ORIGINAL PAPER

Resolving serious seepage through karstified limestone at the Mujib Dam, Jordan

Erich Wiesner · Friedrich-Karl Ewert

Abstract The 67 m high Mujib Dam (Jordan) was built between 1999 and 2003 in Cretaceous limestone with intercalated marls in the middle of the section. Initial investigations indicated only moderate permeability, hence prior to impoundment a normal grouting program was undertaken extending to 50 m in depth. Initially, little seepage was observed when the lower section of the reservoir was filled. After a dry summer, heavy rainfall filled the reservoir overnight to its maximum water level (194 m a.s.l.). A sudden seepage began downstream of the right abutment, which reached 140 l/s by morning, increasing to up to 240 l/s during the following 2 weeks. Although a second grouting program reduced the seepage to some 3 l/s of highly mineralized water, there were concerns as to the adequacy of the Phase 1 grouting. A detailed analysis of the testing and grouting results indicated that the hydraulic pressures created by the sudden increase of water in the reservoir had washed out the natural infill which had previously sealed karstic voids.

Keywords Karstic limestone · Mujib Dam · Leakage · Grouting · Jordan

Abbreviations

GG	Grouting gallery
OA	Open air
U/S	Upstream
D/S	Downstream
P, S, T and Q holes	Primary, secondary, tertiary and quaternary holes
CH holes	Check holes
1 LU	1 l/min × m × 10 bar
WPT	Water pressure test(s)
a.s.l.	Above sea level
TDS	Total dissolved solids

E. Wiesner (✉)
Am Kappelgarten 10, 60389 Frankfurt, Germany
e-mail: erich.wiesner@gmx.de

F.-K. Ewert
Mozartstr. 5, 33014 Bad Driburg, Germany
e-mail: ewert.fk@t-online.de

Introduction

Limestone is a soluble rock and karstification in this rock is ubiquitous except in arctic regions. There are many reservoirs constructed on karstic limestone which were never, or only partly, filled, e.g., the Perdikkas Dam in Greece, the Lar Dam in Iran, and the Wala Dam in Jordan. Karstified limestone is always a major engineering geological challenge requiring an accurate understanding of both the geology and the type of karstification (Wiesner and Wilhelm 1985; Ewert 1996a).
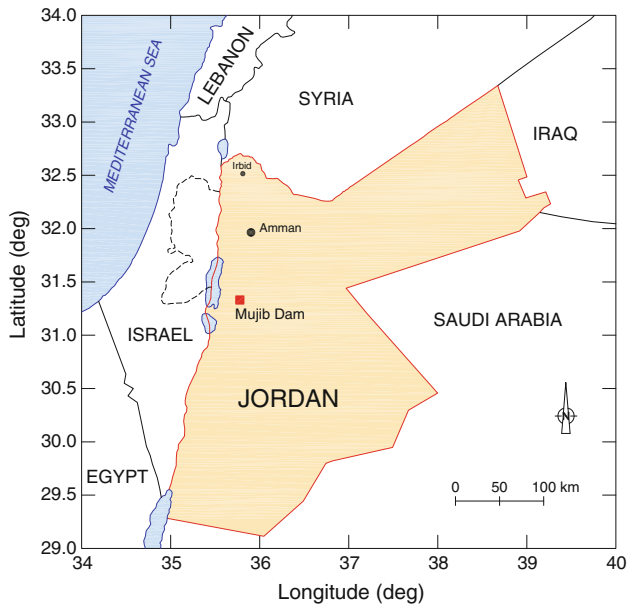
Between 1999 and 2003, a 67 m high dam was constructed in the Mujib River valley (the ancient Arnon, Fig. 1), where it is crossed by the historic King's Highway. The dam has a central concrete section with incorporated spillway and rock-fill flanks, each with an impervious core.

The dam was built in almost horizontal Middle Cretaceous limestone which has been divided into four formations:

1. The Shuayb Formation, mudstones and marls of low strength.
2. The Karak Formation, which in this paper has been sub-divided into three members, where the middle member is well bedded with many layers of marl(-stone) while the lower and particularly the upper member are more pure limestone, observable as karstified in the field (Fig. 2).
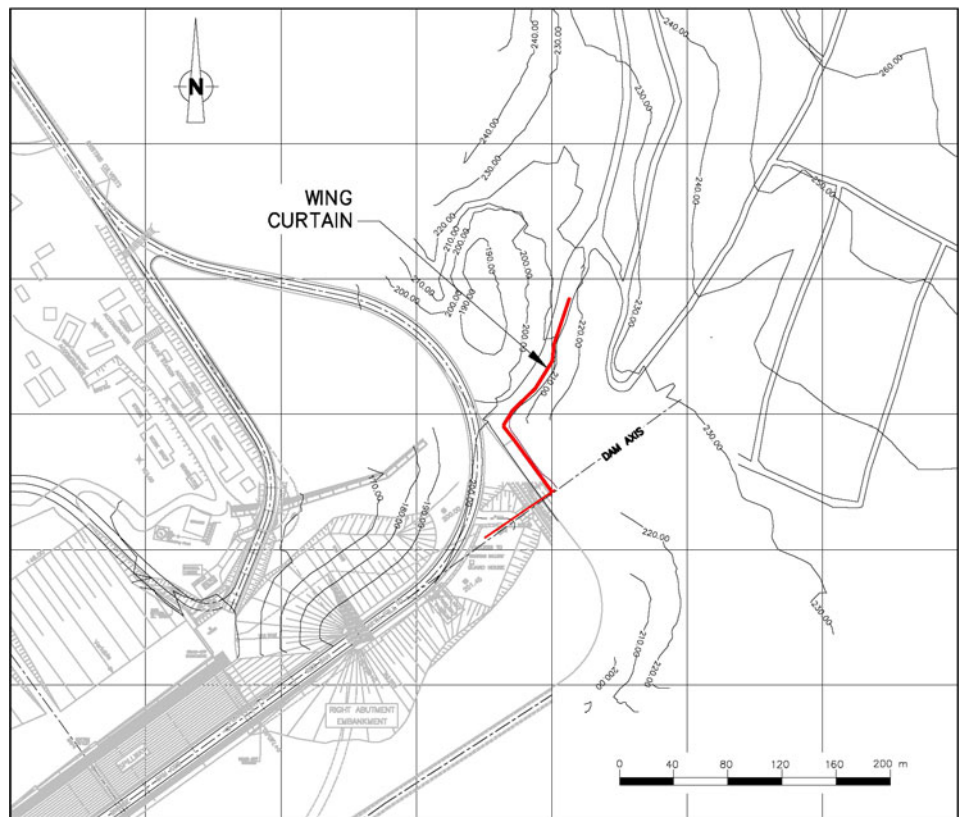
Fig. 1 Location of Mujib Dam, Jordan

Fig. 2 Karst cave, developed on a fissure on the right slope, above maximum storage level. Note difference in karstification on each side, within the same rock

Fig. 3 Layout—right part of dam and abutment. Cut-off-wing course is indicated *red*

3. The Fuhays Shales, with limestone intercalations, especially in the upper part.
4. The Upper Naur Dolomite, a massive or thickly bedded dolomite with frequent pyrite nodules and a low degree of karstification in the form of narrow karst fissures in the valley floor.

In an early phase of the project, the rock of the dam foundation appeared to be effectively impervious. However, as limestone is always prone to karstification, a grout curtain was installed (Fig. 3), extending to a maximum depth of 50 m. The permeability tests yielded low Lugeon values and the grout takes were small except at the left abutment. Substantial karstification was not recognized.

On the right abutment these Cretaceous strata are overlain by some 60 m of terrace deposits while on the left abutment there is an inactive landslide. The effect of the different soils may have caused a dissimilar development: due to its lower permeability probably less landslip materials were spilled into the karst fissures; since they remained open and accessible, they could absorb larger grout takes.

Until autumn 2004, the reservoir level only reached some 38 m above foundation level (177 m a.s.l.) due to the dry summer season. No noticeable seepage was observed. However, in a night of heavy rainfall, the level rose to 194 m a.s.l. and a sudden seepage developed, at the right toe and the downstream slope of the dam. By the morning, the flow was some 140 l/s, but during the following 2 weeks it increased to a relatively constant 240 l/s. Both the sudden initiation of the seepage and its subsequent increase suggested a twofold cause:

1. The original grout curtain did not reach far enough into the right abutment.
2. The seepage eroded the infilling from formerly plugged karst channels crossing the grout curtain.

The approach to the Phase 1 grouting, however, appeared to have been appropriate:

(a) It is generally considered that a grout curtain should extend as far into the abutment bedrock as the proposed top water level. However, when the

Fig. 4 Density of joints at Mujib damsite. *Bold line* indicates dam axis
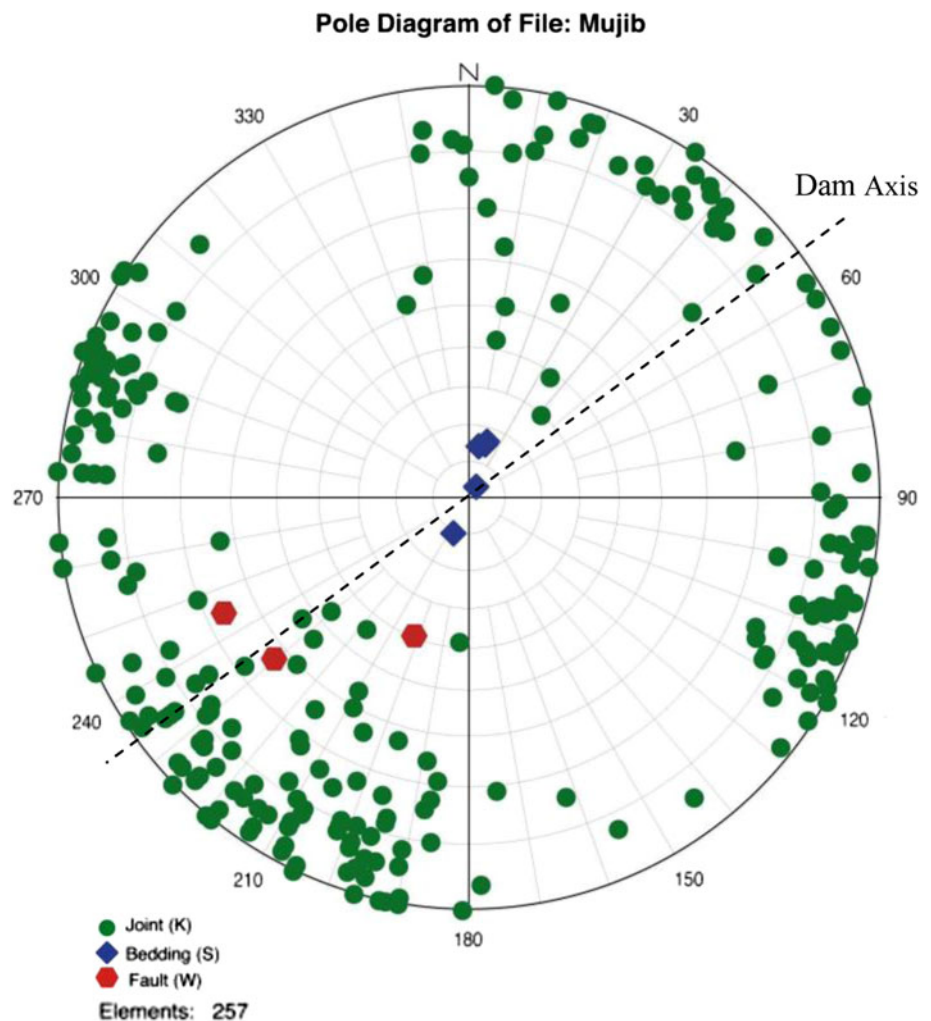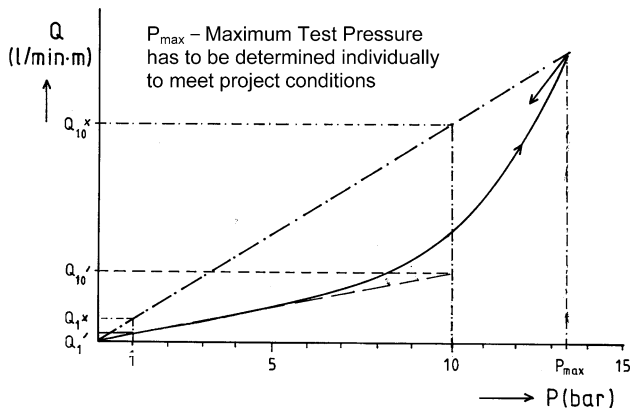
Fig. 5 Definition of WPT parameters Q_{10}' (original permeability) and $Q_{10}*$ (artificial permeability)

elevation of the groundwater table is unknown, the length of the curtain must be estimated, bearing in mind the significance of being too long (costly) or too short (risky). The conditions indicated that a shorter curtain ought to be preferred, with an additional section added if the impounded reservoir proved the first curtain insufficient.

(b) Clearly, if karst channels filled with debris existed within the area of the Phase 1 grout curtain, they would not have been fully grouted. However, the higher pressure of the fully impounded reservoir was capable of removing the erodible infill. Had the

presence of such filled karst channels been appreciated, it would clearly have been expedient to postpone the grouting work until the fillings had been removed such that the channels could be properly grouted.

A second phase of grouting was undertaken which reduced the D/S yield to some 3 l/s. As this water is more of a brine (TDS D/S right abutment max. 2782 ppm, reservoir max. 662 ppm), it is considered to be groundwater rather than seepage from the reservoir.

The paper discusses the groutability of karst channels. While the natural infillings made the passages effectively impervious, when the water pressure rose during the sudden filling of the reservoir, the tolerable hydraulic gradient of these saturated materials was exceeded. No turbid water was observed surging from the newly developed "springs". This suggests the material flushed from the karstic channels was of a sandy nature, probably originally moved into the passageways by percolating groundwater and re-deposited in an intermediate sink.

Phase 1 grouting

Consolidation grouting

Typically, consolidation grouting is undertaken to seal joints and artificial fractures created during excavation in
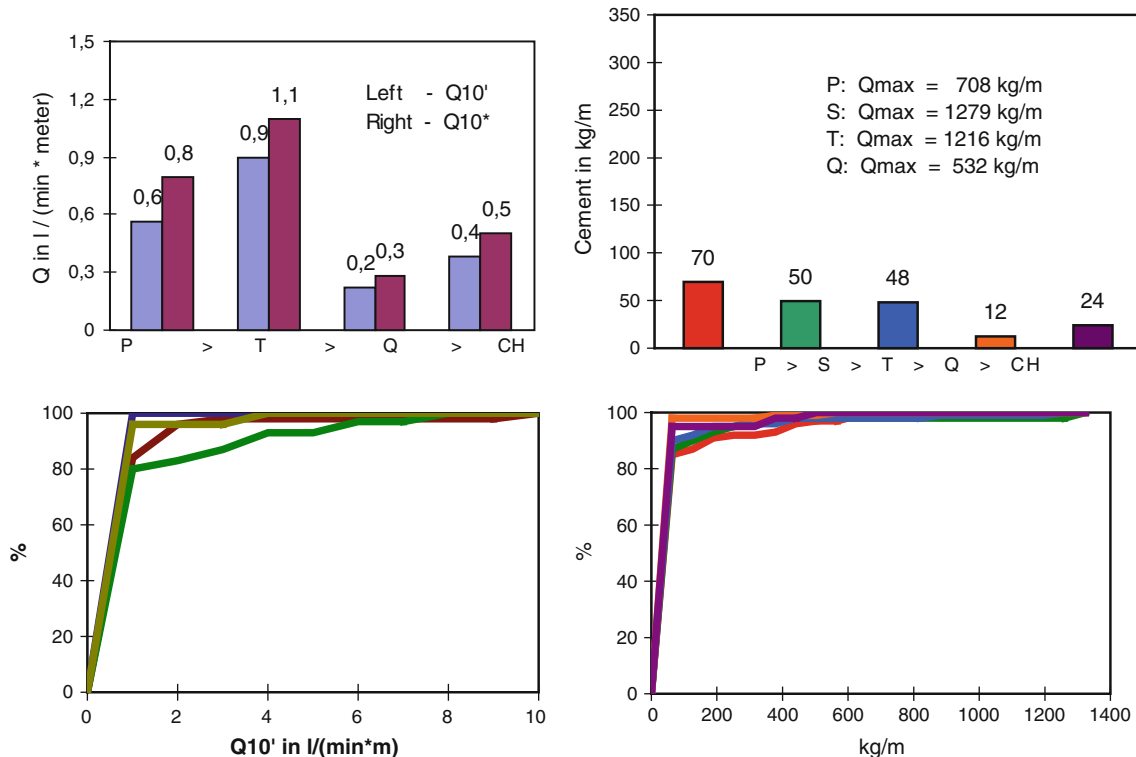
Fig. 6 Mean values and relative frequency distribution of WPT-results (*left*) and grout takes (*right*) obtained in Section GG of Phase-1
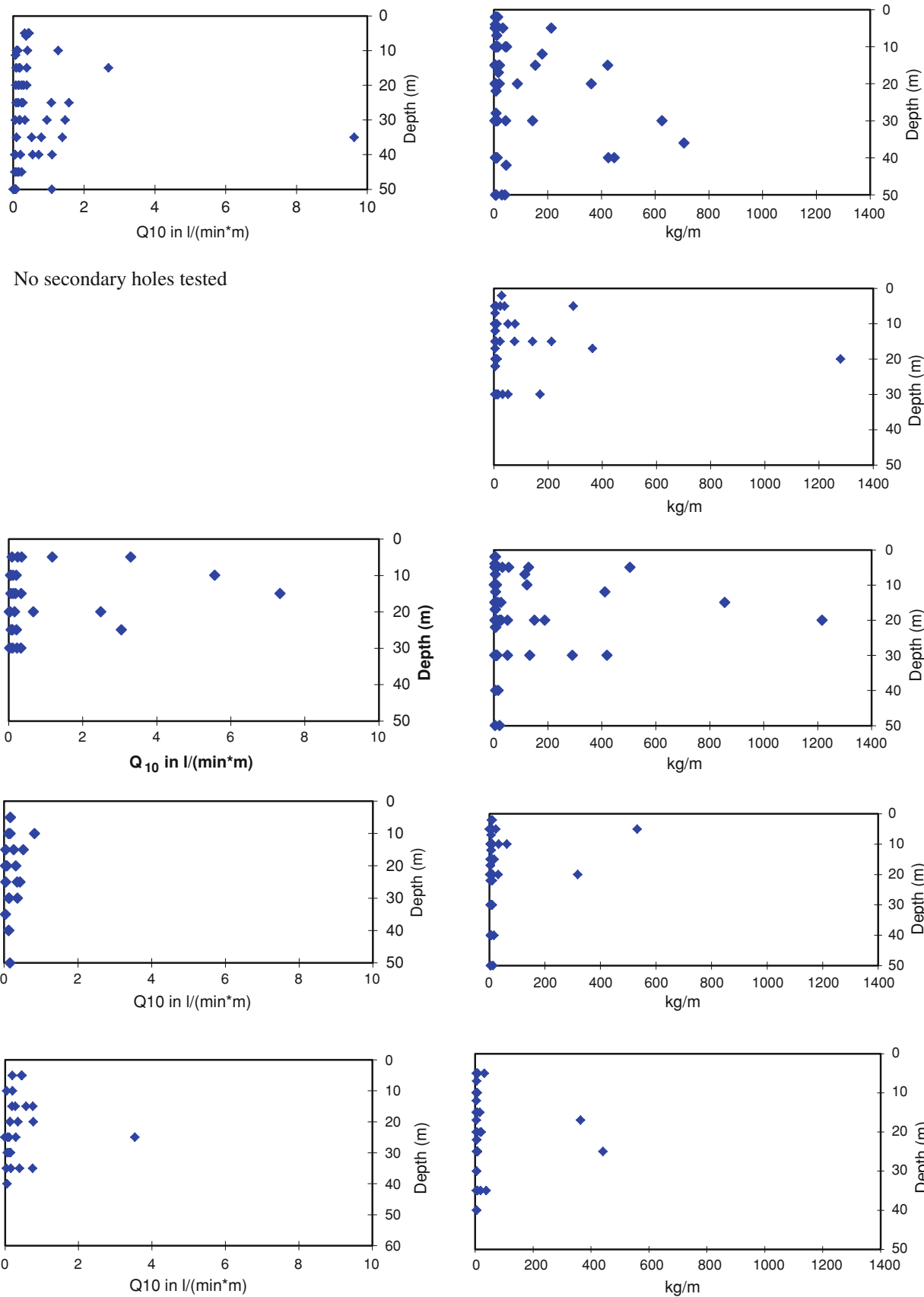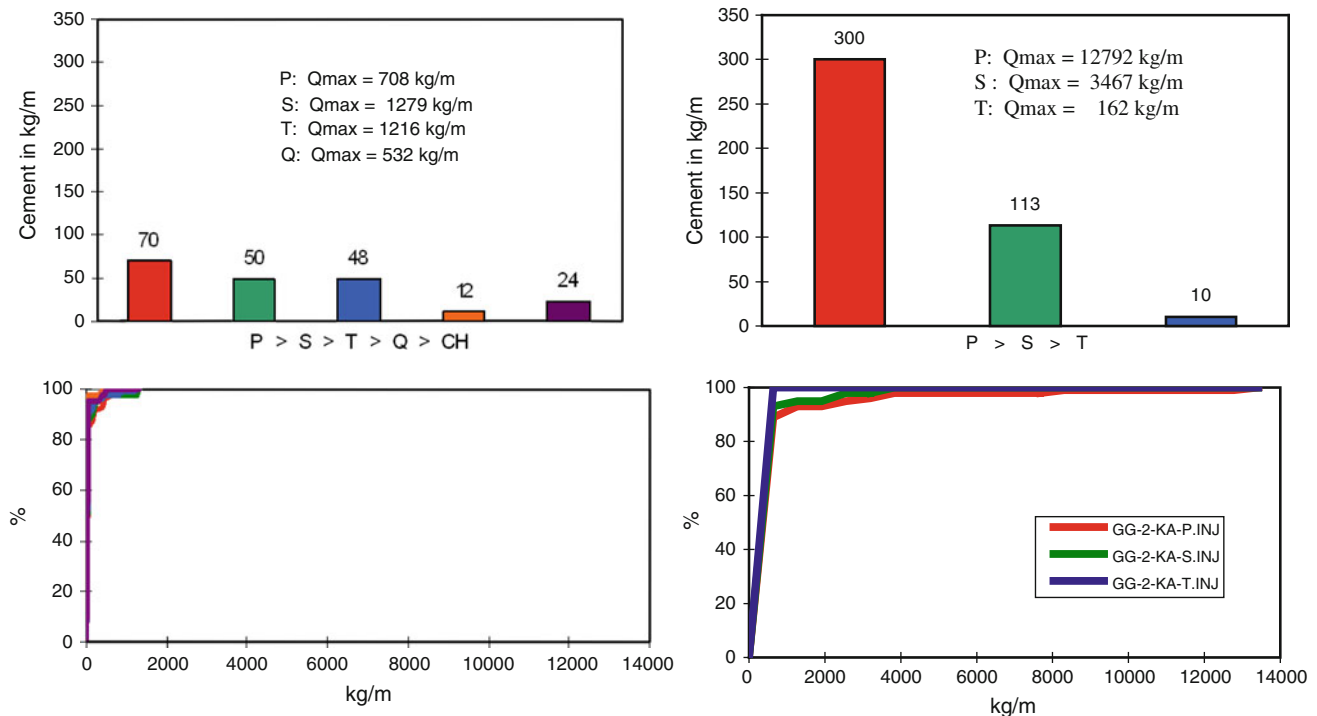
No secondary holes tested

Fig. 7 WPT results (*left*) and grout takes (*right*) obtained in series P, S, T, Q and CH (from top to bottom), Section GG of Phase-1

Fig. 8 Mean values and frequency distribution of grout takes obtained in Section GG of Phase-1 (*left*) and Phase-2 (*right*)

the near-surface area. This was carried out all along the dam foundation prior to the placement of the embankment. In 21 successive sections, two or three lines of vertical grout holes were drilled from the surface of the rock at each side of the grouting gallery with a spacing of 3 m. The lowermost section is located at 160 m a.s.l. and the uppermost at 217 m a.s.l. Three set of holes were undertaken: the Primary (P-) and Secondary (S-) holes extended for 5 m in depth and the Tertiary (T-) holes for 3 m.

About 86 % of all the holes had grout takes of <20 kg/m and some 12 % of the holes had grout takes between 20 and 100 kg/m. However, two holes in sections at El. 172 and 217 m a.s.l., took 1883 and 860 kg/m, respectively, confirming the irregularity of voids in karstic limestone. The very large grout takes revealed for the first time that karstic voids existed below the maximum storage level. As the final pressures reached only 3 bar it was considered that the karstic voids had been filled without hydrofracturing.

Contact grouting

Contact grouting has to seal open gaps which possibly remained between rock and concrete. The holes were placed along two lines in the U/S and D/S walls of the gallery and reached 1.5 m below the contact into the foundation. On the upstream side, 46 holes were drilled at effectively 1.5 m spacings, while on the downstream side, only 33 holes were undertaken, sometimes at up to 3 m

intervals. The P and S holes were 5 m in depth and the T holes 3 m. The final grouting pressure reached 3 bar.

The main results were as follows:

(a) The average grout takes were 23.5 kg/m for the U/S holes compared with 29.9 kg/m for the D/S holes.

(b) The maximum grout takes reached 437.9 kg/m for the U/S holes and 446.9 kg/m for the D/S holes.

(c) The minimum grout takes were 2 kg/m for the U/S holes and 1.5 kg/m for the D/S holes. This is probably due to the fact that in Phase 1 the filling of the hole was never deducted, while in Phase 2 the given quantities are net quantities, hole fills subtracted.

(d) 80.5 % of the U/S holes and 83.3 % of the D/S holes took <20 kg/m.

(e) 6.5 % of the U/S holes and 12.9 % of the D/S holes took between 50 and 200 kg/m.

(f) 2.2 % of the U/S holes and 3.2 % of the D/S holes absorbed >200 kg/m, i.e., 446.9 and 437.9 kg/m.

As these results were consistent with what would be expected for slightly to moderately karstified limestone, the measures were considered to have been effective.

Curtain grouting

Curtain grouting was undertaken through inclined boreholes, as mapping revealed a predominance of near vertical joints (Fig. 4). Holes were drilled from the
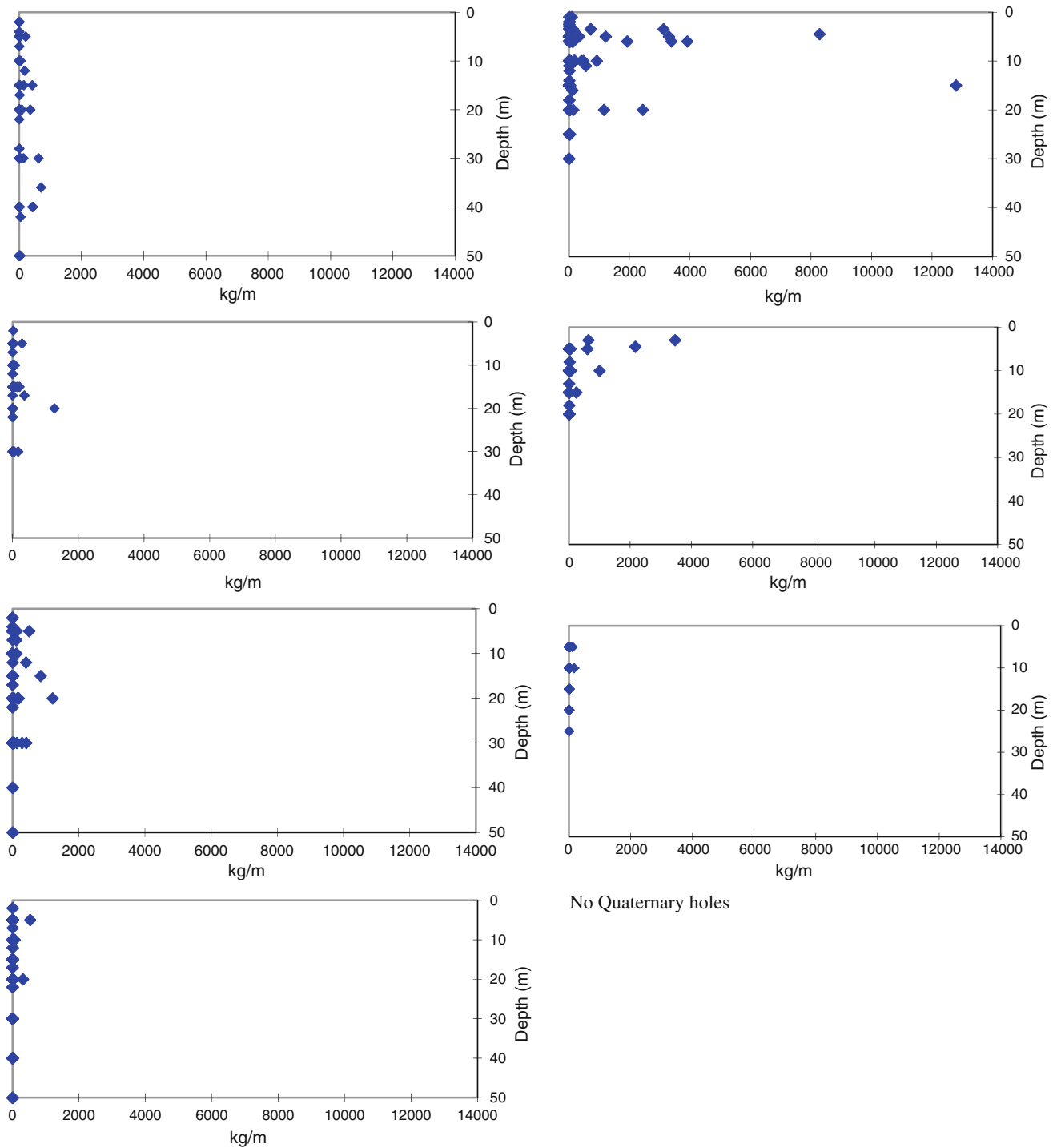
Fig. 9 Depth of grout takes in series P, S, T and Q (from *top* to *bottom*), Section GG, Phase-1 (*left*) and Phase-2 (*right*)

grouting gallery to seal the foundation of the dam (GG) and from the adjacent ground surface to extend the grout curtain into the right abutment (OA and Fan). Holes were placed applying the split-spacing-method in accordance to the findings and with the maximum depth of 50 m.

The grouting work has been carried out in four successive series: P, S, T, and Q; in some places check holes (CH) were also executed. The first stages began between 2 and 5 m where grouting pressures of between 4 and 8 bar were applied. The holes ended between 30 and 50 m and the final grouting pressures ranged from 12 to 15 bar.

**Fig. 10** Linear karst channel in NX core, Wing Section, right abutment

The number of holes and number of grout hole stages are as follows:

- P—11 holes, 75 stages.
- S—12 holes, 70 stages.
- T—22 holes, 120 stages.
- Q—24 holes, 122 stages.
- CH—8 holes, 42 stages.

The maximum grout takes reached 708 kg/m in P holes, 1,279 kg/m in S holes, 1,216 kg/m in T holes, and 532 kg/m in Q holes. About 80 % of the grout stages were within (almost) impervious rock, while only 20 % encountered permeable and groutable voids/fissures. This difference is typical for karstic limestone. Fan grouting into the right abutment did not render noteworthy peculiarities.

Water Pressure Tests (WPT) were carried out in the P, T, Q and CH holes, but not in the S holes. In general, before reaching the reference pressure of the Lugeon criterion of 10 bar, open joints may be dilated or latent planes may be hydrofractured; it is not known whether the Lugeon value reflects the original permeability or is affected by such dilation or fracturing. In order to distinguish between the original and the possibly enlarged permeability, the WPT results are given as Q_{10}' and $Q_{10}*$ values (Fig. 5), where Q_{10}' gives the original permeability and $Q_{10}*$ takes into account dilation or erosion of infillings or hydrofracturing (Ewert 1996b). The statistical evaluation of the results shown in Figs. 6 and 7 indicates:

(a)   About 80 % of the rock was originally almost tight— $Q_{WPT} \leq 1$ LU.
(b)   Permeable and groutable voids existed to a depth of approximately 30 m.
(c)   After the T-series, only one significant test section of permeability was found, indicating a good seal had been achieved.
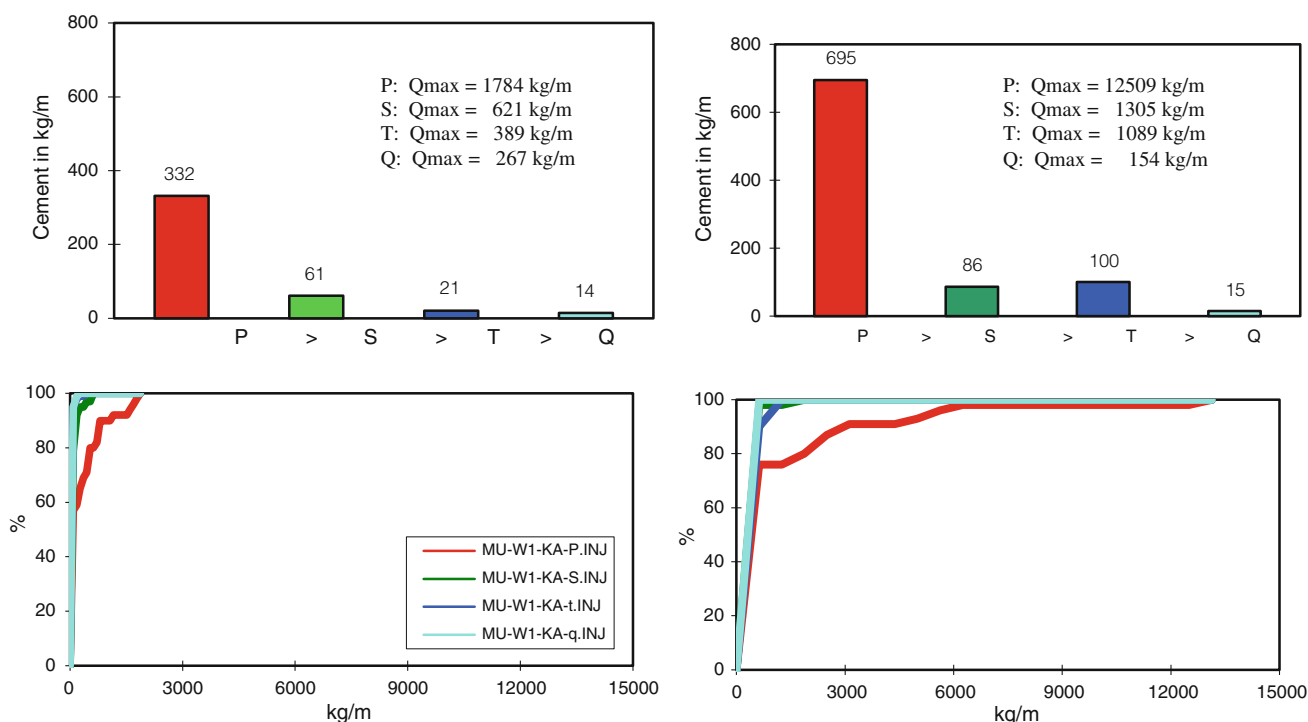
Fig. 11 Mean values and frequency distribution of grout takes in Sections W1 (*left*) and W2 (*right*), Phase-2
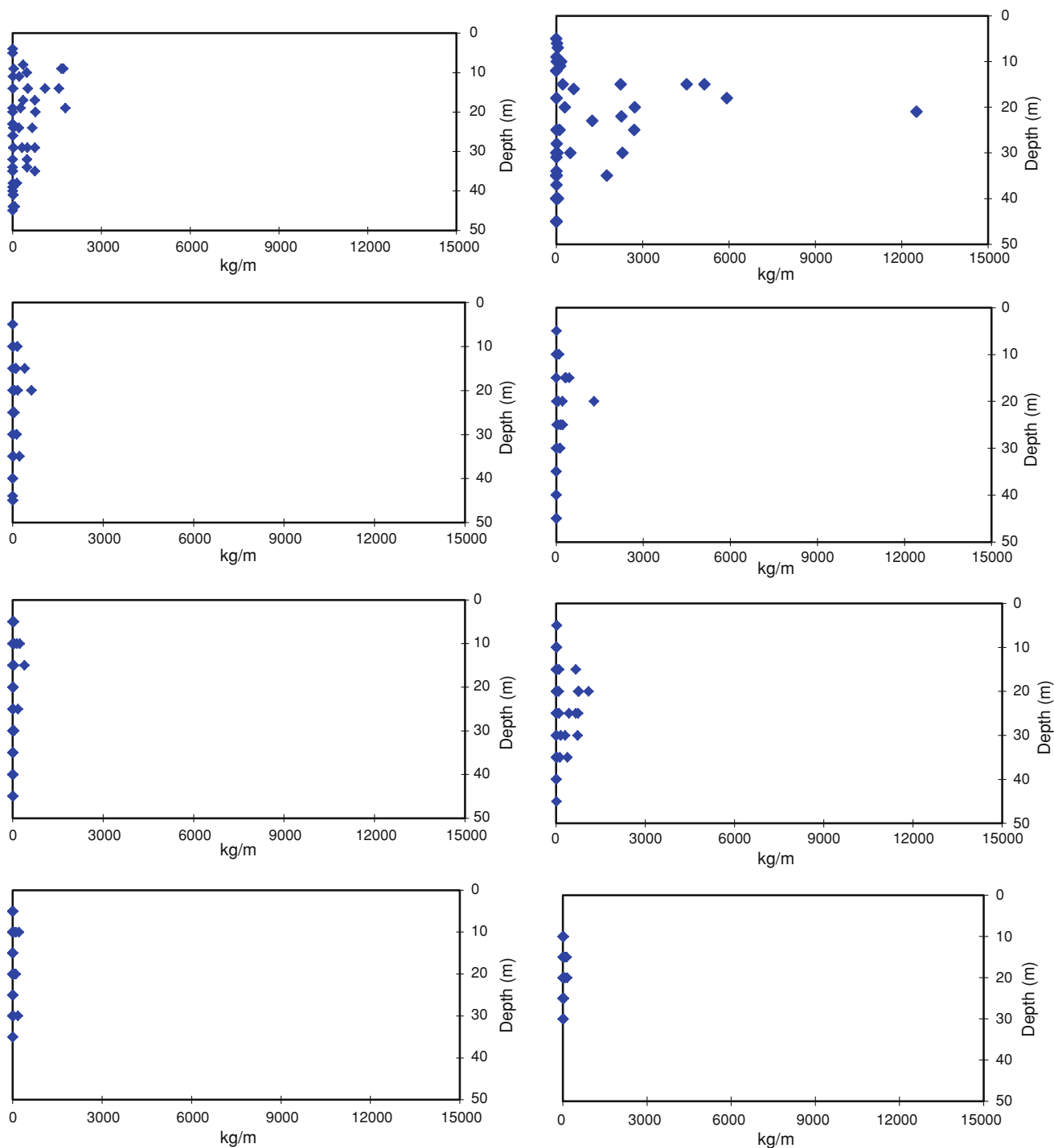
Fig. 12 Depth of grout takes in Series P, S, T and Q (from *top* to *bottom*), Sections W1 (*left*) and W2 (*right*), Phase-2

(d) The differences between the Q_{10}' and $Q_{10}*$ values are small, which indicates low deformability and erodibility of the rock mass under applied pressures.

(e) No clear correspondence between permeability and grout take was observed. To some extent, this may be related to the high pressures applied during the Phase 1 grouting which may have resulted in some hydrofracturing. It should be pointed out that during Phase 2, when more precise observations were at hand, high quantities of grout were always observed at the

beginning of each grouting process without pressure while the final pressures did not result in a significantly greater grout take.

Phase 2 grouting

Grouting beneath the dam

Phase 2 grouting was undertaken after the seepage occurred in 2004. Initially, it was carried out in areas where excessive grout quantities had been recorded during Phase 1. However, it soon became apparent that heavy grout takes were occurring at depths where originally very little grout had been placed, particularly below the reservoir level at the time of the onset of seepage. In view of this, the second grouting phase was undertaken progressively in three series: 27 P, 12 S, and 28 T holes. The holes were inclined as in Phase 1 and the grout curtain was installed in steps down to 30 m in series P (123 stages) and S (43 stages), and to 50 m in series T (80 stages). The results of the statistical evaluation are presented graphically in Figs. 8 and 9. The top stages were grouted with pressures of between 3.3 and 6.3 bar and the lower stages with pressures of between 12 and 14.2 bar, with almost all of the grout injected at the lowest pressures.

The maximum grout takes reached 12,792 kg/m in the P holes, 3,467 kg/m in the S holes, and 162 kg/m in the T holes. The Q and CH holes were not drilled. In Phase 2, all the quantities correspond to the real grout takes of the rock itself as the fill of the hole was always deducted.

Grouting of extension section, right abutment

This part of the curtain was drilled and grouted in an open air section until the fan of Phase 1, then in a somewhat odd configuration along the cut slope in the D/S direction and further in a right bend uphill (see Fig. 3). This configuration was necessary in view of the topography. A straight extension would have required the excavation of a gallery, which could not have been done in time, or to drill many unnecessary metres in terrace material which would have been extremely difficult to stabilize.

This section encountered voids which had either originally been open or had been cleared of debris during the sudden filling of the reservoir. Figure 10 shows a linear karst channel. Such a feature can only be grouted when directly intercepted. Both wing sections had typical grout takes, as shown in Figs. 11, 12:

(a) The majority of the grout holes encountered tight rock; about 60 % in Wing-Section 1 and 75 % in Wing-Section 2.
(b) The groutable voids were relatively large, but as they had generally been filled during the P-series, the take in subsequent series was relatively small.
(c) Especially in Wing-Section 2, however, the rock had longer and wider karstic joints, many of which had not been grouted during the P-series. As a consequence they took significant quantities in the S-series and even in the tertiary holes.
(d) The depth distribution indicates that the open voids were restricted to the upper 35 m below ground surface (165–180 m a.s.l.).
(e) The significant grout takes injected at low pressure had clearly entered open cavities rather any caused by the fracturing of discontinuities.

Table 1 shows the various stages of grouting. It can be seen that despite the reduced spacing of the holes, some high takes still occurred in the later series, probably due to features such as that seen in Fig. 2.

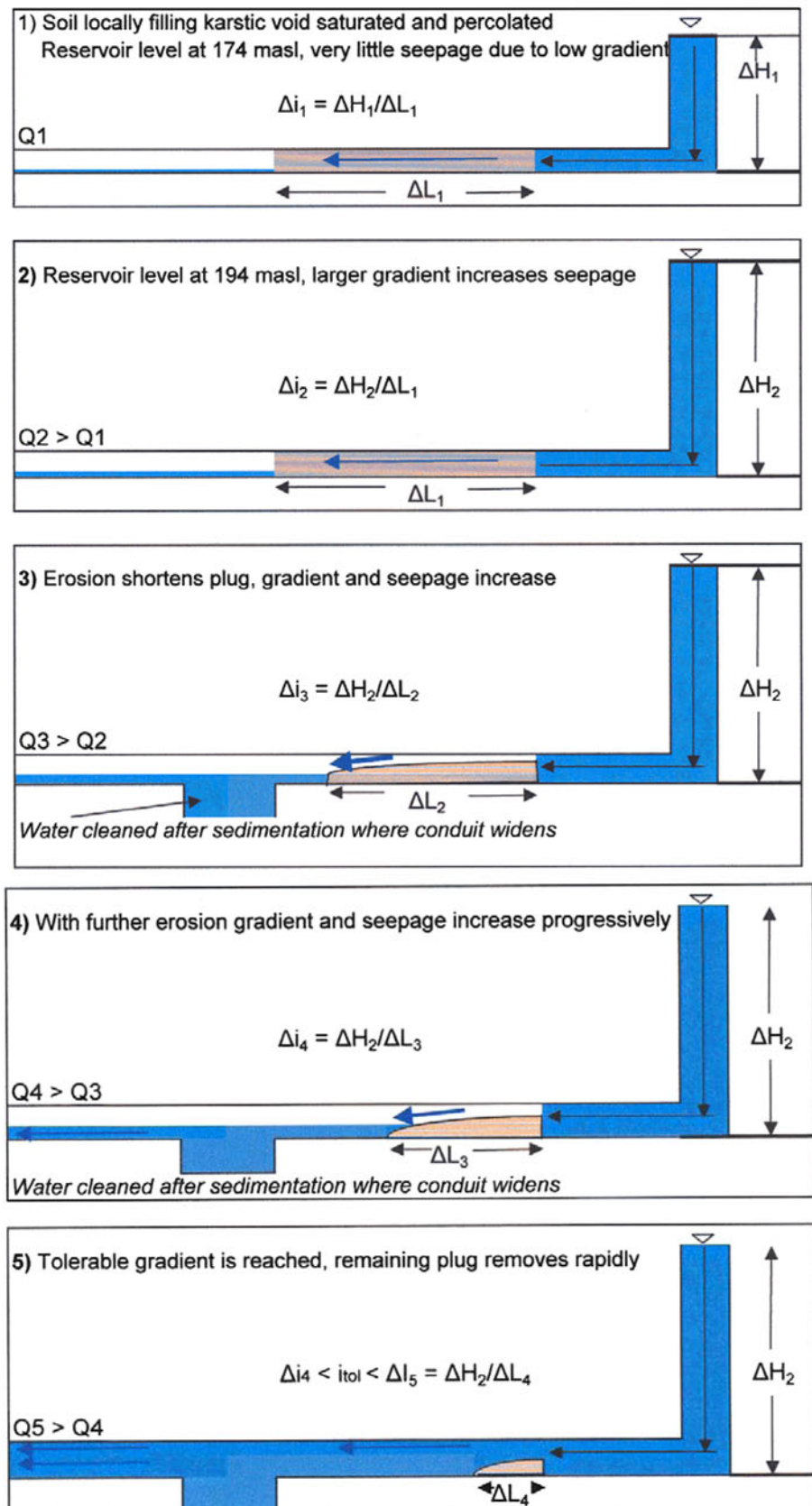## Comparison of Phases 1 and 2 grouting from the gallery

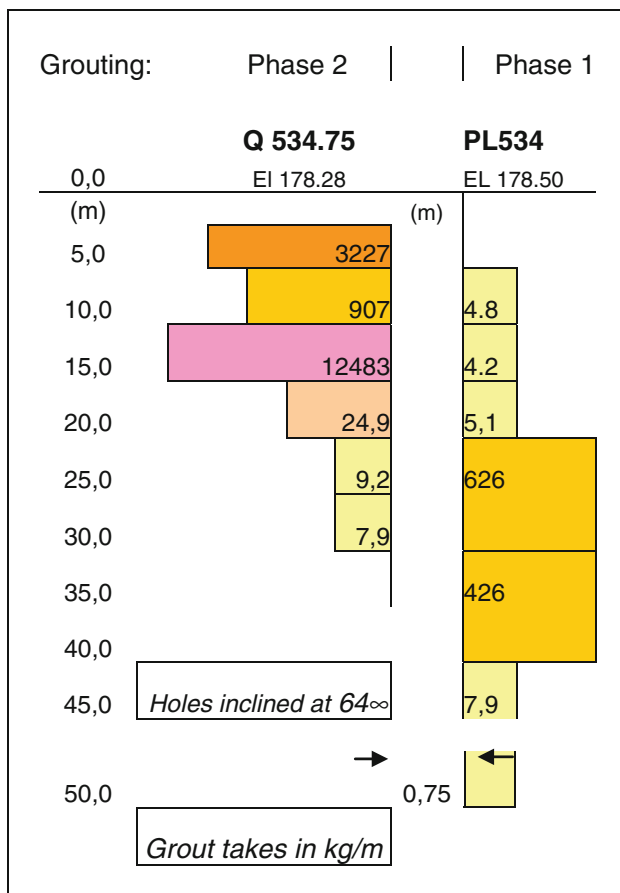The results of GG are compared in Figs. 8 and 9.

(a) The high takes during Phase 2 are below 177 m a.s.l., which was already under reservoir pressure before the accident.

Table 1 Wing Cut-off-Curtain, development of average grout takes from series to series, calculated in kg of solids

	kg/m	Reduction compared to previous series						
Spacing 24 m	689.3	100 %						
Spacing 12 m	433.0	62.8 %	100 %					
Spacing 6 m	98.1	14.2 %	22.7 %	100 %				
Spacing 3 m	79.6	11.5 %	18.4 %	81.1 %	100 %			
Spacing 1.5 m	43.6	6.3 %	10.1 %	44.4 %	54.8 %	100 %		
Spacing 0.75 m	52.3	7.6 %	12.1 %	53.3 %	65.7 %	120.0 %	100 %	
Spacing 0.3-0.4 m	27.8	4.0 %	6.4 %	28.3 %	34.9 %	63.8 %	53.2 %	100 %
Check holes	13.0	1.9 %	3.0 %	13.3 %	16.3 %	29.8 %	24.9 %	46.8 %

Fig. 13 Erosion of infillings from plugged karst channels at higher hydraulic gradient (schematic illustration)

1) Soil locally filling karstic void saturated and percolated
   Reservoir level at 174 masl, very little seepage due to low gradient

   $\Delta i_1 = \Delta H_1 / \Delta L_1$

   Q1

   ΔL_1

   ΔH_1

2) Reservoir level at 194 masl, larger gradient increases seepage

   $\Delta i_2 = \Delta H_2 / \Delta L_1$

   Q2 > Q1

   ΔL_1

   ΔH_2

3) Erosion shortens plug, gradient and seepage increase

   $\Delta i_3 = \Delta H_2 / \Delta L_2$

   Q3 > Q2

   ΔL_2

   Water cleaned after sedimentation where conduit widens

   ΔH_2

4) With further erosion gradient and seepage increase progressively

   $\Delta i_4 = \Delta H_2 / \Delta L_3$

   Q4 > Q3

   ΔL_3

   Water cleaned after sedimentation where conduit widens

   ΔH_2

5) Tolerable gradient is reached, remaining plug removes rapidly

   $\Delta i_4 < i_{tol} < \Delta I_5 = \Delta H_2 / \Delta L_4$

   Q5 > Q4

   ΔL_4

   ΔH_2

Fig. 14 Originally plugged channels absorbed large grout quantities in adjacent hole (75 cm distance) after erosion of infillings

(b) The large individual grout takes indicate sizable voids, extending to depths of approx. 25 m.
(c) The average grout takes in Series P quadrupled—from 69.7 kg/m in Phase 1 to 300.3 kg/m in Phase 2.
(d) As a result of the progressive filling of the voids, Phase 2 needed only three series:

Phase 1:

69.7 kg/m = 100 % in Series P;
49.6 kg/m = 71.2 % in Series S;
48.3 kg/m = 69.3 % in Series T;
12.4 kg/m = 17.8 % in Series Q;
24.1 kg/m = 34.6 % in Series CH.

Phase 2:

300.3 kg/m = 100 % in Series P;
113.2 kg/m = 37.7 % in Series S; and
10.1 = 3.4 % in series T.

Phase 1 encountered 20 % of generally narrow groutable voids, consistent with the WPT results for the Q and CH holes. Phase 2 identified only 10 % of groutable voids, but they took large quantities of material as a result of the

removal of infilling when the water level in the reservoir had created higher hydrostatic pressures (Fig. 13).

Figure 14 compares the grout takes in PL 534 and Q 534.75, within 0.75 m of each other, for the two phases of grouting. It is possible that the large grout takes in Q 534.75 in Phase 2 are the consequence of the erosion of infill from the intercepted karst channels (Fig. 13). Alternatively, a karstic fissure may have been completely filled with sediment, but the apparent or real cohesion of this material may have been strong enough to withstand the initial low water pressures. As the pressures increased, however, the material was gradually pushed towards the open end until the entire plug was removed (retrograde erosion). Depending on the length of the infilled section, both processes may be relatively quick or take place over many months or even longer, especially if the joint had been infilled with gypsum, as observed at Mujib. In contrast to the Caspe Dam in Spain (Manquero Piqueras et al. 2012), however, at Mujib Dam gypsum was of minor importance; the gypsum filled joints occurring only within the uppermost meters of the reservoir where the water pressure gradient and impounding time are lowest.

Adequacy of grouting pressure

The possibility that Phase 1 grouting was inadequate as a consequence of low grouting pressures was considered (Ewert 2005). Grouting of open voids begins at low pressure. After some time the pressure starts rising since the void is getting filled. By contrast, if voids are lacking or quite narrow the grouting pressure rises from the beginning. Supposed latent discontinuities intersect the hole, hydrofracturing occurs as soon as the grouting pressure—if increased that much—exceeds the tensile strength of the latent plane. The fissure pressured open is then filled with grout. Figures 15 and 16 suggest that refilling of hydrofractured planes did not happen: the large grout takes in Phase 1 took place at low to moderate pressures (3–13 bar), and in Phase 2 at even lower pressures (3–6 bar).

Lessons learned

Karstification produces different types of karsticity, which here applies to both abutments.The originally same rock retained under the left slope largely its open fissures since less debris was removed out of a better cemented landslip. Prior to construction it is impossible to determine whether planes are still closed or filled with ungroutable debris. Seismicity is unable to unveil such differences. Drilling cores and water pressure tests do not reveal the real situation either: Flush water removes possible infillings; any
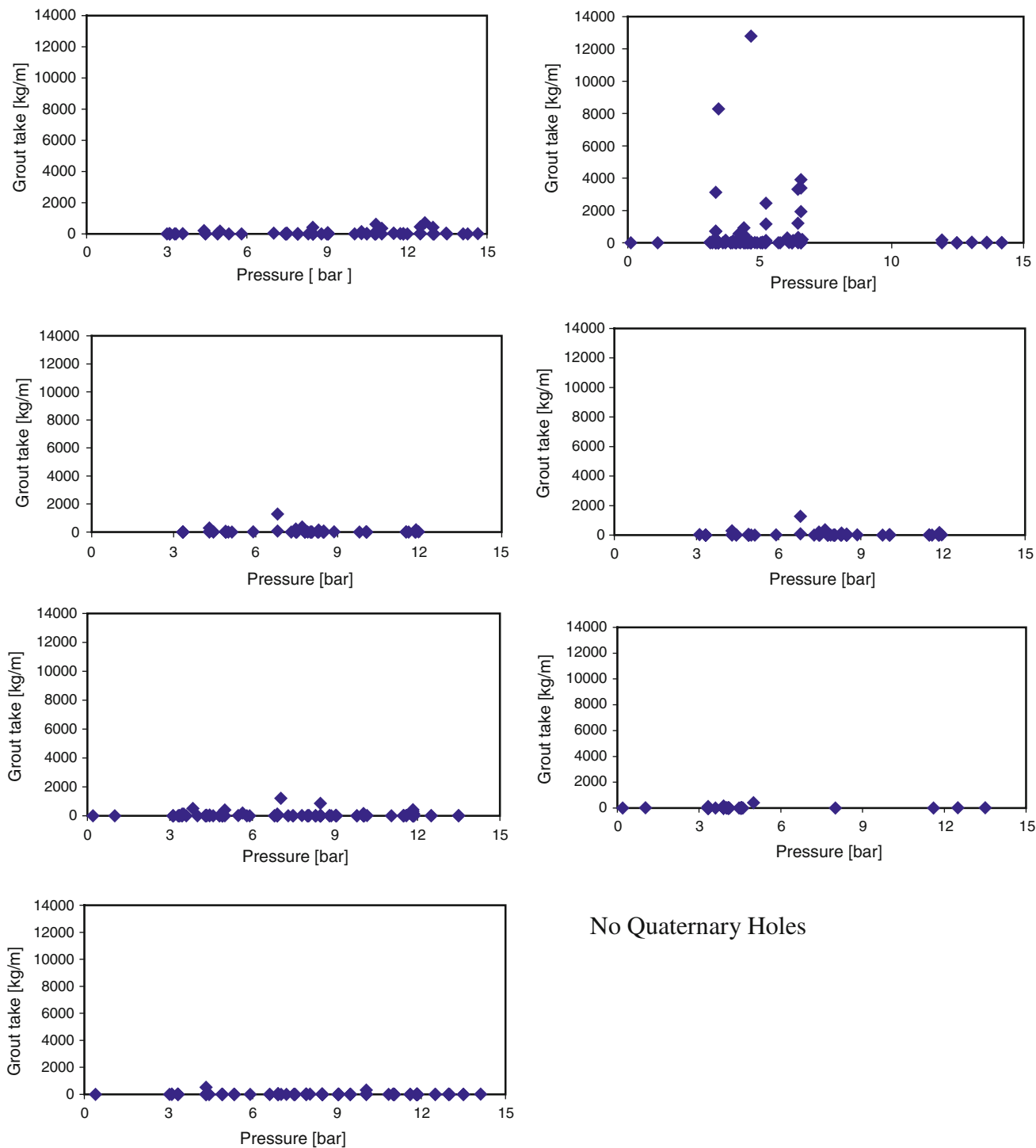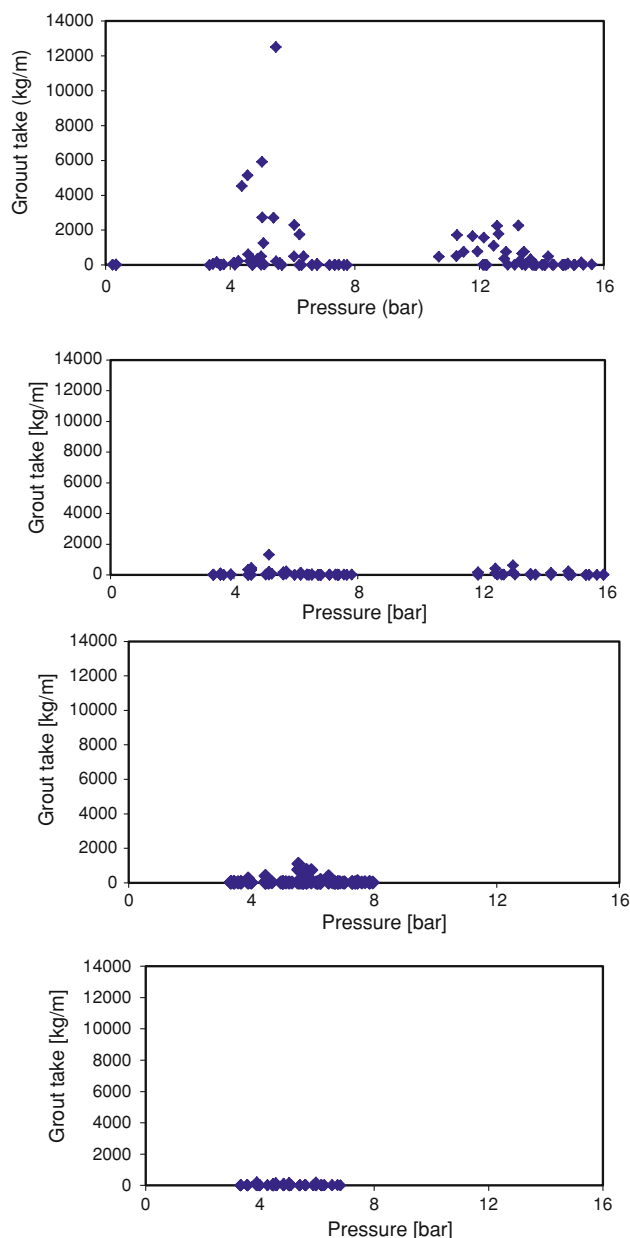
Fig. 15 Relationship between grouting pressure and grout takes in Series P, S, T and Q (from top to bottom) in Section GG, Phases 1 (*left*) and 2 (*right*)

next pressure step causes hydraulic fracturing of latent planes, or erodes debris out of a fissure. Thus, as different processes yield similar results, the problem of possibly existing fissures clogged with debris is difficult to recognize beforehand. This excludes the option of treating them

accordingly already before impounding. Even one or two fissures not detected during excavation or grouting could later cause a significant water loss, hence it is essential to reserve the possibility of later remedial grouting in the case of karstification by including a grouting gallery in the

Fig. 16 Relationship between grouting pressure and grout takes obtained in Series P–S–T–Q (from *top* to *bottom*), Sections W1 and W2 together, Phase 2

design. Many of the problems only became evident during the Phase 2 works, hence in a case similar to Mujib Dam, the presence of an experienced engineering geologist at the site is advocated, at least for some periods if not continuously.

"Dirty" karst, i.e., with filled fissures, is not a uniform material. The lower the cohesion (or the cementation) of the fill the more likely that it will be washed out, and in a shorter time. Again, the longer the filled section the longer time is needed to wash the material out. In view of this, it cannot be assumed that the Phase 2 grouting at Mujib Dam

is necessarily the last. Indeed, it is possible that the breakthrough of a filled karst fissure could occur even after some 50 years.

Final considerations

The possibility of karstification makes limestone a hazardous foundation. During the construction of the Mujib Dam, subsequent consolidation, and contact grouting, relatively few karstic features were encountered and successfully filled; the early impoundment producing very little seepage loss. However, following heavy rainfall the stronger hydraulic gradient caused by the full reservoir initiated the erosion of old infill material in some karstic channels. As a consequence, these channels became permeable, and significant seepage occurred in sections already grouted during Phase 1 when no seepage had occurred for some 6 months under low reservoir pressure. When a second grouting phase was carried out to fill these newly opened voids, it was found that they extended to a depth 35 m below ground surface (El 165–180 m a.s.l) in the extension and some to even lower depths below the right side of the dam. Fortunately, Phase 2 was successful but not predictable with certainty.

References

Ewert FK (1996a) Considerations on grouting of karstic limestone at dam sites. Dam Eng 7:3–32

Ewert F-K (2005) The hydrofracturing behaviour of latent discontinuities in rock and its consequences for the successful and economic execution of grouting work. Dam Eng XVI(I):4–65

Ewert F-K (1996b) Evaluation and Interpretation of Water Pressure Tests, Paper 9. In: Proceedings of the Conference of "Grouting in the Ground", 1992 at The Institution of Civil Engineers, London, p 19

Manquero Piqueras JA, Sanz Pérez E, Menéndez-Pidal I (2012) Water seepage beneath dams on soluble evaporite deposits: a laboratory and field study (Caspe Dam, Spain). Bull Eng Geol Environ 71:201–213

Wiesner E, Wilhelm R (1985) Grouting as underground-sealing of deeply karstified rock at a damsite (Guatemala). 6. Nationales Felsmechanik Symposium, Geotechnik Sonderheft 1985, pp 155-163