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Slab-jacking at Tarbela Dam, Pakistan

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This Paper describes a major slab-jacking programme conducted recently at Tarbela Dam, Pakistan. Some 375m³ of mainly cement-bentonite grouts were injected through 1 302 holes drilled through 39 of the concrete panels forming the protective apron to the upstream clay blanket material of the auxiliary spillway.

The purpose of the work was to restore full contact between the panels and the blanket, and to lift the slabs to their original attitudes following localised erosion of the clay.

Data are provided on the design, control and effects of the slabjacking together with fluid and set properties of the grouts employed.

Introduction

THE TARBELA DAM COMPLEX is one of the world's greatest water resource developments and is located 70km north west of Islamabad, Pakistan (Fig. 1). The principal element of the project is the main embankment dam, 2 740m long, 143m high and comprising 130 million cubic metres of materials. Other major features are two power and two irrigation tunnels on the right bank, the two smaller auxiliary dams (17.5 million cubic metres) and the two massive concrete spillways (Fig. 2), separated by an additional irrigation tunnel on the left bank.

The service spillway, with seven gates discharging water into the Indus River from July to September, has seen enormous rollcrete, concrete and rock anchor protection works being carried out in its plunge pool from 1977 to 1980 (e.g. Lowe *et. al.*, 1979). More recently, similar attention has focused on the auxiliary spillway structure which is designed to operate only for short periods of exceptionally high reservoir inflows, and when for any reason the service spillway is inoperable.

During the 1979-80 construction season, an important undertaking in this latter location was the contact grouting and lifting of certain concrete panels forming the upstream protective apron (Fig. 3). Apart from its large scale — 375m³ of grouts were injected under an area of 6 130m² — the slab-jacking operation was remarkable in that it was conducted with conventional grouting equipment and executed following basic engineering procedures and principles. This Paper details the development of these principles in the light of the successful conclusion of the work.

General and historical background

Structure/geology interaction

The auxiliary spillway consists of 22 concrete gravity monoliths, the crest structure of which is curved in plan. Construction was carried out between October 1970 and August 1974.

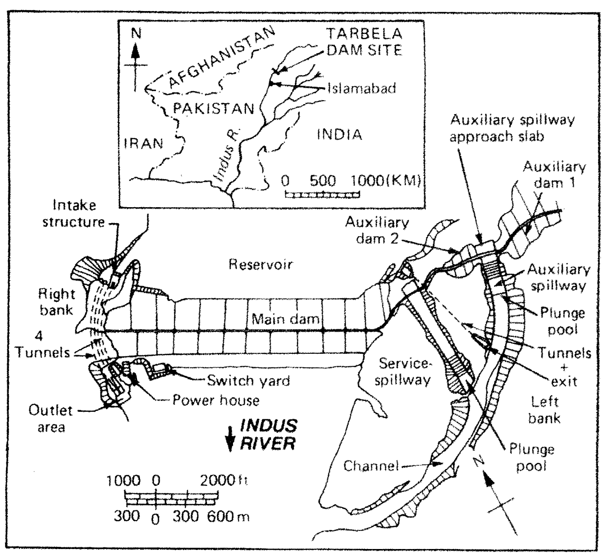
The foundation rock is predominantly limestone (Fig. 4) and dolomitic limestones, massive in the centre but more thinly bedded, and ferruginous and phyllitic to the flanks. The limestones contain small karstic features, with soft infill materials. There are also some medium beds of "sugary limestone" (calcarenite) which are porous. The phyllites are schistose, occasionally talcosic and slightly metamorphosed. To the right of the structure is a major dolerite intrusion with occasional serpentinised seams. All the lithologies are weathered, with some beds of soil-like character.

These rocks constitute a relatively soft and weak zone squeezed between a huge mass of quartzite to the left, and massive dolomite with igneous instrusions to the right. Within this zone there are shear faults, overturned folds and tension cracks. The major discontinuities are *bedding and fold* joints (striking N45 - 65°E, dipping 30 - 75°E), *tension* joints (striking N45 - 55°W, open and spaced 4.5 - 9m) and *shear faults* (striking N30°E, cutting all other joints, and having gouge filled material). The high permeability ($1 - 3 \times 10^{-4}$ cm/sec) of the foundations is related to flow through the karstic network in the limestone, evidently controlled by the jointing.

To control seepage and reduce uplift pressures related to these adverse geological conditions, two grout curtains (Fig. 4), and an extensive drainage system were constructed (Szalay, 1976). The upstream triple row curtain and the spillway structure were linked horizontally by a 68m long blanket of compacted B1 material*,

*B1 material is a well graded blend of sound durable angular gravel, sand and silt, with 20-40% finer than No. 200 sieve. Average permeability 1.3×10^{-7} cm/sec. At the thickest section the design assumed maximum consolidation of 50mm

Fig. 1. Tarbela Dam complex, Pakistan



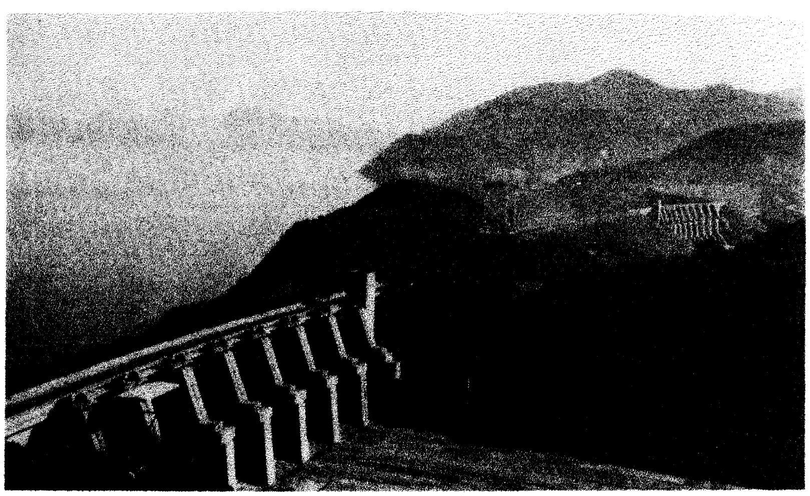


Fig. 2. General view of service spillway gates (foreground), Auxiliary Dam 2, auxiliary spillway gates, and the curved Auxiliary Dam 1, right

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laid on the surface of the rock. The blanket ties laterally into the cores of the Auxiliary Dams 1 and 2, and varies in thickness from 9.5m in the trench immediately on the upstream side of the spillway headworks to 1.5m at its upstream edge. In the trench, the rock had been treated by consolidation grouting and the foundation rock of the B1 was subjected to dental concrete as required. The purpose of the B1 blanket is to lengthen the potential seepage path so as to lower hydraulic gradients and thus uplift pressures under the spillway. The thickening of the blanket next to the spillway structure was to provide a significant seepage path at the soil/concrete interface.

The blanket is capped with a 0.3 - 0.6m thick concrete apron (Fig. 3), constructed in panels, typically 15 × 11m, except at locations permitting articulation so that the slab could follow any differential settlement of the blanket (Fig. 5). Waterstops are installed at all joints between the panels. The purpose of the apron is to prevent erosion of the blanket due to spillway operation and to preclude the introduction of water at full reservoir pressure at the top of the blanket at the soil/concrete interface at the spillway structure.

Monitoring history

Continuous monitoring of the structure/rock system has been maintained from the first impounding in 1975. In 1978, all piezometers recorded significant increases during reservoir infill. Although these increases resulted in pressures which were well within the design requirements for uplift, there was concern that the increases were at least partly due to water bypassing the apron and blanket by seeping down the upstream face of the headworks, and that the situation might deteriorate to dangerous levels with time. While the reservoir was still above apron level, salt conductivity tests were conducted which did indeed indicate an important defect in the blanket integrity close to the headworks.

Investigations disclosed a triangular shaped void about 350mm deep next to the spillway structure. It was continuous and had connection to the reservoir at the left and right slab extremities. The presence of slightly washed zones in the test pits further indicated some movement of fines at the interface. Thus, the intent of the design to preclude having water at full reservoir pressure at the soil/concrete interface had been frustrated. Two phenomena were thought to have led to this situation:

(*i*) Separation occurring along the contact due to the rotation of the headworks under high reservoir pressure, and

(*ii*) The connection between the apron and spillway structure not functioning as intended. As the fill settled under reservoir pressure the apron was hanging up at the spillway structure allowing the formation of a void to which reservoir water had access from the sides. The water then entered the rotation crack washing B1 material down the crack. This resulted in the deeper triangular shaped depression at the concrete/soil interface.

Associated settlements of other panels, including differential movements, had also been recorded, and by early 1980 these ranged from 30mm to 180mm, at distances of 6m to 18m from the headworks. The

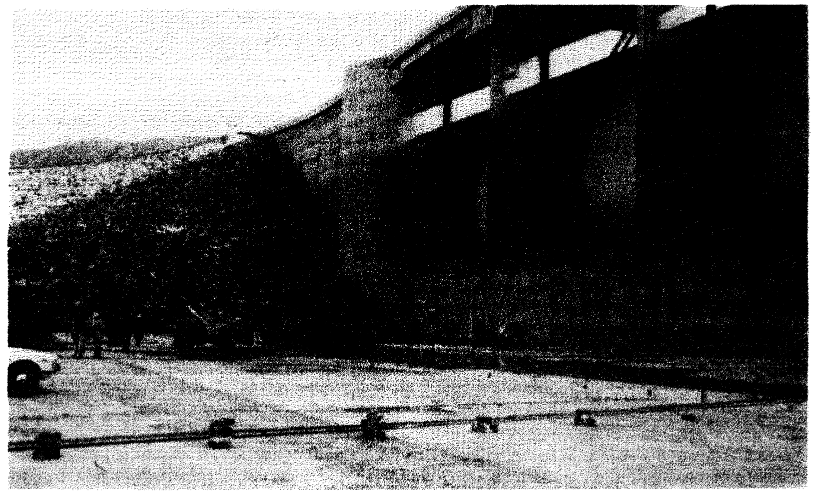

Fig. 3. The concrete panels forming the protective apron on the upstream side of the auxiliary spillway headworks. Auxiliary Dam 1 to the left

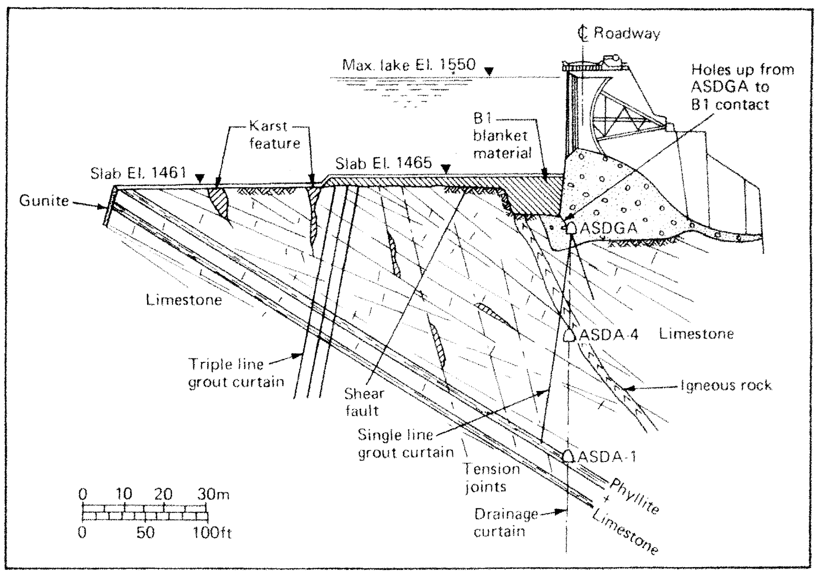

Fig. 4. General cross-section of auxiliary spillway upstream protective works

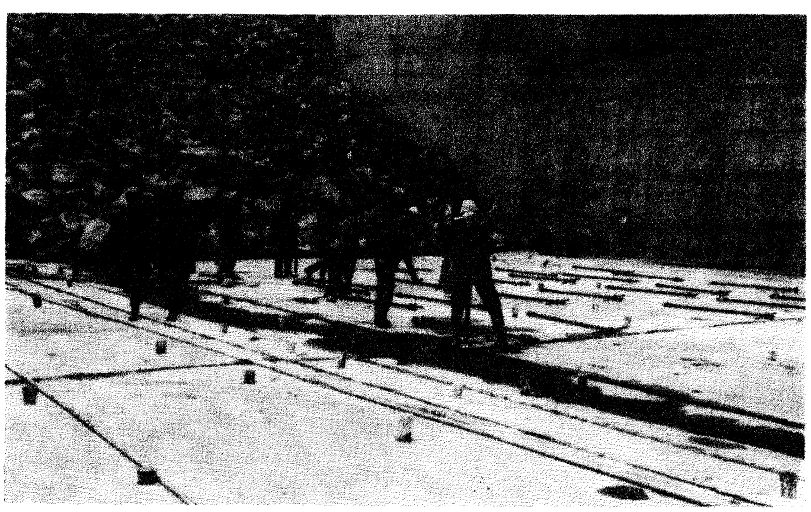

Fig. 5. Wooden bungs placed in holes to arrest grout interconnections. This photograph shows some of the narrow slabs of the articulation

panels were quite uniform above the 1.5m of B1 (Panels 31 - 50), but the greatest settlements generally occurred over the thickest zone of B1, and were on average four times higher. Individual panels were also distorted, with the average of the maximum elevation difference on a single panel being 40mm and the average maximum difference to adjacent panels being of the same magnitude. There was concern that these differential movements could have resulted in voids beneath the hanging slabs and could be distressing waterstops to the point of failure.

As an interim measure the void at the spillway structure was grouted in 1979. Definitive remedial measures were then designed for execution in 1980 and 1981. These measures for the apron included revisions to ensure that the apron could follow differential settlement of the fill without opening voids, and slab-jacking to fill possible voids and to even out the differential settlement which had taken place. The slab-jacking also would automatically check the integrity of the waterstops.

Planning, preparation and monitoring

Proposed method

During the preliminary treatment in 1979, it was verified that without exerting excessive grouting pressures, uplift of individual panels could readily be achieved. Following the experience gained in these early steps, the following outline method was proposed:

(*i*) Drill 75mm diameter holes at approximately 2m centres on a square grid through each slab and up to 300mm into the B1 material, carefully logging the presence of voids or loss of flush.

(*ii*) Backfill holes encountering voids by pouring in a cement-bentonite mix, adding sand to the grout if the take and grout travel were substantial.

(*iii*) After at least 24 hours, pressure grout to 2 bars with less viscous cement-bentonite mixes, lifting adjacent slabs in steps limited to 25mm to prevent damage to the waterstops. Interconnections to be plugged off with wooden bungs (Fig. 5).

(*iv*) Make final fine adjustments to levels, and back-fill drill holes with a compacted dry mix of cement, water and sand, topped with 50mm of an epoxy cement and sand mix.

Grout mix design

The prime criterion was that the set grout should have deformation characteristics compatible with the B1 material i.e. E = 70 - 130N/mm^2 (approx. 10 000 - 19 000 psi). In addition, the mix did not require high strength, but had to be stable (i.e. minimal bleed capacity) to ensure intimate slab/B1 contact after setting. The fluid mix had also to be sufficiently viscous to have restricted, controllable travel during injection.

Earlier grouting operations at Tarbela had led to the development of cement (OPC), or cement-sand, mixes with relatively large amounts of local bentonite, and this general outline was again adopted initially. The bentonite had 100% passing 0.07mm, although 20-30% was in the silt range. The liquid limit was 141, the Plasticity Index 105 and G_s = 2.72.

Back-fill mixes were thus of the form cement (c): bentonite (b): sand (s): water (w) — 20(c): 40(b): 0 to 20(s): 100(w), and pressure grouting was foreseen with c: b: w of 25: 30: 100 ("A/2 mix") — all ratios by weight.

Plant and equipment

The holes were drilled with an Ingersoll Rand CM350 track rig fitted with a light pneumatic drifter, and using air flush. A covered grouting station was assembled nearby featuring standard high speed mixers, storage tanks, and hydraulic ram pumps. It was anticipated that the delicate pressure grouting operations would require the use of a hand pump, but this was soon superceded on grounds of speed and expedience by a circulating line system from the grouting pumps. This permitted the pressure actually exerted at the injection point to be carefully controlled.

Injection was made through mechanical packers, set towards the invert of the concrete slab, and 2m long to ensure the pressure of a nominal head of grout between the cessation of pumping and first set of the mix (Figs. 5 & 6).

Simple tools were made specially for jetting-out holes after first set, and for scouring the apron of surplus grout. Movements of the panels were monitored principally by theodolite and staff, at an average of nine control points per panel (each corner, each side, mid-point and the centre). In addition, visual assessments at slab edges, and observations on surface standing water were also useful in judging the rate and location of uplifts. A level was taken at all points at approximately 5 minute intervals during injection and more frequently when movement was occurring. Consequently, the way in which the panels were raised was tightly controlled, facilitating important operational decisions on, for example, which holes were to be used for injection, and for how long. Throughout the slab-jacking, the drains below the spillway were monitored for grout contamination.

Execution of the work

Hole drilling

A total of 1 302 holes were drilled in the 6 130m^2 area to be treated (average number — 25 per slab: range 12 to 36, depending on size). None were drilled within 900mm of a panel edge, to avoid damage to the concrete.

Voids were logged beneath the apron primarily under the thin articulation panels above the shallowing of the blanket — the zone of maximum depression. The maxi-

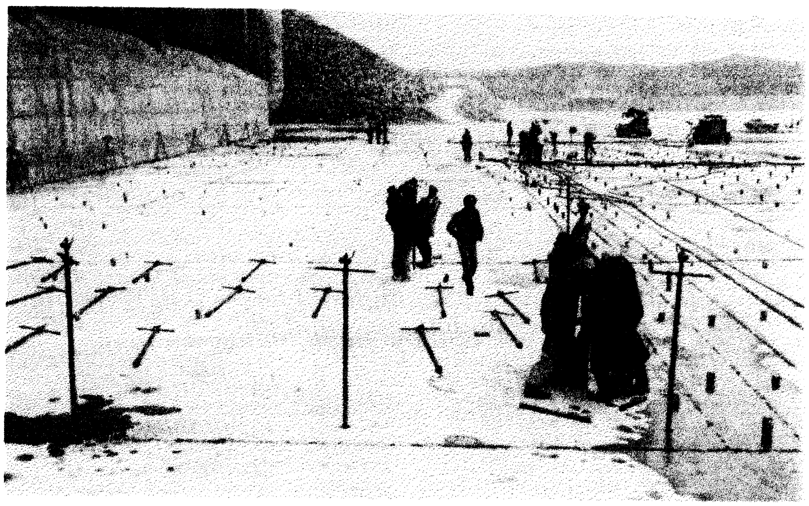

Fig. 6. Mechanical grout packers placed in drill holes in preparation for panel lifting

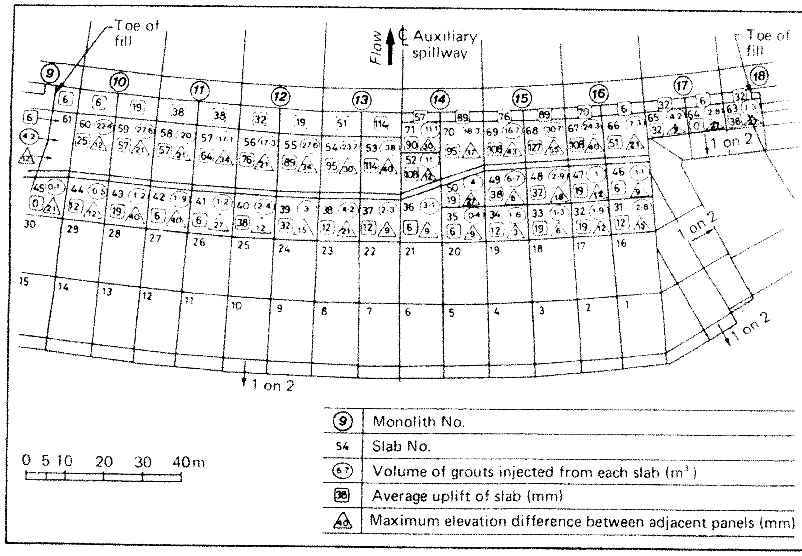

Fig. 7. Grout consumptions, average up-lift and relative attitudes of panels, after slab-jacking

mum void depth was 275mm (Panel 68) with other voids of up to 200mm noted towards the spillway. Elsewhere the general limit was 50mm.

Back-fill grouting

The original concept of gravity backfilling was pursued, but early results indicated nominal consumptions with no interconnections. Indeed, only Panel 38 (Fig. 7) accepted a volume of grout beyond the initial cement-bentonite starting mix. By the same point, it was recognised that the initial pressure grouting with c:b.w — 25: 30: 100 to 70 mixes was easily finding connections with holes up to three slabs distant. It was concluded that the first objective of void infill and contact grouting could be effectively achieved by the methods originally intended for just the uplift grouting. Gravity grouting was, therefore, soon discontinued except for final 'topping up' of holes at the conclusion of the pressure grouting.

Pressure grouting

The pressure grouting commenced under Panel 44 (Fig. 7), and progressed from left to right and in a general way towards the headworks. The lifting of the lower slabs was initiated from one of the peripheral holes on one edge only, and continued across the slab. As the grout migrated, wooden plugs were driven into the holes from which grout emerged. Practice was to permit travel of up to 6m before further packers were then installed nearer to the travelling face of the grout and injection transferred to them. When approximately two-thirds of the slab had been so treated, the grout line was returned to the first packer to attempt to lift that part of the slab.

An initial phase of lifting was conducted on a two shift per day basis, treating each of the lowermost 32 slabs at least once. During this period, experience was gained which proved of great value in the later stages of "fine adjustments". For example, although the mix with 70 litres of water had proved very penetrative, it was, by the same token, too fluid to yield controlled localised uplifts. Progressively thicker mixes were tried, within the overall concept of the design requirements, until a "52 litre" mix was found to be the most viscous that could be mixed and pumped given the site conditions (more than 100m of 20mm i.d. steel grout line from pump to injection point).

The second phase of pressure grouting, including the levelling up of the panels to grade, was conducted during the day shift only, in order to maximise control of operations. As familiarity with grout travels and effects grew, lifting was commenced from the lower one-third of panels, often via up to three simultaneous injection points. This phase concluded with the injection of the '70 litre' mix under Panels 31 to 43 to ensure the filling of any interstices left empty by uplifting with the more viscous mixes. No grout entry to the drains beneath the spillway structure was recorded at any time.

Packer pressures to initiate slab 'floating' varied considerably, with the size of the slab and the joint friction effect being critical factors. Pressures up to 7 bars were required — briefly — in certain cases, although this commonly dropped to well below the limiting 2 bars pressure during continued injection.

Whilst the slabs were all below the required grade, the problem of unintentionally lifting adjacent slabs was not critical. However, as slabs approached level, grout travel and edge friction rendered the achievement of absolute levels extremely difficult. In particular, the intended raising of the narrow panels in the articulation sections proved almost impossible to effect. Accepting that more viscous grouts were not feasible, the best practical compromise was to make the final levelling goal the securing of a smooth profile across the whole area. Caution was also required to forestall slabs from undue bowing, although in a few panels the uneven loading caused by the variable friction effects initiated minor surface cracking.

Joint leakage occurred only between Panels 44 and 45, where the waterstop was subsequently removed and replaced. This verified the continued integrity of all the other waterstops.

Control of the operations demanded the closest liaison between the Engineer and the grouting contractor, especially so in the final phase of delicate adjustments. This control was exercised by a "decision making" engineer, present full time on site, and directly co-ordinating and assessing the grouting survey data. It will be noted that the operation, as conducted, reflected the ASCE "Good Practice" guidelines (1977).

Grouting records and properties

The volume of grouts injected under each panel is shown in Fig. 7, the amounts including both back-fill and pressure grouting mixes. The total number of batches injected was 4 490, giving a total net volume of 375m³. Basic fluid mix properties were determined (Table I) for the three most important mixes, by standard tests on samples drawn from the grouting station storage tanks. In all cases, the bentonite had been mixed with water in the high speed mixer for at least 2 minutes prior to the addition of the cement and a further 5 minutes mixing. Data on strength and deformability were also obtained on cylindrical specimens (Table II). These results confirmed that the grouts matched the characteristics of the B1 material as required.

Effect of the slab-jacking

A detailed survey of the slabs after the "visual smooth profile" aim had been achieved (Fig. 7) showed that the average maximum elevation difference on any panel had been reduced to 24mm (from a maximum of 40mm), and the average maximum elevation difference between adjacent large panels had been dropped to 20mm (from 40mm). Differences between the thin panels were higher. A maximum uplift of 127mm was recorded under Panel 67, whilst by far the lowest region (i.e. the thin strips between Panels 46 and 66) was barely 75mm below the nominal apron elevation of 1 460 ft. The slab-jacking had thus reduced the distortion of the individual panels, and had restored a uniformity of elevation.

By considering the area of each panel, and its average elevation difference, the net volume change was about 240m³, which was less than the volume of 375m³ of injected grout. Part of the difference may be due to the filling of voids, with the remainder due to compression by the overlying slab on the bentonite rich mixes, wastage, and lateral travel of the grout beyond the limits of the apron. Evidence of the last item was seen in later work conducted to improve articulation of the apron. In comparing the grout takes per panel with the changes in elevation, those further from the headworks (i.e. Panels 31 to 45) took more than was required for the measured uplift — a feature attributable to grout interconnection with adjacent panels. Significantly, those closer to the headworks in general showed the opposite relation.

Later indications of the effectiveness of the grouting in restoring the blanket-

TABLE I. FLUID GROUT PROPERTIES

Mix composition (by weight) c:b:w:	Marsh cone (secs)	Colcrete flowmeter (mm)	Bleed capacity %	Fluid s.g.	Comments
25:30:100	39	>>850	0.8-1.4	1.32	Standard "A/2 mix" used in other site applications
25:30:70	60-75	500-550	0.6	1.40	Back-fill mix
25:30.52	Too viscous to exit continuously	210 (virtually zero slump)	Zero	1.53	Final, most viscous pressure grouting mix

TABLE II. STRENGTH AND DEFORMABILITY DATA

Mix composition (by weight) c:b:w:	Maximum compressive strengths* (N/mm²) and strain (%)				Secant Modulus* (N/mm²) at 28 days	
	7 days	14 days	21 days	28 days	At 1% strain	At max. strain
25:30:100	0.23 (3.0)	0.27 (2.0)	0.41 (1.0)	0.48 (1.2)	47.3	38.1
25:30:70	0.68 (2.3)	0.95 (1.9)	1.16 (2.6)	1.38 (1.5)	127.3	91.9
25:30:52	1.17 (2.6)	1.48 (2.0)	1.88 (1.9)	2.07 (1.7)	180.6	118.4

*1N/mm² ≡ 145 psi approximately

structure continuity were provided as follows:

(*i*) After completion of the slab-jacking, water and grout injection tests were conducted on the lower contact of the B1 and concrete, through 20 drill holes angled up from ASDGA (Fig. 4). The majority of the holes were completely water-tight and only the lateral monoliths (5 and 22) had any grout consumptions (58 and 250 litres of "A/2 mix" respectively). No open water paths thus existed along this contact. Furthermore, piezometers installed in ASDGA after the slab-jacking recorded zero pressure under full reservoir conditions, and so confirmed the efficiency of the front seal.

(*ii*) At high reservoir, there was a lowering of pressure at the heel of the headworks on both sides of up to 1.8m. However, this also can be attributed to consolidation grouting of the rock done at the same time as the slab-jacking.

(*iii*) Excavation was conducted during the next low reservoir period of certain of the thin strips forming the upstream articulation and which had proved impossible to accurately slab-jack. Grout was found in intimate contact with the underside of the slab, and no void between B1 and grout could be found.

Summary and conclusions

An important feature of the auxiliary spillway remedial works conducted at Tarbela Dam, Pakistan, in the 1979-80 season, was the slab-jacking of the upstream protective apron. Articulated concrete panels afforded protection to core material laid on the rock surface, and acting as a blanket to reduce hydraulic gradients through the rock mass containing karstic limestones. Differential settlements of the panels, however, were linked with voids forming under the apron and so had threatened the integrity of their interlocking waterstops.

Injection of 375m^3 of stable cement-bentonite mixes, into 1 302 holes drilled over the 6 130m^2 area, effectively succeeded in restoring the pristine smooth profile of the apron with individual panels being jacked by over 120mm. Details of the fluid and set mix characteristics were obtained for the various designs of grout involved.

Further evidence on the effectiveness of the operation was provided by (*i*) subsequent water and grout tests at the lower fill-structure interface, (*ii*) by possibly contributing to the reduction in uplift pressures of the spillway, and (*iii*) by a later exposition of the panel-grout interface.

Acknowledgements

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