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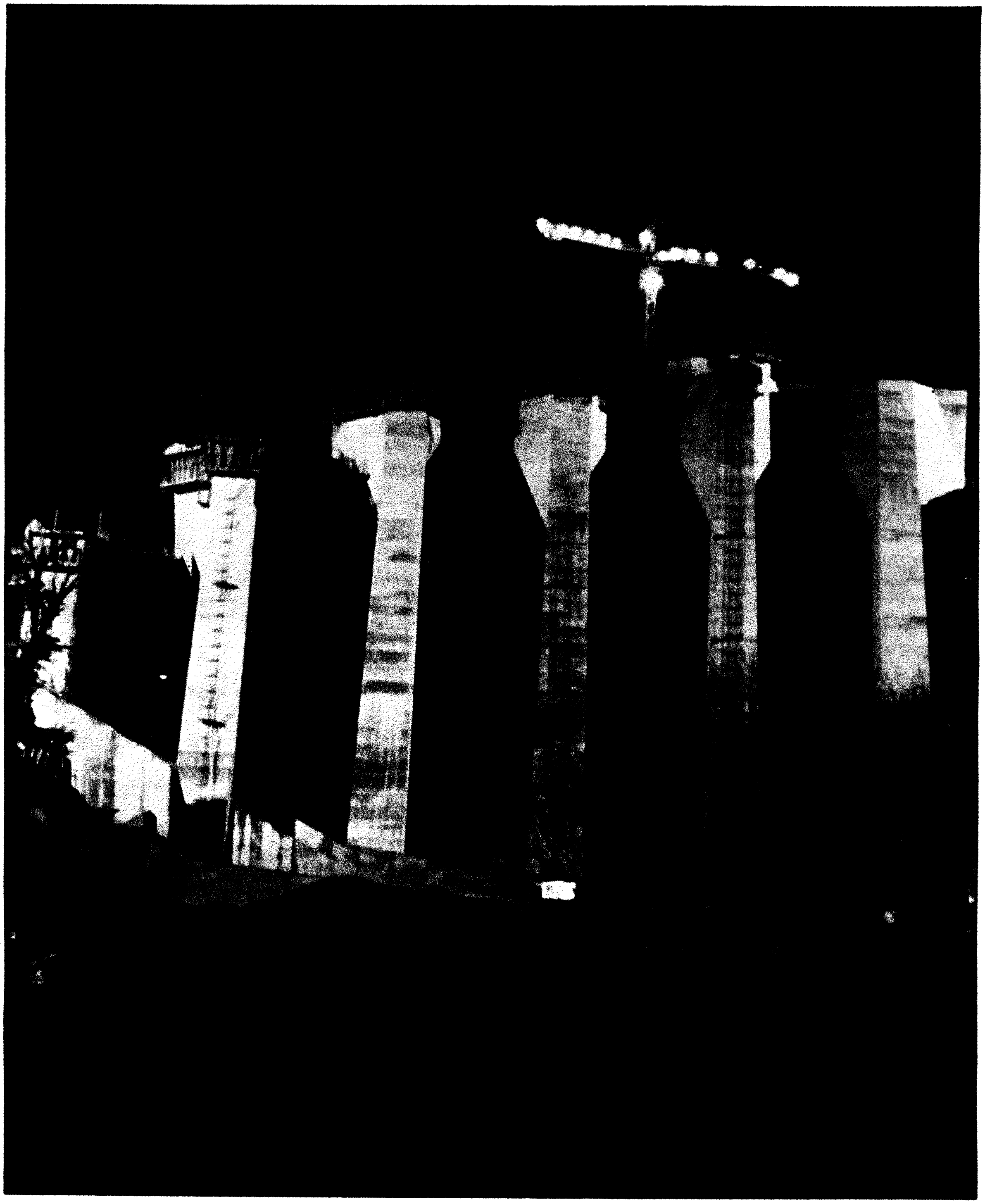
Rock Grouting at Wimbleball Dam

by D.A. Bruce and C.R.F. George



Rock Grouting at Wimbleball Dam

by D.A. Bruce* and C.R.F. George†

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Abstract
The rock grouting programme at Wimbleball Dam in Somerset, involving almost 94km of drilling and 3500t of injected material, was one of the largest contracts of its kind in the UK. The 50m high concrete buttress dam across the River Haddeo is founded on Devonian meta sediments, sheared and faulted especially on the south bank. The bedrock conditions dictated the use of descending stage methods for the blanket (consolidation) and curtain (cut-off) grouting. Over 22km of blanket drilling was conducted to a maximum depth of five 3m stages under the structure. The suspected southwards deterioration of rock mass quality was reflected in a corresponding increase in average grout consumptions (19-165kg/m). The curtain under the dam involved over 20km of drilling and 600t of cement. The pattern of average lineal (19-92kg/m) and areal (22-278kg/m^2) grout takes also highlighted the same variability of bedrock conditions, confirmed by corresponding data from the extensive north and south wing curtains. Observations are made on the relationship of the grout consumption patterns and seismic survey data. Hydrogeological data on the performance of the curtain during reservoir impounding asserted the need for additional treatment to the south bank. The Paper constitutes an important case history and its purpose illustrates contemporary British views on the design, construction, assessment and monitoring of dam grouting.

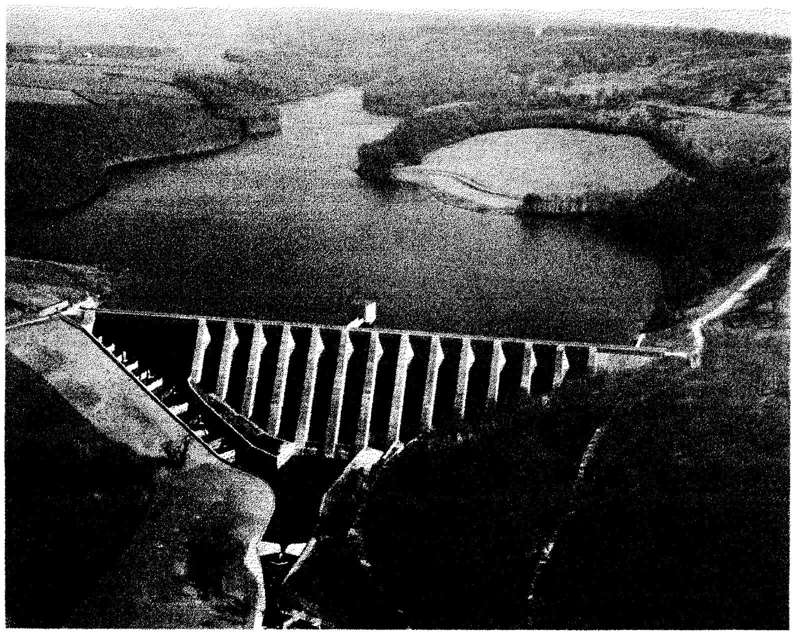


Fig 1. Wimbleball Dam, Somerset, nearing completion.

INTRODUCTION
Wimbleball Dam in Somerset (Fig 1) has been the site of one of the largest rock grouting projects in the UK. Almost 94km of drilling was involved, together with the injection of 3500t of cement and sand. Blanket grouting was done to consolidate the rock immediately under the foundations, but the major undertaking was curtain grouting to form a cut-off under and flanking the dam.

Dam grouting techniques of the type used at Wimbleball have been refined as the theory and practice of dam engineering have become more sophisticated. In addition to constituting an important case history, this project provides an opportunity to illustrate contemporary British concepts regarding the design, construction and assessment of such works. Its relevance is shown indirectly by the US Army Engineer Waterways Experiment Station (1978) which cites 352 technical papers published between 1891 and 1977 dealing principally with dam grouting – only about 20 referred to British sites, and only three appeared in the 1970s.

Relevant data are reviewed, mainly from American and Australian sources, with emphasis on work published since 1977. The additional curtain grouting conducted in a problematical zone of the south bank illustrates several important principles of monitoring and execution.

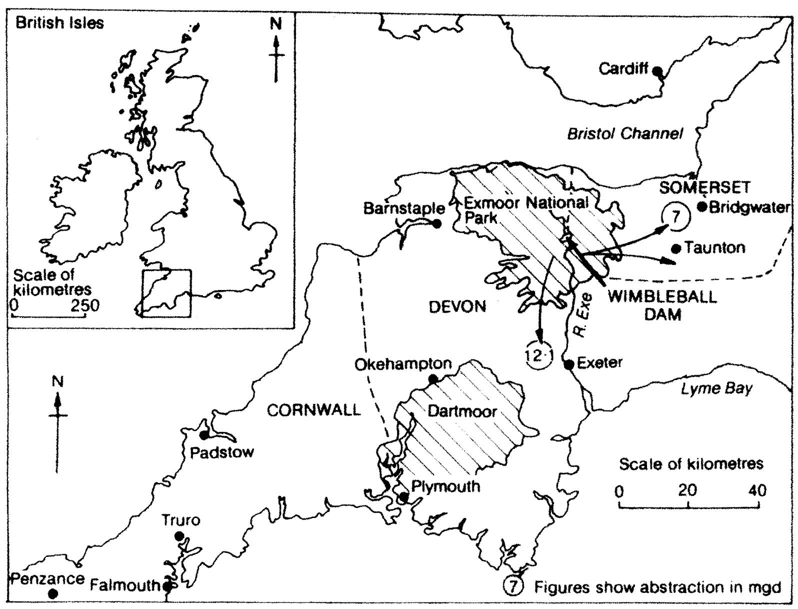


Fig 2. General location map.

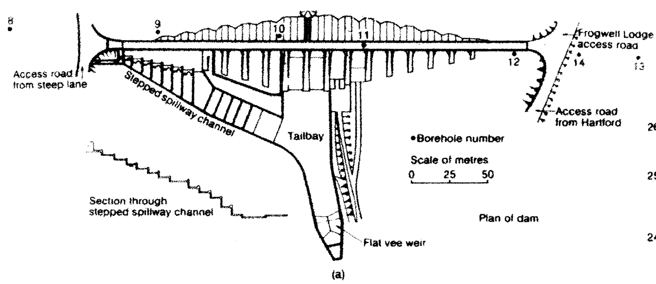


Fig 3a. Site investigation data.

SITE AND GEOLOGY

Wimbleball Dam is situated across the valley of the River Haddeo – a tributary of the River Exe in Somerset. The river runs east–west within a deep V-shaped valley, and the ground rises from 190m AOD at the river to approximately 352m AOD on the north bank. The reservoir top water level of 235.61m AOD necessitated a 50m high dam, the form of which was chosen with particular regard to its visual and environmental impact in this attractive area of the Exmoor National Park (Fig 2). The dam consists of 16 buttresses with four gravity monoliths abutting into each valley side. It impounds 4750 x 10^6 gal and yields 21mgd.

The bedrock of Upper Devonian Pickwell Down beds consists of red, purple, brown and green coloured sandstones, siltstones, mudstones and slates, and exhibits rapid lateral variation (Fig 3a, b and c). Apart from the overlying

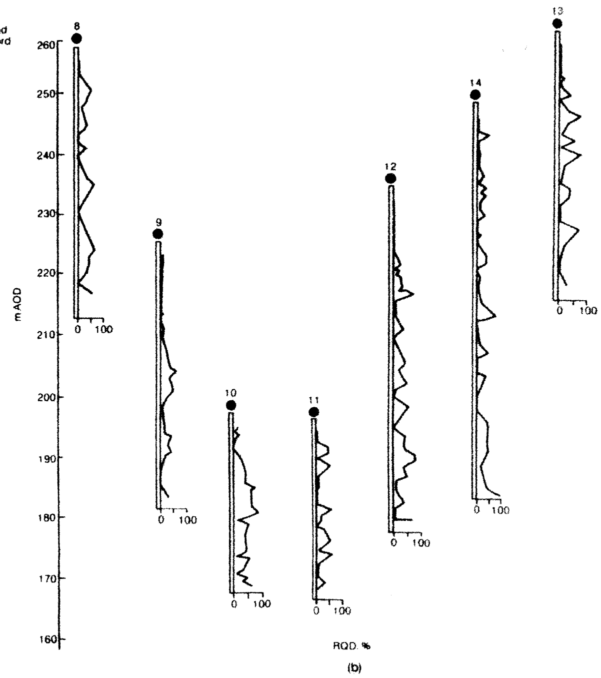


Fig 3b. Site investigation data.

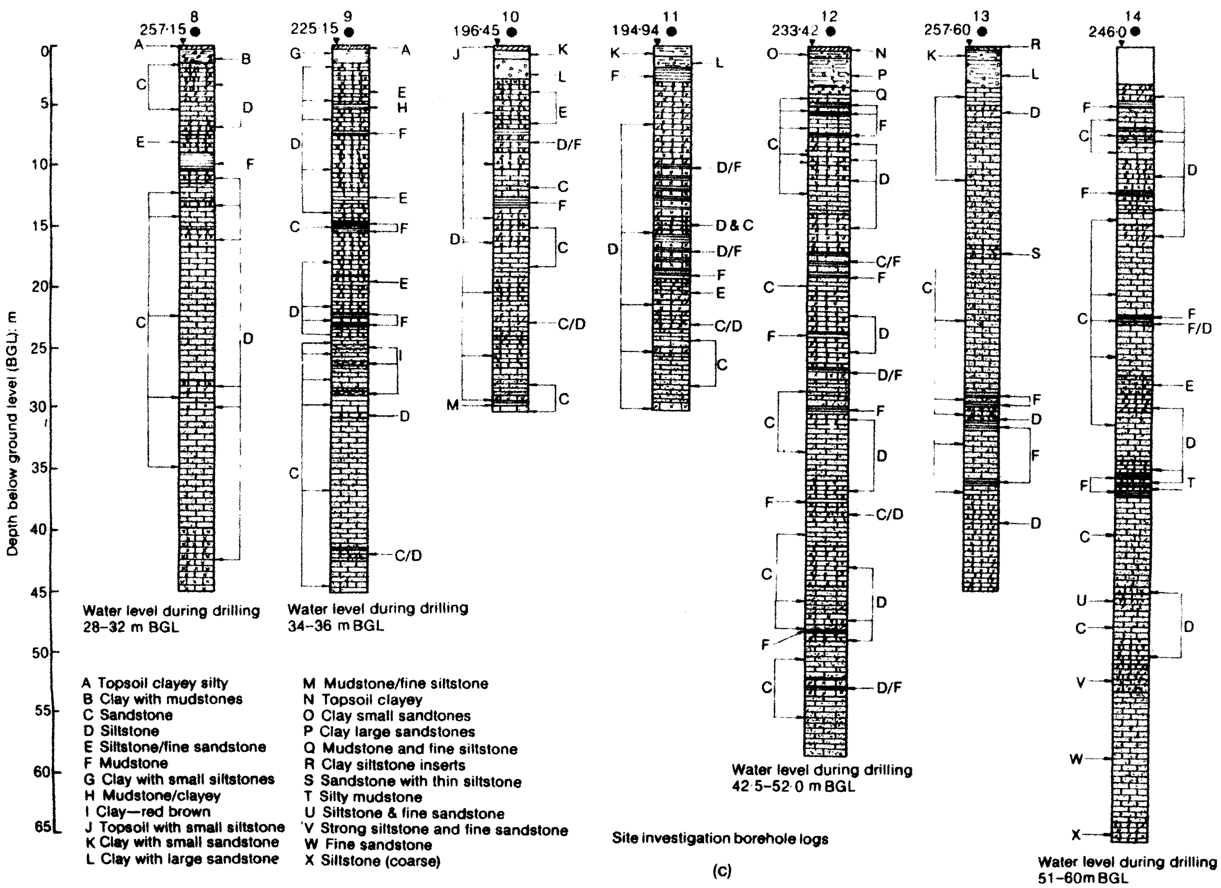


Fig 3c. Site investigation data.

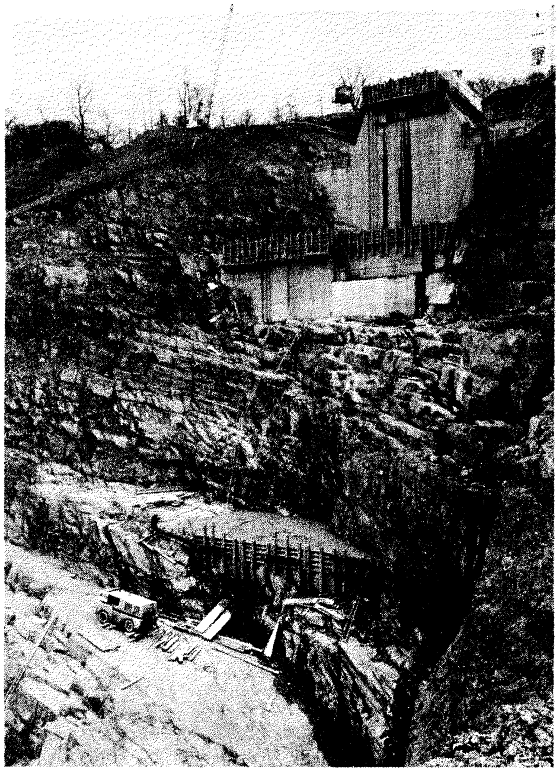


Fig 4. Rock exposed on north side, under stepped spillway channel.

Fig 5. Rock exposed on south side, under buttresses 14-16.

1-2m of apparently sedentary overburden, the rock is commonly indurated, faulted and fractured and frequently metamorphic. The low gradient of the water table suggested that open fissures extend to considerable depths beneath the valley sides.

The rock at the northern end of the site (Fig 4) dips almost due south at about 45°, and is hard and competent. On the south bank are dips are about 14° west (Fig 5) and there are quite thick broken and weathered strata between relatively competent beds. The one inch geological map shows some large faults trending east–west, near vertical and normal, south of Haddon Hill, together with other major faults, striking north-west–south-east approximately 2km to the east. The inferior geological conditions exposed on the south bank were believed to extend southwards along the line of the projected south wing curtain.

Between these two banks of contrasting structure, there are numerous shatter zones within which the mudstone in particular is broken and fragmented in a matrix of red hydrothermically produced clay. In these zones the occurrence of rock fragments of lithology dissimilar to that of the surrounding strata indicates extensive movements. They are of Hercynian origin, strike north-east–east, dip subvertically and 3-5m thick. They were particularly intense in the region of buttresses 7-12, and on the south side foundation levels had to be carried deeper than the 7-10m foreseen. Concrete foundation thicknesses averaged 15-18m, with a maximum of 28m for buttress 12. Laboratory tests on the siltstone and sandstone cores confirmed the high density and low porosity of the rock types. Air-dried density values were 2.51-2.82Mg/m^3 with corresponding saturated densities 2.55-2.85Mg/m^3. Natural bulk densities were 2.48-2.51Mg/m^3.

A seismic refraction survey was carried out before excavation to assess in detail the fracture state of the rock mass at the dam site and along the proposed line of curtain grouting. Compressional wave velocity is highly dependent on the nature, extent and orientation of fissures, particularly above the water table.

The intercept-time method was used to compute the thickness of each major bed. Five traverse lines, involving 32 seismic spreads, were set out and the tests were programmed such that

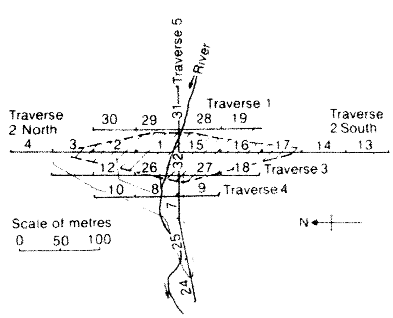


Fig 6. Location of seismic traverses and spreads.

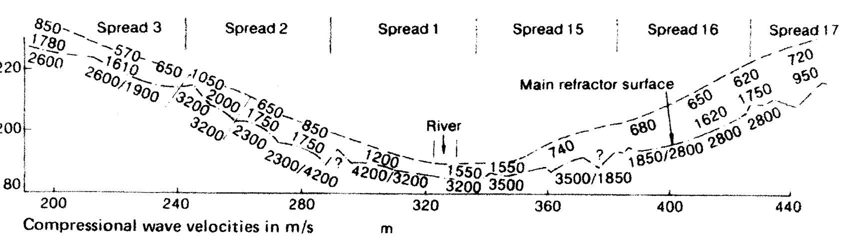


Fig 7. Seismic data, traverse 2.

the true velocity of the main refractor and the lateral variations in it could be measured.

The results from traverse 2 (Figs 6 and 7) along the dam axis were correlated with data from nine other boreholes (Table 1). The profiles confirmed that, except under the river, there existed a thin surface layer (1-2m) of low velocity (170-630m/s, average 300m/s), interpreted as representing soil or degenerated rock. The velocity of each layer was increased by the relatively high water table in the valley floor. Some major zones of weathering or fracturing, possibly produced by major faults, were indicated under the south gravity monoliths. Other areas, under the northern part of the dam, had velocity changes laterally within every layer, possibly representing displacements along steeply inclined fault planes.

For wave velocities of 2500-3700m/s (Fig 7) values of dynamic Young's modulus E_D of 10×10^6kN/m² to 20×10^6kN/m² were calculated. A static Young's modulus E_{st} of 1.5×10^6kN/m² to 3.0×10^6kN/m² may therefore be estimated.

The average depth to the main refractor surface was greater on the south side (to 20m) than on the north (typically less than 10m), and it was closer to the surface under the river than below the valley sides.

Rock type	Compressional wave velocity: m/s	Geological interpretation
I	230-430	Topsoil and soft to firm clay with rock fragments – above water table
II	610-1200	Firm clay with rock fragments, and fragmentary siltstone – above water table
III	1200-1550	Firm clay with rock fragments, and fragmentary siltstone – below water table
IV	950-1900	Weak, laminated to thinly bedded siltstones or completely fragmented sandstones (RQD = 0) – above water table
V	1500-2200	Weak, laminated to thinly bedded siltstones, or completely fragmented sandstones (RQD = 0) – below water table
VI	1500-2200	Weathered, moderately bedded sandstones, siltstones and mudstones (RQD = 12+) – above water table
VII	2200-3200	Strong, moderately to thickly bedded, weathered mudstones, siltstones and sandstones (RQD = 18+)
VIII	3200-4000	Strong, moderately to thickly bedded, weathered mudstones, siltstones and sandstones (RQD = 18+) – below water table

Table 1. Correlation of seismic and geological data (traverse 2)

DESIGN OF GROUTING WORKS

Where the design of the grouting programme is keyed to stage grouting techniques, the descending (downstage) method or the ascending (upstage) method may be used (Lancaster-Jones, 1964).

In the descending method the grout hole is drilled to the depth of one stage (usually 3-5m unless flushing water is lost at shallower depth). After flushing and/or water testing, grouting is accomplished through a packer which is usually placed near the top of the hole. After the grout has attained initial set, the hole is redrilled and the drilling is continued without break to the depth of the next stage. This procedure is repeated by successive stages until the final depth of treatment is reached.

If the packer is always placed at the top of the hole, the upper layers are sealed against subsequent breakout; the more important, and usually more permeable, upper layers thus receive successive retreatment at increasing pressures. The method also ensures that large fissures are located and individually treated during the drilling and grouting.

Disadvantages of the descending stage procedure are the repeated moving of drills and handling of rods, but these can be overcome by suitable equipment and good organization. Also, the material wasted in flushing and redrilling of grouted stages is minimal in the small diameter holes commonly used.

In the ascending method the grout hole is drilled in one pass to the full depth and flushed. A packer is then set at the top of the lowest stage, which may then be water tested before grouting. As soon as this stage has been completed the packer is reset at the top of the next stage, and grouting is continued by successive stages to the top of the hole.

In principle this system is cheaper than the descending method because the drilling of each hole is continuous to full depth, and it is cleaner and less wasteful of materials. However, it is not suitable when rock conditions are such that difficult drilling with loss of water flush is experienced, when an unstable hole interferes with the placing, setting or subsequent removal of packers, or when fissuring permits grout to bypass even a properly seated packer. Other disadvantages are that breakouts from deep injections may exert high pressures on upper ungrouted beds, and that the depth of each grout hole is determined in advance and not as a result of progressive downward investigation and analysis.

The Wimbleball bedrock indicated that ascending stage grouting would present severe operational problems, especially under the southern side of the site, and so the descending stage method was used.

Blanket grouting

Blanket grouting beneath a concrete dam consolidates and strengthens the rock mass immediately under the foundations, and minimizes structural movement during construction and service. The first stage of the blanket grouting between the basal concrete (in which the packer is seated) and the foundation rock surface (Fig 8).

The selection of stage length is largely made with regard to the quality and intensity of the grouting to be undertaken. A programme of short stage lengths yields more information on the variability of ground conditions and promotes more particular treatment. However, this is time-consuming and expensive. In this instance a compromise of 3m stages was chosen.

For rock fissure grouting the diameter of the hole has no significant influence on the efficiency of the treatment, within the normal range of grout hole diameters (up to 100mm), and so the smallest diameter is chosen with due regard to the overall depth of drilling and cost. In the UK 50mm is typical. Primary holes were spaced 3-6m apart, and traditional split spacing or closure procedures (Deere, 1976; Houlsby, 1977a) were used to allocate intermediate secondary, tertiary and subsequent hole positions. This system permits close analysis and assessment of each phase of the treatment and so the overall design can be carefully monitored and adjusted during the execution of the programme.

To expedite the progress of the work, successive phases of treatment were started two stages in arrears of each preceding one, e.g. secondary drilling and grouting were started after the first two stages of the primaries were completed.

The grout holes were located upstream and downstream of the dam from North Gravity Block 2 (NG2) to South Gravity Block 2 (SG2). The geological data obtained before and during foundation construction, together with analyses of grout consumptions and drilling performances, determined the depth of treatment under each buttress. The seismic data indicated the depth of the main refractor surface. This was typically three 3m stages in regions of relatively sound rock, but increased to five where necessitated by poor ground conditions, e.g. under buttress 14.

Hole inclinations were varied from vertical to 45°, and in different directions to ensure a penetration into as much of the bedrock volume under each buttress as possible. This meant that it would be unlikely that, regardless of its

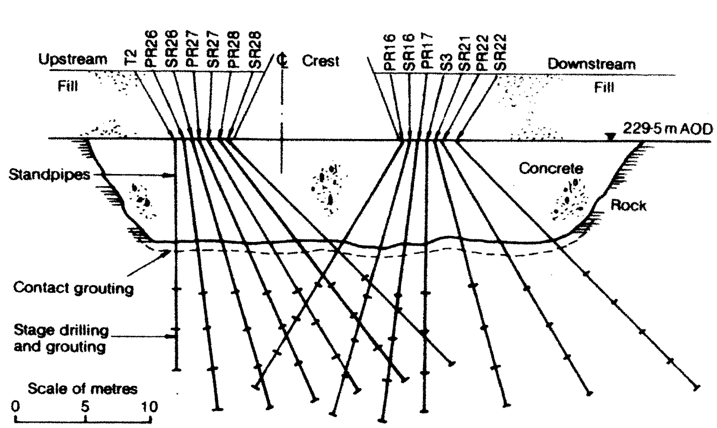


Fig 8. Cross-section through line of blanket.

orientation, any major fissure would remain untreated. Fig 9 provides a typical example, showing the locations, depths, trends and dips of the holes drilled under buttress 14. When particularly poor zones of rock were encountered (e.g. under SG2 (Fig 8)), intensive fans of holes were designed.

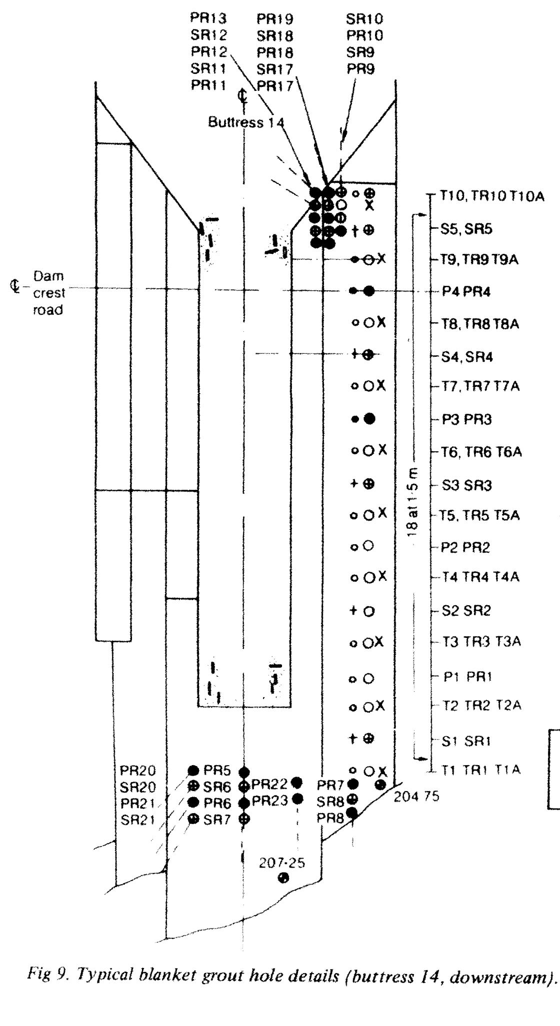
Final stage grouting (refusal) pressures were designed to prevent uplift occurring during grouting. As most of the blanket grouting was carried out early in the construction of each buttress, before a substantial surcharge was exerted by the structure, maximum stage pressures, as measured at the top of the hole, were restricted to the effective overburden pressure at the top of the stage to a maximum of 3.5 bars. The work of Houlsby (1977a) suggests that this approach is unduly conservative, given the rock conditions over a large part of the site (Fig 10). However, where there is a possibility that horizontally bedded strata exist at shallow depths, particular consideration and caution must always be taken to avoid structural movements.

Successive phases of grouting were carried out until the magnitude of the individual stage grout consumptions indicated that all major fissures had been treated and the rock mass was tight in relation to cement-based grouts. Each stage injection started with a thin mix having a water-cement ratio of 5 and an apparent viscosity of about 1.4cP, and was progressively thickened to a mix of minimum water-cement ratio 0.5 (37cP). The progress of the thickening was determined in the field by observing the rate of grout acceptance and the rate of pressure build-up. For example, stages which readily accepted the initial mix with little back resistance had their mix thickening programme accelerated relative to those which were tighter. In this way, the risk of premature blocking off of fissures by pressure filtration of grout particles causing a cake on the borehole wall was minimized.

Where the presence of large fissures was inferred from drilling records or from this grouting procedure, fine sand (zone 2 or 3) was added in the ratio 1:1 by weight with cement, after 1000kg of neat cement had been injected, the mix ending at a water-cement ratio of 0.5. In such cases, towards refusal the grout was reverted to the neat cement mix to ensure the fullest treatment of the finer fissures (down to 160μm; Littlejohn, 1975) which were incapable of being filled by the sand particles. On the rare occasions when even the addition of sand had no obvious effect on rate of acceptance and the stage in question could not be brought up to the required refusal pressure, grouting was discontinued for the remainder of that shift, and started again 24h later. This method of restricting injections was adopted because the grout could otherwise have been travelling considerable distances and in largely unknown directions, which would have been wasteful and potentially dangerous to the structure.

As the upper horizons treated in blanket grouting are frequently highly fissured, connections are commonly established between adjacent holes during pressure grouting. This is also to be expected in the upper stages of curtain treatments. In such cases at Wimbleball, adjacent holes were plugged with top hole packers before injection was resumed using thicker mixes. In this way the grout was constrained to flow into fissures, and not merely allowed to escape from nearby holes. The grout which interconnected was redrilled when set as part of the normal stage drilling procedure.

Clay infilling in joints from boreholes was recognised as being neither extensive nor significant from the geomechanical and hydrogeological viewpoints, and so no major programme of clay expulsion by air and water flushing was specified. Lane (1963) describes the work involving jetting before grouting.



Angles of holes to vertical			
T1A–T10A	Vertical	SR10	10°
P1–4	Vertical	PR11	30°
S1–5	Vertical	SR11	31°
T1–10	12°	PR12	34°
PR1–4	20°	SR12	37°
SR1–5	20°	PR13	40°
TR1–10	29°	PR17	14°
PR5	11°	SR17	17°
PR6	33°	PR18	22°
PR7	15°	SR18	28°
PR8	45°	PR19	34°
PR9	34°	PR20	15°
PR10	18°	SR20	24°
SR6	22°	PR21	34°
SR7	45°	SR21	45°
SR8	29°	PR22	11°
SR9	26°	PR23	33°

Depth of holes into rock	
T2A, 3A, 6A	18 m
P1–4, S1–5, PR1–5, PR17–23, SR1–5, SR11, 12, 17, 18, SR20, 21, T5A, 7A–10A	15 m
PR6–10, SR6–10, T1, 2, 5–10, T1A, 4A, TR1, 3, 4, 7–10	12 m
T3, 4, TR2, 6	9 m
TR5	6 m

Primary:	o	Vertical	●	Raker
Secondary:	+	Vertical	⊕	Raker
Tertiary:	·	Vertical	O	Raker

Scale of metres 0 5 10

Fig 9. Typical blanket grout hole details (buttress 14, downstream).

Curtain grouting

Creating a grouted cut-off reduces the amount of seepage under or round a dam and enables complementary downstream pressure relief wells to control foundation uplift pressures without unacceptably high water losses. At Wimbleball, the wings – the contiguous extensions of the underdam curtain into the flanking valley sides – were also designed to prevent the mobilization of high hydrostatic pressures in these sides downstream of the dam.

The curtain was formed in a vertical plane, descending continuously from level concrete scarcements on the upstream side of the dam (Fig 11). To promote the efficient treatment of the major vertical and subvertical discontinuities and shatter zones, sections of the curtain were locally inclined at 15° (buttresses 12-16) within the vertical plane of the curtain. In addition, access limitations involved the further raking of holes, e.g. under NG4 buttress 8 and SG3 (Fig 12). Inclined drilling is more expensive than

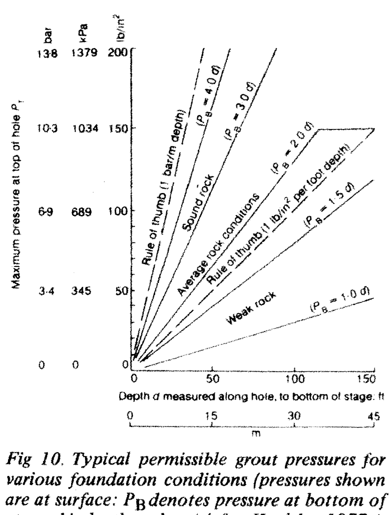


Fig 10. Typical permissible grout pressures for various foundation conditions (pressures shown are at surface: P_B denotes pressure at bottom of stage, d is depth to there) (after Houlsby, 1977a).

vertical drilling and should only be specified on sound geotechnical or practical grounds, as in these examples.

A single row of closely-spaced holes was adopted for the following reasons.

(a) It was felt that it would provide a curtain with a higher degree of physical continuity than one created by the same number of holes in multiple rows and so more widely spaced
(b) Large amounts of erodible joint fill material were neither foreseen or recorded
(c) It was considered that the rock mass could be made satisfactorily watertight with cement-based grouts, and so subsequent phases of clay or chemical injection were not envisaged
(d) The important rock mass discontinuity directions varied considerably across the site, and there was no consistent evidence to suggest that multiple rows would be more successful in intercepting them than an intensive single row
(e) The limited access afforded by the narrow dam scarcements would have restricted the potential width of a multiple row curtain.

In general, the depth of the curtain below dam foundation level was designed to be at least 75% of the maximum depth of the reservoir when full. This gave a curtain depth of approximately 38m. Cope (1978) notes that a common rule of thumb as a guide in selecting curtain depth D is $D = 1/3H + C$, where H is the head of water and C = 8-25m, depending largely on

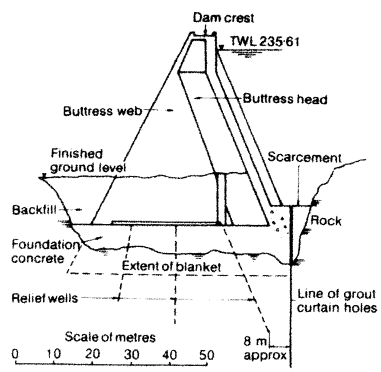


Fig 11. Section through typical buttress showing relative location of blanket, curtain and pressure relief wells.

rock mass permeability. In this case, C would be about 21m – the high value reflecting the overall dam design, the results of the site investigation and the acceptable seepage loss within the project budget. The depth was locally modified in the light of the seismic and borehole data, coupled with the results of the primary drilling and grouting records. Within this framework, the curtain was discontinued at levels where stage water and grout takes suggested that a zone of acceptable impermeability had been reached. General experience indicates that only rarely is a curtain carried to a depth in excess of the storage head.

As in blanket grouting, the split spacing or closure approach was adopted, involving primary, secondary, tertiary and subsequent phases of treatment. The choice of the initial primary spacing is critical and reflects the site structural geology, the required final spacing, and the acceptable limit of grouting effectiveness as related to cost. For each primary hole, there will be one secondary, two tertiaries, four quaternaries and so on. Thus where a tertiary phase of treatment has still not induced the required degree of impermeability, the decision to proceed with a quaternary phase will double the number of grout holes involved, and will have important financial and programming repercussions.

It was estimated that the final hole spacing required to produce an acceptably effective curtain on this site would be 0.75-0.90m. A primary spacing of 3m was selected, with provision for three phases of treatment. The grout consumption analyses illustrate that this estimate was well-founded, although the extremely poor condition of the rock under certain buttresses required quaternary treatment to certain depths. The decision to terminate the grouting in each area was made after close monitoring of the stage water test and grout consumption records.

Before the grouting of each new 3m stage, the stage was subjected to a single 10 minute constant pressure (equivalent to grouting pressure) water test via a top hole packer. Such tests in primary holes indicate the virgin permeability of the rock mass, and in subsequent holes indicate the permeability after each preceding phase of treatment. These tests lack the sophistication of full-scale multiple pressure water testing (e.g. Houlsby, 1976) designed to provide reliable Lugeon (Lu) values.* However, their simplicity permitted their routine conduct in every stage, so providing a consistent and comprehensive record of water acceptance at every point of the curtain.

Grouting was discontinued when stage water tests fell below 3Lu. On these stages, subsequent grout consumptions were commonly less than 50kg at refusal. The adequacy of the treatment under each buttress was further verified by special test holes later drilled on the line of the curtain. To ensure consistency with the grout stage water test data, the same testing procedure was adopted. Grout mix design, grouting pressures and the refusal criterion were as for the blanket treatment.

A final design consideration relates to the possibility of grout hole deviation due to drilling method and rock mass structure. Grundy (1955) advocated a maximum acceptable deviation of $0.4l$, where l is the minimum curtain hole spacing. Such a standard is considered unduly rigorous, especially in the case of a deep curtain with a large number of closely spaced holes. Champion (1961) emphasized that a grout curtain in fissured rock is not a two-dimensional screen. Rather it is a three-dimensional zone of permeability considerably less than that of the surrounding rock mass, by virtue of grout travel in the fissures. The deviation which could reasonably be expected (say 1 in 50) would be small in relation to the true curtain thicknesses and so would not disrupt its continuity.

The wing curtains differed from the dam curtain in several details. For example, the line of the south wing was essentially that of the dam centre line. But the trend of the north wing was 40° downstream. This ensured that this wing was kept orthogonal to the contours of the shoulder between the dam and the main branch of the reservoir, so that the most economic solution was achieved in terms of the length and depth of curtain needed.

The choice of a 6m primary spacing on the north wing reflected the good quality of the rock mass as inferred from the site investigation. The final interhole spacing, with a full tertiary programme, of 1.5m, was adequate except for a few local quaternaries. Although the rock on the south bank was known to be poorer, the same primary spacing was adopted, both to allow an early reconnaissance treatment of the whole length, and to validate comparative analysis with the north wing data. Predictably, a fuller quaternary treatment was required on the south wing, together with lateral extensions beyond the originally foreseen length as dictated by examination of the grout consumptions, water test data and piezometer level observations backed by on-site flow net analysis. The

*1 Lugeon = 1 litre per metre per minute at 10 bars pressure.

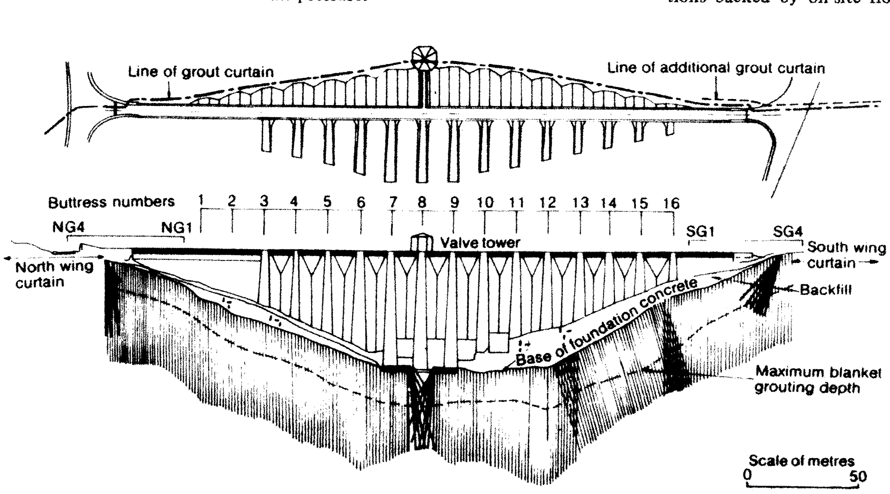


Fig 12. Cross-section along line of dam curtain.

final south wing length was 318m, compared with the north wing's 270m.

The valley sides rose steeply on both flanks of the dam, but grouting of the wings began only from depths equivalent to 4m above projected reservoir top water level (240m AOD). The grout holes were lined with 50mm steel pipes to that depth (maximum 65m) before stage grouting was started. There were fourteen 3m stages in primary and secondary holes.

An intensive programme of grouting was required along the south wing, and a scheme was devised to redrill primary and secondary holes to attempt cement grout reinjection, and thus save time and money as drilling to 240m AOD and placing lining tube would not then be required. However, the consumptions recorded on reinjection were small (Table 2) and the attempt was discontinued.

Phase	Number of holes	Number of stages	Consumption: kg	Average lineal consumption: kg/m	Reduction ratio: %
Primary	54	741	473,175	212.9 (high)	
Secondary	53	719	233,600	108.3 (moderately high)	51
Tertiary	107	1172	268,825	76.5 (moderate)	71
Quaternary (even numbers – first phase)	92	824	81,275	32.9 } 36.4 (moderately low)	
Quaternary (odd numbers – second phase)	40	291	40,325	46.2 }	48
Overall	346	3747	1,097,200	97.60	All Q:P = 17

*Table 2. Grout consumption analysis, south wing (average areal consumption 82.2kg/m²)**

*The consumption is for cement alone; in addition, 100t of sand were injected. The totals exclude the redrilling and regrouting of certain primary and secondary holes for which the average lineal consumption was 39.5kg/m. Including this grout injected into the overall total raises the average areal consumption to 83.4kg/m².

PLANT AND WORK PROGRAMME

Drilling plant

All grout hole drilling was by the rotary percussive method using water flush and a top hammer. This method is faster and cheaper than rotary drilling and superior in terms of the subsequent grout acceptability (see, for example, Doty, 1970; Deere, 1976). In America rotary drilled holes are favoured (Gebhart, 1976). These accept grout three times more readily than percussion holes drilled with air flush (Cope, 1979). However, the Authors' field experiences indicate that it is the type of flush, rather than the method of drilling, which is the critical factor, and would always favour vigorous water flush for all grout hole drilling.

Reflecting the different access limitations and drill capacity requirements across the site, up to ten wheel-mounted and track-mounted rigs were in simultaneous operation. Both air and hydraulically powered types were used with top hammers for the 50mm dia. grout hole drilling (to 107m with cross-bits) and with down-the-hole hammers for the 100mm dia. holes (to 70m with button bits) needed to insert standpipes and piezometers.

Grouting plant

At the most intensive period of the contract, five grouting stations were required. Each station consisted of a diesel driven Colcrete collodial mixer, an agitator tank, a water measuring tank and a Colcrete Evans reciprocating ram pump. Depending on the scale of the grout consumptions in the various regions, different types single or double drum) and capacities (2.5-6.1m³/h) of mixer were used. The grout was pumped to the top hole packer via a single delivery line; pressures were controlled by a manifold at the packer.

Rock grouting requires a pulsating pressure pump to inhibit premature blockages arising from the temporary build-up of coarser grout particles in the fissures. Colcrete Evans pumps – the particular models compatible in output with their associated mixers – were therefore selected. A hydraulic model was used on the south wing where long pumping distances (up to 200m) and considerable changes in elevation (up to 40m) had to be overcome. It also permitted the limiting stage grouting pressures during injection to be preset, thus avoiding the possibility of excess pressure being exerted in any stage.

Programme of work

The major controls on the overall progress of the geotechnical work were:

(a) on any one buttress or gravity block, blanket grouting preceded curtain grouting, which in turn preceded pressure relief well drilling

(b) grouting started from the middle of the dam and progressed outwards and upwards as the flanking elements of the dam became available and river diversion schemes were effected

(c) the acceleration in works which was implemented to offset the delay caused by the adverse ground conditions which seriously influenced the nature of the foundation works in certain areas

(d) the temporary non-availability of certain areas (e.g. on the north wing) which were required for different purposes, e.g. cableway track and haul roads.

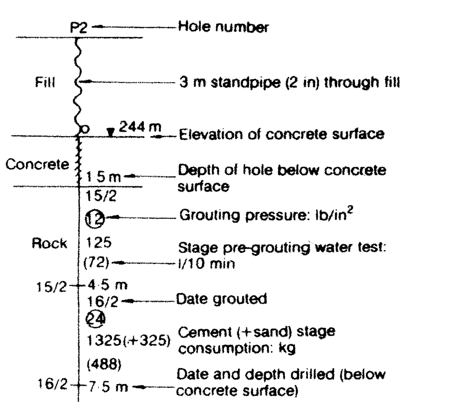
Average lineal grout consumption: kg/m	Designation
400+	Very high
200-400	High
100-200	Moderately high
50-100	Moderate
25-50	Moderately low
12.5-25	Low
0-12.5	Very low

Table 3. Proposed grout consumption classification (after Deere, 1976)

RECORDING AND ANALYSIS OF DATA

Data recording

To facilitate contract control, the measurement of quantities, and analysis of water testing and grouting data, data were presented as shown in Fig 13. Grout consumption is expressed in terms of dry weight of materials per unit length of hole (kg/m) or unit area of curtain (kg/m²) (Houlsby 1977b). This is preferable when unstable mixes are being injected, i.e. those with a large bleed capacity due to their high water-



Other data such as hole spacing, location of line of curtain, interconnections, appear on completed drawings of each buttress

Fig 13. Recording stage grouting data.

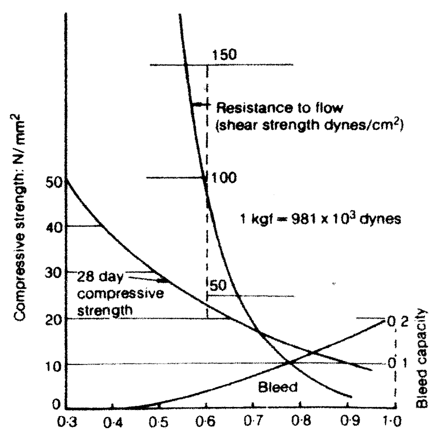


Fig 14. Effect of water content on grout properties (Littlejohn & Bruce, 1977).

cement ratios (Fig 14). Where stable mixes are used (e.g. bentonite-cement compositions with little tendency to bleed) it is valid to analyse takes in terms of total volume of mix per unit length of hole (e.g. m^3/m). The neat water-cement mixes used in the fissure grouting at Wimbleball are in the former category and so consumption is expressed in terms of dry weight of cement. As a uniform basis for quantifying qualitative descriptions of lineal grout consumption, Deere's (1976) proposed classification (Table 3) is gaining acceptance, and is used here.

Examination of daily grouting records is essential during the construction phase as a basis for designing efficiently and recognizing problem areas. The analyses here are retrospective, illustrating aspects of completed programmes. Houlsby (1977a) warns that 'mathematical averaging of the takes of several holes, or even applying statistical techniques . . . is . . . to be avoided at all times during construction evaluations'.

Blanket data

The 1285 holes and 4450 stages involved drilling 8692.15m of concrete and 13,341m of rock, and the injection of 742.3t of cement and 64.4t of sand. Data for individual buttresses are shown in Fig 15. The variable quality of the rock mass led to the following major features, which confirm the original conclusions drawn on the basis of the site investigation studies.

To attain an acceptable degree of treatment, a far greater number of holes (to quaternaries), to greater depths (maximum five stages) were required under the southern buttresses than on the northern side. In view of the fact that the buttress foundations were individually tailored to the conditions exposed on excavation (Bass & Isherwood, 1978), the increasing depth of concrete which had to be drilled, towards the south, is indicative of locally poorer rock conditions, e.g. the maximum concrete depths under buttresses 8 and 14 were 10m and 21m respectively. Lineal grout consumptions were uniformly below the average north of buttress 13 (20-40kg/m, i.e. low to moderately low) with a marked increase to over 165kg/m under SG1 – more than three times the average. Sand had to be incorporated in the mixes from buttress 14 southwards, indicating open and extensive fissures. Primary stages were commonly in the very high category in this region, and poor drilling conditions were frequently recorded.

The ultimate measure of the effectiveness of a foundation consolidation programme is the subsequent performance of the structure. Despite the variability of the rock quality, the response of the dam and its bedrock during impounding was confirmed to be satisfactory by special instrumentation involving collimators, pendulums and deep settlement reference points.

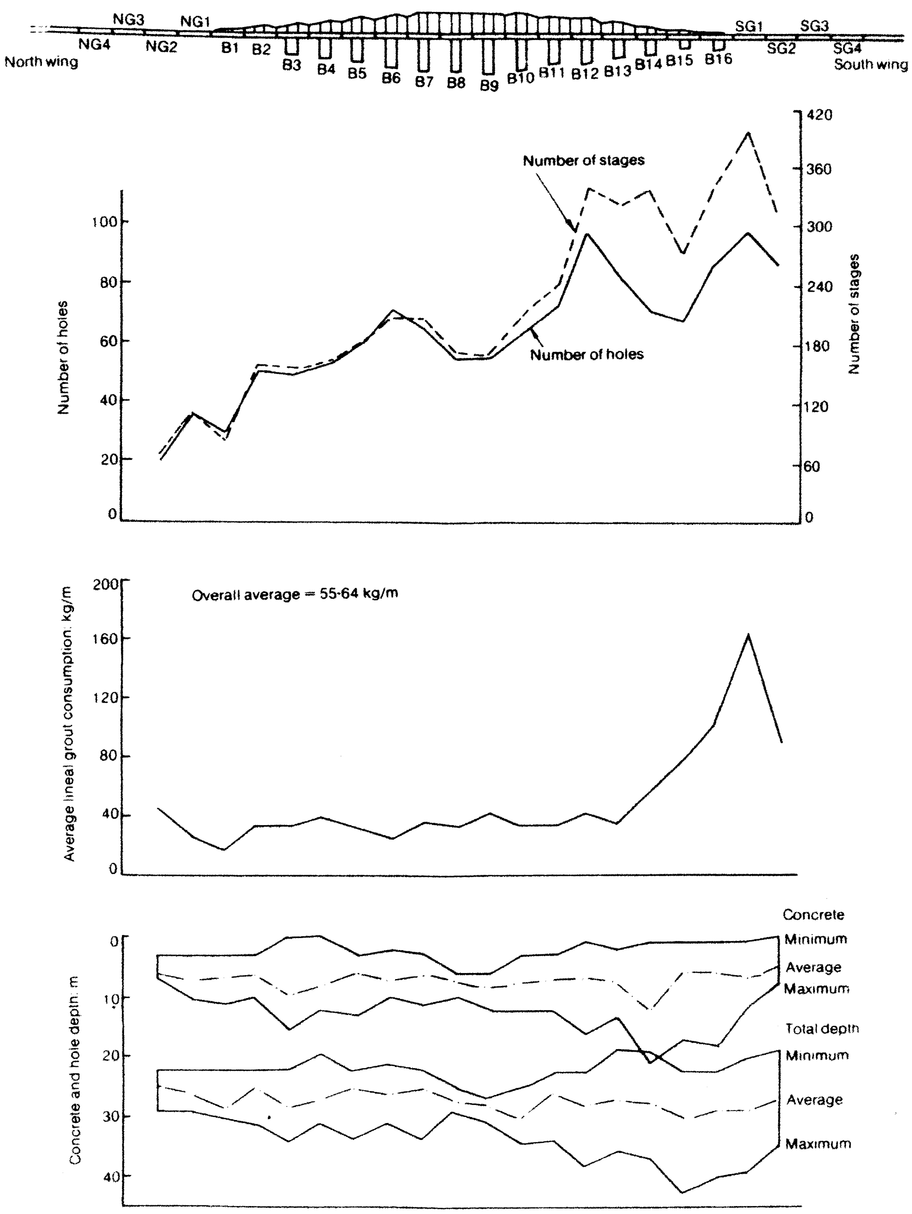


Fig 15. Blanket drilling and grouting data.

Dam curtain

The dam curtain involved 525 holes, 5888 stages (including those in water test holes), 2593.5m of concrete, 17,689m of rock, 596.7t of cement and 7.5t of sand. Data on the distribution of these quantities, and on grout acceptance patterns are presented in Figs 16 and 17. The link between the data and interpreted geology is apparent, especially with regard to the increasing number of holes (quaternaries to 0.45m spacing) and their greater depth (to fourteen 3m stages) southwards. Sand was required in some upper stages from buttress 15 to SG4.

The pattern of overall lineal grout consumptions, from 18.9kg/m (low at NG1) to 92.3kg/m (approaching moderately high at SG4), was illustrated by the pattern of areal grout takes: 22-278kg/m^2(average 52kg/m^2). Cut-offs are not two-dimensional, and the average areal grout consumption (in kg/m^2) of curtain is a more sensitive indicator of ground quality because it is independent of the total distance drilled. A closer review of individual buttress takes showed high primary and even moderately high secondary figures in the south. Deere (1976) recommends that for dams higher than 30m, final phase consumptions should be low. This is in accordance with the criterion adopted at Wimbleball where, of the 24 final phases of grouting (1 secondary, 14 tertiary and 9 quaternary), only three were outside the low category: buttress 13 (25.2kg/m), buttress 16 (27.7kg/m) and SG4 (33.3kg/m).

Reduction ratios, based on the lineal grout consumptions (Cope, 1978), are a useful measure of the effectiveness of successive phases of grouting. For example, if a primary average is 20kg/m and the secondary average 15kg/m, the ratio is 15:20, i.e. 75%. The dam curtain reduction ratios are summarized in Table 4.

Ratios of 25-75% may be considered as satisfactory within the overall design framework, but the magnitude of the results should always be considered before terminating the grouting locally. Thus, although the secondary:primary ratio of 68%, for example, would have satisfied Cope's theory, further treatment was required to meet Deere's (1976) low criterion.

Table 4 shows the drawbacks foreseen by Houlsby (1977a) when grout consumptions are analysed statistically. In areas of low average consumption, the presence of one or two unusually high stage takes will raise the overall lineal figure for that phase. Hence the resultant high reduction ratio will suggest an unsatisfactory treatment. However, a more particular analysis will verify the real effectiveness of the work by pin-pointing the influence of these exceptional stages.

The simple water test holes drilled in the line of the curtain after each buttress had been grouted provided Lugeon values consistently less than 3Lu. Most of the values north of buttress 12 were 1Lu or less, whereas remnant values approaching 10Lu were occasionally recorded under the south gravity blocks. For each buttress, analysis of the grout and water takes indicated that the cement grouting had been conducted as effectively as possible.

The efficiency of the dam curtain in operation was gauged from data provided by the associated Casagrande piezometers and relief wells. The former were installed beneath the foundations of the central part of the dam, under the valve houses and the pumping station, where the foundation loading and the possible uplift pressures are most extreme. After impounding, piezometers terminating 30m below stream level (190m AOD) showed pressures of approximately 3 bars – the value due to natural hydrostatic head alone. It was concluded

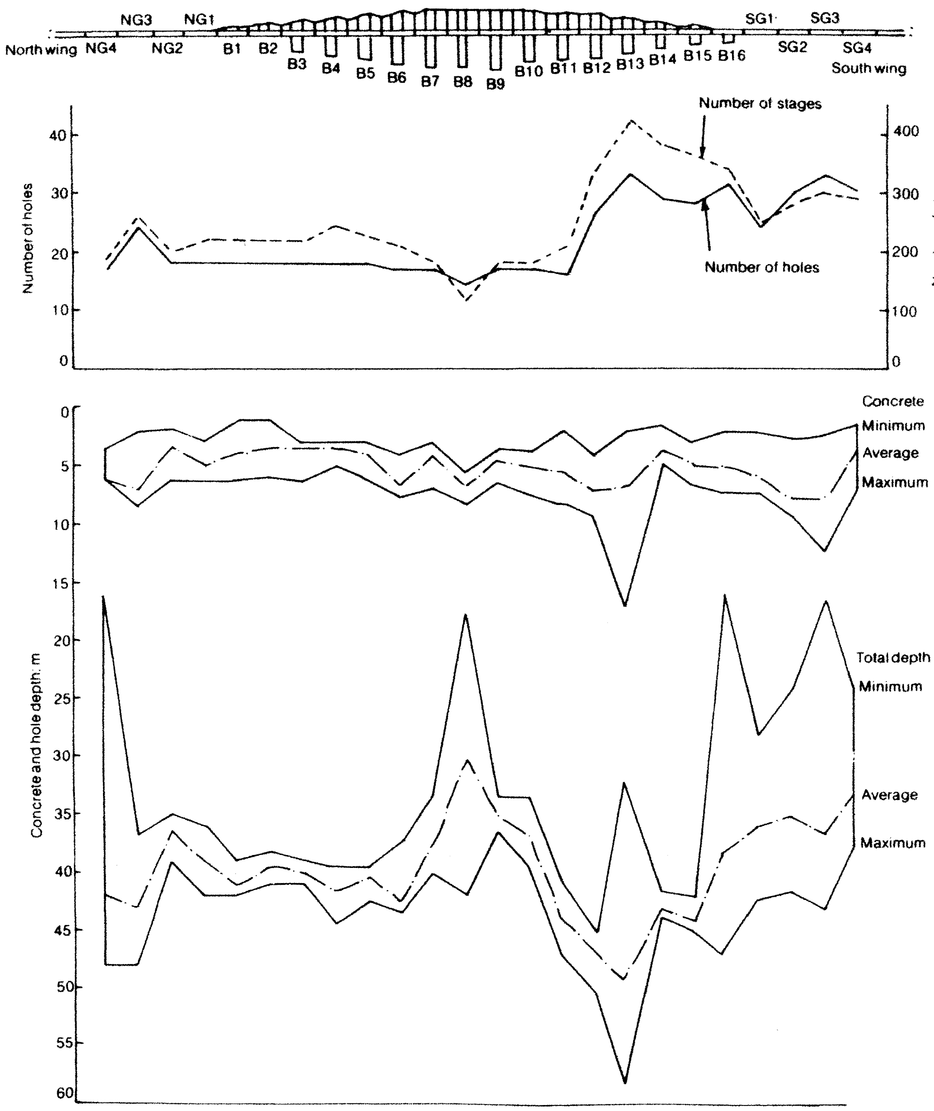


Fig 16. Dam curtain drilling data.

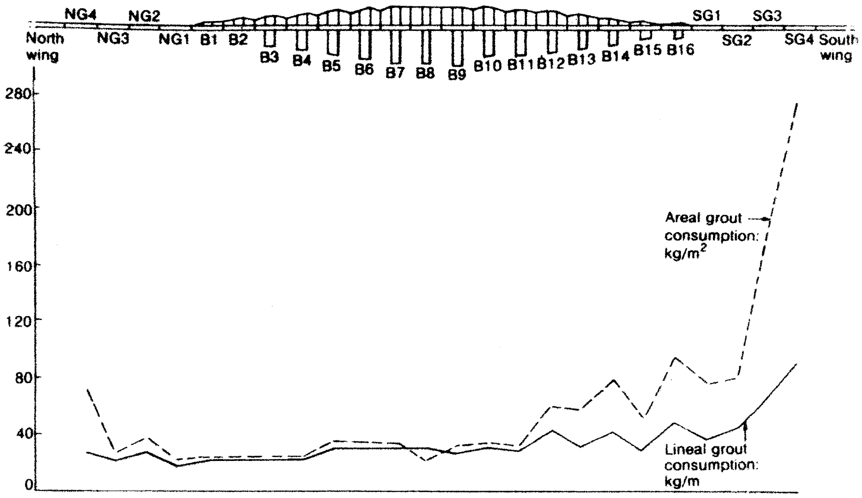


Fig 17. Dam curtain grout acceptance data.

that within the sensitivity of the instrument, no excess head (from the reservoir) was apparent. Relief well monitoring from the different sections of the dam indicated very low flows under the northern part (maximum 2l/s) but substantially higher values (around 10l/s) from the southern section. This was consistent with the conclusions already drawn on rock mass quality.

North wing curtain

The overall lineal consumption (Table 5) of 39.7kg/m (moderately low) is similar to the overall dam curtain figure (35.7kg/m) but higher than that for the adjacent dam curtain section (28.6kg/m for NG4). However, as the wing curtain hole spacing was wider, the more appropriate parameter for comparisons is the areal consumption, which at 24.2kg/m² was within the range experienced on the northern section of the dam curtain and testifies to its geological continuity with that zone. The reduction ratio achieved by each phase of treatment was satisfactory and constant, with the average primary lineal consumption of 61.5kg/m reduced to

Ratio	Range: %	Average: %	Notes
Secondary:primary	28-116	68	12 values within 25% of mean. High values associated with very low take areas and influenced by 1 or 2 exceptional secondary stages
Tertiary:secondary	30-90	56	17 values within 25% of mean. In the 23 areas of tertiary grouting, 17 had ratios less than secondary:primary ratio
Quaternary:tertiary	33-120	92	High average ratio reflects high buttress 16 and SG4 takes. Ratios lowest in high take areas, e.g. SG4
Last phase:primary	17-66	46	North side: overall lineal consumptions modest
	9-58	26	South side: overall lineal consumptions higher Overall ratio 31%

Table 4. Dam curtain reduction ratio summary

Phase	Number of holes	Number of stages	Consumption: kg	Average lineal consumption: kg/m	Reduction ratio: %
Primary	46	634	117,025	61.5 (moderate)	
Secondary	51	664	78,050	39.2 (moderately low)	64
Tertiary	99	986	78,300	26.5 (moderately low)	68
Quaternary (near NG4 only)	2	22	1,075	1.63 (low)	62
Overall	198	2306	274,450	39.7	Q:P = 27

Table 5. Grout consumption analysis, north wing (average areal consumption 24.2kg/m²)

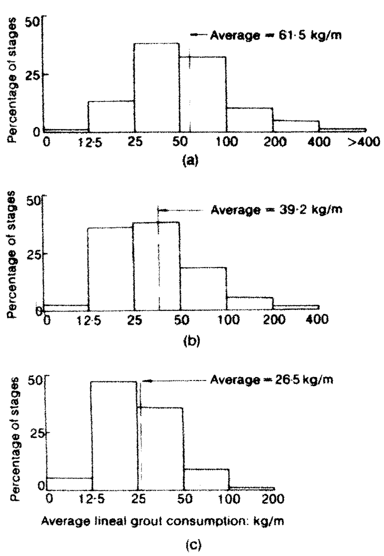


Fig 18. North wing stage consumption analysis: (a) primary holes; (b) secondary holes; (c) tertiary holes.

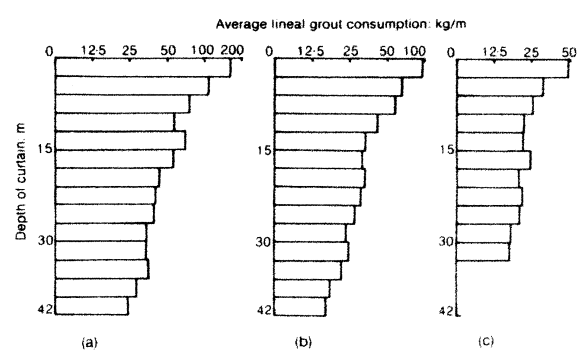


Fig 19. North wing average stage consumption with depth: (a) primary holes; (b) secondary holes; (c) tertiary holes.

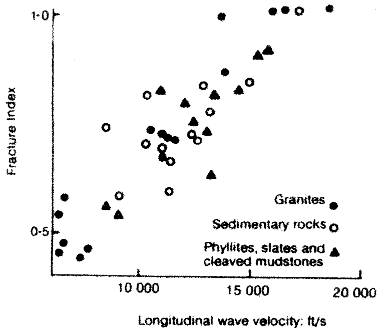


Fig 20. Relationship between V_L and F for three selected groups of rocks (Knill, 1970).

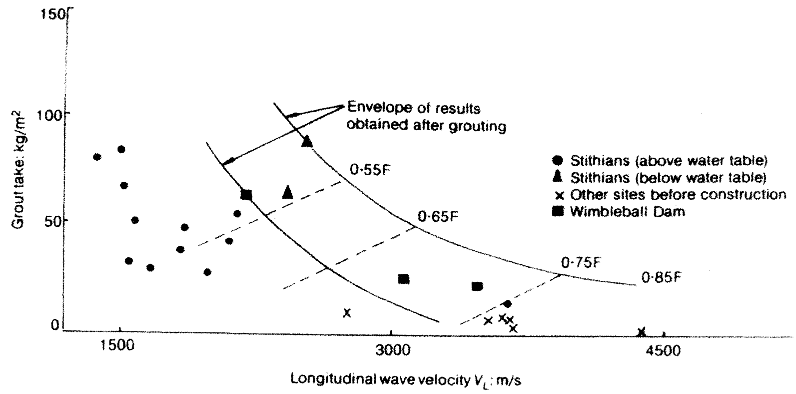


Fig 21. Relationship between V_L, curtain grout take and F, showing selected Wimbleball data (after Knill, 1970).

27% in the few quaternaries installed.

This effect is illustrated by the statistical analysis of the individual stage takes (Fig 18), using Deere's interval boundaries. Similarly, the average consumption with depth can be examined for each phase (Fig 19); the tightening of the rock mass with depth is shown to be consistent.

Correlation of seismic data and grout takes on north wing and dam curtains

Knill (1970) described results from a programme involving the measurement of compressional (longitudinal) wave velocities in rock adjacent to completed concrete dams in the UK. From his analysis of curtain grout consumptions recorded previously from the same 64 sites, he was able to conclude relationships between the two variables. Parallel investigations were conducted on laboratory scale material specimens, in which it was confirmed that the moisture content has a significant influence on velocity: drying the rock results in a velocity decrease varying up to 20%.

The ratio between the in situ and the laboratory saturated velocity measurements constitutes the fracture index *F* – a measure of the state of fracturing of the rock mass. For most rock types, the relation between wave velocity and *F* followed a broadly similar pattern (Fig 20). Allowing for the influence of degree of saturation, the values of velocity given apply to rock below the water table. Predictably, there was a well-marked increase in *F* with corresponding increase in grout take, but Knill concluded that the scatter of the results was such that wide limits had to be set on the data, desensitizing the correlation.

A more satisfactory relationship was found between velocity and grout take (Fig 21) with rock type, based on the broad divisions of origin, which had little influence on the distribution. For seismic observations made on four dams before concreting, this general relationship was repeated, except for a displacement of the results left, apparently due to lowering of the water table during excavation.

Knill suggested that in general the envelope in Fig 21 'provides the basis upon which the prediction of grout take (assuming that the pressures are not excessive) can be made in saturated rock masses'.

Three examples are taken to illustrate the range of rock conditions at Wimbleball (see

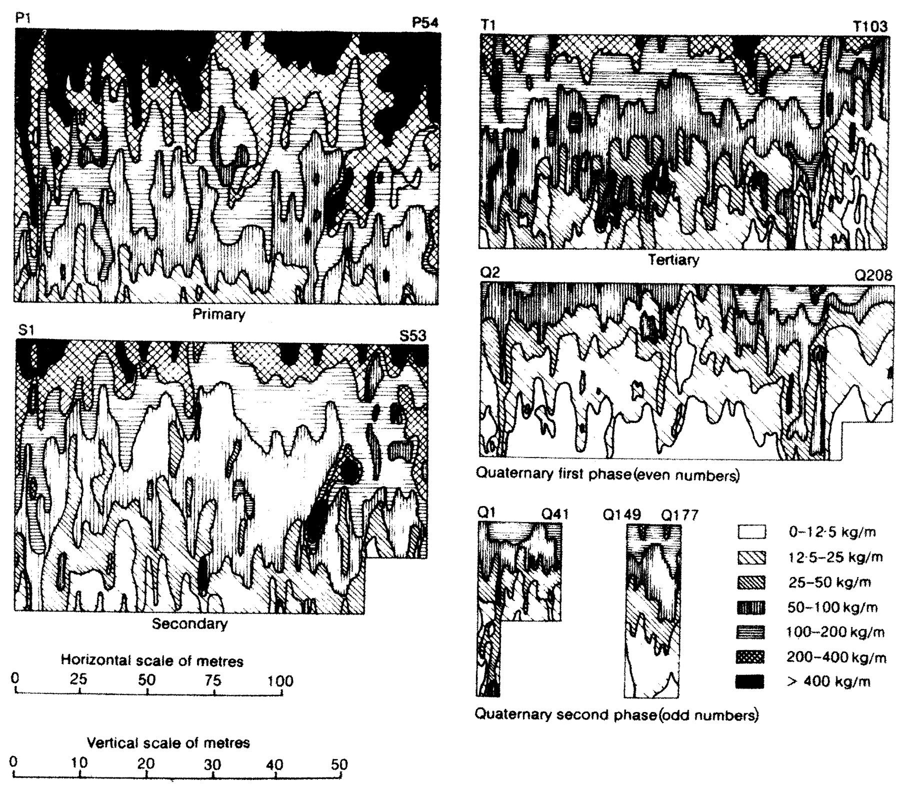


Fig 22. South wing stage consumption contours.

Table 1 and Fig 7): case A (north wing, rock type VII, velocity 2600m/s, dry), case B (valley bottom, rock type VIII, velocity 3500m/s, saturated), and case C (buttress 12-13 region, rock type IV, velocity 1850m/s, dry).

The areal grout takes were 24kg/m², 22kg/m² and 60kg/m² respectively. Assuming that the velocities in cases A and C may be raised 20% to equate with saturated measurements, the Wimbleball data plot as shown in Fig 21. The inference is that the results obtained on this site are consistent with those obtained by Knill (1970).

Knill's data are based on a seismic method valid only to 15m depth. Most curtains are far deeper than this and may be in rock of markedly different properties from that indicated by the method. Hence the technique would appear better suited to the shallower consolidation grouting. However, areal consumptions are not meaningful in this instance, and the relevance of relating them to a voids infilling percentage for cement grouting (Snow, 1968) seems questionable – a view also expressed by Houlsby (1976). Also, most of the seismic data used to establish the relationship were obtained after grouting. Differences in rock mass E values before and after grouting are typically 30-60%, and occasionally far higher, and wave velocity increases of 14-25% have been recorded, varying with depth and grouting method (Lancaster-Jones, 1968). As the relationship is used to provide a predictive capacity on grout takes from data obtained before grouting, extreme care should be exercised in determining the relationship.

A wide scatter of the results is indicated by the width of the envelope (Fig 21). Inaccuracies may arise from the range in measured velocities at any one site (e.g. Fig 7). The accuracy of measurement (claimed to be better than 10%) and the validity of assumptions regarding the degree of saturation have an influence on rock anisotropy of the state of in situ stress.

For these reasons it would seem doubtful if the seismic correlation method could yield a more accurate and reliable estimate of the expected grout takes than an examination of the geological site investigation data. Such seismic studies have value in, for example, suggesting the depth to which consolidation grouting is likely to be required, and the location of major structural discontinuities. However, as a means of providing all but the most general indication of likely curtain grout consumptions it would seem inappropriate.

South wing

The poorer rock and the steeper slope required a longer curtain (318m) with deeper holes (to 107m), more closely spaced (major quaternary programme). The average lineal consumption of 97.6kg/m (over twice that of the north wing, Table 2) was the highest recorded for any section of the curtain, but the areal figure of 82.2kg/m² was of the same order as that for the rest of the south bank. The final phase consumptions were appreciably above Deere's (1976) recommended value. To have attained his limit over the whole length and extent of the curtain would have been prohibitively expensive. Thus particularly open areas were concentrated on by localized quaternary treatment in two phases, principally in the upper stages of the curtain.

The grout take contours for each successive phase (Fig 22) suggested that the particularly open upper regions had been effectively treated. This view was supported by the statistical analysis of the stage takes (Fig 23), although a large number of takes of 50-100kg/m in the second phase quaternaries (i.e. odd numbered holes) boosted their average lineal consumption to 46.2kg/m; that for the even numbered quaternaries was 32.9kg/m. A uniform tightening up with depth (Fig 24) was not so pronounced as it was on the north wing, and a bulge existed 18-21m below the top of the curtain indicating the presence of large, deep fissures. Quaternaries were therefore instructed only to that depth.

However, as impounding progressed, the southern relief well flows reached 10l/s, which is high in relation to the overall picture. In addition, a group of springs were observed, 200m below the dam on the left bank just above stream level. The flow increased with reservoir level and extrapolation suggested that the leakage at full reservoir could reach 114l/s. Chemical analyses suggested that up to 80% of the spring water originated in the reservoir, and data from water level test holes indicated the likelihood of a major seepage path at, or just south of, the southern end of the dam.

The quantity of the flow and its potentially detrimental effect on the stability of the valley side dictated that remedial measures be undertaken quickly, and so a programme of additional grouting was devised.

ADDITIONAL GROUTING – SOUTH DAM AND WING CURTAIN

General programme

A zone from buttress 15 to 130m along the south wing was particularly troublesome. Additional grouting was to begin at buttress 15 and progress in sections southwards for a distance determined by a study of grout takes, relief well, spring flow and water level test hole data. The treatment had to be sufficiently intense to appropriate depths to ensure the final grouting of those fissures which it was suspected had not been intersected during the original programme. To increase the chances of penetrating these fissures, the additional holes were in a second row, offset 1.5m upstream of the original, with a primary spacing of 3m. The expected zone of influence round each hole was increased by raising the refusal pressures to 25% above the original figures.

The surface of the reservoir was at the joint of SG1 and buttress 16 at the time the programme was started and so treatment under the buttress had to be via a fan of raking holes. The other part of the initial section was a line of vertical holes from SG1 to about 30m along the line of the south wing curtain – a phase 1 run of about 80m. A complete secondary phase was conducted, up to four stages in arrears, to the full depth – a maximum of twenty-three 3m stages below top water level. Tertiaries were installed only where zones of high primary and secondary takes were apparent. To expedite their progress, a standard 3m staging of the ter-

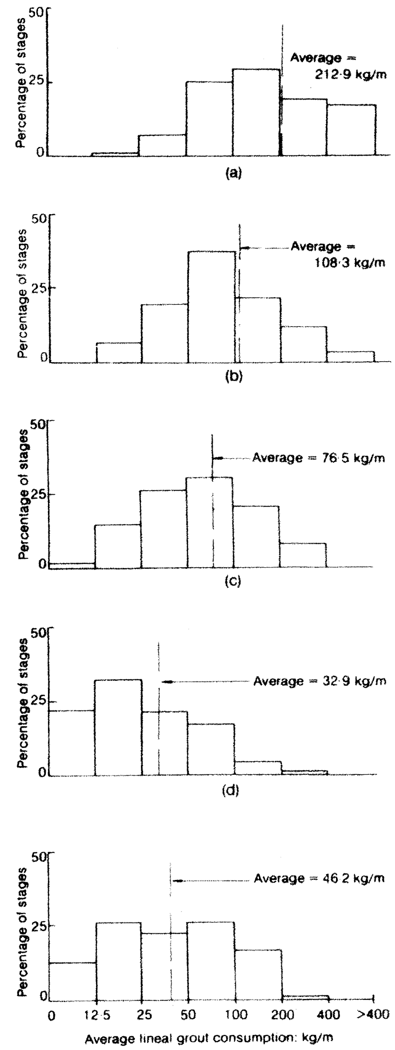


Fig 23. South wing stage consumption analysis: (a) primary holes; (b) secondary holes; (c) tertiary holes; (d) quaternary holes (even numbers); (e) quaternary holes (odd numbers).

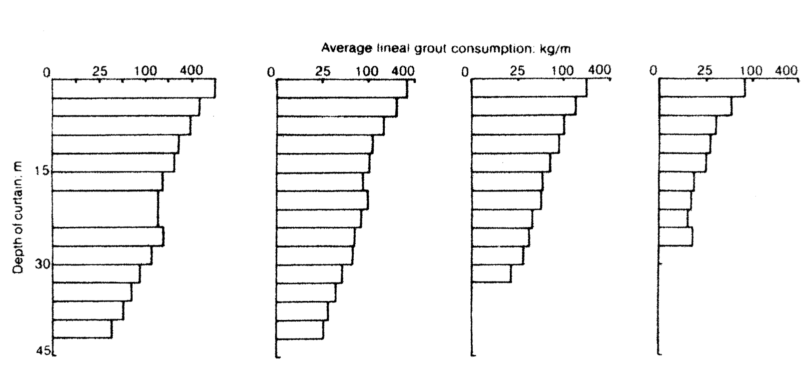


Fig 24. South wing average stage consumption analysis: (a) primary holes; (b) secondary holes; (c) tertiary holes; (d) quaternary holes (even numbers).

tiaries was undertaken only at the level of such zones; the rest of the hole was drilled in a single pass.

The results required a further 30m section of curtain to be retreated. Full primary and secondary grouting was undertaken, although terminated 45-60m below top water level, and had two 9m stages through the upper, tighter, horizons. Local tertiaries were again installed, in 6m stages at critical depths, but with up to 18m stages through low take sections.

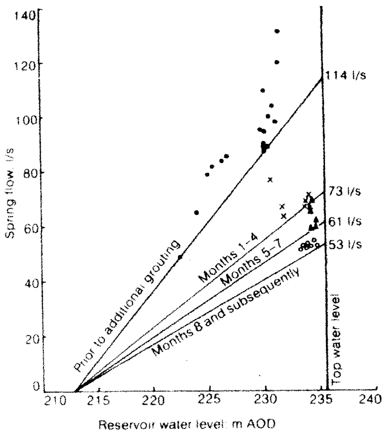


Fig 25. Effect of additional grouting on projected spring flows.

Analysis of grouting data

Table 6 summarizes the results recorded in each of the zones of grouting. The overall areal consumption was approximately 70kg/m², compared with 82kg/m² on the original treatment. Of particular significance was the geographical distribution of the major consumptions across the curtain. Under buttress 16 high stages requiring sand occurred principally in the upper two stages and between nine and twelve stages down. Two stages, including one secondary, in the latter pocket had takes over 9t; grout connections into relief wells and to the tailbay of the dam were also recorded from the same group. In the vertical section of phase 1, high and very high takes (maximum 17t at 35m) were widely distributed, with approximately 25% being secondaries. Particularly dense concentrations were apparent.

(a) in the upper 6m, and 20-30m under SG1 and SG2, i.e. near the base of the original curtain treatment
(b) 20-30m below SG4 and the lower south wing, i.e. the depth of the original tertiary and quaternary grouting
(c) 35-50m (i.e. the maximum depth of the original south wing) at around 30m along the south wing.

Effect of additional grouting

The extent and intensity of the treatment was decided primarily on the records of the spring flow. Relating this flow to reservoir level, and extrapolating from the observed zero level, of 213m AOD, shows the effectiveness of the grouting at three major intervals (Fig 25). Without the additional grouting, the minimum flow at full reservoir level would have been 114l/s. Allowing for a groundwater contribution of 23l/s, the leakage from the reservoir would therefore have been 91l/s. During months 1-4 this throughflow was reduced at the rate of 3.71l/s per working week to 50l/s (73l/s including groundwater). Most of this reduction occurred in the first six weeks (Fig 26), during which time the major pockets under buttress 16, SG2, SG4 and the lower south wing had been located at least by primaries. In the same period well flow dropped over 50%, despite the rise in reservoir level.

Between months 5 and 7, the average spring flow reduction was 1.1l/s per working week, and the piezometric contours migrated towards the centre line of the dam and southwards. Projected spring flow was 38l/s after month 7. The gradient across the curtain steepened further, and well flows again decreased slightly. However, the lower rate of spring flow reduction indicated that in this period of treatment, during which the curtain was deepened and lengthened, new major seepage paths had not been found. Similarly, from month 8 onwards the average reduction had fallen to 0.9l/s per working week, at a time when grouting was concentrated in the form of localized tertiary weak spot treatment. This relatively minor effect, also implying that no important fissures remained untreated, dictated that further cement grouting be discontinued. At this point, projected reservoir through flow as 30l/s – an overall reduction factor of three.

Combined with a similar scale drop in relief well flows, the spring flow data therefore proved the additional grouting programme to have been effective. This success was particularly marked in the early period when major seepage paths below and just south of the structure were treated. These had presumably escaped full injection in the original curtain grouting

Phase	Number of holes	Number of stages	Grouted length: m	OPC: kg	Sand: kg	Average lineal consumption (OPC): kg/m	Lineal consumption (OPC) per hole: kg/m	Percentage of stages >100kg/m	Percentage of stages <25kg/m	Reduction ratio: %
*Fan phase 1**										
Primary	9	130	403.5	29,465	8,450	73.0 (moderate)	10.6-265.5	13	58	
Secondary	8	147	425	22,150	5,100	52.1 (moderate)	21.8-143.9	7	62	71
Tertiary	3	15	63	1,320	–	20.9 (low)	13.1-26.4	0	93	40
Vertical phase 1†										
Primary	27	589	1746.5	222,670	63,050	127.4 (moderately high)	31.8-241.7	24	36	
Secondary	26	561	1657	93,890	7,350	56.6 (moderate)	28.2-166.4	10	38	44
Tertiary	23	151	998	29,435	800	29.4 (moderately low)	14.6-63.9	4	59	52
Vertical phase 2⁺										
Primary	10	139	537	51,070	6,020	95.1 (moderate)	59.9-142.2	22	17	
Secondary	11	132	528	28,690	4,400	54.3 (moderate)	31.4-104.2	8	23	57
Tertiary	6	23	186	4,100	–	22.0 (low)	15.1-35.4	0	78	41
Overall	123	1887§	6544.0	482,790	95,170			62% of stages low or very low		S:P = 49 T:S = 50
Primary						112.8 (moderately high)	10.6-265.5			
Secondary						55.5 (moderate)	21.8-166.4			
Tertiary						28.0 (moderately low)	13.1-63.9			

*Inclined under B16. †SG1 to Ch 30, south wing. ⁺Ch 30 to 60, south wing. § In lengths from 3m to 34.5m.

Table 6. Analysis of additional south bank grouting.

programme because of their orientation and spacing relative to the geometry of that first line.

The pattern of grout acceptance in the later stages of the treatment suggested that the residual flow was occurring through small fissures (less than 160μm) or in slightly larger fissures at greater depth or distance from the dam. To have reduced the flow further would have required either an alternative grouting material, to permit penetration of the fine fissures, or continuing conventional treatment. However, the level of spring flow was considered to be acceptable, and so neither of these options was pursued.

CONCLUSIONS

The Wimbleball case history demonstrates current views on the theory and practice of dam grouting in the UK. A prime responsibility of those in immediate operational control of such works is to review drilling and grouting information on a daily basis, especially for particularly bad or important zones. At Wimbleball the blanket grouting under the southern buttresses required constant review to optimize the intensity and extent of the grouting. The excellent performance of the structure in service proves the soundness of the judgement. A similar attitude was essential during the additional grouting on the south bank, where observations on relief wells, piezometers and spring flows had to be considered in order to achieve a solution which was acceptable both technically and economically.

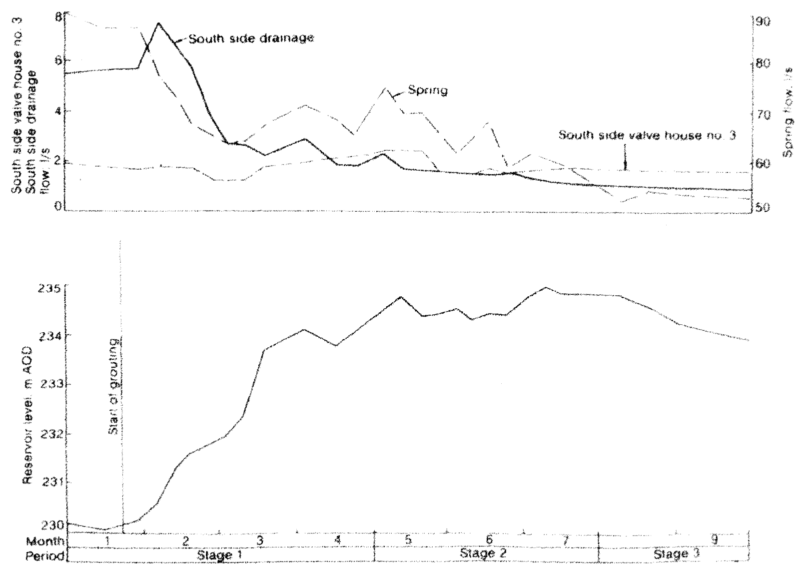


Fig 26. Well and spring flow data during additional grouting.

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