPLICATION OF THE TADAM PROGRAM TO SOHAR COPPER PROJECT, SULTANATE OF OMAN.

21. The Sohar Copper Project is situated 27 km inland from the coast in an area of very limited water resources. Its location is shown in Figure 2. As a result the Oman Mining Company LLC have to rely to a large extent on seawater make-up for the process plant.

22. The mine is required to operate its tailings depository in the Wadi Suq in a manner which will prevent the pollution of adjacent fresh water aquifers.

23. In 1984 WLPU was appointed to advise on methods of maximising the quantity of tailings stored in the dam and the development of a system for recycling the tailings pond water and minimising both the seepage from the reservoir and the importation of seawater (ref. 4).

24. The Sohar Concentrator produces about one million tonnes of tailings each year. Estimates of the size of the existing ore bodies indicate that the plant will operate for about eleven years. At present the tailings dam impounds about 5 million cubic metres of tailings and a maximum of 400 000 cubic metres of free water.

25. TADAM has been used for computer modelling of the filling of the dam with the aims of achieving the optimum deposition pattern and of restricting the passage of seepage water, high in dissolved solids, throughout the underlying gravels. Water balance studies were made in parallel with the filling trials to determine the scope for recycling water from the pond and reducing acid generation therein.

26. In 1984 the pond volume was about 230 000 m^3, the pond level was 245.3 m and the edge of the pond was about 200 metres from the crest of the embankment. Computer aided filling trials indicated that continuation of the filling regime then in use would result in a pond volume of 1.3 million m^3 by December 1989 and a pond level of 256.1 m. WLPU recommended the implementation of pond water recycling which was forecast to result in a pond volume of 25 000 m^3 and a pond level of 251.9 m by December 1989.

27. The anticipated advantages of pond water recycling were:

i) The small pond, distant from the embankment, would result in a large beach with maximum sub-aerial deposition, and hence a higher average density of tailings.

ii) The small pond would result in a larger volume of the depository being available for the storage of tailings rather than water.

iii) The small size of the pond and its distance from the embankment would reduce the volume of seepage loss under the dam wall and hence reduce the pollution risks.

iv) The small pond would result in a reduction in the acidity of the pond water making it more suitable for recycling.

28. In December 1989 the actual pond volume was estimated to be 50 000 m^3 and its level was 250.81 m (ref. 5). The tailings dam had performed just as predicted by the 1984 computer modelling. The pond was confined to the north west extremity of the tailings dam and the edge of the pond was about 600 m from the crest of the embankment as shown in Figure 3. The beneficial effect of the smaller pond on the rate of seepage is demonstrated by the reduction in the rate of pumping of water from the interception trench (Figure 4.)

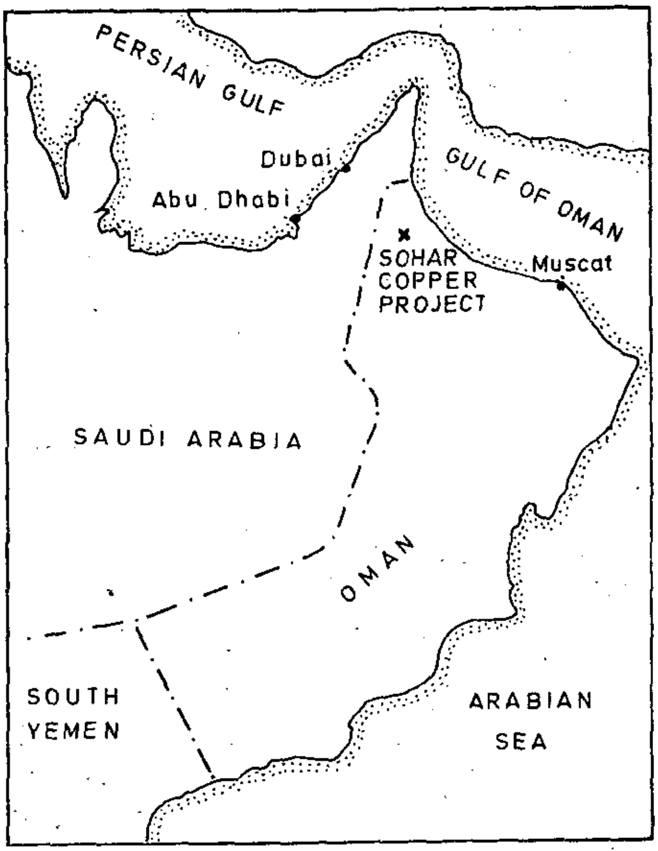

Fig. 2. Location of Sohar Copper Project.

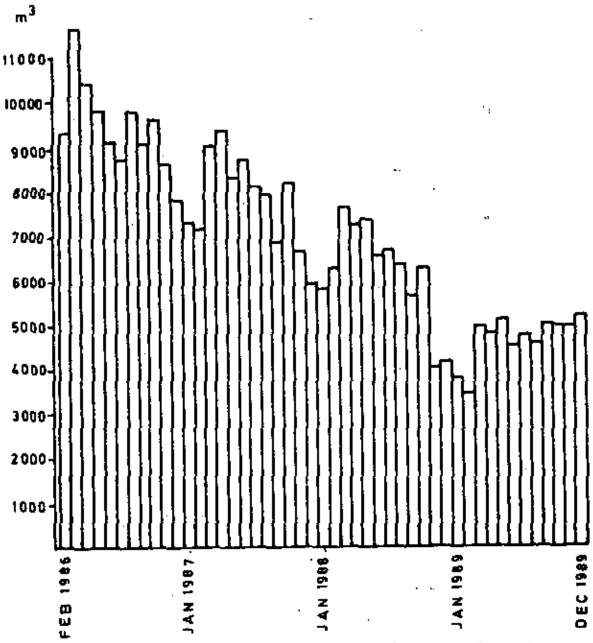

Fig. 4. Sohar Copper Project, volume of water pumped from interception trench to plant; February 1986 - December 1989.

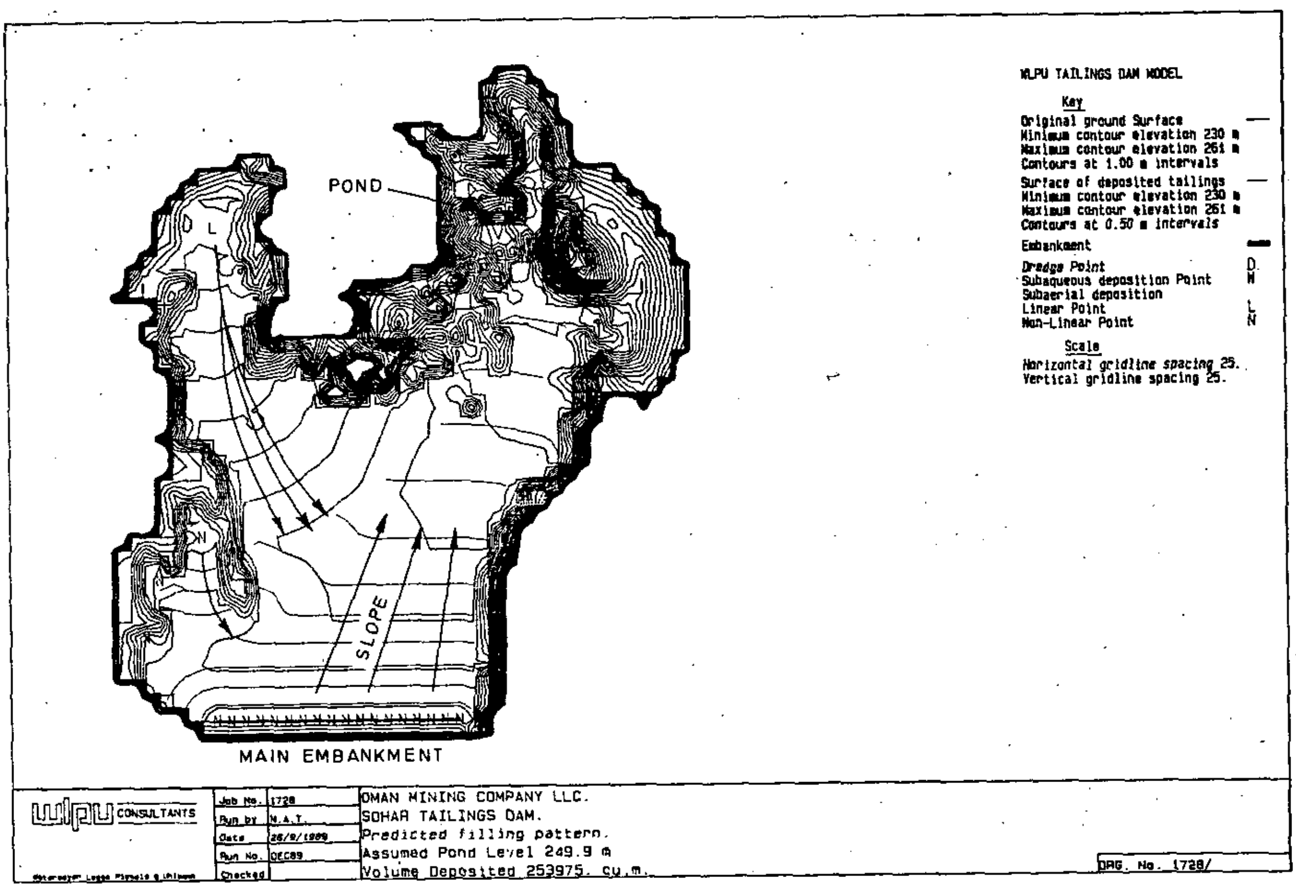

Fig. 3. Sohar Copper Project : TADAM results for December 1989.

APPLICATION OF TADAM PROGRAM AT ANDINA MINE, CHILE

29. The Andina Division of CODELCO, Chile's state copper mining corporation is situated in the Andes, close to Aconcagua, the highest peak in the Americas. Its location is shown in Figure 5. The division mines and concentrates some 40 000 tonnes per day and has two existing tailings disposal dams in rugged mountain valleys at altitudes above 2000 m, overlooking the township and mine support facilities. Disposal is currently to Los Leones dam, a conventional earth and rockfill structure which requires to be raised periodically to create additional tailings storage capacity. At present the dam stands some 160 m high and its heightening by conventional methods is becoming increasingly costly.

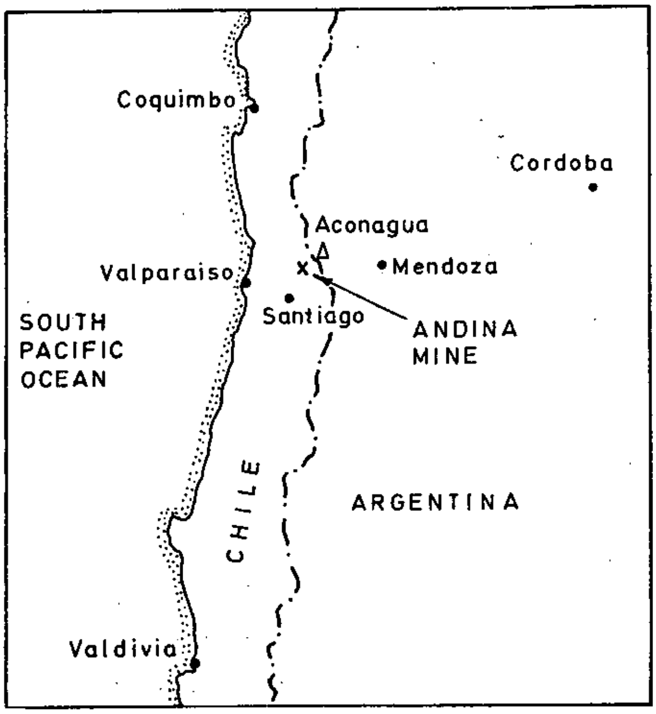

Fig. 5. Location of Andina Mine.

30. WLPU was commissioned to carry out a preliminary study of alternative tailings disposal technologies which might be applied to the existing sites to reduce costs without loss of security (ref. 6). The sites are seismically highly active and there is extensive agricultural, residential and infrastructural development downstream.

31. Five unconventional techniques were examined. These, and various combinations of them, were applied to the two valley sites to create nine disposal options, each of which would provide an additional 30 years storage capacity, amounting to some 430 million tonnes of solids. The nine options were developed to pre-feasibility level and subjected to comparative economic analysis. Single-site schemes emerged more favourably than multiple developments and the further raising of Los Leones dam, by the centre-line technique using compacted hydrocyclone underflow as fill, was identified as the preferred option on both technical and economic grounds.

32. The TADAM program was used to model all the options for tailings deposition in the Los Leones and Rio Blanco valleys. It also provided depth-capacity information for flood routing studies.

33. Original ground levels were digitised from 1:5 000 and 1:10 000 maps onto four 50 x 50 m grids, representing the existing Los Leones and Piuquenes depositories and two additional areas. The model was calibrated by comparing the volume of tailings as "predicted" by TADAM with the actual volume determined from a bathymetric survey. A slope of 1:350 for subaerially deposited tailings resulted in a modelled volume of 35.18 x 10^6 m^3 which corresponded to a measured volume of 37.2 x 10^6 m^3.

34. The favoured option is shown schematically in Figure 6 and the TADAM results are shown in Figure 7. This option would involve, for the 30 year capacity adopted as the basis of the study, the gradual raising of the existing Los Leones dam to a final crest level of 2200 m, an increase of 90 m over its crest in Autumn 1989. Raising of the embankment would be by the centreline method using compacted cyclone underflow. This option would also involve the extension of the foundation downstream to a new minimum level of 1900 m giving a final overall height of 300 m. This would place the Los Leones embankment amongst the world's highest tailings dams. The total cost of the preferred scheme over the 30 year period was estimated to be US$ 197 million at end of 1988 prices, amounting to US$ 0.456 per tonne of tailings.

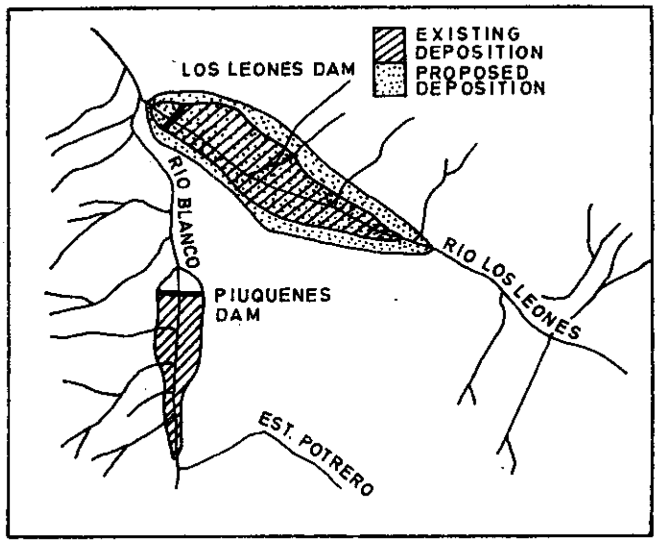

Fig. 6. Schematic Diagram of Favoured Option.

35. TADAM was also used to determine storage capacities for an unusual extension of this study. This involved an examination of the viability of using the raised Los Leones tailings dam for the seasonal storage of 40 x 10^6 m^3 of water for controlled release to augment seasonal low flows to a hydro-electric power station downstream. The additional cost of this option was estimated at US$ 34 million, which was thought unlikely to be matched by the potential economic benefits.

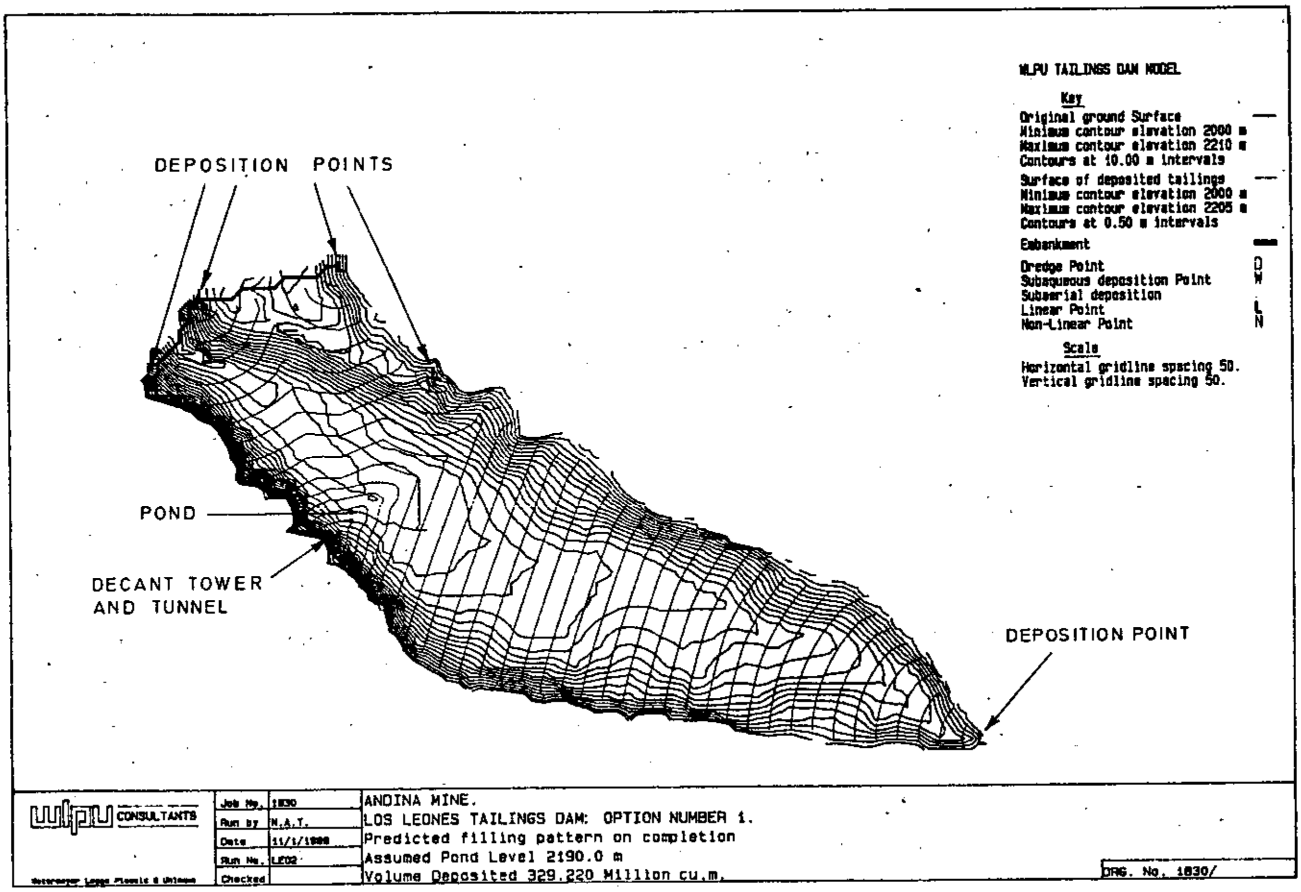

Fig. 7. TADAM Results for Favoured Option.

CONCLUSIONS

36. The dynamic processes involved in the construction and operation of tailings depositories have been outlined. The requirement for an efficient and accurate model of the deposition process has been demonstrated. Development and testing of such a model has been described.

37. The tailings dam model utilises linear or non linear beach profiles, digitised base topography and an unlimited number of deposition points, allowing flexibility during impoundment trials. Confining wall positions are specified in separate data files allowing maximum flexibility, minimising the need for laborious alterations to data and enabling a variety of wall construction methods to be assessed. The program uses virtual deposition points to model subaqueous deposition and topographical effects, it is therefore considerably more accurate than the previous generation of deposition modelling programs, including DUMPS.

38. Applications of TADAM have included tailings deposition and flood routing studies in Africa, Asia, Europe and South America. The accuracy and value of the program have been proven by its success in modelling tailings deposition on numerous projects including the Sohar Copper Project in the Sultanate of Oman and, on a larger scale, at Andina Mine in Chile.

Acknowledgements

The Sohar Copper Project is owned and operated by the Oman Mining Company LLC. Andina Mine is owned and operated by the Andina Division of Codelco-Chile. The permission of both Clients to publish this paper is gratefully acknowledged.

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11. Geotechnical aspects of the construction of tailings dams - two European studies

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The disposal of mine waste behind embankment dams has been practised for many years, but has been poorly served in the technical press. Well publicised failures and environmental concern have led to increasing use of specialist geotechnical engineers in the design process. The safe, efficient and environmentally acceptable deposition of mine tailings relies on engineering, not only of the confining wall, but of disposal methods. Two case studies are reported which indicate the requirement for an integrated multidisciplinary approach to design and stress the need for flexibility to suit mine development and the adoption of technology appropriate for the disposal environment.

INTRODUCTION

1. In the initial planning stage of a new mine attention is focused on proving the mineral reserve and on developing the ore extraction technology necessary to optimise recovery. During the economic and technical evaluation of the project therefore tailings disposal is invariably a secondary consideration. Moreover, the problems associated with the disposal of the waste product are often over simplified, and technology and design principles from similar operations adopted. This approach, once committed to paper, invariably leads to implementation and construction. It is only during full production that the deficiencies of the tailings disposal facility become apparent, often necessitating a review of the adopted disposal method. Major and costly modifications to the disposal system may then be required.

2. The development of a successful tailings disposal scheme involves a multi-disciplinary appraisal of the project in order to provide the mine with a safe, efficient, economic and environmentally acceptable form of storage.

Historical Perspective

3. To many mine operators tailings, the crushed fine waste rock emanating from the processing of the ore, have low priority and are of no commercial value. The tailings product for disposal will, as a result of the concentrating process, invariably be in slurry form often including a suspension of clay sized particles. These waste products have historically been deposited behind crude confining embankments where sedimentation of the product takes place enabling 'clean water' to be decanted into the river system or returned to the plant.

4. For many years, disposal was an unsophisticated process, the demands of production being insufficient to require any but the most simplistic of structures and, as a result, geotechnical design input, inspection and monitoring were minimal. Increased demand, coupled with the processing of lower grade mineral deposits, has led to progressively larger structures with an inherently greater hazard from failure. Much has been gained from the successful techniques and failures of earlier tailings disposal projects, and many lessons remain to be learned, particularly with respect to economic disposal methods, pollution control and the transfer of technology between different climatic zones. In recent years more attention has been paid to the geotechnical aspects of tailings dam construction which, in combination with increased environmental constraints, has led to the employment of specialists at an early stage of project planning. In many cases such technical input has only resulted following well publicised failures, and, due to increased public awareness and environmental concern.

5. The most positive aspect has been the introduction of legislation controlling design requirements, inspection and monitoring routines. This legislation was instigated in the UK following the Aberfan disaster and has formed the model legislation adopted in other countries, e.g. Zambia. All aspects of tailings disposal in the UK now fall under the control of the Mines and Quarries Inspectorate. It is impractical for the Inspectorate to fully appreciate all geotechnical aspects of tailings dam construction and it is necessary, therefore, that great reliance be placed on the advice of specialist geotechnical engineers. However, much of the available information has yet to be disseminated throughout the industry, and consequently the design of tailings dams still remains poorly understood by many practising dam engineers.

6. Many specialist engineers engaged on tailings disposal projects concentrate solely on the design of the retaining wall and neglect serious consideration of the mechanism of tailings disposition. This philosophy, whilst

providing a potentially conservative design, often does not provide the client with the most economic method of disposal and fails to consider the implications on restoration. The aim of this paper is to demonstrate, by means of two examples, the need to adopt an integrated approach to the design of a tailings disposal facility.

PRINCIPAL ASPECTS OF TAILINGS DAM DESIGN

7. The principal factors to be considered in the design of any tailings facility are:-

(i) Ore Characteristics-geology, mining method, previous mining on the site
(ii) Mill Product - concentration method, geotechnical characteristics of the tailings, slurry pulp density, throughput, plant operating hours and any future changes in the processing operation.
(iii) Disposal site - climate, geology, topography, availability of construction materials, hydrology, seismicity.
(iv) *Environment - existing habitats/* population, effluent discharge constraints (including dust and gas emissions), restoration and both long and short-term environmental impact.

8. In addition, the mine operator will require that sufficient storage be provided for the disposal of both proven and projected ore reserves in a safe manner and at a cost which is not detrimental to the viability of the project. During the planning stages, particular emphasis should be placed upon proving the environmental acceptability of the project to local inhabitants and to statutory authorities. Finally, the depository should have sufficient design flexibility to enable changes in grade, ore type, through-put and total storage volume to be accommodated.

9. Such design requirements are illustrated in the following *case studies*. The first demonstrates the need for flexibility, whilst the *second highlights* the problems inherent in adopting techniques from climatically *different regions*.

CASE STUDY NO. 1 - <u>Wheal Jane Tin Mine, Cornwall</u>

10. Cornwall has a long history of tin mining stretching back at least 2000 years and, as a result, old workings have a significant impact on the landscape. The mines to the west of Truro, the area of interest (Fig. 1), were worked from the 16th Century. 19th Century mining activity in particular resulted in the production of large quantities of waste, much of which has been deposited in the rivers and estuaries. The resurgence of mining in the 20th Century in response to rising metal prices, led to a revival of mining in Cornwall and to a reappraisal of tailings disposal policy. The construction of a tailings confinement structure in a tourism orientated County necessitated the design of an environmentally acceptable structure. Hence, when Consolidated Goldfields proposed the reopening of the Wheal Jane Mine on the site of the former Nangiles workings, an initial major concern was the impact of a large tailings disposal facility on the Cornish landscape.

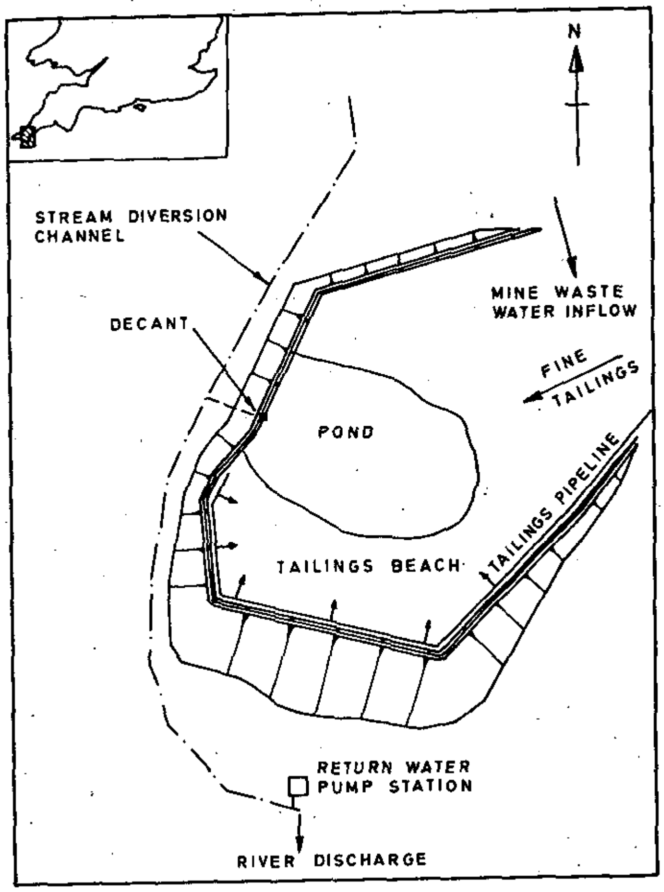

Fig. 1 Clemows Valley Tailings Dam

<u>Design Review</u>

11. WLPU first became involved in the project in 1968, at which stage it was apparent that planning consent for the mining project was dependent on the provision of a satisfactory tailings disposal facility in the adjacent Clemows Valley.

12. An initial appraisal indicated the main design criteria to be:-

(a) economic design utilising, as far as possible, the coarse fraction of the waste product in the confining walls
(b) landscaping to blend with the surrounds as far as practical
(c) effluent control by the design of a suitable settlement area to enable the sedimentation of the finest particles and *to meet consent limits imposed by* the Water Authority
(d) *geology with respect to the provision* of suitable dam foundations, the location of old workings and to minimise any possible risk of a breakthrough of mine tailings into the nearby workings
(e) hydrology, including the provision of appropriate river diversion works and spillways, and the disposal of mine waste (underground) water.

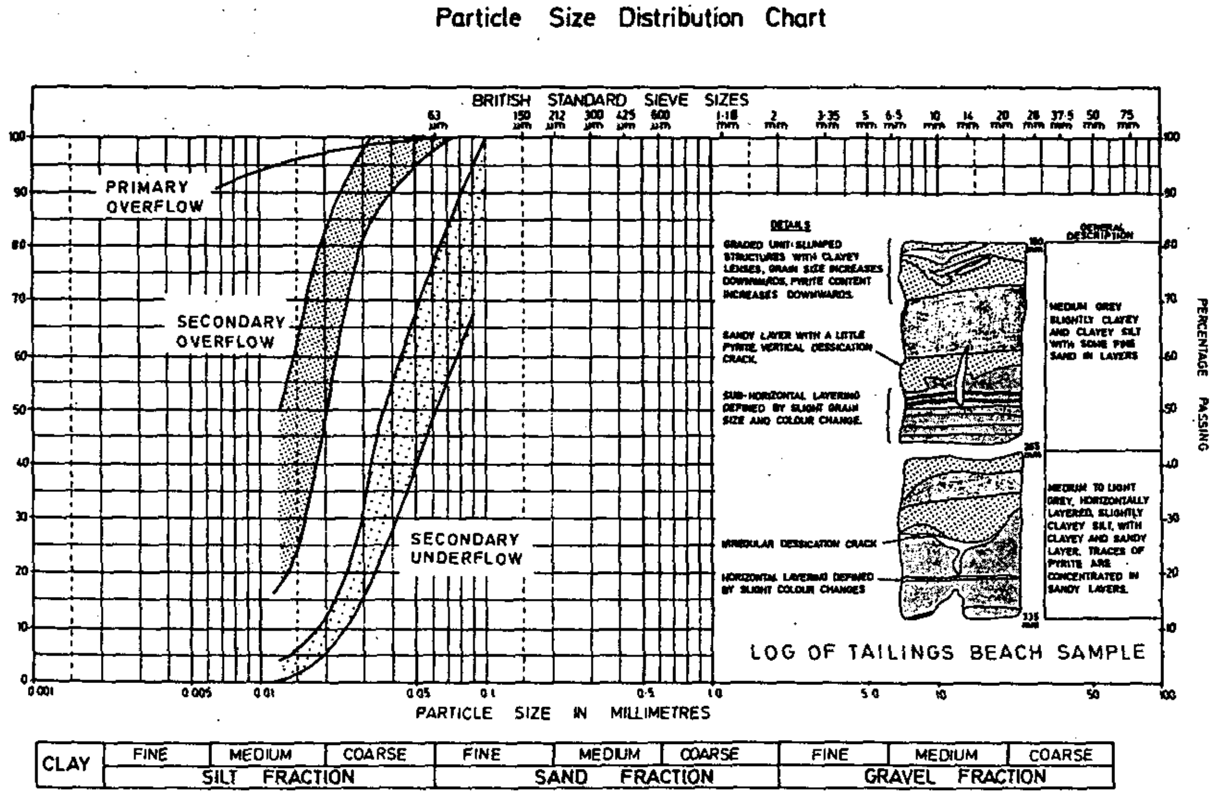

Fig. 2 Typical Tailings Gradings and Structure

Clemows Valley Tailings Dam

13. The initial design located the dam between the surface outcrops of the lodes in the Clemows Valley, the eventual position of the upstream wall being arranged to prevent deposition of tailings above the hanging wall of the productive ore body. The pre-deposition works included a starter wall constructed from compacted earthfill derived from the valley sides, a rockfill toe comprising mine waste, an imported crushed granite filter and appropriate decanting and emergency spillway facilities. The initial design included extensive use of the coarse fraction of the tailings product in the staged construction of the retaining wall. The classified tailings were to provide both the main structural portion and the free draining zone within the embankment, Seepage control through the dam section was to be provided by the coarse tailings, the filter system, and finally by the coarse rock toe laid on the valley floor.

14. Two waste materials (Fig 2.) are produced by the mine comprising a fine overflow and a coarse underflow and are delivered to the tailings dam from the primary hydro-cyclone facility located in the processing plant. The fine product, which predominantly consists of silt and clay sized particles, exhibits poor consolidation characteristics and due to the consequently low effective stress is of limited shear strength. This material is therefore deposited at the northern end of the depository at a location remote from the confining wall.

15. The underflow from the primary cyclone is pumped to a series of secondary hydro-cyclones located on the dam wall. Here the coarse fraction is separated for wall construction whilst the finer, intermediate sized material is subaerially deposited to form a tailings beach. As the slimes fraction has been removed by the primary cyclones, both the wall and beach materials rapidly consolidate and develop a high shear strength, ϕ' values of up to 43° have been measured in the laboratory. The deposition method results in a laminated beach material in which, due to anisotropy horizontal drainage into the higher permeability coarse tailings zone is encouraged and, as a result, consolidation is further enhanced. The tailings beach material is consequently predominantly unsaturated and of sufficient shear strength to enable the upstream method of embankment construction to be employed with the confining wall partially founded on the tailings beach.

16. Tailings deposited sub-aerially from the embankment walls form a gently sloping beach which consolidates rapidly to a density of up to 1.5 t/m^3, allowing access on foot to the edge of the pond. In contrast, the fine tailings deposited sub-aqueously from the northern periphery of the depository, settle to a much lower density of the order of 1.0 t/m^3 and possess insufficient shear strength to allow access.

Design History

17. This initial design was based on early predictions of ore grade, processing requirements, and mining methods. However, within two years of start-up in 1970, major process plant modifications necessitated the re-assessment of the structure. As a result of the smaller proportion of sand sized material

available, considerable field testing of the hydro-cyclones was undertaken in order to increase the efficiency of wall section construction. However, the finer grind produced by the mill reduced the volume of wall building material available and necessitated the revision of the dam section to incorporate a thicker downstream earthfill zone than the original facing proposed for landscaping purposes.

18. Since the first design review, further major modifications of the embankment cross-section and of construction methods have been necessary to accommodate subsequent changes in both mining and milling processes. These changes, which predominantly reflect the vulnerability of the mining operation to fluctuations in world metal prices, are illustrated in Fig. 3 and are summarised as follows:-

1976- Change in mining method with subsequent replacement of coarse tailings by a wide earthfill zone to form the main structural component, thereby allowing the use of the coarse tailings fraction for underground backfill. Inclusion of a chimney drain to replace the embankment drainage zone previously provided by the coarse tailings.

1977-78 Mine closure resulting in cessation of deposition during change in ownership, continuation of monitoring and preparation of abandonment proposals.

1979- Mine reopened and production recommenced.

1980- Major design review to enable full use of the available coarse tailings as a result of increased production, resulting in the reduction in the width of the earthfill zone. Reinstallation of the hydro-cyclones on the dam for coarse tailings wall building and embankment drainage, leading to termination of chimney drain construction.

1988- Purchase of South Crofty Mine leading to increase in mill throughput and the proposed increase in the final crest height by 6m to meet long term storage requirements from both underground operations. The increased coarse tailings available enabled optimisation of earthfill volumes and a more economic cross-section.

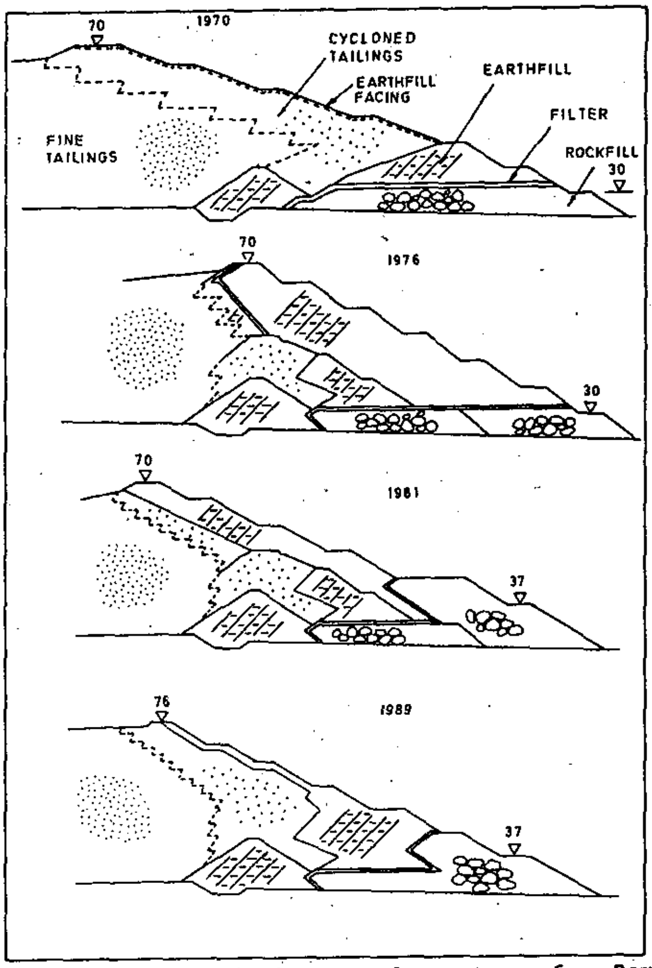

Fig. 3 Chronological Development of Dam Section

Summary

19. To date some 2.5 x 10^6m^3 of storage has been provided by construction of a 1.4 km long confining wall to a maximum height of 40m. Each change in mining or milling practise has been accommodated and the Clemows Valley Tailings Dam has provided an efficient disposal site for some 20 years. On completion of deposition to the proposed final crest level of 76m, the main embankment will be approximately 53m high, providing 5.8 x 10^6m^3 of storage.

20. At the time of writing, due to the recent downturn in the world tin price, underground mining at the site will shortly cease, although processing of South Crofty ore will continue in the short term. Unless metal prices substantially increase, tailings deposition at the site may cease. Restoration proposals may, therefore, be required to provide a suitable long-term scheme of landscaping and reclamation in keeping with the original permission.

21. The flexibility incorporated into the design of the Clemows Valley tailings dam has allowed the structure to be modified throughout its life. The use of the waste products in the embankment cross section has been continuously reviewed to ensure the economic construction of the confining structure and efficient disposal of the mine waste products. The disposal method adopted will also allow the relatively rapid rehabilitation of the site should tailings deposition cease.

CASE STUDY NO. 2 - Neves Corvo Copper Mine, Portugal

22. The complex copper ore body at Neves Corvo mine located near Castro Verde in Southern Portugal is an extension of the "pyrite belt" which runs across the Iberian peninsular. The Neves Corvo mine was identified in the 1970s and was commissioned some ten years later following establishment of a partnership between the Portuguese Government and RTZ (Somincor). The development of this mine, one of the richest copper ore bodies to be discovered in recent times, was undertaken on a design and construct basis by a combination of British, American and Portuguese companies with WLPU involved during the later stages of the design process as Review Consultant.

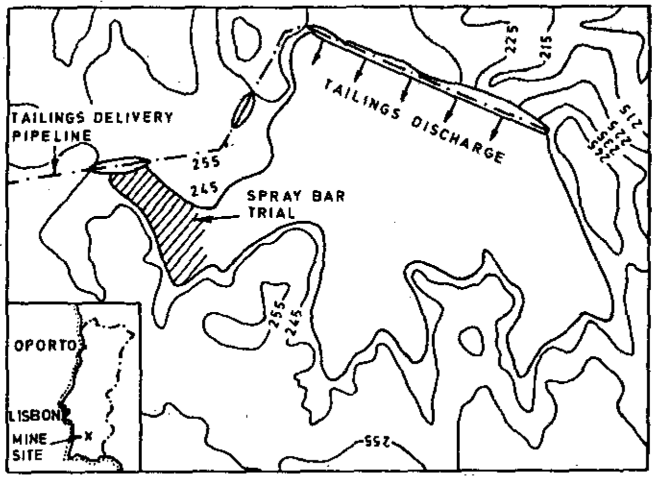

Fig. 4 Cerro do Lobbo Tailings Deposition

Cerro da Lobo Tailings Dam

23. The initial design of the tailings disposal facility was essentially dictated by discussions with the environmental authorities in Portugal who were concerned at the high acid generation potential of the predominantly pyritic tailings. Canadian tailings disposal technology was therefore adopted as most appropriate as the problems associated with the oxidation of similar ores, and the ready supply of water, have led to the adoption of facilities involving total inundation, ie. subaqueously placed tailings remain submerged.

24. The stage 1 Cerro do Lobo tailings dam (Fig. 4) was therefore designed as a conventional 28m high rockfill dam providing some $3 \times 10^6 m^3$ of storage, an additional $7 \times 10^6 m^3$ of storage was to be provided by increasing the dam height to 35m.

25. Initially, the dam core was designed as upstream sloping but following review of the geotechnical characteristics and the availability of the low plasticity clay, a re-design involving a central core was undertaken. For environmental reasons, a zero discharge option was selected and only an emergency spillway constructed. Construction of the dam was undertaken during 1987 and was completed by December in time to receive the first tailings.

Tailings Disposal Review

26. The WLPU review of the tailings disposal facility identified a number of major design items for further detailed study. However, of prime importance was the implication on both the deposited tailings density and final restoration proposal of adopting subaqueous disposal methods. In consultation with Somincor a major study was therefore put in hand to investigate alternative more efficient tailings deposition methods which would remove the onerous long term water demand on abandonment, improve densities and provide an environmentally acceptable method of operation. WLPU instigated a programme of field and laboratory testing to assess the potential improvements in tailings disposal density which could be achieved without significant detriment to the environment.

27. Initial laboratory studies indicated that the subaqueous deposition of tailings would result in the formation of a low density deposit requiring in excess of 300 years to consolidate under self weight. Little consolidation would have occurred at the end of the 20 year depository life and capping to restrict oxidation and permit satisfactory restoration would be impractical for many years. The Company were therefore faced with the prospect of maintaining the depository flooded in perpetuity with the associated provision of long term make-up water to replace evaporation and seepage losses.

28. The period of exposure required to allow access for restoration and capping could be substantially reduced by adopting subaerial deposition methods. Deposition of the tailings above pond level improves density, increases storage efficiency and accelerates the rate of consolidation as a result of both improved drainage and increased effective self weight. In climatically favourable areas the deposited density is also enhanced by evaporation of free water from the tailings surface.

29. The principal disadvantage in using subaerial deposition for the disposal of pyritic tailings is the risk of acid generation due to the oxidation of iron sulphide. Research in Canada (Ref 2 and 3) has shown that pyritic tailings can be satisfactorily deposited subaerially providing the tailings surface is only exposed to atmospheric drying for a limited period. Appropriate disposal methods can limit exposure of the tailings

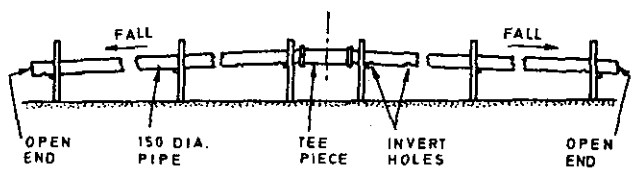

Fig. 5 Spray Bar Apparatus

surface by undertaking systematic deposition of the slurry by means of spraybars or similar techniques.

30. A typical spray bar is shown in Fig. 5 and comprises a 40 m long length of 150 mm diameter tubing with holes drilled at regular intervals along the invert. Tailings are fed into the spray bar via a valved tee connection located in the centre of the perforated pipe. The apparatus is mounted above the tailings beach by means of either timber posts or scaffold tubes. Flow into the spray bar and the gradient of the spray bar limbs are carefully adjusted to achieve a near uniform distribution of slurry throughout the length of the apparatus. Operation will then result in the uniform distribution of tailings slurry over the beach, effectively producing sheet flow within which the velocity is sufficiently low to allow sedimentation of both coarse and fine particles, thereby minimising the amount of particulate matter reporting to the pond.

31. Depending on the tailings production rate, a series of 5 to 10 spray bars would be operated for a fixed period on alternate sections of the beach allowing each area of deposited material to consolidate as a result of both drainage and surface evaporation. As each layer is of the order of 50 to 100 mm thick, consolidation rapidly occurs and by the time the next layer of slurry is deposited considerable densification has occurred.

32. Although the technique is principally used to enhance the deposited density, the method is also advantageous in controlling oxidation of pyritic waste. For the deposition of the Neves Corvo tailings, the controlling factor is, therefore, the control of acid generation, improvement of density being a further benefit.

33. To demonstrate the applicability of the technique in the semi-arid environment at Somincor, a series of both laboratory and field trials were undertaken

Laboratory Studies

34. The subaerial consolidation of a tailings layer was simulated in the laboratory by measurement of moisture content, density, shear strength and pH in a series of 50 mm thick slurry samples, comparative evaporation rates were derived from a control container filled to the same depth with water. The main conclusions drawn from this study were:-

(i) The pH of the initially alkaline tailings samples remained above pH 7 until the samples cracked (Fig. 6), at which point the pH rapidly decreased. The onset of cracking therefore provided a good indicator for subsequent deposition.

(ii) Densities of the order of $1.85t/m^3$ were achieved at the commencement of cracking, some 85% higher than the initial slurry density of $1.0t/m^3$.

(iii) Approximately 20 mm of evaporation took place from the water filled pan prior to the commencement of cracking.

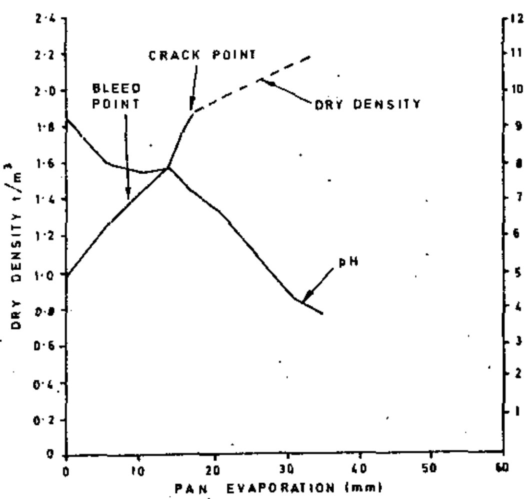

Fig. 6 Laboratory Evaporation Results

35. The laboratory results indicated that significant enhancement of tailings density could be achieved without the generation of an acidic tailings mass. To confirm these results a series of field trials were undertaken in a tributary valley of the tailings dam. Site evaporation rates indicated that, depending on the season, between 2 and 8 mm of evaporation occur each day. Allowing a suitable margin for operating error, a deposition cycle of between 2 and 5 days was proposed. However, to accommodate monitoring and operational requirements, a 7 day cycle was adopted.

Spray Bar Trials

36. A series of field tests were undertaken between August and October 1989 at which time the test programme was prematurely curtailed by abnormally high rainfall flooding the trial area. Measurement of pH and moisture content were carried out, where feasible, on a daily basis as summarised in Fig. 7, which shows average pH remaining above 7 throughout. Accurate measurement of dry density in the semi-solid tailings was extremely difficult, and an indication of density was inferred from the moisture content density relationship, further testing is in hand to confirm the field values.

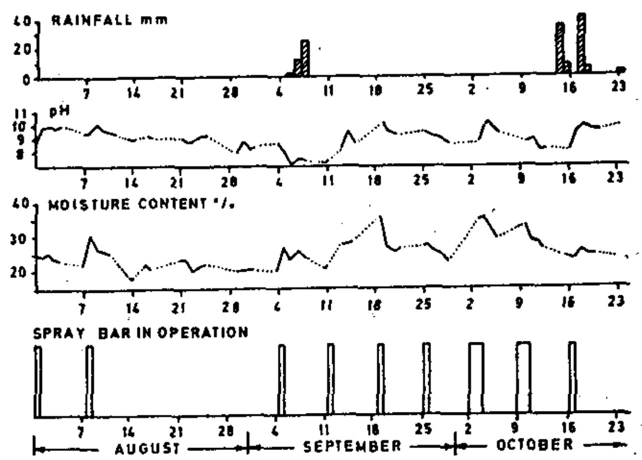

Fig. 7 Spray Bar Field Trial Results

37. The results of the field test clearly indicate that controlled subaerial deposition, by means of spray bars, could be undertaken, without the generation of an acidic tailings mass. In addition, the estimated tailings density for the study period which varied between $1.55t/m^3$ and $1.95t/m^3$ provided an average value of some $1.8t/m^3$.

Summary

38. The attainment of densities of this magnitude throughout the life of the depository would allow the storage of an additional 6 x 10^6t of tailings over that by subaqueous deposition and, more importantly, should enable the company to adopt an alternative restoration scheme for the depository.

39. The tailings deposition technique adopted at Somincor, although adequate to meet short term disposal commitments, will inhibit restoration. The subaqueous method of deposition selected is well suited to the climatic conditions associated with most of Canada, where long term water supplies for restoration by inundation are freely available. The method is, however, less suitable for use in a semi-arid climate in which evaporation exceeds precipitation and water supply involves costly pumping from remote reservoirs. The field trails indicated that subaerial disposal may provide a more appropriate method of tailings deposition for this mine.

CONCLUSION

40. The design of a successful tailings dam not only involves the consideration of technical aspects of the retaining embankment but also the implication of the selected disposal method on both final restoration and operational flexibility. To achieve both of these objectives the mechanism of tailings deposition must be understood, carefully simulated by laboratory testing and, when feasible, by field trials and continuously reviewed during the life of the disposal facility.

ACKNOWLEDGEMENTS

41. The Wheal Jane Mine is operated by Carnon Holdings Ltd and the Neves Corvo Mine by Sociedade Mineira de Neves Corvo S.A (Somincor). The permission of both Clients to publish this paper is gratefully acknowledged.

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12. Spillway systems for tailings dams

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During its operating life economics normally dictate that a tailings dam is constructed progressively, just keeping ahead, with an appropriate freeboard, of the deposited waste and supernatant water. The dam thus normally requires a spillway with a crest level which is at the current reservoir surface and can be varied as an operational procedure to suit deposition progress. The role of this structure, often termed a "decant" is crucial to successful disposal, safety and environmental protection. Some of the commonly used decanting structures are described, with reference to design points and pitfalls which have been experienced with each.

INTRODUCTION

1. The main object of tailings dam construction is the storage of solid waste brought in from outside - and not, as in water storage dams, to catch as much natural runoff as possible. The target in siting a tailings dam is therefore usually to reduce natural runoff into the reservoir to a minimum. A secondary objective, for economic reasons, is to provide a ratio of storage capacity to embankment volume that is as large as possible; a site with the main embankment located in a valley, and therefore across a water course, normally gives a high ratio. In very flat terrain both objectives can be satisfied by choosing a site remote from a valley, resulting in a non-impounding reservoir with embankments around the full perimeter. Occasionally a non-impounding reservoir can be created in a valley site by diversion of the river around or under the reservoir. Usually, however, the planning results in a cross-valley structure and a certain amount of natural runoff into the reservoir has to be accepted. Where permitted by the location of the concentrator, a site commanded by a small catchment area is selected to keep to a minimum the spillway dimensions and provisions for flood absorption and to limit the amount of water which might be contaminated by the waste product.

2. The spillway facilities provided for a tailings dam have the twin duties of evacuating from the reservoir the surplus liquor discharged into the reservoir with the tailings and the natural rainfall runoff. For the former duty, even with the addition of water from direct rainfall, the dimensions of the structure, if the reservoir is non-impounding, may be quite modest and are often determined by the practicalities of construction and maintenance, rather than by the design discharge. In some projects in Canada the principal method of decanting is through a filter drainage blanket specially located on the embankment or natural surface for this purpose. In impounding reservoirs the design discharge originating from natural runoff may be two or more orders greater than that originating from the tailings liquor.

3. As with any spillway, those serving tailings dams must be of such a capacity that the water rise does not encroach on the freeboard provision. This is particularly crucial where the embankment is constructed using part of the tailings product itself. Deposited tailings is highly erodible and any overtopping may lead rapidly to complete dam failure and be followed by liquefaction of the contained solid material. Additionally it is necessary to avoid the possibility of the water rise resulting in the establishment of a temporary subsurface flow regime such that the pore pressures exceed the limitations imposed by considerations of stability and piping.

4. Spillway facilities for tailings dams often take the form of a service spillway for normal discharge and an auxiliary or "emergency" spillway for the more severe and less frequent flood conditions. The former facility is usually termed a decanting system or "decant" - a reflection of its duty to allow free passage of the surface water, while preventing the passage of the stored solids.

5. In this paper some of the features of the arrangements of spillways for tailings dams, particularly those relating to decanting systems, are discussed. Examples are included of typical structures, together with some of the problems encountered.

CONCEPTUAL ARRANGEMENTS

6. One of the major differences between reservoirs used for water retention and those used for tailings storage lies in the rate of construction. The dam for a water retaining reservoir is usually constructed to its full height in a relatively short period and the duty of the structure remains relatively constant thereafter. The spillway arrangements can consequently be designed as permanent structures remaining in place and operable during the life of the reservoir with a minimum

of operational control and with appropriate monitoring and maintenance. The construction of both the dam itself and the spillway is normally fully supervised by engineers.

7. Some tailings dams are similarly constructed to their full height at the start of the mining operation, particularly where the catchment area and therefore natural runoff are large in relation to the reservoir capacity. For the majority of tailings dams, however, this approach would be quite uneconomic and would result in front end capital expenditure on construction which might only benefit the operation in 10 or 20 years. Since the life of the mining operation itself may be uncertain and controlled by market forces, such expenditure could be entirely wasted.

8. The normal approach is to construct the retaining embankment of the reservoir either in stages of modest height increment or even, particularly where the construction makes use of the disposable product, as a gradually rising wall, keeping ahead of the deposition level within the reservoir by only the margin needed for freeboard, flood storage and an operational safety allowance. This approach affects decanting system design in two ways - the sill level has to be capable of being raised in very small height increments and the method of raising it has to be relatively simple and capable of being managed by operatives with minimal supervision.

9. The siting of the decanting system and the level of the sill affect the pattern of deposition profoundly (ref. 1). If it is too close to the embankment the less permeable barrier created by the deposited fine tailings is reduced in effectiveness. If it is incorrectly located in relation to the deposition points, or if the sill level is raised excessively, the deposition level may rise too fast in relation to the embankment construction along a portion of its length and the density of the deposited material may be reduced, depleting the future storage capacity. If the level is set too low the pond of water above the deposited tailings may be reduced to a size too small for the sedimentation of the solids to take place effectively.

10. The following types of decanting systems are those most frequently used to cater for the needs of tailings reservoirs which are non-impounding or subject to modest natural inflows:

Towers

11. This type of system, illustrated in Fig. 1, is probably the most commonly used. It consists essentially of a vertical tower designed to be raised within the reservoir and a sub-horizontal conduit passing under the embankment at or below foundation level, either at the bottom of the valley, for systems inserted at the start of deposition, or higher up on the flank, for systems inserted later in the life of the depository.

12. The tower may be constructed to its full height at inception, with ports in the sides, closed sequentially as the level of deposition in the reservoir rises to prevent ingress of solids. More often, however, it is constructed of steel or reinforced concrete annuli of modest depth, added progressively to perform the same function. The rings are transported to the tower either by raft or using a pontoon arrangement and a floating platform is normally constructed around the tower to facilitate their erection. The system has been employed traditionally in mining operations since the early days of mining activity, when civil engineering played no active role in the operations. The integrity of the structure has varied widely, with many early tailings dams having failed as a result of the collapse of the structure.

13. The conduit portion is similar to the bottom outlet of water retaining dams and has traditionally been constructed of steel piping, concrete piping or, more recently, of either of these encased in a structural concrete surround. Corrugated metal conduits have also been used. The reliability of the conduit has again been variable and many failures have occurred involving piping adjacent to the line or migration of solids into the conduit, as a result of collapse or joint separation, followed by chimney development at critical locations in the embankment.

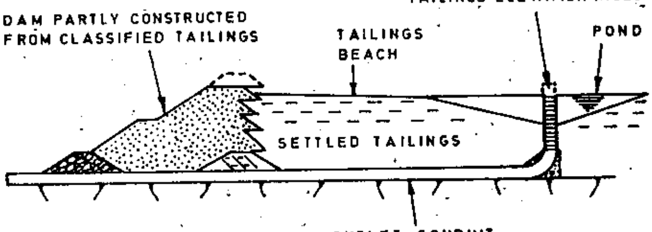

Fig. 1 Tower System

Chutes.

14. This type of decanting structure differs from the tower arrangement in that the receiving portion takes the form of a chute set into a natural slope of the reservoir or occasionally into the embankment. As illustrated in Fig. 2, slabs are used to close the overt of the chute progressively as the tailings deposition level rises. The conduit portion of the arrangement may be connected to the bottom of the chute, as in the tower arrangement, but the opportunity is also afforded in this case for additional conduits to be connected at upper levels. This may be convenient for high pressures to be avoided in the chute itself or in the conduit. The lower portion of the chute and the lower conduit may then be sealed off and filled.

15. Where there is a conveniently steep and regular natural slope this arrangement has advantages over the tower arrangement in respect of its ease of operation. The robust section and thickness of slabs required to resist the pressures and the greater length involved, however, tend to make it more expensive.

16. A variation on the chute system is the "piccolo" decant which normally comprises a steel pipe with numerous tee-pieces. This is laid on the ground and is used where steep

sided valleys permit access to the line at all times and in a convenient position with respect to the proposed filling pattern and pool location. Ports are sealed with blank flanges as the tailings level rises. This is one of the earliest forms of decanting systems and employs simple pipe technology which is usually readily available at most mines.

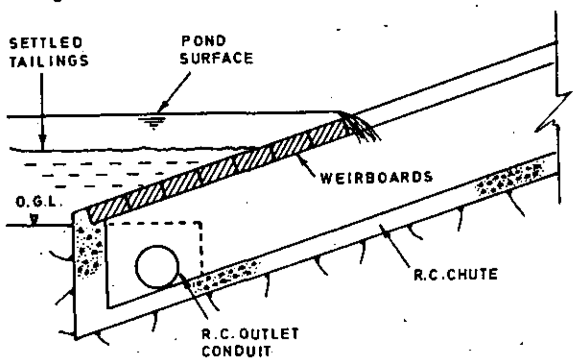

Fig. 2 Chute System

Stoplogs

17. Where saddles are available or the natural slopes under the embankment are shallow, small channel-type spillways, fitted with grooves and stoplogs, may be used for decanting (Fig. 3). These have the merit of permitting avoidance of the uncertainties of the security of structures below the deposited tailings and embankment. For economy, however, it may be necessary to construct a large number of these to serve the full range of deposition level in the reservoir, since the cost of each rises exponentially with its height. In addition it is necessary to ensure that adjacent portions of the embankment are designed to full water retaining standards to cater for the presence of the pool, which for the other systems may be located remote from the embankment. This may substantially increase the cost of the embankment, particularly where tailings construction is being employed, and this type is often reserved for the final arrangement constructed at the time of decommissioning of the tailings disposal facility.

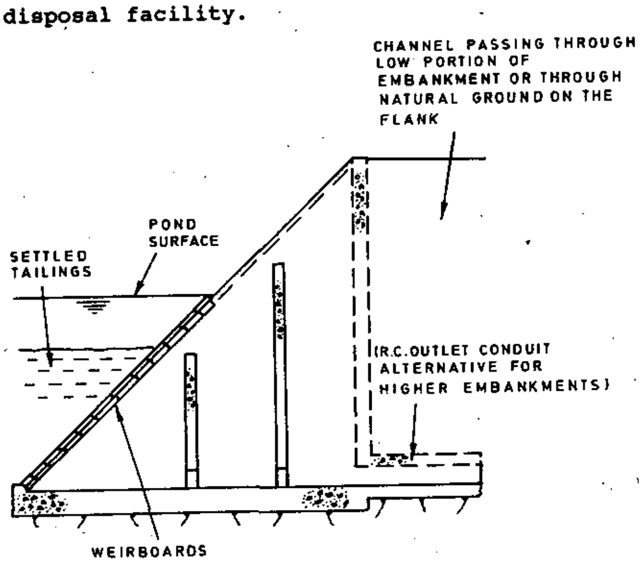

Fig. 3 Stoplog System

Pumps

18. While somewhat stretching the definition of "decanting systems", the use of pumps as an alternative to the provision of fixed free flow structures is frequently the best, and at times the only possible, method of water evacuation. Pumps may be mounted on a raft (Fig. 4), with the discharge and access supported by a floating causeway, or on a raisable framework attached to the reservoir flank.

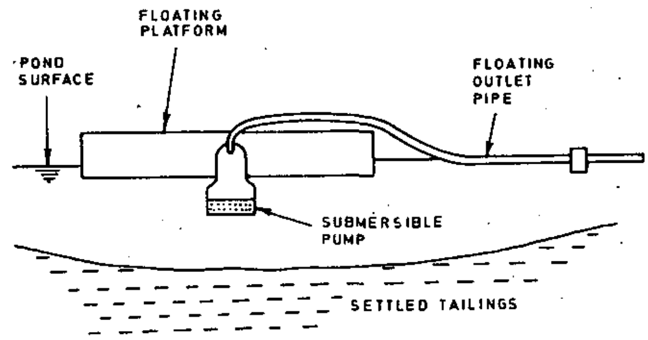

Fig. 4 Pump System

19. The advantages lie in the ease of relocation to suit the prevailing deposition conditions; the clear water pond may be moved in this way further from the embankment or from side to side to raise or lower local deposition levels or to increase the width of the impermeable deposition zone. Where the embankment is located in a steep, narrow-sided valley or where the foundations are inaccessible, as in the case of a site where deposition has already started or where an existing decanting system is no longer functional, a pump arrangement may be the only option, either as a temporary expedient or as the permanent solution.

20. The disadvantages of the system include the cost of the energy to run the pump, the inconvenience of having to have operational control at a possibly remote site, the limitations imposed by the capacity of the pump and the possibility of pump or power failure at critical periods.

21. The prevention of suspended solids from emerging with the discharge is often less easy to control since the intake pipe draws from deeper in the water than do the sills of free-flow decanting arrangements. Small auxiliary reservoirs may be needed to control this if the water pond is to be kept small and, where the pump is located on the flank of the reservoir, earthfill training walls may be needed to achieve a similar effect.

22. Pumps have been used most successfully in large non-impounding reservoirs with high tailings discharge rates such as that at the East Rand Gold Operation in South Africa and in non-impounding reservoirs where the discharged water requires treatment before discharge to the river systems.

23. The increasing need to return water from tailings dams back to the processing plant means that pumps have frequently to be sited at the dams in addition to or instead of other decanting systems. Floating pumps make the maximum use of available head in the reservoir,

a percentage of which will be lost if water is discharged to a fixed pump station sited below the dam.

ENGINEERING ASPECTS

24. Normal principles have been applied to the design of decanting structures, particularly where their engineering has been entrusted to civil engineers. Some features of the design, however, are worthy of special attention and among these are the following:

Hydrology

25. Where the reservoir is large in relation to the catchment area the shape of the inflow hydrograph becomes of lesser importance and the criterion has often been to ensure that the reservoir is able to absorb the largest anticipated storm over a given period, without an outflow allowance, whatever the time distribution of the precipitation. The essential criterion then is to ensure that the discharge is evacuated adequately before the next storm and precipitation/duration/frequency curves are employed.

26. The question of the allocation of an acceptable risk cannot follow exactly the pattern often used for the construction period of water retaining dams. At each incremental height and at each incremental change in the reservoir storage characteristics a new flood assessment must be made. The risk cannot, however, be related to the time in which these new conditions exist but rather to the summation of the different sets of conditions over the entire life of the project, which may be 30 years or more. A conservative selection of design precipitation conditions thus results when the frequency method of analysis is used. In most cases, however, the criterion is based on probable maximum precipitation. The highly erodible nature of the tailings material and the disastrous consequences of overtopping support the need for this conservatism.

27. In many of the tailings dams in which the embankment is constructed of tailings the surface of the less permeable zone is in the form of a beach extending out from the structural portion of the embankment. The freeboard must be measured downward from the top of the beach at the lowest portion along the periphery of the embankment. Encroachment above this level could set up interim untoward flow patterns within the pervious outer shell, even if the exposure is short lived. Where the outer wall is composed of the total tailings project, in which segregation and layering has occurred, the result can be equally serious and the catastrophic failure of the Bafokeng Dam in South Africa is reported to have resulted from such a condition.

28. Where the natural inflow is large or where the reservoir flood absorption is inadequate to prevent encroachment on the freeboard, the decanting system is normally backed by an emergency spillway cut into the flanks of the valley. Minimal erosion protection is usually required for these structures, since each is provided usually for only one season, and it is accepted that there would be some erosion damage, if they were to come into operation, provided the erosion could not extend to the embankment. A precipitation condition assessed to have a 100 year return period may be used as the threshold above which an emergency spillway would come into operation.

Hydraulics

29. Since the decanting level must be continually altered as the elevation of tailings in a depository increases, the hydraulic performance of the decanting system is also continually changing. It is usually impractical to provide a hydraulically efficient structure, such as a tower with a bellmouth entry; instead attention is normally given to ease of construction and operation. However, a number of useful features have been developed for incorporation in these systems to improve performance and a selection are described in the following paragraphs.

30. The hydraulic performance of a typical tower and conduit system is illustrated in Fig. 5 and is seen to be divided into three flow regimes with, respectively, weir, orifice and pipe control.

31. Weir flow occurs at low heads and the flow is controlled by the inlet arrangements. In this flow region the highest discharge per unit head is achieved and it is therefore the region in which the dam should operate. The low head over the crest of the tower permits additional tower sections, or rings, to be fitted relatively easily.

32. With a circular tower, should the height of water over the crest rise above about one quarter of the internal diameter, the nappe will meet in the centre to form a boil. When the height of water reaches about half the internal diameter weir flow will have ceased. Orifice flow then occurs and can be unstable due to air entrainment. Gulping and surging can occur, depending on the configuration of the tower and conduit. This flow region is therefore best avoided.

33. If a further increase in water level occurs the tower and conduit become full of water and pipe flow takes place. In this region control switches to the outlet and the

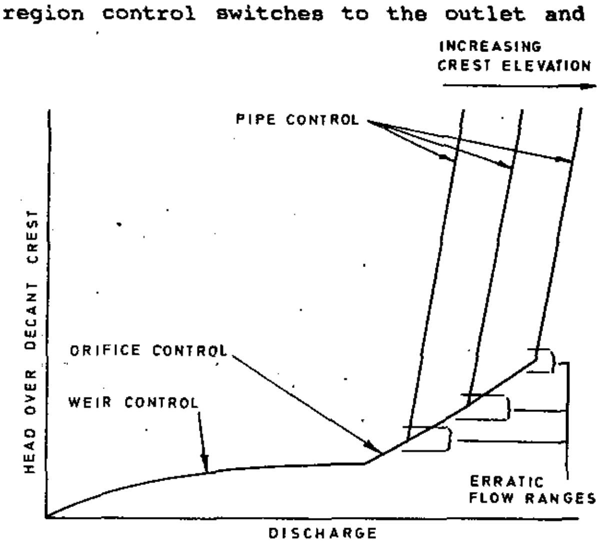

Fig. 5 Discharge Characteristics in Tower Systems

increased frictional forces result in a relatively small increase in flow for a correspondingly large increase in head. Large flood events can therefore cause a large rise in water level against which it must be ensured that the dam embankments are secure. This emphasises the need for a conservative assessment of the likely floods.

34. Consideration must also be given to the hydraulic characteristics of the horizontal conduit and care must be taken that this has adequate discharge capacity to cope with the weir and orifice flow regimes.

35. A typical inflow/outflow hydrograph is given in Fig. 6 and illustrates the low discharge capabilities of a small decant and the need for adequate flood storage or an emergency discharge spillway. The practical difficulties and cost of raising an emergency spillway, contiguous with a decanting system, usually means that attenuation is adopted as the normal method of flood control. Adequate freeboard at all times is therefore required.

36. The construction of towers from either reinforced concrete rings or flanged steel pipe sections has been described. The diameter of the overflow weir is equal to the diameter of the tower and thus limits the length of the weir crest for weir controlled discharge. One way of improving this is to attach a larger diameter pipe section to the top of the decanting tower, effectively increasing the flow into the structure for a given height of water above the sill. The larger section can be moved upwards progressively as subsequent rings are added. This larger diameter pipe section can also provide a useful secondary function; by raising it some distance above the pond level, the new rings may be attached to the tower without the disturbance of flowing water.

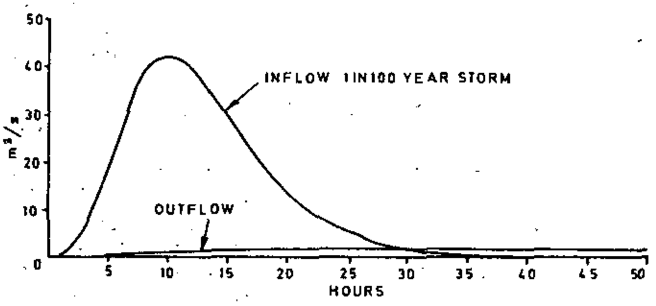

Fig. 6 Typical Hydrographs

Tower Sections

37. The rings used to form the tower structure are normally constructed from precast concrete or flanged steel pipe sections. These should be light enough to be manoeuvrable but strong enough to resist the stresses imposed by the tailings.

38. The tower is located in a medium which is subject to an increase in thickness with time and therefore continues to consolidate throughout the active life of the depository and generally for some considerable time after decommissioning. It is thus necessary for the tower to resist the downdrag to the extent of the limiting sheath friction or to be able to be compress to accommodate the downdrag once the tailings has settled into a solid state. Fine tailings has a high compression index but the coarser material has a moderate value. Some success has been achieved by the use of rubber annular inserts between the concrete or steel sections. The estimated compression of the contiguous tailings is matched with the compression in the rubber such that the vertical stress imparted by the rubber to the concrete or steel ring results in a load not greater than could be sustained by the section. An essential feature of the arrangement is that the rubber has to be restrained from inward movement.

39. At the Wheal Jane Mine in Cornwall the initial decanting arrangement, set up some twenty years ago, comprised a tower formed of precast concrete rings with 45 mm rubber inserts and a conduit section passing under the embankment (Fig. 7). The rings were bolted together through the inserts. Regular monitoring of the compression (Fig. 8) was undertaken and it was found that the compression agreed reasonably well with that estimated, although the rate of deflection tended to be irregular with some suggestion of intermittent sudden relaxation of stress.

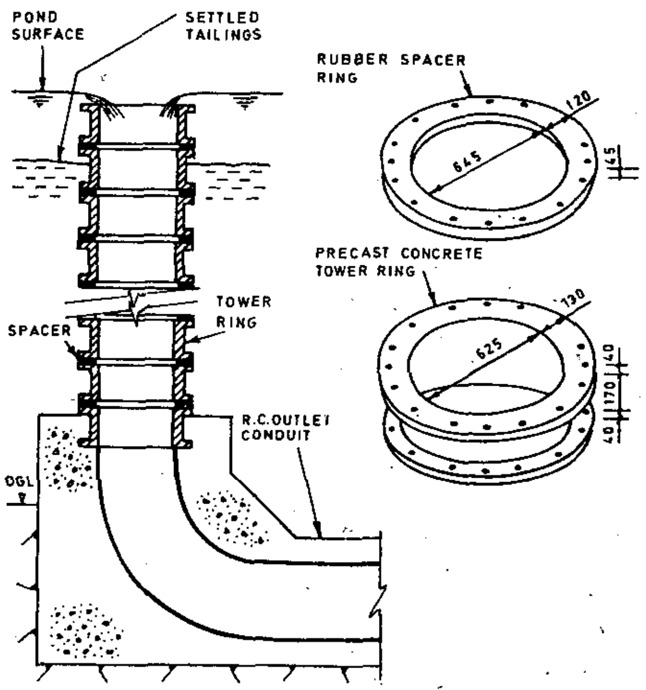

Fig. 7 Wheal Jane Decanting Tower

40. The deformed shape of the inserts was rather different from that anticipated and there was pronounced bulging of the rubber toward the inner face of the tower (Fig. 9). The inserts were glued to the concrete on one side (to the new ring being added); it was of interest to note that the shear stress exceeded the glue bond and on this face the insert slid towards the centre of the tower to the extent that the bolt restraint would permit. On the other, unglued, face the friction resisted the shear and there was considerably less sliding, illustrating the superior bond between the untreated faces of rubber and concrete.

41. These towers are gripped in the tailings downward from about 2 m below the

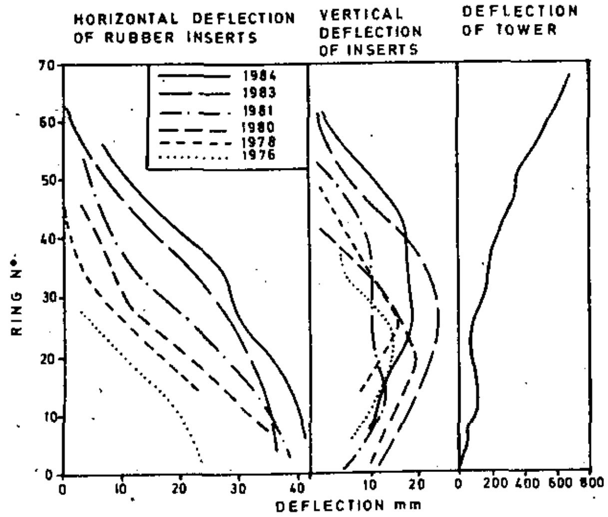

Fig. 8 Monitored Tower Deflection

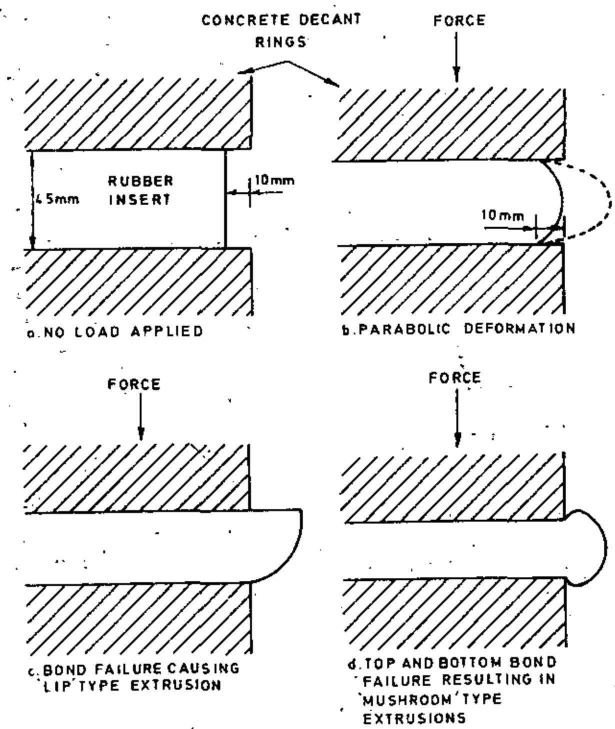

Fig. 9 Rubber Insert Deformation

surface of the tailings and their deflection in the horizontal direction is thus controlled by the strains in the mass of material.

42. In the latter stages of its life the Wheal Jane tower started to show a marked tilt (Fig. 8) which brought forward the date of its replacement with a new structure. The reason for the movement was not determined, since the cost of detailed examination could not be justified, but it was considered that a likely cause was the collapse of ancient shallow mine workings a short distance from the tower, resulting in the horizontal movement of the mass, to fill the void.

43. In other projects, where the care needed to emplace the rubber inserts was not likely to be achieved, the tendency has been toward structures of shorter life. Steel sections have been used with thin rubber gaskets and the height of the tower has been limited, generally to less than 20 m. A new tower and conduit is then started.

44. An alternative arrangement has been devised to cope with the continuing consolidation without the need for the rubber inserts. The tower is constructed helically to a spiral shape as illustrated in Fig. 10. This system has not yet been tested but would be suited best to a rapidly rising deposit of considerable final height and small water discharge requirements, where the additional costs of steel in the structure would not reduce its competitiveness with other systems.

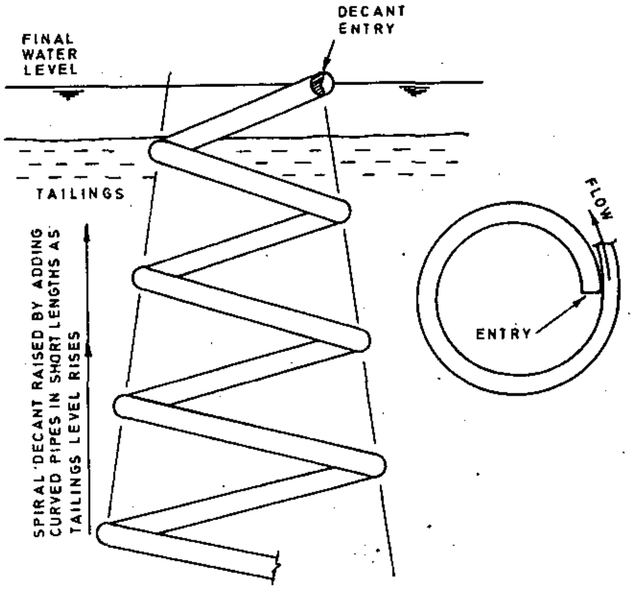

Fig. 10 Helical/Spiral Tower

Sub-horizontal Outlet Pipes

45. The design of the conduit associated with the tower and chute systems involves dealing with the same phenomena as are encountered in the design of bottom outlets for water retaining dams. The tensile stress set up by the horizontal strain generated by the embankment construction has to be overcome by the conduit being made suitably ductile or able to slide relative to the surrounding ground; the variable compression of the foundation has to be allowed for; consideration has to be given to the positive projection condition generated by the different stiffnesses of the conduit and the adjacent ground; and attention has to be paid to the durability of the structure with respect to its projected life.

46. The question of the durability of the conduit is perhaps less emphasised in decanting systems than in water retaining dam bottom outlets since the conduit constructed for the former purpose frequently has a limited useful life, at times of only a few years duration, after which the conduit is sealed and filled. The other design considerations, however, are emphasised particularly strongly in tailings dams, especially in those using tailings for construction, since the material is highly erodible and the smallest breach in the watertightness of the conduit can lead rapidly to internal erosion and failure.

47. In many projects the target has been to found the conduit well below the surface of the

natural ground and, where possible, to seat it on sound rock. The positive projection condition can thereby be countered and both the elongation and differential settlement reduced or eliminated. An additional safeguard to eliminate joint opening has been the encasement of the pipe structures in reinforced concrete, generally quite heavily reinforced in the horizontal direction.

48. A section used in a project in Papua New Guinea, where the bore of the conduit was large and the structure was required to withstand the load imposed by some 80 m of tailings, is illustrated in Fig. 11.

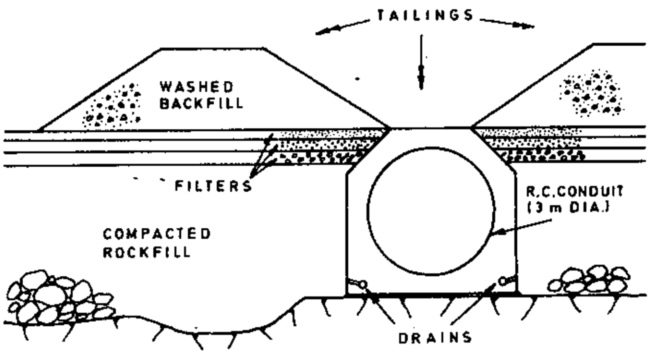

Fig. 11 Conduit under Heavy Loading

49. One of the methods used to reduce the effect of the elongation has involved the provision of a coating on the outside of the conduit, as illustrated for a project in Zambia in Fig. 12.

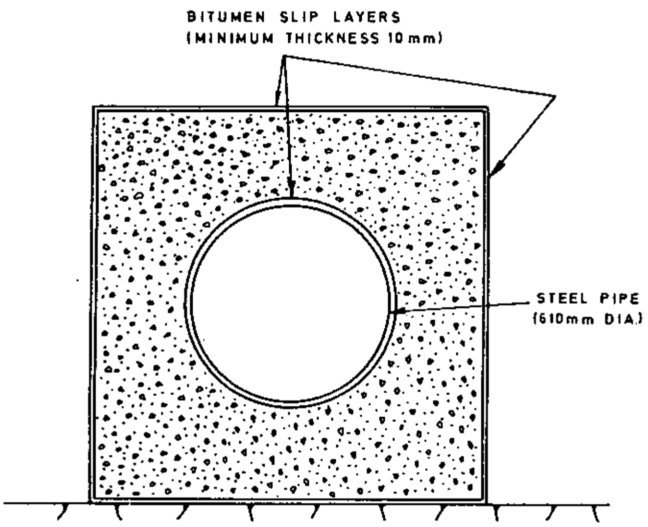

Fig. 12 Friction Reduction around Conduit

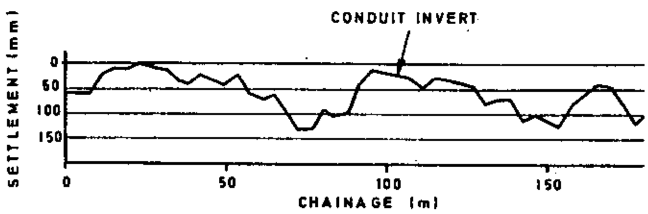

Fig. 13 Conduit Displacement

50. The conduit portion of the arrangement in Cornwall, referred to earlier, was founded on partly weathered rock and considerable care was taken in ensuring its integrity in accommodating the variable settlement anticipated. At the end of the life of the structure the vertical displacements, as determined by closed circuit TV monitoring, were found to be as illustrated in Fig. 13.

Rehabilitation

51. There is an ever greater emphasis on the provisions for tailings dam decommissioning and statutory authorities now require proposals to be formulated in considerable detail. The spillway arrangement is a particularly important feature of the decommissioning and is expected to be trouble-free thereafter, with a minimum of maintenance and monitoring. Arrangements involving conduits and towers are not generally favoured to form the final spillway and it is normally necessary for them to be replaced by a conventional side channel arrangement or for the last stages of deposition to be served by a stop-log arrangement, which can be left in a sound and durable condition with a minimum of capital expenditure at the decommissioning stage.

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13. Clay mining waste disposal problems - central and peripheral

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SYNOPSIS The planning and design of residue disposal schemes for the mineral processing industry raises a variety of problems for the designer, and operator. This paper identifies some of the economic, engineering and physical constraints which face those working in the mineral industry, with special reference to the disposal of china clay tailings.

INTRODUCTION

1.For the purpose of this Paper, Mining Waste Products are divided into three broad groups: Coarse; Intermediate; Residues. The problems and solutions discussed in this Paper are based largely on experience of the disposal of waste materials produced by the method of mining china clay employing high pressure water jetting and wet processing. However, a number of the problems associated with the safe disposal of the waste products - particularly the residues - and the solutions to these problems described in the paper apply also to residue disposal associated with other products.

2.The Paper concentrates on the disposal of the finer waste products, or residues, behind embankment dams, because it is in the handling and safe disposal of these materials that most of the problems arise. Some mention is made of the disposal of intermediate and coarse waste materials (particle sizes ranging from say medium sand through to fine or medium gravel, and including overburden). In civil engineering terms, the disposal of coarse waste products (ranging in size from coarse gravel to up boulders) is, from the mining engineer's point of view, a matter of selecting the most appropriate, economical and convenient plant, and from the designer's point of view a matter of making the best use of such material in the design and construction of retaining structures for the fine residues.

3.Because the proportions in which the three groups of waste occur and arise from the mining operation is determined by the geology of the mine area, and because these proportions are likely to vary as the mine is developed, the design of the most economical overall disposal scheme has to take account of the mine development programme. Because this in turn may be conditioned by unforeseeable market changes, the overall design has to be as flexible as possible in order to accommodate changes in the proportions of construction materials available from the mining operation.

4.From the foregoing general observations, it follows logically that the design problem - as far as the storage of solid materials is concerned - commences with, and depends entirely upon, the safe storage of fine residues. The availability of intermediate and coarse by-products may be seen as an important parameter in this basic design problem; and the disposal of any excess intermediate and coarse materials, over and above that required for fine residue disposal, can be superimposed upon the residue disposal scheme, conditioned by purely physical parameters such as landscaping, optimal land use and, occasionally, subsequent sale or other purposes.

COARSE WASTE PRODUCTS

5.Depending upon the geology of the site, it may sometimes not be economically worthwhile separating the coarse from the intermediate wastes. In other cases, particularly in the early stages of development of a mine or pit, the preoccupation of the mine manager with all the problems of opening a new site - development of access roads, siting of offices and works, and problems of incomplete land acquisition, can lead to the construction of all-in heterogeneous spoil tips comprising a mixture of overburden, coarse and intermediate waste materials stripped from the clay deposit, in such a manner as to deprive the designer of the fine residue disposal scheme of the opportunity to make the best use of these individual ingredients in the residue retaining structures. In either of these situations, there is a clear case for planning the fine residue disposal scheme as an integral part of the overall long-term disposal scheme, at the outset, as distinct from the time when mineral processing has actually commenced, perhaps several months after the initial clearance of topsoil, overburden and any rock overlying in the deposit.

6.The disposal of coarse waste materials alone, as loosely defined earlier in terms of particle size, presents few civil engineering problems. As these materials comprise essentially rock particles having a minimum size roughly equivalent to coarse gravel, there may not be the need for elaborate advance testing in order to obtain reasonable parameters for a central design and it is safe to assume that such materials will have a minimum value of ' of about 35° to 40°. The possible need to crush some material in order to produce drainage or filter materials for incorporation into the fine residue disposal scheme should be borne in mind and their durability under load or in contact with chemical laden process water should be considered, although the double-handling and processing of the drainage materials for such use has been largely obviated - in the UK at any rate - by the use of geotextiles, with or without built-in collecting drainage systems.

INTERMEDIATE WASTE PRODUCTS

7. As defined earlier, these include the sand/medium gravel components of the wastes abstracted in the first of the clay refining processes. By contrast with the subsequent refining and handling processes, carried out by hydraulic methods, the intermediate materials are usually abstracted, and subsequently handled, by purely mechanical means.

8. The designer of the fine residue disposal scheme will probably wish to utilise the sand element in order to form part of the retaining works for the fine residues. He will therefore require

(a) that it be kept separate from the overburden

(b) possibly some advanced testing of its engineering qualities for design purposes

9. As regards sand <u>tips</u>, which may (or may not) form part of, or abut, the residue embankment in a composite scheme there may be a tendency to assume that the material is both non-cohesive and free draining; and to deduce from this that the material may safely be dumped or tipped to a more or less indefinite height, at any angle less than or approaching '. This is by no means a safe assumption, not only because imperfections in the refining process may result in a significant clay content, or solely because the actual permeability after placement may prove lower than expected, but also - and perhaps most importantly - because the possibility of partial or total liquefaction in certain circumstances cannot be casually disregarded. Both research, (refs. 1 and 2) and experience have indicated that 'pure' sand tips can develop flow slides which can only be attributable to some form or other of liquefaction process.

10. Empirically it may be deduced from published accounts of flow slides that they will not (or, more precisely, may not) occur where the exposed slope is at a tolerably shallow angle, perhaps in the order of 20° or so. It is suggested here, however, having regard to the quite frightening properties of flow slides (which may commonly come to rest at an angle little more than 2 or 3°) that empirical rules of thumb are only adequate if:

(a) the designer had considerable experience of the material he is dealing with; and

(b) not only is the design height of the proposed tip limited to a maximum value well within his experience, but he is also absolutely certain that the tip will not be raised for some time after his departure.

11. There has been in recent times at least one significant failure of a sand tip during or immediately following heavy rain and there may be other examples. This has prompted the Authors to ponder another aspect of the design of intermediate waste disposal works, namely the cumulative effect of heavy rainfall on internal stability of sand tips. They do not claim to have pursued this question in any depth, beyond wondering whether the laminar structure of a compacted sand tip, coupled with the comparative readiness with which it might be expected to admit rainwater from its surface, might not in certain circumstances permit the build up of surprisingly high porewater pressures within the lower portions of the tip body.

12. In composite schemes where co-disposal of intermediate wastes and fine residues takes place it is, therefore, absolutely vital that stability analyses should be carried out on the complete cross section, as eventually conceived, as well as on intermediate stages. At the risk of stating the obvious, what may be quite reasonably perceived as a relatively thick outer layer in the early stages will dwindle in relative thickness, in relation to the height, as the scheme develops, rendering the outer layer relatively insignificant, and its strength irrelevant to the stability of the whole mass on completion. However obvious this may appear, it has probably constituted one of the principal causes of general failure in bygone years as generations of site foremen have conscientiously and confidently compacted what appears to be adequate retaining bunds whose relative size progressively shrank as lift succeeded lift (and foreman succeeded foreman).

FINE RESIDUE DISPOSAL

13. We turn now to the principal topic of this Paper, namely the residue disposal works, or 'tailings dam' although quite frequently, the design engineer will be satisfied if the retaining structure acts as a filter rather than as a dam. It may be helpful at this stage to contrast some of the problems arising in the early stages of development between a fine residue disposal structure and an orthodox dam constructed for the purpose of retaining water, for whatever purpose.

<u>Planning and site constraints</u>

14. Having 'prospected' the site for a water storage reservoir, the legal framework exists in which statutory powers can usually be invoked to obtain all the land necessary to complete the scheme, subject of course to public consultation. By contrast, a mining company has no such powers and the search and identification of mineral resources, as well as the acquisition of the land necessary in order to mine and process the mineral and on

which to dispose of waste products, must be achieved against a background of planning control, environmental studies, land ownership constraints and public goodwill generally.

15. The disposal of waste products earns no money for the mining company. The economics of mining dictates that disposal must be achieved as economically a possible, usually making use only of the materials available from the mine and as far as possible, expenditure on retaining structures deferred, for obvious economic reasons, as long as possible. This last point represents one of the most powerful reasons for selecting a 'step by step' method of construction, by contrast with most water storage reservoirs (granted that examples do exist of the latter also being developed in more than one stage as demand grows).

16. The foundations for the disposal scheme may very well not be ideal, because the very soil which is attractive as a mineral ore may well have second rate geotechnical characteristics, simply because it is a clay bearing material.

Hydrological considerations

17. It has been suggested in an earlier paragraph that the design engineer may well often be satisfied if his so-called 'tailings dam' simply acts as a safe and stable filter. This is not always the case and it is sometimes necessary for the tailings dam to act, to a greater or lesser extent, as a reservoir for the reuse of water for the purpose of mineral processing. A water-balance study, taking account of whatever hydrological data may exist for the site, is thus an integral part of the project planning as a whole, but in overseas countries, the lack of adequately documented hydrological data may be a problem, not only in relation to the overall water balance for the mining process, but also in the design of adequate means of coping with flood inflows.

Water control structures

18. Since the cheapest type of site for fine residue disposal (as with water storage reservoirs) is typically a natural valley, it follows that the optimal tailings disposal site will be situated in a natural watercourse having its own catchment area. The design of suitable spillway arrangements may be to some extent simplified by permanent stream diversions or catchwaters; on the other hand the problem is in certain respects more complicated that the design of spillways for water storage reservoirs by the fact that overall economics of the mining development normally dictate progressive development of the retaining bunds in preference to the initial construction of a dam to the full eventual height. The engineer must therefore devise a series of spillways, or other means of safely releasing floodwater, at a succession of increasing levels as the disposal scheme develops. Since both tailings and, often, the materials forming the retaining bunds, are highly erodible, the prevention of overtopping is every bit as vital as in the case of a traditional earth embankment dam impounding a water storage reservoir. At each stage in the tailings dam development, the spillway works must therefore be adequate and yet, each having a limited useful working life, must be designed with a view to reasonable economy.

19. It is fairly typical to make use, as far as possible, of a common wastewater channel downstream of a series of spillways. This can incorporate the necessary drop-structures at the downstream end of the perimeter catchwater, designed to minimise flood flows onto the surface of the tailings dam.

20. The design of stream diversions or catchwaters and, to some extent, of intermediate-level spillway structures, presents the design engineer with an almost philosophical problem concerning the selection of appropriate parameters for the 'design storm'. Both instinctively and logically it would seem uneconomical to design temporary water control works, having a useful life of perhaps two to five years at most, to cope with the same long-term flood risk as the eventual completed scheme.

Abrasive suspended solids

21. The fine residues themselves, carried to their disposal lagoon in suspension in water, are almost certain to be highly abrasive. It is the experience of the Authors that the mining company, with its many years of experience, need hardly look to its consultants for advice or expertise in the design or manufacture of the necessary pipelines. However, in the design of ancillary works, such as catchwaters, diversions and drop structures, the design engineer must be keenly aware of the abrasive nature of even the relatively small concentrates of fine material still carried in suspension in overflows and possible 'natural' surface flows draining toward the lagoon.

22. Indeed, in one case in South Devon, where a public road diversion incorporated a reinforced concrete culvert carrying surface water which had been partly contaminated by tailings upstream, considerable abrasion damage was suffered in the culvert invert concrete, notwithstanding the fact that the flow velocities were not particularly high.

23. In the case of some diversion and catchwater channels, the channel cross section may be designed with a relatively small invert channel incorporating extra, sacrificial, concrete designed to take normal or dry weather flows with a substantially wider cross section above, with more sparing use of concrete, to take the larger, obviously more transient, design flows.

Physical design of the trailings 'dam'

24. Using the traditional nomenclature, the design of tailings dams has often been classified in three groups or types, viz (i) upstream method; (ii) centraline method; (iii) downstream method. Although each of these names may be something of an over simplification, they are acceptable as a basis for the comparison of three approaches to the

step-by-step development of tailings lagoons. Although perhaps almost too familiar to those concerned with tailings dam design, they are reproduced for ready reference in Figure 1. Because of the many and varied site-specific constraints affecting the progressive development, both of the mine and of the disposal area - some of which have been briefly referred to earlier - any generalised comparison between the three types of construction should be read with some caution and may well be invalid for particular sites.

Downstream method

25.An embankment constructed by the downstream method encroaches progressively on the land downstream of the original starter bund and full account must be taken of this in the layout of any stream diversions or overflow works discharging into any original watercourse downstream.

26.Since the downstream slopes are successively covered by fresh lifts, the system clearly does not lend itself to progressive planting.

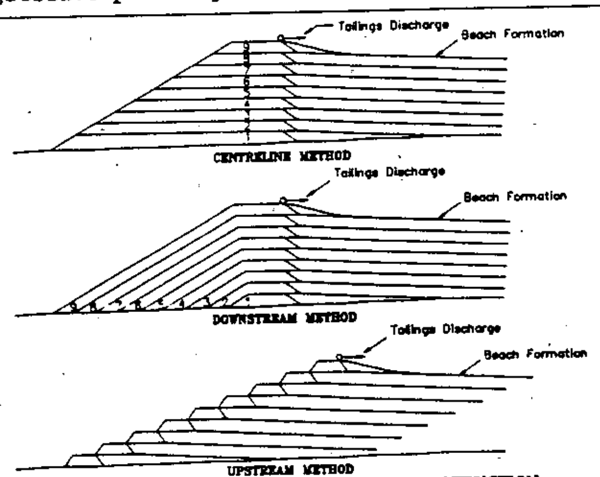

Figure 1 - SCHEMATIC EMBANKMENT CONSTRUCTION

27.Because virtually the whole of the completed dam section will have been constructed of either intermediate or coarse waste fill, the downstream slope is conditioned by the shear strength parameters of the coarser wastes and can therefore in general be steeper than in the case of the upstream method.

Centreline method

28.Centreline construction may be advantageous, for example if the retaining bund or dam has to be constructed between abutments comprising two well-defined narrow spurs of land, such that a shift upstream or downstream would increase significantly the length, and hence the bulk, of the retaining structure.

29.As with the downstream method, this system depends upon the ready availability of relatively large quantities of intermediate or coarse waste fill for the downstream shoulder of the completed cross-section; moreover, for practical reasons either the rate of yield of coarser wastes, as the mining process develops, must closely match the rate of requirement for successive lifts of fill material, or there has to be a convenient source of excess waste material readily available within a fairly short haul distance.

30.Also in common with the downstream method any catchwaters or overflow works must be carried clear of the ultimate positioning of the downstream toe of the completed section. Whilst this point may appear obvious, it is sometimes less obvious that the full extent of the disposal scheme as originally envisaged may be subject to subsequent alteration by reason, say, of production changes or economic factors, in which case the existence of a possible expensive spillway channel and drop structure immediately downstream of the originally proposed profile may be an embarrassing and unwelcome restraint on the additional development of the dam.

Upstream method

31.Upstream construction as currently practised by the Authors and their colleagues, is essentially based upon fairly commonplace methods of stability analysis using measured values of shear strength parameters, forecast (and monitored) values of porewater pressure and incorporating reasonably tight control of the method of introducing the residue into the lagoon from the upstream face of the temporary retaining bunds.

32.Typically, the finished overall slope of such a development will be in the order of 11 or 12 degrees. The retaining bunds or at least most of the upper ones, can be satisfactorily constructed utilising the partly consolidated fine residue from the beach near the perimeter (consolidated by desiccation) although the first one or two retaining bunds have to be constructed using intermediate or coarse wastes, or a combination of the two. To prevent migration of the fine residue particles through these early starter bunds, geotextiles or graded filters are frequently incorporated on the upstream face of the starter bunds and some degree of under drainage, also utilising man-made fibres, is occasionally required.

33.As remarked earlier, there is empirical evidence suggesting that failures by liquefaction, or flow slides, may not be expected in cases where the overall downstream slope is substantially less than, say, 20°. Nevertheless, modern developments may be thought to come close to, or to exceed, the limit of practical experience of upstream-method construction, and, following a study of research work published elsewhere on liquefaction generally, it was considered that further research work was justified, specifically related to china clay tailings, but of probable application elsewhere, to establish, if possible, the essential parameters confirming the susceptibility of the stored residues to liquefaction.

34.In the upstream method of construction, overall stability obviously depends upon the shear strength of the deposited fine residue and upon maintaining as low a phreatic surface as possible. For both reasons, it is highly desirable or virtually essential, that residues should be introduced to the disposal lagoon from the retaining bunds in a controlled manner and the supernatant water kept as far from the perimeter bunds as possible.

35. Similarly in general terms, the nature and permeability of the foundations of a new scheme over which the designer may have little control are of course highly relevant to the likely eventual flow net set up.

Environmental considerations

36. There is a far greater need these days to consider the environmental impact of any disposal scheme than previously, with every indication of stricter restraints in the future. These restraints include, for example:

(a) visual impact - the need to develop a landform sympathetic to the surrounding landscape
(b) progressive restoration, including early development of vegetation and tree cover
(c) Quality of supernatant water (where not required for recycling and processing) discharged to natural watercourses. Hitherto, control has largely been restricted to limiting suspended solids and pH control; analyses and control of other elements may well be required in future
(d) dust control
(e) control of surface erosion and the contamination of surface drainage flows which may themselves have already been contaminated in passing through or near the process plant.

37. From design considerations, the desirability of the formation of beaches near the perimeter bunds, encouraging partial consolidation by desiccation and the lowest possible phreatic surface has already been emphasised. Perversely, this can give rise to the considerable environmental disadvantage of dusting in dry windy weather, not only on the beach itself but also the exposed surfaces of the upper retaining bunds before the establishment of grass or vegetation cover. System planning and budgeting, and the provision of access, should take account of the periodic need for surface watering by bowsers.

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15. Gale common ash disposal scheme - concept, design and construction

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General aspects of ash disposal are followed by particular application to the major Gale Common scheme. The design of the 51m high embankments is presented in the context of the geology, materials available and environmental protection. Construction methods are described.

INTRODUCTION

1. About 20% of the content of British coal burned in thermal power stations remains as ash. A 2000 MW power station is capable of burning up to 100,000 tonnes of coal in a week and records show that in practical terms these power stations produce around 600,000 tonnes of ash per annum.

2. Modern thermal power stations are sited near the coal fields, and in the mining of the coal to fuel the stations, around 60% of the material brought from the face is discarded. For a large productive mine, there can be more than 1,000,000 tonnes of waste per annum.

3. The ash from the stations is of two types, -

pulverised fuel ash - (pfa) and
furnace bottom ash - (fba).

Pfa is generally composed of uniformly graded spherical silt sized particles containing within it around 2% hollow glass balls (cenospheres) which appear as 'floaters' when the pfa is disposed of into lagoons.

Fba is generally composed of well graded sand and gravel particles and is in demand for construction drainage purposes.

Principles and Philosophy

4. Ash disposal is wholly an overhead for a power station and consequently its cost must be minimised. Where disposal is adjacent to the station the ash can be disposed of in conditioned form by truck or conveyor. However, for more distant sites the cheapest form of transportation is generally as a slurry in a pipeline. This means lagooning at the disposal site, and dam structures.

5. Economics dictate that the pfa and fba be used for dam building and while trucking of conditioned material is possible it may not be environmentally acceptable or economical. The dewatering of the slurried ash at the site is the alternative, achieved by vacuum filtration to near optimum moisture content. Fba is produced separately and in relatively small quantities and is normally transported by road. It should be noted that the excavation of lagooned material for embankment construction is not practicable with modern stations due to the separation of the fba from the pfa. With separated pfa re-excavation is difficult as the loose saturated silty material tends to behave thixotropically when disturbed.

6. Materials of lower permeability than pfa may be required in ash dam construction for seals. Because of the relationship of modern large thermal stations with locally dedicated pits, colliery waste (shale) is often the cheapest available material.

7. The pit/power station relationship also gives rise to the common occurrence of mining subsidence under ash dams and lagoons. Additional factors thereby imposed on design are:

- dealing with deformations, strains, and cracking
- allowing for 'earthquake' shock under vigorous subsidence, eg liquefaction of hydraulically placed pfa.

8. It is the authors' practice to arrange disposal schemes so that lagooning can be suspended in a lagoon subject to active subsidence as defined by an upper limit of tensile strain.

9. The CEGB and successor companies have maintained a policy of having their ash dams designed, constructed and certified according to the requirements of the Reservoirs Act.

10. Environmental protection is of great importance in ash disposal and procedures are described in the accompanying paper, (Dennis et al).

11. Because of the pressure on economics, some features may be adopted in excess of their use in conventional dams. Examples are:

- geotextiles and membranes
- "fir tree" construction, where the upstream shoulder is founded on earlier deposits of slurry placed pfa (triangular instead of trapezoidal dam section; liquefaction must always be guarded against).
- flexible pipelines laid directly in

fill of internal bunds, connecting decant towers.

12. Further features distinguishing ash dams from conventional water retaining structures are as follows:-

- The phreatic surface under steady seepage through an homogeneous embankment retaining saturated slurry, is concave upwards instead of the normal convex. This is because a vertical through the "reservoir" is a flow line, not an equipotential, head loss occurring through the lagoon ash.
- The construction period is very prolonged. This requires great care with maintaining setting-out systems design manuals etc, but normally permits exclusion of the 'end of construction' analytical case.
- The retained pfa is sensitive to piping under high exit gradients, and is not self-filtering.

Furthermore ash dam factors of safety tend to be at the lower limits of acceptable norms.

APPLICATION TO GALE COMMON

13. The primary purpose of the scheme is the disposal of pfa from two 2000 MW power stations, Eggborough and Ferrybridge C, located some 5 and 7km distant. Additionally, up to 1m tonnes p.a. of shale (mainly coarse discard with some pressings) from a nearby colliery at Kellingley is disposed of on the site. The ash is pumped to site as slurry and the shale is delivered by lorry. Stage I, started in 1967, is nearing completion, and now contains more than 30m cu.m. of material. Preparatory works for Stage II are well advanced (see Fig.1 of Dennis et al).

14. The scheme disposes of the pfa by the filling of lagoons with slurried ash and by the use of conditioned pfa as the main construction material for the lagoon embankments, part of the slurry being dried to near optimum moisture content in a vacuum filtration plant.

15. The prime use of the shale is to form by fir tree methods, the division bund between the pair of lagoons forming each stage of the development together with the 'water' faces of the enclosing embankments. However, it is available in such large quantities that in Stage 1 it was used to form most of the embankment abutting the succeeding phases of the scheme, and in Stage II there is to be a core of shale downstream of the chimney drain where its greater density enhances embankment stability and improves safety against progressive failure.

16. The lower levels of the Stage II embankments will be built in full width horizontal lifts, each 3.3m in depth. The water face will be inclined at 1:2 at lower levels and will be nearly vertical at upper levels comprising a series of fir tree lifts built out on to the previous lifts of ash settled in the lagoon.

17. The profile of the Stage II embankment is shown in Fig.1. The embankment materials are placed according to normal earth dam methods and standards. Lagoons are filled alternately.

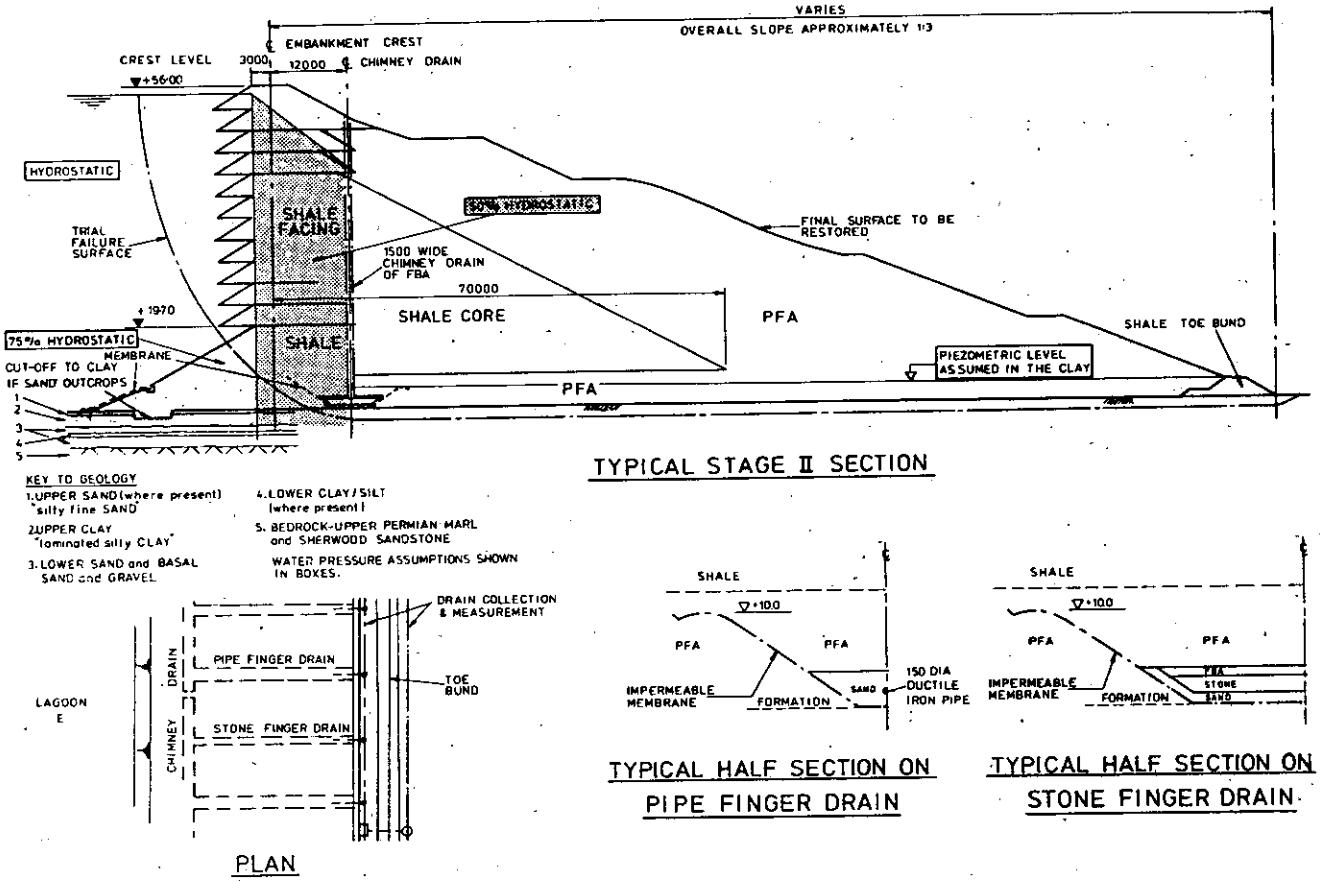

Fig.1 . Typical Stage II embankment details

18. For full protection of the underlying aquifer against leachate, the whole lagoon bed of Stage II is covered by an impermeable membrane. This protection also extends under the chimney and finger drains. In order to expedite draining down of the lagoons upon completion of slurry placing, a dendritic pattern of drains on top of the membrane leads to the outlet culvert, controlled by valves. Some 4 or 5 lifts of the embankments will be built before slurry placing and the lagoons are flooded with water initially to protect the membrane.

THE SITE

19. Gale Common lies on the south western margin of the Vale of York at National Grid Reference SE 535 215. To the west there is a low NW-SE trending escarpment rising to over +30m O.D., passing through Cridling Stubbs northwards towards Knottingley. The River Aire passes within approximately 4km of the site to the north, with the Aire and Calder Navigation and the M62 Motorway situated in between, on low lying ground which extends eastwards away from the escarpment towards the Humber estuary. A Yorkshire Water Authority aqueduct crosses the site on the west side of the main lagoons.

20. The site is generally flat, lying between approximately +6 and +8m AOD with a slight fall from NW to SE. Especially in the area of Stage I, it is naturally marshy with patches of peat deposits. The main site surface drainage consists of a perimeter ditch.

Geology

21. The site lies well to the south of the limit of Devensian tills in the Vale of York. It does, however, lie within the area occupied by Lake Humber in Devensian times and investigations have shown that below a thin cover of Flandrian Stage peat or clay the entire site is underlain by lacustrine silty clay and interbedded sands. Between these deposits and bedrock are sands and gravels of probable fluvial origin.

22. As shown in Fig.1 the superficial deposits across the site typically consist of the following sequence:-

Deposit	Thickness
Upper Sand	0-1.5m
Upper Clay	0-3.7m
Lower Sand	0-2.7m
Lower Clay/Silt	0-1.7m
Basal Sand and Gravel	2-6.0m

23. The solid geology comprises Carboniferous Middle Coal Measures at depth, overlain by Upper and Lower Magnesian Limestone and Permian Marl and partly by the Sherwood Sandstone Group. The boundary between the Sherwood Sandstone Group and Permian Marl subcrops diagonally across Gale Common striking approximately NW-SE. The Permo-Triassic strata dip gently eastwards with the Magnesian Limestone forming the escarpment to the west of the site where easterly dips of up to 5° are recorded. East of the escarpment bedrock is concealed by the superficial deposits. Major faults follow a WSW-ENE trend with the Pontefract-Knottingley trough fault passing northwest of the site. A secondary fault trending WNW-ESE underlies the southern part of the site.

24. The Magnesian Limestone, Sherwood Sandstone Group and the basal sand and gravel are all important aquifers as taken into account in the design of the works.

25. The site lies within the Yorkshire Coalfield and is underlain by several seams of coal. Coal has been extracted from a 1.5m thick seam at a depth of some 700m beneath Lagoons A and B (Stage I) and mining of a 2.3m thick seam has recently been carried out at about 660m beneath the emergency lagoons where over 2m of subsidence has been observed. It is anticipated that mining beneath the Stage II lagoons will eventually take place.

26. Groundwater is generally within 1 to 2m of ground level. Very shallow groundwater gradients have been observed with flow generally to the south east in the basal sand and gravel aquifer. There is evidence for a small upward hydraulic gradient and slightly artesian groundwater conditions in the sand, gravel and bedrock dying out from west to east across the site.

27. Gale Common lies within a region of moderate seismic activity for the United Kingdom where Lilwall (1976) (Ref. 1) calculated Intensity 5-6 ground motions having a return period of 200 years.

GROUND INVESTIGATION

28. Extensive ground investigation has been carried out. The Stage I investigation included 24 percussion borings and 28 shallow auger holes. In 1974 excavation and detailed inspection of pits and trenches was also carried out in Stage I. (Taylor et al Ref. 2.)

29. The 1982 investigation for Stages II and III was carried out in two phases. Phase 1 comprised 38 boreholes, 6 trial pits, 2 trial trenches and 10 static cone penetration tests. The boreholes were sunk using cable tool percussion methods or flight augers through the superficial deposits. 3 boreholes were extended into bedrock by rotary core drilling. Soil samples were obtained for logging and laboratory testing. In selected holes the boring was progressed without the addition of water so that high quality samples of groundwater could be obtained and field and laboratory chemical analyses were carried out. 19 standpipe piezometers were installed.

30. Phase 2 included the sinking of some 117 probes to establish the thickness of the upper clay across the site.and the installation of a further 4 standpipe piezometers for hydrogeological monitoring purposes.

GEOTECHNICAL DESIGN

31. A typical profile of the Stage II embankment is shown in Fig. 1.

32. The geotechnical design is governed principally by the Upper Clay layer which has a relatively low strength compared with the other foundation soils (peaty material having been removed in preparing the site) and with the embankment materials. This deposit is of intermediate to high plasticity and is often laminated, with tree and plant roots. Near-vertical discontinuities noted in trial pit and trench inspections are considered to result from desiccation following the draining of Lake Humber. Tests indicate that the deposit is slightly overconsolidated, considered to be due to the effects of vegetation and desiccation.

33. From consideration of the geology and topography of the site and detailed inspection the clay deposits are not believed to contain pre-existing shear surfaces. Based on the results of the laboratory tests design parameters given in Table I are believed to be reasonable lower bound peak parameters for shearing along the laminations.

Table 1. Properties and parameters assumed in design for the Upper Clay.

Natural moisture content	24 to 40%
Liquid limit	40 to 75%
Plastic limit	22 to 30%
Bulk density	1.7to2.0 Mg/m^3
Drained shear strength	
Ø' peak (design value)	17 degrees
Ø' residual (min value)	11 degrees
c' peak (design value)	3 kN/m^2
c' residual (min value)	0
Coefficient of compressibility m_v	0.16 to 0.30 m^2/MN
Coefficient of consolidation c_v	2.0 to 4.0 m^2/yr

34. The parameters determined from laboratory tests for other construction materials and for the lagoon pfa are given in Table 2.

Table 2. Engineering properties assumed in design for the embankment construction materials and lagoon pfa.

	Compacted Colliery Shale	Compacted pfa	Lagoon pfa
Bulk Density Mg/m^3	2.07	1.60	1.63
Maximum Dry Density Mg/m^3	1.78	1.46	-
Optimum Moisture Content %	8.5	17.5	-
Permeability m/s	< $1.0x10^7$	$1.5x10^{-6}$	-
Ø' degrees	29	29	22
c' kN/m^2	0	0	0

35. For drainage zones use has been made of local limestone and fba. Typical particle size curves for these materials together with those of pfa and shale are shown in Fig.2 .

36. Extensive embankment stability analyses have been carried out for Stage I and in the design of Stage II. Because of the soil types and the slow rate of construction it was determined that construction excess pore pressures would not occur and so only drained conditions need considering. Effective stress analyses using pore pressures predicted from seepage theory were therefore carried out.

37. Typical piezometric design assumptions are shown in Fig. 1 together with an example of a trial failure surface analysis. Analyses used computer programs of the Sarma and Morgenstern and Price methods and hand wedges.

38. The static factor of safety required against both overall and local toe failure is 1.40. This was judged to be the minimum allowable commensurate with the assumptions, particularly those for the Upper Clay Layer. Following Carsington, the design was reviewed to assess any potential for progressive failure particularly in view of the relatively brittle nature and thinness of the Upper Clay layer which would tend to concentrate strain. It was concluded that the assumptions made concerning the drained shear strength of the clay and the factor of safety criterion in the conventional stability analyses were such that progressive failure was unlikely to be a problem. However, it was considered prudent to carry out some finite element analyses to check the potential for progressive failure.

39. The embankment has also been checked to ensure a factor of safety greater than unity for seismic loading or shock loading due to mining induced movements. For these cases it was assumed that the lagooned material had zero strength. (Haws et al Ref.3).

40. Foundation and embankment pore pressures are monitored. Most of the original 22No twin tube hydraulic piezometers continue to function satisfactorily 25 years after they were installed. As Stage I developed additional piezometers were installed, and there is currently a total of 46.

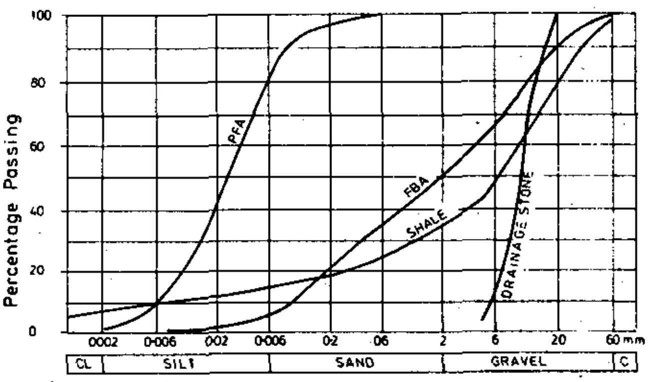

Fig. 2. Typical particle size curves.

41. For Stage II 33 hydraulic piezometers have been installed in the foundation soils and additional ones will be installed in the embankment during construction to monitor the seepage condition between the lagoon and the chimney drain.

42. The piezometer monitoring to date has not indicated any cause for concern and has confirmed that there is no build-up of construction pore pressures.

CONSTRUCTION

Materials supply, characteristics, laying, & testing

43. Pfa The pfa is pumped from the two power stations via 530mm pipelines. These are of asbestos cement which, although cheap, have proved to be susceptible to erosion. The pipeline from Ferrybridge is now being replaced by one of high density polyethylene which trials have shown to be highly resistant to pfa erosion.

44. At rest the pipeline is kept filled with water so that when ashing out a power station an initial slug of water of low slurry content enters the lagoon.

45. When slurry of solids content 20 - 40% reaches the site it is diverted to the vacuum filtration plant. Here as much as possible is dewatered to about 20% moisture (optimum being around 17.5%) and the rest is disposed of into the lagoons as unsuitable. About 55% of the pfa reaching the site is thus processed and used in the embankments.

46 As pfa is made up of spherical particles it is easily compacted and has a flat standard compaction curve. This makes it possible to lay throughout the year in layers up to 500mm thick, achieving compactions in excess of 98% maximum standard compaction with 4 passes of a T182B Vibrating roller.

47. The in situ density test uses the larger 150mm diameter core cutter to reduce the effect of dimensional error on a very sensitive result. The top 150mm layer of material is discarded because of overstressing or disturbance by traffic. Three cores are then taken going down in 180mm units so that the bottom of the lowest core is between 600 and 700mm below the surface covering the re-worked top of the previous layer.

48. Shale Although the site is owned and operated by National-Power, it is also the principal disposal area for discard from Kellingley Colliery. Included in the discard are press tailings, the amount being limited by the need to stockpile the material in a tip throughout the winter.

49. The material arrives saturated from the colliery washer, so when placed direct into the works it is spread in 100mm layers to dry for a day to lower the moisture to the 8.5% optimum before being rolled by two passes of a 10 tonne grid roller. This leaves a high surface area to aid drying and absorb surplus moisture from the succeeding layer of wet shale. Laying can only be carried out when drying conditions prevail, which experience has shown to be dry weather with a day temperature in excess of 15°C.

50. Testing is by the normal sand replacement method ignoring the 'top 150mm' of temporarily saturated material.

Construction cycle

51. At all times construction must provide new capacity before existing lagoons are full. In the case of 'A' lagoon (Dennis et al, Fig.1) crest raising is carried out when the lagoon is out of commission. However owing to the filtration plant effluent only passing into B lagoon that lagoon never comes out of service so that the placing of a nib of shale into the lagoon followed by crest raising has to be done with the lagoon live. The amount of shale 'lost' into the lagoon is clearly minimised by carrying out this operation towards the end of the lagoon filling cycle.

52. Where the outfall has to be raised for the live lagoon the method adopted has been to isolate the area by forming a shale bund around the outfall tower, having first raised the tower concrete, if necessary over water. Cleaning out, infilling the tower and placing damboards follow as well as the placing of stone filled gabions 2m high so as to form a barrier to prevent the layer of 'floaters' and vegetation from reaching the tower. The 'floaters' rise with each lagoon filling and are now between 1 and 2m thick. The lagoon being live means that pumps are required to be on hand when men are working in the pond area around the tower. In operation each tower has a collar boom around it as a secondary protection to prevent floaters being carried back to the river in the return water which must not exceed 40 ppm.

53. The Stage I construction was dictated by the initial shortage of embankment material, so that for each raising of the embankment crest the form most economic in use of material was adopted. This led to a continuous operation of placing a minimum of material on the temporary outside face up to a level from which the crest raising for a new lagoon could be executed in a comparatively short time in the summer (necessary because of the use of shale at the water face). Repeated temporary treatment by hydroseeding was thus necessary to the outer face of the embankment for dust prevention and environmental reasons.

54. For Stage II, construction will be by complete horizontal layers across the embankment permitting outerface sowing as the embankment rises, thus presenting a better image.

Survey controls

55. For Stage I, survey stations were established on the toe bund wherever cross section checks were desired. This arrangement was suitable for the tacheometric survey method applied until the advent of the EDM. The stations also provide control for the landscaped outer face. Here the practice is to raise the embankment in 2m stages, survey in pegs on the correct outerface and then trim and soil to this line. This not only enables irregular patterns to be followed but also

ensures that the soil is placed on a prepared surface uncompacted by plant. It also leaves a degree of unevenness which helps to avoid the "engineered slope" normally found in earth dams.

Other works

56. On a scheme of this size the ancillary works are substantial, examples are:

- the provision and maintenance of 5km of 7.9m HRA road for 17 tonne axle loads.
- the provision of a monitored drainage system isolated from surroundings, the water being returned to the river with lagoon outfall water.
- the provision and replacement of slurry pipelines within the site both on and off the embankment.
- embankment, drainage and survey work arising from mining subsidence.
- the provision and operation of instrumentation.

Initial development of Stage II

57. Apart from an initial contract to establish the road system, the outfall arrangement, and some of the embankment formation, the work of placing the foundations of the Stage II embankment has been carried out as an extension to the normal work of operating the site.

58. Both stripped topsoil and subsoil have been stored in two separate groups according to a plan prepared by the Landscape Consultant and MAFF.

59. This has been followed by the initial embankment construction of shale with a membrane lining of black POLYTARP laid on the floor of the lagoons and up the inner slopes for the first 3 m above ground level.

60. Between the dendritic drains (para. 18) were laid dumplings of press tailings from the colliery to hold down the membrane against wind action. In spite of this, considerable amounts of air were trapped under the membrane and it was necessary carefully to vent the "whales" which arose as the lagoon filled.

61. Finger drains leading outwards from the chimney drain encircling the embankment were first shaped, lined with membrane up to +10mAOD and then filled with a stone drain and a piped drain at alternate locations. At their outer ends these drains are led via individual manholes into measuring chambers so that embankment seepage can be examined in sectors.

62. Two blocks of instrumentation, one in the south side and one in the east, record foundation piezometric levels using hydraulic instruments.

Outfall system

63. The effluent from the lagoons is handled identically to that for Stage I with discharge via concrete outfall towers and a 900mm diameter pipeline laid through the embankment in a reinforced concrete tunnel. The towers will be raised in 6.6m stages with continuity of reinforcement maintained by the use of couplers. The height of these stages is dictated by the need to place an access platform on the top of the tower. Lagoon water level is controlled by damboards placed from the platform. These are of reinforced concrete with their faces recessed so that they can be removed with a scissor grab.

64. On completion of each 3.3m lift of the lagoons reinforced concrete is placed to fill the damboard slots and complete the octagonal tower, the damboards being retained as a back shutter for safety.

65. As developed for Stage I, shale is placed around each tower to form a platform to carry stone filled gabions to hold back the floaters.

CONTRACTS

66. Construction has been by a series of contracts of two years duration, the new succeeding the old without any time gap. Early contracts were undertaken by Lehane, McKenzie & Shand and the more recent ones by Taylor Woodrow (Northern), with a Stage II preparation contract going to Henry Boot.

ACKNOWLEDGEMENT

Acknowledgement is made to National Power Co. for permission to publish this paper.

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16. Gale common ash disposal scheme - planning, environment, operation and restoration

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The scheme is for the disposal of waste materials from two power stations and a colliery, by forming circumferential embankments of these materials and filling the central voids with slurried ash. In view of its prominence, great attention has been paid to the profile and restoration of the 'hill' so formed. The life of the project is some 50 years and it is being developed in 3 stages. Work on the second stage has recently commenced.

PLANNING AND ENVIRONMENT

Introduction

1. The disposal of ash from a major coal-fired power station can be a considerable problem, not least environmentally. When Eggborough Power Station (2000 MW) was being planned, a number of schemes for the disposal of the ash were considered, but finally, with the nearby Ferrybridge 'C' Power Station (also 2000 MW) being built at about the same time, the concept of a joint disposal site had obvious advantages. A new colliery was also being opened at Kellingley and the waste from here was considered to be a useful material for embankment construction. This opened up the possibility of dealing, on one site, with yet a third disposal problem.

2. The general area in the vicinity of Gale Common is flat and featureless, and the scale of disposal - some 65 million cubic metres over the full life of the scheme - meant that an above-ground scheme would lead to the creation of a man-made hill on a scale rivalling Brayton Barff near Selby; offering the prospect of adding a pleasant feature to the local landscape.

3. From the outset great attention was paid to finished profile and final landscape treatment. Lagooning was planned to a total height of about 50 metres, making the embankments - which in effect are peripheral dams - among the highest earth dams in the United Kingdom. The scale of the scheme is such that the horizontal embankment crests would, from certain viewpoints, stretch for over a kilometre across the skyline without a significant change in level. Therefore, notwithstanding that the outline of the scheme in plan is shaped so as to offer interesting changes of silhouette and light pattern, the crest, if left without further treatment would have been a disastrous culmination to an otherwise well-planned scheme. Hence it was decided to place dry material on the finished lagoons to form a crown, thus presenting a more pleasant and natural final appearance; and by forming this material to a general south westerly slope gaining some agricultural advantage.

4. The lagoons are planned to be finished to different levels to follow as closely as practicable the overall final profile and hence reduce the volume of material which has subsequently to be placed dry.

5. Originally the site was of only poor to medium quality agriculturally. The worst areas were in the northern part and planned coal mining would eventually reduce yet further its agricultural value.

6. In view of the considerable life-span of the Power Stations (which could be extended by re-planting) it was clear that a disposal scheme based on a staged development would be the most appropriate.

7. A 3-stage scheme was developed, with the first stage at the northern end of the site (for the reasons explained above), a second stage to the south east, and a final stage to the south west. Each stage comprises two lagoons, so as to permit their being raised and filled alternately. In addition, a separate pair of low-level lagoons, for emergency purposes, was to be constructed in the north west area of the site. (See figure 1).

8. Figure 1 also shows the Yorkshire Water aqueduct reservation which traverses the site. This reservation is kept clear of the works and is crossed by "bridges" so that heavy loads will not damage the pipelines.

9. The whole arrangement was discussed with the local planning authority who gave approval to the outline scheme.

Stage 1

10. The main constructional materials for the "hill" are pulverised fuel ash (pfa) and colliery waste. (Conditioned pfa and colliery waste for the embankments, slurried pfa for filling the lagoons.) The initial embankment construction had to be wholly of colliery waste since pfa would not be

available economically until the stations were commissioned. The preliminary site works for Stage I therefore included the construction of initial bunds (in colliery waste), separately enclosing the main and emergency lagoons, and allowed for continued construction in both colliery waste and pfa as the latter became available.

11. These preliminary works also included the construction of the vacuum filtration plant-house and associated tanks, etc., the slurry pipelines from the power stations, the on-site pipelines, roadworks, site drainage, drains for the main embankments, effluent settlement ponds and a booster and return water pumphouse. The latter, is firstly to pump the slurry into the lagoons when these reach a level beyond the capability of the power station pumps, and secondly to pump the effluent back to the River Aire, from which river the power stations drew the water in the first place.

12. Eleven feet, the height of the initial embankments, (translated more recently to 3.3 metres) has remained the lift height by which the embankment and lagoons have subsequently been raised.

13. Because of the great width of the main embankments at their base and the limited supply of colliery waste and conditioned pfa, it was not possible until comparatively recently, to construct these embankments in full-width horizontal layers. It was essential to build up the inside of the embankments to keep pace with lagooning, but little more material was available than that required to ensure the stability of the rising inner face. This led to a scar face being visible from outside the site, and (later) to stricter requirements governing the construction of Stage II (see para 18).

14. This scar face was open for most of the life of the first stage of the work. It was not until 1989 that the outside face everywhere caught up with the lagoon face and the soiling and seeding of the outer face, which followed construction as soon as possible, was able to provide an environmentally satisfactory appearance overall.

15. Dust nuisance is always an environmental problem when constructing earthworks in pfa. Although a thin crust does form on finished surfaces, this is fairly easily disturbed. In order to reinforce this crust, temporary surfaces, where of short duration, were sprayed with vinyl, and where of longer duration, were hydroseeded.

16. Floaters, which form a thick crust on the tops of the lagoons, are very easily disturbed by wind. However, the surface can be bound together by a covering of vegetation, which also breaks up the air flow immediately above the vulnerable surface. At Gale Common, rushes, reeds, long grass and even small trees have flourished on the floater crust, providing cover and a habitat for an interesting variety of wild life.

17. A variant to the originally planned constructional sequence has recently been investigated and will probably be followed. The total volume of material in the "crown" on Stage I is about 2,750,000 cubic metres. It has now been shown that by stepping each subsequent lagoon well within the outline of the one below, further lagooning above the originally envisaged levels will be possible, so that a large part of the "crown" can be formed from settled ash, thus offering a considerable constructional economy. The scheme would be incorporated in the next contract which is due to commence in the autumn of 1991.

Stage II

18. It is estimated that lagooning in Stage I will have been completed to the originally planned levels by early in 1992. However, there has already been a surfeit of bank-building material over and above that required for Stage I and it has been possible to commence the construction of the Stage II embankments well before lagooning (in Stage II) has had to start. This enables the embankments to be built to full width from the outset (a requirement of the Planning Authority) and soiling and sowing of the outer faces to commence as well.

19. Another constructional difference from Stage I, again of environmental significance, is the fact that for Stage II an impermeable membrane has been laid across the floors of the lagoons and below the chimney and finger drains. This is to ensure that the contaminated leachate from the lagoons is prevented from seeping downwards and entering the underlying aquifer. So far, there has been no indication of such being a problem in the case of Stage I, but it was nevertheless considered a worthwhile precaution in this environmentally sensitive era.

20 Although the bank building of Stage II starts from a more favourable position than did Stage I, construction is still controlled by the rate of material supply. The first target - which must be achieved before Stage II becomes operational - is to complete the banks to such a height that the lagoons provide sufficient head to drive the return water to the settling ponds, which are located near the return water pumphouse, some 2500 metres from the outfall towers.

21. The second key stage occurs when the

banks have reached a level, such that with their decreasing width (with increasing height), the rate of supply of bank-building material can support construction at the same rate as the lagoons are being filled. Clearly, from this stage onwards spare material will be available for completing the "crown" on Stage I. In fact, this critical level (+19.7m) should be reached well before the corresponding depth of ash has been deposited in the lagoons and therefore, for some considerable period before the slurried ash reaches this level, all the "dry" material should be available to be placed in the "crown" to Stage I.

22. The requirement to build to a level of +19.7 in advance of lagooning explains the inner profile of the Stage II embankments, which have to be free-standing (trapezoidal) up to this level, but can adopt a "fir tree" configuration at higher levels. (See fig 1 of Haws et al)

23. Stage II lagooning should continue until about the year 2007, when it will also be crowned with dry material. However, by that time, the Stage III embankments will have been completed to their initial critical level and these lagoons will be ready for operation. Stage III will not be complete until about the year 2020.

OPERATION

Owner Operations

24. The Booster and Return Water Pumphouse and the Vacuum Filtation Plant (VFP) at Gale Common are manned and operated by National Power on a 24-hour basis. National Power also organises the delivery of furnace bottom ash (fba) to the site by road.

25. Pumping around and off the site is controlled at the pumphouse; but the bulk of the controls are at the filtration plant where density meters determine when slurry is to be taken into the plant for filtering.

26. Once in the filtration plant, the slurry is thickened by transferring slurry from the bottom of a secondary tank while the surplus water and thin slurry are decanted off via a perimeter trough in the primary tank. The thickened slurry is kept in motion by recirculation and a rake arm while it is progressively drawn off for filtration. After a time the grading of the remaining slurry starts to blind the filters and at this stage filtration ceases and the slurry is pumped away to 'B' lagoon while the filters are cleaned.

Contractual Arrangements

27. The initial construction works for Stage I of the Gale Common scheme were mainly carried out prior to 1970 and were completed during the first operational contract, 1970/71. Since then, the operational contracts have normally been for two-year terms.

28 Originally, each two-year contract was not subject to variation of price and this situation existed until 1975. However, by 1976, two-year contracts were embracing variation of price clauses. Consequently it was considered that a long contract period might be financially advantageous to the Employer. Accordingly, in 1976, tenders were sought for both 2-year and 5-year terms, each incorporating variation of price. In the event, however, the "5-year" tenders did not offer obvious financial advantages and an alternative 2-year "fixed price" offer by one of the tenderers was accepted. Thereafter, all normal contracts have been subject to variation of price.

29. Until 1984 the Contractor was Lehane, Mackenzie and Shand Ltd, and since then it has been Taylor Woodrow Construction Northern Ltd. The current contract has another year to run.

30. In addition to the contracts described above, there have been two others, namely a Site Investigation Contract for the Stage II/III area of the Site (by Soil Mechanics Ltd in 1982) and the Stage II Initial Works Contract in 1986/7 carried out by Henry Boot Northern Ltd.

31. On the Employer's instruction, contracts are based on the CEGB General conditions of Contract for constructional Works, 1971 Edition (which document is similar to the old ICE 4th Edition) and on the old ICE Standard Method of Measurement, 1974 Edition.

32. Over the years the details of the contracts have been refined and adapted, but essentially they have been contracts to operate the Site for a specific period of time, including any incidental works that might become necessary (although the last two contracts have also included further preparatory works for Stage II). Consequently the Bills of Quantities have tended to include a fairly large P.C. and Provisional Sums element to cover operational and maintenance requirements unknown in detail at the time of tender.

33. Since 1980 the contracts have included intermediate completion date requirements. Such Key Dates relate to items of work for which there is a definite programme requirement, such as clearance of shale stockpiles by the end of the summer period (so that the maximum storage capacity is

available at the onset of winter, during which little or no shale can be placed in the Works). Other Key Dates have applied to such items as emergency lagoon embankment raising, road resurfacing, pipeline replacement etc. Monthly valuations are checked on Site and passed to the Engineer's Head Office where a payment certificate is prepared and issued to the Employer, for payment within 42 days of receipt of the valuation on site.The minimum amount for interim certificates is currently stipulated as £50,000, but the volume of work is seasonal and actual monthly payments vary considerably.

Site Organisation

34. The current contractor, Taylor Woodrow Construction Northern, operates the site as a management contractor with all work being carried out by the principle subcontractor, V.H.E. The latter keeps a small permanent staff on site while maintaining a flexible position with the bulk of his plant and labour.

35. Pfa is handled by five Moxy articulated dumptrucks, a loading shovel, a bulldozer and a vibrating roller. Normally shale can only be placed into the Works between April and October and involves a loading shovel, lorries, two or three bulldozers and a compactor. At least one bulldozer, for stockpiling, and one backactor, for all the trimming and trenching associated with the job, remain on site full time. The total manpower varies from 20 - 35 men, winter to summer.

36. The Consulting Engineer provides on site an ER, inspector and clerk, with appropriate additional seasonal and specialist back-up from head office as required. The ER has responsibility for supervising the Works and for managing the site on behalf of the Station Manager of Eggborough Power Station, who has responsibility for the overall operation of the lagoons.

Communication

37. The scheme is monitored by both the Employer and the Consulting Engineer at regular monthly meetings with the Contractor. In addition, the site staff, Contractor and Employer meet British Coal respresentatives on a monthly basis at "pit" level to resolve any problems in connection with the shale supply.

38. At a higher level there are meetings at least annually with the appropriate authorities to discuss mining subsidence, landscaping, overall management, and planning and the environment.

Checking

39. Routine records are maintained with reference to:

- weather (the site is a Met. office rainfall station)
- piezometric levels (6 locations in the embankments and borehole dip readings elsewhere)
- mining subsidence (levels and strains)
- shale and pfa densities, and furnace bottom ash (fba) grading
- daily lagoon levels and monthly seepage readings
- daily pipeline and embankment inspections.

Environmental Protection

40. The day to day control of dust on the site is covered by the inclusion of two water bowsers in the contract - one on permanent duty and the second on standby. The machines are alternated weekly to ensure, as far as possible, that the standby will operate effectively when required.

41. The extensive interim embankment surfaces, which are beyond reach of the bowsers, are vinyl sprayed or hydroseeded (see para 15).

Certification of Lagoons

42. Originally the Stage I main lagoons and the emergency lagoons were inspected and certified under the Reservoirs (Safety Provisions) Act, 1930 and although it has not been necessary to register them under the Reservoirs Act, 1975, National Power, as a responsible operator, treats them as though they were so registered. Accordingly, they are inspected and certified by Mr E T Haws as Construction Engineer, and preliminary certificates are issued by him, to the Employer, each time a lagoon embankment is raised. Ultimately a final certificate will be issued for each lagoon.

43. Because the Stage II lagoons will stand filled with water (to protect the membrane lining) for some time before being used for slurry, they have been registered under the Act. They, too, have been inspected and certified by the Construction Engineer and no doubt this procedure will continue, although the present intention is to de-register the lagoons when they become operational (say 1991/92).

44. The preliminary certificates issued to date for the various lagoons have sometimes contained conditional clauses specific to individual lagoons and to situations pertaining at the time, in addition to normal clauses stipulating requirements for daily inspections, keeping of appropriate records, maximum permissible slurry inputs, water and spillway levels, etc.

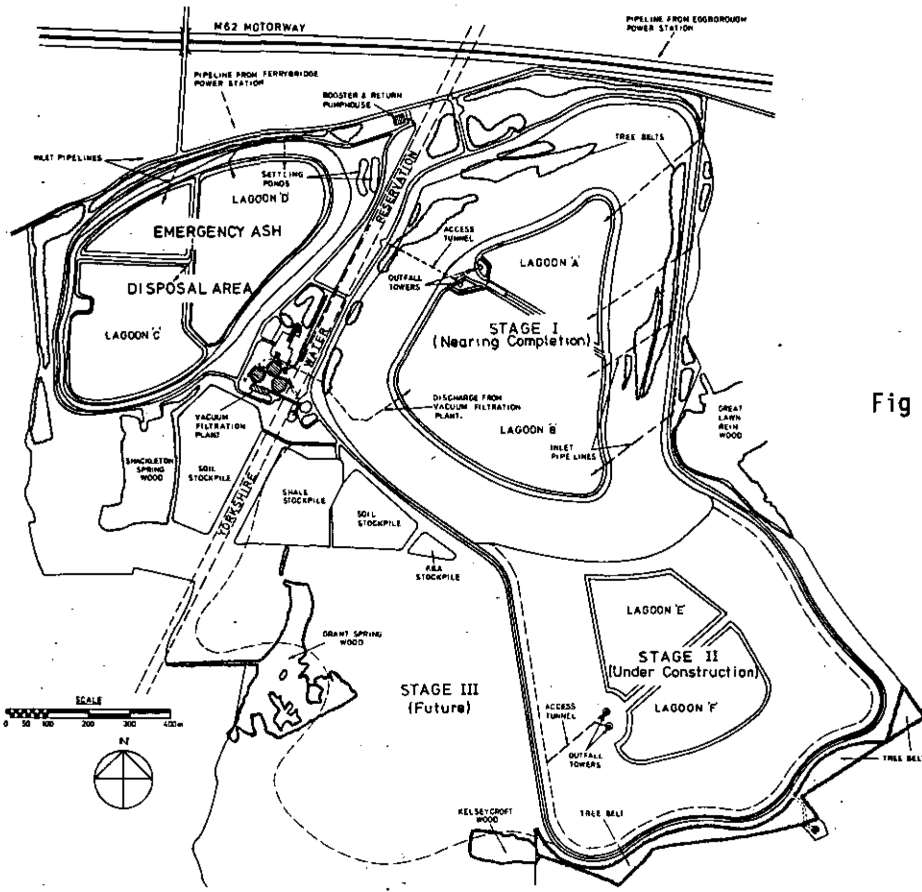

Fig 1. Layout of Scheme

Fig 2. South-east Corner of Stage II (model)

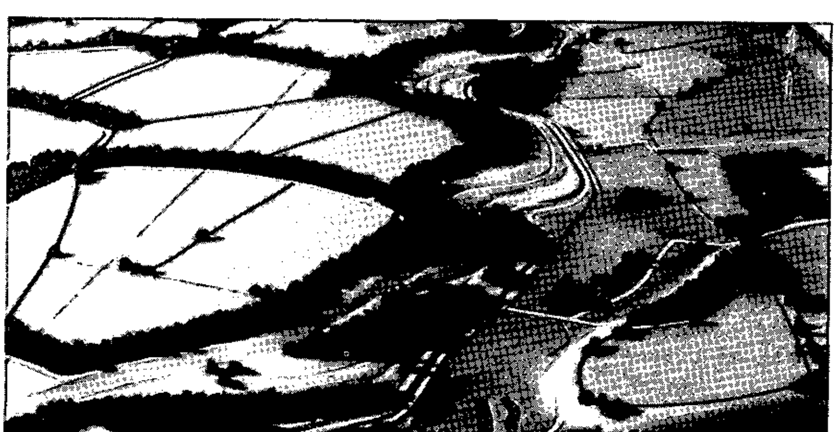

Fig 3. West Side of Stage I (model)

Examples of additional conditions are those pertaining to pipelines on the embankments, the required action when settled ash builds up local to the points of input, action to be taken in the event of outfall valve malfunction and clauses relating to mining subsidence.

Costs

45. The disposal of ash is an inescapable facet of coal-fired electricity generation. Some ash can be solid, but overall the disposal is effectively an operational overhead.
There is generally a ready market for any fba and Eggborough also sells some of its pfa make as a pozzolan, but the Authors have no information regarding the receipts from such sales. Neither have they any details of National Power's own costs relating to ash disposal.

46. Therefore the following approximate costs are based solely on the civil engineering contract sums at the date of those contracts and on the total quantities of pfa sent to Gale Common.

47. For the whole period, to the end of 1989, the cost (including initial construction works amounts to approximately 80p/tonne of dry ash; for the more recent period 1987/9 it is approximately £1.75/tonne (including the concurrent initial works for Stage II). The latter figure reflects the more up-to-date costs of the preparatory works for Stage II, compared with the Stage I pre-1971 values included in the former figure. The most recent "disposal only" contract (1986/7) gives a value of approximately 80p/tonne.

LANDSCAPE AND RESTORATION

Concept

48. The design strategy for restoration of Gale Common hill was established soon after the engineering concept. The late Brenda Colvin CBE PPILA produced the first design in the early 1960s, the CEGB at that time pioneering the idea that a restoration scheme should be prepared before site work started. The design approach was a poetic response to a new problem; the preliminary report of 1962 for instance cited "the objective of creating a completely new landscape feature, of distinguished and pleasing form, contributing to the interest of the landscape, as do existing hills".... "High contours, thus indicated, will be visible from great distances in this flat area. Crops on these contours exposed to higher winds than existing levels will require shelter planting in the form of tree belts. High priority should, however, be given to the visual effects of tree belts."

49. The top of the hill was designed as an arable plateau sloping towards the south-west. The sides of the hill, pasture or wood because of their steepness, were imagined as terraces spiralling upwards, the corners of the hill being elongated into terraced silhouettes. The report explains the ideas behind this design: "In Britiain man-made terraces were common in neolithic and iron age periods both for agriculture and defence. If this new hill can be made in terraced form it would be of comparable scale and character to those early ones; an abstract sculptural group. No attempt at reproduction of other earthworks is proposed. A frank artefact may be preferable to a naturalistic hill in this flat area where it can scarcely be seen as part of the surrounding geological structure".

50. These themes have guided development of the project over the succeeding two and a half decades, latterly under the guidance of the Author who, as partner of Brenda Colvin from 1969 until her death, is still retained as Consultant.

Engineering Restraints

51. The cross section of the bulk of the hill is determined by the need for steep side slopes to maximise the proportion of interior volume to dry-placed banks. The average slope is 1:3, with engineering restrictions on the acceptable length of steeper slopes and on deviation from the engineering section line. However, at outer corners it has proved possible to increase the distance between crest and toe without placing much extra material, by taking advantage of the horizontal difference between the tightest lagoon shape possible on plan and the angular corners of the site. This has given the opportunity to design an interesting and less steep profile where the hill is most visible in silhouette. The photograph (figure 2.) of a model made in the early 1980s illustrates how the south-east corner of Stage II, for instance, takes advantage of this concept.

52. The plan form of the hill, already curvaceous due to exigencies of the site, has been further enriched by such devices as taking advantage of re-entrants which occur where a later stage is butted against an earlier. Rather than smooth out the form, the hollow is emphasised. Tracks and berms, to prevent run-off water accelerating too fast down the steep sides, spiral up the side of the hill, not parallel but in a slightly variable relationship to each other. "Though the separation between the berms never reaches a distance beyond acceptable engineering criteria, the

possibility of variability has made it possible to design a rich superficial form. This flexibility also makes it possible to arrange tracks to leave the base and arrive at the top in convenient locations.

53 The concept of dealing with berms in an irregular spiralling manner derived from following what at first seemed very constricting engineering criteria to solve all the demanding structural problems. Yet the very strictness of the engineering disciplines imposed - minimum and maximum berm widths, minimum and maximum gradients, maximum deviations from structural cross sections, strict rules at the most fragile positions, maximum distances between berms - all these give an underlying unity of form to the apparent casualness of layout in the final landscape details. In just such a way a tree which looks haphazard in form is constructed by the hidden geometry of natural forces.

Woodlands and Pastures

54. The photograph (figure 3.) of the model's west side, giving some idea of this underlying land form design, shows the principles of woodland layout adopted. On the top plateau, simple edge and intermediate belts shelter potential arable fields; small clumps pick out high points to enrich the silhouette of the hill. On the sides of the hill trees are blocked into elongated woods frequently continued from existing woodland below, picking up the spiral lines and leaving large areas of hillside under pasture. During the layout of the Stage I hillside woods on site, long sweeping lines have been emphasised to show more clearly the land form below, as this was perceived to be more attractive in appearance than too great an artifice of casual informality.

55. Since the model in the photographs was made, studies of pasture field patterns for the hillside have resulted in fragmentation of open land into enclosures 1-2 hectares in size. These have been grouped according to MAFF recommendations for sheep farming. The detailed shapes of the fields was determined by trial and error on a working model. At the request of the planning authority a larger 'outbye' pasture is proposed for the south-east corner of Stage II to show the land form more clearly at the point.

56. At original ground levels, several old woods have been kept, and are being managed to enrich wildlife; a ground cover of bluebells and anemones has been given renewed vigour by gentle management improvements. An attempt has been made to move the floor of a wood, lost under Stage II, onto the woodland sides of Stage I; it is too soon to know with what success. One ancient wood is receiving a special management regime following ecological advice. New screen woods have also been established along the edges of the site at ground level to hide low level vehicular movements and stockpiles of colliery waste from local roads and villages.

57. The chemicals in pfa and its very small particle size limit the diversity of tree species which can be grown. The CEGB has long been carrying out scientific studies of what will grow. Site trials were also undertaken as soon as an ash face was available; these revealed that minor variations in restoration technique were less important than the correct choice of species and certain basic amelioration measures. A soil layer on top, ripped into the compacted ash surface at the interface, induced root growth in tolerant species. Root pits were dug after five years which showed good penetration into the pfa, so that both stability and tolerance of the dry Yorkshire plain climate can be hoped for in future.

Soils and Maintenance

58. Great care has been taken at Stage II with soil stripping. Four separate long-term soil heaps cater for top soil in two grades and subsoil in two grades. The top grade is the light sandy soil which forms the limited areas of top class farmland beneath Stage II; this will be used to restore the top of the hill for arable use. The sides of the hill use second grade subsoils for restoration, which is often heavier in texture, or a mixture of soils from Stage I stripping.

59. Vegetation establishment at Gale Common has been excellent. This is partly due to thorough trials and demanding specifications. Much of the credit however must go to excellent standards of planting and aftercare achieved by the Mobile Resources Unit of National Power. In the creation of landscapes all efforts of imagination and technical understanding are in the end dependant upon good husbandry of the living skin.

60. The aim of restoration is to create a new living skin across the surface of the hill to profiles and plan shapes which both please the eye and satisfy foreseeable functional requirements.

ACKNOWLEDGEMENT

Acknowledgement is made to National Power Ltd for permission to publish this paper.

Discussion

In view of the large number of papers in this session the chairman for the session, Dr A D M Penman, introduced the subject and reviewed the papers.

A D M PENMAN
This subject attracted 35% of the papers submitted to the Conference. The nine papers in this session (papers 7-16) can be considered in terms of the organisations that have submitted them (Table 1), or in terms of the heights of the tailings dams (Table 2).

The Bulgarian dam described in paper 8 was the only one built almost entirely from tailings. During the work, the effect of delivery pressure on the sizes of coarse grains from the cyclones was measured (2.0 to 2.3 bars produced particles of 0.25 to 0.26mm, while 1.6 to 1.9 bars gave 0.20 to 0.23mm). Slurry density in terms of solids to water by weight was 1 to 3.0 - 3.5 while by volume this was 1 to 7.6 - 9.6. In the early stages when, because of the valley shape, it was difficult to keep dam height safely above lagoon level with the coarse fraction available, design might have been helped if they had had access to the TADAM programme described in paper 10.

The large pulverised fuel ash dams and lagoons at Gale Common have been described in papers 15 and 16. While tailings dams do not come under the Reservoirs Act, CEGB decided to have Phase 1 designed and constructed in accordance with the Act. With the formation of National Power in preparation for the privatisation of CEGB, Phase 2 will not be put under the Act when it is being built with pfa in the lagoon. Initially, however, a layer of water has had to be put over the impervious plastic sheeting covering the foundation to hold it down. The volume of water involved brings it into the Reservoirs Act, but apparently the intention is to bring it out again once fly ash is pumped into the lagoon. The earthquake intensity mentioned in paper 15 is apparently Modified Mercalli and the "dendritic" pattern of drains means simply that the drains were laid out like the roots of a tree or system of tributaries going into a river.

Tailings from the Neves Corvo copper mine in Portugal is rich in pyrites and to avoid the acid caused by oxidation, the tailings was discharged into the lagoon below water and kept below the water level. This produced very low densities. To get a greater weight of tailings into the storage volume, spray bars were used to place the tailings above water but keeping it continuously wet (paper 11).

Paper 9 is a geotechnical paper on the art of building low banks on very soft clay. It can usefully be compared with Paper 1. The soft clay clearly had a dried crust and was reinforced with tree-roots. It is not clear how gypsum cakes are to be stored to a height of 12m without instability.

The problems of waste disposal from china clay mining are discussed by paper 13. Paper 12 gives a valuable assessment of things that can go wrong with overflow systems: a vital factor for the safety of tailings dams which in general are particularly sensitive to overtopping. It is suggested that most failures initiate from faults in the overflow arrangements.

The safety of British tailings dams is discussed in paper 7. It points out that there is no register of all tailings dams and so it is not known how many exist. A comparison is drawn between the requirements of the Reservoirs Act and those of the Mines and Quarries Act that relate to tailings dams. The high standard of safety of embankment dams owes much to the detailed studies of behaviour that have been made. We need to apply this approach to tailings dams.

Various names have been given in the papers to the same thing, eg in South Africa the impoundment is often called the dam and the retaining dam is called a wall. The terminology given in Table 3 is therefore proposed for use in describing tailings dams.

Table 1. Organisations submitting papers

Organisation	Paper No
WLPU (now Knight Piesold)	10 11 12
Rendel Palmer & Tritton	15 16
MRM Partnership	13
Binnie & Partners	9
Building Research Establishment	7
Higher Instit Arch & Civ Eng, Sofia	8

The embankment dam. Thomas Telford, London, 1991

Table 2. Heights of tailings dams

Paper No	Tailings dam	Material stored	Country	Height (m) Prest	Height (m) Propd
10	Los Leones	Copper tailings	Chile	160	300
8	Elatzite Mine	Copper tailings	Bulgaria	95	145
15,16	Gale Common	PFA	UK	51	51
11	Wheal Jane	Tin tailings	UK	40	53
11	Cerro da Lobo	Copper tailings	Portugal	28	35
9	Tioxide Group plc	Gypsum waste	Malaysia		2.5

Table 3. Proposed terminology

Preferred term	Non-preferred term
Dam	Wall
Lagoon	Reservoir, Dam
Wall drain	Chimney drain
Continuous wall drain	
Tailings - singular	Tailings - plural

C B ABADJIEV (Higher Institute of Architecture and Civil Engineering, Sofia)
I should like to express my gratitude to the British Dam Society for the opportunity to share experience with British colleagues. I wish to give a brief overview of paper 8 with particular attention to three problems:
- The area of the tailings dam was the same as that of Nottingham University (where the Conference was held), about 300 hectares. The dam was built by the downstream method using 75 cyclones on the starter dam crest, the number increasing to 130 during this last year, the 8th year of construction.
- It was difficult to achieve the design slope from the coarse discharge from the cyclones. This was overcome by enlarging the discharge aperture of the cyclones to give a wetter mix that spread more readily.
- Dust control. Rotational sprinklers with a range of 45m, then vertical wetted screens were used but with various problems. Currently two new methods using chemical and chemical/ biological stabilisation are being used. This involves waste from the timber paper cellulose industry and seeding grass, which has to be kept watered.

S A CALE (Knight Piesold)
Following Dr Penman's remarks, I should like to make several points:
- There is an increasing use of sand filling in underground operations and this tends to decrease the sand content of tailings available for dam construction.
- Tailings are not always released from the dam crest. In cases where it is more economic, tailings are released at the head of a valley and retained by a traditional earthfill dam which, initially may simply retain water, before the tailings in the valley extend to it.
- There should be a register of all the tailings dams in the UK.
- There is no doubt that earthquake risk must be taken seriously.
- Instrumentation and monitoring should be similar to that used for embankment dams but because of the much longer construction period, redundancy needs to be built in to cope with losses of equipment that may occur.

N THOMPSON (Knight Piesold)
Dr Penman asked about the value of computer software such as TADAM (paper 10).
Tailings dams involve retaining structures (referred to as the "wall" in the ICOLD manual on tailings dams and dumps) and retained material (referred to as the "stockpiled material"). The ratio of volume of stockpiled material to volume of wall varies widely depending on the topography. One of the aims of the design process is to maximise this storage ratio within the constraints of the site. TADAM is a design tool. It models tailings deposition and calculates the capacity of an irregularly shaped impoundment filled with a material that has a sloping upper surface. This calculation can be carried out by hand using contour plans and a planimeter. The value of TADAM lies in the automation of an otherwise tedious and time-consuming design process.

Paper 10 describes the use of TADAM for the design of two tailings dams. The Los Leones dam at Andina is constructed in extremely rugged terrain and would have a final overall height of 300 m. The Sohar dam in Oman is constructed in a relatively flat area and will have a final overall height of about 33 m. The Andina mine study involved the investigation of four dam sites with approximately one hundred possible filling patterns for each site. Each filling pattern would take about one man-day to calculate using contour plots and a planimeter so the whole volumetric study would take about 400 man-days. The actual study using TADAM took about 40 man-days. The value of this software in saving time and reducing tedium is obvious.

The effort involved in optimising the layout of a tailings dam decreases as the original topography becomes flatter. TADAM would not have saved a significant amount of time if it had been used for the design of the Gale Common ash disposal scheme. For this reason the software is not used by Knight Piesold's office in Sydney, Australia. However, the value has been proven by use on many schemes at their Ashford, Derby, Denver and Vancouver offices.

S HAWES (Independent Consultant)
For some twenty years, my practice has been retained by a major agricultural produce processor to design, construct, maintain and repair settlement lagoons and effluent reservoirs. The ten million tonnes of produce processed each year results in 75% being discharged as water, and 7% as topsoil. Thus, with rainfall and plant foul water run-off, storage lagoons are required for nearly ten million tonnes of water and seven hundred thousand tonnes of soil. This output is stored in lagoons awaiting purification before discharge to river, or excavation to drying beds. These lagoons vary in depth from two to ten metres, and have a capacity of between thirty thousand and two hundred thousand cubic metres.

In some cases the mud, which varies from fine sand to clay, is dropped as "tailings", the water passing on into storage in reservoirs which the client accepts as such under the 1930 and 1975 Acts. More recently there has been a change to combining settlement with storage, thus a lagoon will contain about one tenth settled mud, nine tenths water with a BOD of 4000 and a fair amount of rotting organic matter.

Failures during some thirty years have varied from breaches due to overfilling, land drains below embankments blowing, causing water supply intakes in the rivers below to be closed, banks slipping due to their having been cut away at the base when mud was being dug out and cracking due to inadequate compaction. Old mild steel pipes have rotted, and in one case the pipe re-activated itself and sent the contents of one lagoon down to the town sewage works which it closed for several days. Frequently ponds are emptied of mud by cutting the banks away, and then re-building without compaction.

Whilst Section 1 of the Act refers to "water as such" it immediately goes on to say that accordingly it does not include a mine or quarry lagoon which is a tip within the meaning of the Mines and Quarries (Tips) Act 1969. What it does not say, is that any lagoon which is not a tip within that Act is automatically outside the Reservoirs Act 1975. It then says quite unequivocally that it covers any reservoir "designed to hold" or "capable of holding" more than 25 000 cubic metres of "water". Note that it here refers to "water", and not "water as such".

Whilst the bulk of the water which my clients' lagoons hold comes as "water as such", it is within a short time dirty water with a BOD of 3000 to 4000, and, as it enters the lagoon, it carries topsoil which has been washed off the produce. I feel I must now refuse to advise on such lagoons until they have been registered as large raised reservoirs, since whilst they are unregistered they are, in my opinion, illegal structures, and since I would have no legal power to fix a top water level, if failure occurred I could be an "accessory before or after the fact". Many of them have not been designed to hold water, simply because they have not been designed. They just grew. But they are capable of holding water as defined in Section 1 of the Act, and I cannot see that legally or logically they do not hold "water as such".

Discussion amongst engineers is often biased by those who had a hand in fashioning the Act, who know what was intended. But now we have to interpret the Act as it is written, and not as it may have been intended. What is rather alarming about the present situation is that those with experience of cost effective solutions in this area, particularly those with responsibilities for pollution prevention, are to be excluded from practising by edict of the Reservoirs Committee. They refuse to recommend the Minister to create a suitable Panel for those engineers whose experience is in the field of industrial and agricultural medium sized water and effluent holding structures. This is a field where more, not less, engineering input is necessary, and where the very large consultancies have little experience of the techniques which produce the safe and cost effective solutions so necessary to the competitiveness of British industry.

A D M PENMAN
During the drafting of the 1975 Reservoirs Act an endeavour was made by the committee of the Institution of Civil Engineers to bring in the storage of liquid other than water with a view to tailings dams and the fact that in America they had been storing oil behind embankment dams. It was turned down as something which would put additional complications into the rulings of the Act.

The greater density of tailings makes its destructive power much greater than that of water. Escaping tailings can crush cars and demolish buildings which, under the same depth of escaping water, would only be damaged by being wetted. As Mr Hawes has indicated, many tailings lagoons could hold more than 25 000 m^3 of clean water above the tailings if they become flooded and it might be argued that they should come within the existing Act, although clearly it would be preferable to have special provision for tailings dams.

A STREET (MRM Partnership)
In his introductory presentation Dr Penman suggested that the upstream method of tailings dam construction tended not to be the preferred approach, indeed it was frowned upon. There are of course a wide variety of mining and quarrying processes, each producing different types and quantities of solid waste (overburden, waste rock, crushed fines, etc) and tailings. In addressing the design of mining and quarrying waste disposal schemes it is necessary to work within certain constraints which include:
- the need to utilise the waste materials being produced; the rate of production may vary and this may be a significant controlling factor in the choice of tailings dam construction method

- the need to ensure the stability of the structure, both during construction and long term
- the need to develop a scheme which has minimal visual impact; this is becoming a key concern, particularly in the UK

For certain schemes the upstream method of tailings dam construction may well prove to be the preferred approach. There are a number of good examples of such dams currently under construction in the UK which provide an economic and visually unobtrusive solution, as well as being technically sound.

With regard to the need for a register of tailings dams, it is my opinion that such a register is required. A number of tailings dams figure among the largest "embankment" structures in the UK. Under the Mines and Quarries legislation the "competent person" is required to inspect and report on such structures at prescribed intervals. Unfortunately the qualifications and experience required for the "competent person" are not defined. There clearly would seem to be some merit in extending the existing 1975 Reservoirs Act to cover tailings dams since this would bring the "Supervising Engineer' and "Inspecting Engineer" into play. If this is considered to be going too far then a formal register of dams would at least be an improvement on the current, ill-defined position.

E T HAWS (Rendel-Parkman)
There has been considerable discussion on anomalies within the Reservoirs Act. One such item concerns the exclusion of ash lagoons from the legal requirements of the Act. However, the responsible attitude of the CEGB and its successor companies has been to operate such lagoons as if they were within the purview of the Act. The final legal step of registration has naturally been avoided. As an indication of the effects of the legal distinction it is worthy of note that at Gale Common the new lagoons for Stage II are firmly within the Act whilst they contain only water as temporary protection to a sealing membrane, whereas they will come out of the Act directly ash starts to be placed. Regarding the cost of applying the regulations of the Act to such lagoons, the only cost item is the fees for certification which are very small indeed related to project cost.

Studies are currently underway into the possibility of achieving part of the domed finish required for landscape architecture on Stage I by lagoons within embankments set in from the main crest. This implies founding the additional bunds on lagooned ash from which it is hoped that floaters have been displaced. Nevertheless allowance will have to be made for the residual presence of floaters which pose an interesting soil mechanics problem as they are of less density than water. Floaters or cenospheres are hollow glass balls of very small dimensions. They are of considerable value as filler or base for such items as paint and face powder if they can be retrieved economically. Such retrieval is currently underway but is a difficult operation as handling is troublesome and ash blow must be avoided at all costs, particularly with the adjacent M62. Removal of the floaters also requires stripping off the substantial vegetation which has floated up thus leading to a chance of wave action and also removing the habitat currently enjoyed by wildlife.

The remark has been made that dams have been put out of operation because of mining subsidence. However, it is the case with power stations on the coalfields that, although the power stations themselves are on a pillar sterilised from extraction, the ash lagoons are not so protected. Movements of a metre and above have been experienced and the lagoons have been put back into service satisfactorily after repairing cracks, drainage systems etc. For this reason the lagoon arrangements commonly include alternative containers so that active slurry placing can avoid lagoons currently suffering subsidence.

The culvert containing the outlet pipework from the Gale Common outfall towers has been subject to foundation settlement and stretch under 50 m of fill and a sag of 600 mm has been accommodated safely.

Finally on the subject of membranes, a comparatively cheap polythene sheet has been placed over the whole of the base area of the Stage II lagoons to ensure protection of the underlying aquifer from leachate. It was not feasible economically to underdrain or surcharge this membrane and consequently "whales" appeared when the lagoon was flooded. This phenomenon is apparently common in these circumstances and we believe was entirely due to trapped air with no known presence of methane. In view of the limited purpose of the membrane the procedure adopted was to make a small incision in each "whale", thus releasing the trapped air. It is believed that any residual seepage through these small cuts will be quite negligible in terms of any possible effect on the aquifer.

R ORANGE-BROMEHEAD (Rendel, Palmer and Tritton)
The new ash lagoons at Gale Common have been lined with a plastic membrane to minimise the possible pollution of the underlying aquifer. When laying the membrane, material was placed on it only sufficient to keep it down against the effects of wind. Upon filling the lagoon it was expected that the water would drive the air out from under the membrane. In the event, air became trapped in bubbles which appeared above the rising water giving the impression of a bay of whales. The solution finally adopted was to lower the lagoon level so that a man in waders could puncture each 'whale' with a small incision. The loss of impermeability to the future ash filled lagoons is not seen as significant.

H T MOGGRIDGE (Colvin & Moggridge)
I wish to refer to some landscape aspects of the Gale Common scheme:
- The plants on the top of the lagoon, growing on the floaters, were not an accident, but were designed as a means of suppressing dust.
- The landscape consultant can make a contribution to the edge of the site in terms of saving existing woods, planting etc. This is important for good public relations.
- Some trees have been planted in the ash-hill and have 6 years growth. Trial holes have shown roots entering the ash layer beneath 150mm of topsoil. There is a limited number of species that grow successfully on ash.
- The after use of the site will be mainly for agriculture including fields for sheep and some for arable with some areas of woodland.
- The cost of this landscaping work is about 1% of the total. We were landscape consultants at Roadford reservoir and there the cost was about 2%.

A R GRIFFIN (Binnie and Partners)
Further to the comments made by Dr Penman during his summarising of paper 9:
- Gap in the piezometric data. It should be explained that data was obtained during this period. Readings showed a rapid increase in pore pressures suggesting that failure of the trial embankment was imminent. However, visual observations and readings of deformation and settlement did not indicate that the embankment was under undue stress. Subsequently the supervising engineer obtained a second pneumatic piezometer readout unit to check results and found that the original unit was malfunctioning. As a result it was decided to remove the erroneous or suspect data from the graph.

- Vegetation under the trial embankment was cut and rolled to form a mattress under the embankment fill. Larger tree roots were grubbed out. This is normal practice in south east Asia particularly under road embankments crossing soft alluvial clays. The mattress acts like a reinforcing fabric and supports the fill but makes stability analysis difficult.

- The original design was based on limited vane strength data obtained from boreholes put down by others nearby for factory construction. The data was insufficient to enable a detailed design to be carried out and subsequently a more extensive investigation included closely spaced in situ vane tests and piston sampling. However, the results of laboratory tests on the piston samples suggested that the material was disturbed and therefore could not be relied on. It was necessary before this data became available to produce a preliminary design with estimates of quantities for tender purposes. The preliminary design was therefore conservative and the aim of the trial embankment was to define a more realistic but stable structure.

The trial embankment enabled a reassessment of this design with savings on materials. Also the revised design will enable a review of the process of placing the waste gypsum material behind the retaining embankments and the heights to which the gypsum can be stacked. The preliminary design allowed the waste material to be stacked in layers 1 m thick with 20 to 35 m wide benches between layers raising the whole structure to about 12 m high in the centre. This design is being reviewed as a result of the trial embankment test with a view to reducing the bench widths and increasing the storage capacity of the tip.

- Gypsum waste or tailings. This material consists of fine coarse silt size crystals (generally needle or lozenge shaped) mixed with iron hydroxides. It can be placed in the landfill area and rolled and compacted. Laboratory compaction tests (2.5 kg) indicate a maximum dry density of about 1.27 Mg/m^3 and optimum moisture content of 24%. The moisture content of the waste on production is about 23% but 'ages' rapidly, the iron hydroxide rapidly dehydrating to iron oxides which in turn act as a cementing agent between the gypsum crystals. Consequently its properties are time dependent. Quick undrained tests on compacted samples with moisture contents ranging from 38% to 13% gave strengths of 119 kN/m^2 to 317 kN/m^2 respectively. Effective stress tests gave ϕ' values in excess of 50° at low effective cell pressures.

Tests and observations therefore indicated that the properties of the soft marine clays would dictate the final design of the embankment and tailings retention area. Also since the factor of safety of the embankment was very sensitive to small changes in the adopted strength parameters of the soft marine clay it was considered that a trial embankment was essential to enable a sensible economic design to be produced.

B G CHIN (Klohn Leonoff)
Dr. Penman has drawn a comparison between papers 1 and 9. A difference is that a total stress method was used in paper 9, whereas we used effective stress methods in paper 1. When using undrained strengths, you must predict how fast they will increase during construction, whereas with effective stresses you take pore pressure dissipation into account. In our Forty Mile East Dam we observed virtually no dissipation, while at the trial embankment in Malaysia r_u was of the order 0.5, indicating quite a lot of dissipation during construction, giving a gain of strength in the foundation.

C G GREGORY (Rofe Kennard and Lapworth)
With reference to the question of whether ash lagoons came under the provisions of the Mines and Quarries (Tips) Act 1969 or the Reservoirs Act 1975 and its predecessor the Reservoirs (Safety Provisions) Act 1930. My firm has been responsible for the detailed design and supervision of construction of several large ash lagoons and the most recent lagoon at Fiddlers Ferry Power Station stored water to a depth of

approximately 10 metres above the natural level of Cuerdley Marsh on which it was constructed. The enclosing lagoon embankment was approximately 3 kilometres in length and hence it could be seen that a very considerable volume of water was stored. Such a structure surely came within the Reservoirs Act 1975 and the Act was followed in that an appropriate panel engineer was appointed to undertake the design and supervision of construction of this and other similar ash lagoons elsewhere.

G N JONES (National Power)
There have been a number of references in the papers presented today and ensuing discussions to ash lagoons operated by the CEGB and one of its successor companies, National Power. Firstly, I would like to offer some reassurance to everybody that National Power, as a responsible company with operational safety as one of its highest priorities, intends to continue the policy of the former CEGB and to treat large ash lagoons as if they are reservoirs under the Reservoirs Act. This will even apply when they are not registered as such with the local authority.

In the discussions about extending the scope of the Reservoirs Act to cover tailings dams, I query whether this could be done without considerable thought and probably re-drafting to cover extra provisions particular to this type of structure. A couple of points come to mind:
- Tailings type dams typically have very long construction periods and the Reservoirs Act requirement for a single Construction Engineer may be difficult to meet. It would be nice to think that Panel Engineers were immortal but, in any case, consultancy agreements of the required length would not be palatable to clients such as ourselves. A greater emphasis on supervision of construction is, I believe, required.
- The definition of capacity and therefore the lower bound definition of reservoirs under the Act needs careful thought in a tailings dam. At what point does the material contained cease to be liquid or to potentially liquefy during a failure?

17. Evaluation of dam safety at a series of hydropower dams including risk assessment

D. S. BOWLES, L. R. ANDERSON, and T. F. GLOVER, Utah State University, USA, G. S. TARBOX, Consultant Engineer, Wanconda, IL, USA, R. B. WAITE, Utah Power and Light Co./Pacific Corp. USA, and P. E. YIN AU-YEUNG, ECI, Denver, USA

Utah Power and Light Company (UP&L) owns a series of dams on the Bear River in Utah and Idaho, U.S.A. These dams are regulated by the Federal Energy Regulatory Commission (FERC). Not all the dams currently meet the FERC's standards for flood and earthquake loading. A dam safety evaluation study was performed using an incremental consequence assessment and a risk assessment. Each dam was evaluated considering its potential for complete or partial failure due to floods, earthquakes, internal causes, or upstream dam failure. The safety evaluation procedure and results are presented. Also the dam owner's perspective on the role of the dam safety evaluation results in the selection of remedial measures which were accepted by the FERC is presented.

INTRODUCTION

1. Several Utah Power and Light Company (UP&L) dams on the Bear River in Utah and Idaho, U.S.A. were found inadequate based on the current probable maximum flood (PMF) and the recently revised maximum credible earthquake (MCE). This finding was surprising to UP&L management because the dams had a satisfactory record of performance and because the highest floods of record were less than 10% of the new PMF's. Also the very competitive market for electrical power did not justify the investments that the FERC appeared to be requiring. According to UP&L's Waite (1989a) the estimated cost of remedial upgrades was $20 to $25 million. This translated to $155 to $195 per installed kilowatt or an increase in the average cost of generation of three to four mills per kilowatt-hour.

2. UP&L decided to commission a comprehensive risk-based evaluation of their dams in order to provide them with a thorough understanding upon which to base their proposals to the FERC. The evaluation was costly to perform, but the savings in remedial action costs made possible as a result of the study more than justified the costs (see Para. 36). The study was conducted by ECI, Denver, Colorado, and RAC Engineers and Economists, Logan, Utah. A detailed description of this study is presented in ECI/RAC (1988).

3. The Bear River UP&L dam safety evaluation was performed through an incremental consequence assessment (ICA) and a risk assessment (RA). ICA, which is recognized by FERC regulations, provides estimates of increases in economic damages or life loss for postulated scenarios of dam failure compared with cases which consider no dams on the Bear River (i.e., natural flows). While ICA deals with incremental consequences due to a dam being added to a natural river system, it does not include consideration of the chance of these failure scenarios actually occurring; therefore, it is a "what if" type of assessment. To add the perspective of the chance of occurrence, risk assessment was performed for the UP&L Bear River dams. The risk assessment approach is specifically mentioned in FERC Engineering Guidelines, and it is currently used by the U.S. Bureau of Reclamation (Von Thun 1987) for evaluating the safety of existing dams. Also, its use has been recommended for evaluation of existing dams in reports by the American Society of Civil Engineers (1988) and the National Research Council (1983, 1985).

4. The remainder of the paper is divided into four sections. The next section summarizes the procedure followed in the Bear River study. The four study dams are briefly described in the following section, together with their hydrologic and seismic setting. In the fourth section results of the evaluation are summarized for one of the dams and the use of these results in the dam safety decision-making process is discussed. The final section contains some conclusions on the value of information obtained from incremental consequence assessment and risk assessment in dam safety decision-making.

PROCEDURE

5. The Bear River study was conducted in accordance with the procedures described by Bowles, Anderson and Glover (1987) and by the U.S. Bureau of Reclamation (1986). The overall framework for risk assessment in dam safety evaluation is summarized in Fig. 1. The figure is divided vertically into the four major steps of the risk assessment procedure. These steps are described in the next subsection. The development of a risk model, upon which the risk assessment is based, is described in the following subsection. Fig. 1 is divided horizontally into the sequence of events represented by the model.

<u>Risk Assessment Steps</u>

6. In this section the four major steps in a dam safety evaluation risk assessment are described (see row headings in Fig. 1). <u>Risk identification</u> involves recognizing and listing the various factors which could contribute to the risk of dam failure and organizing these into logical event sequences which cover all reasonably

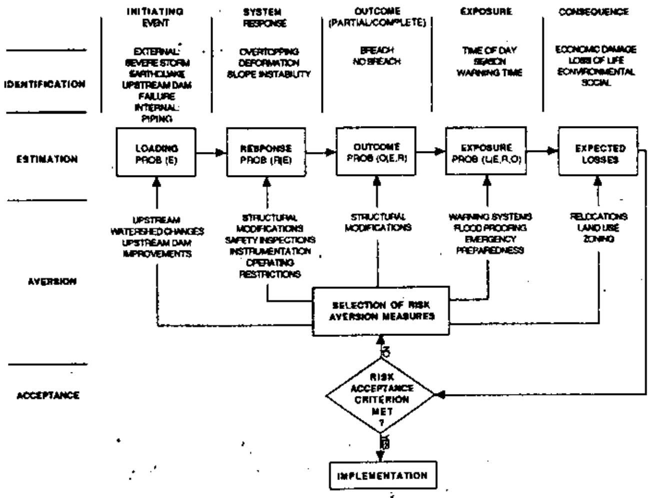

Fig. 1. Risk-Based Method for Assessing Dam Safety Improvements (Adapted from Anderson et al, 1987)

probable failure modes. Such an organization is referred to as an event tree. It serves as the risk model for evaluation of existing dam safety, or the effectiveness of proposed rehabilitation (risk aversion) alternatives. The second step is <u>risk estimation</u> which involves assigning probabilities and consequences to each failure mode, represented by a branch in the event tree model.

7. The product of the second step is an estimate of the probability of failure, and life loss or economic consequences associated with each failure mode, or combination of failure modes, for the existing dam (i.e., the do nothing alternative). If these risks are unacceptable, the assessment proceeds to the third step of <u>risk aversion</u>. This involves formulation and evaluation of remedial action (rehabilitation) alternatives. Risk aversion can be achieved by reducing the probabilities associated with an event tree branch, or by reducing the consequences. In both cases structural and nonstructural measures should be considered. Fig. 1 lists examples of aversion measures in the aversion step part of the diagram. These examples are linked by arrows to the probability or consequence that would be expected to be reduced by their implementation. The product of the aversion step is an estimate of these reductions for each aversion measure.

8. The final step in the risk assessment process is the decision on what degree of safety, or equivalently what residual risk, is acceptable. Although the engineer can supply information and recommendations for the <u>risk acceptance</u> decision, the decision is usually made by the dam owner, operator or regulator. Information available from a risk assessment can be categorized into several types: probabilistic, economic, safety, legal liability, and insurance. Bowles (1990) discusses the role of risk acceptance criteria in dam safety decision-making.

<u>Risk Model Development</u>

9. Risk model development commences with the identification of a sequence of events (see column headings in Fig. 1), beginning with events that can initiate dam failure, and ending with the consequences of failure. <u>Initiating events</u> can be classified as external or internal. External events include earthquakes, floods, and upstream dam failure. Internal events include chemical/physical changes in soil or concrete properties or latent construction defects. At low levels these events would not normally lead to dam failure. However, at high inflow rates a rapid rise in pool level could lead to overtopping, or a severe earthquake could result in structural deformation or liquefaction. These and other dam-foundation-spillway-reservoir <u>system responses</u> can lead to the <u>outcome</u> of the sudden release of the reservoir contents. The magnitudes of the resulting life loss and property or environmental damage will depend on various <u>exposure factors</u>. These can be defined by flood routing to determine the path of the flood wave, area of inundation, and travel time; the time of the day and season of the year; and the effectiveness of any warning systems and evacuation plans. <u>Consequences</u> are classified as life loss and economic loss which includes property damage, cost of dislocations, and lost project benefits. Environmental and social consequences also can be considered.

10. During the identification step, professional judgment and experience, review of available information, and site visits are used to develop a list of the types of initiating events, system responses, outcomes, exposure factors, and consequences which apply to a particular dam-foundation-spillway-reservoir system. Using this information an event tree is developed. Each branch in the event tree represents a failure mode.

11. To implement the risk model, requires the estimation of probabilities and consequences for each event tree branch. Several cases should be considered: 1) natural flow (i.e., no dam); 2a) existing dam without failure; 2b) existing dam under various failure modes; and 3) various structural and non-structural rehabilitation alternatives (including different levels of each alternative, e.g., various spillway capacities).

STUDY AREA AND DAMS

12. In downstream order the study dams are: Soda Point, Grace, Oneida and Cutler. UP&L regulates flows in the Bear River by diverting water to and from Bear Lake which is located upstream of Soda Point Dam. The hydrologic regime is dominated by the spring snowmelt event. A general storm of regional extent, combined with snowmelt, defines the PMF. However, the risk assessment also considers local summer thunderstorms since these independent events can occur at high enough magnitudes to threaten the safety of the Bear River dams. Procedures used for estimating flood frequency relationships and extending them to the PMF are described in Au-Yeung and Anderson (1989).

13. The seismic hazard in the region is strongly related to the presence of the 370-km long Wasatch Fault zone which extends from Gunnison, Utah on the south to Malad City, Idaho on the north. The Wasatch Fault zone is an active westward-dipping, normal fault. Geologic evidence indicates that it has experienced many large magnitude earthquakes during the last 10,000 years. Future activity is expected to produce earthquakes with a maximum Richter magnitude in

the range 6.5 to 7.5. A uniformly-distributed background seismicity was used to represent other faults in the region that are capable of generating small to moderate earthquakes, which would not rupture the ground surface. The historic record was used to establish the annual frequency of exceeding given levels of acceleration (Au-Yeung and Anderson 1989). Maximum accelerations ranged from 0.45g at Soda Point, Grace, and Oneida Dams to 0.66g at Cutler Dam.

UP&L Dams

14. The four UP&L study dams are briefly described in this section. For the Oneida Dam potential failure modes and remedial action alternatives are also described as background for the presentation of results in the next section.

15. Soda Point Dam is a concrete gravity dam with a short embankment section at the left abutment. It is 103-feet high and 490-feet long with a 14,000 kW generating station integrated into the concrete section. It has a spillway capacity of 63,000 cfs. and a PMF of 72,100 cfs. The dam was built in the mid-1920's.

16. The 52-foot high Grace Dam is a timber crib structure, which serves a 33,000 kW powerplant through a 5.5 mile penstock. It has a long but low embankment section extending to the right abutment. The spillway capacity is about 14,000 cfs. and the is 63,700 cfs. Grace Dam was built in 1910 and modified in 1951.

17. The Oneida Dam, built shortly after 1910, is a concrete gravity structure, 110-feet high. An earth embankment dam, separated from the main dam by a ridge, closes off a low saddle to the left of the main dam and has a crest elevation six-feet higher than the concrete dam. The total spillway capacity is about 12,000 cfs. and the PMF is 74,700 cfs. The hydropower generating station has a capacity of 29,000 kW. The potential hydrologic failure modes are overstress of the concrete dam and overtopping of the embankment. Overstress failure of the concrete dam and slope instability of the embankment were considered as the earthquake failure modes. Failure modes of the concrete dam was postulated to result in total failure of the dam and failure of the earth embankment was postulated to range from partial failure to total failure. Several structural and non-structural remedial actions were developed to the conceptual level for the Oneida Dam site. They were: 1) no action, 2) decommission the dam, 3) anchor the main concrete dam to allow overtopping and raise the embankment dam to prevent overtopping, and 4) install a flood warning system.

18. The 112-foot high Cutler Dam is a concrete gravity arch structure that was built in the late 1920's. The hydropower generating station has a capacity of 30,000 kW. The spillway capacity is 22,000 cfs. and the PMF is 195,500 cfs.

SUMMARY OF RESULTS

19. Each of the four UP&L dams was assessed individually and for its serial interactions with other dams. To illustrate the type of information obtained from the study results from Oneida Dam are used. The event tree risk model for Oneida Dam is presented in Fig. 2. A summary of results for the Oneida Dam is presented in Table 1. The

Table 1. Summary of Findings - Oneida Dam

<table>
<tr><th colspan="2"></th><th colspan="2">Hydrologic</th><th>Earthquake</th><th colspan="2">Internal</th></tr>
<tr><td colspan="2">FERC guidelines</td><td colspan="2">X</td><td>X</td><td colspan="2">√</td></tr>
<tr><td rowspan="2">Incremental consequence assessment</td><td>Life loss</td><td colspan="2">None</td><td>8 lives</td><td colspan="2">N/A</td></tr>
<tr><td>Economic damage</td><td>General:
Non $ 0·5 m
UP&L 27·5
$ 28·0</td><td>Thunder:
Non $ 0·3 m
UP&L 25·7
$ 26·0</td><td>Non $ 0·8 m
UP&L 26·2
$ 27·0</td><td colspan="2">N/A</td></tr>
<tr><td rowspan="9">Risk assessment</td><td>Prob. (net life loss)</td><td colspan="2">None</td><td>8 lives
1 in 43 500 y</td><td>Concrete
8 lives
1 in 18 000 y</td><td>Embankment
6 lives
1 in 110 000 y</td></tr>
<tr><td rowspan="3">Cost-to-save-a-life</td><td colspan="2" rowspan="3">∞</td><td colspan="3">Decommission $ 1·6 B</td></tr>
<tr><td>Anchors $ 2·7 B</td><td colspan="2">–</td></tr>
<tr><td colspan="3">Warning system $ 107 m</td></tr>
<tr><td rowspan="2">Prob. (breach failure)</td><td colspan="2">1 in 17 500/y</td><td>1 in 43 500 y</td><td colspan="2">1 in 15 500 y</td></tr>
<tr><td colspan="5">1 in 6500/y</td></tr>
<tr><td>Prob. (partial failure)</td><td colspan="2">–</td><td>–</td><td colspan="2">–</td></tr>
<tr><td>Benefit–Cost ratio</td><td colspan="3">Anchors <1%</td><td colspan="2">–</td></tr>
<tr><td>Total annual cost</td><td colspan="3">Do nothing $ 2 400 (min)
Decommission $ 249 000
Anchors $ 420 000</td><td colspan="2">–</td></tr>
</table>

table is divided vertically into sections for conventional dam safety assessment following the FERC guidelines and sections for ICA and RA. Results are summarized for hydrologic, earthquake and internal initiating event types. Reference is made to the following remedial action alternatives: do-nothing, decommissioning the dam, anchoring the concrete dam and raising the embankment, and implementing a dam failure warning system.

Incremental Consequence Assessment

20. Incremental Hazard to Human Life: Failure of the existing Oneida Dam is not expected to result in additional life loss above that projected due to the effects of a natural flood without the dam in place. This finding also applies to cases where Oneida Dam fails due to flood-caused failure of the upstream Soda Point Dam. Therefore, upgrading of the dam to safely pass the PMF (by installing anchors in the concrete dam and raising the embankment) is not projected to reduce hazard of life loss.

21. For an earthquake-caused failure of Oneida Dam, life loss is predicted to be about eight lives. Upgrading the dam to withstand the maximum credible earthquake could be achieved by adding anchors to the concrete dam which would reduce predicted life loss to zero.

22. Incremental Economic Damages: Increases in economic damages due to dam failure vary with the flow rate at which dam failure is postulated (see Fig. 3). The maximum increase for the existing Oneida Dam for a general storm flood is projected to be $28 million, with only $0.5 million of non-UP&L losses. For thunderstorm floods, the maximum increase is projected to be $26 million with only $0.3 million of non-UP&L losses. These levels of UP&L damages, while not small, are according to UP&L representatives, within insurance coverages that UP&L carries.

23. Damages for earthquake failure of the existing Oneida Dam are estimated to be up to $27 million for an overstress failure of the concrete dam with only $0.8 million of non-UP&L losses. No earthquake failure damages are predicted for the embankment dam because it is considered to meet the MCE standard.

Risk Assessment

24. Risk of Incremental Life Loss: No chance

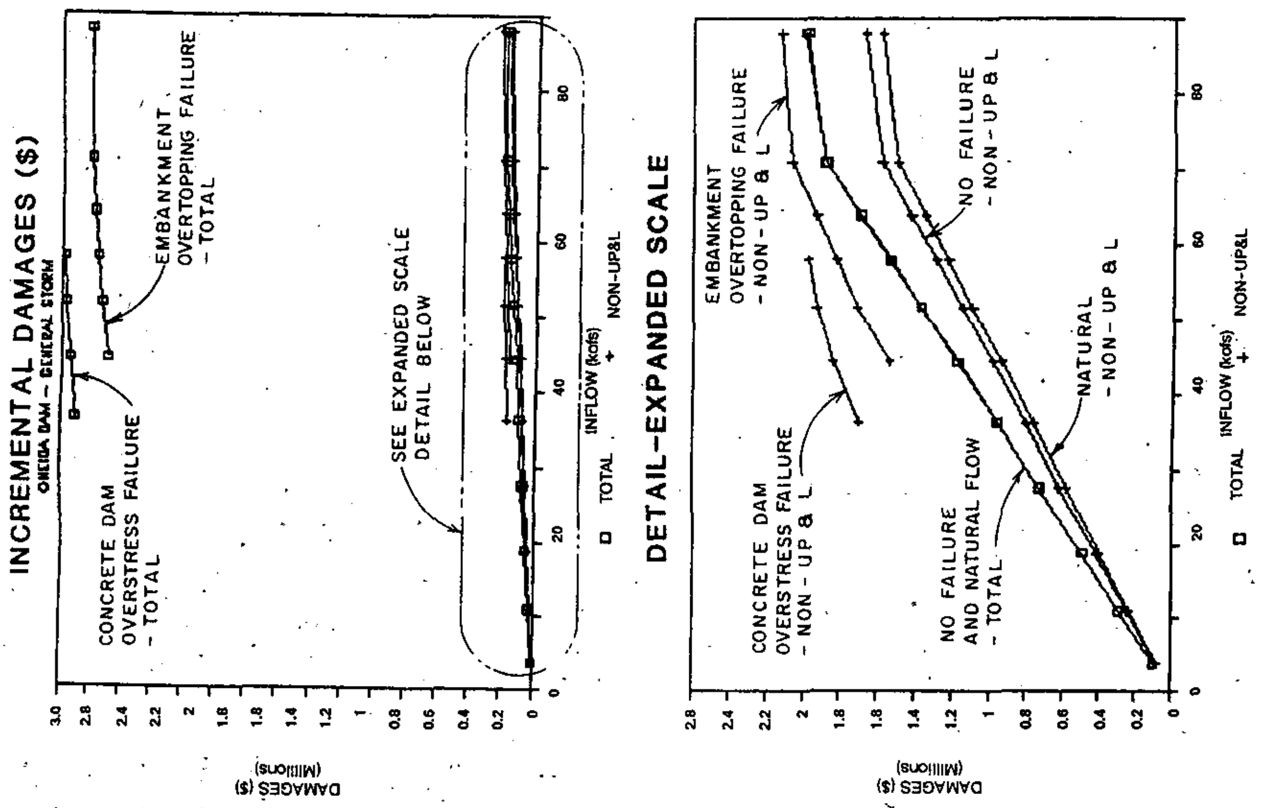

Fig. 3. Incremental Damages for Oneida Dam - General Storm (ECI/RAC 1988)

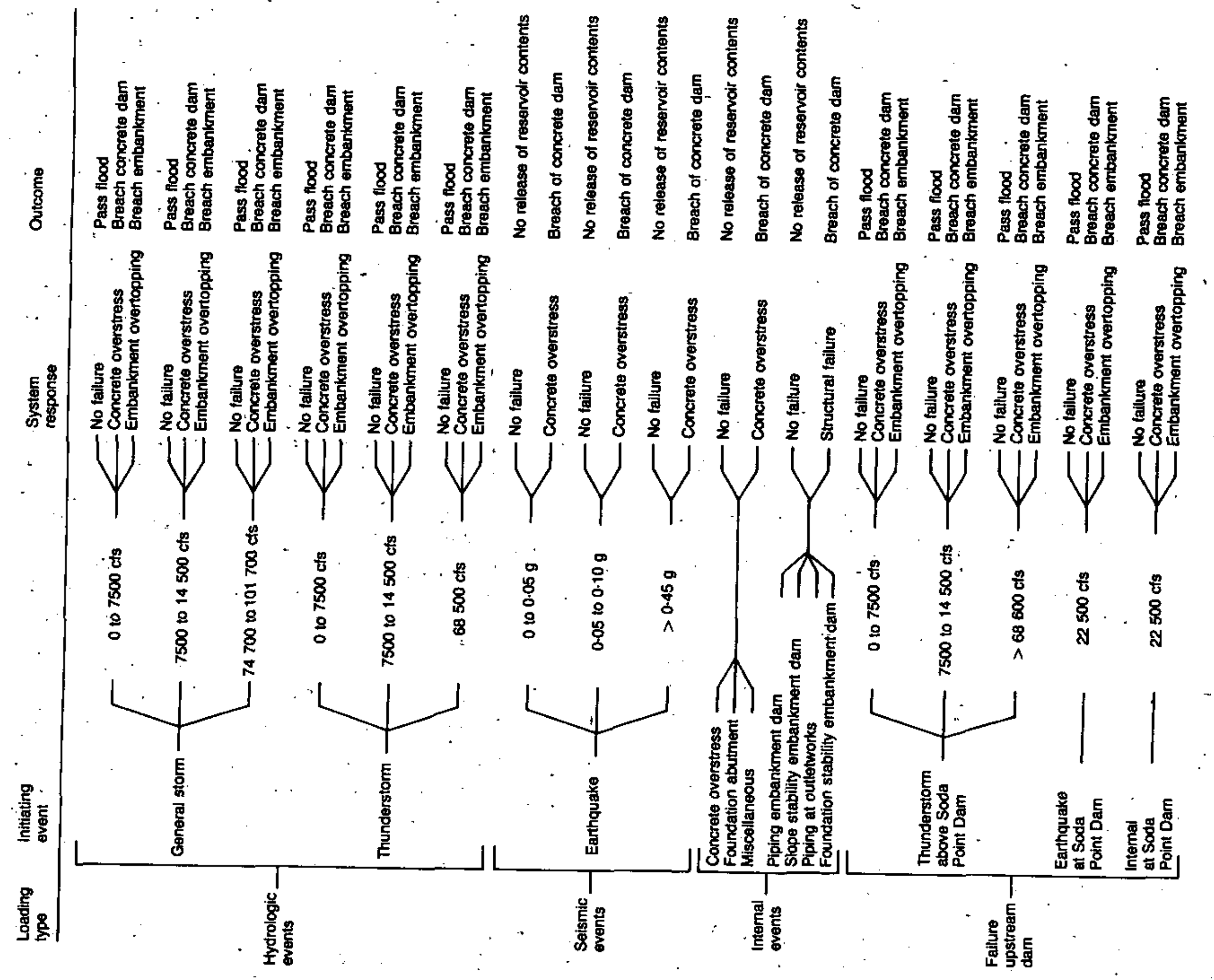

Fig. 2. Event tree for Oneida Dam (ECI/RAC 1988)

of incremental or increased life loss is projected due to flood-caused failure of Oneida Dam when compared with the case of no dam.

25. It is predicted that if an earthquake failure of the concrete dam occurred, then about 8 lives could be lost. The chance of failure of the Oneida concrete dam is estimated to be 1 in 43,500 per year for earthquake failure, and 1 in 18,000 per year for an internal failure. If internal failure of the embankment occurred, then it is estimated that about 6 lives could be lost. The chance of such a failure occurring is estimated to be about 1 in 110,000 per year. These chances of life loss resulting from the failure of Oneida Dam are much lower than the historical probability of life loss from dam failures in the United States due to all causes (i.e. flood, earthquake and internal failures), which is 1 in 5,000 per year. However, the existing concrete and embankment sections of the Oneida Dam have been found to satisfy FERC criteria with respect to their internal condition. Also, the embankment section is considered to meet the MCE standard.

26. Cost-to-Save-a-Life: The cost of increasing human safety can be expressed on a "per statistical life saved" basis (i.e. cost-to-save-a-life). This is the cost of providing safety and is not in any sense a value for human life. Since no life loss could be attributed to the Oneida Dam under flood loading, it follows that upgrading of the dam would not be predicted to save any lives. Therefore, the cost-to-save-a-life for remedial upgrading of the flood performance of the dam is infinitely large.

27. The cost-to-save-a-life for installing anchors in the concrete section of the Oneida Dam to withstand the maximum credible earthquake is calculated to be approximately $2.7 billion per life saved. A dam break/flood warning system was considered for reducing the hazard to human life in the event of an earthquake or internal failure of Oneida Dam. It was calculated that the cost-to-save-a-life for this system is approximately $107 million per life saved. However, this system is not expected to reduce life loss at the Oneida Hydro Facility itself, since it is located immediately below the dam. If the Oneida Dam is decommissioned, the cost-to-save-a-life is calculated to be approximately $1.6 billion per life saved.

28. These costs can be compared with costs-to-save-a-life calculated for regulated areas such as nuclear power plant design ($4 - $10 million), environmental protection ($4 million) and occupational health and safety ($4.5 million and $300 million for OSHA Benzene regulations).

29. Probability of Dam Failure: The chance of a breach failure of Oneida Dam, from floods, earthquakes, internal causes and upstream dam failure (see Fig. 4), is estimated to be 1 in 6,500 (1.6×10^{-4}) per year. This is lower than the historical probability of dam failure in the United States due to all causes. The chances of failure of either the concrete or embankment dam are estimated to be approximately equal (see Fig. 5).

30. Information on annual failure probability is combined with the incremental consequence assessment results (incremental economic damages) in histograms in which incremental damages are shown separately for general and thunderstorm initiating events, and for damages occurring to the owner (UP&L) and to other parties (non-UP&L). Figs. 6 and 7 are examples of histograms of net damages to UP&L and to other parties for the general storm. Failure probabilities are divided between components attributed to the concrete dam and the embankment dam hydrologic failure modes.

31. Benefit-Cost Ratio: Economic benefits are predicted to be less than one percent of the estimated cost for installing anchors in the concrete dam and raising the embankment. No structural alternatives were considered for internal failure modes since the Oneida Dam has been found to meet FERC standards for these cases.

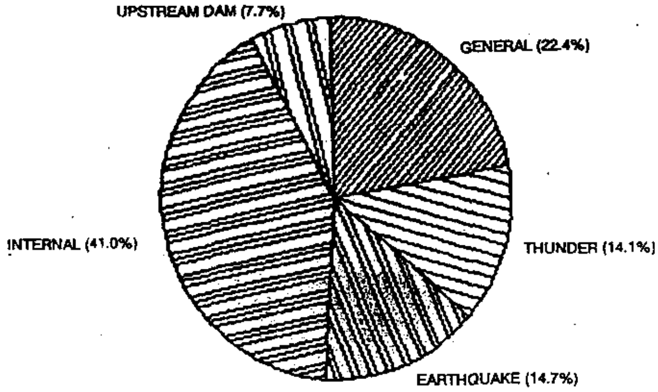

Fig. 4. Probability of Failure by Initiating Event for Oneida Dam (ECI/RAC 1988)

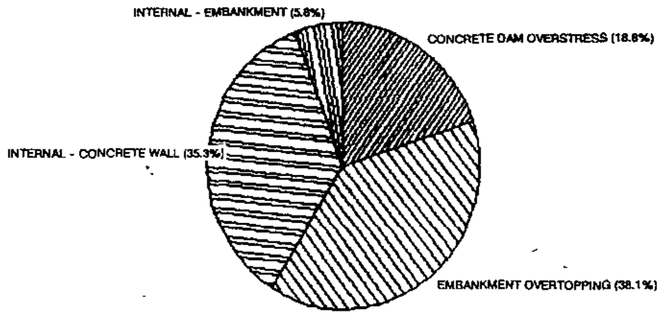

Fig. 5. Probability of Failure by System Response for Oneida Dam (ECI/RAC 1988)

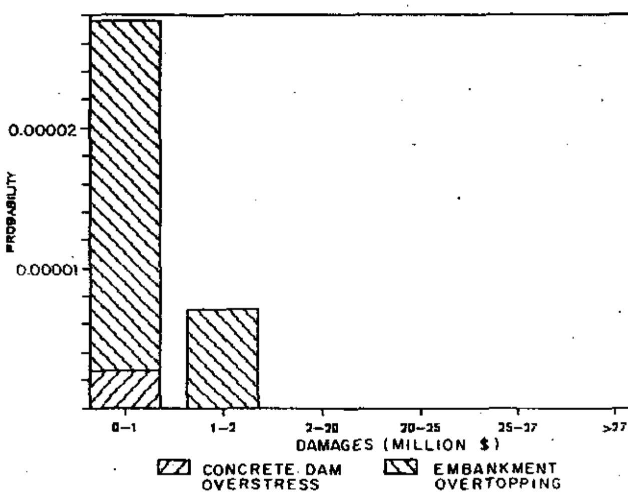

Fig. 6. Histogram of Net Non-UP&L Damages for Oneida Dam - General Storm (ECI/RAC 1988)

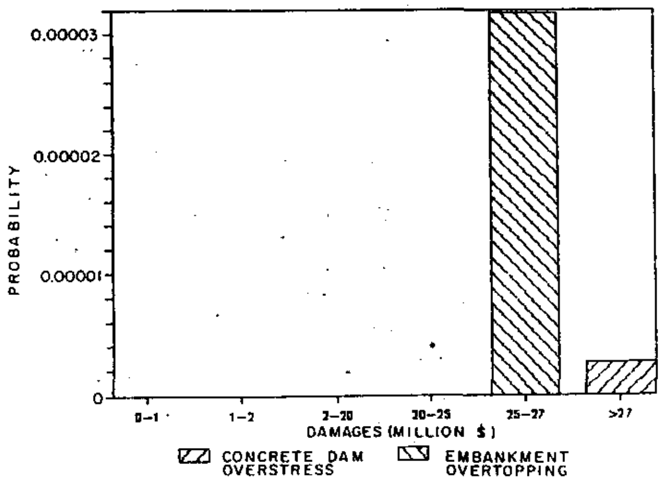

Fig. 7. Histogram of UP&L Damages for Oneida Dam - General Storm (ECI/RAC 1988)

32. Total Annual Cost: The sum of the predicted annualized damages (net risk costs) and estimated annualized costs is $420,000 for installing anchors in the concrete dam and raising the embankment, $2,400 for the do-nothing alternative (i.e. maintain the existing dam) and $249,000 for decommissioning the facility. Thus, the existing dam alternative was found to have the lowest total annual cost of these three alternatives.

33. Environmental Impacts: A reconnaissance-level environmental evaluation of dam failure impacts was performed. For flood-caused failure scenarios, the additional area of environmental impact is predicted to be small when compared to the natural flooding case. The probability of dam breach impacts occurring was found to be approximately 1 in 6,500 per year.

Serial Dam Failure Modes

34. The potential for serial dam failure was assessed for all reasonably probable failure modes at upstream dams. This involved many flood routings and the evaluation of downstream consequences. Under the general storm the probability of dam failure was slightly increased by the presence of upstream dams for all dams except Grace Dam which is protected at intermediate flood magnitudes by the upstream Soda Point Reservoir. For thunderstorm loading it is only in the case of an overstress failure of Soda Point Dam that the probability of downstream dam failure was predicted to increase slightly. Seismic or internal initiating events are not expected to lead to serial dam failure except at Grace Dam as the result of this type of failure at Soda Point Dam.

Study Outcome

35. The Bear River Risk Assessment was presented to FERC by UP&L, to justify their proposals for upgrading the dams. These proposals were accepted by the FERC. The views expressed by the owners representative (Waite 1989b) are testimony to the value of the risk assessment of the four Bear River dams and another on Hams Fork in Wyoming:

36. "Was the Study Worth It?: Including the internal utility costs the risk assessment study for the five dams cost close to $500,000. What did we get for our money besides FERC's discomfort? First and foremost, we developed an in-depth understanding of these dams' potential for failure, and we internally justified the necessary remedial activities. Without this thorough review, we would probably have had bad feelings about any of the work for a considerable time to come, and we may have otherwise sought very costly legal remedies. Second, the study developed alternatives that would probably have been missed or bypassed without this penetrating scrutiny. Based on our initial estimates and contingency plans, we feel the study came very close to saving us $10 million in current remedial costs, about 40% to 50% of the money we had anticipated spending. Third, we felt that FERC was better able to appreciate the benefit of avoiding some of the work we would have otherwise done, and we were better able to appreciate some of their concerns ... Fourth, some of the work, such as the incremental flood studies would have been needed in any case, and they were a material portion of the study cost, perhaps 20% of the total ... On the whole, it was very well worth the effort."

CONCLUSIONS

37. The Bear River study showed that estimated dam failure probabilities were low. Predicted incremental damages were low, and in most cases damages would affect the owner to a far greater extent then other parties. The probability of life-loss was estimated to be low, and the cost-to-save-a-life was calculated to be high for all structural and nonstructural alternatives. No economic justification for alternative fixes could be shown. Evaluation of potential serial failure modes did not show large increases in failure probabilities from this type of initiating event. In the case of Grace Dam, a decrease in failure probability can be attributed to the protection provided by the upstream Oneida Dam. The approach to risk assessment used in the Bear River study did not involve placing a value on human life, nor did it involve using a specific decision criterion, such as minimum total annual cost. The selection of remedial actions was made by the dam owner and regulator using study results and giving weight to any case where incremental life loss was identified (however, unlikely) and where social impacts of dam failure would be severe.

38. The results summarized in this paper illustrate some types of information which can be provided by incremental consequence and risk assessment dam safety evaluation studies. Other uses of information obtainable from these approaches include: the assessment of liability exposure for dam owners and operators, the choice of interim measures for improving safety at dams which are awaiting permanent rehabilitation, the efficient allocation of effort for dam safety studies, the sequencing of remedial actions at a group of dams which cannot be budgeted or scheduled to be performed simultaneously, and the provision of a basis for insurance coverage of dams. Bowles (1990) describes several risk assessment studies performed for U.S. dam owners which illustrate some of these applications and their value in dam safety decision-making. In each

case the information obtained has proven useful in dam safety decision-making process.

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18. Safety considerations with existing embankment dams and in their raising

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SYNOPSIS. Experiences with a variety of embankment dams of greatly differing ages in various parts of the world are described in relation to safety, both in their existing state and in their raising. The dams involved contain: a concrete core wall; homogeneous sections; an upstream face core; a central core with rockfill shells; a deteriorating draw-off culvert and ancient sluices. Safety lessons are summarised in respect of the inspection and investigation of, and the design and construction of works on, operational dams.

INTRODUCTION

1. Dams, by their nature, are generally amongst the longest functioning of the structures built by man, and often still present much of their general original appearance to the present observer. They are seldom discontinued or removed, because the water they retain is increasingly valued as both a basic and a precious resource.

2. With the increased emphasis quite properly being given to their continued use and safety, dams-particularly embankments - are being regularly inspected, and where necessary investigated, repaired or improved. Because of their increasing age, these actions involve safety considerations at each step of that process. Some of those considerations are described for two cases of existing dams, the first relating to the 66-year-old Coedty dam in North Wales and the second to ancient embankment dams in Sri Lanka.

3. The need to maximize water resources can sometimes require the raising of existing dams, which involves additional safety problems during the design and construction stages. Two examples follow, in which the raising of the Gaborone dam in Botswana and Hinze dam in Australia are described.

SAFETY OF EMBANKMENT DAMS IN THEIR EXISTING STATE

Coedty dam - North Wales

4. History. Coedty dam is situated in the Conway Valley, and was built in 1924 to augment the hydro-electric capacity of the Cowlyd and Eigiau dams conveying water by a series of tunnels, pipelines and leets (surface channels) to the Dolgarrog power station.

5. The dam (Fig.1) is a 250 m long, 11 m high earth embankment with upstream and downstream slopes of 1v:2h and 1v:2.5h respectively. The upstream slope is protected against wave action by stone pitching, and the downstream slope is grassed. The embankment incorporates a thin reinforced concrete cutoff wall placed centrally on the dam axis, and extending to 3.7 m below natural ground level.

6. Bedrock at the dam site is overlain by some 6-10 m of glacial till drift deposits. The drift typically comprises poorly sorted deposits of generally granular material, but including variable amounts of silts and clays. The embankment is founded on, and constructed of, the till.

7. The draw-off works comprise a 1.8 m diameter reinforced concrete culvert feeding the penstock which leads to Dolgarrog power station 2 km away and 250 m below. A line valve and automatic self-closing butterfly valve are installed on the draw-off pipe immediately downstream of the embankment.

8. In 1925, the completed embankment was overtopped following failure of the Eigiau dam upstream, forming a breach 60 m wide at the top and 18 m at the bottom. Sixteen lives were lost in Dolgarrog village. This event, recorded by a bronze plaque adjacent to the road through the village, led directly to enactment of the 1930 reservoir safety legislation. The dam was rebuilt, since when a number of other repair works have been carried out in attempts to reduce the quantity of seepage passing through the dam, and including the installation of a riveted steel liner to the draw-off pipe.

9. An attempt to reduce seepage by grouting through the core was made in 1972; grout takes were high and grout appeared on the surface. In 1986 further wet areas appeared on the downstream face, and it was then decided that the reservoir should be maintained some 1.5 m below full storage level. A full safety assessment of the embankment was initiated.

10. A ground investigation was undertaken in 1988 to establish the soil properties and seepage conditions within the embankment, as well as to obtain data for stability analyses and the design of any necessary remedial works. Standpipe piezometers were installed in a number of boreholes and a monitoring programme established.

11. The problem identified. The safety assessment yielded the following conclusions:

(1) The concrete core wall was in a poor condition, and was expected to continue to deteriorate due to soft water attack.
(2) In the area where the embankment was reconstructed following the breaching in 1925 the water level in the downstream shoulder was low, being apparently drained by the permeable strata in the foundations. Elsewhere the phreatic surface was high.
(3) The stability of the downstream shoulder with the reduced reservoir level was satisfactory. However, if the reservoir level was restored to its original level further deterioration of the core wall would result in increased seepage. If the phreatic surface rose to a level consistent with no core wall, the stability would give cause for concern.
(4) Although the upstream slope had shown no signs of distress as a result of operating under conditions of continuous and rapidly fluctuating water levels for many years stability analyses for the rapid draw-down condition showed a factor of safety of near unity.
(5) The draw-off conduit was in poor condition. The steel lining was corroded and a short unlined concrete section at the downstream end showed signs of leakage. Also, the draw-off conduit was not provided with an upstream guard valve.

12. A number of V-notch weirs were constructed to monitor seepage flows. However, because of the permeable strata underlying the embankment it was considered that flow measurements would be unlikely to indicate actual seepage losses realistically. Nevertheless, measurements made before lowering the reservoir level had shown a substantial increase in flow as the reservoir approached the full storage level. The elevation at which such increases in flow occurred was consistent with the areas of defective concrete encountered during the drilling. The highest measured seepage losses amounted to some 3 l/s, as compared with a calculated value based on flow rates of some 20 l/s at maximum water level.

13. The solution adopted. In view of the dam's history of problems, and its location in a narrow valley perched high above Dolgarrog village, it was considered prudent to design remedial works on pessimistic assumptions about future behaviour, viz:-

* The core wall being totally ineffective in reducing seepage
* Severe rapid draw-down conditions.

Because of failure to reduce seepage through repair of the concrete cutoff wall it was decided that provision would be made only to control rather than reduce seepage.

14. Remedial works (Fig.2) to the embankment therefore comprised:-

(1) Widening the crest and placing stabilizing fill incorporating a drainage layer on the downstream slope, divided into five separate seepage collection panels in plan (Fig.1).
(2) Placing stabilizing fill to about half embankment height on the upstream slope.

15. Similarly pessimistic assumptions were made on possible future deterioration of the draw-off culvert, and the following works were undertaken:-

(3) Lining the existing steel-lined culvert with a 1.5 m dia. glass-reinforced plastic liner.
(4) Demolishing the existing 1.8 m dia. unlined concrete culvert section immediately downstream of the embankment and reconstructing, with fabricated steel liner pipes, and including an access branch for inspection.

16. The absence of a shut-off guard valve on the upstream end of the culvert represented a serious shortcoming, both from an operational point of view and also for the

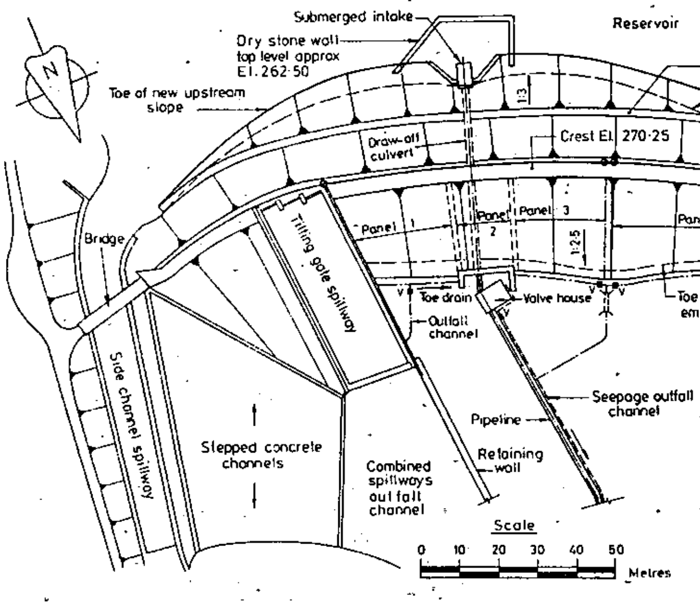

Fig. 1. Plan of Coedty dam

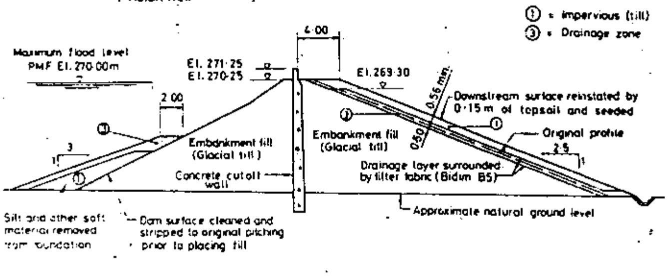

Fig. 2. Modifications to Coedty embankment

safety of the embankment should leaks develop in the buried section of the culvert. The normal safety standards called for any pipe into which people can enter for inspection purposes to be protected by two guard valves. In this instance, however, it was agreed that for various reasons a single valve would be acceptable. Accordingly, the remedial works (Fig.3) included the demolition of the existing intake screen supporting structure and its reconstruction incorporating a butterfly valve and air vent pipe. An extension of the existing steel access bridge to accommodate the valve-operating headstock was also necessary. A number of other works unrelated to safety were also undertaken.

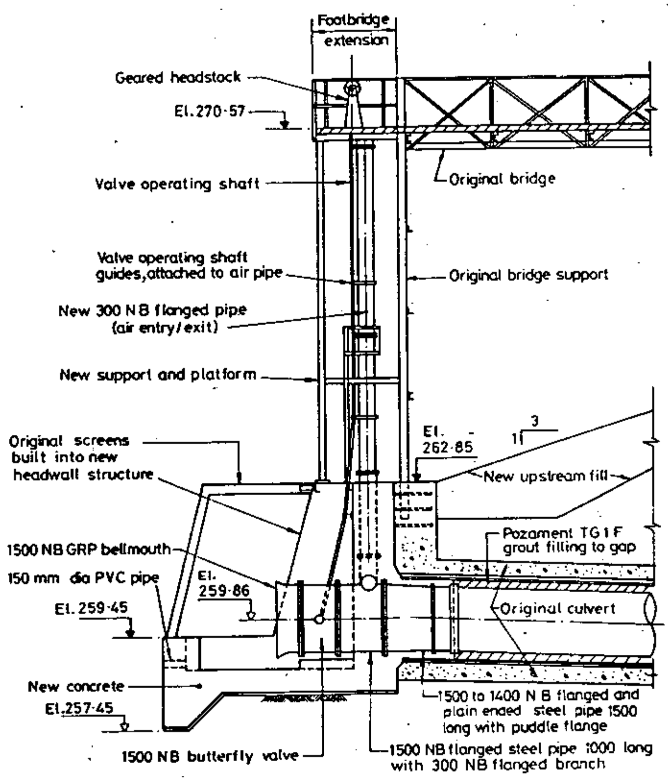

Fig. 3. Sectional elevation showing modifications to Coedty draw-off intake

17. Construction. Construction was fairly straightforward. The main problem was how to deal with the river inflows while working on the draw-off culvert, as this was the sole outlet from the reservoir. The only solution was to pump. It was decided that the pumps should be able to deal with low flows only, and the possibility of the works being flooded occasionally was accepted by the client as a risk to be paid for if it occurred. In fact, because of the long dry summer it was only towards the end of the work that any significant flooding occurred, but no damage was caused.

18. The work was carried out between June and December 1989. In January 1990 the spillway gates were raised and the original maximum reservoir operating level was restored. Seepage flow measurements and piezometric levels indicate that the embankment is functioning satisfactorily.

Ancient embankment dams in Sri Lanka

19. Safety assessment of embankment dams in Sri Lanka presents a number of unique problems. There are some 300 or so major reservoirs (those supplying 80 or more hectares of irrigation) and tens of thousands of smaller ones. These are all impounded by low earth dams or bunds designed to serve the commercial needs of the country's densely populated rural areas. Many of the reservoirs (referred to as 'tanks' in Sri Lanka) have been in existence since the last century and records show that some of them have been in almost continuous use since first constructed by Sri Lanka's ancient kings (Fig.4), some over 1600 years ago. The tanks form an integral part of the rural environment and the communities depend on them for much more than just their irrigation needs. They are part of the social fabric of the population so that their safety is taken as a matter of course. The failure of the Kantalai Tank bund in April 1986 (ref. 1) with the loss of 127 lives therefore came as a devastating shock to the Sri Lankan people.

Fig. 4. Typical ancient Sri Lankan embankment dam

20. The need for a universal system for hazard rating for Sri Lankan reservoirs, one that can be accepted and applied by all agencies responsible for reservoir operation in the country, has now been recognised. The case in Sri Lanka for adopting a hazard rating system is the same as for other countries with a large stock of dams, namely:-

* to provide the basis for a unified approach to assessing flood criteria for dams
* To formalise the need for more intensive inspection procedures and maintenance resources targeted to high hazard structures.

21. In order to promote the concept of hazard rating within the country's engineering community (ref.2), proposals were formulated for a hazard rating table based on that adopted by the Australian National Committee on Large Dams' dam safety sub-committee.

Table 1. Hazard rating for reservoirs in Sri Lanka

Very high	High	Significant	Low	Very low
Excessive numbers of lives at risk	Loss of identifiable life expected because of community or other significant developments downstream	No loss of life expected, but the possibility recognized. No urban development and no more than a small number of habitable structures downstream	No loss of life expected	Reservoirs in remote jungle or sea coast locations presenting no measureable risk to persons or property
	Excessive economic loss such as serious damage to communities, industrial, commercial or agricultural facilities, important utilities, the dam itself or other storages downstream	Appreciable economic loss such as damage to secondary roads, minor railways, relatively important public utilities, the dam itself or other storages downstream	Minimal economic loss, such as farm buildings; limited damage to agricultural land, minor roads, etc	
	Dam essential for services, and repairs not practicable	Repairs to dam practicable or alternative sources of water/power supply available	Repairs to dam practicable. Indirect losses not significant	

Their table was used as being the most appropriate for Sri Lankan conditions, but with an extension to the number of categories to reflect the wider range. The benefit of adopting a system which is compatible with practice in other countries, so that common experience can be shared, is clear. The proposed hazard rating table has now been applied in a preliminary exercise to rate all of the major irrigation tanks, and is reproduced here as Table 1.

22. In conjunction with hazard rating, the standards for flood design generally adopted in Sri Lanka have been reviewed. Table 2 sets out flood standards which are believed generally to reflect current practice for design of new dams in Sri Lanka, and may therefore be used to check the adequacy of existing dams.

23. In order to improve the evaluation of the safety of major irrigation dams, model forms for dam safety inspection were prepared and applied to twelve representative inspections. The dams selected for such inspection were all major structures, but displaying particular symptoms which had been causing concern. The model form for dam safety reporting provided for making recommendations for measures to be taken in the interests of safety.

24. The following summarises the main or recurring features of those measures in respect of the twelve dams inspected.

25. Reservoirs. The water level was to be kept down in some cases until remedial works had been completed.

26. Bunds. Further slope erosion was to be arrested by remedial works; a berm at one dam was to be completed to ensure stability; crest low points were to be raised to restore freeboard; drainage measures were to be implemented.

27. Sluices. Ancient dry stone barrels were to be demolished, as posing a severe risk to safety (being similar to the Kantalai type which collapsed after piping, ref.3); gates

Table 2. Minimum flood safety standards as generally represented by current design practice in Sri Lanka. (Return periods of floods in years)

Hazard rating	Reservoir size factor - V x H		
	Greater than 100	100 to 10	Less than 10
Very high	PMF	PMF	PMF
High	10 000	5000	2000
Significant	1000	500	200
Low	100	50	20
Very low	no minimum	no minimum	no minimum

V = Volume of water in million m^3 retained by dam with reservoir level at highest flood level

H = Height of dam measured in metres as vertical distance between crest level and level of lowest adjacent natural ground

PMF = Probable Maximum Flood as derived by rainfall maximisation procedures

were to be checked for higher heads at one dam and replaced at another by larger ones to allow faster emergency drawdown; barrels were to be strengthened where the need was shown by leakage testing; stilling basins were to be repaired and a new sluice was to be constructed to allow more rapid draw down in an emergency.

28. Spillways. Aparc from usual matters relating to capacity and gate mechanisms,

radio equipment was being considered for one remote dam for warning of emergency use of the spillway.

29. Downstream flood corridors. Flood warning systems were being considered as well as arrangements with local authorities to secure life and property in the event of major release from two spillways located in close proximity to built-up areas.

30. The above points are thought to typify the dam safety problems likely to be encountered at most of Sri Lanka's ancient dams.

SAFETY ASPECTS IN RAISING OF EMBANKMENT DAMS

Gaborone dam raising, Botswana

31. The design for the raised dam, constructed 1983-84, is based on an 8 m rise in the full supply level for the reservoir, and includes flood discharge arrangements capable of passing a probable maximum flood of 7000 m^3/s. The original zoned embankment has an upstream sloping silty clay core and a downstream random zone separated by a chimney drain. Investigations were limited by the need to respect the integrity of the core and avoid deep penetration into it, while construction records from the early 1960s were unavailable. The upstream face of the core was subject to a degree of softening and the downstream random zone was largely of impermeable material. The permeability of the chimney drain was in doubt until exposed during the new construction work, when it was shown to be a stiff gravelly clay rather than a modern permeable chimney.

32. The sandy alluvium of the original foundation was cut off by a contiguous piled wall linked to a grout curtain in the underlying weathered granite, and historically significant seepage water existed at the downstream toe. The only suitable dam alignment, through a granite outcrop offering a spillway site, necessitated incorporating the existing dam into a 25 m final height embankment (Fig. 5).

33. It was not safe to drill through the existing core and improve the cutoff provisions, so the design relies on the lengthened seepage path maintaining similar hydraulic gradients in the foundation alluvium. Internally the raised dam has a new sand chimney zone and a high capacity horizontal drainage blanket. The safety of the foundation against piping failure was further enhanced by the addition of a trapezoidal collector drain of graded filter material, running along the downstream toe. The whole of the existing dam and the raised core were thus treated as an integral upstream core zone, supported by a massive new downstream random fill shell zone.

34. The crucial stage of construction for upstream slope stability occurred when the reservoir was at an unusually low level and comprised removal of the upper 4 m of the existing dam core, while the advanced height of new core fill downstream maintained the required freeboard of the existing reservoir.

35. The fill had to be placed as rapidly as possible to reach crest level, consistent with stability of the upstream slope. Pore pressures both within the existing core and above the core junction level were therefore monitored with hydraulic piezometers and control surveys of the upstream slope were carried out. In the event pore pressure rises were not significantly high (ref. 4).

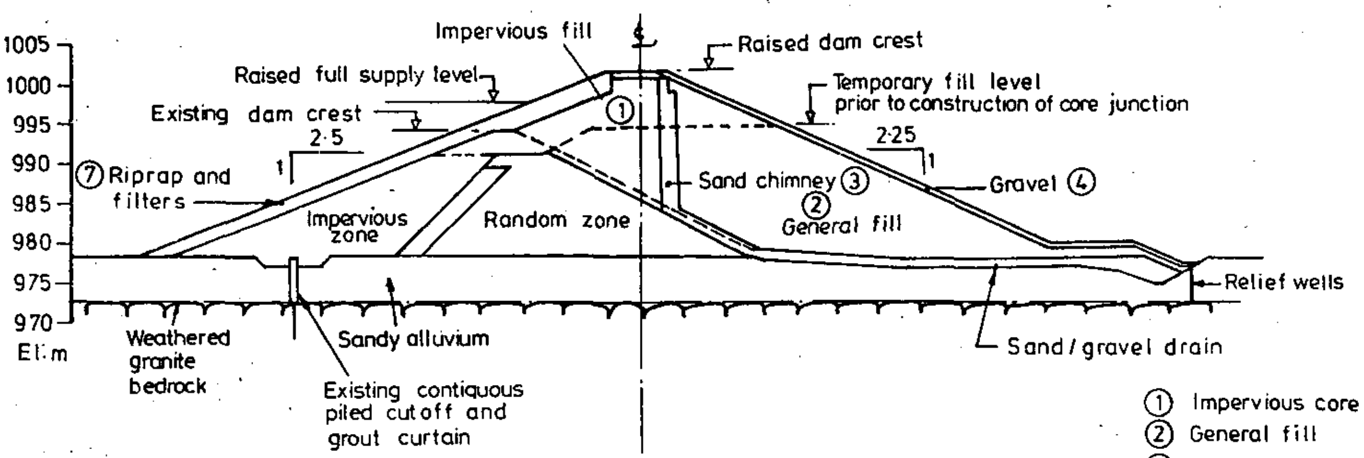

Fig. 5. Raised section of Gaborone dam

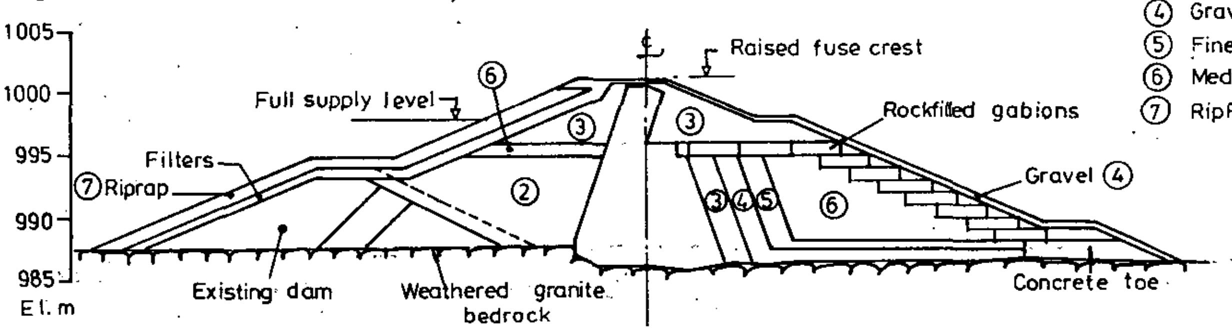

Fig. 6. Fuse section of Gaborone dam

36. In line with advances on flood risk analysis a spillway design based on a 1000 year flood was upgraded to PMF. The dome of sound granite offering a foundation for the ungated spillway weir was of strictly limited length. To avoid the excessive cost of increasing crest height along the full embankment length it was decided to follow a common precedent in Africa and provide a fuse section within the embankment. If the flood exceeds 0.5 PMF the fuse is overtopped and its design (Fig. 6) ensures rapid erosion, down to a protected base level to retain a residual reservoir. The lack of data from actual operation of such fuses, and the conjectural form of the PMF flood hydrograph, led to the adoption of a wavewall to increase the available emergency freeboard. Standby plant was also provided to initiate erosion if necessary. The risk of occurrence of a flood that would activate the fuse provision is less than 1:1000 years, and the fuse merely concentrates the breach at a selected discharge channel rather than damaging the entire dam crest and downstream toe.

37. The existing concrete draw-off tower and culvert were incorporated into the new scheme. The heightened tower was tied to anchorages in the bedrock by prestressing bars to avoid flotation problems. The existing length of culvert, set in a trench through the bedrock below the embankment, was provided with a reinforced concrete liner to take the extra vertical loading. A new extension length and downstream valve house were then added, and grouting and drainage provisions incorporated to reduce seepage along the outside of the culvert. The draw-off water passes through a steel pipe within the concrete culvert and township supplies were not interrupted for more than 24 hours during construction.

Hinze dam raising, Australia

38. Hinze dam, located 70 km south-south east of Brisbane, Queensland, was built 1974-76 as a 48 m high rockfill embankment with a central clay core. It was designed with the intention of future raising by 22 m as the next stage in augmenting water supplies to the Gold Coast and adjoining areas of Albert Shire. Since then, however, the forecast water demand almost doubled for the period 1985-2000, with the existing yield expecting to be reached by 1990. These considerations, together with optimisation studies, formed the

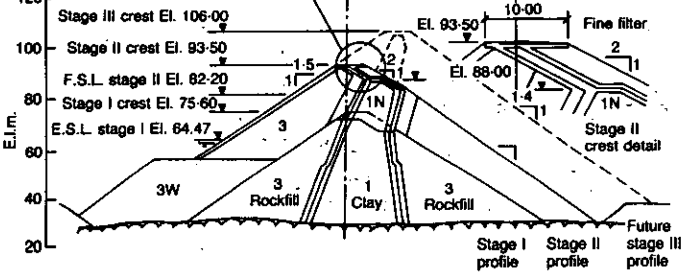

Fig. 7. Hinze dam two-stage raising

basis for a two-stage raising of the dam by a total of 30 m, Fig.7. The first stage raising was designed and constructed, with completion in 1989, to facilitate subsequent raising to the ultimate height.

39. Two other factors significantly influenced both design and construction of the raising, namely: (a) changes in procedures for estimating probable maximum floods in Australia since the original design, resulting in considerably increased peak outflows; these could not be accommodated by the earlier envisaged raised dam and spillway arrangement; (b) a decision at the beginning of construction to improve the flood mitigation aspects of the dam by incorporating a rectangular slotted spillway in the raising project: a lower 24.5 m wide part reduced the peak outflow for floods up to the design 100 year flood, whilst greater floods passed also over the adjacent shoulders of the spillway. This required a combined surcharge and freeboard of 11.1 m, resulting in a revised Stage II dam crest level at El.93.5 m, in order to discharge the PMF of 2420 m^3/s. This represented an additional 3.5 m to the original raised crest level to which construction had already begun.

40. The Stage II crest arrangement (Fig.7, inset) features a flat-inclined filter barrier to water above El.88.0 m, which represents a 25 year return period flood, whilst El.89.2 m represents the 100 year design flood and El.92.5 m the PMF. None of the floods would be of long duration and the PMF would result in water levels above El.88.0 m for a period no greater than 3 or 4 days. During such a period the top 5.5 m of the Stage II dam would act as a permeable bund. The quantity of seepage would be controlled by the permeability of the flat-sloping one metre thick fine filter zone acting as a core, protected by the outer coarse filter and discharging into a special general drainage zone.

41. The design concept is simply an extension of the existing (Stage I) clay core and rockfill shells, with the core slightly inclined downstream to minimise the amount of rockfill (zone 3W) to be placed in the reservoir. Such rockfill was restricted to fresh or slightly weathered only, to minimise pollution of the water. To avoid damage to the exposed right hand side of the drawoff culvert, 3W material over the culvert was required to be placed from the left towards the right abutment. Fine, clean rockfill of maximum size 200 mm was specified to be placed around the intake tower to minimise damage to the concrete. The upstream rockfill berm 3W would ensure that there will be no rockfill placed through water for the ultimate Stage III embankment.

42. To safeguard freeboard and to minimise distortion of the Stage I core, new rockfill was to be constructed to at least Stage I crest level before new core construction began, and maintained at as similar a level as possible upstream and downstream.

43. A minimum thickness of 2 m of core material was required to be stripped from the

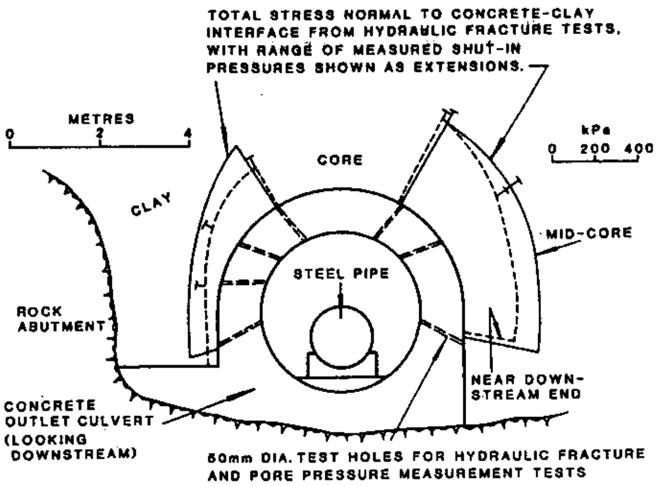

Fig. 8. Hinze dam core/draw-off culvert area stress conditions

Stage I embankment to eliminate a slightly dry and stiff zone from the core at its geometric change point, and thus reduce the risk of arching and cracking. At 2 m depth the 10 year old existing core had a moisture content of about 20%, which was within the range of the original placement moisture contents. The new core was benched into the existing core on a 1v to 1.5h slope, with a 3 m working bench downstream. To avoid further drying out and to minimise risks due to temporarily reduced core freeboard, cutting down of the Stage I crest and replacement with Stage II fill was to proceed in sections of a minimum length necessary for construction purposes only.

44. Low stress conditions within a portion of the Stage I core, between a rock abutment and the side of the nearby concrete draw-off culvert (Fig. 8), and a state of hydraulic fracture were suspected and proved (ref.4). Although the hydraulic fracture was safely contained by the downstream filter, it was decided to enhance existing stresses in the core zone. This was achieved by a programme of squeeze grouting from within the culvert. The first three series of injections over the upstream two thirds of the core width took one month, and succeeded in raising the minor total principal stress by about 150 kPa, with the piezometric profile rising correspondingly. A fourth series of injections was made after a seven month wait, when testing showed that the stress levels achieved by the previous work had been maintained, and a modest additional enhancement was then achieved. Overall stress levels on completion of the dam raising were found to be acceptable.

45. In summary therefore, the safety issues involved with this raising were related to: (i) a reappraisal of procedures for estimating the PMF, (ii) incorporating flood mitigation requirements within the project after start of construction, involving a permeable barrier, (iii) construction sequences and rockfill sizing, and (iv) stress enhancement of the core adjacent to the draw-off culvert. First spilling for the raised dam occurred in April 1989 after rapid filling.

CONCLUDING REMARKS ON SAFETY LESSONS DERIVED FROM CASE HISTORIES DESCRIBED

Regarding the inspection and investigation of operational dams

46. Investigations can at best only partially discover the internal condition of an old dam. The older the dam the more likely it is that full design and construction details will not be available. In such cases external inspection will provide the only direct knowledge of a dam's condition, unless internal investigations are made. The planning and doing of investigations in operational dams regardless of age, however, must proceed with a high level of caution. At all times they must be capable of being quickly stopped, and procedures modified where they are suspected of causing a local but significant deterioration in an embankment's integrity. This particularly applies to all work within a dam core, and at interfaces with adjacent structures such as draw-off culverts.

Regarding the design and construction of works on operational dams

47. Safety must be the primary concern in the design and construction of works on operational dams and will include: the maintenance of freeboard and the ability to pass floods; control of temporary and long-term seepage by adequate geometry and drainage; careful construction sequences to avoid damage to existing work; realistic assumptions about the possible long-term deterioration of significant elements such as draw-off culverts and their surrounds, and concrete core walls in old dams; continued monitoring and assessment of a dam's behaviour after completion of remedial works or raising.

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20. Woodhead Reservoir - remedial works

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Remedial works to Woodhead Reservoir were required to alter the overflow provisions to handle a probable maximum flood (PMF) inflow while restricting the outflow to lower reservoirs in the Longdendale Valley cascade. Woodhead Dam has had a noteworthy history with several modifications to the original design since construction first started in 1847. Several restrictions were placed on the remedial works as a result of the dam being situated in the Peak National Park. The risk to the Works, as a result of various factors, are considered for both the temporary remedial works phase and the permanent works.

INTRODUCTION

1. Following an inspection in 1986 under the Reservoirs Act 1975 of North West Water's five reservoirs in the Longdendale Valley, 25 km east of Manchester, the AR Panel Engineer recommended that the safety provision of all reservoirs in the valley be increased to withstand a probable maximum flood (PMF) condition.

2. This recommendation was based on the method of predicting severe floods developed by the Institute of Hydrology, "Flood Study Report", NERC, 1975 (Ref1).

3. Several alternative methods of achieving the required degree of flood protection were examined. Nine of these alternatives were the subject of an environmental appraisal including consultation with over 50 interested parties. The preferred method, selected on the basis of environmental grounds, involved concentrating the major works at Woodhead Reservoir, with only minor work required at the lower reservoirs.

4. At the time of the AR Panel Engineer's inspection the outlet capacity of the spillway at Woodhead was assessed as 170 m^3/s. This corresponded with the reservoir at crest level, less an allowance for wave run-up, and a flood with a return period of the order of 1 in 3,000 years. The spillweir was not the control on the discharge capacity but rather the relatively narrow channel leading from the weir. The dam had been classified in accordance with the ICE Notes for Guidance (Ref 2) as being a category A dam and therefore should be able to accommodate PMF inflows. By increasing the discharge capacity of the overflow at Woodhead the discharge capacity of the subsequent reservoirs in the cascade would have proved to be insufficient and increased provisions at all five reservoirs would be required. However, by restricting the flow and holding the flood water at Woodhead, only minor work in the four lower reservoirs is required.

5. From the analysis of the outlet works of Torside Reservoir, the next reservoir in the cascade, and its time lag effect, it was established that the inflow to Torside reservoir required to be limited to a maximum of 128 m^3/sec to ensure that no major works were required. By restricting the flow from Woodhead reservoir to Torside to 128 m^3/s, flood inflows to Woodhead in excess of this outflow had to be stored. Therefore a raised embankment and throttled outlet was required to increase the flood storage volume available.

HISTORY OF THE DAM

6. The storage and use of the Etherow River in the Longdendale Valley was covered by an Act of Parliament which received Royal Assent on 9 July 1847. The Longdendale scheme, and its contemporary, the Rivington scheme for Liverpool, were the first major works of this extent in the country. Very few engineers were experienced in this size of project. Manchester City Council, the promoters for the scheme, selected John Frederick Latrobe Bateman as the Engineer.

7. Construction of the Woodhead embankment commenced in August 1847. The embankment was formed where the Manchester to Sheffield turnpike road, on the north side of the valley, and the Manchester, Sheffield and Lincolnshire Railway, on the south side approached one another. The embankment comprised of a central puddle clay core supported by granular shoulders of locally won weak and weathered sandstone. The puddle clay extended 3.3 m (10 ft) into the

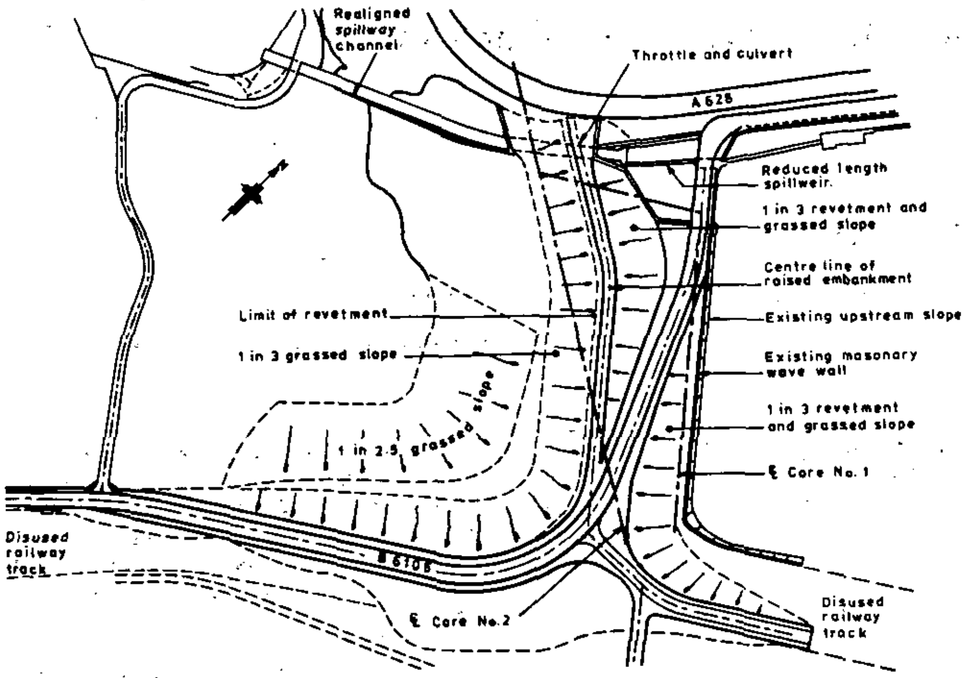

Figure 1: Plan of raised embankment

foundation (Ref 3 and 4).

8. This original embankment had side slopes of 1 to 3 for the upstream slope and 1 to 2 for the downstream slope. The upstream slope was pitched with stone. The embankment was within 7 m (20 ft) of its final 29 m (90 ft) height when the impounded water reached a depth of 9 m (30 ft). Seepage at this point had reached a balance with the inflow and no further water was being stored. Leakage had been observed at the downstream toe. Many years were spent trying to stem the leak by a variety of methods, with several causes being postulated.

9. In 1864 Bateman decided that the only way to fully impound was to construct a new embankment with a deep concrete cut-off extended through the under-lying landslip deposits into the almost impermeable shales. A study of the valley downstream of the first embankment indicated to Bateman that the most suitable location would be immediately downstream and slightly at an angle to the original embankment. Construction of this additional core, cut-off and shoulder was quickly started and was completed in 1877, a full 30 years since commencement of construction.

10. The embankment has remained virtually unchanged to the present. However, since 1887 there has been doubt about the capacity of the flood control works and from that date until 1945 the reservoir was held down 1.52 m (5 ft) to provide flood storage capacity. Several flood studies have been carried out throughout its life (Ref 5 and 6). In the late 1930's an additional length of spillweir, deepened watercourse and a new spillway chute were constructed to meet the requirements of the 1933 Interim Report of the Institution of Civil Engineers Floods Committee.

GENERAL DESCRIPTION OF THE WORKS

11. Following a detailed study, by Binnie & Partners, of the whole of the Longdendale Valley and the flood safety provisions at each of the five reservoirs in cascade (Ref 7), it was determined that all reservoirs in the valley could not accommodate a PMF event. Remedial works were therefore required for all five reservoirs to meet the requirement of the 1975 Reservoirs Act. The study also considered the necessary actions to meet the requirements and it concluded that the most favourable means of providing additional safeguards for the whole of the valley was to carry out all the major works at Woodhead.

12. To maximise the benefits of the works, while minimising the environmental impact, flood routing and increasing reservoir retention time was considered the most applicable. Woodhead being at the head of the cascade and having almost 50% of the catchment area of the valley has particular influence on the whole of the Longdendale system. Therefore, the majority of the work was concentrated at Woodhead.

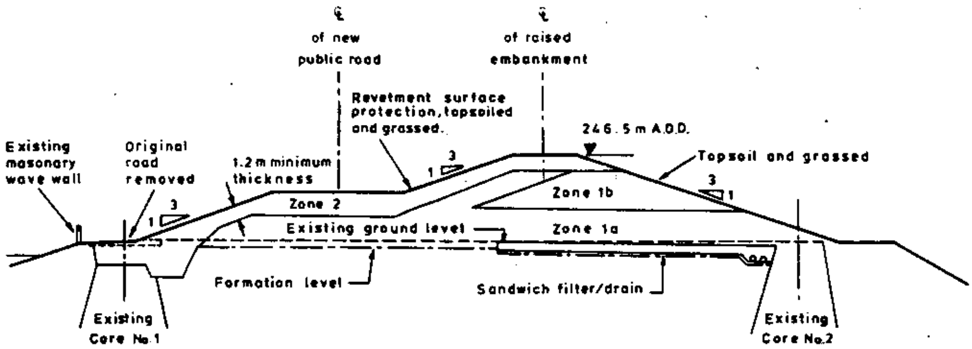

Figure 2: Typical section of raised embankment

13. The time lag of flood flows through the reservoir was relatively short and in order to increase this it was necessary to restrict, or throttle the outflow. From the study of the lower reservoirs it was necessary to limit the outflow to around 128 m^3/s during PMF conditions. This could be achieved by forming a throttle 4.5 m wide by 3.5 m high. This, however, results in the inflowing flood water being retained in the reservoir and the reservoir rising by approximately 6.0 m. The flood storage requirement is therefore to increase the height of the embankment of the order of 7.0 m.

14. The raised embankment was designed as a simple section (Fig 2) with two fill zones, an inclined upstream rolled clay zone to form the water barrier, supported by a zone of granular fill. The upstream clay membrane was connected to the original upstream puddle clay core (Core No 1). However, it was known that this core was not effective, as Bateman had demonstrated. Piezometers installed, during the site investigations, in the infill between the two puddle clay cores reflected the reservoir level. Study of the monitoring results indicated that, if the reservoir level rose as a result of storing the PMF inflow, the phreatic level in the infill zone would rise above the maximum height of the downstream puddle clay core (Core No 2). However, there was a sufficient time lag to limit the maximum rise possible, during a single flood event significantly below the maximum reservoir level. This would cause severe leakage and instability problems with the raised embankment. The phreatic level was therefore held below the level of the downstream core by the inclusion of a drainage blanket which intercepts the rising water in the infill and conveys it to discharge into the spillway channel. The quantity of water loss is anticipated to be of the order of 150 l/sec.

15. This large loss of water is only acceptable as it occurs during severe flood conditions, and would not be stored. The drainage blanket is a graded filter sandwich which will prevent loss of material from the infill. The grading was designed using the rules proposed by Sherard (Ref 8). Other details of the design are indicated on figures 1 and 2.

RESTRICTIONS ON THE CONSTRUCTION

16. During the consultation stage restrictions were placed on the construction phase to reduce its impact on the environment. In particular, as the works were located in the Peak National Park winning of fill material in the valley was not allowed. It was anticipated that all material necessary for the works would be imported into the valley. However, there were also concerns about the traffic impact on the already heavily used Woodhead Pass (A628). During the investigation stage several sources of fill were examined. The Manchester to Sheffield Railway, which runs along the south side of the valley and through the Woodhead Tunnel at the head of the reservoir, was closed in 1982. However, the track was still ballasted and was being considered as a haul route to the dam, to reduce the load on the public highways. Examination of the track revealed that the quantity of ballast was apparently in excess of the required granular fill quantity for the works. This was investigated and found to be suitable for granular fill. There was therefore only a requirement for clay, to form the water barrier, and granular filter material to be imported from out of the valley.

17. During a previous investigation into the operation of the discharge arrangements the reservoir was emptied. The silt in the

reservoir basin was observed to be several metres thick. The stream flow eroded the silt leaving near vertical walls. During a period of heavy rain following a dry spell a large volume of silt slipped and liquefied causing the draw-off tunnel to be blocked by silt. To ensure that a similar situation did not recur North West Water required that the remedial works be carried out without taking the reservoir out of commission.

18. The reconstruction of the spillway channel and throttle was required to be on the line of the old channel. Physical hydraulic model studies were carried out at Salford University in 1986 to establish the optimum alignment. The decision to locate the new channel on the line of the old channel and with the reservoir not out of commission, has posed problems in the maintenance of an adequate discharge capacity through the Works during the construction period.

RISK ASSESSMENT

19. The construction of an orifice controlled discharge channel for an existing reservoir previously controlled by a larger

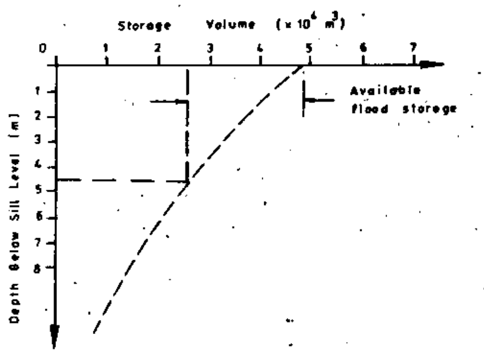

Figure 3a: Reservoir capacity below sill level

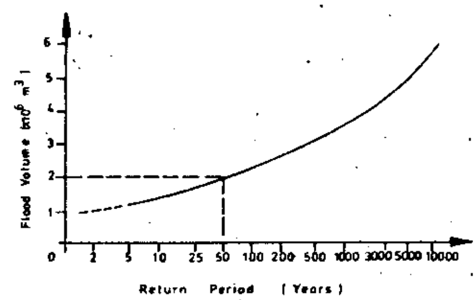

Figure 3b: Flood volume against return period (Modified spillway)

capacity side-spillweir introduced several factors which required to be considered in comparison to the normal spillweir widening project. These additional factors all had a bearing on the risks imposed on the Works.

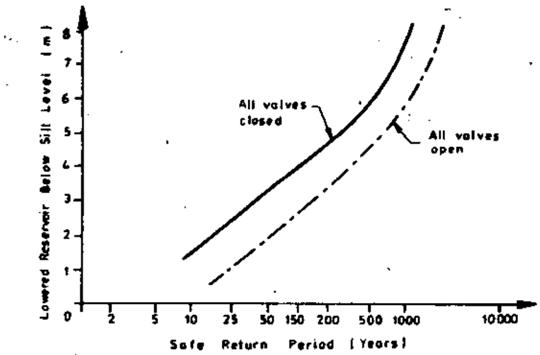

Figure 4: Overtopping risk

20. During the design, the hydrological analysis indicated that the inflow for PMF conditions would more than fill the reservoir basin even if it had initially been completely empty (Fig 3). As previously stated, problems had been experienced with the draw-off arrangements during a period with the reservoir empty and therefore it was decided that the reservoir could not be completely emptied during the construction period.

21. For the construction period it would be necessary to accept a risk of the spillway channel works being inundated due to flood water discharging through the channel. It was also necessary to accept a risk of the embankment being overtopped during the period of the works. The acceptable risk is a very difficult concept for most engineers to embrace and fully quantify. It was accepted that the relatively short duration of the works could allow a theoretical risk factor to be calculated. However, this only factorised the long term acceptable risk. The question as to what is acceptable still remained. It was considered that as the existing works were lower than the final acceptable risk, but at a risk level that had been carried for over forty years, this level of provision during the Works would be appropriate.

22. It was accepted that the reservoir would have to be held down to avoid floods of short return period inundating the spillway channel works. The risk of inundation of these works was based on precedence and a balance between the economic value of the loss of storage and the cost to the works from damage as a result of the inundation.

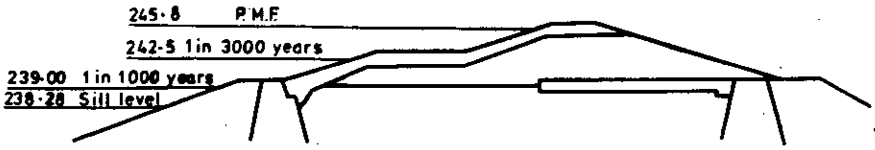

Figure 5: Section showing flood lift

This risk element was set as a flood event with a return period of 1 in 50 years which would cause the reservoir to increase in level to that of the weir level, any greater inflow not being fully stored. To achieve this the reservoir had to be lowered and held at maximum level of 4.5 metres below the weir sill level (see Figures 3a and 3b).

23. With the lowered reservoir level, the risk of overtopping the original embankment crest level, if no water could pass through the spillway channel was that with approximately a 1 in 200 year return period (Figure 4). This was significantly below the previously defined acceptable risk.

24. The winters in the Peak District can be very severe and it was the Employer's view that the limited draw-off facilities could not maintain the reservoir at the reduced level below the overflow weir during the winter period due to the high run-off with a frozen catchment or with snow melt. It was therefore required that no work be carried out in the spillway channel during the winter period, October to March inclusive.

25. With the orifice fully constructed a flood with a return period of approximately 1 in 1,000 years would cause the reservoir to rise from a water level at the sill level to a level in excess of the original crest level. It was therefore stipulated in the contract that provison be made at all times for facilities to handle flood inflow with a return period equal to or in excess of that which the original spillway could accommodate. This could be achieved by using either, or a combination of, a purpose built channel, the original channel or the new permanent works channel.

26. A further requirement in the contract was that if the culvert roof was constructed prior to the winter closedown then the embankment had to be raised to a level of 242.5m OD over the full length of the dam. Therefore, even with the reservoir level uncontrolled, a combination of flood storage and discharge would ensure that the risk of overtopping would not exceed that previously defined as acceptable.

27. Following normal tendering procedures the contract was let to Alfred McAlpine Construction Limited and construction commenced in early April 1989 on an eighteen month contract. An auxiliary spillway channel was provided during the initial phase of the culvert construction to maintain the required discharge capacity. Once the new culvert was functional the auxiliary channel was removed and the fill was raised to above an elevation of 242.5m OD. This work was carried out during the first construction season.

28. Weather conditions during the last quarter of 1989 were very good and a relaxation was granted for the contractor to continue working in the spillway channel until the end of October.

29. To improve on the accepted risk, North West Water attempted, and generally succeeded, in holding the reservoir level down 4.5m below the sill level. Only during periods of flood inflow was this not achieved and at no time over the 1989/90 winter did the reservoir level exceed 2.0m below sill level.

30. Construction continues through 1990, with the contract completion programmed for early October 1990. The risk of overtopping the embankment will continue to reduce throughout 1990 until when the embankment is at final level and final surface protection is complete at which point the works will be able to accommodate full PMF conditions.

CONCLUSIONS

31. Risks are imposed on the Works from various quarters. These must be assessed to ensure that they are being fully catered for and that no unacceptable risk is being carried by any of the interested parties. By detailed analysis the level of risks being carried during remedial works can be determined and a level of risk determined which is acceptable to all parties.

ACKNOWLEDGEMENTS

32. The Author is grateful to North West Water for their permission to publish this paper and to my colleagues and the Partners of Babtie Shaw & Morton for their assistance and encouragement in its preparation.

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21. The Bureau of Reclamations new downstream hazard classifications guidelines

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SYNOPSIS
A new Bureau of Reclamation (Reclamation) document, "Downstream Hazard Classification Guidelines" (ref. 1), has been published for guidance in determining a dam's downstream hazard classification. This document reflects Reclamation policy and philosophy regarding flood danger to people, combined with new quantifiable depth-velocity flood danger level criteria. This document includes guidance and criteria for performing dambreak/inundation studies suitable for downstream hazard classification purposes. This paper summarizes key portions of the "Downstream Hazard Classification Guidelines".

INTRODUCTION

1. A downstream hazard is defined as the potential loss of life or property damage downstream from a dam and/or associated facility (e.g. dike) due to floodwaters released at the structure of waters released by partial or complete failure of the structure (ref. 1). Downstream hazard classification is not associated with the existing condition of a dam and its appurtenant structures or the anticipated performance or operation of a dam. Rather, downstream hazard classification is a statement of potential adverse impact on human life and downstream developments if a designated dam failed.

2. Although the Guidelines are intended to be used for all dams, they are especially useful for dams whose failure flood would affect only a small population (e.g. small dams). The purpose of the Guidelines is : (1) to define the Safety Evaluation of Existing Dams (SEED) method for assigning a dam's downstream hazard classification; (2) to provide guidance and present methods, for the purpose of downstream hazard classification, for estimating the downstream area susceptible to flooding due to a dam failure; (3) to provide guidance and criteria for identification of downstream hazards; and (4) to bring objectivity and consistency into downstream hazard classification.

3. Reclamation's "Downstream Hazard Classification Guidelines" can be obtained by sending a request to: Bureau of Reclamation, Denver Office, P.O. Box 25007, Denver, Colorado 80225, Attention: D-7923A. The cost is $2.00 per copy (Foreign handing: Surface mail $0.50, Air Mail $1.00).

PURPOSE OF DOWNSTREAM HAZARD CLASSIFICATION

4. Dams within the U.S. Department of the Interior (DOI) are given a downstream hazard classification for two reasons:

1. The DOI Manual, part 753 (ref. 2), established that a downstream hazard classification is to be assigned to every DOI dam.
2. Downstream hazard classification serves as a management tool for determining which dams are to undergo the full SEED (Safety Evaluation of Existing Dams) process. Dams having a low downstream hazard classification are excluded, whereas those having a significant or high downstream hazard classification are included.

DOWNSTREAM HAZARD CLASSIFICATION SCHEME

5. The system presented in table 1 is used by the SEED program for classifying Reclamation and other DOI dams.

Table 1. Downstream hazard classification scheme

Classification	Lives-in-jeopardy	Economic loss
Low	0	Minimal (undeveloped agriculture, occasional uninhabited structures, or minimal outstanding natural resources)
Significant	1-6	Appreciable (rural area with notable agriculture, industry, or worksites, or outstanding natural resources)
High	More than 6	Excessive (urban area including extensive

community,
industry,
agriculture, or
outstanding natural
resources)

Lives-in-jeopardy

6. Lives-in-jeopardy is defined as all individuals within the inundation boundaries who, if they took no action to evacuate, would be subject to a dangerous situation commensurate with the depth-velocity flood danger level criteria explained later in paragraphs 11 through 14.

7. Lives-in-jeopardy is divided into permanent and temporary use. Permanent use includes permanently inhabited dwellings, worksite areas, and industrial areas, whereas temporary use includes roads, campgrounds, and other recreational facilities.

IDENTIFICATION OF DOWNSTREAM HAZARDS

8. Identification of downstream hazards is one of the most important topics of the Guidelines. Therefore, it will be given extra consideration in this paper.

9. Sometimes downstream hazards classification is obvious. That is, an analysis is not necessary because lives-in-jeopardy and/or property damage could be determined with little doubt. If a downstream hazard is not obvious, and/or "possible downstream hazards" have been identified, then a dam-break/inundation study is performed for the purpose of determining the impact of a dam failure flood on the possible downstream hazards. A possible downstream hazard is one that has been identified as having the possibility to constitute a downstream hazard, but field work and/or analyses need to be performed for confirmation. Possible downstream hazards are identified from topographic maps, photographs, field surveys, and information from "locals". Downstream hazards include any situation that is suspected of having a potential for lives-in-jeopardy or economic loss due to a dam failure.

10. Analysis does not always prove a possible downstream hazard to be a confirmed downstream hazard; many "gray areas" exist in downstream hazard classification. Analysis may indicate that a residence may be flooded by 1 foot (0.3 m) of water, but will this result in loss of life? If a failure flood overtops a highway bridge, will the bridge be destroyed? If not, will a vehicle be carried by the floodwater or go out of control due the hydroplaning? Or, will a vehicle crash due to a damaged road or bridge after the flood has passed? Questions and gray areas such as these are the underlying reasons for creation of the Guidelines.

11. Figures 1-5 contain curves of depth versus velocity that are indicative of dangerous floodflows for various possible downstream hazards. These curves assist the analyst in making decisions regarding questions such as those addressed in paragraph 10. The curves are presented for the following situations: (1) houses on foundations, (2) mobile homes, (3) passenger vehicles, (4) adults, and (5) children.

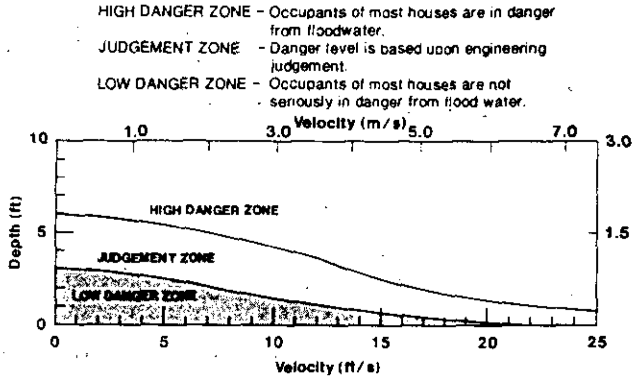

Figure 1. – Depth-Velocity flood danger level relationship for houses built on foundations.

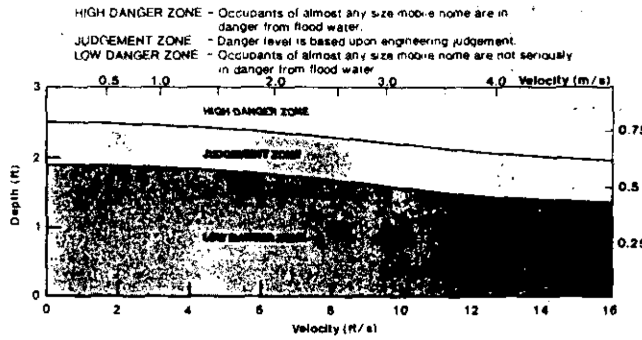

Figure 2. – Depth-velocity flood danger level relationship for mobile homes.

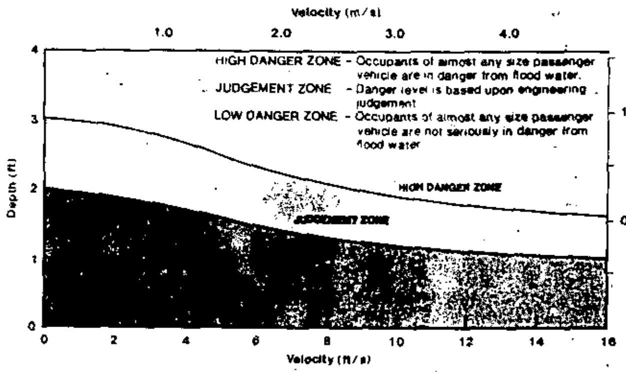

Figure 3. – Depth-velocity flood danger level relationship for vehicles.

12. Figure 1 is a modification by the author of a study performed by Black (ref. 3). The curves in Figs 2-5 were derived theoretically by the author. Figure 3 is in reasonable agreement with a theoretical analysis performed by Simons, Li and Associates (ref. 4). The lower curve in Fig. 4 is in reasonable agreement with a theoretical analysis performed by David J. Love and Associates, Inc (ref. 5), and a laboratory flume study performed at Colorado State University by Abt and Wittler

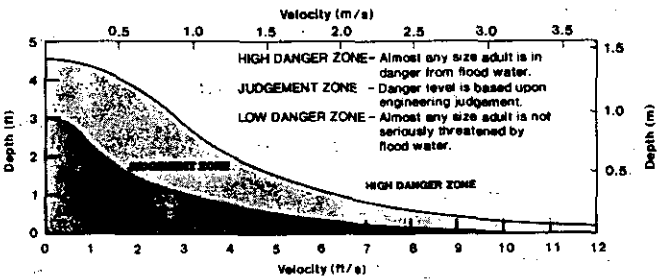

Figure 4. – Depth–velocity flood danger level relationship for adults.

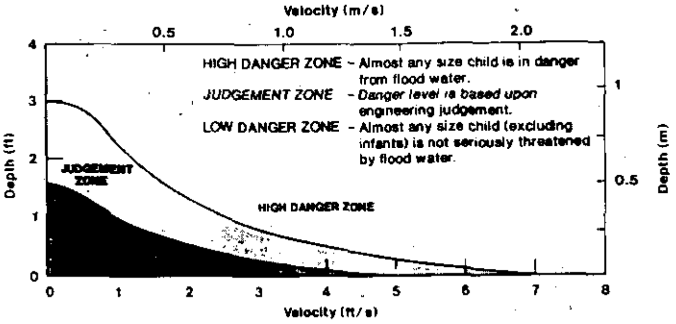

Figure 5. – Depth–velocity flood danger level relationship for children.

using monoliths (ref. 6). Very little research has been done on this topic; even if this were not the case, there would always be discrepancies which could not be avoided due to the many initial assumptions that would have to be made, very large number of variables that would have to be considered, and personal philosophy. The relationships presented in Figs 1-5 are very reasonable for estimating lives-in-jeopardy for downstream hazard classification needs, and satisfy one of the purposes of the Guidelines - to bring consistency and objectivity into downstream hazard classification.

13. The depth-velocity flood danger level relationships are divided into three zones: low danger, judgement, and high danger. An explanation of these three zones follows:

Low-danger zone. - If a possible downstream hazard is subject to a depth-velocity combination plotting within this zone, then the number of lives-in-jeopardy associated with possible downstream hazards is assumed to be zero.

High-danger zone. - The low-danger and high-danger zones represent the two extremes of reasonable certainty regarding the occurrence of no lives-in-jeopardy associated with possible downstream hazards is assumed to be zero.

Judgement zone. - The low-danger and high-danger zones represent the two extremes of reasonable certainty regarding the occurrence of no lives-in-jeopardy and some lives-in-jeopardy, respectively. Between those two extremes exists a zone of uncertainty with respect to assessment of lives-in-jeopardy. Because every flood situation is unique, it is impossible to account for all of the variables that may result in lives to be in jeopardy. Thus, in this case, it is left up to the analyst to use engineering judgement for determining lives-in-jeopardy. Whenever possible, several opinions and a common agreement among analysts should be reached in making this determination.

14. In many downstream hazard classifications, especially where large dams and catastrophic flooding are involved, reference to Figs 1-5 is superfluous because of the obvious flood danger. But, for situations where the downstream hazard classification of a dam is solely dependent upon an isolated flood situation where occupants of a dwelling or vehicle may be in danger, or a person having no protective environment (e.g. house, vehicle) may be in danger, Figs 1-5 should be used. In such situations, the analyst will have predicted a reasonable maximum depth and velocity, with confidence, at the possible downstream hazard sites.

ESTIMATING INUNDATED AREA

15. Determination of downstream hazard classification based on the downstream hazard classification system presented in table 1, and the flood danger via Figs 1-5, is straightforward, providing the extent of flooding is known. The following methods for determining the extent of flooding are recommended in the Guidelines: (1) use of an existing inundation study, (2) engineering judgement, and (3) a dam-break/inundation study.

Use of existing inundation study

16. Many dams have comprehensive dam-break/inundation studies prepared for the downstream area. If these studies exist, they should be used as the basis for downstream hazard classification.

Engineering judgement

17. In some situations the downstream hazard classification may be obvious - a large dam with a populated area located in the flood plain immediately downstream from the dam, for example. In these cases the downstream hazard classification is based solely on engineering judgement using information from a field survey and/of other current topographic maps.

A dam-break/inundation study

18. If a comprehensive dam-break/inundation study does not exist, or the downstream hazard classification is not obvious, then an analysis should be performed to define the inundated area. There are three main phases to a dam-break/inundation study: (1) assume a dam failure scenario, (2) determine the downstream terminal point of flood routing, and (3) perform the recommended analytical procedure.

19. Assume a dam failure scenario. The results of a dam-break.inundation study would be the most accurate if we knew the failure scenario a priori. However, for dam-break/inundation studies, this is uncertain and can only be assumed.

20. The failure scenario possibilities are nearly infinite. Because of this, and for

safety of dams conservativeness, the Guidelines suggest a procedure that seeks the highest downstream hazard classification that is reasonable. This procedure begins with the evaluation of a sunny-day failure. If the sunny-day failure scenario results in a high downstream hazard classification, no further analysis is necessary because this is the highest downstream hazard classification possible. However, if the downstream hazard classification is less than high, then additional analysis is necessary to determine the incremental flooding, that is, if a more severe downstream hazard classification can be obtained if the dam should fail in combination with the "incipient danger flood" prior to the dam failure. The incipient danger flood is the natural runoff flood that results in the possible downstream hazard to experience incipient flooding. For example, a runoff flood that causes flooding at the foundation/ground interface of a house is the incipient danger flood for that particular house. After the dam-break flood is combined with the incipient danger flood, for all possible downstream hazards, the magnitude of flooding is compared to Figs 1-5 and the lives-in-jeopardy and, subsequently, the downstream hazard classification determined.

21. Determine the downstream terminal point of flood routing. A dam-break flood routing needs only to be performed for a distance downstream from the dam until the downstream hazard classification can be ascertained, or until "adequate flood water disposal" is reached. Adequate flood water disposal is that point below which potential for loss of life and significant property damage caused by routed floodflows appear limited.

22. Recommended analytical procedure. The Guidelines recommend use of the National Weather Service Simplified Dam-Break model (SMPDBK) (ref. 7) for estimating flood depths and velocities, and offer specific criteria for breach parameters. SMPDBK is recommended because of its ease of use together with reasonable accuracy. However, it is cautioned that the analyst be knowledgeable of SMPDBK's limitations. Other methods should be used, such as the National Weather Service DAMBRK model (ref. 8), when more accurate results are desired and/or the analyst has the background, time, and resources needed to apply such methods. Additional information regarding dam-break/inundation studies is provided in the appendix of the Guidelines.

CONCLUDING REMARKS

23. While downstream hazard classification may be obvious in situations pertaining to large dams and populations, it often requires detailed analysis for other situations such as those involving small dams and populations, However, detailed analysis does not always result in a firm downstream hazard classification. Many unknowns exist with regard to loss-of-life to persons in dwellings, vehicles, or on foot. Due to these unknowns, agency guidance is important to give consistency in assignment of downstream hazard classification. The Guidelines are intended to provide such assistance.

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Information in Database.

22. Mitchell's House reservoirs - investigations, monitoring and remedial works

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SYNOPSIS

Mitchell's House reservoirs are retained by an earth embankment and form a single impounding reservoir divided by an intermediate embankment.

Survey stations installed on the embankments indicated that differential settlements were occurring. Together with damp areas observed on the face of No. 2 embankment, this led to a geotechnical investigation being carried out.

Analysis of the data showed that settlement of the main embankment was continuing; drainage of the base of the dams was taking place into the foundations; high pore pressures existed in the downstream shoulders; and the stability of the downstream shoulders of the embankments was inadequate.

Sand drains have been installed immediately downstream of the core to intercept seepage and drain it into the foundations. A rockfill berm installed on the downstream face of No. 2 embankment hs reduced the rate of settlement. A similar berm is to be placed against No. 1 embankment.

INTRODUCTION

1. Mitchell's House reservoirs are situated 3 km SE of Accrington, Lancashire. They were built in the latter half of the 19th Century to the conventional design of the time with earth embankments and puddle clay cores. No. 1 reservoir was built first, with embankments facing West and South, and No. 2 reservoir was formed by extending the Western embankment, leaving the South embankment as a dividing embankment. The total length of the main embankment is 525m, with a maximum height of 20m. The reservoirs share a single overflow.

HISTORY

2. Construction of No. 1 reservoir began in 1855, the embankment being built with a puddle clay core connected by a clay blanket at formation level to a cut-off trench along the upstream toe. However, the dam was not watertight until extensive repairs were carried out to the core between 1881 and 1891. As was the practice at the time, a number of headings were driven into and under the dam to find the source of the leak, and eventually a large part of the core was excavated and reconstructed.

3. Work began on No. 2 reservoir some time after 1876, but apparently even before No. 1 was satisfactory. Again there were problems in making the reservoir watertight, and these were not overcome until 1892, when a wing trench in the Southwest corner was filled with clay.

4. Coal mining took place beneath the reservoirs, but ended in 1905, after which no further problems appear to have occurred, although some reservations were expressed about the possibility of long term effects. Work was done in 1958 to raise the main embankment and puddle clay core and enlarge the overflow. At the same time some of the exploratory headings remaining from the 1881 repairs were sealed.

5. The reservoirs appear to have been satisfactory for some years following this work. However following an Inspection in 1980 the Engineer reported that the embankment of No. 2 reservoir was "distinctly soggy" in places. He recommended that a system of survey stations should be installed, and that the old heading system should be inspected, with the next Statutory Inspection to take place in 1983.

GEOLOGY

6. The reservoirs are constructed on a silty/clay glacial till, 10 to 15m thick, which overlies horizontally bedded sandstones and mudstones of the upper coal measures. The upper 2m of rock is moderately weathered with water passing through it - lower down the rock is solid and less permeable.

7. The strata below the till is intersected by Northeast/Southwest faults, one of which lies upstream of the main embankment, with a width varying between 12 and 30 metres.

8. The Lower Mountain coal seam is located between sound sandstone at a depth of 70m. Mining of this seam took place in the area beneath the reservoirs between 1886 and 1905.

MINING

9. The first report of coal mining under the reservoirs was in 1886 when it was stated that a considerable quantity had been worked under No. 1 reservoir, but very little under No. 2; another report in 1892 drew attention to the risk of settlement and fracture if the pillars under the reservoirs were removed. Further reports in 1893 and 1896 detailed the risks to the reservoirs if all the coal were to be removed, and in 1897 agreement was reached with the mine owners to leave 50% of the coal as pillars immediately beneath the main embankment, although it appears that the coal beneath the reservoir basins was removed completely.

10. In 1903 mining to the West of No. 2 embankment caused subsidence at the ground surface which damaged sand filters under construction. It also resulted in No. 2 reservoir having to be "emptied for repairs" but there is no record of what this involved. Mining was abandoned in 1905.

EXPLORATORY HEADINGS

11. The headings driven below the North end of No. 1 embankment in 1881 were on two levels - an upper level varying between 6m and 9m below the original ground surface, and a lower level some 8 to 9m deeper. The upper headings were in the glacial till, and lined with masonry, the lower ones were in sandstone, and unlined. The upper headings, and short drives from them, appear to have followed the line of the centre of the dam, while remaining on the downstream side of the core. The lower headings went beyond the centre line, and originally had two spurs extending well under the reservoir basin. These spurs were sealed with concrete in 1958, as were parts of the upper system.

12. An inspection of the heading system in 1958 found deposits of ochreous sludge in the lower headings, and water flowing in them. There was concern that this might indicate that material was being removed from below the embankment.

SURVEY STATIONS

13. Following the 1980 Inspection, 63 permanent target stations were installed on the crests and downstream shoulders of the main embankment to monitor surface settlements.

14. By the next Statutory Inspection in 1983 four sets of levels had been taken. Although there were some inconsistencies between individual results, the general indication was that settlement of 20 - 30mm had taken place over a period of 30 months on parts of the embankment, affecting both reservoirs.

SITE INVESTIGATION

15. In his report on the 1983 Inspection of the reservoir, the Engineer recommended that a site investigation should be carried out: (a) to establish whether settlement was taking place within or below the embankment; (b) to establish the main parameters of the material used in the construction of the embankment; and (c) to establish the piezometric gradient through and under the embankment.

16. It was decided to investigate one section through No. 1 embankment, and two through No. 2, selected as representing areas of settlement and wet areas on the downstream face. Six boreholes were drilled on each section, and instrumentation including both pneumatic and standpipe piezometers, and vertical extensometers, was installed. A number of trial pits were also excavated to establish the core position at the crest of the embankment.

17. A geophysical survey was also carried out using resistivity and conductivity methods to identify seepage zones and variations in materials. The results confirmed the interpretation of the ground investigation and piezometer readings, and indicated that while seepage from No. 1 reservoir was concentrated in the bottom layers of fill, that from No. 2 appeared to occur at a number of levels.

INTERPRETATION

18. The results from the Site Investigation established that the embankments consist of a silty clay fill of a soft firm consistency, poorly compacted, and containing a variable quantity of gravel sized fragments of sandstone and shale. However, the fill material is generally of low permeability of the order of 1 x 10^{-7}m/s. The embankments overlie a firm silty clay glacial till which is 10 - 15m thick with lenses of peat and other organic matter.

19. The bedrock is a horizontally bedded strong sandstone with mudstone bands, moderately weathered to approximately 2m below rock head, forming a permeable layer. Below this, the worked out lower mountain mine coal seam was encountered at 70m depth and had sandrock above and below. There was no evidence of subsidence at this depth in the single borehole drilled to this level. The rock below the seam was used as a base for a surface benchmark.

20. Evidence of a puddled silty clay core in No. 1 Reservoir was firmly established, but this had not been brought up to the raised height in the 1958 remedial work and stopped at a depth of 1.2m below the crest. No recognisable core was encountered in the investigation into No. 2 Embankment, and no cut-off excavation into the foundation material or rock below. Again, the central clay material does not appear to have been carried up to the crest in the raising carried out in 1958.

The general findings from the site investigation are shown diagrammatically in Figures 1 and 2 for Nos. 1 and 2 Embankments respectively.

21. <u>Seepage Analysis</u>

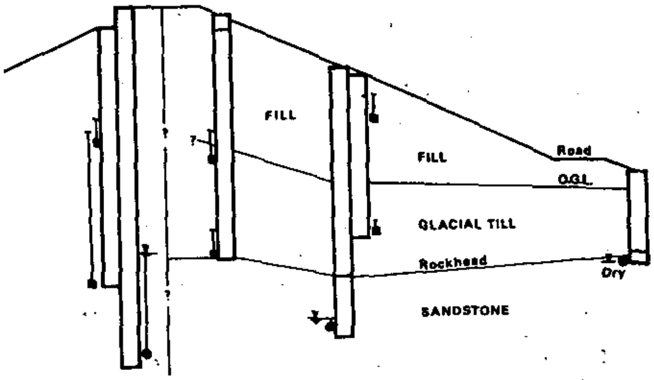

Figure 1 – No.1 EMBANKMENT CROSS-SECTION

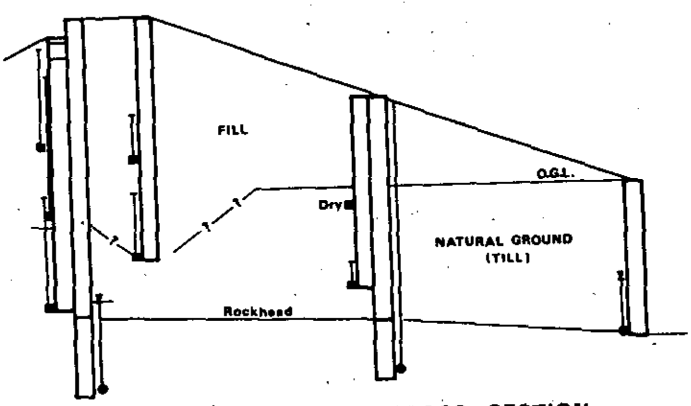

Figure 2 – No.2 EMBANKMENT CROSS-SECTION

A finite element model of the embankments was formed with sections divided into five soil types and their permeability characteristics altered within realistic limits in order to reproduce the piezometer readings at the time of the investigation. Flow patterns arising from these assumptions were then produced, indicating that the weathered rock head horizon acts as a drainage lair and has an appreciable effect on the piezometric surface, and indicated generally good drainage in the downstream shoulder. However, there is also indication of some seepage flow at higher levels emerging in the downstream slope indicated by some surface piezometers. The typical flow pattern is illustrated on Figure 3 for No. 1 Reservoir.

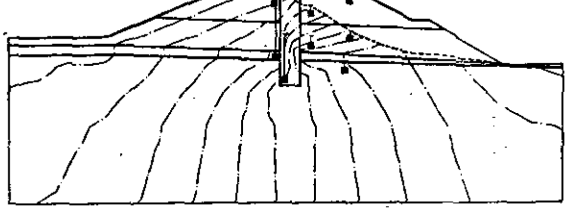

Figure 3 – FLOW PATTERN THROUGH No.1 EMBANKMENT

22. <u>Stability</u>

A number of features gave cause for concern in regard to the stability of the embankments:-

(a) the downstream slopes of the banks were steep and near the equilibrium point for surface slips;

(b) the downstream face of No. 2 Embankment was irregular;

(c) no obvious drainage facilities were found in the toes of the embankments, and damp patches were common on the downstream faces;

(d) at high reservoir levels water issues onto the road crossing the downstream shoulder of the No. 1 Embankment.

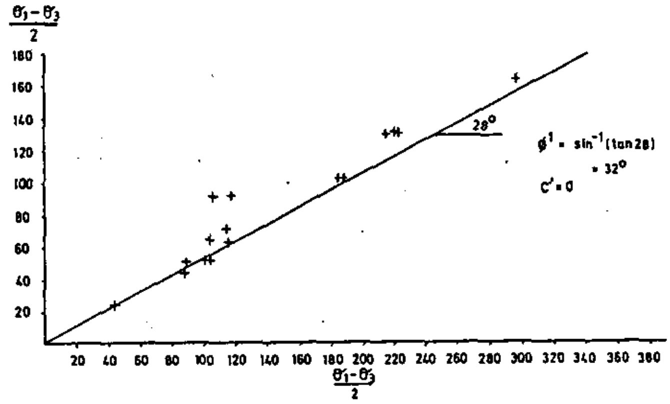

Figure 4 – EFFECTIVE STRESS PIEZOMETER RESULTS

Based on the stress parameters obtained from the consolidated undrained triaxial compressive test results, shown plotted on Figure No. 4, the effective stress parameters used in the calculations were:- $\phi' = 32°$ and $C' = 0$. Check calculations were also carried out in the range $\phi' = 30 - 34°$ but this was found to have little practical effect on the results obtained: The main reason for this being that factors of safety were dominated by the pore pressures. The analysis was carried out for circular slips with a range of piezometric surfaces under the worst assumptions, factors of safety less than 1 were recorded but the piezometric surface levels supported by actual readings indicated the factors of the safety of the order of 1 to 1.25.

23. It was concluded from this analysis that steps should be taken to improve the drainage in the downstream shoulder to control the pore pressures and at the same time increase the factor of safety against shallow slips by adding berms to both embankments. The extent of the berms was to some extent dictated by the space available but it was decided that berms of 4m width to safe slopes could be established on both embankments, leading to increases in factors of safety between 15 and 25%, which should relieve the stress on the embankments and ensure continuing stability.

24. Remedial Measures

A number of alternative remedial measures were considered to improve the factor of safety including the following:-

(a) reducing the level of the spillway and hence the top water level in the reservoir. This was unacceptable as the reduction of 2m in water level would have reduced the capacity by over 25%. Thus, making the storage inadequate for the source;

(b) The reservoirs could be taken out of commission. This would involve substantial works and lead to the possibility of additional flooding below the site of the embankments. In any case it was decided as a matter of principle that the source was a valuable commodity and should be maintained;

(c) provide a new core seal to the embankments to obviate the possibility of further seepage and settlement. Alternative methods including grouting, sheetpiling, pile replacement and slurry changing were considered but the cost of such works was considerably in excess of alternative solutions;

(d) the installation of additional drainage. This could be achieved by installing trench drains, a series of finger drains in the upper sections of the embankments, installing horizontal drainage holes within the embankment, or installing vertical drainage holes from the crest. On the grounds of cost and effectiveness the vertically inclined holes were finally installed because it was felt that this would provide further site investigation data and enable control on the drainage to be exercised whilst further longer term stability solutions were considered;

(e) to improve the stability by installing berms on the downstream toe of the embankment as a free draining weight block to increase the factor of safety against a slip. This was adopted as an economical solution but subject to programming of other works has yet to be completed.

25. To summarise; the agreed remedial measures were to install the inclined vertical drainage immediately downstream of the crest of the embankment. This was achieved by drilling holes inclined at 7.5° from the vertical at 3m centres and installing a 75mm slotted tube surrounded with gravel. A section is shown on Figure 5.

Figure 5 – VERTICAL DRAINAGE DETAIL

Berms were to be installed on both embankments: No. 2 Berm was placed during the winter period 1988/89, and appears to be performing satisfactorily. The berm on No. 1 Embankment will be installed in 1990/91 as soon as the new treatment works have been completed allowing for demolition of the existing works at the toe of No. 1 Embankment.

26. The berms were designed with a width of 4m (dictated by the needs of the construction plant) and slopes of 1:3 (vert:horiz) along the length of No. 1 Reservoir, and a slope of 1:2.5 along No. 2. These required a fill volume of approximately 7,000m^3 and consisted of well graded limestone compacted in layers to give a density of 2.1 Mg/m^3. A crushed sandstone filter layer was installed beneath the limestone to dissipate excess pore pressures generated during construction. Additional counterfort drainage was provided above the berm laid to a drain along the inner edge of the berm and piped across to a ditch along the toe. During the construction of No. 2 Berm some old stone slab counterfort drains were discovered in the slope of No. 2 Embankment, indicating that this had been a long standing problem with this embankment.

MONITORING

27. Throughout the period following the initial site investigation in 1983 through to the present time the embankments have been monitored to assess the ongoing situation in regard to both seepage and settlement. Piezometric levels have been measured in piezometers installed in the original and subsequent investigation work, and also recorded in the drainage system installed in 1986. Settlement has been measured on the extensometers installed within the embankment and on surface monitoring points installed originally in 1981.

28. The piezometer readings have indicated generally that the drainage system is now performing satisfactory, and the readings taken on the vertical inclined drains show that these have continued to operate and respond slightly to increases in reservoir level or heavy surface water.

29. The degree of settlement has always proved very much more difficult to evaluate, particularly in regard to readings on the surface monitoring points. These have been read on average once a year since installation in 1980, but unfortunately by different teams of surveyors giving results which have been difficult to correlate. Readings since 1984 have been more consistent as they were carried out by the same independent firm of surveyors. During the period there has continued to be a slow but steady settlement occurring with differential movements peaking with the installation of the crest drainage system in 1986 but generally continuing at a slow rate. At peak, settlements of about 20mm were recorded but have generally averaged a few millimetres per year.

30. These have been confirmed by the more precise readings taken within the extensometer tubes showing settlement taking place of the order of 5 - 10mm per year. It is difficult to be certain of the reason for these, but there appears to be no evidence of the removal of material from the bank or foundations by seepage, and it is more likely that this is due to continuing consolidation and settlement of a poorly compacted embankment subject to additional loading and improved drainage in recent years. There is no doubt that the understanding of the performance of the embankment given by the comprehensive set of instrumentation now installed enables it to be carefully and properly monitored. Any sudden alterations in the pattern of settlement or seepage should show up within the pattern of readings to give adequate warning of any problem. These are plotted on an ongoing programme with a time base with groups of instruments associated with reservoir water levels showing each part of the embankment.

CONCLUSION

31. When the remedial works are completed with the installation of the berm on the No. 1 Embankment we believe that the Mitchell's House Nos. 1 and 2 Reservoirs will then be in a stable and safe condition to give service for many years to come. Nevertheless, because of the complex nature of the structures and the fact that they overlie old coal mine workings it will be necessary to maintain vigilance and careful monitoring to ensure they continue to remain in a safe and stable condition.

Discussion

N. CULLEN (Water Research Centre)
As part of its 1988/89 subscription research programme for the then Water Authorities, WRc had published a methodology for the preparation of innundation maps for use by emergency bodies in the unlikely event of a threatened reservoir failure. The work had been undertaken by Binnie and Partners under a sub-contract, but remained confidential to the original clients (who include the Scottish Regional Councils). The work had been strongly encouraged and assisted by North West Water and Yorkshire Water.

Mr. Cullen expressed the personal hope that, because the DAMBRK program was now available and because of the increased attention being paid in all branches of industry to hazard assessment and contingency planning, that Inspecting Engineers would recommend to their clients that an inundation map be prepared for embankment dams where a significant number of people - say 100 or more - were at risk.

He had recently attended a conference on Emergency Planning and felt confident that mechanisms existed to handle such information sensitively and confidentially, and to incorporate it within the "All Hazards" contingency plans for civil disasters which are currently under preparation at County and District level.

Based on the track record of performance of UK dams over the past 60 or so years, the probability of failure in the future could be encapsulated by the "Rule of 63" - there is a 63% chance of a major failure during the next 63 years. This risk was very low in comparison with most others which UK society faces, but the threat was not inconceivable.

F.M. LAW (Institute of Hydrology)
In Mr. Trieste's paper the hazard classification for USBR was based on lives-in-jeopardy being within or outside a grouping of 1-6 people. It appeared to lay stress on the number of individuals under threat. By contrast the British approach was to recognise the unacceptability of any community of people being under threat from an engineered structure. This was evidenced by public reaction to the Flixborough and Aberfan disasters, as well as to the Dolgarrog and Lynmouth floods. Building a dam with a planned threat to a community, even if 10000 to 1 against it happening next year, was not our standard. Society accepted much larger death tolls on its road systems so long as they were perceived as random individual threats; however a coach crash killing, say, a primary school group drew a far sharper call for action in safety terms.

Unfortunately the USBR Guidelines recommend using the NWS Simplified Dambreak model (SMPDBK). This was known to have clear drawbacks (Ref. 1) compared with DAMBRK proper and it is recommended that the latter version be used in the user-friendly form developed for the Department of the Environment by Bradford University and Binnie and Partners.

It was interesting to hear Mr. Knight stress the 1000 year life of dams in Sri Lanka. Once one recognised that dam life was of that order (and not just until the capital loans are paid off) then the high risk of building any important spillway with a 1000 year flood capacity was clear!

Ref. 1 Water Resources Commission of New South Wales (1986), Dambreak Seminar Papers (especially that by R. Stack)

J.H. PHILLIPS
What consideration is given or should we give, to the effects of the P.M.F. emerging from our spillways. Over a number of years when I was involved with reservoirs, land drainage and tidal retaining embankments in the Lower River Severn basin, I was concerned that little attention was given to the large variation in the return times catered for and accepted by the public. From 1 in 2 years for the overtopping of tidal embankments which could cause loss of life in farms and hamlets, to in excess of 1 in 10,000 for reservoir flooding from a dam, where very soon on the flood route water would be retained by road and rail embankments leading to deep flooding of housing, or should a breach occur, similar but more damaging flooding. At best the culverts in these embankments will only cater for a 1 in 150 event.

In the light of these variations I support the flood routing investigation concept, the results of which should be passed to the Town Planning Authorities. This may prevent further development on flood routes and lead to a wider appreciation of the risk and acceptance of realistic common standards.

E.M. GOSSCHALK (Halcrow and Partners)
Research on seismic risk to U.K. dams is being carried out for DOE by Halcrow and Building Research Station in collaboration. A draft guidance document for those concerned with the safety of UK dams was in course of preparation and was due to be submitted to DOE by the end of October. DOE's comments and approval were not as yet available. It was intended to propose to DOE that the draft should be circulated to representative authorities for comment before publication.

It is the case that there is a risk, which is described as extremely small, that earthquakes of greater magnitude than generally perceived could occur in the region of the UK. Figures by Ambraseys and Jackson (Ref. 1) showed epicentres and focal depths of recorded earthquakes which had Richter magnitudes of up to about 6 and focal depths generally of 15 km or less. These suggested that the occurrence of events of higher magnitude is conceivable and at such relatively shallow depth (in international experience) surface effects could be very serious.

Risk Factor	Extreme	High	Moderate	Low
	Contribution to risk (weighting points)			
Capacity ($10^6 m^3$)	> 120 (6)	120-1 (4)	1-0.1 (2)	< 0.1 (0)
Height (m)	> 45 (6)	45-30 (4)	30-15 (2)	<15 (0)
Evacuation requirements in case of danger (No of persons)	>1000 (12)	1000-100 (8)	100-1 (4)	None (0)
Potential downstream damage	High (12)	Moderate (8)	Low (4)	None (0)

Table 1

Risk Factors

Total Risk Factor	Risk Class (Risk Rating)
(0- 6)	I (Low)
(7-18)	II (Moderate)
(19-30)	III (High)
(31-36)	IV (Extreme)

Table 2

Risk Classes

Figures published by Long (Ref. 2) and Irving (Ref. 3) were being used to assess the probabilities of exceedance of peak ground accelerations. The risk classification for reservoirs proposed in ICOLD Bulletin 72 is being used (Tables 1 and 2 attached), based on capacity of reservoirs, height of dam, evacuation requirements in case of danger and qualitative assessment of potential downstream damage. Ted Gosschalk commented that reservoirs of capacity one million m^3 or less or liable to necessitate the evacuation of 100 persons or fewer would not necessarily be allocated to other than low or moderate classes of risk.

Dr. Roger Musson of BGS had prepared a map which divided the UK on the basis of experience into zones of three levels of seismicity and this map was shown on the screen. Even in the zones of the lowest level of seismicity, some events would be possible, for example the damaging event at Colchester in 1884. Unprecedented events do occur even in regions which have been previously seismically quiet. A recent example was the events of magnitudes 6.3, 6.4 and 6.7 at Tennant Creek in Western Australia within 24 hours in January 1988. Well defined associations between events in the UK and geological features are lacking because the geological features are obscured by superficial deposits and the relatively small magnitudes rarely cause surface ruptures. Thus it is difficult to predict the likely location of epicentres.

It is the intention that the results of the research will include guidance on seismic loading, based on peak ground accelerations graduated in accordance with the risk posed by the dam in question, together with guidance on appropriate methods of evaluating safety of dams.

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D.E. EVANS, BINNIE & PARTNERS

The work leading up to the construction at Woodhead described by Mr. Chalmers in Paper 20 was the study of the flood hazard in Longdendale carried out for North West Water by Binnie & Partners between 1984 and 1986. Our brief was to study how the cascade of 5 reservoirs (Woodhead, Torside, Rhodeswood,

Table A Flood handling capacity of Longdendale system

Reservoir	Woodhead	Torside	Rhodeswood	Valehouse	Bottoms
Catchment:					
Direct (km^2)	33.5	24.5	4.51	3.62	1.23
Indirect (km^2)	-	33.5	58.0	62.51	66.13
Total (km^2)	33.5	58.0	61.51	66.13	67.36
Total (%)	49.7	86.1	91.3	98.2	100
Available maximum flow capacity past dam prior to work at Woodhead (m^3/s)					
	204	254	351	342	324
Required flow capacity past dam prior to works at Woodhead (m^3/s)					
(a) PMF	319	471	480	481	481
(b) 10 000 year	215	308	309	314	316
Adopted solution with throttled spillway at Woodhead					
Heightening (m)	5.93 (+ wave surcharge allowance)	1.28	0.25	0.56	0.51
Peak discharge in PMF (m^3/s)	128	345	369	388	392
Further spillway modifications	None	Minor	Minor	None	Minor

Valehouse and Bottoms) could be brought up to modern standards. We were to assist North West Water in development of proposals for the reservoirs up to but not including the design stage. In consultation and co-operation with interested parties (Longdendale lies within the Peak National Park) we were to prepare engineering alternatives and general arrangement drawings. The scope was extended to include close definition of works required for the selected alternative.

The studies concluded that the most economical way of providing adequate flood discharge capacity for the 5 reservoirs was to throttle outflow from Woodhead by a substantial amount so as to exploit to the practical maximum temporary storage in that reservoir. There was some flexibility in the choice of schemes which concentrated the major civil engineering works at Woodhead and minimised the modifications needed at the other four reservoirs. The method selected on economic grounds was also the preferred method on environmental grounds as it caused least disruption to the valley as a whole. This conclusion was endorsed by the environmental appraisal referred to in paragraph 3 of the paper.

The flood hazard problem in Longdendale is illustrated by Table A which shows the flow capacity of the whole system prior to construction of the new works at Woodhead was well below that needed to meet Category A General Standard. Moreover the two major dams would be overtopped in the 1 in 10000 year event. Generally the limitation on the existing spillways was lack of channel capacity downstream of the overflow. The cascade effect of 5 reservoirs in series meant that improved flood attenuation at the head of the system, where Woodhead commands half of the total catchment, was particularly effective as all the reservoirs would benefit. With the adoption of a throttled spillway at Woodhead the need for new spillways at the dams downstream is avoided and modest heightening of the embankments at Torside, Rhodeswood, Valehouse and Bottoms combined with some work to the existing spillways is sufficient to bring the whole system up to Category A standard. The work at these four reservoirs remains to be done.

When developing the design of the 7m heightening of Woodhead dam and adaptation of the existing spillway to give throttled discharge at high flows, Binnies realised it would be possible to construct what would be, if effect, a flood embankment on top of the wide crest available. This embankment could be set back from the No. 2 dam core and contain arrangements to intercept and divert the leakage passing through and under this embankment so that the water level just upstream of the Woodhead 2 core would never rise above the core top. Seepage through the new embankment and the existing dam fill below it would occur only in flood conditions under a head approximately equal to the flood surcharge depth.

Carefully designed filter and drainage layers would carry the seepage safely to a generously sized collector pipe set just above reservoir conservation level. This pipe would drain floodwater leakage to a discharge point at the top of the spillway chute. There appeared to be no virtue in trying to tie into the Woodhead 1 core which was known to be ineffective. Perhaps Mr. Chalmers would enlarge on the reasons why in the Babtie design (Fig. 2) it has been felt necessary to provide a clay blanket tied into the Woodhead 1 core.

M. AIREY (Mott MacDonald)
Referring to Paper 20 on Woodhead Reservoir

(a) As the reservoir was drawndown for a long period, were any desilting measures ever considered?

(b) Did the prolonged period of drawdown have any adverse effect upon the quality of water at the treatment works downstream?

(c) With the very wide crest of the existing Woodhead embankment the "heightening and throttling" solution adopted seemed the most obvious. With a normal crest width would this approach have been technically viable and/or economic?

MR CHALMERS made the following response:

(a) The reservoir was maintained in service and the silt itself was not exposed over the majority of the contract. Only during one period when the reservoir was empty, accidentally, was the silt fully exposed during the period of the contract. So there was no consideration given to desilting at that stage.

(b) It is a feed reservoir to the Torside reservoir which was being used throughout the contract period and there was no adverse comment made by our client.

(c) Yes, it was the obvious solution and there would have been considerable difficulties if the crest had not been so wide. It would definitely have required far more consideration before embarking upon raising such an old reservoir if it had a narrow crest.

23. The design and operation of flood storage dams for recreational uses

J. B. ELLIS, M. HALL, and D. L. HOCKIN, Consultant, Middlesex Polytechnic, Enfield, UK

SYNOPSIS The successful development and enhancement of recreational and amenity activities on flood storage ponds in urban catchments depends upon extending the scope of the hydrological and hydraulic criteria normally applied in their engineering design. The improvement of water quality and ecology within the pond can be achieved through the application of a variety of management procedures relating to landscaping, the status of the surrounding land and the configuration of inlet and outlet control structures.

INTRODUCTION

1.One of the most common flow control strategies in urban catchments is to provide purpose-built detention storage and developers are being increasingly obliged to provide attenuation storage for new developments (ref. 1). A recent Local Authority survey commissioned by Thames Water (ref. 2), identified over 200 individual storage schemes in the Greater London area of which the large majority were associated with surface water control on new (58%) or existing (18%) developments. Although catchment sizes ranged up to 1000 hectares, 82% were less than 100 ha and 50% less than 50 ha. Design discharges and storage volumes are typically based on a return period of either 10 or 50 years such that live storage volumes varied between 360 and 185 000 m³. The majority of these flood storage facilities were excavated basins with an earth embankment retaining structure.

2.Although flood alleviation is the primary function and justification for the construction of such impounded basins, the 1988 Land Drainage Improvement Works Regulations, as well as the 1981 Wildlife and Countryside Act, require that such objectives must be achieved in a way which protects and (or) enhances nature conservation and the environment. In considering such environmental enhancement it is essential to be mindful of local community interests including the need to make open water bodies visually acceptable whilst at the same time achieving naturalistic landscapes and habitats. Full amenity development may require the provision of special facilities which need to be landscaped into the overall reservoir design. Additionally, if the storage reservoir is to be developed for multifunctional uses which include direct contact recreational activities such as canoeing or sail boarding, there is a need to ensure that the retained water quality is 'clean' and 'safe'.

3.It is now widely recognised that surface runoff from impervious urban areas can present water quality and ecological problems in receiving water bodies (ref. 3). The discharged annual loading per unit effective hectare of a separately sewered catchment is of a similar order of magnitude as from a combined sewered catchment irrespective of the pollutant considered. Therefore urban flood storage reservoirs can be subject to polluted discharges which can potentially restrict their development for recreational, nature conservation and amenity purposes. However, a number of design criteria can be utilised to improve the environmental status of an impoverished water quality and wildlife-poor open water body in an urban area. These include considerations of water quality, physical characteristics, access and usage as well as habitat creation features.

BASIN GEOMETRY AND CONFIGURATION

Basin size

4.A number of studies in both Europe and North America (refs. 4, 5) have shown that pollution retention in reservoirs is primarily a function of water residence time and turbulence. The former factor is directly related to basin size and field studies suggest that the ratio of basin surface area to drainage area should be 1 to 2% for residential and 2 to 3% for commercial developments in order to achieve a total solids removal in excess of 70% for annual average rainfall conditions. Hydraulic residence times are another way to express basin volume. Long detention times, of the order of 12 to 36 hours, will result in good pollutant removals; the longer residence times are needed for settlement and decay of bacteria which are of obvious significance for direct contact recreational activities.

5.Optimal removals occur at water depths of between 1.0 to 1.5 metres which will also encourage oxygenation as well as serving bird roosting and feeding purposes. Deeper permanent pools are needed for amphibians and reptiles whilst fishpools need to have a minimum depth of 2.5 m. A shallow fringing (but discontinuous) platform for emergent vegetation is desirable as it is not only ecologically valuable but will also enhance nutrient, metal and oil uptake as well as absorbing wave impacts and concealing unaesthetic changes in water level and litter.

Basin configuration

6.The use of indented, irregular shorelines can provide territorial shelter, seclusion and feeding grounds for a wider range of wildlife and waterfowl as well as helping to divide the reservoir surface into discrete zones for different recreational uses.

7.The underwater bank profile should have various gradients of 1:6 or shallower to allow the development of bands of emergent vegetation of varying widths. Both depth and shape can be modified by excavation, although this may necessitate the provision of protection to the reformed banks and bed. Rapid and cheap solutions include placing rubble or rip-rap to create an irregular marginal profile with spits and bays and to grade gently shelving underwater banks suitable for planting emergent species. Use of sub-soil filled sandbags on the edges of a water body can also provide a habitat for emergent plants whilst anchored willow logs can provide offshore nesting and roosting sanctuaries or can grow to form fringing willow in shallower water.

8.The inclusion of islands into the reservoir design will help serve as dividing walls, increasing flow paths and hydraulic residence times, encouraging mixing and eliminating the formation of 'dead' zones. In addition they can provide extremely valuable refuge sites for wildlife.

9.The use of a two-cell basin design achieves two objectives. A forebay (or diversion structure) will act as a sedimentation chamber and oil interceptor minimising 'first-flush' of pollutant discharges into the main pool and thus improve the water quality. It will also enhance plug flow and minimise short-circuiting. This latter phenomena, which exacerbates the disturbance of polluted bottom sediments, is a common occurrence in flood storage reservoirs and the smaller the basin in relation to the inflow volume, the more pronounced is the problem. Short-circuiting can be controlled by lengthening the flow path (minimum 3/4:1 length to width ratios are recommended), submerging the inflow below the permanent pool level or by the use of inflow baffles to diffuse the inflow.

TURBULENCE

10.In order to achieve water quality improvements in the main basin, one of the most important objectives must be to ensure an inlet design which will minimise the turbulence generated by the inflowing water which is the root cause of much of the observed poor pollutant performance of storage reservoirs. The inlet function should distribute the influent uniformly over the

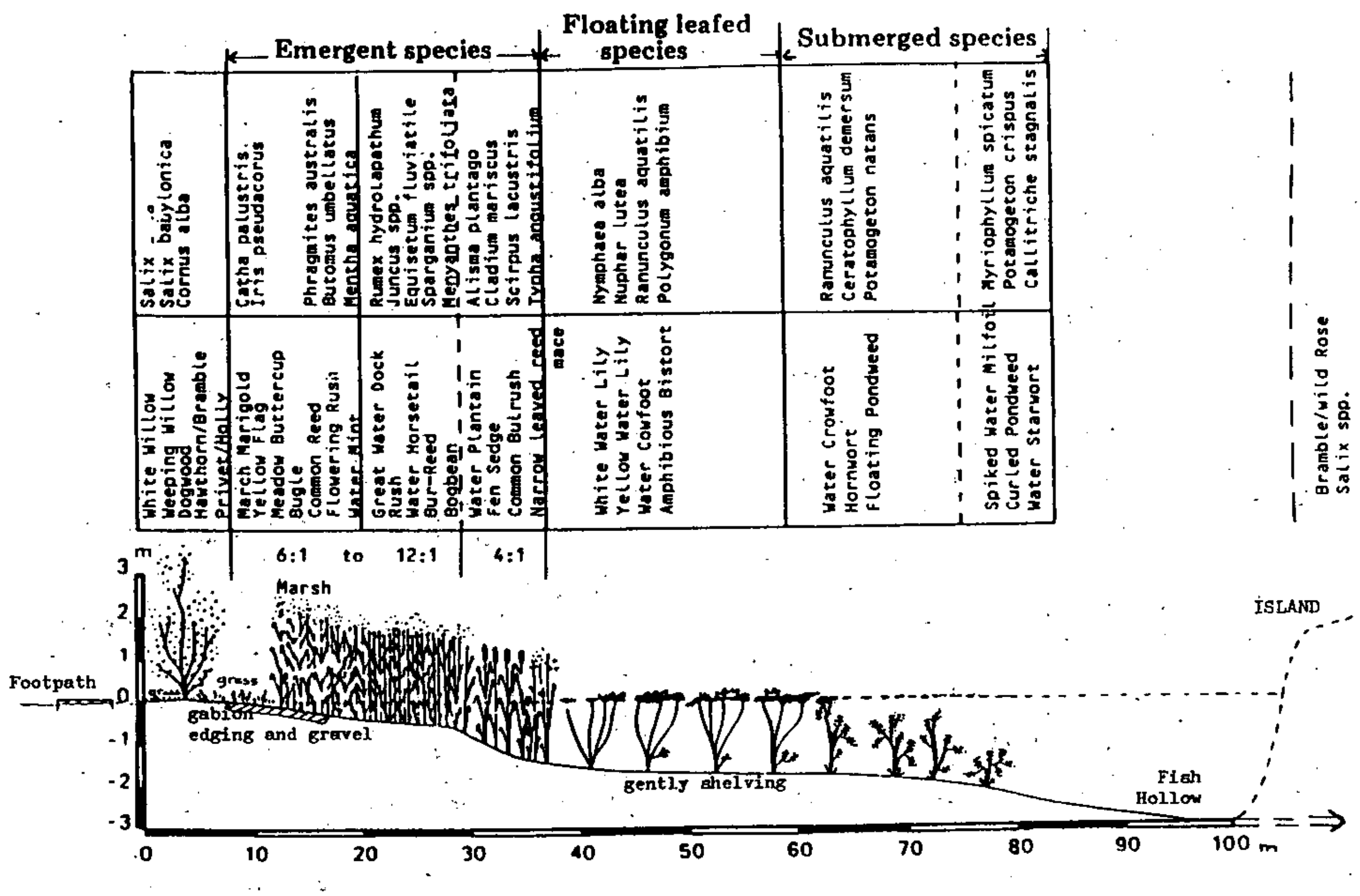

Figure 1: Vegetational Sequence Across Reservioı

cross-sectional area of the settling zone. Inlet baffle walls, submerged weirs or gradually expanding inlet openings to reduce flow velocity will all allow a more controlled release of water (at reduced energy) to the main basin where quiescent settling could take place. Release into the second basin cell at depth through a horizontally slotted baffle, for instance, would ensure maximum quiescence as well as encourage plug flow (ref. 6).

11.Use of wide, gently sloping grass swale channels as inlet conduits can reduce inflow energy due to bottom and side slope friction. Swales also offer the opportunity for biofiltration of solids and adsorption of soluble and toxic pollutants prior to entry into the flood basin (ref. 7).

12.The accumulation of sediment shoals near the inlet zone will also help dampen turbulence in the main basin and also support an emergent mactophyte reed marsh which will enhance biofiltration and biological uptake of pollutants. Figure 1 provides an idealised vegetational sequence for a flood storage reservoir based on a framework of dominant marginal, floating leafed and submerged macrophytic species, with emergent species such as Phragmites located adjacent to inlets. The sequence provides maximum hydraulic resistance, effective polysaprobic conditions as well as offering a range of nesting and insect habitats. Sediment bars and shoals will also be particularly attractive to wading birds and waterfowl.

13.Reductions in turbulence as well as enhanced retention times can be achieved through modifications in outlet design. Dual outflow structures for quantity and quality control have long been in use in the United States (ref. 8). Larger storms can be controlled through the use of high level outlets and overflow spillways, whilst the smaller, more frequent (and more polluted) storm events can be retained for a longer period of time through the use of short, small diameter outlets risers. A variety of outlet designs and their quality characteristics are discussed in references 4 and 9.

LANDSCAPING

14.Planting and landscaping of the adjacent land should be undertaken to create a natural environment as far as is possible. It is highly desirable to include such landscaping as part of the initial design with future management in mind, although considerable improvements and facilities for passive recreation and amenity can be introduced retrospectively (refs. 10, 11).

15.Well designed, landscaped vegetation zones around the storage basin can provide excellent and integrated urban wildlife habitats particularly for 'edge' species of songbirds and mammals. A minimum 300 to 350 m wide buffer strip is recommended for screening purposes, to overcome the so-called 'island' effect on species diversity and for general preservation of scenery. Figure 2 provides some general guidelines which can be used which include the use of vegetative screens for car parking and buffer zones to restrict access as well as gravel beaches, board walks and pond dipping platforms designed for disabled access.

16.Shoreline tree and shrub planting needs to be done with care as shading of marginal aquatic vegetation and related accumulation of excess organic materials can contribute to both nutrient and oxygen demand problems. Species that are conducive to passive recreation include dogwood, yellow stemmed willow and holly, blackthorn and privet mixes. Additional tree plantings could include alder, white and weeping willow with the surrounding meadow grass laid down with suitable slow growing cultivars (ref. 10).

17.The surface of embankments, spill banks and spillway channels which are subject to high velocities can utilise reinforced grass designs which can provide protection against velocities of 3 m/s for as long as nine hours (ref. 12). Proper management of such dense, well-knit grass swards is essential and this entails regular cutting, the application of fertilisers and weed control.

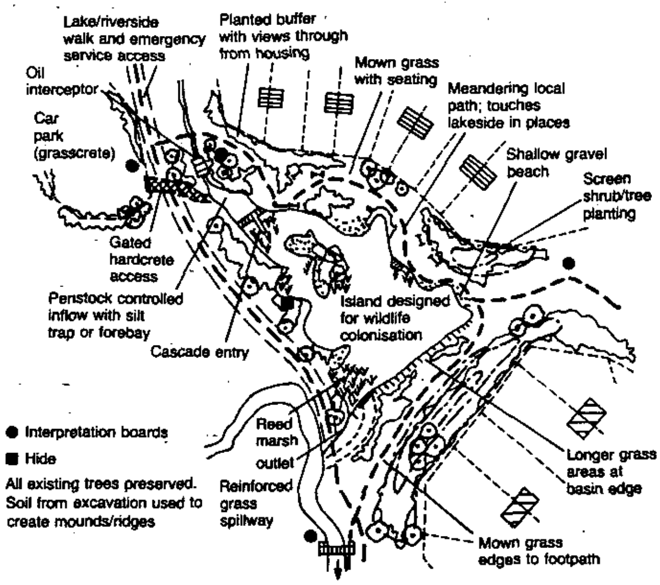

Figure 2: Idealised Environmental Layout for Flood Storage Basin

MAINTENANCE AND SAFETY

18.It is essential that the responsibilities for future maintenance and safety should be decided during the planning stage and formal arrangements agreed for regular inspection and maintenance. These should include considerations of amenity, ecology and water quality. A formal protocol for inspection, maintenance and servicing should be drawn-up, staff allocated and their duties and responsibilities confirmed in writing. Administrative and managerial procedures can be simplified by the routine use of standardised reporting forms which should be countersigned by senior staff. Operational information can then be placed on a database and accessed to check basin performance and operating costs. The information can also be used to identify problems that might be overcome in future reservoirs by design modifications. Information on costs would provide a rational basis for calculating commutation payments to be made when an authority takes over responsibilities from a developer.

19. Where a lessee develops on-site facilities for recreational and amenity use, they should be required to accept full maintenance and safety responsibility for both water area and the adjoining land. The responsibilities may therefore include land management, keeping the basin free of silt and weed, macrophyte control as well as providing safety equipment and controlling public usage. However as recreation, amenity and nature conservation are secondary uses, the lease should specifically exclude any powers relating to the control structure or operation of the flood storage basin.

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24. The use of close-range photogrammetry for reservoir embankment monitoring

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Photogrammetry is an established technique for the presentation of three dimensional imagery. Close-range photogrammetric techniques have been developed to measure structures and landforms. This paper details studies carried out to monitor reservoir embankment deformation and develop methods of presentation of data.

INTRODUCTION

1. Photogrammetry utilises photography in conjunction with conventional surveying techniques to produce plans, sections or digital data, and the opportunity to present these in a three dimensional model of the relevant imagery. Its major use has been in the preparation of topographical maps from aerial photography. More recently techniques have been developed to apply close-range photography to the measurement of structures and landforms by Cooper et al (ref.1) and Chandler et al (ref. 2).

2. The majority of earth embankment dams owned by North West Water (NWW) have survey stations installed on the crest and in some cases on the downstream face. Additional stations have been installed in areas of instability. These stations consist of a stainless steel pin set in a concrete block at least 500mm deep. These stations are regularly monitored for level and in some cases for alignment, using conventional geodetic instruments. This is effective in monitoring movement at discreet points but provides no information on the rest of the embankments.

3. Two reservoir sites were selected as being suitable to study the effectiveness of close-range photogrammetric techniques and to experiment with various methods for representing graphically any movements which had occurred. Spade Mill Nos. 1 & 2 are liable to rotational slips, shallow flow slides and sink holes in blockwork pitched internal slopes. Dean Clough Lower is a grass covered earth embankment which suffered from instability of the downstream face whilst under construction in the 1870's. The downstream face shows much evidence of slides although no major movement has occurred in recent years. The Engineering Photogrammetry Unit of City University was commissioned to carry out these studies.

4. The anticipated precision for measuring movement was +/- 5mm on the pitching at Spade Mill, and +/- 20mm on the grassed embankment at Dean Clough, depending on the vegetation.

PHOTOGRAMMETRY

Theory

5. Figure 1 illustrates the situation that exists when a camera photographs an object. The camera produces a central perspective projection of the object on the negative where the centre of the camera lens, O, is the perspective centre for the projection. A point A on the object is imaged at a on the negative and Object point B is imaged at b. The angle θ subtended at O by A and B is recreated inside the camera by θ', the angle subtended at O by a and b. Therefore the camera may be regarded as a form of theodolite which instantaneously records an infinite number of angles between an infinite number of points on the object photographed. In this way taking a photograph is a remarkably efficient way of recording information. The mathematics of photogrammetry, in effect, allow any desired angle to be recreated from measurements between two points on the photograph. By measuring the positions of a series of image points on a photograph the resulting series of angles produces what is known as a bundle of rays.

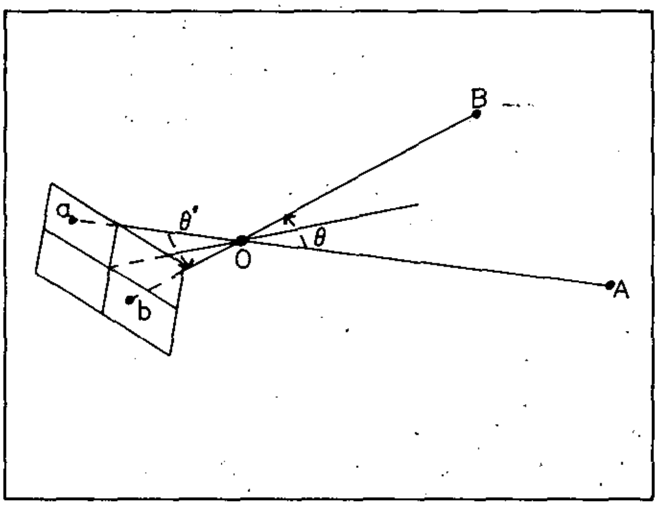

Fig. 1

6. By taking more than one photograph of an object the three dimensional coordinates of points on the object can be computed by the intersection of two or more bundles of rays. For most purposes two photographs, known as a stereopair, are sufficient.
7. Unless the camera is set up over a predetermined point and pointed in a known direction, a procedure which is very difficult to carry out precisely, it is normally necessary for the position and orientation of the camera to be computed from the measurement of images of points whose object point coordinates have been determined by some other method. These points are known as control points. This is known as a space resection and is similar in principle to the standard resection technique carried out with a theodolite, but is three dimensional.

Equipment

8. For photogrammetry two major items of equipment are required - a camera for taking the photographs and an instrument for measuring them.
9. When the first series of field visits was being planned, the most suitable camera available was a Carl Zeiss Jena UMK 10/1318 photogrammetric camera. This is a wide angle camera with a nominal focal length of 100mm and a format size of 130mm by 180mm. The photographs were to be recorded on glass plates for maximum image flatness and stability. By the time of the second series of visits a Carl Zeiss Jena UMK 30/1318 camera was available. This camera was fitted with a 300mm telephoto lens, with the same format size as the UMK 10/1318.
10. The photographs were measured on an Intergraph Intermap Analytic (IMA) photogrammetric colour graphics workstation. This system consists of a photogrammetric analytical plotter with computer controlled measuring stage plates interfaced with an Intergraph colour graphics workstation running Intergraph's Interactive Graphics Design System (IGDS) software. In this way, as information is measured from the stereopair it is immediately stored in a "design file", a computer model of 3-D data, which can then be manipulated by various graphics routines to produce plots, or can be used as input for other Intergraph software packages such as Digital Terrain Modelling (DTM).
11. Provided with the IMA were a number of specialist software packages. These included IMAN for carrying out space resection and other photogrammetric tasks, IMAPF for collecting three-dimensional spatial data in a design file and IMAPD for specialised data collection for input into the DTM package.
12. For the measurement of the control survey a Zeiss Oberkochen Elta 2 electronic tacheometer with automatic recording of data on a MEM 400 memory module was used along with a Carl Zeiss Jena Ni 007 precise level. On later surveys a Wild TC-1 Tachymat was also used.

RESERVOIRS

Spade Mill Nos. 1 & 2

13. Spade Mill reservoirs form two adjacent storage units situated near Longridge, Lancashire(Fig. 2.) No. 1 reservoir was originally constructed in 1862 by forming an earth embankment across the valley. Between 1905 & 1908 the reservoir was enlarged by raising the embankment and carrying out excavations in the basin. All the internal slopes were graded and pitched with sandstone blockwork. No. 2 reservoir was constructed between 1952 & 1959, primarily by excavation, with the internal slopes being graded and pitched with concrete blockwork. Both reservoirs have a maximum storage depth of approximately 10m.
14. 68 survey stations are installed on the crests of the embankments; 36 on No. 1 and 32 on No. 2. A control pillar at P provides the local datum.

Areas of Instability

15. Six areas of instability have been monitored (Fig. 2).
16. Area A is a series of depressions situated in the top third of this natural slope. The depressions are associated with flow between the reservoir and a surface water drain in a sandy lens in the glacial till, removing fine material.
17. Areas B, D & F are shallow slides probably taking place in softened fill or regraded natural ground. Two survey stations are situated within slips B & D.
18. Area C is a deep seated rotational slip the upper part of which is in full with the slip plane following the underlying sandstone rock surface. The rock is subject to artesian water pressures. 20 survey stations are situated within the slip.
19. Area E is a deep seated rotational slide in glacial till. 6 survey stations are situated within the slip.

Field Visits

20. Three field visits have been made to Spade Mill reservoirs. These have provided photography of each area at least twice to enable comparisons to be made.
21. 15th October 1986 - For this first visit Areas A and B in No. 1 reservoir and Areas C and D in No. 2 reservoir were recorded.
22. At this time only the 100mm focal length UMK camera was available. In order to obtain photography at a sufficiently large scale photographs of the sites were taken from a small dinghy. One pair of photographs were taken of each site, between 50 to 100 metres offshore, depending on the extent of each site. Additional photography of each site was taken from the water's edge, and, except Area C, from the back of a Land Rover above the wave wall.
23. To provide control for the photography, rotatable targets, set in gaps between the blockwork, were positioned throughout each site. The control survey and photography proceeded simultaneously.

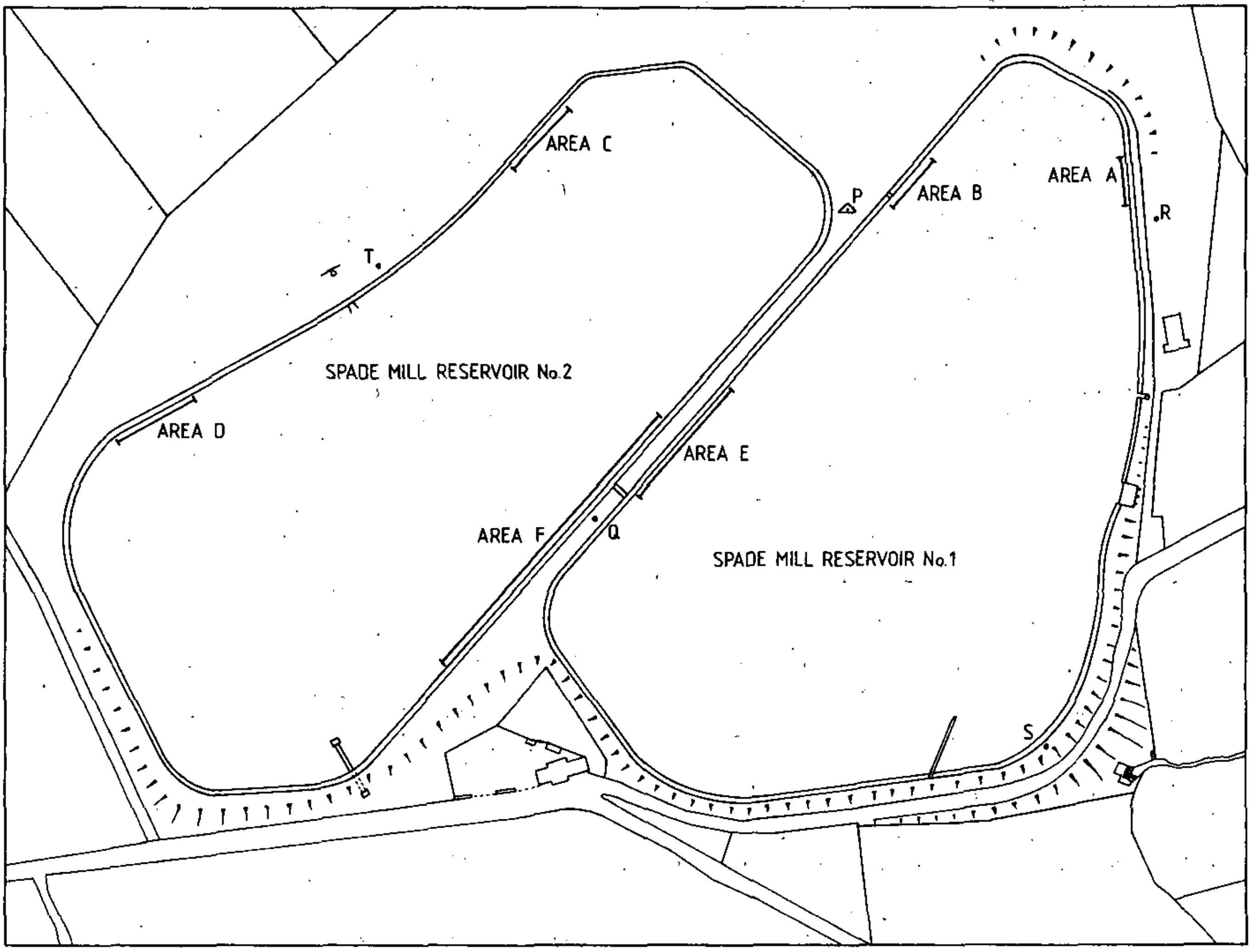

Fig 2. Spade Mill Reservoirs

24. Control targets on Areas, A, C and D, and part of B, were coordinated by bearing and distance measured with the Zeiss Elta 2 electronic tacheometer from pillar P. Targets below the wave wall in Area B and other check measurements were co-ordinated from temporary stations at Q and R. Distances between a sample of targets were checked using a steel tape.

25. 29th October 1987 - During this second visit Area E in No. 1 reservoir and Area F in No. 2 reservoir were recorded in addition to the four areas recorded on the first visit.

26. The 300mm focal length UMK camera was used. This enabled the photography to be taken from the opposite side of the reservoir. The higher viewing angle removed the need to photograph each site from above the wave wall at the top of the slope.

27. For the control survey fixed targets, larger than used on the first visit, were positioned around each site. Two additional stations, S and T, were positioned and the targets co-ordinated as before.

28. 30th November 1989 - On the third visit to Spade Mill only area E in No. 1 reservoir and Area F in No. 2 were photographed. The 300mm focal length UMK camera was used, and control targets were positioned and coordinated as before.

Dean Clough Lower

29. Dean Clough Lower reservoir is situated near Great Harwood, Lancashire. (Figure 3). It was constructed in the 1870's and 1880's by forming an earth embankment across Dean Brook. The dam is 350m long, with a maximum height of 22m. The downstream face has a slope of 3:1, with a berm just below half height. It had an early history of instability. The embankment was not constructed to the full height intended and the berm was added. Uneven areas near the top of the embankment may indicate some slipped zones, but there has been no evidence of any recent movement. The crest and downstream face have a grass turf grazed by sheep and reed patches are periodically mown. 43 survey stations in four rows are installed on the crest and downstream face of the embankment. Two control stations, "Stream" and "Track East", provide the local datum.

Field Visits

30. Two field visits have been made to Dean Clough Lower Reservoir. The central area above the berm which has indications of earlier instability, has been photographed twice to enable comparisons to be made.

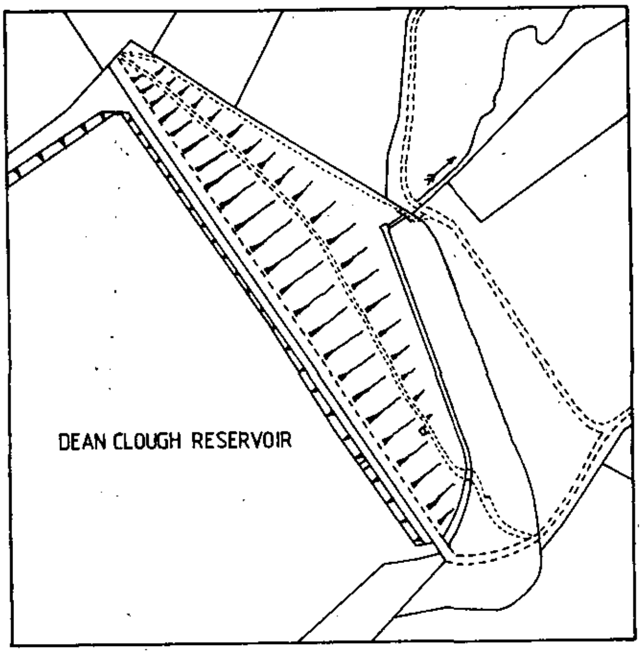

Fig. 3 Dean Clough Reservoir

31. 4th March 1987 - Only the 100mm focal length UMK camera was available.
A strip of photography was taken to provide cover along the dam face. Control was marked using large fixed targets. These were positioned in three rows; along the top, middle and bottom of the embankment. The targets were co-ordinated by bearing and distance from site control station Stream, and checked from Track East.
32. 17th March 1988 - The second site visit was similar to the first. A strip of photography of the dam was taken with the 100mm camera as before. Additionally, oblique stereopairs were taken with the 300mm camera from further back on the valley sides at approximately 45 degrees to the embankment.

ANALYSIS

Computation of Control Surveys

33. After each site visit, the recorded field survey observations were downloaded from the data recording module of the Elta 2 into a microcomputer. The data was run through a reformatting file to produce an input file for the Three Dimensional Variation of Co-ordinates (TDVC) program.
34. TDVC is an interactive computer program for least squares estimation of spatial co-ordinates. Field survey measurements are input. The output gives estimated co-ordinates with their standard errors and 'adjusted' observations with their discrepancies. Observations with large discrepancies can be removed and the adjustment recomputed.
35. For all the Spade Mill surveys the standard errors of the estimated co-ordinates of the control targets were of the order of 5 to 7mm in X, Y and Z axes. No observations were removed from the computation.

Measurement of Photography

36. The measurement of photography of Spade Mill was delayed until two sets of photographs had been obtained. This was so that corresponding photography of each area at both epochs could be measured in quick succession.
37. Each pair of photographs being analysed were first orientated by space resection using the Simultaneous Orientation section of the Intergraph IMAN nucleus package. Where plan detail, such as header stones, was to be measured for monitoring purposes both epochs of photography were measured using the IMAPF Map Feature Coding applications program to create an Intergraph design file of this detail. Where other methods of presentation were to be used then only one epoch of each site was measured using IAMPF to provide a base plot for superimposing additional information. In these instances both epochs of photography were then measured using the IMAPD DTM Data Collection applications package.

PRESENTATION OF RESULTS

Spade Mill Nos. 1 & 2

38. A number of different methods of presentation were used for depicting the results of the measurement of the Spade Mill photography.
39. Plots of Plan Detail - This method was used for the analysis of Area C in Reservoir No. 2, and required data from both epochs. It demonstrated that Area C had suffered significant slumping of the lining.
40. Significant physical features - the wave wall, water's edge, edge of pitching and header stones were measured on each epoch of photography to produce colour coded graphics design files. These were then plotted at a scale of 1:500. One plot could then be laid over the other to check for any changes in the plotted features. One of the main drawbacks of this method of presentation is that at 1:500 only changes larger than 150mm in plan could be detected. Also, no change in height could easily be depicted.
41. However, this method of presentation could prove useful if a graphics screen display was used instead of a paper or film plot. In this way the two design files could be referenced together. It would then be possible to zoom in to specific areas, to check for changes smaller than 150mm. Also by viewing the three-dimensional data from the front or side, changes in level could also be seen.
42. Digital Terrain Models (DTMs) - One data collection package used in the study produced three-dimensional graphics files which were used as input into the DTM processing package. During analysis of the photographs the normal procedure was to collect three-dimensional coordinates across the site in a regular grid. A suitable grid spacing, 5 or 10 metres, was selected and a point measured at a corner of the site. The IMAPD software then drove the stage plates of the IMA so that the measuring mark had moved the equivalent of the selected grid spacing across the site. The operator then placed the measuring mark on the surface of the site and recorded the Z coordinate

along with the X and Y coordinates the IMA had driven to. This process was repeated until coverage of the site was obtained. Any significant changes in the surface of the site which fell between the regular grid were then digitised either as break lines or as spot heights.

43. The DTM processing package was then used to produce a number of different outputs:

44. Visualisation - One output was a visualisation of the surface of a site in the form of a rectangular grid of lines. When viewed as an isometric projection, all areas of slumping could be easily seen. This method was tried on Reservoir No. 2, Area C. Although this method provides a good indication of irregularities in the surface it is not suitable for depicting changes between epochs as one grid overlaid on another produces a very confusing image.

45. Contour Plots - The DTM can be used to calculate contours. This was also used to depict changes in Area C. A contour plot was generated for each epoch and plotted at a scale of 1:500. When one plot was laid over the other some indication of change in shape could be perceived. This method suffers from the same disadvantages as plots of plan detail.

46. The DTM was also used to produce contour plots of difference by subtracting the generated surface of epoch 2 from the generated surface of epoch 1 to produce a surface of change, i.e. peaks were where the surface had risen and troughs were where the surface had dropped. A contour plot was then generated and plotted at a scale of 1:500. Area C was again used for this study. The advantage of this method over the previous methods was that it immediately showed areas where change in shape of the surface had occurred.

47. A slightly different method was used for the depiction of Area A in Reservoir No. 1 where there are a number of small areas of settlement. It was decided to try to depict these sink holes and any changes relative to the general plane of slope, using a contour plot based on a rotated plane.

48. A DTM was measured for each epoch. The two resulting design files were rotated into the mean plane of the slope. A contour plot was then generated for each epoch as well as a contour plot of difference, showing movements relative to the surface plane.

49. The major problem on this site was that the large rectangular stone blocks and the relatively small contour interval (100mm) produced a rather jumbled series of plots which were difficult to interpret. A soil slope or small blocks would follow settlement more closely giving a better representation of movement.

50. Vectors - Although the plots of plan detail gave some indication of physical movement of particular features, all the other representations described above could only indicate changes in shape of the surfaces of the sites. In an attempt to represent the movement of individual points it was decided to measure the same points of detail - in this case corners of stone blocks - at both epochs. This was done by studying epoch 1 photography in the IMA and measuring particular corners of stones. Epoch 2 photography was then mounted in the IMA and the software drove the measuring mark to the previously measured coordinates. This greatly helped the identification of the selected block corners on the second set of photographs. The operator then placed the measuring cursor on the new position of the corner and recorded the coordinates. A program then subtracted epoch 1 coordinates from epoch 2 coordinates and generated a graphics design file of the required vectors.

51. One problem to be overcome was how to depict a three-dimensional movement on a two-dimensional plot. It was decided to show the X-Y movement by a line and the Z displacement by a circle at the epoch 2 end of the line. The magnitude of the Z displacement was represented by the radius of the circle and the direction of the displacement by the colour or representation of the circle.

52. This method of representation was used for Areas B, C and D.

Dean Clough Lower

53. For the Dean Clough photography it was decided to carry out a two-stage measurement process. The first stage was to determine the areas on the dam where movement had taken place and roughly indicate the magnitude of the movement. The second stage, if required, would be to measure, in more detail, the areas where movement had occurred. To date only the first stage has been undertaken.

54. For the first stage the epoch 1 photography was placed in the IMA and a coarse DTM collected for a spacing of 10 metres. The DTM collected from epoch 1 was then used to back-drive the IMA with the epoch 2 photography mounted. If the measuring cursor appeared to lie on the surface of the dam the operator instructed the IMA to drive to the next point. If the cursor appeared to lie off the surface the operator moved the mark down onto the new position of the surface, noted the change in the Z coordinate reading and annotated this change on an enlarged photograph of the dam. In this way areas of localised movement were very rapidly identified.

55. Settlements of between 5 and 12mm were identified in specific areas which coincide with those considered susceptible to movement.

CONCLUSIONS

56. The study has shown that photogrammetry is a very powerful tool for monitoring localised movements on reservoir embankments. It is a very efficient data gathering technique. Up to six separate sites at Spade Mill were photographed and control surveyed in a single day.

57. The measurement of the photography can also be carried out efficiently, particularly with an analytical plotter. The ability to backdrive the plotter using the results of a previous set of measurements enables selected points to be rapidly identified, or, in the case of Dean Clough, to identify areas where localised movement had occurred.

58. Of the photography taken at Spade Mill, the second and third sets, taken with the 300mm UMK, proved more successful than the first set. The higher viewpoint, from the opposite bank as opposed to a boat, made measurement easier. In addition, vegetation covering sections of Area C at apoch 1 obscured large areas.
59. The main purpose of the investigations was to develop systems to enable reservoir slope movements to be monitored and quantified. Two-dimensional representation of three-dimensional movement, at an economical price, was seen as a central requirement.
60. Plans and sections are readily produced. These give a good representation of the plan or sectional profile, but at a reasonable scale small changes are difficult to detect. Depiction is in two dimensions.
61. Digital Terrain Models are very powerful for providing input data for a wide range of applications. However, if produced on a fine grid spacing suitable for quantifying local movement they are relatively expensive to produce. This is justified where the DTM is to be utilised for a graphics screen display or further modelling or measurement, but is not cost effective for monitoring of movement. The DTM results can be viewed as an isometric projection or utilised to produce contour plots. All methods have draw-backs in trying to produce a clear three-dimensional representation of movement.
62. Vector plots give a visual indication of relative movement superimposed on a plan. It is also possible to scale dimensions to quantify movement in three dimensions. This form of presentation meets the study objectives of producing a cost effective visualisation of three-dimensional movement in two dimensions.
63. On Dean Clough a simplified first stage analysis has been utilised as a low cost indication of whether any movement has taken place. Detailed analysis of local areas can then be carried out if required.
64. In addition to the monitoring of embankments with known or suspected movements the technique could be utilised to produce a reference photography/control data set against which suspected movement in the future can readily be quantified.

ACKNOWLEDGEMENTS
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25. Accommodating rare floods over embankments and steep reinforced channels

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SYNOPSIS
Based on research and case history data available from the United States (US) and the United Kingdom (UK) engineers can now feel confident in the design of overflow protection for low embankment dams. The introduction of articulated concrete block revetments, originally designed for coastal protection in wave environments, has produced a viable option for application to steep-slope, high-velocity flow conditions where, if left unprotected, the channel banks and foundation material would be subject to damaging and potentially catastrophic erosion.

INTRODUCTION

1. With the growing concern about dam safety throughout the world and increasing awareness of the hydrologic inadequacy of many older dams, civil engineers and dam designers are now looking to innovative alternative designs in modifying these dams. The traditional approach for accommodating design flows at embankment dams is to design spillway and outlet structures with sufficient capacity to avoid overtopping of the embankment. Improvements in the collection of historical flood data has resulted in significant increases in the predicted probable maximum flood (PMF). Therefore, many older dams are now considered unsafe due to inadequate spillway capacity. Conventional modifications include increasing the spillway size and/or raising the embankment. However, in many cases, these have shown to be costly or impractical.

2. Since 1983, research in the US and the UK has identified several innovative alternative designs to the more costly conventional modifications. These alternative designs provide methods for protecting the steeply sloped erodible embankment faces to achieve a high discharge capacity by allowing the entire crest, or significant portions thereof, to be overtopped. Traditionally, cast-in-place reinforced concrete would be used to achieve the desired level of performance and stability; however, more recently, roller-compacted concrete (RCC) has been used on several embankment dams in the US at a significant cost savings. Also, the introduction of articulated concrete block revetments, originally designed for coastal protection in wave environments, has produced another viable option for application to steep-slope, high-velocity flow conditions where, if left unprotected, the channel banks and foundation material would be subject to damaging and potentially catastrophic erosion.

3. Earlier performance testing of articulated concrete block systems at Jackhouse Dam in the UK (refs. 1-2) stimulated recent testing to these systems under high-velocity, steep-slope flow conditions (refs. 3-4). The research revealed the characteristics of hydraulic stability and nature and magnitude of destabilizing processes associated with these systems under bare (unvegetated) conditions. Both cabled and non-cabled concrete block systems were tested. Earlier research (1986-1987) of the articulated concrete block systems required installation be in strict compliance with the manufacturers specifications. However, after some failures, modifications to the installation procedures were deemed necessary and tests performed later in 1988 showed improvement. This paper describes the hydraulic testing program, results, and conclusions derived from approximately four and one-half years of study. The paper also briefly discusses current in-house research by the Bureau of Reclamation (USBR) to evaluate the capability of RCC and wedge-shaped blocks to protect the downstream face of an earth and rockfill dam, measuring 48.5 m high, during overtopping flows. Modifications made to other embankment dams to allow overtopping are also discussed.

TESTING PROGRAM

4. In early 1986, a large-scale flume and recirculating water supply system were constructed by Simons, Li and Associates, Inc. (SLA) to examine the performance of embankment protection systems under steep-slope, high-velocity flows. The flume, was 3.35 m high, 1.2 m wide, and 27.4 m long. An erodible embankment 1.8 m high with a crest surface of 6.1 m and downstream slopes ranging from 2H:1V to 4H:1V was placed on the flume, Fig. 1. Various protection systems were installed on the embankment surface and subjected to overtopping flows of up to 2.8 m^3/s. This flow rate yielded 1.2 m of overtopping head, with maximum velocities of approximately 5.2 to 6.7 m/s, depending on system roughness, measured near the downstream toe. This testing program was unique from earlier tests in the US and UK in that the embankment tested was a highly erodible silty sand (SM) which tested the effectiveness of the protection systems rather than the erosion resistance of the embankment soils.

5. Initial performance studies of various protection treatments were sponsored by the U.S. Federal Highway Administration (FHWA) and USBR. In general, the high velocities and large tractive forces developed on the downstream slope of the embankment caused deformation and/or failure of meshes, mats, and wire-enclosed riprap. This type of failure was characteristic of shear-stress dominated deformation at overtopping heads typically in the 0.3 to 0.6 m range, with measured shear stresses ranging from approximately 190 to 720 Pa. Treatments which successfully resisted the hydraulic stresses at full discharge included soil cement, which for purposes of evaluation is considered the same as RCC, placed in 10-cm-thick steps, and several articulated concrete block revetment systems.

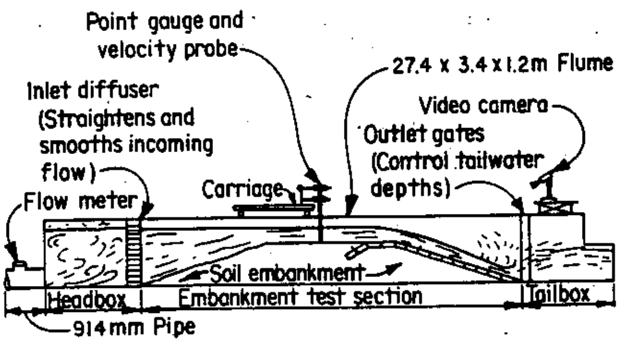

Fig. 1. Profile of testing facility

6. For the final phase of the SLA studies, the FHWA and USBR were joined by the Soil Conservation Service (SCS) and the Tennessee Valley Authority (TVA) to extend the hydraulic testing program to focus directly on the performance of the articulated block systems. Five systems were investigated: three cabled systems (Armorflex, Petraflex, and Dycel) and two noncabled systems [concrete construction blocks and wedge-shaped blocks of Soviet design (ref. 5)]. Fig. 2 provides information relating to the geometric configuration, weight, and dimensions of each of the five systems.

7. The Dycel system, with the largest, area-to-thickness ratio, failed during 0.3-m overtopping head by allowing excessive underflow to accumulate beneath the system. The other four systems performed successfully during overtopping heads as high as 1.2 m.

HYDRODYNAMIC FORCES AND REVETMENT BLOCK STABILITY

8. Hydraulic Forces - An individual block surrounded by a matrix of identical blocks is subjected to the forces of lift and drag under the action of flowing water. The lift force acts in a direction normal to the plane of the bed, and is typically comprised of the buoyant force

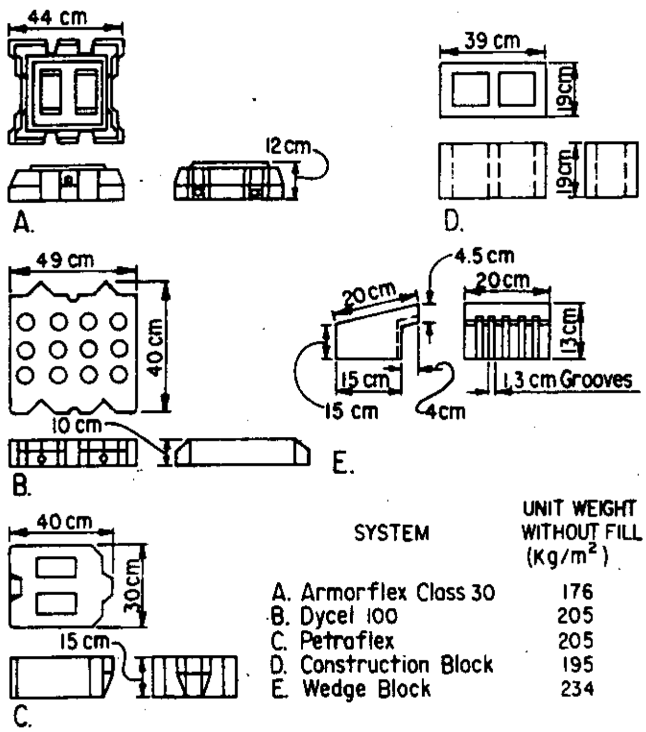

Fig. 2. Sketches of five types of concrete blocks tested. (NOT TO SCALE)

and differential pressure across the block due to local accelerations. Lift forces can be substantially increased due to excessive seepage pressures beneath the block, and by flow separation which causes a negative pressure to occur on the upper surface of the block. The latter commonly occurs at sharp transitions from a mild bed slope to a steeper one i.e., at the transition point between dam crest and the downstream slope (ref. 6).

9. The USBR recently conducted a study to measure the pressure profile along a horizontal surface and over a sharp transition to a steep slope. This study revealed a large reduction in surface pressure in the vicinity of the change in slope. This pressure change is present over a very short distance in the direction of flow. Apparently the curvilinear flow over the intersection returns to the original flow profile in this short distance. Fig. 3 shows typical piezometric pressure profiles for three different changes in slope at an overtopping head equal to 0.66 m.

Fig. 3

10. The drag force acts in the direction of flow, and is comprised of frictional drag and form drag. Form drag, in particular, can lead to the creation of forces large enough to initiate block movement (rotation) where the block in question presents a frontal profile which is subject to direct impact by the flow (ref. 7). This is possible in instances of irregular subgrade preparation or poor installation where an individual block protrudes vertically above its adjacent neighbors. Cabled block systems have the ability to maintain the amount of projecting frontal area at a practical minimum when cable runs are oriented in the direction of flow, with the maximum height of projection limited to the difference between the diameters of the cable and the cable tunnel (Fig. 4a). Wedge-shaped block systems negate this effect entirely by providing a thin upstream cross section and a thicker downstream one. All upstream edges are therefore effectively "shielded" from direct impact (Fig. 4b). Drainage slots are also provided at the downstream edge, thereby relieving seepage and uplift pressures on the underside of the revetment and enhancing the intimate contact between the blocks and the subgrade (ref. 8).

Definition of Failure

11. Loss of "intimate contact" between a block, or group of blocks, and the subgrade which they are to protect has been identified as the primary indicator of incipient failure (refs. 2-3). Given the nature of revetment mattress installation in typical steep-slope applications, failure due to slipping or sliding of the revetment matrix along the plane of the bed is remote and has never been observed under controlled test conditions. This includes steeply sloped embankments where mechanical or vegetative shear

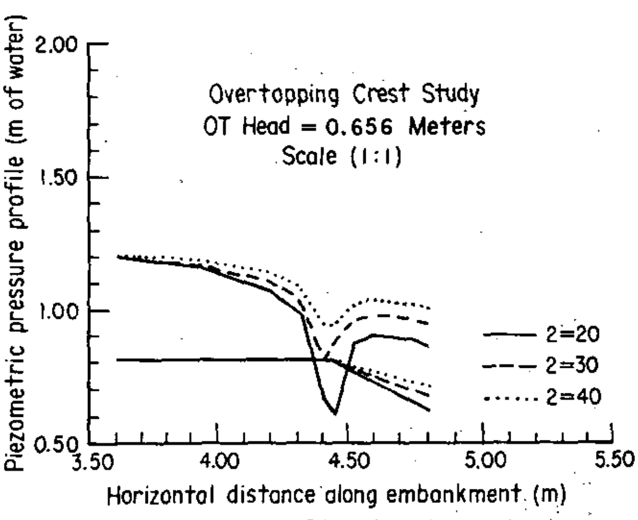

Fig. 3. Pressure profile at a change in slope.

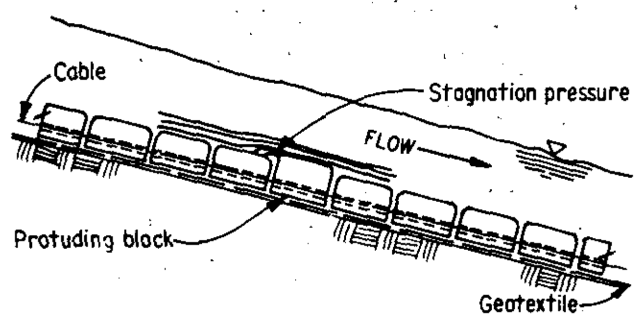

Fig. 4a. Typical profile of cabled revetment system cables in direction of flow.

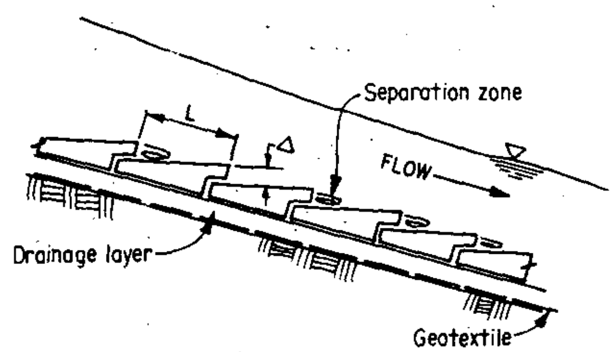

Fig. 4b. Typical profile of wedge block system.

restraint was not provided. Apparently the frictional resistance developed between the blocks, geotextile and/or granular filter, and subgrade soil is usually sufficient to prevent sliding occurrence. The loss of contact, therefore, is the result of overturning forces levered about the downstream edge, or about the downstream corner point when the block is located on the sideslope of an already steeply sloped channel. However, physical dislodgement or even measurable movement does not need to occur in order for the undesirable seepage flow to initiate and progress within the subblock environment, causing erosion of the embankment.

12. Therefore, the definition of "failure" of an articulated block revetment system is when overturning moments are exactly balanced by resisting moments. The dominance of overturning moments denote the condition where the ingress of flow beneath the system is imminent, and loss of contact is initiated. This definition of failure appears reasonably conservative in that the additional shear and uplift restraint provided by vegetative or mechanical anchorage systems is not depended upon by the designer. Likewise, any restraining force which can be attributed to cables should not be considered, because the mobilization of tension forces in cables can only come into play once finite rotation has occurred, by which time the system has already been defined as having "failed."

Stability Analysis

13. Both lift and drag on a block produce overturning moments proportional to their magnitude and to the length of the moment arms through which they act. The resistance to overturning is provided by the submerged weight of the block acting through the center of gravity and its moment arm. Hydraulic stability is thus dependent on the hydraulic conditions of flow and the size, weight, and geometric characteristics of the block. The analytical method for determining revetment stability by way of the "factor of safety" method was developed originally by Simons and Senturk (ref. 9) in their derivation of a methodology for evaluating the stability of rock riprap in open-channel flow. In their method, the critical shear stress at which particle motion is initiated was determined by the Shields relationship. In the case of articulated block revetment systems, the critical shear stress is determined through controlled hydraulic testing and measurement.

14. The factor-of-safety procedure can be extended to blocks of different dimensions and weights, provided they are geometrically similar to the system for which the critical shear was previously determined through laboratory testing. The blocks must be of the same "family" in terms of method of interlock, profile configuration, and characteristics of boundary roughness and interaction with the flow field. Given this basic similarity, the weight and dimensions of the block in question can be compared with those of a tested block to determine the critical shear stress and a force balance approach.

MODIFICATIONS TO ALLOW OVERTOPPING AT EXISTING DAM SITES

15. During the 1980s a number of cost-effective modifications have been made to existing embankment dams and grasslined waterways to prevent overflow erosion. Modifications such as the use of gabions, grouted riprap and even well-maintained grass lining have proven to be effective under low overtopping flows (ref. 6). The use of simple concrete construction blocks has shown to be effective in preventing erosion of SCS chute spillways.

16. The use of RCC has been demonstrated to be a cost-effective alternative on several embankment dams in the US by providing erosion protection during overtopping heads in excess of 0.6 m deep. Spring Creek Dam, located in Colorado, has a 15.2-m-high embankment and was modified in 1986 to allow overtopping by the stairstep placement of RCC on the downstream face.

17. The use of articulated concrete blocks to prevent overtopping erosion was first used in the UK (ref. 1), and the wedge-shaped blocks were developed and used in the USSR (ref. 5). These applications allowed for the articulated concrete mats to be placed on the downstream face of an embankment or steep waterway. With the use of soil anchors and grass vegetation these applications have been effective in preventing erosion.

18. Articulated concrete block mats are also currently being used to modify three embankment dams in the Blue Ridge Parkway located in the

Eastern United States (ref. 10). These dams range in height from 8.4 to 12.0 m at their maximum section. The modifications are designed to prevent breaching and erosion from overtopping flows 1.2 m deep.

RECENT DEVELOPMENTS AND FUTURE DIRECTIONS

19. With the sufficient research and case history data currently available engineers can now feel confident in preparing designs for overflow protection for low embankment dams. However, USBR now faces the challenge of protecting much larger dams during overtopping flows. A. R. Bowman Dam is a 48.5-m-high earth and rockfill embankment located in Central Oregon which is projected to be overtopped by flow depths of up to 6.3 m during the PMF event.

20. A research program has been initiated by USBR, and has the following objectives regarding dam overtopping: 1) design criteria utilizing RCC technology for new dams and overlays for rehabilitating existing embankment dams that comply with Safety of Dams criteria, 2) to determine optimal step geometry as a function of hydraulic forces and energy dissipation, 3) to develop step geometry for embankment dams that uses hydraulic forces to enhance subsystem pressure relief during operation (ref. 11).

21. In addition to laboratory studies, field studies will be made to test and compare at near-prototype conditions the performance of RCC, Russian wedge-shaped blocks, and riprap embankment protection systems. These data will evaluate flow aeration, dynamic pressures and embankment drainage as important variables which influence protection method stability under large overtopping flow conditions.

22. Two adjustable slope test facilities have been built to develop step geometry. One is to investigate embankment overlays on slopes of 2:1, 3:1, and 4:1. The other will be used for concrete dam slopes of 0.6:1 and 0.8:1. These flumes are each 0.45 m wide by 0.75 m high and have a vertical fall of about 4.5 m. The maximum unit discharge for each flume is 1.67 $(m^3/s)/m$. The sidewalls of the flumes are formed in clear plastic. A rail mounted instrument cart is provided along each flume for laser velocimetry, air content measurement, other instrumentation, and photography.

23. These indoor facilities will be used to optimize the spillway step geometry for relief of uplift pressures under RCC overlays on embankment dams by venting through the overlay to low pressure zones of the steps and for increasing of energy dissipation to minimize required toe protection. Pressures, velocity profiles, flow depths and hydraulic jump characteristics will be measured.

24. Early results of this research have confirmed a previous hypothesis that the velocity attained along the downstream slope is directly proportional to the depth of overflow across the embankment crest rather than the length of chute or slope. Once uniform flow is reached the velocity remains constant.

25. A new prototype outdoor test facility will utilize a 2:1 slope, have an approximate 15 m vertical drop, be 1.5 m wide by 1.5 m high, and have a unit discharge of 4.65 $(m^3/s)/m$. An existing prototype chute or a university test facility will be used for a series of near prototype tests to evaluate the effects of aeration, dynamic pressures, embankment drainage, and natural freeze-thaw phenomena on protective system stability.

SUMMARY AND CONCLUSIONS

26. Based on prototype experience and/or large-scale testing of many erosion-protection systems, those showing sufficient stability to perform reliably under steeply sloped high-velocity flow conditions appear to be limited to (1) traditional cast-in-place reinforced concrete, (2) roller-compacted concrete (or its lower-strength alternative, soil cement), and (3) selected articulated concrete block revetment systems.

27. The articulated concrete block systems may be the most cost effective alternative in many typical low head project settings, subject to availability and proximity of materials and equipment.

28. The proper selection of block type, weight, and dimensions are critical to design performance. These dimensions can be determined by a factor-of-safety method of analysis, provided that initial determinations of critical shear stress are performed under controlled conditions as described in this paper. Also installation procedures should conform to site specific conditions. The block manufacturers' specifications have not always been appropriate.

29. The RCC and Russian wedge-shaped blocks appear to have application for protecting high embankment dams from overtopping flows.

30. Recent research verifies that terminal velocities are directly proportional to the depth of overflow rather than the length of the slope.

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26. Deformation of Ramsden dam during reservoir drawdown and refilling

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Ramsden dam is an earthfill dam some 25m high with a puddle clay core. It is estimated that the crest has settled more than 1m since construction in 1883. Measurements have shown that long term settlement is largely due to drawdown of the reservoir. The net crest settlement due to a cycle of drawdown and impounding was affected critically by the magnitude of the drawdown. Settlements occurred throughout the full depth of the core.

INTRODUCTION

1. Ramsden dam was constructed between 1879 and 1883 for Batley Corporation Waterworks and the engineer was G H Hill. The reservoir is one of four in the Holme valley in West Yorkshire supplying water to Dewsbury. The maximum height of the dam is about 25m and the length is just over 120m. The upstream slope is 1 in 3 and the downstream slope is approximately 1 in 2. The lower part of Ramsden dam is submerged by Brownhill reservoir. The original construction drawings indicate that selected fill was placed either side of the core. The puddle core is founded on a concrete filled cut-off trench that has a maximum depth of 22m in the centre of the dam. The clay core is 3m wide at the crest and increases in width with depth with both faces having batters of 12 in 1.

2. Ramsden dam shows obvious signs that considerable settlement has occurred. It is estimated from records that there has been more than 1m of settlement since construction in 1883 (ref.1). Since 1977 precise surveying of the level and horizontal alignment of stations on the crest close to the wave wall has shown that the continuing and varying rate of settlement and downstream movement was related to reservoir fluctuations (ref. 2). The maximum settlement has occurred at the central section of the dam where it is deepest. The average rate of settlement between 1977 and 1985 was approximately 8mm per year and the average rate of horizontal downstream movement was 3mm per year.

3. It was originally intended to construct a new wave wall on Ramsden dam in 1987, but this was deferred following consideration of the detailed design with the panel engineer. Due allowance could have been made for the present rate of settlement, but uncertainty over the cause of the settlement raised the possibility that adding a substantial wave wall could affect local stability. It was decided to delay construction pending an investigation in which the effects of reservoir fluctuation on embankment deformations were monitored. Hoyle (ref. 3) suggested in 1975 that those engaged in research would find this subject worthy of further investigation.

INSTRUMENTATION

4. Figures 1 and 2 show the location of some of the instruments installed at Ramsden dam. In 1987, surface surveying stations were installed on the downstream slope, the crest and the upper part of the upstream slope. Settlement and horizontal upstream/downstream movements were measured relative to two survey pillars on the valley sides. Also in 1987, magnet settlement gauges and inclinometers were installed in the core to measure sub-surface movements. Precise levelling was continued on levelling points installed in 1977 along the crest close to the wave wall.

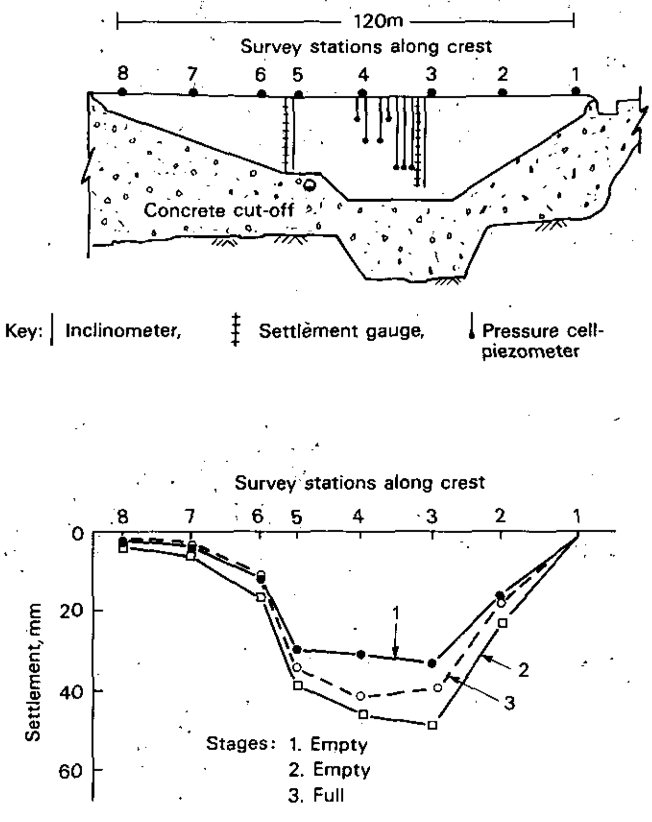

Fig. 1. Longitudinal section through core and settlement along the crest

5. During earlier investigations (ref. 1) to assess the susceptibility of the core to hydraulic fracture and to examine the effectiveness of the downstream fill to act as a filter, push-in earth pressures cells and pneumatic piezometers were installed in the core, and standpipe piezometers were installed in the core and the upstream and downstream fill. The shear strength parameters of the downstream fill were also measured.

OBSERVATIONS DURING THE 1988 AND 1989 DRAWDOWNS

6. Detailed observations of movements are presented for the reservoir being emptied completely during 1988 when repair work was carried out to the draw-off works and for the controlled drawdown to 6m below TWL in 1989. Six stages are identified in describing the results (see Table 1). They correspond to periods of time and various states of Ramsden reservoir. The dates for the beginning and end of the stages correspond to when observations were taken. During Stage 2, the "empty" state, the reservoir was partially refilled, however Stage 2 can still be regarded as a period when any movements were due largely to time dependent effects. Because of the silt and the upstream slope of the valley bottom, the empty state corresponds to a level approximately 18m below the crest (16.6m below TWL).

Table 1. Stages during investigation

Stage	Dates	State of Ramsden reservoir
Stage 1,	24/6/88 - 6/10/88	Drawn down from full to empty
Stage 2,	6/10/88 - 24/11/88	"Empty"
Stage 3,	24/11/88 - 5/4/89	Impounding
Stage 4,	5/4/89 - 28/7/89	Full
Stage 5,	28/7/89 - 12/10/89	Drawn down to 6.0m below TWL
Stage 6,	12/10/89 - 1/2/90	Impounding

Surface movements

7. Except for observations on the upstream slope, all movements presented are relative to the beginning of May 1988 when both reservoirs were full. Little movement was measured from when the instruments were installed in 1987 to May 1988 with the reservoir remaining full. Surveying stations could not be installed on the upstream slope until the reservoir had been drawn down slightly. Figure 1 shows the longitudinal section and the crest movements at the end of various stages and Fig. 2 shows surface movements on the crest, and upstream and downstream slopes at the deepest cross section at the end of the various stages. For clarity the movements for stages 4,5 and 6 are only shown for the crest survey station. Figure 3 shows the level of Ramsden reservoir and vertical and horizontal movement time plots for a selection of the survey stations.

8. Figure 2 shows that settlement and upstream horizontal movement due to drawdown (Stage 1) were largely confined to the crest and the upstream slope. The largest settlements occurred in the middle of the crest, above the centre of the clay core. Figure 1b shows that the magnitude of crest settlement was related to the height of the embankment. Some movement of the downstream survey stations close to the crest occurred (see Fig 1), but the magnitude of the movement decreased with distance from the crest. The settlement component of the crest movement was very much larger than the horizontal upstream movement. On the downstream slope, the magnitude of the settlement component reduced so rapidly with distance from the crest, that the horizontal component of movement was greater at stations B,C and D. At stations E, F and G, the small settlements up to 6mm may be due to the drawdown of Brownhill reservoir. The magnitude of the movements on the upstream slope will have been greater than that shown in Fig. 2 since measurements could not be started until the reservoir was drawn down by 2m.

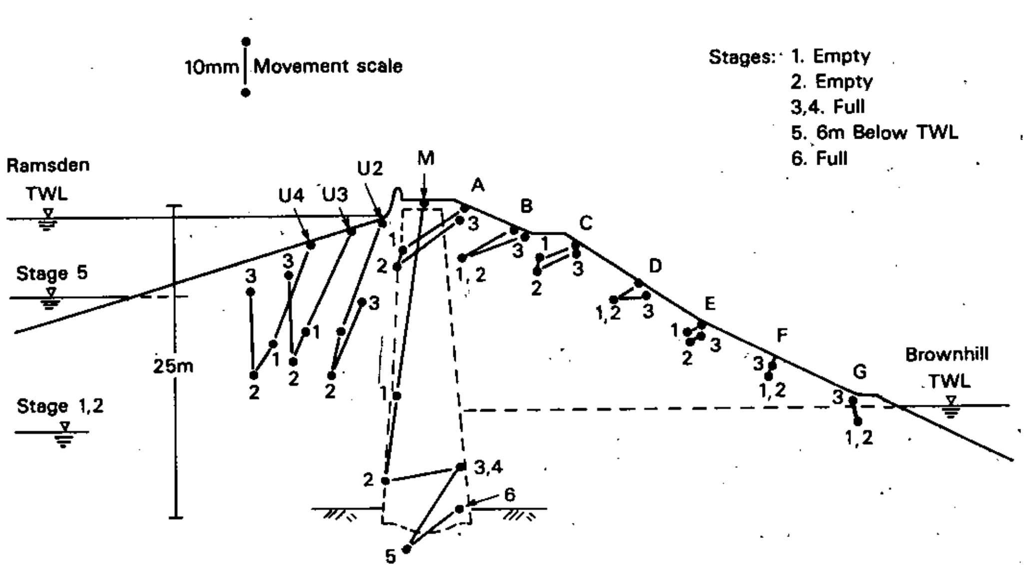

Fig. 2. Summary of surface movements at the end of the stages

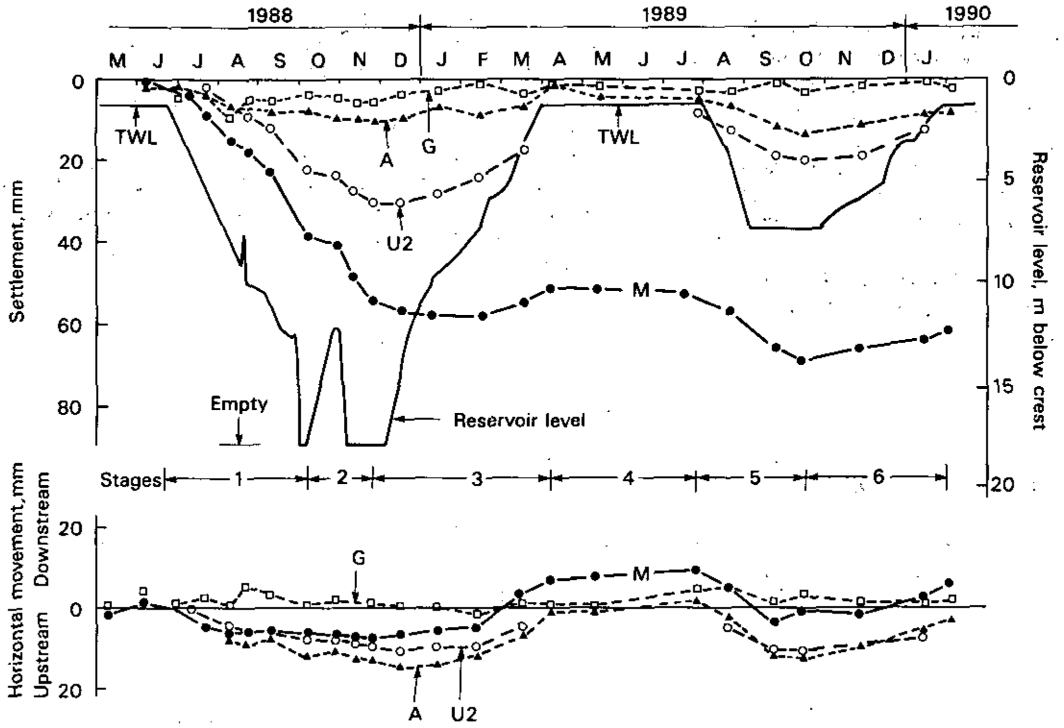

Fig. 3. Development of surface movements and changes in reservoir level with time

9. Figure 3 shows that settlement and upstream movement occurred as soon as the reservoir drawdown began. The rate of settlement of the crest stations was not significantly larger than those on the upstream slope particularly in the early stages of drawdown. Settlements on the crest and upstream slope continued during Stage 2 when the reservoir was empty and even when partial refilling occurred between the 3rd October and 7th November 1988 (see Fig. 3). Partial refilling caused the rate of settlement to decrease. The rate of decrease was larger the further away from the core. Emptying the reservoir again in November caused a rapid increase in settlement, again with the magnitude being greatest at the crest. Further settlement occurred when the reservoir was completely empty. There was little movement of the survey stations on the downstream slope during Stage 2.

10. Refilling the reservoir from empty to full (Stage 3) caused only 6mm of heave at the crest, about 10% of the total settlement. Settlement at the crest was still continuing (see Fig 3) when refilling commenced and continued until the reservoir level was approximately 11m below the crest level. Measurable heave did not occur until the reservoir was 5m below crest level. In contrast heave of the survey stations on the upstream slope was measured shortly after reservoir impounding began. A much larger proportion of the settlement on the upstream and downstream slopes recovered on reservoir refilling (see Fig. 2). Again measurements could not be taken on the upstream stations when the reservoir was full. Refilling of Brownhill reservoir during December 1988 probably caused the heave of the station G.

11. Refilling also caused the crest to move horizontally downstream by an amount greater than it had previously moved upstream, resulting in a net downstream movement. However, on the downstream slope, at survey stations A,B,C and D, the upstream movement caused by drawdown was generally equal to the downstream movement on refilling with no net downstream movement. As with the heave on refilling, the majority of the horizontal downstream movement occurred during the last stages of refilling.

12. Very little movement occurred during Stage 4 when the reservoir remained full for 4 months, but as soon as the reservoir was lowered (Stage 5), settlement and upstream movement of the crest occurred. Refilling the reservoir caused the same pattern of movements as with the earlier refilling, but the amount of crest settlement recovered as a percentage of the maximum settlement was much larger than when the reservoir was completely emptied, see Table 2.

Table 2. Crest settlement and recovery

	Complete drawdown	Partial drawdown
Maximum settlement on drawdown	58mm	16mm
Settlement recovered on refilling	6mm	8mm
Percentage recovered	10%	50%

Note: The maximum settlement includes Stages 1,2 and part of 3 for the complete drawdown

Sub-surface movements

13. All sub-surface movements presented in Figs 4 and 5 are absolute movements having been corrected for movement at the top of the instruments. Figure 4 shows the horizontal and vertical movements with depth at the end of various stages. All movements are relative to May 1988, except for the settlements presented in Fig. 4c. where they are relative to the end of stage 4. Figure 5 shows the vertical movements with time at the surface, and 9.1m and 18.5m below the crest together with the level of Ramsden reservoir.

14. Complete drawdown of the reservoir (Stage 1) resulted in vertical displacement throughout the depth of the core. Linear extrapolation of the settlement below 22.5m gives zero settlement at the top of the cut-off at depth of 25m. The average strain over the full depth of the core was approximately 0.16% although it was significantly smaller above a depth of about 9m and no strain appears to have occurred in the upper 2m of the core.

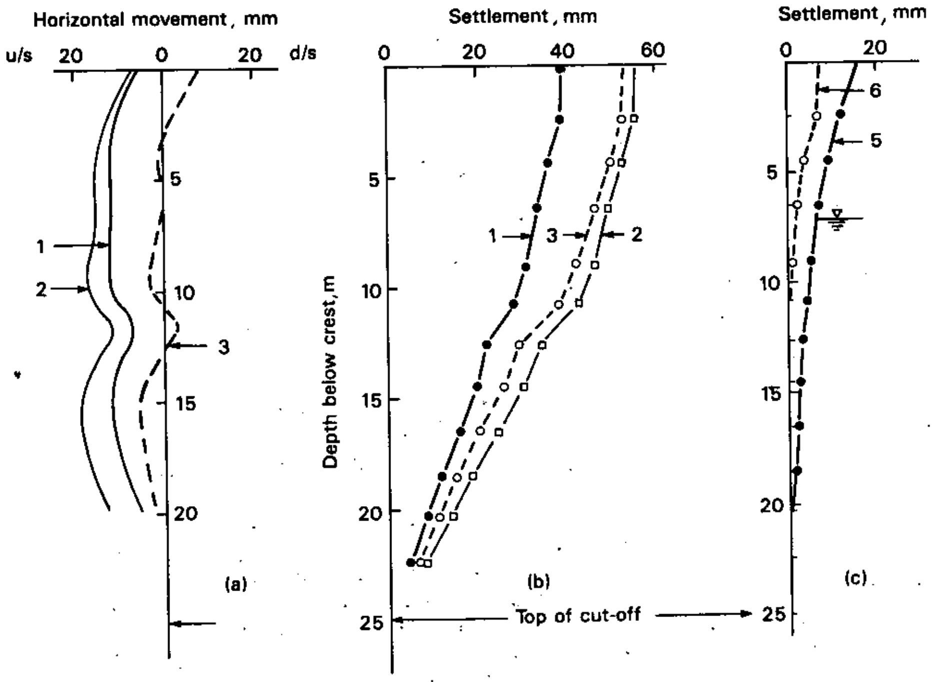

Fig. 4. Sub-surface movement of the core at the end of the stages (a) horizontal movement, (b) settlement, stages 1,2 and 3 and (c) settlement, stages 5 and 6

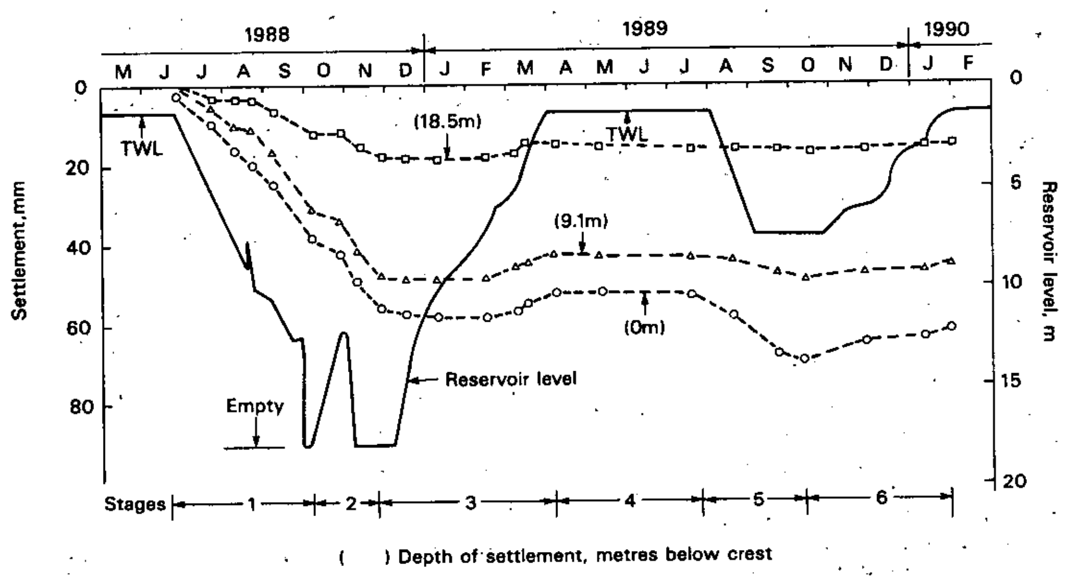

Fig. 5. Development of sub-surface vertical movements in the core with time

15. The inclinometer results (see Fig. 4a) showed upstream horizontal displacements to increase with depth to about 5m as a result of reservoir drawdown. From 5m to 10m there was no increase in upstream movement. The small discontinuity in the measurements below 10m corresponds with the discontinuity in vertical strain between 11m and 13m, (Fig 4b).

16. The effect of partial refilling of Ramsden reservoir during Stage 2 was accompanied by a decrease in the rate of settlement at all depths. A very small heave may have occurred below 18m depth, (see Fig 5). Emptying the reservoir again in November caused a rapid increase in settlement at all depths at both sections. Further settlement occurred at all depths when the reservoir remained empty during November.

17. On refilling (Stage 3) heave began to occur at depths equivalent to the reservoir level while the crest was still settling. When Ramsden reservoir was full again, a net settlement had occurred at all depths. The discontinuity in vertical strain between 11m and 13m was more evident. Above and below this small discontinuity the strain was reasonably linear. The strain above this depth was less than the strain below, see Fig 4b. Refilling caused little change in the horizontal movement profile except for the apparent total downstream movement of the whole core.

18. Very little movement occurred during Stage 4 when the reservoir remained full. Figure 4c shows the profile of settlement with depth caused by drawing the reservoir down to 6m below TWL, (end of Stage 4 to end of Stage 5) and then refilling (end of Stage 6). The settlement measurements in Fig. 4c are relative to the end of Stage 4 when the reservoir was full.

19. Piezometric levels in the upstream and downstream shoulders close to the core followed the water levels in Ramsden and Brownhill reservoirs respectively with little time lag. Permeability measurements in the downstream piezometers gave values of 10^{-7}m/s. The measurements showed that water pressure from Ramsden and Brownhill reservoirs act against the core of Ramsden dam. Above the level of Brownhill reservoir the piezometric levels in the downstream fill were very small.

20. Piezometric and earth pressure measurements in the core showed a steady decrease on reservoir drawdown and increase on refilling. The ratio of change in horizontal earth pressure and piezometric pressure to change in reservoir head was approximately 0.3. This is similar to that measured in other dams (ref. 4).

DISCUSSION

21. Long term continuing deformation (settlement and horizontal movement) can be an important indicator of performance of a dam. Apart from localised surface disturbances caused by, for example, traffic and animal activity, there are four main mechanisms that could cause long term deformation.

a. Internal erosion
b. Slope instability
c. Secondary consolidation of puddle clay and creep of shoulder fill
d. Stress changes due to fluctuations in reservoir level

22. The first two are due to some malfunction of the embankment and will at some stage require some remedial work to be carried out to the dam. Earlier investigations, (ref. 1 and 4) indicated that internal erosion of the core as result of hydraulic fracture was unlikely to be occurring at Ramsden dam and even if it were occurring, the fill immediately downstream of the core should act as a filter and prevent further erosion. It would also seem unlikely from the movement observations, the shear strength parameters and the pore pressures that slope instability of the downstream fill was a cause of the movements at Ramsden.

23. The second two mechanisms are due to normal behaviour of the dam, but it is important to be able to quantify the magnitude of these effects. To determine whether settlements measured many years after the completion of a dam can be attributed to secondary consolidation of puddle clay core and creep of the shoulder fill or erosion or slope stability, Charles (ref. 5) has proposed the following settlement index (S_I).

$$S_I = \frac{s}{1000\ H \log(t_2/t_1)}$$

where s is the crest settlement measured in mm between times t_1 and t_2 since the completion of the embankment at a section of the dam H metres high. Where values of S_I are greater than 0.02 it is suggested that some mechanism other than creep may be causing the settlement and the situation should be seriously examined. Settlement measurements at Ramsden dam (ref. 5) give a S_I of 0.077 between 1977 and 1985, and 0.12 between 1983 and 1985. During 1988 when the reservoir was emptied, the S_I increased to 0.46.

24. At Ramsden dam the majority of the deformations measured during 1988 and 1989 were clearly due to reservoir fluctuations. Similar behaviour has been observed at a number of other embankment dams with puddle clay cores (ref. 6). A complete drawdown of Ramsden reservoir such as occurred in 1988 has been a rare event and therefore the relatively large net crest settlement of 50mm that was measured is unusual. Partial drawdown of the reservoir to 6m below TWL has been more common and therefore a net crest settlement of approximately 8mm per 6m drawdown might be expected. However, prior to the present investigation, one particular cycle of drawdown to 6m below TWL and refilling in less than 2 months in 1986 caused less than 1mm of crest settlement. The rate of drawdown on this occasion was much less than in 1989, but refilling was much quicker. It would therefore appear that the rate as well as the magnitude of the drawdown influences the net settlement.

Principal mechanisms causing deformations due to reservoir fluctuation

25. Upstream fill: The piezometric levels in the upstream fill follow the reservoir level with little time delay. Therefore reservoir drawdown causes a rapid increase in the vertical effective stress approximately equivalent to the decrease in reservoir head. The effective stress may be nearly doubled. The settlement of 30mm is equivalent to a modulus of approximately 100,000 kPa. This would be very large for initial loading of such a fill, but would be reasonable for a reload situation. Resubmergence of the upstream fill causes a decrease in effective stress and more elastic recovery of surface settlement than on the crest.

26. Puddle core: Two mechanisms could account for the movement of the core. Firstly, lowering the reservoir level reduces the lateral pressure on the upstream side of the core and leads to horizontal upstream movement. Initially this movement would be due to undrained deformation of the core. On refilling the reservoir the horizontal pressure of the reservoir water against the upstream side of the core causes a horizontal downstream movement of the core and probably some heave of the core. Horizontal downstream movement of the core also allows some downstream movement of the upstream fill close to the core.

27. Secondly removal of the water from the upstream side of the core also changes the drainage conditions and allows consolidation and time dependent settlement of the core. Most of the settlement that occurred during Stage 2, when the reservoir was "empty", will have been due to the dissipation of pore water pressures. Refilling will have caused swelling of the core.

SUMMARY AND CONCLUSIONS

28. Reservoir drawdown caused significant surface settlement and smaller upstream horizontal movement of the crest and the upstream slope. Impounding reversed the movements, resulting in a net settlement and downstream horizontal movement at the crest when the reservoir was full. The magnitude of the net crest settlement was affected critically by the magnitude of the drawdown in terms of height and time. Only 10% of crest settlement due to complete drawdown was recovered on refilling, but 50% was recovered following refilling after drawdown to only 6m below TWL. There does not appear to be any appreciable net surface movement of the downstream fill following a cycle of drawdown and impounding. Drawdown caused settlement and changes in pore water pressure throughout the depth of the core.

29. Settlements occur in the upstream fill because reservoir drawdown increases the effective stress. Settlement of the core occurs because reservoir drawdown causes a reduction in lateral support on the upstream side of the core and hence undrained deformation. Further time dependent deformation of the core occurs as excess pore water pressures dissipate.

30. The magnitude of net crest settlement following a cycle of drawdown and refilling depends on the magnitude and rate of the drawdown, and on the time the reservoir is left in a drawndown state. Research is continuing to investigate the effects of reservoir fluctuations on embankment deformations at other dams.

ACKNOWLEDGEMENTS

31. The work described in this paper forms part of the research programme of the Building Research Establishment and is published by permission of the Chief Executive. The main client for the work is the Water Directorate of the Department of the Environment. The initiative of Mr J D Humphreys in instigating the work at Ramsden dam is appreciated. The General Manager of Yorkshire Water Western Division, Mr J.R. Layfield, permitted the use of the dam for research purposes and encouraged publication. Mr I R Holton has made a valuable contribution to the field work and the analysis of the results. Mr D Burford designed and installed the surveying system.

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27. The routine monitoring of embankment dam behaviour

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The paper outlines the various requirements for the structural monitoring of embankment dams within the Western Division of Yorkshire Water which is responsible for the operation of over 70 such dams. It details the ways in which the work is routinely carried out with a particular emphasis on the monitoring of deformations. The methods adopted for the storage of these records is described in some detail and the paper concludes with a presentation of some of the results that have been observed.

INTRODUCTION

Yorkshire Water - Western Division

1. The Western Division of Yorkshire Water is responsible for the supply of water and treatment of waste water for a population of just over one million in the Metropolitan Districts of Bradford, Calderdale and Kirklees and in part of North Yorkshire County Council. In order to supply water to this population there are 80 reservoirs within the Division which have a capacity greater than 25tcm and are therefore covered by the provisions of the Reservoirs Act 1975.

2. Over 70 of the reservoirs within the Division are formed either wholly or in part by earth embankments with the remainder consisting of either concrete dams or covered concrete service reservoirs. The average age of the dams is approximately 100 years with the oldest being completed in 1827 and the newest in 1985. The capacities of the reservoirs vary from just over 25tcm to approximately 22,000tcm in the case of Grimwith reservoir and the highest embankment is Scammonden Dam which, with a height of over 60m, is the second highest earth dam in the United Kingdom.

Reservoir Safety Management

3. The reservoirs in Western Division are operated by water supply staff in the Bradford, Calder and Skipton Operational Areas. A Divisionally-based Reservoir Safety section has been in place since 1983 and currently consists of a Reservoir Safety Manager, a Project Engineer, a Surveyor and a Technician. The section is responsible for the following activities:-

(a) Ensuring compliance with the provisions of the Reservoirs Act 1975.
(b) Providing advice to the Operational Areas on Reservoir Safety matters.
(c) Coordinating and progressing the implementation of repair and improvement schemes at reservoirs.
(d) Monitoring the behaviour and structural performance of the dams within the Division.
(e) Maintaining the statutory and other associated records for all the reservoirs within the Division.

4. The remainder of the paper describes the various methods that have been adopted within the Division to monitor the structural performance of the dams and to keep records of their behaviour. The survey methods that are described should not be regarded as the only methods that are available but they have been found to provide acceptable results over a large number of years.

EMBANKMENT MONITORING REQUIREMENTS

5. The extent of embankment monitoring in Western Division has primarily been governed by recommendations made by Inspecting Engineers in their statutory inspection reports under the Reservoirs Act 1975. The introduction of Supervising Engineers in the mid-1980's also resulted in an increase in monitoring work with recommendations for new monitoring systems and requests for the reading of established systems at increased frequencies.

6. The monitoring of embankment behaviour or performance can essentially be considered to fall under three main headings:-

(a) Deformation monitoring
(b) Pore-water pressure monitoring
(c) Leakage/drainage flow monitoring

7. In the case of deformation monitoring, prior to 1970 recommendations from Inspecting Engineers do not appear to have been specific enough as to necessitate the setting up of permanent monitoring systems and the only work that was undertaken was to level along the crest of a dam to establish its freeboard. This was obviously not a suitable arrangement for the

on-going monitoring of behaviour as no permanent stations were involved. Similarly, the early methods for monitoring groundwater levels involved the use of "observation wells" which were slow to react and unsuitable for detailed monitoring. Flow monitoring is generally the only measurement procedure which provides long-term records against which current behaviour can be assessed.

8. The mid 1970's witnessed a new emphasis on embankment monitoring with more detailed recommendations from Inspecting Engineers requiring the setting up of permanent levelling and/or alignment stations on dams. This new approach also coincided with a dramatic growth in site investigation work and since then there has been a continuing increase in the number of piezometers installed throughout the Division.

9. In addition to the requests from Inspecting and Supervising Engineers, Western Division has also adopted a policy whereby additional measurements are taken at reservoirs which are undergoing substantial drawdown and refilling operations. These additional measurements primarily relate to the monitoring of deformations but the frequency of other readings may also be increased. It is also a policy of the Division to annually monitor the vertical movement of all earth embankment dams whether or not this is the subject of a recommendation from an Inspecting Engineer although permanent stations have not yet been established at all dams.

10. The frequency of monitoring varies according to the requirements of a particular dam, however, broadly speaking the most common frequencies can be summarised as annually for movement monitoring, monthly for the reading of piezometers and weekly for drainage/leakage flows. There are obviously a considerable number of exceptions to these frequencies with, for example, movement monitoring reducing to monthly in some cases and leakage monitoring increasing to three monthly. In some cases it is recommended that monitoring be carried out with the reservoir at its highest and lowest water levels during the year although in practice this is often difficult to achieve.

EMBANKMENT MONITORING SYSTEMS

11. Table 1 sets out the full extent of monitoring work which is undertaken in Western Division at all its 80 reservoirs and illustrates the considerable workload that is involved. The monitoring of piezometric levels and leakage/drainage flows generally use the traditional measuring systems that are in widespread use. Since such basic systems as V-notch weirs, the timed filling of measuring containers and the details of standpipe, hydraulic and pneumatic piezometers will be familiar and commonplace to most readers the remainder of the section will concentrate on deformation monitoring and will describe the systems adopted in Western Division to establish patterns of vertical and horizontal displacements.

	No of Dams	
	Earth	Concrete
Movement		
Vertical	66	1
Horizontal	9	1
Piezometers	18	1
Flows	39	3

Table 1. Dam Performance Monitoring Systems in Western Division

Deformation Monitoring

12. As can be seen from Table 1, the majority of embankment dams in Western Division have systems for the monitoring of vertical deformations, with most of the levelling stations being positioned along dam crests, usually in or above the core material. However, individual circumstances sometimes dictate the siting of additional levelling stations and there are several sites within Western Division where such monitoring stations can be found on the upstream and downstream sides of the crests, on wave walls and on the upstream and downstream shoulders.

13. In the case of horizontal movement monitoring, the most common system used is again to install stations along the crests of dams with the upstream-downstream component of the movements being treated as the most important to be monitored. Should the need arise to monitor other parts of an embankment such as the upstream and downstream shoulders, systems can be readily designed and adapted to cater for such needs.

Surveying Techniques - Levelling

14. Vertical deformations are invariably monitored by precise levelling techniques using a Wild NA2 Automatic Level with parallel plate micrometer attachment which enables readings to be taken to the nearest 0.1mm. Sightings are taken to a 4 x 1 metre sectional aluminium-alloy staff with centimetre graduations. The use of an invar staff has been considered but has not been adopted as it is thought to be impractical and unnecessary, mainly due to the adverse weather conditions which are prevalent at many of the sites within Western Division but also because acceptable results can be obtained using the more traditional staff.

15. Crest-levelling stations are spaced at no greater than 30 metre intervals and at least one bench mark is established on stable ground away from the dam. A point on the overflow crest is also used as an additional datum wherever possible.

16. A closed loop system of levelling has been adopted with each station being levelled on the outward and return legs, thus providing a check on the readings. The level is set up adjacent to every other monitoring station with a short intermediate sight being taken to this station. Backsight and foresight distances are kept the same to eliminate collimation error between the change points. No adjustment is made to the readings to distribute any closing

error and the calculated levels from observations on the outward leg are used if the closing error is within acceptable limits and its pattern of accumulation is satisfactory.

17. In addition to traditional levelling techniques, trigonometrical heighting is also undertaken at some sites incorporating precise theodolite observations and distance measurements. This method usually forms part of a larger system whereby three-dimensional coordinates for stations on a dam are established from remote instrument stations. This is discussed in more detail in the following section.

Surveying Techniques - Alignment

18. Whilst the monitoring of vertical deformations on old embankment dams has become standard practice since the late 1970's, the same prominence has generally not been given to the checking of horizontal deformations. In the case of new embankment dams, such monitoring systems are established as a matter of course in order to monitor their initial behaviour. However, the setting up of similar systems on older dams has only been undertaken as a result of areas of instability becoming evident in the form of increased settlement, cracking, slumping etc.

19. Observations are taken to alignment monitoring stations using a Wild T2000 Electronic Theodolite with Wild DI4 Electromagnetic Distance Measuring attachment. Raw field data is transferred directly to a Husky Hunter data-logger which is equipped with Optimal survey software to present the results in coordinate form as and when required.

20. Method 1: The basic method used for the monitoring of horizontal movement within Western Division is to construct a control pillar on stable ground in line with the long axis of the dam crest. The theodolite and EDM are set up on this pillar and observations are first taken to reference points around the site and then to the monitoring stations on the dam. The small angular changes to the monitoring stations with time represent upstream or downstream movement which, along with a horizontal distance measurement, can be converted to an actual movement in millimetres. The limitations of this method of monitoring include:

(a) It is difficult to incorporate on long dams, curved dams and dams with more than one embankment.
(b) Whilst it provides excellent results for monitoring stations on the crest and wave-wall, this system becomes less accurate for stations on the upstream and downstream shoulders.
(c) It only detects horizontal movement in the upstream-downstream direction.

21. Method 2: An alternative method that is used is the network approach whereby a number of survey pillars are set up on stable ground around the site and coordinated to provide local control. Bearings, vertical angles and slope distances are then taken from an appropriate pillar to the monitoring stations on the dam from which X, Y and Z coordinates for each station can be calculated. Positioning of the instrument stations is of vital importance and wherever possible steep sights are avoided as slight errors in the measurement of the vertical angle or dislevelment of the instrument will result in exaggerated inaccuracies in the calculated coordinates of the monitoring stations. This method is perhaps a more flexible system of monitoring and is favoured to Method 1 where dams have monitoring points on both crest and upsteam and downstream shoulders. Movement is also detected in all directions and therefore a more comprehensive analysis can be made of the deformation characteristics of the dam. However, the accuracy suffers slightly with this system, especially in the vertical plane, due mainly to the more indirect way of arriving at such a value as compared to spirit levelling. This method also tends to be much more time-consuming which is especially important with such a large number of dams to be monitored.

Monitoring Stations

22. The first crest levelling stations in Western Division were set up in the late 1970's and consisted of a length of reinforcing bar set in approximately 0.3 cu m of concrete with the top of the bar positioned at or just above ground level. Such stations have proved unsuitable for long-term monitoring due to their susceptibility to damage and disturbance by

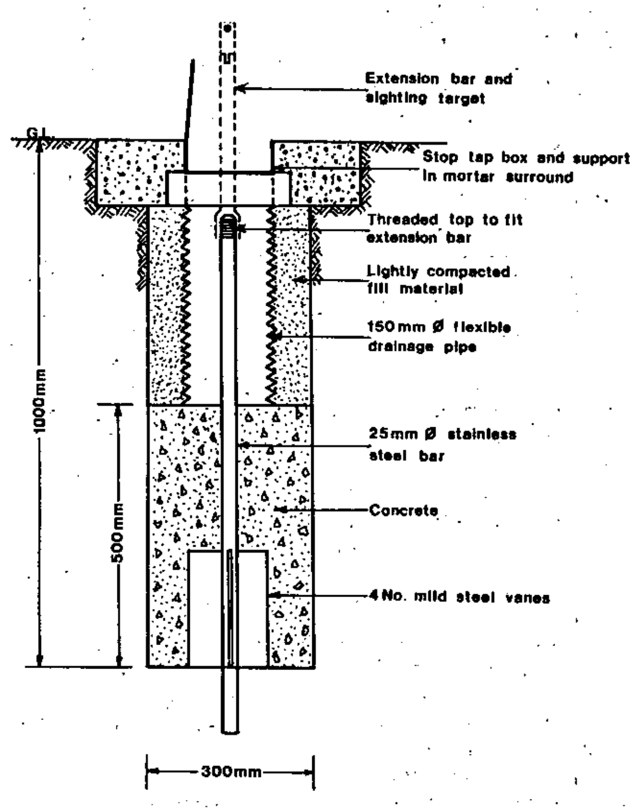

Fig 1. Embankment Crest Monitoring Station

vandalism, heavy plant, grass cutting equipment, frost heave and corrosion. A programme of replacing old stations with a new installation design has therefore been implemented (see fig 1). The location of the monitoring station below ground level and the introduction of a flexible sleeve provides adequate protection against damage.

23. A facility for monitoring horizontal alignment has also been incorporated into the design whereby an extension bar with a survey target can be attached to the underground levelling pin. This brings the installation above ground level and therefore able to be observed from an instrument station off the dam. This new design of monitoring station is being restricted to use on embankment crests only as they are relatively easy to locate and lines of sight are not likely to be obscured by excessive vegetation cover. When monitoring downstream shoulders the use of conventional pillar-type monuments is preferred which can be set at varying heights above ground level thus allowing unobscured lines of sight and ease of location.

Surveying Errors associated with monitoring work

24. Apart from mis-readings by the Surveyor, the other errors associated with the precise levelling of monitoring stations are generally either weather related or instrument related.

25. Windy and sunny conditions both result in a marked drop in the accuracy of readings. The majority of reservoirs in Western Division are sited above 250m AOD and in very exposed locations so surveying in windy conditions becomes virtually unavoidable if monitoring requirements are to be met. Also irregular refraction on hot sunny days makes reading of the staff extremely difficult with a resulting reduction in accuracy. This is especially apparent in conditions of intermittent sun where fluctuations in the same staff reading of up to 0.3mm occur when lighting conditions change during sighting.

26. The main instrumental error is associated with the line of collimation of the level being out of adjustment and therefore not projecting a truly horizontal line of sight. Errors resulting from this factor can be kept to a minimum by equalising the backsight and foresight distances between change points and using the same approximate instrument positions at each visit. In order to achieve the maximum continuity between readings, adjustment of the level is avoided unless the errors become unacceptable. In practice, critical examinations of the levelling errors have never indicated a requirement for adjustment.

27. The errors mentioned previously have been found to have a cumulative effect on the accuracy of the levelling run resulting in a close relationship between the length of survey and the closing errors arising. This relationship is shown in Table 2, the information being derived from precise levelling work undertaken on dams in Western Division over several years.

Crest Length (m)	Expected Closing Error (mm)
100	± 1.0
200	± 1.5
300	± 2.0
400	± 2.5
500	± 3.0
600	± 3.5

Table 2. Closing Errors during Monitoring Work in Western Division

28. Table 2 assumes a closed loop of levelling along the dam crest and back. The expected closing error indicates the error which can be expected to accumulate using the automatic level with parallel plate micrometer and aluminium staff in calm, overcast conditions. It must be stressed that these values for expected closing errors can only be used as a rough guideline and that other factors must be taken into account when assessing the degree of reliability of a particular set of readings.

29. In alignment monitoring the weather conditions are again a major factor influencing accuracy, with sunny conditions having a marked detrimental effect on angular readings, and damp, misty conditions causing great problems when attempting precise distance measurements using EDM. Obviously in these extreme weather conditions it would be inadvisable to attempt to carry out such precise surveying work, however, if one was to be entirely governed by the ever-changing nature of British weather the monitoring work would become virtually impossible. Therefore, as ideal weather conditions cannot be guaranteed, a compromise has to be reached whereby as many sightings as possible are taken to each monitoring station from which mean values are extracted.

30. As with levelling, instrumental error can again be a major problem when monitoring horizontal alignment and regular checks are undertaken to check whether instruments are in correct adjustment or are in need of repair.

RECORD KEEPING

General Introduction

31. There are generally three types of records which must be maintained for the proper management of large reservoirs and these can be summarised as follows:-

(a) Statutory.
(b) Monitoring.
(c) Operational.

32. For the owner of a large number of reservoirs there is a requirement to keep large amounts of information which is of the same basic format and, as such, is amenable to the use of computer-based storage systems for the efficient handling of the data. With this basic premise in mind a decision was taken in 1983 to implement the use of such systems and a start was made on the first of the monitoring systems which was for the storage of settlement records. The original proposal was for the storage of

monitoring and operational data only but the phased introduction of the Reservoirs Act 1975 from April 1985 onwards provided an opportunity to extend the scope of the systems to provide a comprehensive Reservoir Safety Records System covering the three items listed above.

33. The original decision to computerise the storage of reservoir-based data coincided with the introduction of a "Distributed Data Processing" approach to the provision of computer services across the whole of Yorkshire Water. This was a proactive approach whereby staff were actively encouraged to develop and implement computer-based solutions to their problems and was backed up by a heavy expenditure on the introduction of new hardware and support facilities. The initial investment consisted of the purchase of five DEC VAX 750 mini-computers to supplement the existing ICL main-frame together with a considerable expansion of the hardware and software support arrangements including the introduction of a large training programme. The VAX computers were located at 5 sites within Yorkshire Water and interconnected by a communications network with terminals connected to each VAX by a Wide Area Network. The basic configuration has been considerably uprated since 1983 with the introduction of additional and more powerful VAX's which can be clustered together to work as a more efficient unit.

34. The individual components of the Reservoir Safety Records Systems were completed by 1986 and have remained essentially unaltered since then apart from minor modifications to improve their efficiency and the introduction of arrangements whereby other persons outside the Reservoir Safety section can have access to the data.

Reservoir Safety Records System

35. Access to the VAX network is by a series of hierarchical Users which are each allocated an amount of disc-storage space. Each User is protected by a password and has access to the following software facilities:-

(a) Database Management (DATATRIEVE)
(b) Spreadsheet (DECALC)
(c) Graphics (DECGRAPH)
(d) Office Management (Word Processing, Electronic Mail).

A major advantage of the multi-user approach is that it is possible, by the establishment of appropriate path names within the software, for Users to have shared access to data records. The access capabilities of the various Users can be controlled by the granting of appropriate privileges to give Read, Write or Modify access to each individual User depending on their requirements.

36. The Reservoir Safety Records are held in DATATRIEVE databases which are located in two interlinked Users named RESSURV and RESSAFE and the various links with other Users are as shown in Figure 2.

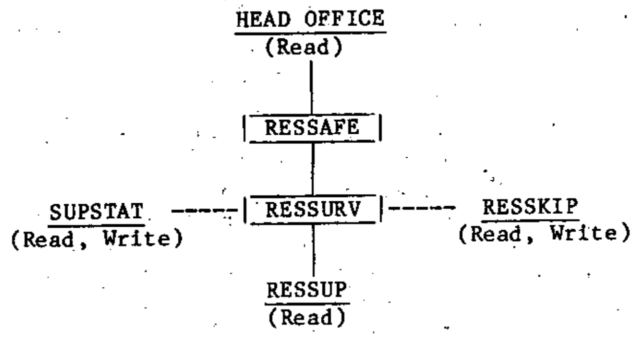

Fig 2. User Access to Reservoir Safety Records

This figure illustrates the use of privileges to restrict the type of access granted to each User. The SUPSTAT User is controlled by the Divisional Water Supply Statistics section and is the means by which reservoir levels are transferred into Part 1 of the prescribed form of record at weekly intervals after having first been used to calculate reservoir stocks. The RESSUP User is available to all Divisional Supervising Engineers for read-only access to all the statutory and monitoring records. The RESSKIP User is controlled by the Skipton Area office which is remote from the Divisional Headquarters and is used for the input of raw monitoring data and the output of calculated results (drainage flows, piezometric levels, reservoir levels, etc). The HEAD OFFICE User is located on a different VAX but network linkshave been established to enable Regional staff to have access to data from all Divisions relating to the appointment of Panel Engineers and their progress with Inspections.

37. All components of the Reservoir Safety Records System are accessed by a series of menus. The three main sub-divisions of the system are as follows:-

(a) Head Office Statistics.
(b) Prescribed forms of record.
(c) Dam Performance Monitoring System.

The Head Office statistics have been briefly described above and the next two sections will describe the other components of the system.

Prescribed forms of record.

38. The relevant information for each Part of the prescribed form is held in database form and can be accessed from menus for storage, deletion, modification and reporting purposes. A reporting procedure has been established for each Part so that the printout resembles, as closely as possible, the form that is set out in the Regulations. Hard-copy printouts are kept in individual files for all 80 reservoirs and are continually updated as changes occur with all superseded information being retained so that the reason for the change can be deduced at a later date. Copies of the appropriate monitoring information for each reservoir is also contained in the file and is periodically updated with information from the Dam Performance Monitoring System.

39. There are a number of uses to which the information contained in the prescribed form can be put. As mentioned previously, weekly records of water level are automatically transferred into Part 1 of the system from an associated system and these levels are used to assist in the interpretation of the structural monitoring data. Part 10 of the prescribed form contains details of all recommendations made by Inspecting Engineers in the interests of safety. The relevant database has been expanded to include all other recommendations which may have been made by either Inspecting or Supervising Engineers and also includes details of any work arising from valve tests. Reports can be produced of only those items to which Part 10 refers or alternatively they can be produced of all recommendations, complete with target and completion dates, which can be used by Operations for maintenance programming purposes. In addition to the two specific applications referred to above, the information in the records can also be used for the production of ad-hoc reports and even for distribution lists.

Dam Performance Monitoring System (DPMS)

40. Records are not incorporated into DPMS without some consideration being given to the capabilities of existing systems and, for example, the manual system for recording flows in the Calder Area has been retained as it is considered to give a satisfactory presentation of the results. Despite this, however, the vast majority of the monitoring information from dams within Western Division is now recorded in DPMS.

41. The information contained within DPMS is held in a number of databases covering the following subject areas:-

(a) Movement.
(b) Piezometric Level.
(c) Drainage/Leakage Flows.

Each database is accessed by a menu for storage, reporting and plotting purposes although deletion and modification are only available by directly entering the database management system. All the databases include records of reservoir level for the appropriate date and the piezometer and flow records can also include rainfall if required.

42. The movement system has been set up so that surveying information in both the horizontal and vertical planes can be stored. The information is held in absolute form so that relative values can be calculated from any given starting date, eg. levels are stored in metres above Ordnance Datum and routines have been written to calculate settlement values from a given date for plotting purposes.

43. The piezometric information is entered in the form of either depth readings to the water surface in the case of standpipe piezometers or pre-calculated pressures. Tables of top of standpipe levels or appropriate datum levels are held within the computer from which piezometric levels can be calculated for either standpipe, hydraulic or pneumatic piezometers. Tip levels are also held in tabular form so that the head above the tip can be calculated for reporting purposes.

44. The flow system allows for the entry of readings in either time taken to fill a given container, head measured over a weir or direct entry of previously calculated flows.

Head-discharge tables for various weir types are held within the computer so that flows can be automatically calculated and whichever method of entry is used, the flows are always presented in litres per minute for reporting and plotting purposes.

45. With such a large number of reservoirs it can be appreciated that a correspondingly large amount of data has been, and is constantly being, entered into the various systems. In the case of the settlement system alone there are now over 10,000 records held in the computer and this figure is considerably exceeded by the number of reservoir levels contained in Part 1 of the prescribed form. The capabilities of the DATATRIEVE system are such, however, that even with such large quantities of data the retrieval times are still perfectly adequate at the present time although arrangements are being considered for the archiving of data on magnetic tape so that it can be stored off-line should this ever be required.

ANALYSIS OF MONITORING DATA

Production of Results

46. Results are initially analysed by the production of reports and a number of standard reports can be selected from the appropriate menus of DPMS. The reports are designed such that all the data for the chosen monitoring point, ie. survey station, piezometer or drain, between selected dates, is printed in date order enabling a rapid appraisal of the records with respect to time, reservoir level and/or rainfall to be carried out. If further investigation is felt to be justified then a number of standard graphs can be generated or the data can be transferred to the DECALC spreadsheet package for further manipulation prior to plotting.

47. Although any graphics terminal can be used to produce the plots, a more versatile workstation has been established within the Reservoir Safety Management Section and the relevant details are shown in Figure 3.

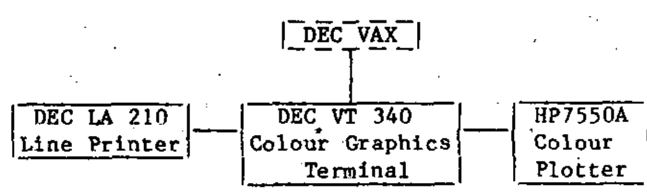

Fig 3. Diagrammatic Representation of DPMS Workstation

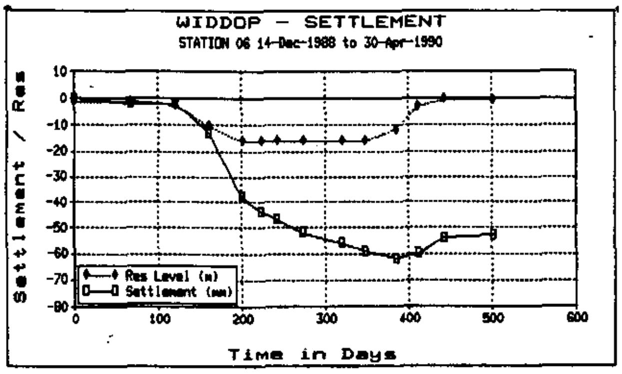

Fig 4. Crest settlement at Widdop Dam

Selection routines have been established within DPMS to choose the data to be plotted and the actual plotting is carried out by either the line printer using the DECGRAPH package or by the plotter using a more complex package called SIMPLEPLOT. The main difference between the two packages is that the DECGRAPH system has been set up to plot only one point plus reservoir level whereas the SIMPLEPLOT system can plot up to nineteen points plus reservoir level to a higher standard of presentation in up to eight colours with a selection of line-types. Figure 4 shows a graphical representation of the settlement of a survey station on the embankment crest at Widdop reservoir and was produced from a menu in a matter of seconds by the DECGRAPH system.

Analysis of movement data

48. Figure 4 has been specifically included as it illustrates a number of points which have been observed at Widdop and at other dams of a similar construction. The dam at Widdop consists of an earth embankment over 20m in height with a relatively wide crest and a puddle-clay core which extends into a shallow puddle clay filled trench. The graph refers to Station 06 which is located at the upstream side of the crest in the middle of the dam and shows the settlement of the point as the reservoir was emptied and then refilled between the beginning of 1989 and the beginning of 1990. It can be seen that the drawdown caused a sharp increase in the rate of settlement which was of the order of 10 times the previous long-term rate and amounted to approximately 40mm from the reservoir full to the reservoir empty condition. Whilst the reservoir was empty the settlement continued but at a reduced rate such that by the time the reservoir was ready to be refilled a further 20mm of settlement had occurred. Initial refilling of the reservoir did not result in any discernible change in the settlement rate but a rise of approximately 8mm did occur as the reservoir approached the full condition. The whole full-empty-full cycle resulted in a net settlement of just over 50mm.

49. Analysis of the associated horizontal data shows that the crest of the dam as represented by the wave-wall moved upstream by approximately 40mm during drawdown and then moved downstream by approximately 20mm on refilling. Observations to the wave-wall in this case may have produced a false impression for the movement of the dam as a whole as experience on other dams has indicated that, in general, dams are left with a net downstream movement as a result of the reservoir being drawn down and refilled.

50. Movement observations of the large number of dams within Western Division over the last 13 years have led to the formation of the following general conclusions:-

(a) The movement of a dam is directly related to the height of fill. Any departure from this relationship is a clear indication of either survey errors or the development of a problem.

(b) Drawing a reservoir level down is likely to produce a rapid increase in the rate of settlement accompanied by an upstream movement. Completely emptying a reservoir is likely to produce disproportionately large movements.

(c) Maintaining a reservoir in an empty condition eventually causes the settlement to return to the long term reservoir full settlement rate.

(d) Refilling a reservoir does not initially alter the rate of settlement but eventually heave does occur although this does not usually fully compensate for the initial settlement. Generally the dam also moves back downstream more than it moved upstream during emptying of the reservoir.

(e) It has generally been found that relatively short dams in steep-sided valleys move more than longer dams of a similar height.

51. The basic patterns of movement described above enable a fairly accurate prediction of the three-dimensional movement of a dam caused by reservoir drawdown to be made so that any departures from the pattern can be critically examined. Tedd, Claydon and Charles (ref. 1) examine this matter further in their analysis of Ramsden dam which is also located in Western Division.

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28. Embankment dam behaviour: the contribution of geo-chemistry

A. MACDONALD and J. M. REID, Babtie Shaw and Morton, Glasgow, UK

Geochemical degradation can have a significant effect on the geotechnical properties of embankment fill materials and on the quality of drainage waters. The most susceptible materials include mudstones, weathered igneous and metamorphic rocks, and rock with a high proportion of secondary minerals. Geochemical information should be collected at the site investigation stage, including from trial embankments. The design should allow for any geochemical degradation likely to occur and contractors should be made aware of the problems. Chemical monitoring should be carried out during and after construction to check the design assumptions.

INTRODUCTION

1. When material is excavated and placed as fill in an embankment, its properties will be liable to change from those in its natural state. This is due in part to the mechanical breakdown of the material during the earthworks operations, but also to chemical reactions resulting from the change of environment. The greater the contrast between the conditions under which the material formed and those pertaining in the embankment, the greater is the potential instability of the minerals of which the material is composed. Thus bedrock is more potentially susceptible to geochemical degradation than soil, in particular igneous and metamorphic rocks which formed under conditions of high temperature and pressure very different from those pertaining at the earth's surface.

2. In practice, however, the susceptibility of a material to geochemical degradation is dependent on a number of kinetic factors rather than on its mineral composition alone; fresh igneous and metamorphic rocks form some of the strongest and most durable aggregates available. The initial weathering grade of the material is of vital importance to both its physical and chemical stability, as are the grain size, presence of secondary minerals and degree of leaching to which the material is subjected both during placement and in the longer term dam environment.

3. The subject of geochemical degradation or weathering of rock as a factor in the design of engineering structures such as embankment dams has been attracting attention in recent years. The Department of the Environment recently commissioned a research contract on chemical deterioration of fill material in earth dams in the United Kingdom (ref.1). A number of papers have been published over the years by Taylor, Spears and co-workers on the breakdown of Carboniferous Coal Measures mudrocks in colliery spoil tips and tailings dams, eg (refs 2,3) The subject of rock weathering in engineering time has been comprehensively discussed by Fookes et al (ref.4), who proposed Rock Durability Indices to measure the susceptibility of materials in various engineering situations.

4. The processes which can cause geochemical degradation of fill materials are basically those involved in the natural weathering of rocks and soils, namely solution and precipitation, oxidation and reduction, hydration, hydrolysis, dispersion and cation exchange. These processes are described in more detail in (ref.1) and in standard geochemical and geological texts, eg (refs 5,6). The effect of excavation and placement in an aerated, active leaching environment such as an embankment can be to greatly accelerate these processes compared to the natural state.

5. Because of the large volumes of fill involved in dam embankments, even a small percentage of chemically active material can have a major effect on the quality of drainage water or the generation of hazardous gases. If the proportion of chemically active material is larger, there may be significant effects on the geotechnical properties of the fill, such as density, shear strength, compressibility, grading and permeability. These effects need to be investigated and quantified at the design stage if the embankment is to function satisfactorily throughout its design life.

6. Materials susceptible to geochemical degradation can vary widely. Fookes et al (ref.4) found that in-service deterioration occurred mainly in basic igneous materials and that the cause of degradation was either the presence of secondary minerals or active in-situ weathering. The study carried out for the Department of the Environment (ref.1) found that a number of rock types could be susceptible to deterioration in dams in the United Kingdom : these are; (1) calcareous

The embankment dam. Thomas Telford, London, 1991

materials in contact with acidic water; (2) clays or mudstones with a high proportion of expandable clay minerals; (3) clays or mudstones with a high exchangeable sodium percentage; (4) clays and mudstones containing pyrite; and (5) highly weathered rock, particularly of igneous and metamorphic origin. Apparently similar materials can behave very differently in different situations, depending on slight variations in mineralogy and weathering state. This is illustrated later by a comparison of the behaviour of Carboniferous mudstones at Roadford and Carsington dams.

7. Geochemical degradation is likely to be most intense during the initial stage of dam construction, when the material is newly broken up and exposed to an oxidising and leaching environment. Once the embankment is completed and the reservoir impounded the rate of reaction will decrease. If the material is below the water table the supply of oxygen will be greatly reduced and the rate of removal of weathering products will be limited by the rate of seepage through the embankment. This will generally be very slow. Further reactions may occur between the seepage water and other materials in the embankment, however, and it may take many years for all the reaction products to be leached out of the embankment and into the drainage system. Above the water table, reactions may be limited by the reduced rate of percolation of surface water once the surface of the dam is sealed by topsoil and drainage installed.

8. In a loosely tipped 50 year old colliery spoil tip, Spears et al (ref.7) showed that the maximum depth of chemical weathering was 3.81m below the crest, while the surface zone of intense weathering only extended to a depth of 1m. Excavations in the downstream side of Burnhope dam, County Durham (ref.8) showed that marks formed by the bucket teeth of the excavator were still visible in the mudstone fill after 25 years, indicating minimal deterioration of the material. At Carsington dam, the highly weathered mudstone fill was found to have undergone further change to a depth of 0.3m between 1984 and 1987. The fill was left fully exposed to the atmosphere during this time, and the surface layer had degraded to a soft dark grey clay. This material formed an impermeable layer protecting the underlying fill from further physical and chemical deterioration.

9. From the foregoing, it can be seen that the geochemical behaviour of fill materials should be investigated and taken into account in embankment dam behaviour. In the following sections guidance is given as to the actions required at various stages in the design and construction process, illustrated by examples from the authors' experience and other examples. It must be emphasised that every site is unique and that strategies for evaluating and dealing with geochemical degradation have to be developed to suit the particular circumstances of each site. Awareness of the potential problems is the key to success.

DESK STUDY

10. At the desk study stage information should be gathered on the mineralogy, degree of weathering and geochemical behaviour of any potential fill materials and of the dam foundation. In the U.K., published data may be available in memoirs and reports of the British Geological Survey, who should be approached for advice. Data may be available from articles in learned journals and site investigation reports. Inquiries should be made regarding any problems encountered with similar materials in existing embankments or other works.

11. The chemistry of the waters to be stored in the reservoir should be determined and compared with the mineralogy of the potential fill materials. Natural waters in the U.K. are generally not aggressive chemically, but acidic waters resulting from peat in upland areas or pollution in lowland areas can cause dissolution of carbonates in fill material (ref.1). Reaction between stored effluent and fill material is a significant factor in the design of tailings dams (ref.9).

12. The aim at the desk study stage is to identify potentially susceptible materials and prepare a programme of sampling and testing to be carried out at the site investigation stage.

SITE INVESTIGATION

13. The site investigation is the stage at which a comprehensive databank of the chemistry and mineralogy of all potential fill and foundation materials, groundwater, surface water and gases should be obtained. This should enable prediction of the likely geochemical reactions to be made, which can then be verified by monitoring fill, drainage water and gas composition during construction and operation. It is much harder to assess problems which arise during construction if little or no pre-construction chemical or mineralogical data is available.

14. The nature and scope of the testing will depend on the materials involved. An essential requirement is high quality engineering geological logging of all

exposures, trial pits, cores and soil samples. Correct assessment of weathering grade and identification of secondary minerals are particularly important. Geochemical problems are very often due to secondary minerals such as calcite, pyrite, gypsum, jarosite, zeolites and clay minerals. Methods of rotary coring should be specified to ensure that these minerals are not destroyed during drilling. The use of triple tube core barrels and foam or mud flush may be required.

15. The sample description should be backed up by an appropriate programme of chemical and petrographic analyses. For igneous rocks such as basalt, this may involve making thin sections and examining them in a polarising microscope as well as carrying out X-ray and chemical tests. Fookes et al (ref.4) give a review of proposed petrographic indices for assessment of unsound materials. For clays and mudstones X-ray diffraction tests to determine the clay mineralogy are essential. Standard chemical tests such as pH, sulphate and carbonate should always be carried out in sufficient numbers to determine the chemical characteristics of the materials. Sulphide or total sulphur tests should also be carried out. Details of test procedures are given in BS1377 (ref.10), BRE Digest CP2/79 (ref.11), Head (ref.12), BS1047 (ref.13) and Taylor and Spears (ref 14).

16. The chemical composition of natural materials is often highly variable, because of the irregular distribution of secondary minerals. Sufficient tests should therefore be carried out to assess this natural variability. One method of achieving this is to carry out intensive chemical testing in a small number of trial pits or boreholes, combined with more general testing at a much lower frequency. This enables a geochemical profile of the material to be built up and zones of active and inert material to be identifed. This approach was adopted for the site investigation for the redesign of Carsington dam in 1987. The proposed fill materials are highly weathered mudstones of Carboniferous age and were thought to contain high concentrations of sulphates and sulphides. Testing was carried for pH, total and water soluble sulphate, sulphide, carbonate, and organic matter. A profile from one of the trial pits is shown on Fig. 1. Sulphide and carbonate were found to be very low throughout the weathered strata, but the sulphate content was high and the pH acidic. The sulphate was in the form of jarosite, visible as a yellow-green powder on bedding planes and joint surfaces. Only the

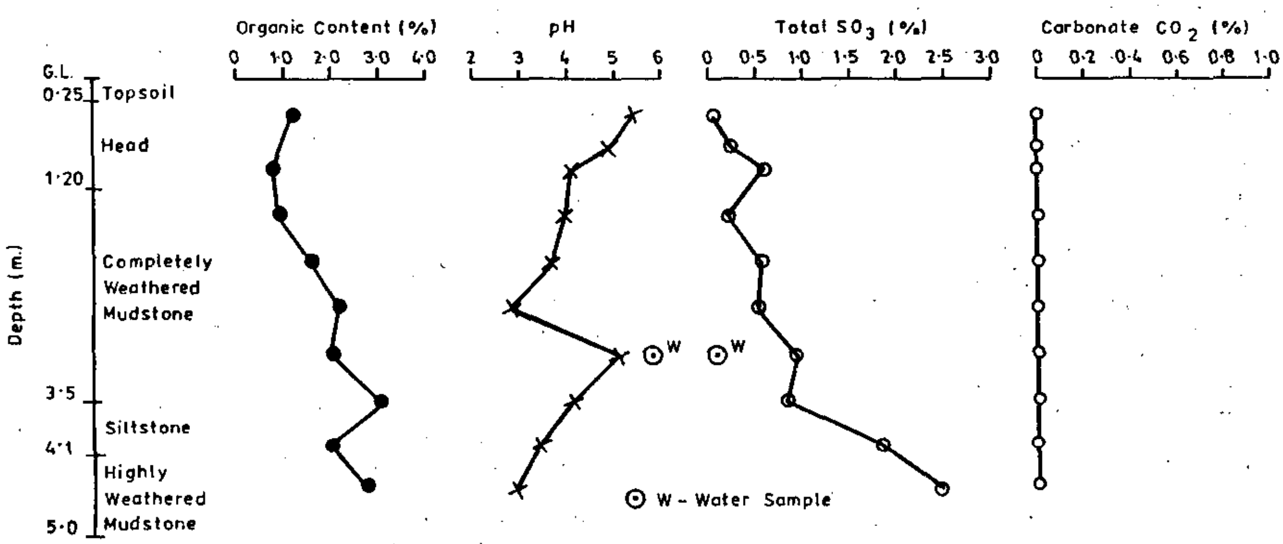

Fig.1: Geochemical Profile of Trial Pit, Carsington

surface layers of head and residual mudstone were chemically inert, the jarosite having been further oxidised to limonite and goethite.

17. Physical and mechanical tests can also be used to assess the susceptibility of fill materials to degradation. Fookes et al (ref.4) proposed rock durability indices based on point load strength index, magnesium sulphate soundness test, water absorption, specific gravity and modified aggregate impact value. These indices are of great value for assessing the durability of rock fill in engineering structures. However they do not give any information on chemical properties of the fill and should not be used as a substitute for chemical analyses.

18. Most site investigations for dams involve the construction of trial embankments. These provide excellent models for the final structure. The chemistry of the fill materials and, particularly, the seepage waters from the embankment should be closely monitored. As trial embankments are generally left unsealed and free standing, they present an environment which is very favourable to accelerated weathering reactions. If geochemical degradation is going to be a problem, a trial embankment should give some indication of it. At Roadford dam, trial embankments were constructed from local fresh to moderately weathered mudstone and sandstone of Carboniferous age. Seepages from the embankments were found to be acidic with high concentrations of sulphate, iron and manganese. Chemical and mineralogical analyses of the embankment fill indicated that some oxidation of pyrite and formation of jarosite had taken place. Similar seepages were observed from the trial embankment at Carsington.

19. Simulated weathering tests can be carried out on fill materials in the laboratory in an attempt to obtain more quantitative data on anticipated rates of weathering and their effect on the geotechnical properties of the material. These tend to be expensive and are often inconclusive. One of the main difficulties with such studies is the difference in physical and chemical mechanisms operating in a small, closed system in the laboratory compared to those operating in a dam embankment. In-depth studies are best carried out as research projects over a period of at least a year and require appropriate resourcing. Shorter, semi-quantitative tests may give useful guidance on the likely behaviour of the materials. Some examples are given in paragraphs 20 and 21.

20. At Colliford dam in Cornwall, the embankment was constructed with sand waste from china clay workings. This material contained about 12% of kaolinised feldspar, and there was concern that this might degrade to clay during the lifetime of the dam. Accelerated weathering tests were carried out by passing hot water continuously through columns of the fill and monitoring the leachate. The results indicated that some degradation was taking place, but the reaction tended to be self-limiting; as more feldspar was converted into clay the permeability decreased and so did the rate of the reaction (ref.15).

21. At Roadford dam, Devon, simulated weathering tests were carried out on the mudstone fill by leaching with peaty water and acid and measuring the resulting changes in chemistry and mineralogy. The results were used as a guide to the likely long term properties of the fill assuming full degradation of the material. At Carsington dam, where the mudstone fill was known to be acidic, tests were carried out by Sheffield University Departments of Geology and Civil Engineering in collaboration with Babtie Shaw & Morton to assess the effect of acid leaching on the shear strength of the material. A slight decrease in shear strength was noted, but this was dependent on the density of the sample. There was a decrease in the grain size of the material during the tests. Acid leaching tests were also carried out on site at Roadford Dam to determine the effect on the filter material of acid leaching from the embankment fill. Leaching with sulphuric acid over a period of 48 hours was found to produce a very slight decrease in the permeability of the filter and a weight loss of 1%.

DESIGN

22. Having obtained the required information, due allowance for the effects of geochemical degradation must be made in the design. For some effects, such as sulphate and acid attack on concrete and other construction materials, guidance is available from standard publications (refs 16-18). For others the criteria are less clear cut and some degree of interpretation is necessary.

23. A number of reactions may affect the density and permeability of fill material. For example, solution of calcite or other minerals may decrease density and increase permeability. Conversely, precipitation of gypsum following pyrite oxidation and reaction of the acid with calcite results in a volume increase of over 100% and can

generate significant heave pressures (ref.3). Several examples of this phenomenon have been reported in the U.K. recently on the floor slabs of buildings (refs 19,20). Nixon (ref.21) gave guidelines for potentially troublesome shales viz; (1) total sulphur content in excess of the acid soluble sulphate content, and (2) acid soluble calcium content not less than 0.5%. The extent to which these reactions take place depends critically on the availability of air and the rate of seepage through the fill.

24. In general the effects of geochemical degradation are to cause accelerated mechanical breakdown of the fill, leading to a decrease in grain size, permeability and shear strength. This can be accomplished by acid attack on cementing materials or clay minerals, or simply by uptake of water by certain clay minerals; the so-called "swelling clays" such as montmorillonite, vermiculite and mixed layer illite - montmorillonite intergrades. The mudstones at Carsington, for example, break down rapidly an exposure to moisture. X-ray diffraction tests revealed that mixed layer illite - montmorillonite accounts for about 40% of the clay fraction. At Roadford the clay minerals consisted dominantly of illite, kaolinite and quartz, and the mudstones are much more stable when exposed to moisture.

25. The effects of weathering processes on the shear strength of Coal Measures mudstones has been studied extensively by Taylor (refs 2,3). He gives lower bound peak shear strength parameters of $\phi' = 22^\circ$ and $c' = 0$ for loose tipped colliery spoil heap material, and suggests that this is similar to the fully weathered peak shear strength of the weakest Coal Measures mudstones and clays. However, he highlights the fact that intense chemical weathering of the surface of a spoil tip over a 50 year period had only reduced ϕ' by about 1.5°. Skempton and Coats (22) stated that at Carsington dam deterioration of the mudstone fill used in the dam shoulders had caused a decrease in 0' from about 28° to 25°. This was thought to be largely due to breakdown of the fill by absorption of water by swelling clay minerals.

26. If laboratory simulated weathering tests have been carried out, it may be possible to determine the likely drop in shear strength directly from them. In addition some allowance for deterioration based on comparison of the shear strength of fresh and weathered material from the potential borrow area can be made. This dual approach has been adopted at Colliford (ref.15), Roadford and Carsington, the appropriate parameters for the main shoulder fill materials being indicated on Table 1.

CONSTRUCTION

28. Within the tender documents for a project it is important that potential problems caused by chemical degradation of the fill material are clearly set out for contractors in order that their tender price adequately reflects the difficulties of dealing with such material. It is equally important that these are explained concisely to the Client beforehand. For example, for the Carsington reconstruction contract the Contractor priced the items for pollution control at around £900,000, some 5% of the tender total, even though extensive pollution control lagoons were already in place. There are three main aspects which should be considered during the construction stage; namely safety, pollution control and fill workability.

29. Safety - Leaching of minerals from the fill can result in gaseous emissions. These may be toxic or explosive in their own right, or simply lead to an oxygen deficiency in confined spaces on the site. Such confined spaces include trial trenches excavated in the fill for testing purposes as well as manholes in the drainage system. Monitoring for gases should be carried out in all boreholes and confined spaces; many of the more common geochemical reactions generate carbon dioxide, and methane and hydrogen sulphide may also be produced within the fill or migrate upwards through the foundation in certain strata, in particular mudstones. Good design, for example by limiting manhole depths, should help to mitigate these problems. However the contractor must be made aware of the dangers in order that he can take the necessary safety precautions.

Table 1. Examples of allowances made for changes in shear strength due to degradation.

Dam	Shoulder fill type	Design shear strength parameters			
		Newly won fill		Degraded fill	
		c': KN/—²	ϕ'	c': KN/m²	ϕ'
Colliford (ref. 15)	China clay sand waste	0	39°	0	35°
Roadford	Moderately weathered to fresh sandstones and mudstones				
	(i) Low normal stress	0	40°	0	35°
	(ii) High normal stress	0	32°	31	29·5°
Carsington	Highly weathered mudstone	10	22°	10	22°
	Moderately weathered mudstone	15	25°	15	24°

30. Pollution Control - Pollution of downstream watercourses by high suspended solid loads has long been recognised as a problem during embankment construction and it has been good practice for many years to install settlement lagoons to deal with this. Geochemical changes in the fill and on borrow area surfaces can result in water which is acidic in nature and has high metal concentrations from underdrainage systems and from rainfall run-off. This must be dealt with possibly by chemical dosing and settlement, if the run-off is of unacceptable quality to discharge to the river. At Carsington, caustic soda and lime dosing facilities were found to be required together with large settlement lagoons. Prior consultation with the authority responsible for monitoring discharges should enable parameters to be included in the contract documents for water quality immediately downstream of the site compared to that upstream. The contractor is then responsible for installing temporary works to ensure these parameters are met. Alternatively the Employer can design and operate pollution control facilities, hence relieving the contractor of the risk of system failure provided he diverts all site run-off through the facilities.

31. Fill Workability - Geochemical reactions can result in rapid changes in material properties with the effects of air and moisture on the fill. This is further accelerated by vehicle movement, and it therefore has to be made clear in the contract documents that for the more susceptible fill materials, operations must cease whenever rain threatens.

32. During construction, procedures should be established to monitor the physical and chemical properties of the material in the borrow areas, the fill placed in the embankment, the quality of run-off and drainage water, and the presence of gases in relief wells, manholes and other confined spaces. This will enable a check to be made against design assumptions. Such information will also be required to enable decisions to be taken on the need for treatment in the post-construction stage. A database of results can be established to enable trends to be reviewed and long-term monitoring procedures and frequency adjusted to suit.

IN-SERVICE CONDITION

33. Once the dam has been completed, and responsibility for operating the reservoir has been taken over by the Client, continued monitoring will be required to determine if geochemical reactions are continuing, or have started to occur, within the fill. Monitoring of drainage flows is good practice on any dam, both during initial filling and subsequent operation, and is regarded as one of the best means of highlighting problems with the structure at an early stage. Similarly, monitoring of drainage water quality is the best means of establishing the rate of geochemical change within an embankment. It should be carried out at frequent intervals from commencement of impounding until the mineral composition of the drainage water becomes stable. Water quality analysis should be continued thereafter, but on a less frequest basis, to monitor any long-term changes.

34. If the pattern of usage of the reservoir changes at any time, for example by the transfer of water from a different catchment to supplement yield, the frequency of water quality analysis may have to be increased to monitor the effect of this new source. Similarly the testing frequency should be increased if there are major fluctuations in water level, such as could occur in a drought situation.

35. The drainage waters emanating from the embankment may be of too poor a quality for discharge to the downstream watercourse. Some form of treatment may therefore have to be installed. The cost implications of this can be considerable and so an attempt should be made to assess at an early stage the period for which treatment will be required in order to provide the most economic solution for what is likely to be a temporary condition. The chemical testing carried out on the fill during construction coupled with the results of water quality testing can be used to obtain an approximate estimate of rate of leaching of minerals from the fill.

36. It is not possible to be precise about the form of treatment required to deal with geochemical pollution as this will depend on the nature of the fill and the quality of the watercourse into which the discharges are to be made. The economics of each situation will have to be assessed. However typical examples that have been considered for some sites are:

(a) trickle discharge over wooded hillsides downstream of the embankment

(b) pumping back into the reservoir and relying on dilution

(c) full treatment in a 'package' skid-mounted plant

(d) utilisation of downstream pollution control facilities installed for embankment construction purposes.

37. Operations staff must be made aware of the potential dangers from gases, and safe

entry procedures for manholes, tunnels etc. should be clearly set out. Wherever possible gases should be allowed to vent naturally. However manholes may require to be purged using small mobile plant, and a permanent forced ventilation system is likely to be required in all tunnels and galleries.

CONCLUSIONS

38. Considerations of the geochemistry of the fill material should be an inherent part of embankment dam design. The financial implications of geochemical degradation can be considerable. Embankment side slopes will require to be flatter to allow for lower long-term shear strengths and measures will have to be taken both during and after construction to control pollution of watercourses. The design of concrete structures and specification of imported materials should take cognisance of the potential for aggressive groundwaters.

39. Of prime importance in predicting long-term geochemical behaviour is the collection of data from chemical testing of the fill and drainage waters at all stages from initial site investigation through to construction. Testing and analysis should continue throughout the operation of the structure until a stable environment is achieved.

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29. Reservoirs - a legacy of opportunity

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Whilst reservoirs are a vital management tool for the water industry, their construction and management impinge upon the local built and natural environment. A wide range of impacts have been identified in this paper both relating to the reservoir's primary use and any secondary uses which may result. Both direct and indirect impacts are outlined and a discussion of mitigation measures is included.

Introduction

The impoundment of water, forming reservoirs behind embankment dams, is necessary in both developed and developing nations. Reservoirs are an important water management tool, supplying water for domestic and industrial use and regulating water availability. In the UK the equivalent of 350 litres of water is added to the public supply per head per day, of which 270 litres per head per day is actually delivered to customers. Of this, half the water is delivered to households (135 litres per head per day) and half to industry. Reservoirs are especially important where groundwater is of inferior quality or is unavailable.

While reservoirs are clearly necessary, the environmental implications of their construction and management, their effect on the local community and economy, and the potential for secondary usage and development cannot be ignored.

A wide range of impacts can be identified. These impacts can be classified as short or long term, primary (direct), secondary and tertiary (indirect). They can be due to the construction operation itself, changes in site afteruse (including secondary use and secondary development) or more indirect impacts such as change in groundwater level and hydrological gradients.

Impacts

The impacts of dam and reservoir construction can basically be divided into two clearly distinguished classes, those of short-term construction phase impacts, and those of longer term, post-construction phase, impacts. These are discussed in the following sections. However, before discussing these it is important to consider what we mean by 'Environment'. Do we only mean aspects of the natural surroundings such as ecology and geology? Do we include aspects of the site such as rights of way, land use and human perception of the site? Or, do we include financial aspects such as land value, effect on the local economy and secondary development? These are often investigated separately as Environmental "habitat" assessment, social impact and economic impact. The inclusion of social and economic aspects often dramatically affects the cost benefit equation of many developments, including reservoir and dam projects. However, the Chambers English dictionary defines environment as "external conditions influencing development or growth of people, animals or plants". Indeed the EC Directive specifically states that impacts on the human environment should be included (1). Thus environmental assessments should investigate all aspects of impact. One should remember that impacts need not always be adverse. Benefits of dam and reservoir construction should be included in any EA.

Short Term Impacts

These usually tend to occur during the construction phase and relate to the construction operation itself together with associated social and economic changes. Impacts due to aspects of the construction operation itself include landscape and land use changes, noise, vibration and transportation impacts, and temporary disruption of ecology. Landscape changes include landscape alteration and consequent visual impact. The visual impacts of embankment dam construction are not as great as those of concrete dam construction. However, they are often the most obvious to the local populace during the construction phase. Where relocation of residents in the lands to be flooded or local opposition to the scheme has occurred it is the visual changes in topography that induce most of the social problems experienced.

It is often necessary to restrict access to operating areas of the site during construction in the interests of public safety. This alters land use and may interrupt public rights of way. Access routes also often sever public rights of way such as footpaths or bridal ways. Alternative routes must be provided and well signposted. It is advantageous if these alternative routes are those which will be used post construction ie they do not cross land to be flooded. This avoids having to change routes more than once. This type of disruption is caused not only by the construction site itself but also 'transit camps', stores and vehicle parking and maintenance areas. These operations also create changes within the land use itself and may lead to the severance of agricultural lands belonging to a single farm.

The construction phase involves much machinery movement, releasing energy into the environment in the form of noise and vibration. Transportation of men and materials to the site spread these effects away from the construction site itself. Poor site access and insufficient road capacity often cause problems which directly affect the local population which may already be 'sensitised' by the development. Such problems must be mitigated wherever possible.

Disruption of the local ecology often occurs during construction, either due to release of components into the natural environment, such as particulates into local water courses, or severance of habitats. Disruption of water flow in local streams, or release of components such as particulates or oils,into local water courses can cause dramatic effects on the 'downstream' ecology, either by restricting oxygen exchange at the water surface, cutting down photosynthetic efficiency by particulate settlement on aquatic plants leaves, direct phytotoxic effects, or changing the flow regime and thus the stream bed substratum.

The influx of men and money into a local community often causes short term social and economic effects. There is frequently a requirement to house many workers in the local community. Temporary manual work is often available to the local unemployed, and there is always the need for goods and services for the construction operation and men employed on site. This can cause problems, especially in small communities.

The last group of temporary impacts relate to the reservoir filling operation itself. Progressive land take and disruption in the downstream water flow regime are the most important effects on the natural environment. The sudden removal of construction workers from the local community itself, can cause as many problems as those created by the intial influx of workers at the beginning of the construction phase.

Long Term Effects

Long term effects relate to the after-use of the site rather than aspects of construction itself, and remain long after the reservoir is constructed and filled. The most obvious impact is land take. Reservoirs remove large tracts of land from their previous use. The filling operation also causes changes in the groundwater regime of lands bordering the new reservoir which influences the land use options on adjacent lands. Many reservoirs are also used for leisure activities such as sailing and fishing. The leisure uses often encourage secondary development of adjacent lands for club-houses, restaurants, holiday facilities, marinas and boat yards with all of their associated infrastructure.

The ecological impacts may be diverse. Changes in water flow and nutrient status of the water (by precipitation of dissolved ions, settlement of particulates and utilisation of nutrients by the lake's flora and fauna) can substantially alter the downstream ecology of the watercourse. Construction may also disrupt fish migration routes and introduce species not previously associated with the area. Changes in the groundwater regime adjacent to the site may cause vegetation and consequent faunal changes within the lakes littoral zone. Lastly the infrastructure associated with the reservoir and any secondary development may cause severance of habitats or disruption to terrestrial flora and fauna.

The increase in use of a particular area due to the reservoir/dam construction may induce the local authority or statutory conservation agency to place statutory ecological designations (eg Site of Special Scientific Interest status in the UK) on that area for protection of the local ecology. The increased use will also lead to increases in noise and transportation effects and permanent changes in the local economy to service any secondary development that takes place.

Reservoir management techniques to prevent eutrophication or mitigate silt accumulation will also affect the environment by release of particulates and/or potentially toxic chemicals into the environment. These will also lead to increases in traffic and noise impacts. Toxic chemicals may also be released by any secondary use of the site eg sailing (eg TBT paints now banned on small craft) or secondary development (solid and liquid wastes).

Mitigation

The mitigation strategy should consider, carefully, each aspect of reservoir construction and operation balancing environmental susceptibility with the control measures that would need to be implemented.

The mitigation process is facilitated by a comprehensive review of existing data and of the nature and characteristics of the reservoir. Where adverse impacts are apparent, mitigation measures for minimising and/or offsetting these impacts, and opportunities for enhancing natural environmental values can be explored.

It is essential to understand the utilisation, alteration and impairment of natural resources affected by the reservoir so that mitigation measures can be proposed and incorporated during the reservoir design process.

Changes in microclimate in the vicinity of the reservoir should be reviewed in terms of humidity levels which may affect insect populations. An analysis should be made of the hydrologic regime of the stream or river systems to be modified by the reservoir. During this review, it is essential to assess the overall impact on the monthly and annual mass water balance so that potential changes in the hydrological balance can be identified. Likely modifications to average and seasonal water quality should be detailed for both the reservoir and the downstream flow(s).

An assessment should be made of the effects on groundwater quantity and quality in the vicinity of the reservoir and downstream, alterations to water table, wells/aquifers, and infiltration rates in the watershed.

Sediment influx from watershed runoff and downstream erosion, tectonic/seismic activity, mineral resources, physical and chemical weathering, landslide and subsidence characteristics should be considered.

A reservoir development may have effects on soil erosion in the watershed, slope stability, bearing capacity and settlement/heave and soil structure.

The assessment should include details of the impact on fauna and flora in the watershed area above the reservoir and in the downstream zone(s), including those caused by associated developments such as access roads. Inundated fauna and flora may need to be rescued, relocated or re-established in new areas free from encroachment.

Anticipated physical, chemical and biological changes within the reservoir may affect the water column (nutrient trapping and thermal stratification) and in the benthos (animal communities living in or on the reservoir bottom sediment). Existing fisheries may suffer as a result of changes to the quantity and quality of downstream flow directly or indirectly during migratory periods. Mitigation measures may need to be implemented to prevent overfishing or problems of storage, processing and marketing of fish. An altered fishery may have implications for local nutrition and diet.

Any potential for the growth of weeds in the reservoir would need to be considered as should the uses of the drawdown zone for agricultural and other purposes.

The mitigation plan should consider and identify the impacts resulting from flood control, including potential reduction in flood damage and reclamation of lands for agricultural use. The effects of the reservoir on navigation such as those related to low flow, proposed transhipment facilities and landing facilities may require mitigation and have wider implications for land use patterns and land capability. For instance, downstream aquaculture could be improved by low flow augmentation.

The social/economic conditions of the local population in the region affected by the reservoir should be examined in order to identify the need for mitigation in terms of welfare, new roads and new industries. Mitigation plans for managing the resettlement of populations (wholly or partly), provisions for rehabilitating families in their new living/working conditions and alternative choices for resettlement (rural and urban) need to be assessed. If resettlement necessitates changed agricultural methods (eg growing upland crops, use of irrigation), the mitigation programme should contain provisions for assisting the resettlers.

The mitigation study should assess the degree to which the reservoir may affect the likelihood of contracting water-borne disease and should propose corrective measures. An assessment should also be made of plans for adequate community sanitation, both to enhance quality of life and to minimise pollution of the reservoir. The study should assess the possible recreation and aesthetic values of the reservoir and the plans for effective development of its recreational potential, such as the planting or replanting of borrow areas and other disturbed areas. Measures for any inundated archaeological, historical and cultural sites should be assessed and the appropriateness of salvaging or preservation schemes be explored.

Secondary Uses

The 1980's saw the "Water for All" decade and positive efforts to improve water availability for use in Less Developed Countries where the main primary uses of embankment dam reservoirs are for hydro-electric power and for irrigation. Approximately one and half billion hectares are used for agriculture worldwide. About 13% of this area receives irrigation water, providing 40% of the crop production.

A secondary use of irrigation dams is the control of the seasonal floods, although such control is not always beneficial. Some populations rely on seasonal flooding for the establishment of flood plain fisheries and have to learn new techniques for lake-fishing of the reservoir to replace their established riverine fishing methods. The type of fish caught often changes also. Annual floods also have the advantage of the deposition of nutrient-rich sediments as the flood recedes from the flood plain, improving the agricultural quality of the land. Also the impoundment of water within a reservoir can greatly reduce the nutrient status and suspended sediment load of river water downstream of the dam. Major changes in the flow regime downstream can have serious effects downstream, as occurred with the Aswan Dam in the 1960's. Reduction in fresh water flow led to the loss of 4000 km of the Nile delta, a vital food producing area and allowed the ingress of saline water from the estuary, destroying a highly valuable sardine industry.

However, irrigation dams can support valuable secondary uses. The creation of Lake Volta resulted in the establishment of very important Tilapia and freshwater sardine fisheries, valuable year round commercial resources.

In the UK the 1973 Water Act enforced the multi-purpose use of reservoirs, provided the secondary uses were commensurate with the primary purpose for the construction of the dam.

In the UK such multi-purpose uses are actually recreational but there are potential conflicts with the primary management aims of the reservoir. The main primary use is the provision of a potable water supply.

Fisheries

A popular secondary use of reservoirs is fishing, but the establishment of fisheries within reservoirs can cause management problems. A major component, and expense, in the treatment of potable water supplies is the removal of algae through filtration. Dense algal growths rapidly clog filters. Some algal species produce toxins which may taint the water even after standard treatment processes. Corrective treatment of such 'tastes' and odours is expensive and not always possible. In extreme cases toxins from algal blooms can kill livestock, that drink untreated water. This occurred in the UK in the summer of 1976 and 1989. Fish eat zooplankton, the organisms which normally graze on algae. A good zooplankton population can significantly reduce water treatment costs by 'cropping' much of the algae. The zooplankton are much larger than algae and are removed much more easily without clogging filters. Introduction of fish into a reservoir can therefore increase water treatment costs.

If, through public pressure, or other reasons there is a management policy to maintain a fishery within a reservoir, there are two alternatives: either a coarse or game fishery. Coarse fish species are more suited to high nutrient lowland waters but can cause management problems by shoaling and damaging pumping equipment. The use of ground bait can cause clouding of water, increasing the organic loading and treatment costs. The establishment of a game fishery, usually a 'put-and-take' trout fishery causes fewer problems with respect to reservoir management. But the lobby in favour of coarse fishing is very strong within the UK.

Coarse fishing attracts a very different consumer to that for game-fishing. It is a much more social activity with competitive matches, whereas game-fishing tends to be a more solitary occupation. A different management strategy is required for the 'consumer' as well as the game fish stock itself. A lowland reservoir is not a natural habitat for game fish such as trout which will need frequent restocking. The high cost is generally reflected in licence fees, again emphasising the different type of consumer attracted and the management approach required.

Sailing

Reservoir sailing has become extremely popular within the UK. Generally only sailing boats are encouraged as potable water supply reservoirs are often too vulnerable to pollution from oil and fuel spillages from power boats. Reservoir sailing can be seasonal, restricted to the winter months with boats used elsewhere in the summer and requiring thorough cleaning before they are allowed to be returned to the reservoir.

A large reservoir may support simultaneous uses of both fishing and sailing if spatial separation is possible. Alternatively temporal separation is used with summer fishing and winter sailing.

Nature Reserves

A general reduction in the extent of water bodies within the UK over the last 50 years or so has encouraged the use of reservoirs as wild life sanctuaries, especially for waterfowl. The establishment of new reservoirs which can provide good waterfowl habitats is often actively encouraged by the nation's "birders" who have an extremely strong lobby. The potential for the multipurpose use of reservoirs can be pushed very succesfully at planning enquiries when such secondary uses are proposed. Nature reserves tend to be restricted to the shallows of the reservoir, which are unsuitable for other secondary uses and thus give the undisturbed conditions necessary for wildlife. Conflicts with the primary management aim of the reservoir occur due to the seasonal drawdown of the reservoir - a problem which has been very successfully solved at Bough Beech in Kent where small retaining bunds maintain shallow pans of water throughout the year. The provision of islands, for breeding pairs of fowl, also provides some protection against egg collectors.

Sightseers

Day trippers or sightseers have been found to remain very close to their cars. Habitat erosion and disturbance can be prevented by channelling such visitors to 'honeypot' sites where lookout posts are provided with information boards, picnic sites, toilets and parking facilities.

A major source of friction in the multiple recreational use approach is between ramblers and all other users. Dedicated footpaths, well signposted, with suitable information boards etc will keep ramblers away from other users.

One of the largest secondary multi-use reservoirs in the south east UK is Datchet which successfully integrates year round sailing, a put-and-take trout fishery and night-clubs and restaurant facilities.

With the privatisation of a water industry it is anticipated that profitable secondary uses of reservoirs will continue. It is very much hoped that the less profitable secondary uses such as waterfowl reserves will not become less attractive propositions.

Concluding Remarks

Reservoirs are important in water management terms. They provide essential supplies of water for domestic and industrial usage yet the environmental implications of their construction and operation cannot be ignored. Awareness of the potential short and long term impacts of reservoirs can assist in incorporating mitigation strategies so that the benefits of reservoir development are not eroded by adverse effects. Subsequent, secondary use of the reservoir can enhance the value of the development in addition to that of the primary purpose of potable water supply. Existing reservoirs provide a legacy of opportunity whilst those at the planning or construction stage can have their realisable benefit programmed into the reservoir development programme.

Discussion

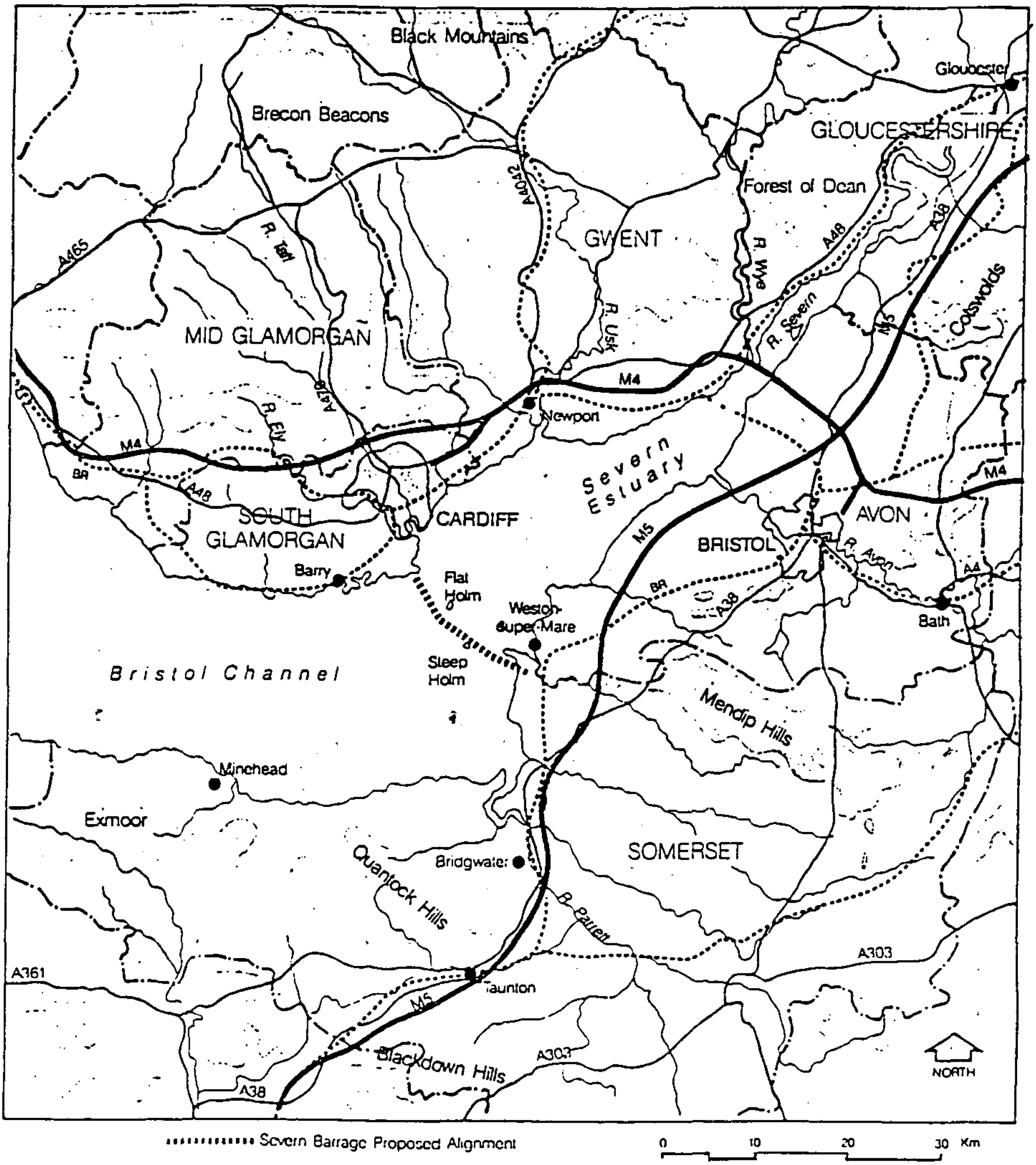

Fig. 1. The Severn Estuary region

H.J. MOORHEAD

The Authors of paper 29 are to be congratulated on producing a paper which gives an excellent overview of the main environmental impacts and effects of the construction of a dam and the impoundment of water. The paper is a good starting point for consideration of the environmental effects of dams. Saying starting point, does not in any way intend a criticism of the paper. The subject is a complex one and involves many disciplines and there is difficulty in communicating across the inter-disciplinary boundaries. The paper indicates what has to be covered. The following diagrams indicate how a particular environmental study was organised using the recently completed Severn Tidal Power Studies as an example.

Figure 1 shows the extent of the area which would be affected by a tidal barrage on the Cardiff-Weston alignment, and Figure 2 shows the effect of the barrage operation on the tide levels where it will be seen that in the with-barrage situation the level will go from normal full tide level to half tide level with consequential environmental effects. Figure 3 shows the organisation of the studies and it will be seen that a separate co-ordinator was appointed for Tidal, Environmental and Regional Aspects. Figure 4 shows the links between the Environmental Studies and other project aspects. The central role of hydrodynamics, sediments, water quality and salinity can be seen leading to two main streams of environmental work:

- the Natural which comes down through the food chain and also includes geology and geomorphology, and
- the Anthropogenic which runs down the human related aspects as shown.

Finally all are drawn together in the Appraisal Assessment to produce the Environmental Statement. In this particular case the results are being made publically available so that there can be a full debate on the Environmental Effects.

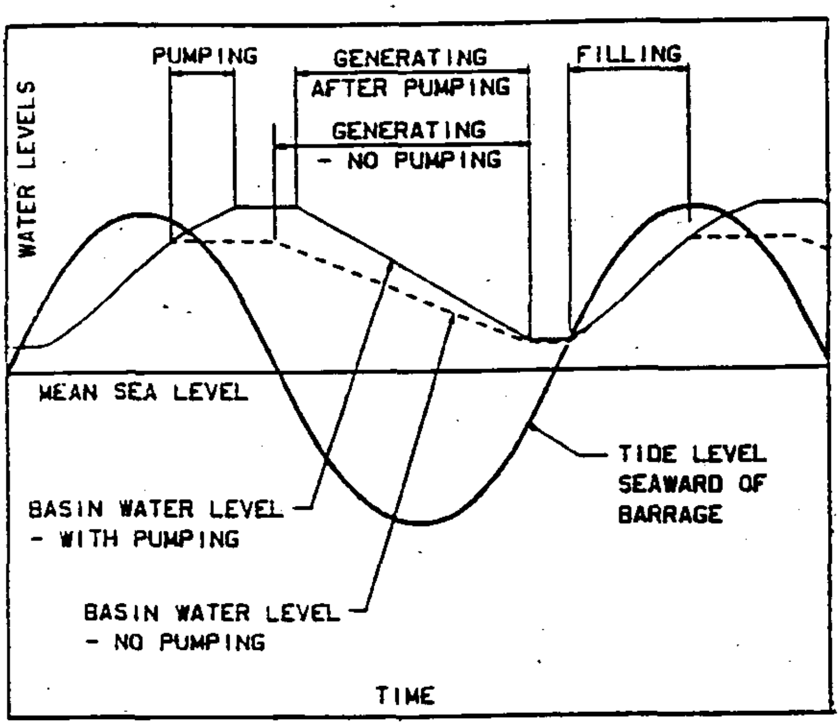

Fig. 2. Estuary and water level basins

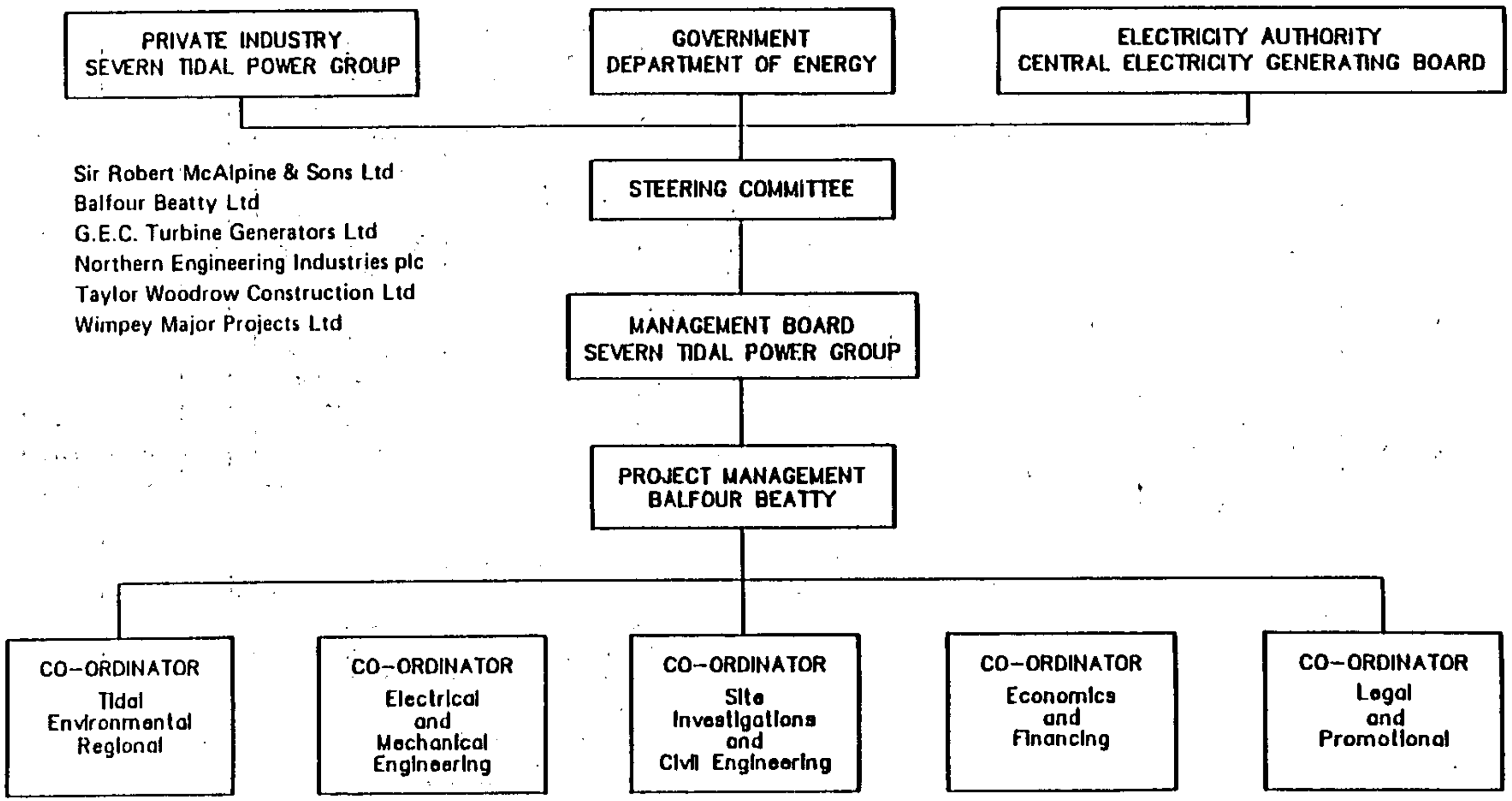

Fig. 3. Project organization

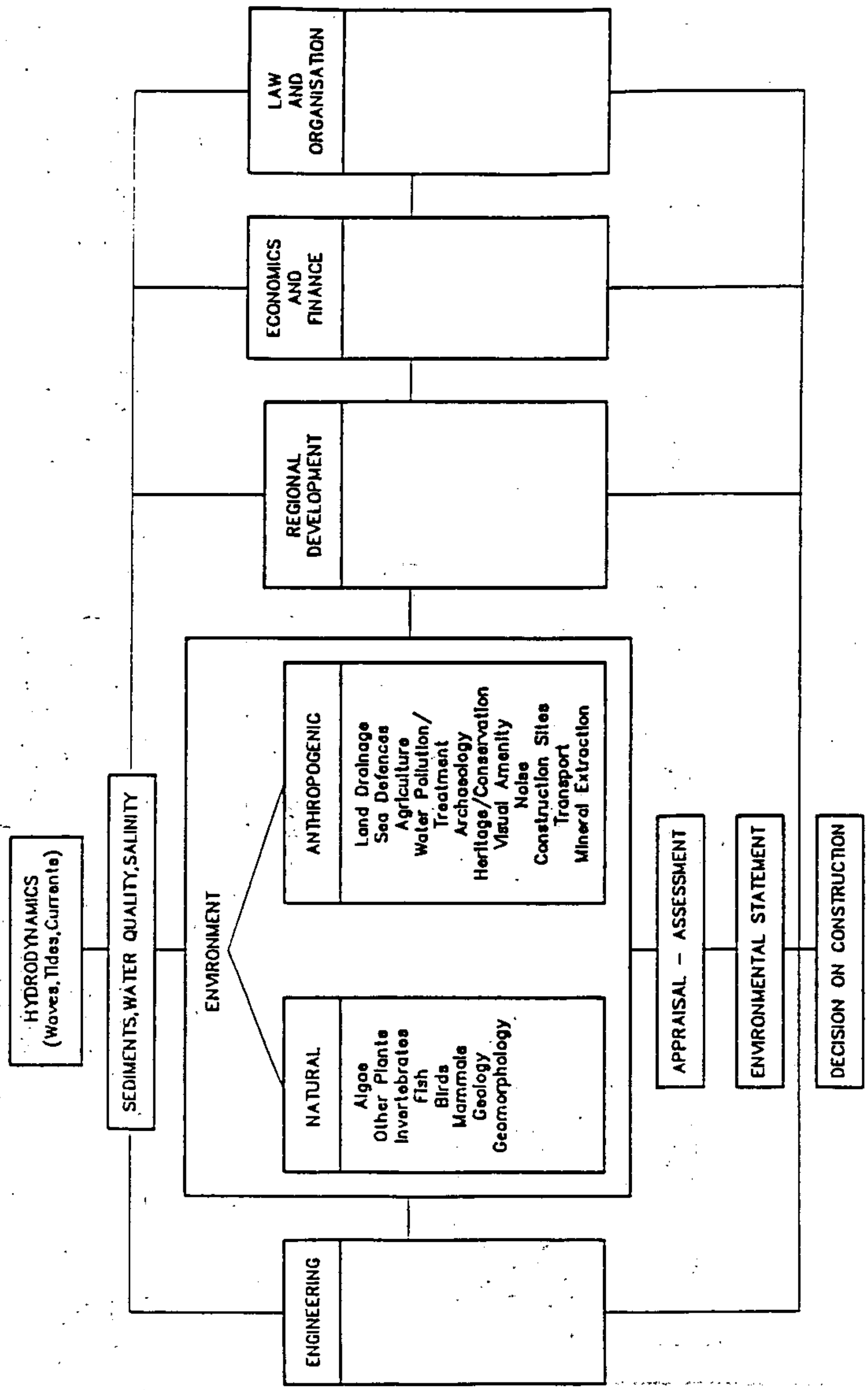

Fig. 4. Links between environmental studies and other projects aspects

M. BRAMLEY (R & D Co-ordinator, National Rivers Authority)

With regard to Paper 25, it is worth adding some further words about work carried out in the US on the subject of dam overtopping and the protection of embankments and auxiliary spillways from erosion. It needs to be mentioned that George Powledge, the author, chaired the recent ASCE Task Committee on Mechanics of Overflow Erosion on Embankments.

The Group's two reports are commended. Both were recently published in ASCE Journal of Hydraulic Engineering (their second report is quoted as Reference 6 to Paper 25, their first report on "Research Activities" in the same ASCE volume). Reference 26 in particular provides a useful tabulation of American overtopping events at dam and levee embankments. Both reports also draw on UK information exchanged with UK engineers during the DTI-sponsored Overseas Scientific and Technical Expert Mission (OSTEM) mission with which Michael Kennard and I were involved in mid 1988.

Further information from the US on protection and provision for safe overtopping of dams and flood banks is given in the CIRIA Report on this mission (1).

The paper refers to the Russian design of wedge-shaped protection blocks (Figure 4b) whereby the stepped upper surface provides enhanced stability to the protection. The six stepped block service spillways which have been built in Russia over the past 10 years have all performed satisfactorily. However it is worth noting that in cases the stepped surface has been achieved using standard precast concrete road slabs (generally of length in direction of flow about 1m and thickness about 150mm). These are placed in a "stair-step" fashion on a hand-trimmed drainage layer. This simplification has enabled economies to be made in the manufacture of the otherwise complex block shape.

With regard to Paper 29, the authors are congratulated on a useful review of the environmental issues associated with reservoirs and reservoir construction. It may be beneficial for dam engineers, who may feel that environmentalists regard all effects of reservoir development as negative, to reflect on the very positive interaction which has now been developed between engineers and environmental interest groups developed in flood defence and land drainage works. Here the conservation officer or environmental interest groups have progressively been involved earlier and earlier in the planning of the works. This is not simply to address environmental assessments, but also to identify the environmental parameters which - alongside engineering and economic parameters - must form the basis for the design.

It is important to emphasise that positive impacts occur, particularly through the substantial amenity value which water space now provides. At Milton Keynes, property and land values in the vicinity of the flood storage reservoirs are considerably higher than at distance.

Reservoir water quality is becoming an increasingly important issue, particularly with the recent emergency of problems of euthrophication and algal blooms. Reservoir engineers are likely to become increasingly involved in mitigation strategies, not only for new reservoir development but also for existing reservoirs, which enable reservoir water quality to be managed and controlled. Such works might, for example, involve improved mixing, aeration, draw-off or dosing facilities.

Finally, it is important to put the reservoir within the context of the catchment as a whole and recognise that problems of conflicting interests in water use are prevalent throughout. In England and Wales, the NRA is now developing the Catchment Management Plans as the basis of its multi-functional water management responsibilities. These involve water quality, water resources, fisheries, conservation, flood defence, navigation and recreation interests. The Catchment Management Plan, and the related designation of use- related environmental quality objectives for controlled waters within the catchment, should in the medium term provide a clearer framework within which environmental assessment of reservoirs is carried out in England and Wales.

Reference

1. Protection and provision for safe overtopping of dams and flood banks; Report to Department of Trade and Industry on OSTEM mission to USA, Project Report 2, November 1987. CIRIA, London.

G. STEPHENSON (Research Manager, CIRIA)

CIRIA has recently started work on two projects that will be of interest to dam's engineers. They are:-

The Performance of Wedge Shaped Blocks in High Velocity Flows (Stage 2)

The Performance of Block Work Protection for Dam Faces

The first project on the performance of wedge shaped blocks for spillways, has been mentioned in paper 45 of the proceedings and the first stage of the work was more fully reported in the July 1989 edition of Water Power and Dams. (Pravdivets and Bramley)

The original concept behind the stepped block was developed in Russia by Dr. P.I. Giordienko of the Moscow Institute of Civil Engineering when looking into methods of protecting erodible surfaces subject to high velocity flows. It was further developed by Professor Yuri Pravdivets of the same Institute into a practicable workmanlike solution. Eight successful stepped block spillways have been constructed in the USSR in the past decade.

Outside of the USSR relatively little interest had been shown in the concept prior to the CIRIA study, most probably because of a lack of first-hand understanding and experience in their use. The CIRIA project came about because of interest by USA and UK engineers in low-cost methods of upgrading existing spillways using new design concepts. Stage one of the study concentrated on:

- Demonstrating the potential benefits and mode of functioning of the wedge-shaped blocks.
- Setting out a design methodology.
- Providing the information necessary to undertake stage two.

The work undertaken included:

- A review of published data.
- Assembling of existing practical knowledge.
- Model testing.
- Outline specification for the design guide.

Work already undertaken by Salford University and Hydraulics Research Ltd. under a sub-contract to CIRIA has shown that the blocks show considerable advantages over more conventional methods of chute construction. In particular:

- The upstream edge of the block is shielded from potentially dirsuptive flow stagnation pressure, which can give rise to extreme lift and drag forces on a protection system.
- The flow pattern produces a low-pressure separation zone downstream of each step. This zone is connected by drainage vents to the underlayer and controls the build-up of seepage flow.
- The block shape is inherently stable. If any block moves perpendicularly off the slope (either by lateral displacement or by rotation about one end), the sloping upper surface experiences a stabilising downthrust.
- The stepped upper surface has a high roughness which helps to dissipate the energy of the flow and reduce flow velocity, hence reducing the amount of energy to be dissipated at the tailwater or toe.

Initial work on the design guide (to be completed as part of stage 2) has shown that it is feasible to provide a simple framework within which a competent engineer can develop site-specific solutions. In stage one of the project an outline specification for the guide was set out.

In stage 2 of the project, due for completion late 1991, further laboratory tests at Salford University are planned. These will look more closely at:

1. Pressure distribution, with the data collected on a data logger and analysed statistically.
2. The effect of longitudinal joints on the stability of a panel of blocks, by laying the 17mm average thickness blocks with increasing gaps between them until failure occurs.
3. Block lifting with a 50mm block resting on a micro-switch that detects when the block first lifts. These tests will be conducted with and without interblock restraint on the test block.
4. The force parallel to the slope on a 50mm block resting on rollers with and without interblock restraint.
5. The stability of the blocks in a hydraulic jump and the design of a toe area using all three block sizes. Blocks of a different weight will also be investigated by attaching metal strips to the base of the existing model blocks.

The design guide will incorporate the conclusions from the laboratory and desk studies together with any further information obtained from Professor Pravdivets. Work on the design guide will involve the expansion of the draft included in the Stage 1 report to address the gaps identified in the design approach.

The work is being funded by the Department of the Environment (Water Industry Directorate), United States Corps of Engineers, Severn Trent Water plc and Salford University.

The second project which has just started is a joint project between CIRIA and Hydraulics Research Ltd. The objective of the study is to produce a guide for the use of engineers responsible for the design, maintenance and rehabilitation of blockwork protection against wave attack. Specific phases of the work are:

- Conduct a general survey of blockwork protection to UK dams including present and past practices.
- Examine in detail selected samples, including instances of blockwork movement.
- Derive local wind, and hence wave conditions, at selected sites for past extreme events.
- Calculate conditions for blockwork movement, movement/stability for selected sites using results of previous research and other members for conditions identified above.
- Describe and consider options for improving stability of existing blockwork protection.
- Produce a report to give guidance on best practice in design, maintenance and/or rehabilitation of blockwork protection for dams.

Publication of the final document is due spring 1992 and funding is being provided bv the

Department of the Environment (Water Industry Directorate).

For further information on either project please contact Garry Stephenson at CIRIA, 6 Storey's Gate, Westminster, London, SW1P 3AU.

N. HOYLE (Colquhoun, formerly N.W.W.)
In relation to Robertshaw and Dyke's paper, elbows in embankments, and more especially corners of hillside reservoir embankments, were areas prone to disturbance. Supervising engineers should pay particular attention to such locations. These areas were particularly difficult to cover by dimensional monitoring, due to the difficulty of establishing adequate instrument stations and sight lines.

Had the authors encountered such difficulties and had their monitoring revealed movement at corners?

MR. ROBERTSHAW in reply agreed with Mr. Hoyle's remarks about both the difficulty of surveying and the tendency to movement at corners of embankments. His experience had nevertheless been that both vertical and horizontal movement were largely related to the height of the embankment.

G.P. SIMS (EPD Consultants)
Would the authors please confirm that it is possible to use photographs taken by 35mm or similar cameras for photogrammetric purposes? Can advantage be taken of pre-existing photographs? How would such photographs be used?

In response MR. HOPKINS confirmed that 35mm cameras could be used, but with a much lower precision because nothing is known about the camera. Historical photography can be used, using new photography and identifying features around the site which didn't move and which appeared on the previous photography. These could be used to work backwards and work out some of the properties of the earlier, unknown camera. The results would not be as good, but may be all that is available.