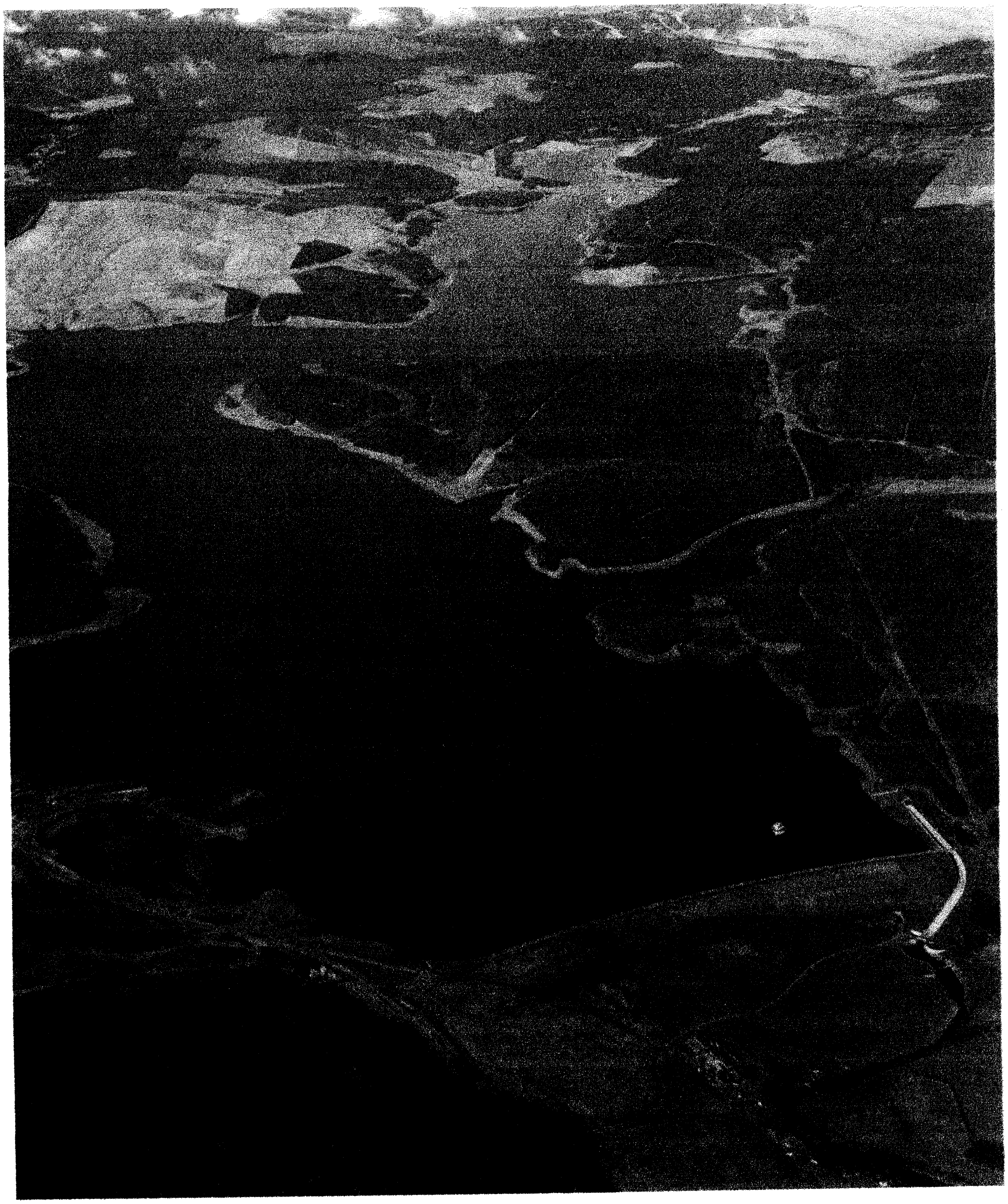
Rock Grouting and Water Testing at Kielder Dam, Northumberland

by Donald A. Bruce & James P. Millmore



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Abstract
A cement grout curtain has been formed in the North Abutment bedrock of Kielder Dam in Northumberland. In order to gauge the effectiveness of the grouting, a series of pre- and post-grouting Lugeon water tests were conducted in special control holes. This programme was executed according to proposals made in this journal on Lugeon testing (Houlsby 1976), which have now gained world-wide acclaim and acceptance. The paper demonstrates clearly both the practicality of the method and the usefulness of the results. Safe grouting pressures were selected following hydrofracture tests, again using similar methods. Data on curtain grouting design, execution and analysis are also provided, and fundamental observations made on the relationship of water takes and cement grout consumptions.

INTRODUCTION

Kielder Dam in Northumberland impounds one of Europe's largest artificial lakes with an effective storage capacity of 188 million cubic metres, and a surface area of 1,086Ha – approximately three-quarters that of Lake Windermere. The earth embankment dam has a designed crest length of 1,140m, a maximum height above river bed of 52m and a volume of over 4 million cubic metres. The scheme will be used by the Northumbrian Water Authority to regulate flows into the River Tyne (Fig 1), from where water will be fed to the Rivers Wear and Tees. In this way, the needs of the industrial conurbations of the north-east of England will be satisfied until well into the next century.

Hydrogeological analyses indicated that water seeping beneath the dam would emerge in the river valley, downstream, and so would not be lost. The design of the dam, therefore, could accommodate a certain controlled underseepage, and so it was unnecessary to provide a cut-off under its entire length. Rather, the water tightness was in general achieved by linking the clay core to the upstream clay key by means of a horizontal clay blanket (Fig 2). For most of its length, the clay key is formed in the natural

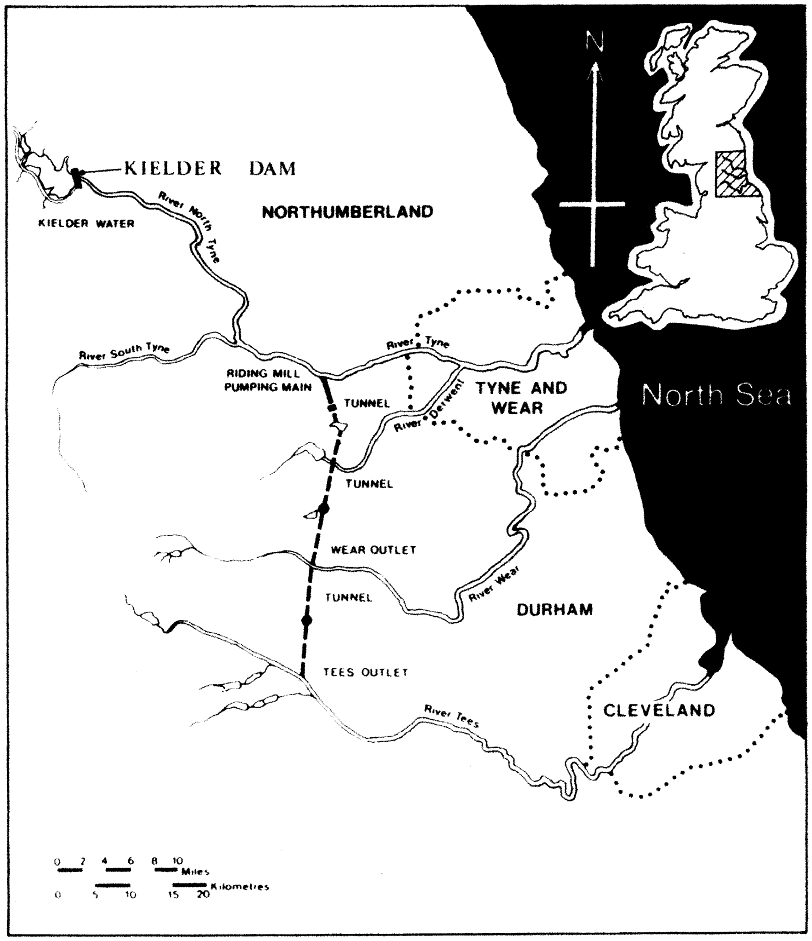


Fig 1. The Kielder Water Scheme

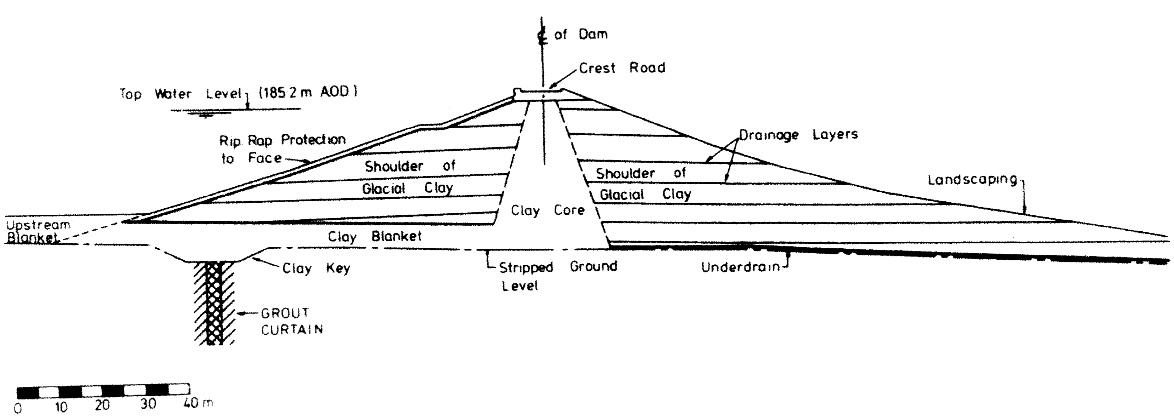


Fig 2. Cross-section through Kielder Dam

	Kielder (77 tests) % Group	Kielder (77 tests) % Overall		Houlsby (1976) (811 tests) % Group	Houlsby (1976) (811 tests) % Overall	
Less than 1 Lugeon	(100)	29		(100)	23	
1, 2 or 3 Lugeons						
Group A	70	10		78	34	
Group B	–	–		13	5	
Group C	30	4	14	1	1	44
Group D	–	–		2	1	
Group E	–	–		6	3	
4 or more Lugeons						
Group A	21	12		5	2	
Group B	23	13		53	17	
Group C	21	12	57	9	3	33
Group D	28	16		21	7	
Group E	7	4		12	4	

Table 7. Analysis of flow types and Lugeon values in comparison to Houlsby (1976)

Table 7 provides an analysis of the Lugeon values as related to inferred flow regime. In general, the analysis of the tests and the selection of the flow type were straight-forward and in accordance with the forms in Fig 6. An exception was that in Group D ('Washout'), the Lugeon pattern was often of the type abccc as opposed to abcde. 70% of the stages of 3L or less were 'laminar', the balance being 'dilation'. This predominance of 'laminar' stages agrees with Houlsby's findings. For stages with takes above 3L, all the groups except E ('void filling') were fairly evenly distributed, contrasting with Houlsby's concentration of 'turbulent' stages. The relatively high proportion of Group D ('washout') stages resulted from the presence of gouge infilled fissures. Eight of the 12 stages in question were in the deepest sections of the curtain and, therefore, subjected to the highest water test pressures. This indicated the vulnerability of the infill to erosion and so justified the selection of a multirow curtain design. Overall, the Kielder tests reflected the markedly dif-

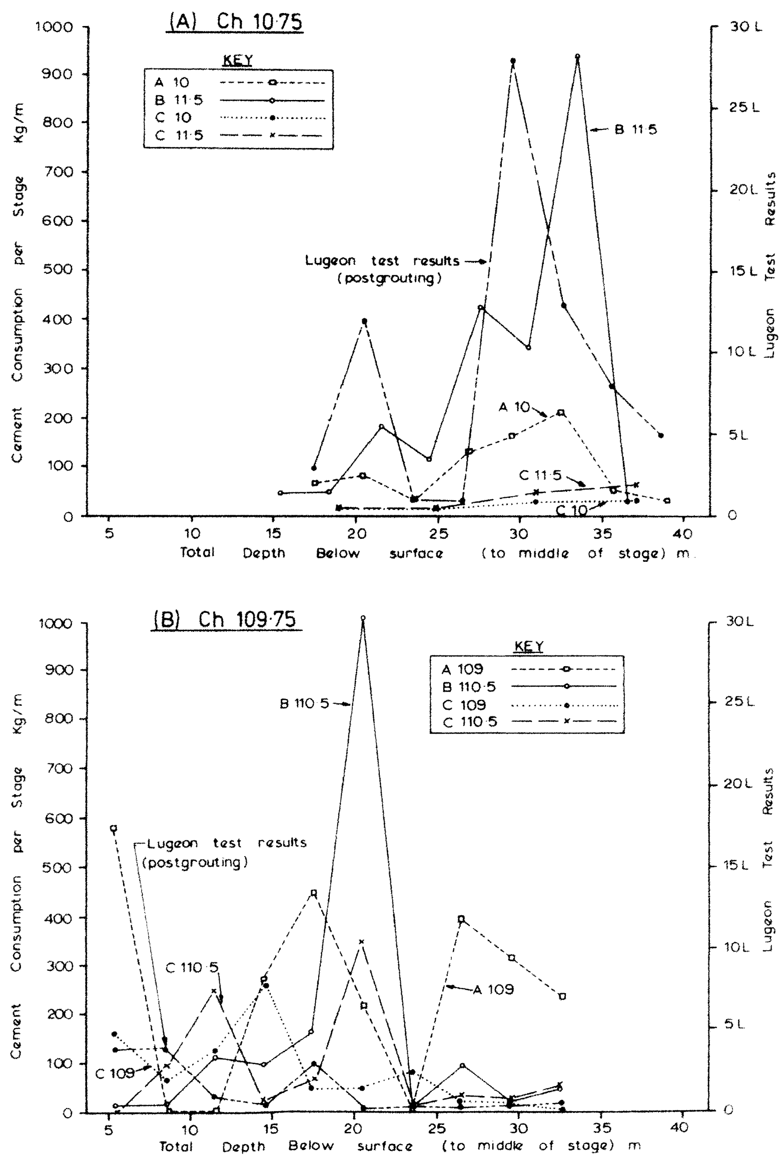


Fig 8. Relationship of Lugeon values from post-grouting water test holes at (A) Ch 10.75 and (B) Ch 109.75 to grout takes from the four closest grout holes.

Equation :-

$$H = \frac{6\eta}{\pi \rho g}\,\frac{Q}{nLt^3}\,\log_e \frac{R}{r}$$

where :-

t = average fissure width.

r = radius of hole.

η = viscosity of fluid.

ρ = density of fluid.

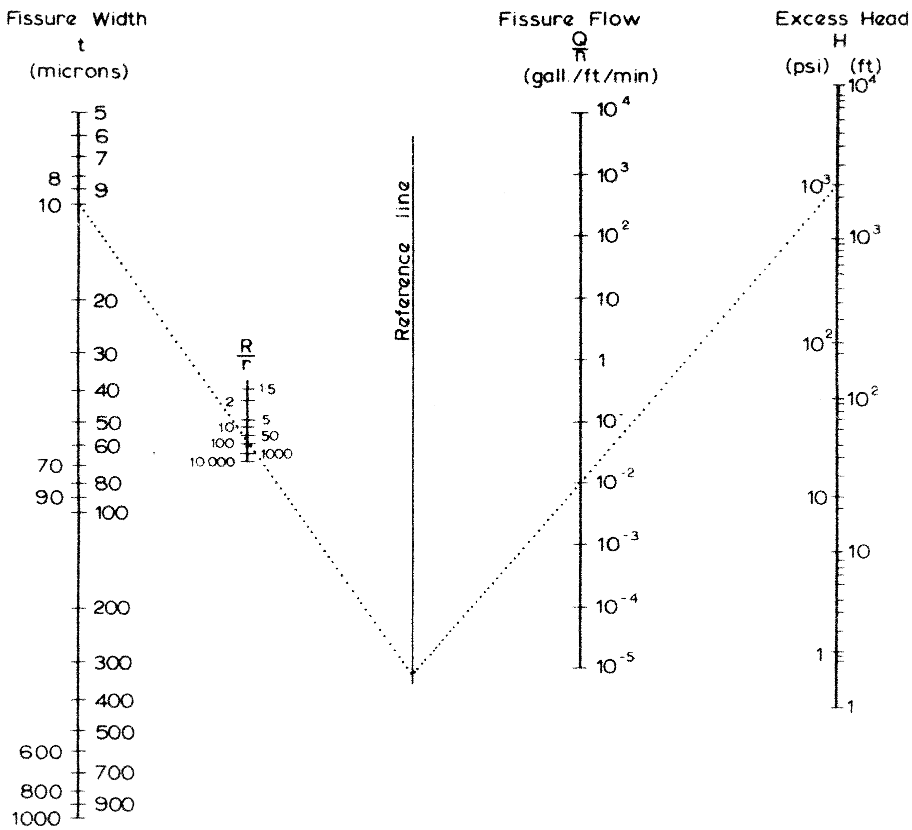
R = distance to source of water (expressed as effective radial extent of a disc shaped stratum

L = length of hole or stage under test.

Q = flow rate into hole or stage.

n = No of fissures per unit length of hole.

H = excess head at mid depth of section being considered.



Example :-

Determine the flow rate of water into a 10 ft long borehole through fissures of mean width 10 microns, where there are on average 10 fissures / foot length of hole, under an excess pressure of 1000 psi.

The average value of R/r is equal to 100

1. Place a ruler across t = 10 and R/r = 100 to give point on reference line.
2. Place a ruler across reference point and H = 1000 psi. Read $Q/n = 10^{-2}$
3. Since n = 10 and L = 10 ft, $Q = 10^{-2} \times 10 \times 10$. Therefore Q = 1 gal / min.

Fig 9. Nomogram relating geometry of radial fissures and excess head to rate of flow of water into a cylindrical hole (after Littlejohn 1975).

glacial clay overburden which mantles the rock. The overburden does shallow to the North and South Abutments and on these flanks the key was formed in the rock. Downstream, a system of relief wells was established to control seepage pressures and contribute to the stability of the structure.

These seepage control schemes were supplemented by further works in two locations:

(i) Adjacent to the Culvert (Fig 3), a plastic concrete diaphragm wall was constructed through the river bed sands and gravels in which it was impractical to form a clay key; and

(ii) Below the clay key in the North Abutment, 150m of curtain grouting was conducted in an area where bed rock was exposed and known to be extensively fractured in a complex fault zone.

The prime purpose of the grouting was to seal major fissures or voids, and so reduce the mass permeability of the area to the same order as the remainder of the site bedrock. This programme featured a series of Lugeon test holes to gauge the permeability of the rock mass before and after grouting. These tests were conducted and interpreted according to the proposals of Houlsby (1976), and this paper illustrates the usefulness and practicality of the method. Description is also provided of two 'hydrofracture' test holes installed prior to the grouting, and tested to rationalize the selection of appropriate grouting pressures.

GEOLOGY

The core and blanket of the North Abutment were keyed into a 3m deep trench excavated into Lower Carboniferous sediments of the Scremerston Coal Group. The major lithologies and their properties exposed in the trench (Fig 3), may be summarized as follows:

Sandstone. Light grey (weathered to orange), medium grained, medium bedded (200-600mm). Closely to moderately widely spaced joints (100-600mm separation), generally fairly tight (4-6mm maximum opening), moderately weathered, moderately strong (12.5-50MN/m^2). Main joint orientations as in Fig 3. Expected to be moderately permeable.

Mudstone/shale. Dark grey, very fine grained, thinly laminated, slightly weathered, weak, relatively impermeable. Very thin coal horizons.

Coal. Black, vitreous, very thinly bedded (20-60 mm). Very closely jointed (20-60mm), slightly weathered, very weak, moderately-highly permeable.

The general dip on the North Abutment was less than 10° to the south-east with considerable local variations in the vicinity of the fault zone.

During the site investigation, major fissures and cavities (up to 0.5 x 0.9 x 2m in one place) were encountered in this complex fault zone. Several faults, filled with argillaceous gouge material, were noted, especially north of Ch 100, and running obliquely across the line of the curtain with steep dips to the south-west. [The curtain extended from Ch 0 near the Culvert ('stream' or south end), northwards to approximately Ch 150.] The biggest intersected the trench at approximately Ch 100, and was flanked by a zone of shattered rock well over 1m wide.

The total seepage through the entire rock foundation was estimated to be in the region of 5,000-8,000m^3 per day, with approximately 95% flowing through the sandstone strata.

CURTAIN GROUTING: DESIGN AND PLANNING

General design and specification

Cement grouting was specified with injections to begin at a water:cement ratio of 14:1, and progressing, if necessary, to w:c = 0.7:1, by weight, as determined by the rate and amount of grout acceptance. Provision was made for the addition of inert fillers, such as sand, if especially large takes were encountered. The descending stage (or down stage) method, and 'split spacing' (or closure) techniques were specified. Primary stages, 3m long, were instructed with the length of subsequent phase stages to be determined by the results and progress of the early work.

Maximum grouting pressures at refusal, as measured at the top of the hole, were to be as high as was consistent with minimum surface displacement, or splitting of the rock. The exact stage pressures were to be determined following special 'hydrofracture' tests, conducted in the line of the curtain.

A minimum of 3m of clay embankment was required to be placed before grouting, to provide an appropriate reaction against the grouting pressures in the upper stages in particular. In addition, the grouting of the first stage was to effect the contact grouting of the clay-rock interface.

Two primary rows of vertical holes (Fig 3) were designed, 1.5m upstream and downstream of the curtain centre line. The holes in each row were offset from each other at 3m centres, and were foreseen as being from three to ten stages into rock with provision to deepen locally to terminate in mudstone. Possible secondary or further holes were to be at 1.5m intervals along the centre line. This multi-row system was

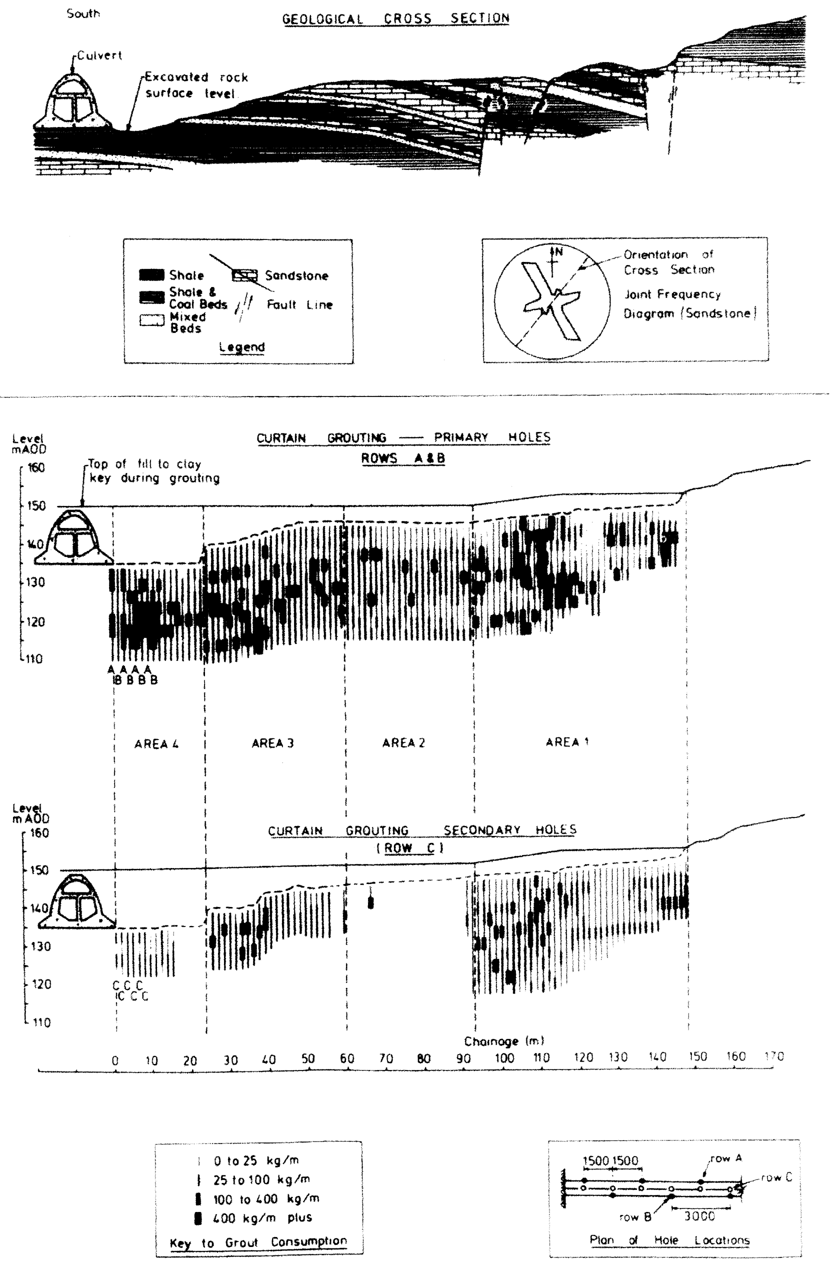


Fig 3. Cross-sections through North Abutment showing geology and grout curtain details

CH 110, Stage 9-10m (i.e. 6-7m into rock). Fresh sandstone
At top of stage:

Theoretical overburden pressure	9 x 0.226 = 2.03 bar.
Hydrostatic pressure:	9 x 0.100 = 0.90 bar.
Specified maximum total stage pressure:	4.83 bar.
∴ Maximum gauge pressure (top of hole):	4.83 – 0.90 = 3.93 bar.
Make first gauge pressure half theoretical overburden:	½ x 2.03 – 0.90 = 0.12 bar.
∴ Range of gauge pressure (R):	3.93 – 0.12 = 3.81 bar.

Hence R/5 = 0.76 bar and R/3 = 1.31 bar.
Thus, applied gauge pressures are:

Step		Gauge pressure (bar)	Note
Ascending stage	0	0.12	Too small to conduct
	1	0.12 + 0.76 = 0.88	–
	2	0.12 + 1.52 = 1.64	–
	3	0.12 + 2.28 = 2.40	–
	4	0.12 + 3.04 = 3.16	–
	5	0.12 + 3.81 = 3.93	Or hydrofracture
Descending stage	6	0.12 + 2.62 = 2.74	
	7	0.12 + 1.31 = 1.43	
	8	0.12	Too small to conduct

Table 1. Calculation of gauge pressures for hydrofracture tests

Stage (hole/ depth)	Theoretical overburden pressure at top of stage (bar)	Maximum total test pressure if WL at rock head (bar) (i.e. gauge + 3m)	Maximum total test pressure if WL below stage (bar) (i.e. gauge + full head	Maximum ratio range
110/9-10	2.03	4.14	4.74	2.04-2.33
110/23-24	5.20	6.43	8.43	1.24-1.62
130/5-6	1.13	2.73	2.93	2.42-2.59
130/8-9	1.81	4.20	4.70	2.32-2.60
130/20-21	4.52	6.70	8.40	1.48-1.86

Table 2. Comparison of theoretical overburden and test pressure

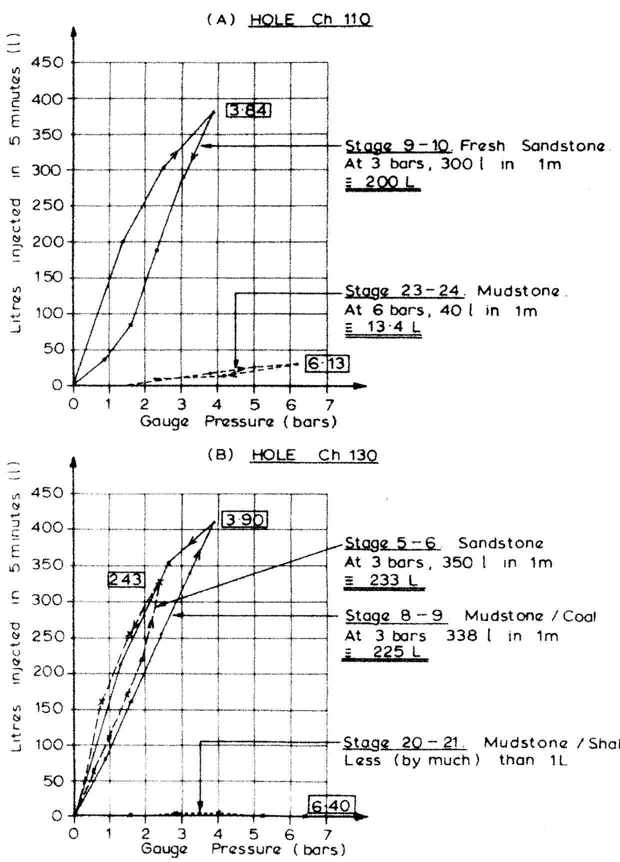


Fig 4. Pressure-flow relationships recorded in hydrofracture test stages in holes at (a) Ch 110 and (b) 130. Maximum test pressure (bars) shown for each stage.

selected as being the most reliable method of intersecting any major discontinuities running obliquely across the line of the curtain.

Reflecting the general design philosophy of 'controlled underseepage', it was intended that the target permeability of the grout curtain should be 10 Lugeons or less. This is in accordance with the views of Houlsby (1976) who, in the same paper, proposed methods for the conduct and interpretation of Lugeon water tests. These methods were to be used in a programme of pre- and post-grouting test holes to monitor the effect of the treatment. Experience in rock of a similar nature suggested that to meet the 10L target, it would not be necessary to continue with successive phases of grouting until the final phase average lineal consumption of 25kg/m, recommended for 'high dams' by Deere (1976), was met. Overall, however, there was a general intention that, although specific procedures were laid down in the design, early water test and grout take data would be assessed with a view to amending and optimizing the execution of the work.

Hydrofracture tests

Two test holes were located at Ch 110 and Ch 130. The concept was to test at least three 1-m stages per hole (two in sandstone, and one in mudstone/shale) in five ascending 15-min steps to a maximum pressure of 8.6 bar, or hydrofracture, and then descend in three steps. The first pressure was half the theoretical overburden pressure. Calculation of step and stage pressures was made as in Table 1. This assumes that overburden pressure is 0.226 bar/m and that the water table is below the stage being tested, thereby ensuring that the maximum specified stage pressure was not exceeded.

Each hole was drilled 50mm diameter to the depth of the bottom of the 1-m stage to be tested. The stage was then isolated by a 3-m long pneumatic, down-the-hole packer. The integrity of the packer *in situ* was checked by holding its inflating pressure for about 10 min prior to the test. Water was supplied to the stage via a high pressure centrifugal pump, through a test apparatus to control and monitor the applied pressure and to measure the flow. During testing the following observations were made, and related to times;

(i) rate of water flow, in 5-min intervals;
(ii) pressure;
(iii) water levels in holes within 50m radius; and
(iv) ground levels at selected locations to check on any heave resulting from the testing.

Five of the six planned stages were tested – stage 5-6m in hole Ch 110 was abandoned since the irregular borehole wall caused the rupturing of three packers in succession. As shown in Fig 4, even in very permeable strata, little deviation from a linear pressure-volume relationship was recorded, although there was a slight increase in the apparent permeability of the ground on the descending leg of each curve.

Very high Lugeon values were associated with sandstone or coal horizons, whilst stages in mudstone/shale were much less permeable (e.g. Ch 130, stage 20-21m). No interconnection of water to adjacent holes or uplift was recorded.

Regarding the relationship between the applied gauge pressure and theoretical overburden pressure, Table 2 shows that these tests covered factors from 1.24 to 2.59 times, depending on the position of the water table. Due to the complex geology involving perched local water tables, and the effects of 'charging' the rock mass, the phreatic surface during the testing proved to be very variable. In general, however, it may be regarded as being below the level of testing.

It was concluded that no hydrofracture had been recorded, despite the application of pressures well in excess of twice theoretical overburden. Actual grouting pressures were thus selected to give a suitable safety factor against hydrofracture of uplift, bearing in mind the variability of the ground water level:gauge pressure equivalent to theoretical overburden pressure at the top of the stage, to a maximum of 4 bar.

The selection of safe grouting pressures has received lengthy discussions, for example, by Bjerrum *et al* (1972). As opposed to their findings for clays, where hydrofracturing was recorded at excess head to effective stress ratios of 0.5-1.0, ratios for rocks are much higher.

In cases of laminar rock mass structures, this increase reflects the tensile strength contribution (Haimson 1968), and the '1 psi per foot' rule of thumb is most commonly followed as the safe grouting pressures. Analogous to the Kielder tests, Morgenstern & Vaughan (1963) experimented in thin sandstones and shales, and recorded hydrofracture at 1.2-2.4 times total overburden pressure. More recently, however, Houlsby (1977) re-emphasized the need to consider the rock mass structure when selecting safe grouting pressures (Fig 5), another aspect of his work which has gained general usage. These researches confirm the suitability of the pressures selected for the Kielder grouting.

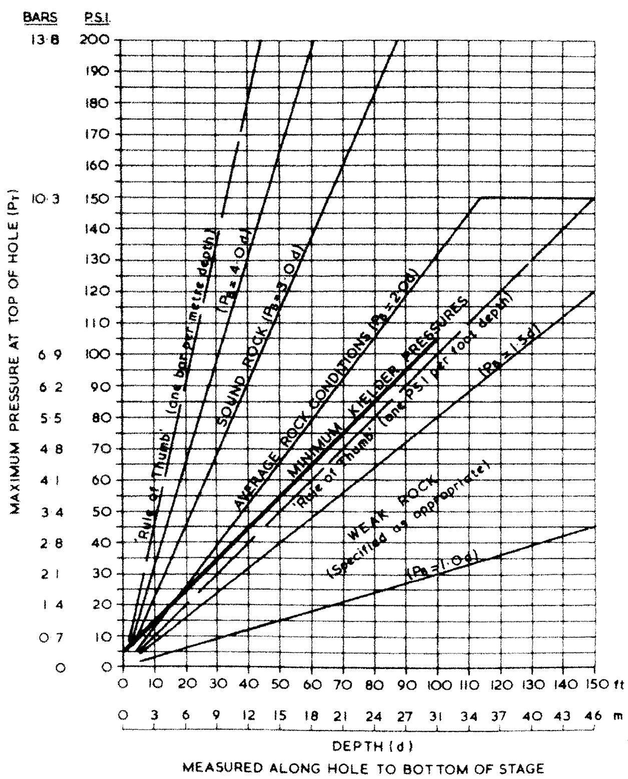


Fig 5. Typical permissible grout pressures for various foundation conditions (pressures shown are at surface: P_B denotes pressure at bottom of stage; d is the depth to there) (after Houlsby, 1977)

CURTAIN GROUTING: EXECUTION

Most of the drilling was conducted with wagon drills, equipped with air-powered rotary percussive top hammers. This method was used to drill through the clay (with air flush), to a maximum depth of 11m (at Ch 25), and 1m into rock. A 50-mm diameter UPVC standpipe was then placed and sealed into the rock with a thick near cement grout. Descending stage drilling (with water flush), water testing and grouting were conducted thereafter (Ferguson & Lancaster-Jones 1964).

From the culvert (Ch 0) to Ch 25 the clay was up to 16m thick and could not be 'open-holed' by the wagon drills. A track rig with top hammer was therefore employed to case the clay to rock head using the ODEX system of overburden penetration. The plastic standpipes were placed inside the steel casing, which was then extracted prior to sealing in the standpipe. Descending stage work then progressed as above.

Grouts were mixed in diesel powered Colcrete colloidal mixers, and passed via agitator tanks to air powered Colcrete Evans ram pumps (Gourlay & Carson 1982) for injection. The grouting station was located overlooking the clay key.

Injection of grouts, and water for simple stage tests, was made through top-hole mechanical packers, set in the standpipes.

The rate and location of clay placing in the trench were controlled by local exigencies and effectively divided the curtain into four areas. In order of treatment, these were:

Area 1 Ch 148.0-94.0
Area 2 Ch 92.5-61.0
Area 4 Ch 23.5- 0
Area 3 Ch 59.5-25.0

A summary of the grouting quantities executed is provided in Table 3, whilst the following features of the work are noteworthy.

Area 1. The curtain varied in depth from five to ten 3m stages (Fig 3), with those in the B row one or two stages in advance of the A row holes. A full secondary programme (C holes) was conducted at 1.5-m centres, and also in 3-m stages. The fact that grout emerged around the periphery of the clay key, and from adjacent boreholes, testified to the thoroughness of the clay-rock contact grouting. Data from the pregrouting water tests (Section 5 & 6) plus experience gained in the early stages, led to the adoption of grouts from w:c (by weight) 7:1 in ten gradual steps to 0.49:1, or until refusal (Bruce 1982).

Area	Holes (no.)	Clay (m)	Rock (m)	Grout stages (no.)	Cement (kg)	Lugeon tests	Simple stage water tests (% of total stages)	Clay thickness (m)	Stage length (m)
Ch 148-94									
A	19	76	450	153	35,802		50	4	3
B	18	71	434	146	53,676	Before & after	57	3-5	3
C	41	163	1,003	333	55,846		97	3-5	3
Ch 92.5-61									
A	11	44	330	110	13,775		99	4	3
B	11	44	330	110	16,400	Before & after	98	4	3
C	2	8	24	5	1,275		0	4	3 + 6
Ch 59.5-25									
A	12	79	357	120	30,650		13	4-11	3
B	12	86	359.5	120	30,200	After	1	4-11	3
C	22	176	594	99	17,900		0	5-11	6
Ch 23.5-5.0									
A	8	125	195	64	18,808		100	15-16	3
B	8	124	190	61	23,550	After	100	14-16	3
C	11	173	264	44	5,500		–	15-16	6
Overall	175	1,169	4,530.5	1,365	303,382				

Table 3. Quantities of work conducted in grout curtain

Area	Cement (kg)	Rock (m)	Curtain area (m^2)	∴ Average lineal take (kg/m)	∴ Reduction ratio % $\left(\frac{C}{A+B}\right)$	Average areal take (kg/m^2)
1. (Ch 148-94)			1,225			
A	35,802	450		79.6 } 101.2		
B	53,676	434		123.7	55	118.6
C	55,846	1,003		55.7		
2. (Ch 92.5-61)			992			
A	13,775	330		41.7 } 45.7		
B	16,400	330		40.7	116	31.7
C	1,275	24		53.1		
3. (Ch 59.5-25)			1,103			
A	30,650	357		85.9 } 84.9		
B	30,200	359.5		84.0	35	71.4
C	17,900	594		30.1		
4. (Ch 23.5-0)			622			
A	18,808	195		96.5 } 110.0		
B	23,550	190		123.9	19	76.9
C	5,500	264		20.8		
Overall	303,382	4,530.5	3,942	74.4 } 84.2	51	77.0
				94.3		
				42.7		
				Average overall = 67.0		

Summary	No.	Rock (m)	Cement (kg)	Average consumption (kg/m)
All A holes	50	1,332	99,035	74.4 } 84.2
All B holes	49	1,313,5	123,826	94.3
All C holes	76	1,885	80,521	42.7

Table 4. Lineal and areal grout take summary

Based on the results from this first, intensively grouted area, the following philosophy was declared by the Engineer for the remaining work.

(i) Stage length to be 3m in primary holes and 6m in secondary and subsequent phases.

(ii) If the stage consumption is less than 500kg in any stage of a completed primary hole, no adjacent secondary holes will be required.

(iii) If the stage consumption is frequently and significantly greater than 500kg in several holes, secondary holes will be required in that area.

(iv) If a single primary hole has one stage of consumption above 1,000kg, a single secondary hole will be installed in 6-m stages to that depth.

(v) No tertiary holes will be required unless in any one 6-m secondary stage, the total grout take exceeds 1,500kg in the corresponding levels of two or more adjacent holes.

Area 2. Grouting proceeded to this pattern: the primary holes were ten 3-m stages into rock. Two secondaries descended in 6-m stages to the level of a large primary take (1,000kg).

Area 3. The A row grouting was usually one stage in advance of the B row treatment, and both extended for ten 3m stages. This was followed by an almost full secondary programme in 6-m stages, with again, no call for tertiary holes.

Area 4. A full set of secondary holes was installed to Ch 16, the curtain depth being 24m into rock, and the B row one stage in advance.

A summary of lineal and areal consumptions, and Reduction Ratios, is provided in Table 4.

These data amplified by diagrammatic representation of lineal grout takes (e.g. in Fig 3), highlighted the especially badly fissured and faulted nature of the rock in Area 1. Grouting to an average areal consumption of almost $119 kg/m^2$ was necessary there to produce a curtain to acceptable standards – over three times the intensity which proved satisfactory in the adjacent Area 2.

CURTAIN GROUTING: EFFECTIVENESS – LUGEON TESTING PROGRAMME

The specification (see section on design) called for Lugeon testing before and after grouting, this being a more sophisticated and reliable approach than surveying grout stage water test values, conducted at one constant pressure over only 10 min. Such simple tests do, of course, provide useful background data in the course of routine grouting operations, and were conducted where thought most appropriate (Table 3). Two pre-grouting tests were conducted in each of Areas 1 and 2 (Ch 120, 100, 80 and 60), and were followed by post-grouting tests at Ch 109.75 and 82.75. By this point, it was felt that sufficient data had been accumulated on the permeability of the virgin rock mass, also from the stage water test and grout details, and so only post-grouting tests were carried out in Areas 3 and 4 (Ch 34.75 and 10.75).

Thus, a total of four pre-grouting holes (37 3-m stages) and four post-grouting (38 3-m stages) were tested. It was considered that the intensity of testing (one hole per 18m of curtain length) was adequate for a contract of this scale.

Standpipes were installed and the stages drilled as for the grouting. Down-the-hole packers were used as for the hydrofracture testing with the bottom of the hole forming the base of each 3-m stage. Holes were vertical and located on the curtain centre line. They generally extended to the base of the grout curtain, although they could be terminated at 30m if shale were encountered.

Each stage was tested at five gauge pressures in the sequence a-b-c-b-a, with the flow over a 10-min period of stable conditions being recorded. These pressures were calculated as follows:

Step a. Low pressure up and down: 0.4 x 0.226 x depth to bottom of stage, to a maximum of 3.4 bar.

Step b. medium pressure up and down: 0.7 x 0.226 x depth (m), to a maximum of 6.9 bar.

Step c. High pressure: 1.0 x 0.226 x depth (m), to a maximum of 10.0 bar.

An equivalent Lugeon value was calculated for each test pressure:

$$\text{Lugeon value} = \text{water consumed (l/min)} \times \frac{10}{\text{test pressure (bar)}}$$

and then a representative Lugeon value was selected for each stage with regard to Houlsby's chart (Fig 6). As in the Houlsby method, no correction was made for head loss, frictional effects, or location of water table.

Details of the stages tested are provided in Tables 5 and 6. 'Flow Group' refers to the flow type designations shown in Fig 6.

Regarding the pre-grouting results, a wide range of values may be noted, from virtually impermeable (mudstone/shale) to unmeasurably large in fractured in fractured sandstone. Although 49% were 5L or less, 35% were greater than 20L, and these stages, like the extremely low values, tended to congregate in distinct pockets. Conversely, the post-grouting values did range to 28L, but 74% were 5L or less and barely 5% were above 20L.

Table 5. Pre-grouting Lugeon test results (numbers in margins refer to depth below surface (m))

CH60

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–4	–	–	standpipe through clay
4–7	1.66	>100 B	–
7–10	2.27	>100 B	–
10–13	3.00	73 B	–
13–16	3.67	7 A	–
16–19	4.33	<1 A	–
19–22	4.93	<1 A	–
22–25	5.66	<1 A	–
25–28	6.33	<1 A	–
28–31	4.00	76 E	7 bar unattainable due to high flow
31–34	2.67	>100 B	7.66 bar unattainable due to high flow

CH80

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–4	–	–	standpipe through clay
4–7	1.66	>100 B	3 packers burst in shale/sstn sequence
7–10	2.27	<1 A	–
10–13	3.00	1 C	–
13–16	3.67	<1 A	–
16–19	4.33	1 A	–
19–22	5.00	7 C	–
22–25	5.33	73 D	5.66 bar unattainable due to high flow
25–28	6.33	23 A	–
28–31	2.06	>100 D	7 bar unattainable due to high flow
31–34	2.46	>100 D	7.66 bar unattainable due to high flow

CH100

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–6	–	–	standpipe through clay and rock
6–9	1.93	0 –	–
9–12	2.63	5 C	–
12–15	1.00	>100 B	3.35 bar unattainable due to high flow
15–18	3.60	>100 B	3.95 bar unattainable due to high flow
18–21	4.60	5 D	–
21–24	5.27	3 C	–
24–27	5.80	4 C	–
27–30	6.53	4 A	–
30–35	7.60	9 C	–

CH120

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–8	–	–	standpipe through clay and rock
8–11	2.40	1 A	–
11–14	3.00	6 C	–
14–17	3.66	3 C	–
17–20	4.26	49 C	–
20–23	4.93	6 C	–
23–27	5.73	9 A	–
27–29	6.27	5 D	
29–32	6.93	0 –	–

Table 6. Post-grouting Lugeon test results (numbers in margins refer to depth below surface (m))

CH10.75

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–16	–	–	Standpipe through clay
16–19	3.79	3 A	–
19–22	4.48	12 D	–
22–25	5.17	<1 A	–
25–28	5.86	<1 A	–
28–31	6.55	28 B	–
31–34	7.24	13 D	–
34–37	7.79	8 A	–
37–40	8.62	5 C	–

CH34.75

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–9	–	–	standpipe through clay
9–12	2.07	3 A	–
12–15	2.76	5 A	–
15–18	3.45	5 E	–
18–21	4.14	5 A	–
21–24	4.83	5 A	packer leakage
24–27	5.52	4 C	–
27–30	6.61	5 D	–
30–33	6.90	2 A	–
33–36	7.59	14 D	–
36–39	7.59	9 D	8.28 bar unattainable due to high flow

CH82.75

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–4	–	–	standpipe through clay
4–7	1.03	0 –	–
7–10	1.72	<1 A	–
10–13	2.41	<1 A	–
13–16	3.10	<1 A	–
16–19	3.79	<1 A	–
19–22	4.48	<1 A	–
22–25	3.10	22 B	5.17 bar unattainable due to high flow
25–28	5.86	18 D	–
28–31	4.83	11 B	6.55 bar unattainable due to high flow
31–34	4.83	9 D	7.24 bar unattainable due to high flow

CH109.75

	Maximum pressure (bar)	Lugeon value and flow group	Notes
0–4	–	–	standpipe through clay
4–7	1.03	4 E	–
7–10	1.72	4 A	–
10–13	2.41	1 A	–
13–16	3.10	<1 A	–
16–19	3.79	3 A	packer burst on last step
19–22	4.48	0 –	–
22–25	5.17	<1 A	–
25–28	5.86	<1 A	–
28–31	6.55	<1 A	–
31–34	7.24	<1 A	–

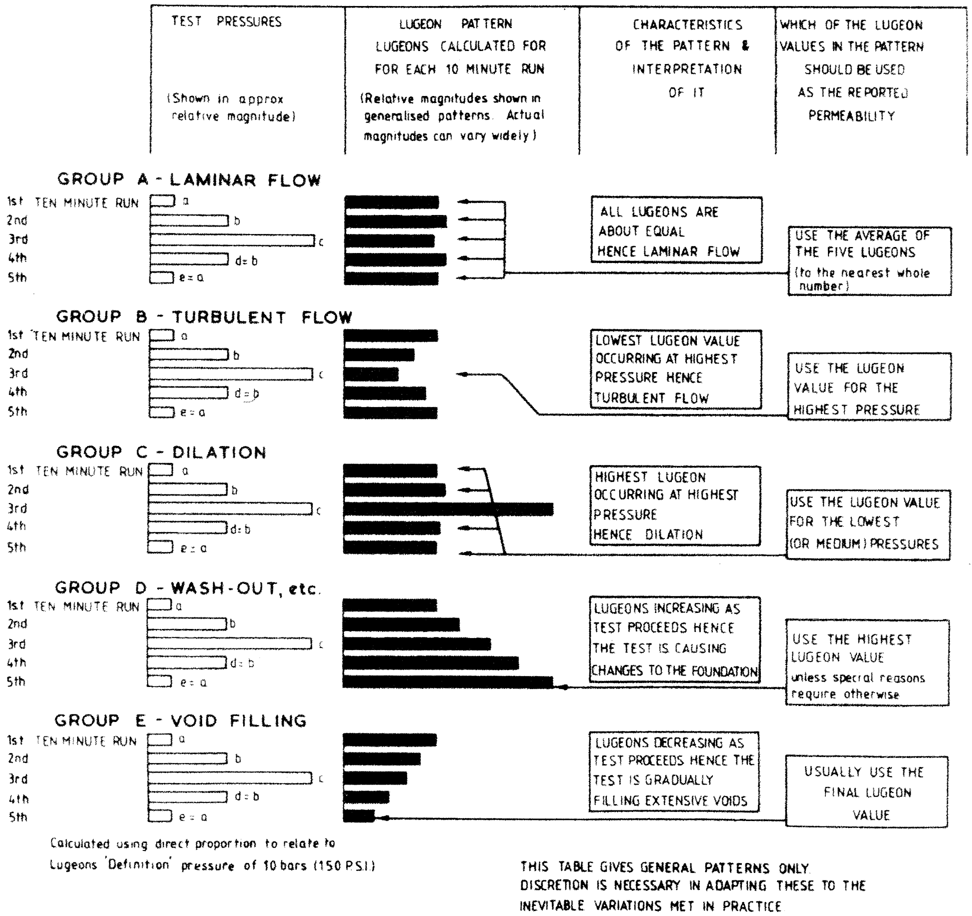


Fig 6. Interpretation of Lugeon test results (after Houlsby 1976).

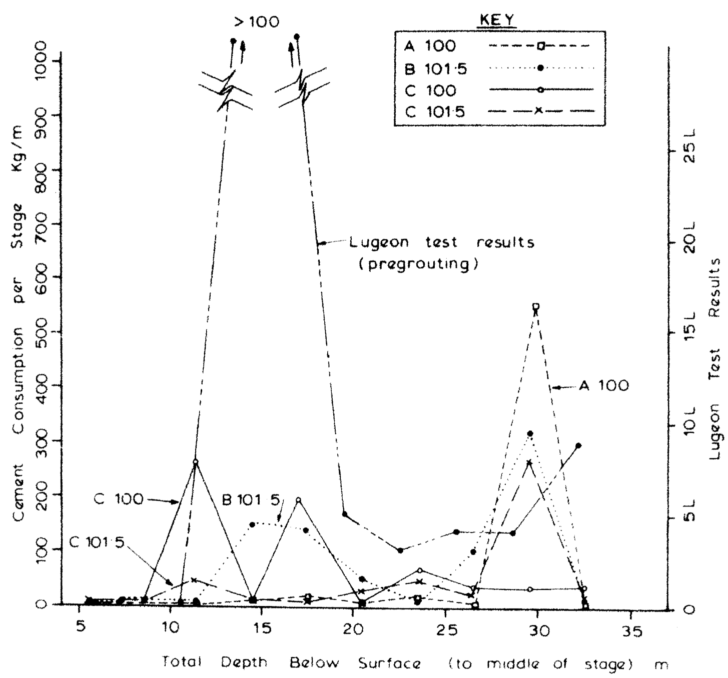


Fig 7. Relationship of Lugeon values from pre-grouting water test hole Ch 100 to grout takes from the four closest grout holes.

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

Table 8. Proposed grout consumption classification (Deere, 1976)

ferent response of the two major lithologies, also noticeable from the grout takes: a large proportion of 'tight' stages (corresponding to mudstone/shale), and an even larger percentage of takes of 4L or more (fissured sandstone).

RELATION OF WATER TESTING AND GROUTING DATA

Houlsby (1976) notes that geological factors such as wall roughness, frequency, orientation and geometry, strongly influence flow through fissures. One consequence is a considerable scatter of results from even neighbouring holes in apparently uniform areas. On these grounds alone, close correlation may not be expected between water test results and earlier, or subsequent, grout takes. Three examples are of particular interest:

(i) Figure 7 shows the Lugeon values obtained from the pre-grouting Test Hole Ch 100, plotted with the subsequent stage grout takes from the four closest holes. The Lugeon values are very high, between 13 and 19m, and moderately high value below 30m. The other seven stages gave values of 5L or less. Conversely, the range in grout takes for the primary holes (especially the A hole) was much smaller, with only minor peaks from 13-19m, but more significant maxima from 28-31m. Likewise, the secondary takes, uniformly smaller, also showed their major peaks in the 28-31m band. Zones impermeable to water, however, also proved tight to grout.

(ii) Figure 8 shows similar graphs for post-grouting Test Holes Ch 10.75 and 109.75.

In the former, secondary hole grouting gave 'low' to 'very low' (Table 8) takes of the scale acceptable as 'refusal' under Deere's 'high dams' target (1976), and excellent reduction ratios were achieved. However, the subsequent water test holes still showed stages with permeabilities of up to 28L, and an overall pattern not dissimilar to the first phase of grouting (B holes). In contrast, the grout holes around Ch 109.75 showed high and irregular takes although peaks were apparent from 17-23m and around 26m. The post-grouting water tests were, however, 4L or less, indicating a marked tightening of the ground, especially in the zones of highest grout consumption.

From these examples, the following fundamental observations may be made.

(i) Even when holes are as close as 1.5m, direct correlation of grout and/or water test values may not be noted.

(ii) A low water absorption will forecast a low grout take, but a low grout take need not be followed by a low water absorption.

(iii) The relative magnitude of Lugeon values is no certain guide to the magnitude (relative or absolute) of the subsequent grout takes.

(iv) When an area is grouted virtually to refusal, as indicated by cement grout takes, it may still yield moderate to high Lugeon values.

Explanation for the last three points in particular, is most plausibly found in a paper by Littlejohn (1975), and is related to the particulate nature of cement grouts. It is generally assumed that the minimum fissure width which can be suitably treated by cement grouts is 160μ. Using the nomogram (Fig 9), it may be estimated that a single 160μ fissure in a 3-m stage will give a flow equivalent to 10L. A single 100μ fissure will give a flow equivalent to 2L. Thus, a stage with a measured permeability of, say, 20L could, in a simple example, be intersecting (a) 10 No.100μ fissures or (b) 2 No.160μ fissures. Even using correct injection procedures, cement grout *would not* penetrate into the rock mass around the stage in case (a) but would merely clog the inside surface of the borehole by the pressure-filtration effect. Conversely, case (b) *could* be grouted and the fissure sealed for a certain distance around this hole. A post-grouting water test located nearby would, therefore, indicate *20L* still, in case (a) (despite the grouting record indicating a 'tight' hole, e.g. Fig 7) or *a very low value* (e.g. Fig 8) in case (b), where the previous grout take may have been substantial.

It is further significant that, of the 31 post-grouting stages of permeability 10L or less, 22 were interpreted as having 'laminar' (Group A) flow characteristics, whilst none appeared 'turbulent' (Group B). As Houlsby (1976) notes, 'finer' fissures are 'liable to exhibit laminar flow' whilst 'wider ones will have turbulent flow'. Of the seven takes above 10L, three were 'turbulent', suggesting, retrospectively, that further grouting would have been efficient in treating the remnant wider fissures and providing locally a tighter curtain. However, a sufficient grouting standard was considered to have been achieved by the work conducted, especially so since the final permeability of the grouted section overall was substantially lower than that of the remainder of the site.

CONCLUSIONS

The rock grouting programme at Kielder Dam has provided an early opportunity to implement in Britain the proposals on Lugeon water testing made by Houlsby in 1976. These tests, in special holes located along the curtain axis, were conducted before and after curtain grouting and illustrate clearly the effects of the grouting on rock mass permeability. The proposed method was found to be practical and, understandably, yielded a quality of information far higher than that obtained from the short duration, single pressure tests commonly conducted on grout stages prior to their grouting. When using cement based mixes, the particulate nature of the grout must always be borne in mind if attempting assessment of 'refusal' standards or comparing grout take and water test values. Conclusions drawn on the basis of only one of these sets of data may be erroneous. The Kielder work also provides a good example of the rational selection of maximum grouting pressures, when based on field trials conducted in a similar manner to the Lugeon test programmes.

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Front cover photograph courtesy of Northumbrian Water.

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