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# Rock Grouting at Dam Sites

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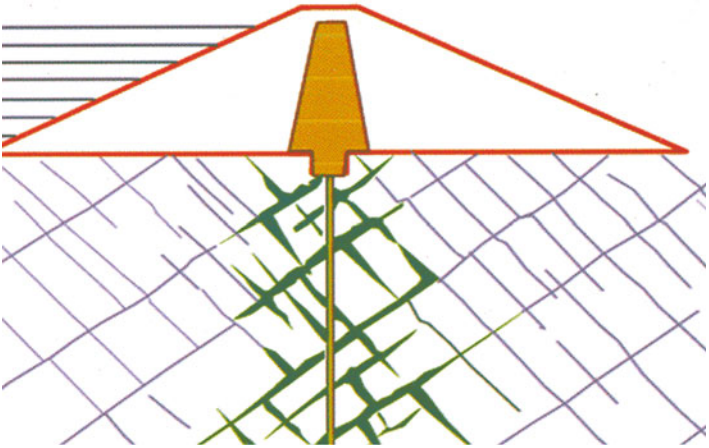

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# Rock Grouting at Dam Sites

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# Task

Grouting is the usual method to seal a permeable dam foundation. In spite of alternative technologies—diaphragm walls, for instance—many sites will remain whose conditions still favour conventional grouting. Dam foundation grouting today is mostly rock grouting. The decision to grout is usually based on the results of water pressure tests (WPT), and it is assumed that the permeability can be reduced by means of grouting to the desired degree. The possibility to press grout suspension into the rock was supposed to confirm both the need and the success of a grouting program. This concept resulted in many inconsistencies. It is prudent to establish a new one: WPT-results should not be the only basis for the decision to grout, the hydrogeological setting of the whole foundation should be also considered. Rock types are not equally groutable but have individual groutabilities which only permit a specific reduction of their permeability. Grouting pressures should not be related to depth but to both the individual geological setting and the purpose of a grouting program.

# Contents

# Chapter 1
# Introduction

Grouting applied at dam sites today is usually carried out to consolidate the foundation ('*consolidation grouting*'), to seal the contact between rock and concrete ('*contact grouting*') and to tighten the rock down to a certain depth ('*curtain grouting*'). The grouting of open fissures or other natural voids is called '*penetration grouting*', the grouting of latent discontinuities to achieve a pre-stressing of the rock is called '*displacement grouting*', it is mostly applied in tunnelling. This book intentionally deals with penetration grouting for curtains.

A comprehensive literature is available on the technology of rock grouting, mainly dealing with the procedure itself. However, '*how to grout*' should be the second question, the first one should be '*whether to grout, and where, and how much*'. All this depends on the hydrogeological situation of the entire dam site and on the groutability. The permeability of the rock surrounding a borehole is often not identical with the permeability of a large rock mass and a permeable rock is not necessarily groutable. These complexities have received too little attention in the past although they determine both the under-seepage and the success of the grouting work and they are much more complicated than technological issues. Therefore, dealing with this first question means examining whether the rock at the dam site is permeable, whether this permeability determines the entire hydrogeological regime and whether the rock is groutable. Rock masses comprise a great variety of permanently changing conditions. This makes it difficult to recognize their permeability and groutability. In penetration grouting, the task is not simply to press grout into the rock but to fill actual water paths—quite often not the same. The first task is possible in most cases, the latter is not everywhere achieved, mainly because the given groutability is not duly considered and grouting is often carried out rather schematically instead.

A comparison of many projects shows that there is hardly any recognizably consistent concept in assessing the actual permeability and groutability of the rock or in sealing the foundation by means of grouting. Sometimes even a tight rock was extensively grouted while in other projects, also well performing, only little grouting was done in spite of an apparently high permeability. Thus, there are

 
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reasons to improve the situation by considering both the possible differences between permeabilities of the local rock sections tested and the permeability of the entire dam foundation as well as the individual groutabilities. Positive examples demonstrate that there obviously exist possibilities to achieve progress and also in terms of a more economical treatment. Investigation methods have now improved and a new concept of dealing with this important complex is also available. Both are helpful to avoid schematic treatments. Of course, even improved methods and concepts will never meet the degree of accuracy generally applicable in engineering, nevertheless, considerable progress is achievable which can help to avoid unnecessary measures.

Dams built a century ago did not receive a grout curtain but later, grouting has been applied almost at every dam. The installation of a grout curtain always involves considerable expense. It should only be done if necessary. The decision to install a grout curtain still depends largely on the results of water pressure tests (WPT) introduced by Lugeon. Usually a grouting program is carried out provided the quantity of the water absorbed in a borehole ($Q_{WPT}$) exceeds 1 l/min/m/10 bar, called "Lugeon-criterion" (i.e. 1 LU). For some time, inconsistent observations have been made which raise doubts as to the reliability and expediency of that criterion.

The definition of an impermeabilization criterion of general applicability—in accordance with Lugeon or others—inherently presupposes that rocks of higher permeability are groutable and can be sealed by grouting to a defined degree. The general applicability of such a criterion would strictly mean that individual geological conditions do not control the course and the success of the grouting work; all rock types should be equally groutable. Grouting would be easy to handle if that were true! Unfortunately, it is not, because all geological settings have their own specific "water routeing" and their own strength. Since these properties determine the groutability, it is quite likely that each rock has its own individual one. Studies carried out by several colleagues and Ewert confirmed the existence of such individual groutabilities. Thus, the Lugeon criterion as a compulsory measure should be replaced by individual concepts. The Lugeon criterion still continues to be the guideline for dealing with this matter in many projects. It will need some time and steady discussion to change this situation.

Rock grouting is often discussed controversially. One of the reasons for differing opinions is different personal experience. If an engineer is dealing mainly with hard rock he will not be afraid of generally applying high grouting pressure whereas his colleague, working in soft rock, tends to prefer lower pressures. However, different rock types need their own treatment; hard rock requires a high grouting pressure, and soft rock a low one. Unfortunately, grouting programs are often carried out in accordance with outdated recommendations and specifications; they are not evaluated thoroughly and critically enough. Commercial aspects sometimes also influence the decision in a grouting program and the way the work is done.

In order first to illustrate the state of the art several typical examples will be presented. They comprise grouting programs carried out still rather schematically and others where the more economical approach "appropriate grouting if required" has already been applied. In spite of such achievements, old concepts still dominate often causing unnecessary treatments. To contribute to a wider application of an improved methodology this book deals with all relevant aspects. All conclusions have been derived from actual programs. In attempting to improve the current situation, and achieve a better result at lower costs, the following aspects are to be considered:

- The permeability of the rock mass at the dam site including the hydraulic characteristics of water conducting openings and their effect on the reliability of WPT-results.
- The influence of particular geological factors on the hydrogeological regime at the site.
- The hydrofracturing behaviour and individual groutability, and the relationship between quantities of grout takes and the degree of impermeabilization.

The common approach in dealing with the foundation of dams is rather simple: if the results of water pressure tests exceed a few Lugeon-units it is decided to seal the subsoil. The authors experienced that in many cases they could have abstained from expensive treatments if all the potentially effective factors were analysed and duly considered. Unfortunately, they largely remained disregarded—as the following examples confirm which frequently occur:

- In water pressure testing large water takes do not necessarily indicate a substantial permeability of the rock—a multitude of fine fissures are able to absorb a lot of water but considerable head losses due to friction yield a small coefficient of permeability which determines the seepage losses through the underground.
- Rock masses of little strength containing fissionable planes—bedding, cleavage, metamorphism—are susceptible to hydrofracturing. The fractured planes absorb large quantities of grout even at moderate grouting pressures already.

A particular problem causes the construction of dams in limestone areas where we have to deal with karstification. The dissolution of limestone produces water conducting voids. They are irregularly distributed and their locations are unpredictable. Karstic voids appear as fissures, cavities and even caves while the rock itself is usually impermeable. The degree of karstification determines whether still narrow or already wide fissures or cavities or even huge caves prevail. Grouting of karstic limestone is difficult: cavities or caves absorb enormous amounts of cement often without success in spite of great expenses, while in case of an initial karstification an extremely close spacing of the groutholes is required to find the few small voids. There the boreholes primarily serve for detection rather than for the grouting itself. In general, grouting of karstic limestone requires a schematic treatment, however in case of favourable hydrogeological conditions such a treatment may not be warranted any more and can possibly be replaced by more economical concepts. Thus, limestone confronts us either with a technological problem—how to close large cavities—or in case of an initial karst with that question difficult to answer: is it possible to disregard the few small voids assumed, although their unknown seepage capacity may be large enough to empty the reservoir? This contribution analyses typical cases; an attempt is made to develop conclusions regarding alternative solutions.

It has been dealt with repeatedly with this very complex subject and most parts have been published already. The reason to do it again is quite simple: New data permanently obtained in practical programs and their thorough evaluation and critical discussion confirmed the main conclusions drawn in Chap. 8 and permit a continuous updating required to achieve further improvements. Thus, it is prudent to summarize the older findings again and to report on the younger projects somewhat more in detail which particularly applies to dam sites in karstic limestone.

A substantial portion of new data were obtained in testing and grouting programs executed at 23 dam sites in Spain. They were collected during a 3-months-stay at the Escuela Técnica Superior de Ingenieros de Caminos, Canales y Puertos de la Universidad de Cantabria in Santander 1994 financed by the Spanish government. Ewert thanks all Spanish authorities involved and, above all, his colleague and friend Alberto Foyo who supported this project with great efforts.

# Chapter 2
# Water Flow in Rock: Geometry of Water Conducting Paths and Lugeon-Values

Each rock mass has its own pattern of water bearing paths. In Germany, in the 1970s the term “Wasserwegsamkeit” was in common use to describe this individual setting. An equivalent term is unknown in English and the direct translation is not concise—the term ‘*conductivity*’, for instance, does not precisely meet the meaning. At first the term ‘*hydraulic routeing*’ was suggested for this subject; but probably ‘*water routeing*’ seems to be more adequate for the arrangement of the water-percolating paths: which paths exist, what is their geometry (size, shape and length) and how are they arranged along the discontinuities. Both the joint pattern and the degree of weathering determine this routeing. Figure 2.1 illustrates extreme cases, actual rock masses range in between:

- Vertical fissures crossing sandstone bands are completely open, they form a network of communicating water conducting openings; piezometer hydrographs run parallel.
- The discontinuities are still latent, some contain open veins of very thin apertures, they are connected only locally; hydrographs show largely independent courses.

The size and the shape of the paths are relevant for defining both the permeability of a rock mass and the penetrability of the grout slurry.

Textbooks deal with hydrogeological features mainly in terms of the ‘productivity’ of aquifers while the details of the water paths are not considered. Details need to be studied in assessing the permeability of a dam site. Very small geological features can create an anisotropic permeability across the foundation, and the often disregarded groundwater regime. The interaction between groundwater and reservoir is also quite significant for the under-seepage. Eventually, the groutability of a given rock type is very much influenced by the size and the shape of the individual paths. WPT’s usually do not explore these details. Thus, the hydrogeological details of the foundation need due consideration.

Wolters et al. in 1972 published the results of specific investigations concerning the percolation behaviour of a Cretaceous marlstone in Westphalia (Germany):



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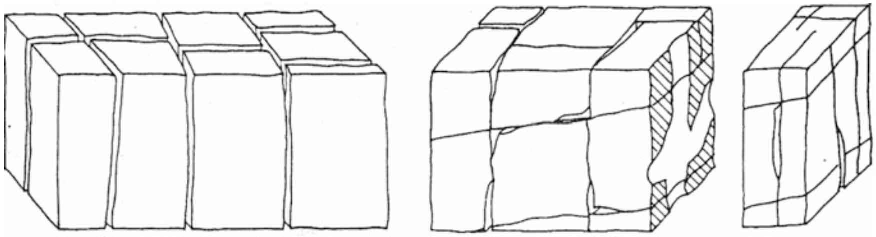

**Fig. 2.1** Schematic illustration of typical arrangements of water paths: interconnected open fissures versus partly opened joints

In that rock the water flows along channels of mostly small diameters and extensions. The paths are arranged along discontinuities. They are often not connected with each other.

Ewert extended this research to several other rock types where a similar routeing was found. Weaver also gives examples of such channels encountered in Venezuela. Meanwhile many observations made in various rock types and regions all over the world support the idea that water preferably flows through veins developed along discontinuities, except those rock types where tension tectonics caused a splitting of whole joints which directly produces open fissures. The size and the shape of the veins depend on the development reached at the time; very small nearly circular paths of <1 mm in diameter may prevail or may exist side by side with wide ones, >10 cm in diameter. As the paths are getting wider, the previously circular or elliptical shape turns into flat fissures following the discontinuities and covering an increasing portion of the joint. This type of water routeing is principally the normal one.

Rock mechanical engineers often base their models used for computation on the assumption that joints are open throughout—at least the joints of one set. However, such cases are seldom, the opposite prevails: most joints are not completely open but are furnished with local water paths. They exist in form of veins of many very different sizes; they are irregularly distributed along the joints whose major parts are still closed.

Depending on the type and degree of both jointing and weathering we observe different arrangements of the water conducting voids. '*Voids*' mean any form of water conducting path. We have to distinguish between latent discontinuities, isolated paths, locally open joints and fissures (Fig. 2.2):

A Latent discontinuities are still closed, (bedding planes, for instance)

B Isolated paths (veins) are arranged along discontinuities. Very small veins have a quasi-circular shape, getting larger and following the joint they evolve first into quasi-elliptical and then longish paths.

C Joints opened 'throughout' have a rather limited extension and their widths are usually very small (order of magnitude: $\leq 1$ mm).

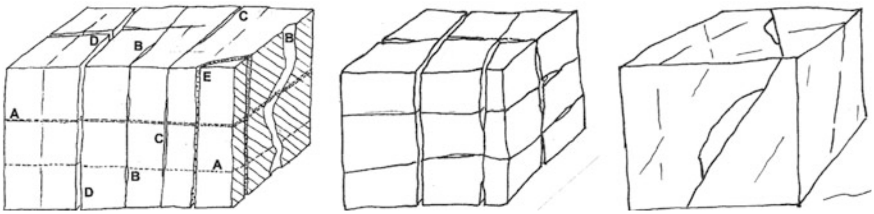


**Fig. 2.2** Arrangement and course of most water conducting paths in rock (schematic illustration), *left* Regular rock, *middle* Fissure karst, *right* Cave karst

D Open joints of a considerable width are classified as fissures (order of magnitude: >1 mm).

E Fissures can be filled with a sandy or loamy material.

F Cavities in karstic limestone differ very much in shape and size: planar fissures to huge caves from a few centimeters to many kilometers—fissure karst and cave karst, respectively.

In order to accomplish an optimal result of a grouting program we should know the type of the water routeing. Of course, a rock mass contains mostly a variety of different forms and sizes of water routes. Nevertheless, depending on the geological environment the same types of water paths prevail, e.g. planar of tubular ones. This should be taken into account since their penetrability for the grout suspension can differ very much, although they may even produce similar LU-values. In deciding on the appropriate impermeabilization criteria the penetrability of the paths, their size and shape are really important.

Since the water take of 1 LU played such a key role in deciding on the execution of a grouting program it was interesting to find out the size of the respective path. Lab tests using geologically defined models showed that a circular path of $\leq 1$ mm in diameter and 300 mm in length absorbs at a pressure of 10 bar 1 l/min, i.e. 1 LU. An elongated joint of an area of 0.2 mm $\times$ 20.8 mm and 700 mm in length yields a similar water take. Water takes $\leq 1$ LU signify even smaller entrances or longer extension of the paths. From these tests it can be inferred that small LUGEON-values originate always from isolated little paths (Type B) or from narrow joints of limited extension (Type C). Large veins, joints or even fissures produce much larger LUGEON-values. A water take of 1 LU reflects a rock which is practically impervious.

Besides the effect of the rock permeability regarding water losses their head losses have to be considered as well. If the hydrogeological setting could be assessed beforehand, an economical approach could be made concerning all aspects relating to groundwater. This has not been achieved so far and the situation concerning the effect of groundwater at construction sites is determined more by chance than by clear results of a hydrogeological investigation. Although it remains impossible to predict a given situation with accuracy, a substantial improvement could possibly be achieved if more systematic research was done. Unfortunately, investigation programs for individual projects covering all these details are expensive, and hence are not carried out. It may possibly help to improve our situation in the long run if hydrogeologists, wherever doing research work, would also observe the characteristics of this water routeing. The findings should be related to the various types of the rock masses including all the individual factors which influenced the development of water routeing and permeability. Presumably, a classification could be established after some time. On that basis, it should be possible to assess the type of the routeing and the permeability for new projects beforehand.

In order to relate WPT-values to the relevant hydrogeological features and to understand better the individual groutability, this water routeing has been examined in open pits and tunnels. The available space does not allow to present details which are described elsewhere. The results were valuable for practical purposes. This justifies the suggestion that this research should be done in as many rock masses as possible. Thus, colleagues involved in this matter are encouraged to make their contributions.

The frequency and distribution of water flowing out of rock walls varies considerably. In tunnels or other pits there are sometimes completely dry sections, several hundred meters in length, while in other water seepages are quite frequent at a spacing of a few centimeters. Studying the frequency and distribution of water seepages at rock surfaces confirms that the pattern of seepage corresponds to the type of water routeing described before. Water flows mostly along smaller or wider openings developed along discontinuities. The arrangement, course and size of the water carrying openings justify the old idea of 'water veins'. Only in a minority of cases and in tunnel sections near to the surface is the water flow along fissures open throughout.

This is no novelty and was known already to miners in ancient times. Thus, one could conclude that research on this subject is not required anymore. Nevertheless, the analysis is quite helpful:

- For hydraulic calculations, we need the coefficient of permeability $k_f$. The permeability is investigated by means of WPT's. The direct conversion of $Q_{WP'T}$ into $k_f$ is impossible; $Q_{WPT}$ does not disclose whether the water is absorbed by many thin or a few wide paths, producing different $k_f$ values as discussed below. Examining the setting of the paths allows us to recognize the prevailing type of the paths in view of their width and shape which is helpful to appraise $k_f$.
- Impermeabilization measures at dam sites are usually based on the results of WPT's. However, the tests only disclose the permeability of the rock around the borehole. WPT-results reflect the permeability of the foundation more or less appropriately if the rock tends to be homogeneous (Fig. 2.3a). If to the contrary, the rock mass encompasses impervious intercalations, WPT-results do not reflect the actual permeability (Fig. 2.3b); in such cases an impermeabilization scheme is not required.
- It is usually assumed that a rock mass below the groundwater level is filled throughout with communicating groundwater. This is not always true. There

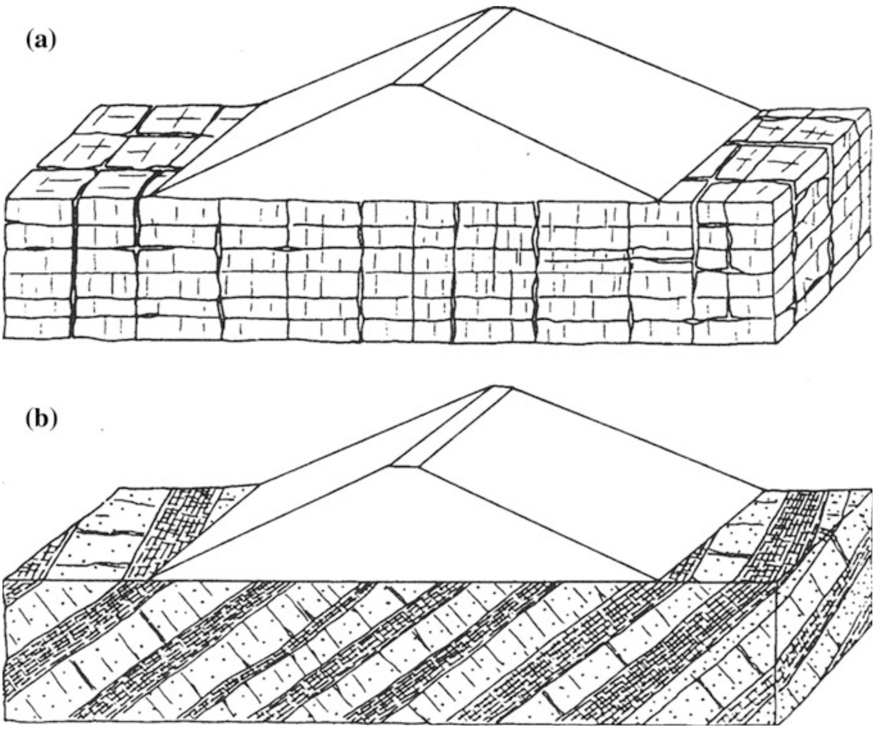


**Fig. 2.3** Dam foundation of approximately isotropic (*above*) and anisotropic permeability (*below*); the latter is not reflected by WPT-results

often exist large areas of completely dry rock and such areas can extend over many hundreds of meters, particularly in deeper zones. Piezometer hydrographs sometimes indicate not only different groundwater levels but also independent fluctuations which discloses separate groundwater systems side by side. Thus, it is not always appropriate for static calculations to consider the full hydraulic head. A different approach would be expedient.

- Driving tunnels through permeable rocks can cause a regional lowering of the groundwater table with serious consequences for buildings or for agriculture and forests above the tunnels. It is highly desirable to predict such an impact and to find out whether this is a permanent or temporary one and what should be the appropriate counter measures.
- Papers and discussions often reveal that hydrogeological facts verified by practical observations are not appreciated by everybody involved. Several scientists base their conceptions on models not completely in harmony with the real hydrogeological situation. If the facts were taken into account, a more realistic—and more valuable—result could be achieved. This is discussed in detail later.

# Chapter 3
# Examples of Grouting Programs

The following grouting programs have been selected to demonstrate both the main problems occurring in rock grouting and the achievable improvements. These examples concentrate on the installation of grout curtains because this complex is more often controversially discussed than displacement grouting for pre-stressing the rock or contact grouting for sealing the construction joints. Consolidation grouting to improve the bearing capacity is also a difficult subject; here it will be left out. The author uses the chronological order of dealing with the projects as this reflects the development in recognizing the problems and in understanding the essential factors in improving rock grouting. Dealing with rock grouting over more than forty years the authors became familiar with a considerable number of projects situated in all essential rock types, this discussion can only consider a few typical ones.

## 3.1 Tavera Dam

At the Tavera Darn (Dominican Republic, 1969–1973) a double-row curtain was originally designed. During the first phase of the grouting work it was found that the massive conglomerate with only a few intercalated siltstone layers, steeply inclined and diagonally crossing the dam, was practically impervious. A grouting pressure of $\leq 5$ bar was applied. Nearly all the grouthole stages refused to absorb either water or cement, except a very few ones yielding the combination '*no water but moderate grout takes*', which originated from infrequent cases of hydrofracturing as the grouting pressure, much higher than the maximum test pressure, was too high for a few intercalated siltstone layers. The mean values for both water and cement takes convincingly demonstrate the imperviousness of the rock (Fig. 3.1). Thus, the scope of the work was drastically reduced. Finally, a shallow contact grouting was carried out, widely spaced deeper holes served more for a tighter

 
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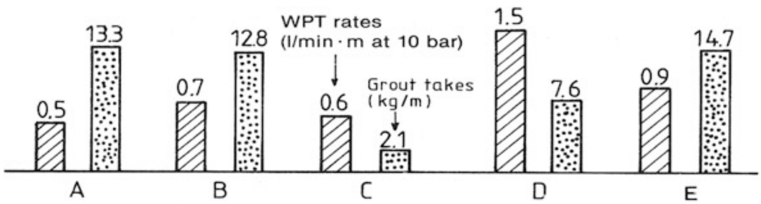



**Fig. 3.1** Tavera Dam. Mean values of grout takes for Sections III–VII, Series D and E were only made in Sections III and V

investigation rather than for an overlapping sealing which was neither required nor achievable; Fig. 3.2 displays a typical part.

Additional grouting was ordered for the spillway section because a failure which occurred during the test of the spillway led to the assumption that the foundation was too permeable. The results of these further series (D + E) confirmed the low permeability. But as higher grouting pressures were prescribed for the deeper zones ($\sim$20 bar), the combination '*little water, large cement takes*' occurred more frequently.

To explore whether the high grout takes were caused by hydrofracturing, several test holes were carried out to analyse this behaviour. The results confirmed the expectation. The example in Fig. 3.3 demonstrates this hydrofracturing very clearly. While grouting pressures of $\leq$15 bar did not initiate fracturing, or no more than a few times, the higher grouting pressures of $\approx$20 bar always initiated fracturing. Higher grouting pressures applied in Series D and E produced larger grout and water takes due to fracturing and dilation of open paths. The high pressures applied already in Series D loosened the rock bond, thus Series E yielded a further increase of grout and water takes.

Figure 3.4 shows further P/Q-diagrams in detail. The statistical analysis also shows the higher grout takes: The summation curves of series D + E displayed in Fig. 3.5 indicate a comparatively larger portion of higher takes. It is interesting to note that they occurred more often in series E rather than in series D. This indicates that too high a grouting pressure produces a certain loosening of the rock bond.

The hydraulic function of an effective grout curtain is the over-proportional reduction of the water head imposed by an impounded reservoir. The recorded uplift pressures show a linear decrease indicating an ineffective grout curtain (Fig. 3.6). The hydraulic situation of the underground remained unchanged also by secondary grouting.

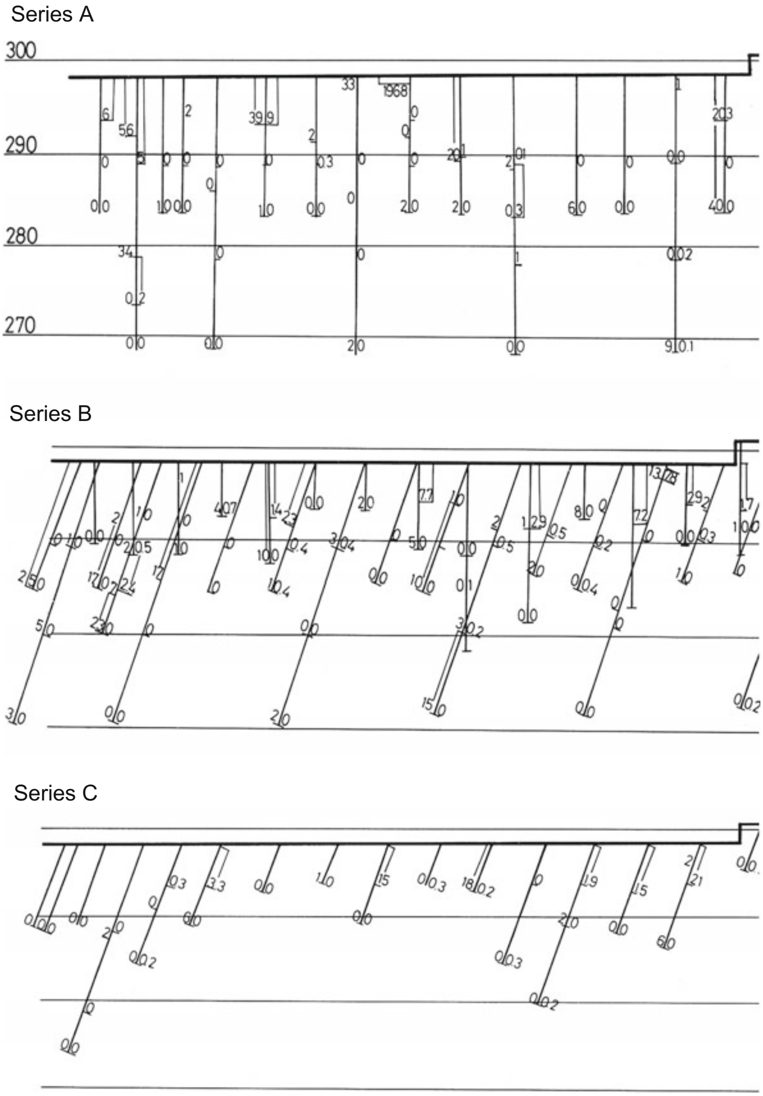



**Fig. 3.2** Tavera Dam: typical part of grout curtain. Subsequent series A, B and C are plotted separately. WPT-values at the *left side* of the hole, individual grout takes at *right side*

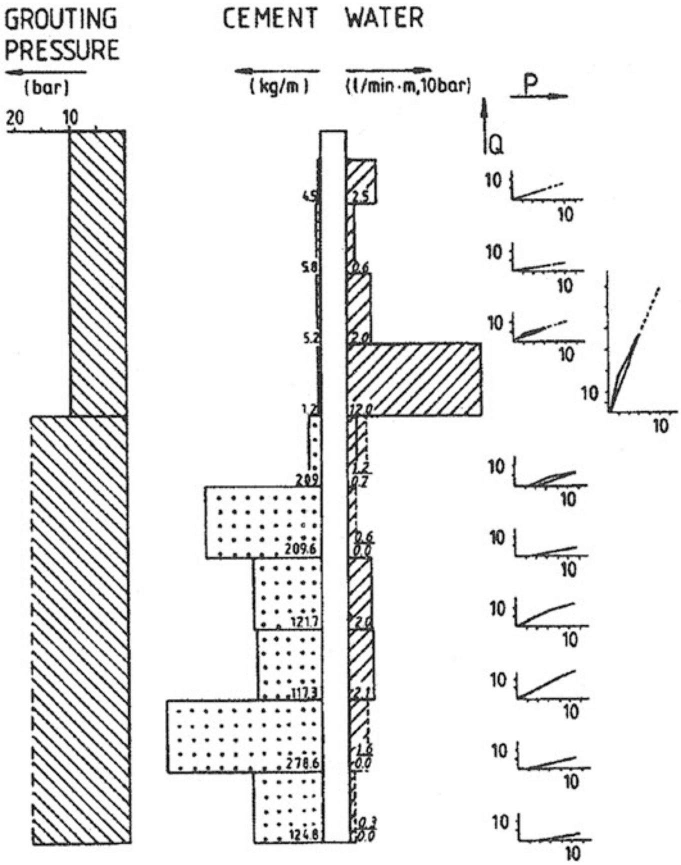



**Fig. 3.3** Tavera Dam. Higher grouting pressures cause larger grout takes due to hydrofracturing

## 3.2 Antrift, Haune and Twiste Dams

Between 1975 and 1985 these three smaller dams were built for flood control in Hesse, Germany. All foundations consisted of a well bedded and alternated sandstone–siltstone sequence. The horizontal bedding planes are closed; they are covered with mica providing a distinct fissility. The joint pattern is quite intense but different for siltstones and sandstones, respectively. The siltstones have closed joints or very narrow fissures. The sandstones besides having closed or narrow fissures also have wide ones extending over longer areas. This fabric of discontinuities causes anisotropic permeability: it is quite high along the sandstone bands and comparatively low across the bedding. Due to the distinct fissures along the bedding and the intense jointing the rock units are not tightly connected but move with respect to each other. Thus, the rock is particularly susceptible to hydrofracturing along the latent bedding planes. WPT's disclosed that the critical pressures

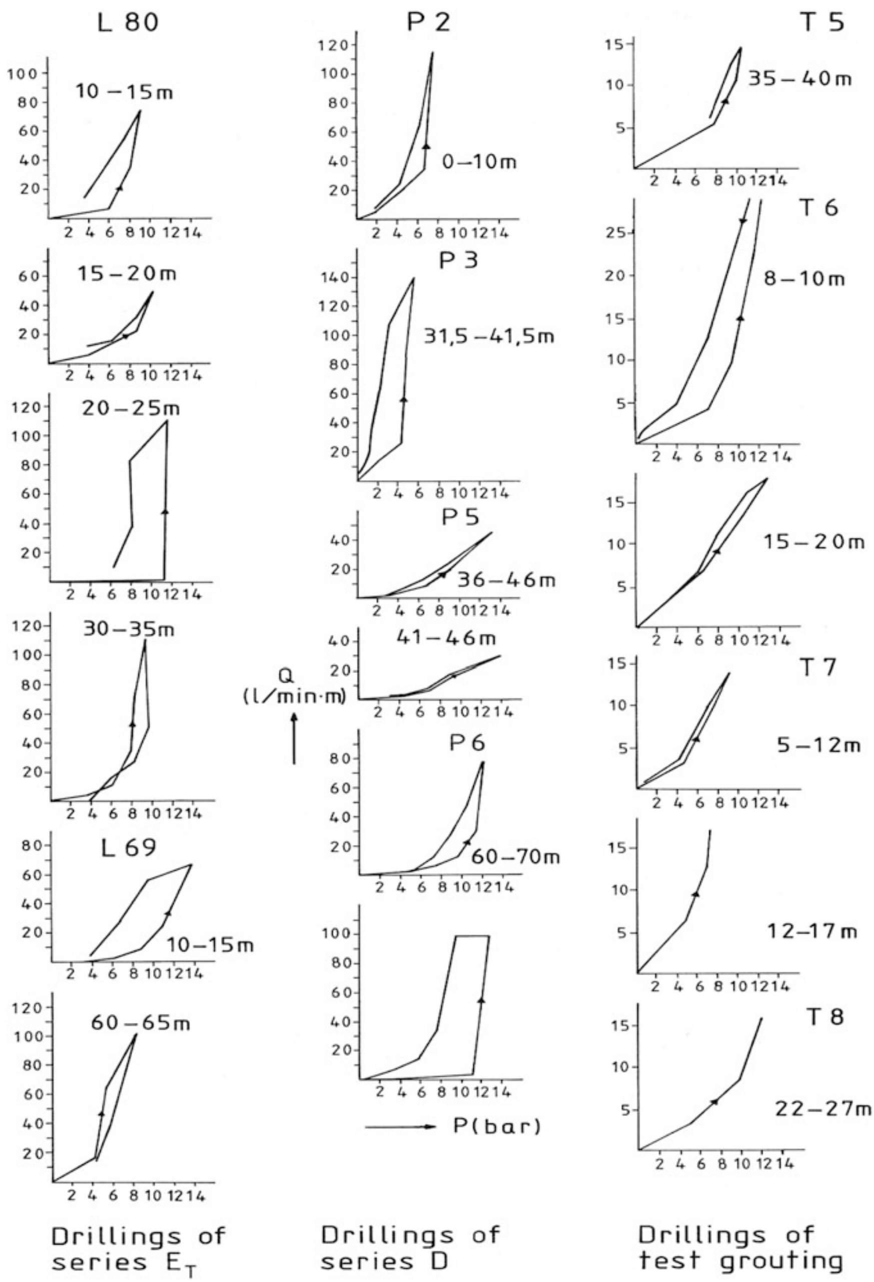



**Fig. 3.4** Tavera Dam: typical classification of P/Q-diagrams—tight rock sections are fractured and permeable sections are dilated

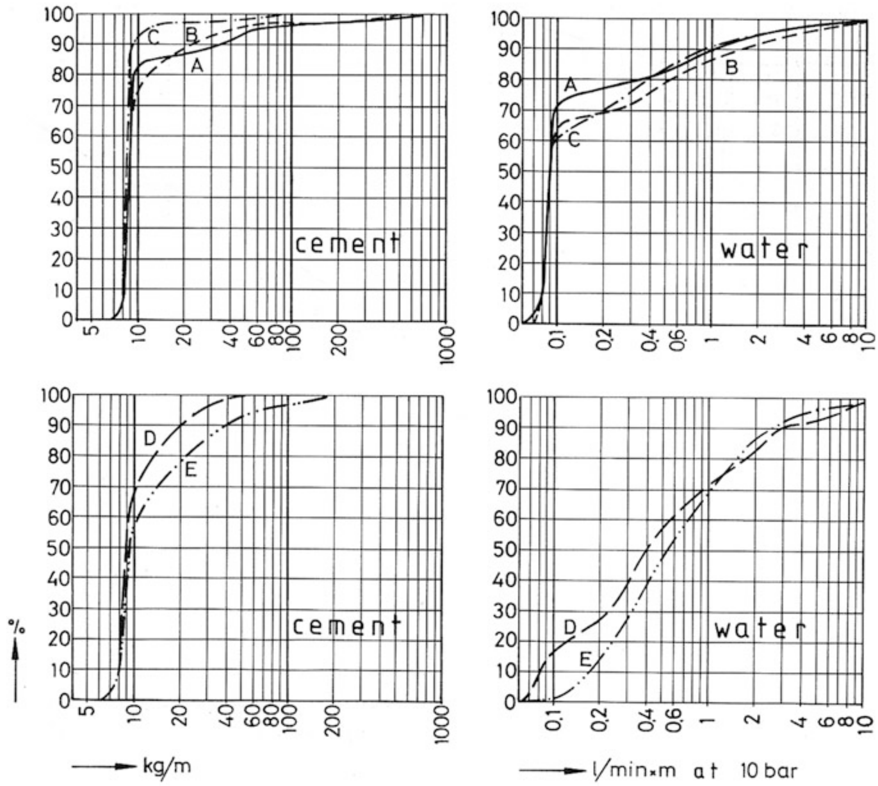



Fig. 3.5 Tavera Dam. Summation curves showing the relative frequency distribution

increase with depth because they are controlled by the overburden pressure; in shallow zones pressures of $\leq 3$ bar already heave the rock (Fig. 3.7a). The dissimilar fissures and the distinct susceptibility to hydrofracturing cause a poor groutability. As shown in Fig. 3.7b, low grouting pressures are sufficient for the grouting of the wide fissures ($P_w$) while the finer paths would require higher grouting pressures ($P_f$) which are not attainable because the latent bedding planes are hydrofractured already ($P_{hf}$) and subsequently re-filled.

The grouting work to be done at the Antrift Dam and at the Twiste Dam started at nearly the same time. During the first phase of the grouting work the poor groutability was not discovered. At both dams this caused intense hydrofracturing of the latent bedding planes at grouting pressures of $\leq 3$ bar. At Twiste the results were even worse, therefore this discussion deals only with this example.

There, the repeated horizontal and easily fissionable bedding planes caused unfavourable geological properties of this rock mass, and thus a really poor groutability illustrated already by the mean values of the grout takes in Fig. 3.8.

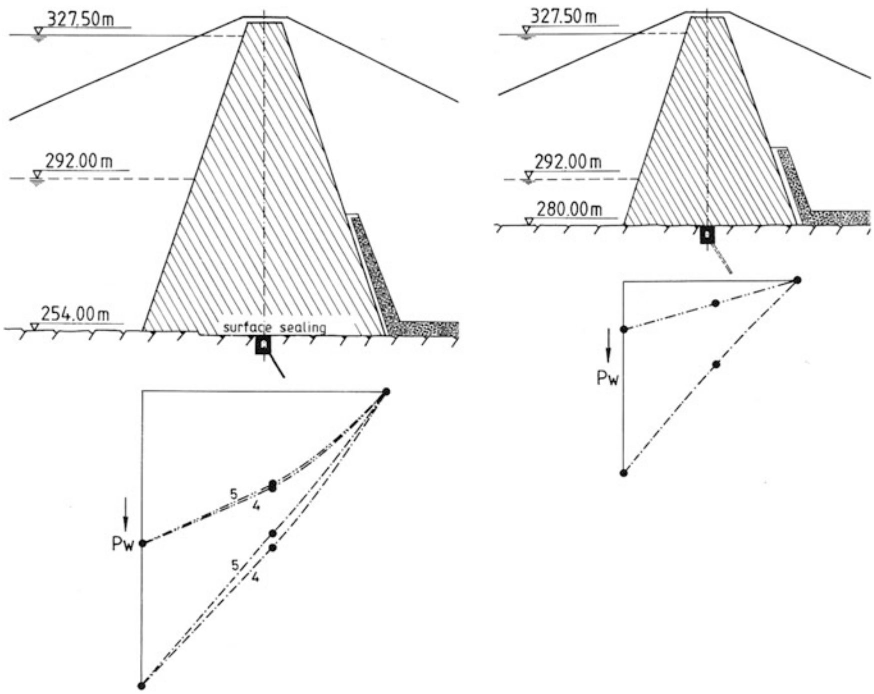



**Fig. 3.6** Tavera Dam. Linear reduction of uplift pressure

Rock grouting at dam sites is usually carried out in series. The grout takes of the later series decrease when voids are filled, i.e. decreasing of mean values indicate a progressive sealing; vice versa: increasing characterizes inappropriate grouting technology, probably fracturing of closed bedding planes. Here, almost 50% of the whole injected volume (average: = 1250 kg/m) was deposited along fractured bedding planes. The wide fissures absorbed less grout and the finer fissures were not grouted.

Figure 3.8 shows such mean values for typical sections:

- In Sections II, III and IV the grout takes of the later series increased—faulty grouting work.
- In Sections VI, VII and VIII the later series decreased—adequate grouting work.

The unusual large grout takes occurring particularly in the section of the valley area were also caused by a technological failure explained in Fig. 3.9. Downstage grouting was applied but not in the usual way with the packer set at the top of each section, and with a maximum allowable grouting pressure related to both the depth of the section the and properties of the rock type. By contrast, the packer was set always at the top of the first section directly below the beginning of the rock. Instead of drilling, grouting, re-drilling of the section grouted before and drilling of the next section, the packer remained in its first position while the grouting pressure

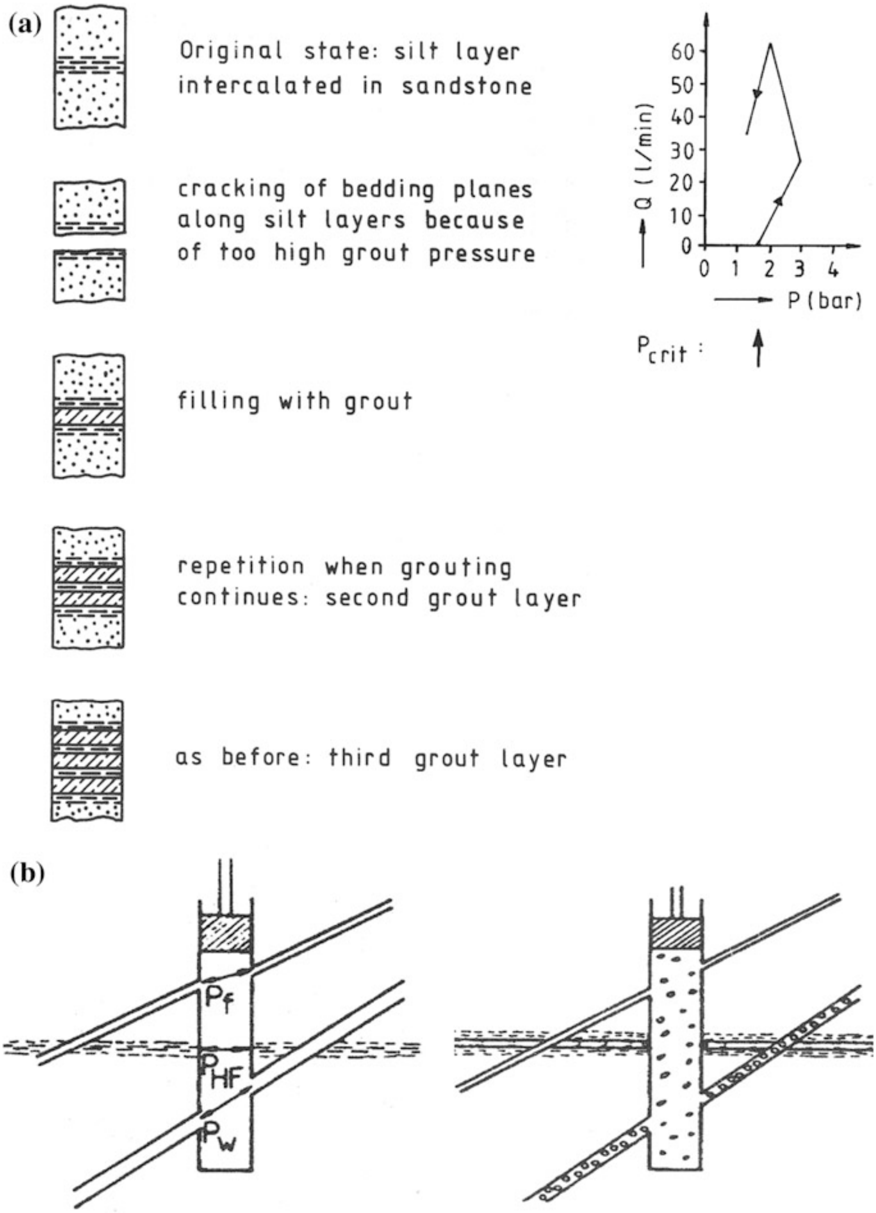



**Fig. 3.7** **a** Twiste-dam horizontal bedding planes are fractured at low pressures and filled with grout. Further grouting phases yield repetition of fracturing along bedding planes subsequently also grouted. **b** Twiste Dam: wide fissures are grouted at low pressures, fine fissures need high pressure but bedding planes are fractured and grouted before

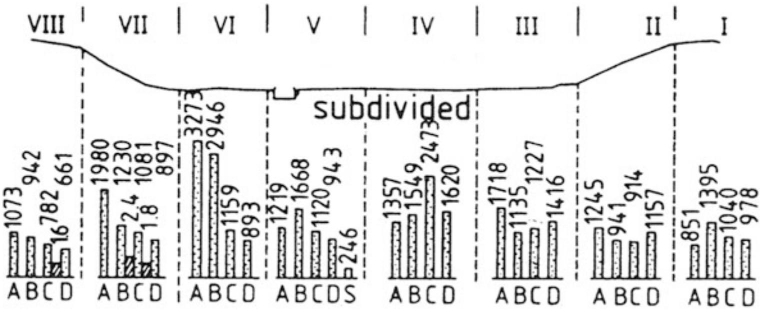



**Fig. 3.8** Twiste Dam: mean values of grout takes

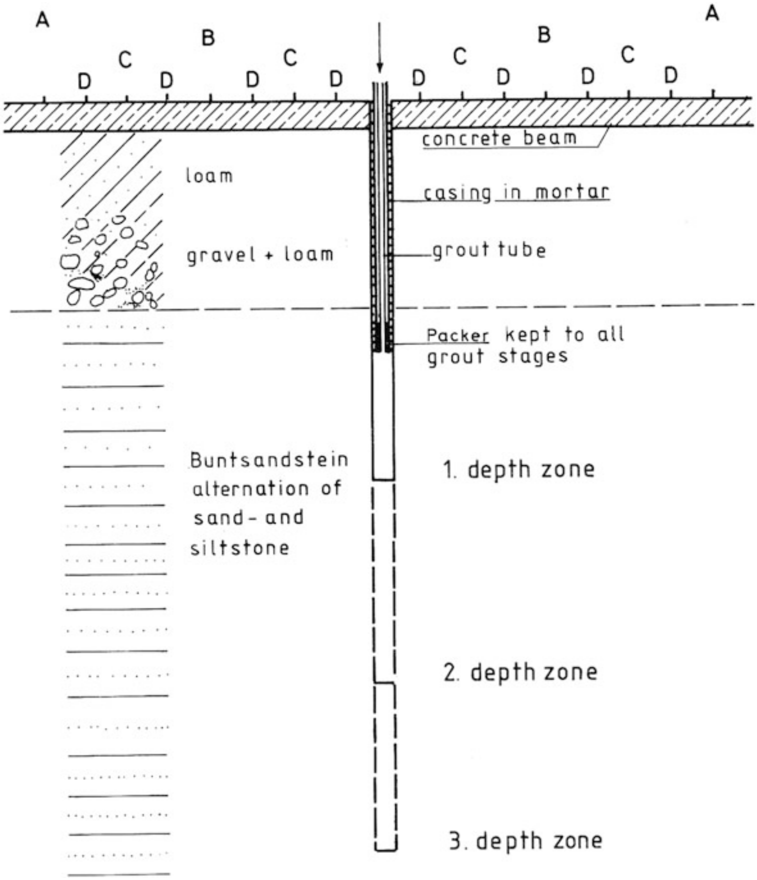



**Fig. 3.9** Twiste Dam, grouting scheme, with packer fixed inside casing

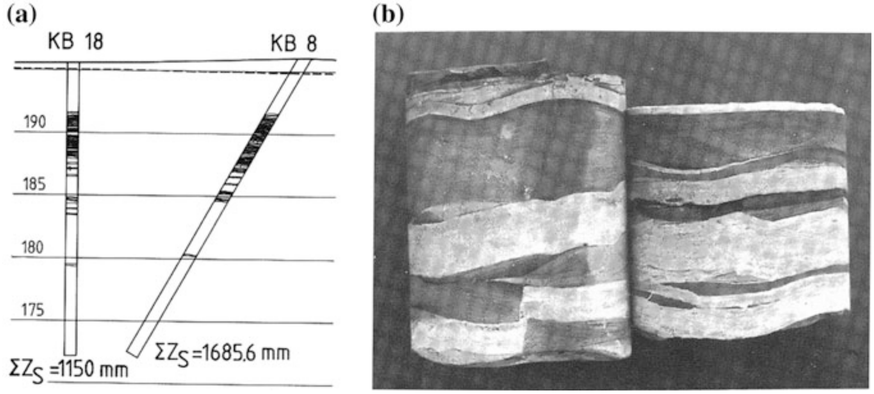



**Fig. 3.10** **a**, **b** Twiste Dam: groutstone layers deposited along fractured bedding planes demonstrate inadequate grouting technology

increased with the depths of the deeper sections. When the third depth zone was grouted its higher grouting pressure defined for that depth was partially also effective in the upper sections of the drilling where it was much too high for the lower strength of their rock bond. That discrepancy yields the result shown in Fig. 3.10.

Figure 3.10a exemplifies the results of the faulty execution of the grouting work carried out along the bottom of the valley. Control drillings showed in the uppermost zone many groutstone layers deposited along fractured bedding planes. All such groutstone layers together registered in Hole KB 8 reached 1.69 m in thickness, in Hole KB 18 1.15 m, respectively. The concrete slab installed for the execution of curtain grouting was really heaved up that mount. Figure 3.10b: The position of the groutstone layers demonstrate that they were not inserted simultaneously but during several grouting processes following each other. These results confirm convincingly that by far too high a grouting pressure was applied.

To allow a clear interpretation of the mean values, they ought to be complemented by the frequency distribution of the individual grout takes. If the subsequent grouting series progressively seal the voids, smaller grout takes are absorbed and less large ones. The frequency distribution changes in favour of the smaller grout takes and the respective summation curves shift towards the left (Fig. 3.11a). The summation curves in Fig. 3.11b indicates the contrary: the grout takes of the later series have not decreased, hence their curves share the same position. This unchanged frequency distribution indicates a similar deformation behaviour of the rock; sealing of unequal voids was not achieved.

Not only the increasing mean values in Fig. 3.8 but also the identical summation curves in Fig. 3.11b suggest that the grouting technology was not appropriate: too high a pressure caused hydrofracturing of beddings and re-filling with grout. While the statistical evaluation is still an indirect proof of such an occurrence other findings together give very clear evidence of this:

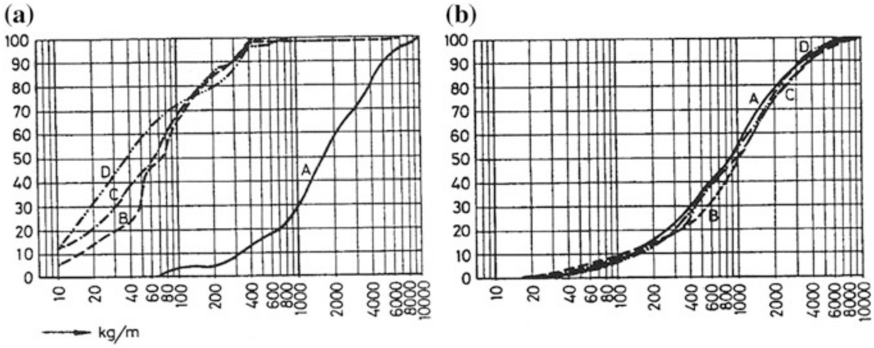



**Fig. 3.11** **a**, **b** Twiste Dam, frequency distribution of grout takes for series A, B, C, D

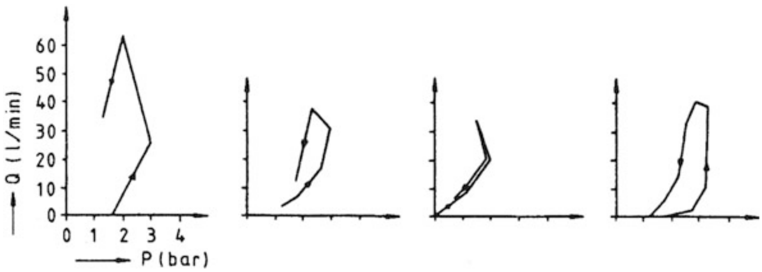



**Fig. 3.12** Twiste Dam. Typical P/Q-diagrams indicating low critical pressure

- The grout cap has been lifted, unfortunately up to more than 1 m in some sections (Fig. 3.10a).
- The groutstone layers encountered in control drillings along fractured bedding planes occurred repeatedly (Fig. 3.10b).
- The WPT-diagrams clearly identify the low critical pressures (Fig. 3.12).

The photographs in Fig. 3.13 indicate the existence of wide fissures and fault zones containing debris. Both are groutable yielding a tight groutstone. Therefore, it was concluded that a grouting program was required. The groutstone filling the fissures and the cementation of the debris justify not only that conclusion but also showed that sealing of the rock and stabilization of the debris is really attainable. However, the results reveal also that the grouting technology was not adequate. The left sample shown in Fig. 3.13 confirms this assessment convincingly: the groutstone filling of the wide fissure was cut by horizontal groutstone layers which intruded later fractured beddings during subsequent grouting phases.

The expensive and time-consuming results of the grouting work carried out along the bottom of the valley initiated an investigation program. Soon it became obvious that a faulty technology was applied: in shallow sections the critical pressures of the intercalated siltstone layers were not higher than 3 bar which made

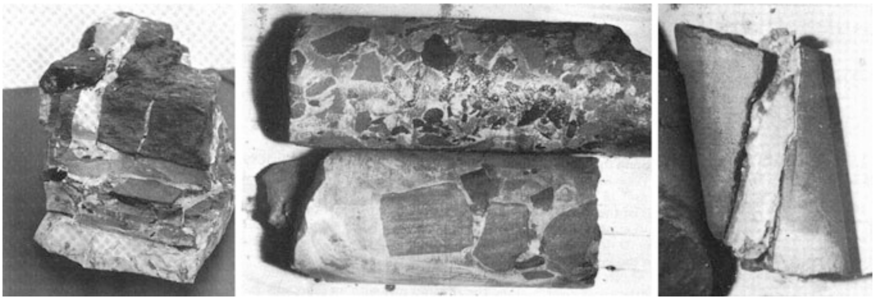
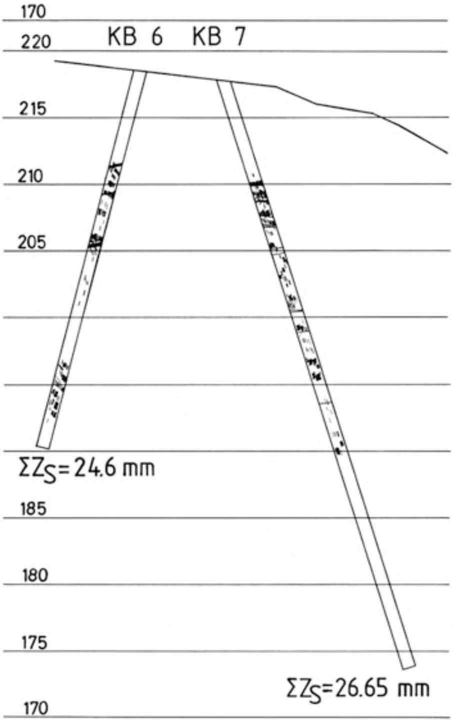


**Fig. 3.13** Twiste Dam: results of grouting work Phase 1: grouted fault zone filled with debris, steep wide fissures filled with grout but interrupted by intruded horizontal groutstone layer

**Fig. 3.14** Twiste Dam: improved grouting technology yielded better results—groutstone deposited in fissures and much less on fractured bedding planes




the rock quite suitable for hydraulic fracturing which, in fact, frequently occurred, in particular within the surface near a 1 m deep zone, as displayed in Figs. 3.7 and 3.10.

Consequently, the grouting technology was improved and was applied for the grouting work still to be done in the slope sections. Low pressure grouting and limitation to comparatively short grouthole sections permitted to reduce the thickness of all horizontal groutstone layers resulting from hydrofracturing of bedding planes in control drillings KB6 and KB7 to 25 and 27 mm, respectively, while grouted fissures were found. In Sections VI, VII and VIII (left slope) the mean values of the grout takes decreased accordingly (Figs. 3.8 and 3.14).

Nevertheless, the average grout takes of the last series are still very high which justifies the conclusion that only wide fissures could be sealed while the narrow ones remained largely ungrouted. This was confirmed by core samples. The poor groutability caused the still substantial residual permeability of 30 LU. Moreover, the siltstone was erodible. Under such conditions considerable constructional measures were required to achieve a satisfactory performance of these small dams. Higher dams would have demanded even more expensive measures which were not warranted. All grouting works and other measures are comprehensively described.

The Haune Dam was built later and all the experiences could be utilised. The foundation was partially sealed by a diaphragm; the remaining part was grouted. Low pressure grouting was used to minimize the hydrofracturing of the latent beddings. After all, a comparatively better result was achieved; nevertheless, finally it had to be realized that a considerable permeability remained which, due to the limited groutability, could not be reduced any further by reasonable means. Conclusively, it would have been difficult—or impossible—to manage a higher dam creating a greater hydraulic head.

## 3.3 Möhne Dam

In the course of more than 50 years of operation of the Möhne Dam (Germany) the seepage through the foundation consisting of an alternating sequence of sandstone and shale had increased. For this reason, it was decided to grout the rock which had not been treated before. The following summary characterizes the course of this program and the groutability of this rock:

- The examples of the grouted fault zone shown in Fig. 3.15 demonstrate that this grouting improved the foundation.

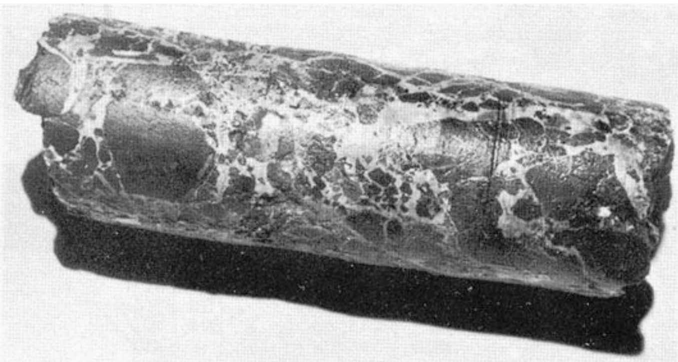


**Fig. 3.15** Möhne Dam. Grouted faultzone

- The groutstone layers shown in Fig. 3.16 were not deposited along natural openings but pressed into split bedding planes as a result of too high a grouting pressure. Vertical Quartz-filling of fissures were also fractured. Detailed studies disclosed that this dissimilarity was also caused by hydrofracturing.
- P/Q-diagrams demonstrate that hydrofracturing begins at critical pressures between approximately 7 and 11 bar (Fig. 3.17).
- Figure 3.18: The WPT-results of series A revealed that the original permeability in most sections was quite small (<3 LU), it was a little larger (3–10 LU) only under the slopes, but more permeable locations ($\geq$ 10 LU) were identified only in very few places. WPT's of Series B testified that grouting of the Series A reduced this permeability.

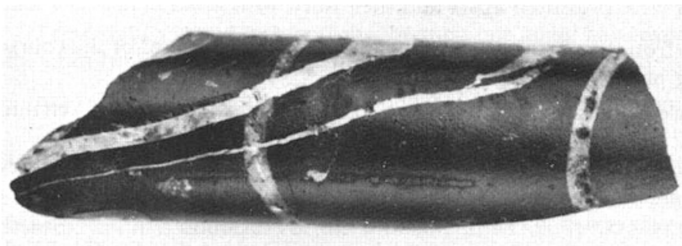


**Fig. 3.16** Möhne Dam. Groutstone deposited in fractured beddings, vertical natural Quartz-filling of fissures also fractured

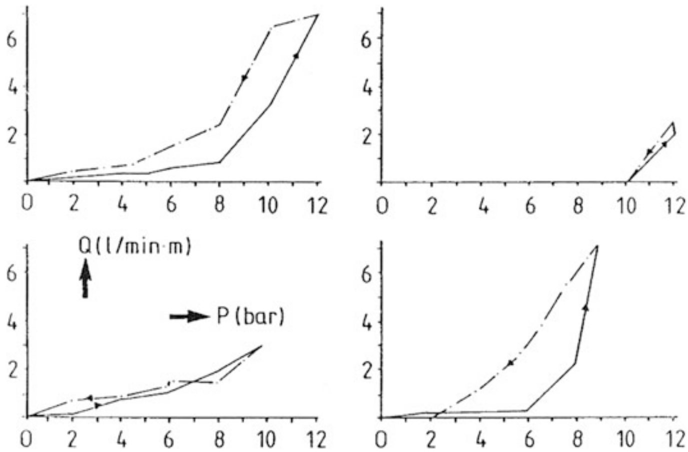



**Fig. 3.17** Möhne Dam. Typical P/Q-diagrams indicating widely ranging critical pressures; fracturing falsifies WPT-values

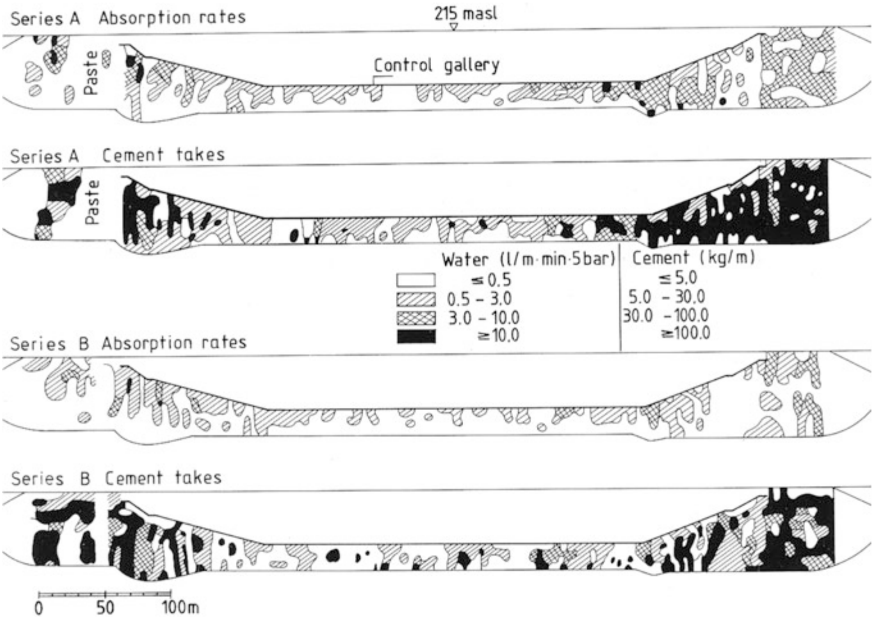



**Fig. 3.18** Möhne Dam. WPT-results and grout takes of series

- Figure 3.18: Many grout takes of Series A + B do not correspond to the WPT-results because small water absorptions ($Q_{WPT}$) are combined with large grout takes ($Q_C$): $Q_{WPT} \leq 10$ LU and QC $\geq 100$ kg/m.

The result of this grouting was to seal only a few permeable fissures and faults; too high a grouting pressure caused considerable hydrofracturing impairing the economy of this program.

## 3.4 Prims Dam

At Prims Dam (Germany) a grout curtain was installed to diminish the seepage through the permeable foundation consisting of quartzites and slates. WPT's yielded for the upper 25-m deep zone of the valley section $Q_{WPT} \geq 20$ LU, and at a greater depth 6 LU $\leq Q_{WPT} \leq 20$ LU. The slope sections showed $Q_{WPT} \geq 20$ LU down to a depth of 70 m. Due to a careful execution of the grouting work it was possible to achieve a substantial impermeabilization in spite of a limited groutability.

Due to a distinct fissility of the pronounced bedding planes this rock is particularly susceptible to hydrofracturing. The tensile strength across the bedding is low

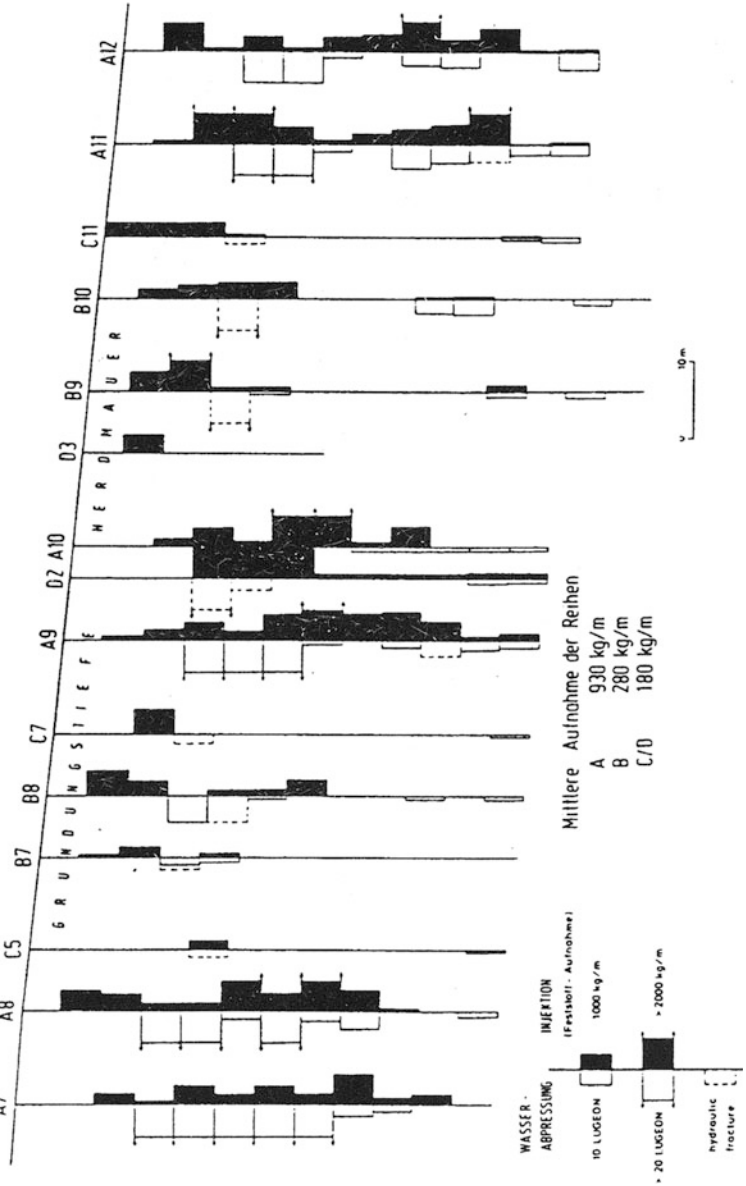



**Fig. 3.19** Prims Dam. Part of grout curtain—WPT-results and grout takes

in spite of a considerable strength of the rock itself. The bedding planes and several fault zones cross the dam from upstream to downstream.

During the test grouting (Fig. 3.19), a maximum grouting pressure of 20 bar had been applied in zones below 35 m. It was clear that this pressure would cause intense hydrofracturing, but at the same time it was hoped that this would seal the finer paths. This attempt failed. It caused grout takes up to several tons per meter, which were intercalated along a few bedding planes. These lenses, crossing the dam, do not form an overlapping curtain. The finer paths remain ungrouted because the high pressure required to penetrate them was not reached (Fig. 3.19).

WPT's disclose clearly the susceptibility for hydrofracturing:

* Near to the surface fracturing begins at a critical pressure of $\leq 3$ bar, and it depends on the overburden pressure, i.e. it increases with depth;
* Between 30 and 60 m fracturing occurred occasionally at pressures between 6 and 8 bar and regularly at pressures between 10 and 12 bar.

Thus, the grouting pressure was reduced to 3 bar for the zone between 5 and 10 m, 5 bar for the zone between 10 and 15 m, and 6 bar for the rock below 15 m, respectively, though even the low pressure of 3 bar led sometimes to a fracturing of latent planes. With these grouting pressures and a double-row curtain with an alternated 3 m spacing of the groutholes an optimum result was achieved. The grout takes in three subsequent series decreased as follows: 930–280–180 kg/m. After grouting, the residual permeability ranged between 2 and 5 LU.

## 3.5 Almendra Dam

The Almendra Dam (Spain) is an example that an allegedly hard rock—competent granite—may also be prone to hydrofracturing which is not usually expected. WPT's carried out before grouting allows a direct comparison of water and grout takes; the water takes are related to 10 bar (reference pressure of the Lugeon-criterion), the grout takes result from higher grouting pressures. The relationship $Q_{WPT}/Q_C$ shown in Fig. 3.20 demonstrate the combination '*little water but large grout takes*', particularly in deeper zones. As this combination originates from hydrofracturing of a largely impervious rock, it is evident that already not too high a grouting pressure causes intense hydrofracturing in rock types of different hardness—Almendra versus Tavera—but with a laminated texture possessing pronounced planes prone to fracturing.

As often observed, it was difficult also in this project to find the injected grout in core samples. In many cases the grout is rare also in joints clearly being percolated, even if those sections of the grouthole received large quantities of grout. Cases prevail showing grout on a few joints only while their majority remain untreated. At first glance this discrepancy between large grout takes and this scarceness of grout layers is surprising, however it corresponds with hydraulic fundamentals (Fig. 3.21): dissimilar paths are not equally groutable. The smaller the width of the path the larger the head losses due to friction—and vice versa. The pressure required to initiate penetration and to keep the grout flowing increases accordingly. Thus, unequal joints are not grouted simultaneously, the wider ones are penetrated first at a comparatively low pressure.

**Fig. 3.20** Almendra Dam. Small Lugeon-values (*left*) indicating impervious rock versus large grout takes frequently caused by hydrofracturing

| Depth | Almendra | | | Tavera | | |
|---|---|---|---|---|---|---|
| (m) | WPT (Q10) | PG (bar) | C (kg/m) | WPT (Q10) | PG (bar) | C (kg/m) |
| -5 | 1 | 10 | 15 | 2.5 | 10 | 5 |
| -10 | < 1 | 10 | 20 | 0.6 | 10 | 6 |
| -15 | < 1 | 15 | 30 | 2.0 | 10 | 5 |
| -20 | < 1 | 15 | 10 | 12.0 | 10 | 1 |
| -25 | < 1 | 15 | 20 | 0 | 20 | 21 |
| -30 | < 1 | 15 | 10 | 0 | 20 | 210 |
| -35 | < 1 | 15 | 20 | 2.0 | 20 | 122 |
| -40 | 0.2 | > 20 | 520 | 2.1 | 20 | 117 |
| -45 | 0.1 | > 20 | 50 | 0 | 20 | 279 |
| -50 | < 1 | > 20 | 360 | 0 | 20 | 125 |
| -55 | 0.9 | > 20 | 580 | | | |
| -60 | 0.9 | > 20 | 80 | Almendra: Paleoc. Granite | | |
| -65 | 0.8 | > 20 | 350 | | | |
| -70 | < 1 | > 20 | < 10 | | | |
| -75 | < 1 | > 20 | < 10 | Tavera: Oligoc. Conglom. | | |
| -80 | < 1 | > 20 | 1040 | | | |
| -82 | < 1 | > 20 | 1040 | | | |

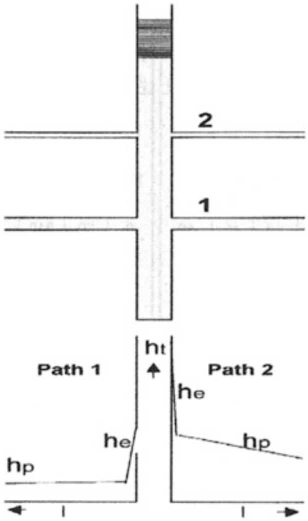



**Fig. 3.21** Almendra Dam. Unequal penetrability of dissimilar paths in grouting due to different head losses at their entrances: $h_{e1} < h_{e2}$

The project Almendra presents a multitude of cases, which convincingly demonstrate the effect of too high a grouting pressure. Figure 3.22 illustrates such courses: The fissure filled with cement-stone (left) and the suspension flown out of the rock and, after hardening, forming layers of cement-stone cover an area of many square meters (right). This spot is in the middle of the left slope far outside of the gravity dam, i.e. several tens meters away from the grouting gallery.

**Fig. 3.22** Almendra Dam. Grout suspension flew out of the surface of the slope and was deposited in layers of various centimetres in thickness

If WPT's are schematically carried out in all borehole sections subsequently grouted, the pairs of values Water/Cement-take indicate the susceptibility to hydrofracturing. The alleged incomparability between water takes and grout takes has been often discussed. The discrepancies between both values were always diagnosed, often lamented and, after all, resigned about the facts. However, this alleged incompatibility is not an enigma but explainable because it has geological reasons as the author repeatedly demonstrated.

Here is only room to discuss the discrepancy between little water takes and large grout takes: it results from the relatively small pressures applied in WPT's ($P_{WPT}$) and the much higher ones mostly used during grouting ($P_{req}$). As soon as the condition $P_{WPT} < P_{crit} > P_{req}$ rules the work and latent discontinuities exist, the course of grouting is impaired—or even completely determined—by the hydrofracturing behaviour. This yields a relationship between WPT-values and grout takes as presented in Fig. 3.23 for the Almendra Dam. There, the large grout takes of a rock mass originally nearly impervious result from a backfilling of latent discontinuities pressured open, e.g. that work was not required. Although the rock consists of a solid and strong granitic gneiss but the latent discontinuities cracked at $P_{crit} < 20$ bar.

The following feature makes the example Almendra interesting and important: the susceptibility to fracturing is not caused by well developed discontinuities as bedding planes, for instance, but by the parallel texture of the crystals originating

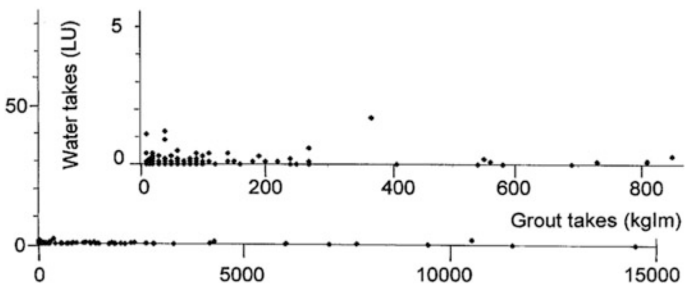



**Fig. 3.23** Almendra Dam. Relation between water and grout takes. Explanation: the upper coordinates present the segment out of the lower one at an enlarged scale: x = 800 or y = 5

from metamorphism. The detail of the grout curtain installed at Almendra shown in Fig. 3.24 demonstrates convincingly the discrepancy between the small Lugeon-values and the very large grout takes. After all it is concluded that the relationship between water and grout takes (LU/kg $\times$ m$^{-1}$) is worth to be evaluated and graphically presented because it yields valuable information about the hydrofracturing behaviour.

Horizontal groutstone pillows deposited along fractured bedding planes do not necessarily reduce the under-seepage because the vertical joints remain mostly open (Fig. 3.25, left). Cases of parallelism between the susceptible planes and the axis of the grout curtain may occur but on the whole are certainly advantageous exceptions (Fig. 3.25, right).

The examples presented here are by no means isolated cases. They confirm that the critical pressures causing hydrofracturing do not increase with depth. This proves that the conventional grouting pressures increasing with depth do not correspond with the behaviour of latent discontinuities.

## 3.6 Aabach Dam

The Aabach Dam (Germany) demonstrates very clearly the decisive role of the hydrogeological setting. WPT's apparently proved the rock very permeable while test grouting revealed a poor groutability as displayed in Fig. 3.26 much water in tests, but little grout. Further details are discussed later.

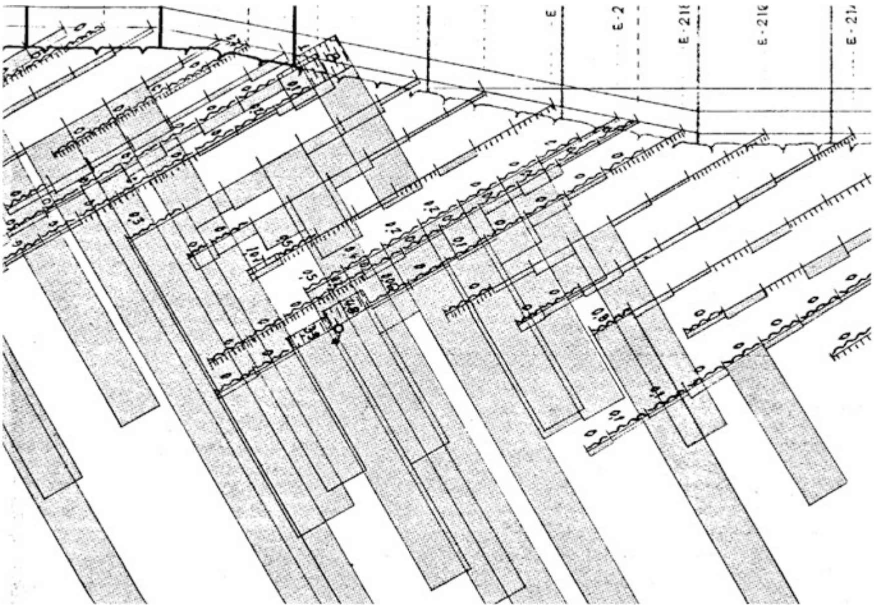


**Fig. 3.24** Almendra Dam. Segment of the grout curtain installed in nearly impervious rock

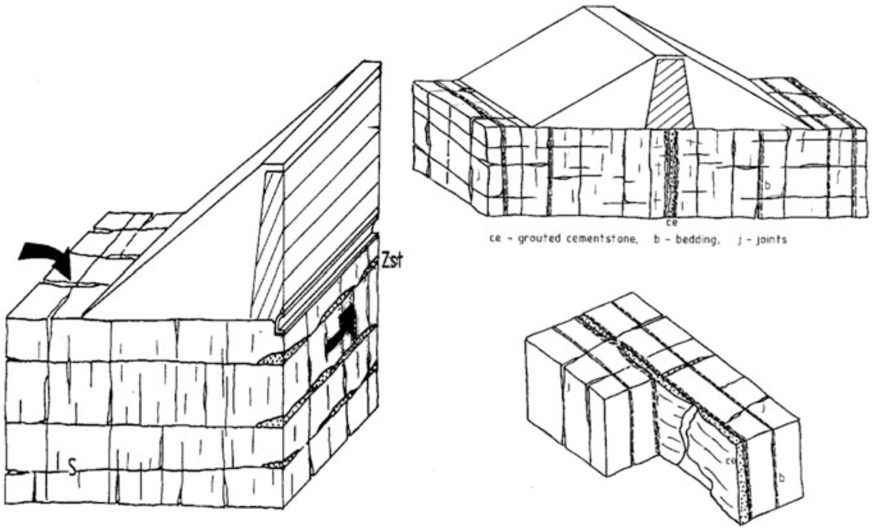



**Fig. 3.25** Groutstone pillows intercalated along horizontal bedding planes have little hydraulic effect only (*left*), vertical pillows are possible only in exceptional cases (*right*)

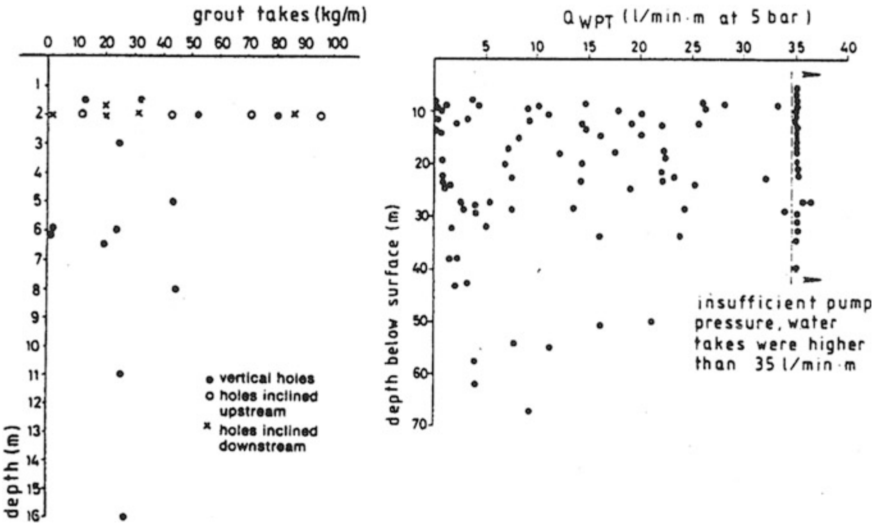



**Fig. 3.26** Aabach Dam: WPT-values (*right*) and grout takes (*left*) resulted from test grouting

At this dam site the permeability originates from narrow paths, thus, grouting requires high pressures. Such pressures were not attainable because the bedding planes showed a distinct fissility causing a significant susceptibility to hydrofracturing. The '*critical pressure*' able to split closed bedding or cleavage planes are a rock-type specific property. Therefore, it was prudent to examine the rock

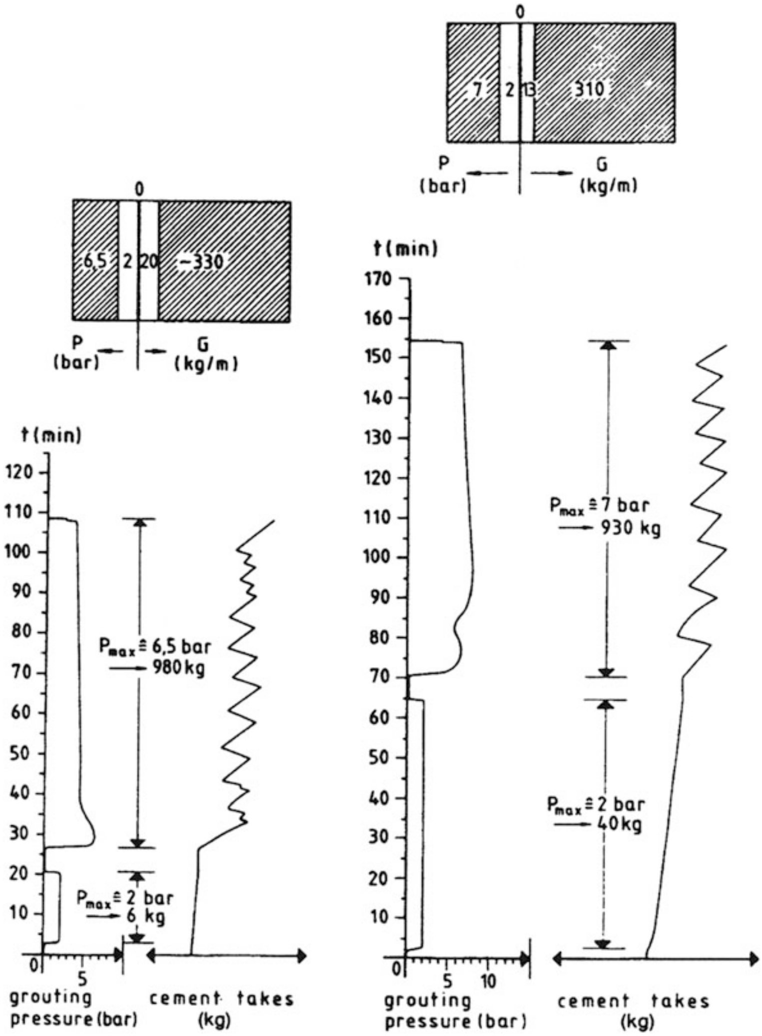



**Fig. 3.27** Aabach Dam. Tests to investigate course of grouting—very small grout takes during low pressure but large take after intentional hydrofracturing

accordingly. Here, tests in two holes were carried out to investigate the hydrofracturing behaviour occurring during of grouting; the results are shown in Fig. 3.27. In the first part of the test at a pressure of 2 bar—still without fracturing—only little grout was absorbed: 20 kg/m. After an interruption of 5 min the tests continued. At a pressure of 6.5 bar hydrofracturing occurred, the grout started flowing while the pressure decreased. After about 80 min the tests were stopped. Then the two holes had absorbed 980 and 930 kg of cement, i.e. 330 and 310 kg/m. It had to be concluded that a multitude of fine fissures produced at low pressures a

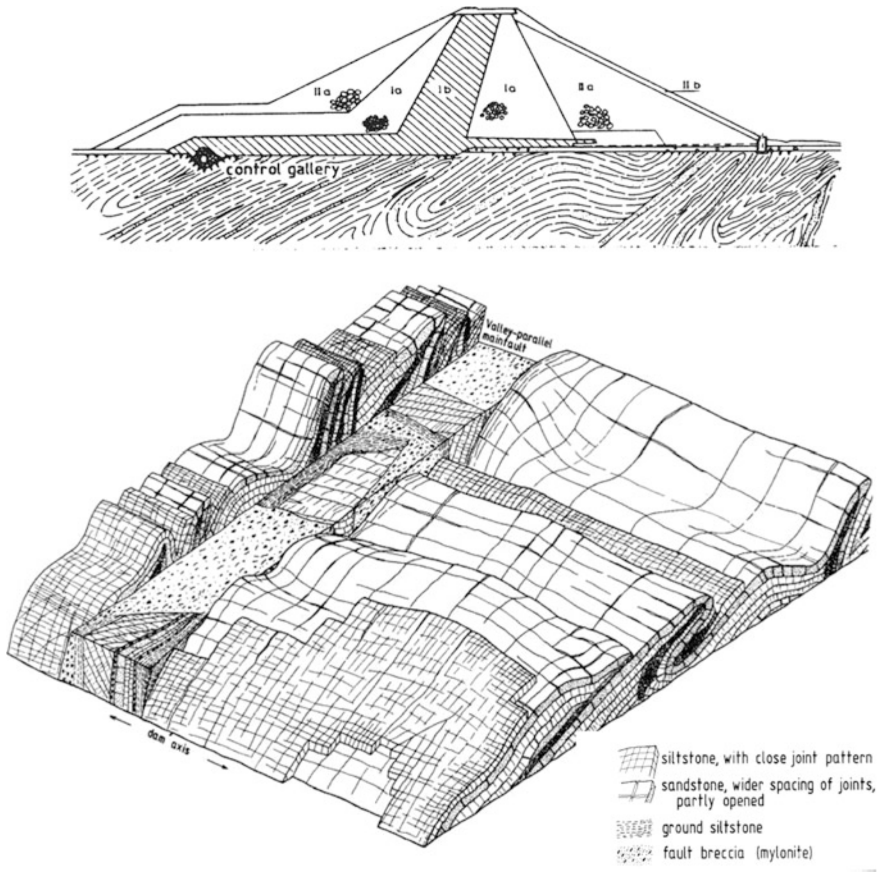



**Fig. 3.28** Aabach Dam. Geological situation in the dam area

considerable permeability but was not yet groutable. The rock first became groutable when hydrofracturing split the originally closed planes and opened access to sufficiently wide groutable fissures.

Due to (a) the impervious carpet incorporated in the dam, (b) the alternation between permeable sandstones and nearly impervious siltstones altogether heavily folded and faulted and (c) the favourable orientation of the beds, in most sections parallel to the dam axis, the hydrogeological setting in total was favourable; it is illustrated in Fig. 3.28. This allowed the dam engineers to abandon the grout curtain and confine themselves to a short contact grouting, a decision which proved very much appropriate.

Due to the heavy folding and faulting the geological situation of the dam foundation changes every several meters. The reduction of the hydraulic head caused by the reservoir changes accordingly. A rather tight net of piezometers was installed and is maintained to record the uplift (Fig. 3.29).

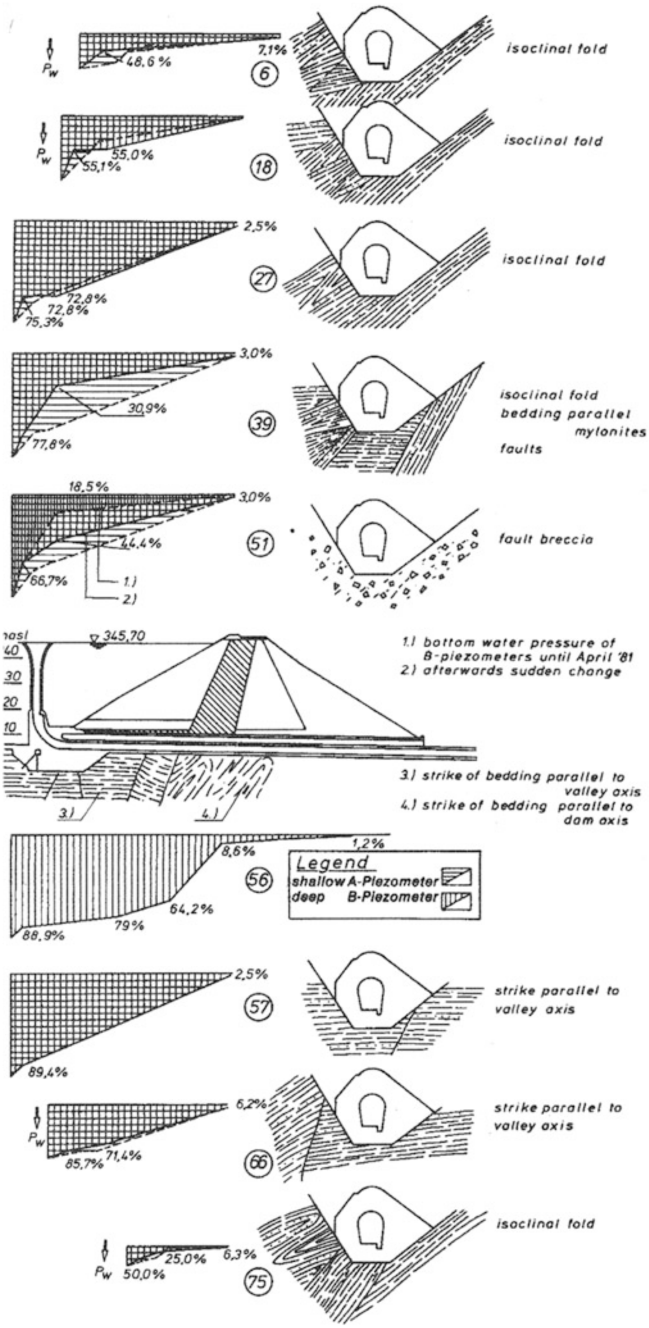



**Fig. 3.29** Aabach Dam. Reduction of uplift pressure across the inspection gallery

## 3.7 Yuracmayo Dam

The Yuracmayo Dam (Peru) is founded on glacial deposits (left slope and valley floor) and young pseudo-stratified rhyolites (right slope). The grouting of this rock has been carried out in accordance with the GIN-principle introduced by Lombardi and Deere. Unaware of a distinct susceptibility to hydrofracturing a Grouting Intensity Number (GIN) of 2000 with a maximum grouting pressure of 30 bar was used. Latent bedding-like planes were pressed open at critical pressures between 7 and 10 bar as exemplified in typical P/Q-diagrams shown in Fig. 3.30.

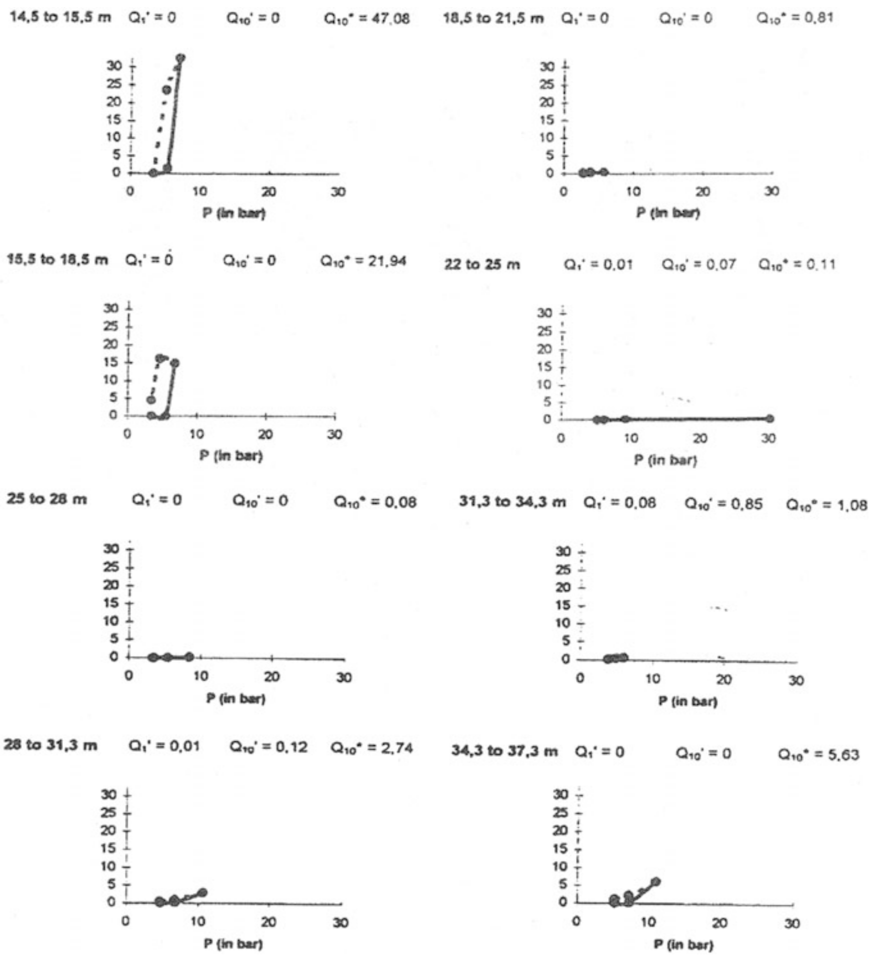



**Fig. 3.30** Yuracmayo Dam. Hydrofracturing of latent bedding-like planes at pressures $\leq 10$ bar hydrofracturing caused bad results

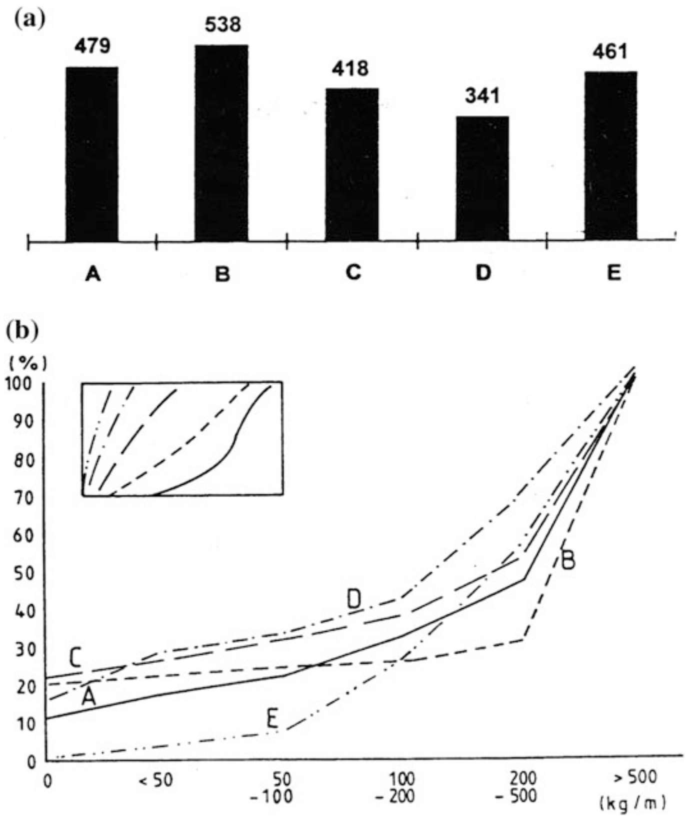



**Fig. 3.31** Yuracmayo Dam. **a** Mean values of grout takes in subsequent series, **b** summation curves of frequency distribution of individual grout takes of subsequent series; small one inscribed exemplifies progressive sealing

The grout takes were quite large: the usual decrease through subsequent series was not achieved as the mean values (A) and their summation curves of the frequency distribution (B) give evidence—even the fifth series (spacing: 0.5 m) yielded an average grout take of 461 kg/m (Fig. 3.31).

In spite of this large cement consumption an overlapping curtain could not be installed because the re-filled fractured discontinuities traverse the dam (Fig. 3.32).

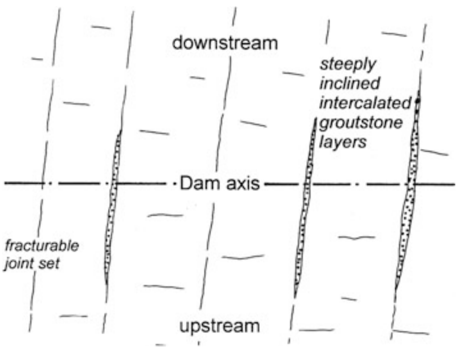



**Fig. 3.32** Yuracmayo Dam: Grout takes of all series do not reduce hydraulic head because groutstone layers cross dam axis towards downstream

## 3.8 Grouting in Karstic Limestone

### *3.8.1 Introduction*

The karstification of limestone produces a peculiar setting of water conducting voids. In an initial state only a few joints are locally open. Later on, the progressive solution of lime results first in a finer and then wider fissure, possibly followed by cavities and even huge caves. All rock masses have their degree of karstification, evolved during the geological history. Beside the karstic features the rock itself is usually tight. In normal rock, the arrangement and the location of the water paths are unknown, nevertheless, roughly a regular distribution can be assumed, for each rock type a specific one, of course. This is principally different in karstic rock where the water conducting voids are largely irregular and appear just by chance. Their arrangements concerning volumes, width, extension and direction is unpredictable. Cavities far from each other may be connected while others just nearby exist separately, even in limestone of mature karstification. When such a rock mass has to be sealed, not the rock itself but the karstic fissures, channels, cavities and carves have to be filled; this simply means that the karstification has to be eliminated.

Grouting measures in karstic limestone confront with extraordinarily difficult conditions usually causing an unfavourable relationship between expenses and benefits. We are facing two extremes. Where cavities or even caves exist, the most difficult problem is to achieve a tight filling, but where, vice versa, only a few and small openings exist, the real problem is to find them.

Medium-sized cavities readily need grout masses of up to several tons per running meter of grouthole. Huge caves are practically not groutable as the grout flows away without clogging. In order to keep this scope of the grouting within reasonable limits, special grouting techniques are used—unfortunately not necessarily successful. If so, other technologies have to be envisaged, the excavation of access shafts and the filling of the cave with suitable materials, for instance.

Small and isolated voids have to be detected. This is usually done by means of a close spacing of the groutholes which primarily serve to detect the voids. This is particularly expensive when only a few exist because many unproductive drilling meters are needed to locate them. With less voids it becomes less likely to meet them in spite of a very close spacing of groutholes.

This book does not deal with the first issue: the technology of filling large cavities. Instead, based on typical projects, it will be discussed how the hydrogeological frame can be utilized to determine whether and to what extent the treatment of a karstic limestone by means of grouting is required or when it is not warranted anymore.

Three projects are discussed because all sites had difficult geological conditions—the Pueblo Viejo Dam (Guatemala), with a very long sideways extension of the grout curtain installed from tunnels situated in three levels, the Panix Dam (Switzerland), where several springs inside and below the reservoir area indicated karstification, and the Mujib Dam (Jordan) where despite curtain grouting a sudden partial filling of the reservoir initiated growing water losses through the right abutment.

## *3.8.2 Pueblo Viejo Dam/Guatemala*

The nearly 130 m high Pueblo Viejo Dam built on the Chixoy river in Guatemala belongs to the major projects built on karstic limestone. The dam, commissioned in 1983, received an extended grout curtain. Various complexes have been described. Because of important aspects, the example of this treatment and the hydraulic behaviour of the subsoil are worth to be discussed also here.

### 3.8.2.1 Concept

The karstic limestone was investigated to some extent during the feasibility study; further investigations were done during the grouting work and the impoundment. The limestone itself is tight except in some sections of the abutments and adjacent slopes. Besides there exist wide fissures (centimeters to decimeters), smaller cavities (decimeters up to one meter) and extended caves (tens of meters). The voids are partly filled with volcanic ashes. It was concluded to install a deep reaching grout curtain and to backfill accessible cavities after the removal of the ashes.

The curtain has been installed from the inspection gallery and to both sides from the various grouting galleries and tunnels shown in Figs. 3.33 and 3.34; (GI 1 to GI 7). It was considered necessary to extend the grout curtain up to the spots where the natural groundwater table exceeds the reservoir level. This caused a considerable length of the curtain: At the left side it reaches 850 m into the mountain, at the right side 450 m, respectively. The curtain on both sides is changing in orientation in order to sooner reach the fault against the impervious serpentinite (right) and the power-tunnel (left).

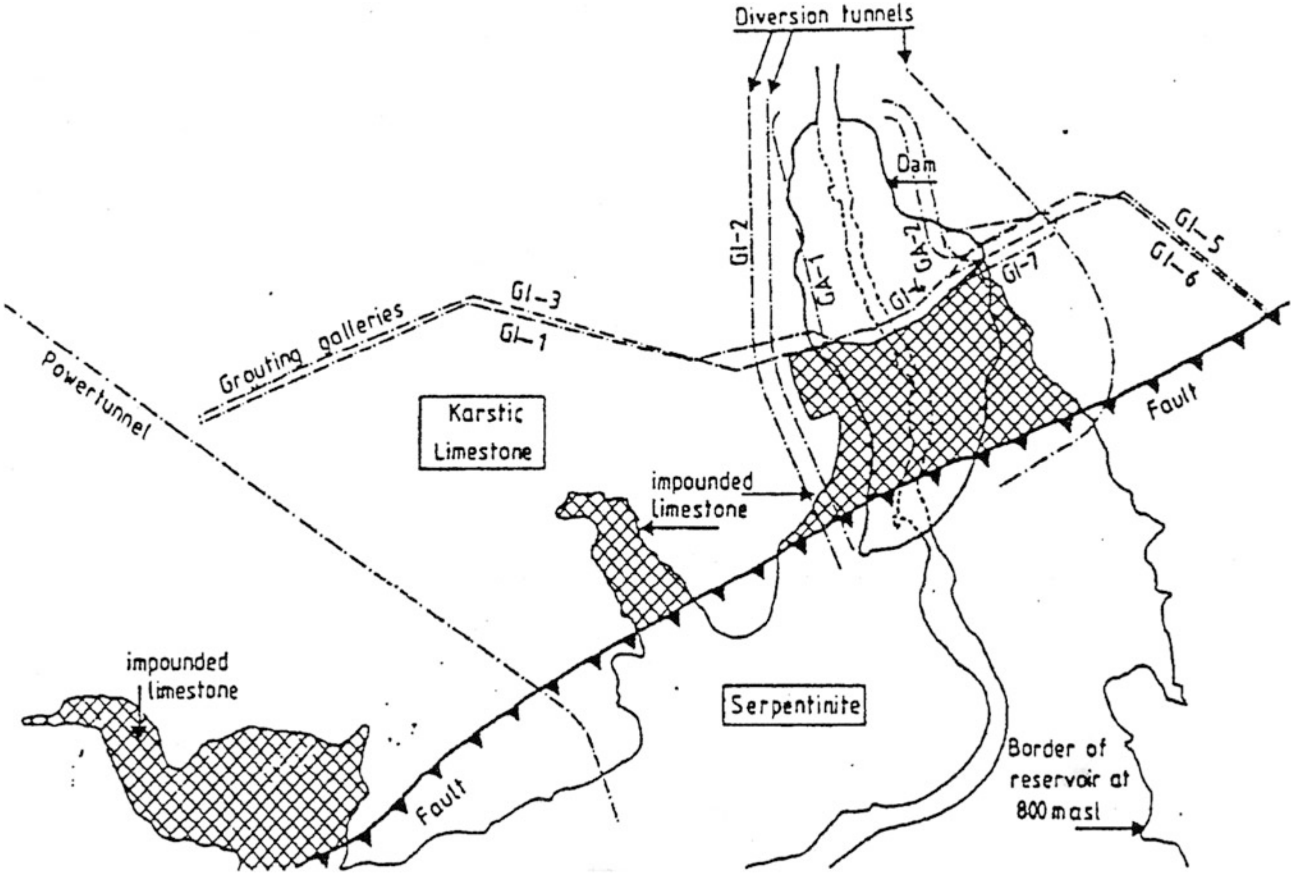



**Fig. 3.33** Pueblo Viejo Dam. Main fault. Areas of limestone impounded. Diversion tunnels, inspection and grouting galleries

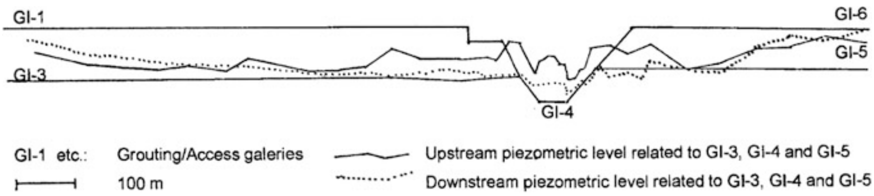



**Fig. 3.34** Pueblo Viejo Dam. Section along grouting galleries/tunnel showing piezometric levels related to fully impounded reservoir on upstream and downstream side of the curtain and changes between November 1983 and December 1985

### 3.8.2.2 Pattern of Groutholes and Piezometers

Even with a grouthole spacing of $\leq 0.5$ m it would not have been possible to meet all the small voids, hydraulically still capable. They easily cross the curtain in between of two groutholes and it is most likely that even high grouting pressures would not have opened an access by hydrofracturing. To avoid too tight a grouthole pattern in the interest of economy, it was decided to space the holes not too close together and instead to set up a relatively dense piezometer system. For the groutholes, a 1.5-m-spacing was chosen for the dam area, and a spacing of 3.0 m was deemed sufficient for the tunnel sections, except local spots of a higher karstification. The piezometers were installed along the entire grout curtain, at the abutments, on the surrounding slopes, on the downstream face of the dam and its valley flanks (Fig. 3.34).

It was uncertain whether the volcanic ash filling many cavities had enough resistance to mechanical erosion. The ash consists of a very fine grained ungroutable material. By means of hydrofracturing and high pressure grouting it was possible to intercalate isolated groutstone pillows. This improved the density but could not encase the ashes. Thus, if ash-filled cavities were to cross the curtain and had a free outlet somewhere downstream, regressive erosion was possible. In that case, it could be expected that the uplift would drop on the upstream side of the curtain and, contrarily, rise on the downstream side. To recognize such occurrences immediately, the piezometers were installed on either side of the curtain; in addition, deep reaching piezometers were placed on the downstream side, aimed at indicating to what extent the reduction of the uplift is taking place in depth. The downstream piezometers can also be operated as drainage holes.

### 3.8.2.3 Results

The average specific grout take amounted to 63.3 kg/m. In case of a normal rock types this small figure would signify a little permeability only, hardly warranting a systematic treatment. For karstic limestone, however, the meaning is different: The distribution of the individual grout takes is very unequal. While the tight rock itself does not absorb any grout, the karstic openings often cause extraordinary takes.

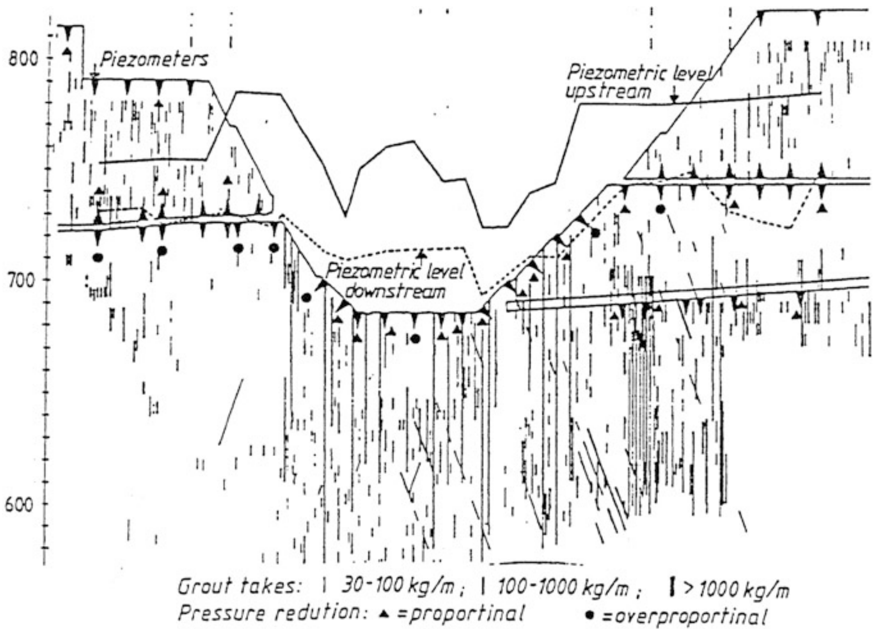



**Fig. 3.35** Pueblo Viejo Dam. Grout takes of the dam section, locations of piezometers, piezometric levels in November 1982, uplift reduction

Figure 3.35 shows the grout takes for the dam section, the original groundwater table and the piezometric levels after the impoundment. It is evident that the reactions are different from one piezometer to the other; the reduction of the uplift across the grouted zone varies accordingly.

### 3.8.2.4 Effectiveness of the Grout Curtain

The piezometers along the various tunnels indicate different groundwater tables, thus perched groundwater exists (Fig. 3.34). The excavation of the long access tunnels should have affected the upper groundwater because in the course of the construction time lasting for years, dropping of its table should have occurred down to the level of the lowermost tunnel. This did not happen. On the contrary, the upper groundwater table approximately remained in its original position. All piezometer-hydrographs revealed individual reactions to precipitation and reservoir filling. This confirms that separate water-carrying openings exist side by side, being independently connected with both the surface above and the reservoir. It also demonstrates that despite of local connections, the rock mass is impervious enough that drainage is unable to empty the upper groundwater.

The grouting changed the level of the groundwater table inside the right mountain-side. Upstream of the curtain the groundwater table rose from elevation

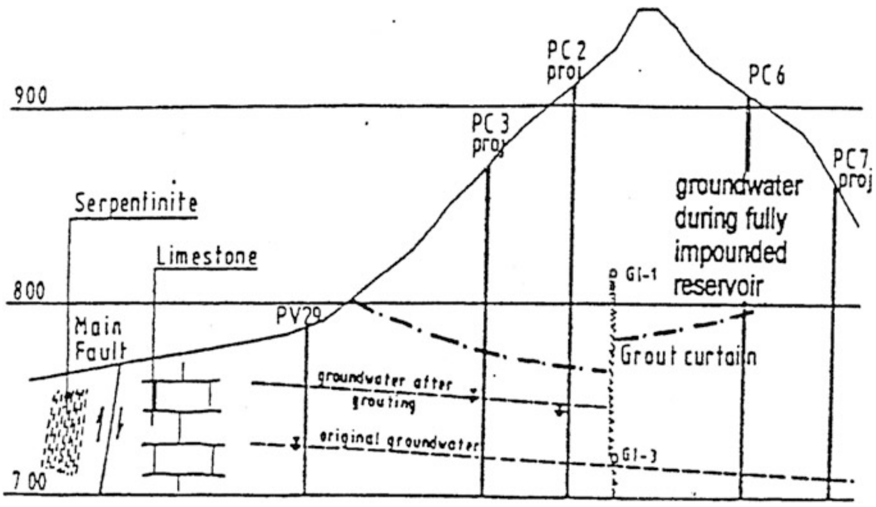



**Fig. 3.36** Pueblo Viejo Dam. Rise of groundwater table in front of the curtain due to grouting

725–753 m.a.s.l. already during the grouting work before the filling of the reservoir because thermal water, ascending along the fault against the impervious serpentinite, was stored along the curtain (Fig. 3.36). This also changed the previous north-eastern direction of the groundwater flow.

The filling of the reservoir took place between January and October of 1983 (Fig. 3.37). In November 1983 the steady state appeared to be reached, thus the hydraulic behaviour was comprehensively analysed. It is interesting to compare this initial situation with the one observed two years later (December 1985). The potentials shown in the graphs indicate the percentage of the remaining hydraulic head; the original groundwater table serves as the '*zero-state-basis*'.

The results indicate a favourable hydraulic behaviour of the foundation which, in principle, confirms the concept established for the treatment; however, a substantial over-doing has to be acknowledged. The conclusions are summarized as follows:

- The conditions regarding the permeability distribution, also including the effectiveness of the grout curtain, change from place to place resulting in varying reductions of the uplift (Figs. 3.34 and 3.35).
- Within the dam area the downstream piezometers indicated residual potentials of 5–16% (Figs. 3.35 and 3.38). This means that the decrease of the uplift across the basis of the core from upstream to downstream is satisfactory. This situation did not change in the subsequent years.
- The upstream piezometers reacted much stronger and much more differently. In November 1983 their potentials ranged between 90.0 and 23.5%. The low potentials revealed that in their sections the rock in front of the curtain had about the same permeability as the grout curtain itself, i.e. obviously a linear reduction of the uplift occurred. This applied, against all expectations, to the central

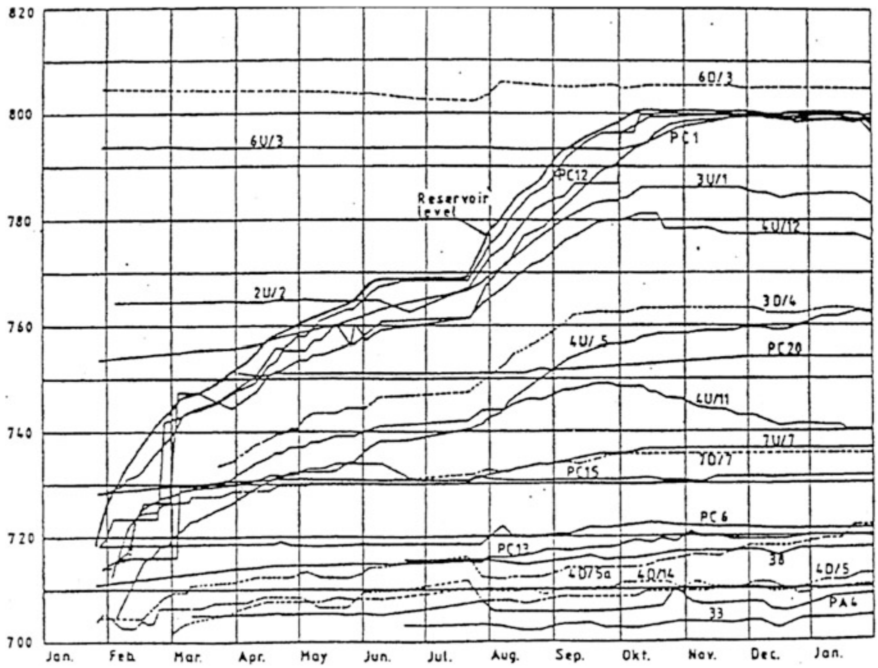



**Fig. 3.37** Pueblo Viejo Dam. Typical piezometers hydrographs during impoundment, U—piezometers towards upstream, D—piezometers towards downstream, PC—piezometers outside of dams and galleries

section of the valley and to several sections of the abutments. Two years later the situation had changed (Figs. 3.34, 3.38 and 3.39): Several upstream piezometers meanwhile indicated a much higher potential. This means that a built-up of uplift pressure took place. It occurred very slowly because of the very small differences in permeability between the grout curtain and the rock in front of the curtain.

- The hydraulic situation is especially complex in section below the spillway (Fig. 3.34). Directly beneath Gl-2, the upstream piezometers showed nearly the reservoir level while the downstream piezometers varied considerably. In some sections the grout curtain is effective, in others it is not. This, however, does not necessarily mean that there grouting was not required.
- The little seepage also confirmed the satisfactory performance of the foundation. In November 1983 the recorded water losses reached the total amount of 28 l/s. Two, years later, this had dropped to 24 l/s. The right abutment just delivers 1.0 l/s. the left one 23 l/s, in which the natural groundwater prevails.
- In the back sections of the access tunnels of both mountain-sides the groundwater table downstream of the curtain is higher than upstream. The normal flow direction of the groundwater and the bending of the curtain cause the paradoxical situation that the groundwater flows from downstream towards upstream

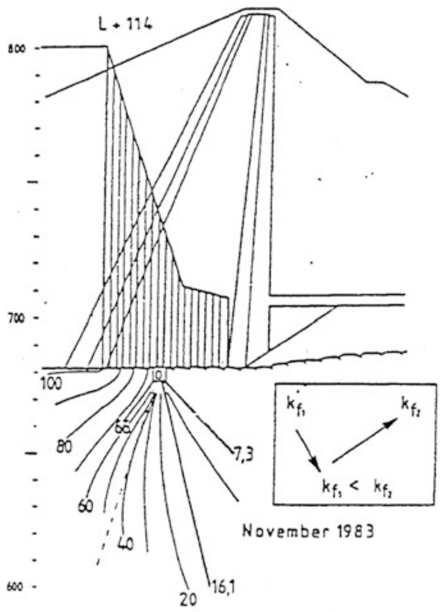



**Fig. 3.38** Pueblo Viejo Dam. Uplift reduction during fully impounded reservoir in November 1983 and December 1985, section indicates the situation within the middle of the valley

against the curtain. Seepage water from the reservoir and groundwater store each other along the curtain compensating the hydraulic gradient. The respective sections amount to 360 m on the left and 200 m on the right side (Figs. 3.34 and 3.36). Provided, the conditions for such a development principally recognizable were known beforehand, the curtain could have been shortened accordingly.

#### 3.8.2.5 Geological Setting of Voids and Possible Effect of Grouting on Uplift Reduction

Taking into consideration the nature of the outcropping rock, the state of the core samples, the results of WPT's, the grout takes and the form of the uplift reduction across the grout curtain (linear or over-proportional), this rock comprises various types which can be classified as follows (Fig. 3.40):

- A: Practically impervious, not groutable, linear uplift reduction because of hair cracks.
- B: Low permeability due to very fine ungroutable joints, linear uplift reduction.
- C: Distinct permeability due to general opening of joints, groutable, little to medium grout takes diminish permeability, thus over-proportional reduction of uplift.

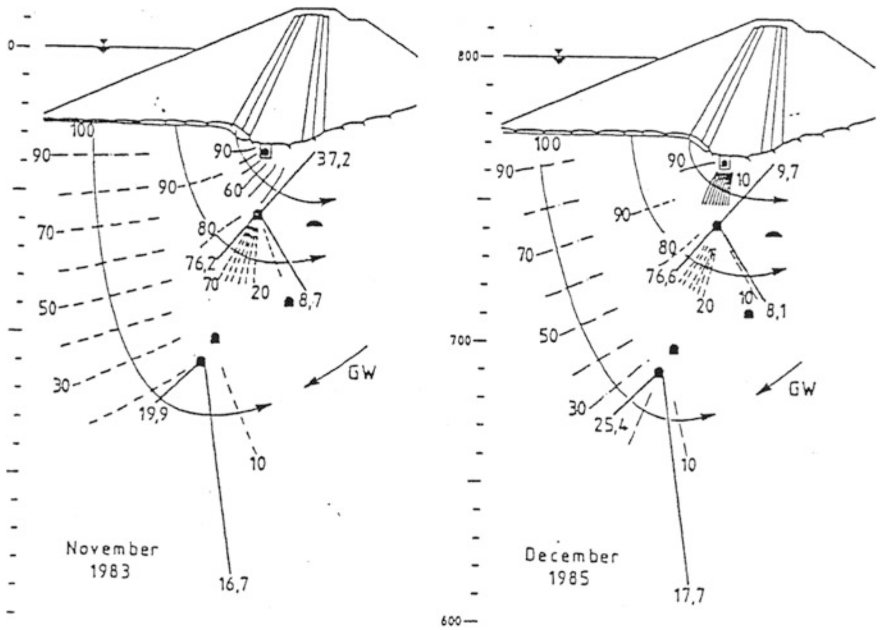



**Fig. 3.39** Pueblo Viejo Dam. Uplift reduction (given in %) during fully impounded reservoir in November 1983 and December 1985, section indicates the situation beneath the right abutment

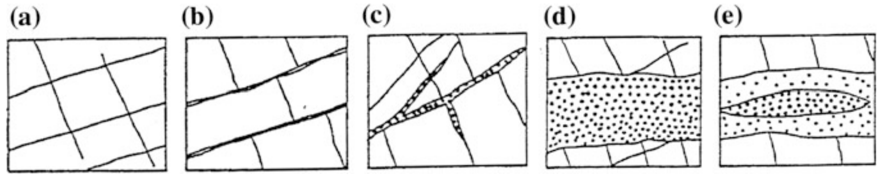



**Fig. 3.40** Pueblo Viejo Dam. Classification of rock regarding water routing, permeability, effectiveness of grout curtain altogether determining the form of uplift reduction

- D: Open karstic cavities, often very large dimensions. Unless cavities are too large, they can be filled by grouting which reduces karstification. Reduction of uplift depends on permeability conditions:
  (a) If the surrounding rock is impervious—over-proportional uplift reduction;
  (b) If the rock is permeable—linear uplift reduction.
  The amount of grout allows an estimate which of the cases apply. In the latter one, the linear uplift reduction does not mean that the treatment was unnecessary: Large cavities have to be sealed.

- E: Ash-filled cavities, often of very large dimensions, groutable; intercalated groutstone pillows may stabilize the ash but as it remains permeable, the uplift reduction occurs linearly.

#### 3.8.2.6 Conclusion

On the basis of (a) the grout takes, (b) the residual permeability determined by means of WPT's, and (c) both the form of uplift reduction and its temporal changes, it was possible to estimate what type of rock existed in each section. This approach is applicable also for other projects with similar conditions to be realized.

The analysis of the results underlines the requirement for a hydrogeological investigation of a large surrounding area. A net of piezometers should be installed and periodical recording should be carried out. The position of the groundwater table with all its possible deviations from the surface including the long-time changes due to the rainy seasons ought to be known because a much more appropriate concept for the underground treatment could be developed.

### *3.8.3 Panix Dam/Switzerland*

#### 3.8.3.1 Introduction

The concrete gravity dam at Panix (Switzerland) is founded on slightly karstic limestone. But also small karstic cavities may cause unacceptable seepage losses. A tight grouthole pattern was considered necessary to pinpoint all the voids. At Panix we were facing the condition that less extensive karstification requires more groutholes in order to detect them all. However, economic considerations restrict the number of groutholes—too close a grouthole spacing is not warranted everywhere and anymore.

The concrete gravity dam, a component of a hydropower scheme, 50 m in height, 240 in in length, was built in the Alps on limestone. Originally it was planned to carry out both consolidation and curtain grouting:

- Consolidation grouting of 10–15 m deep holes, set at spacing of 4 and 8 m, respectively, was designed to strengthen and to seal the rock of the upper seam.
- The curtain grouting was considered to tighten the rock down to 55 m in the section of the valley and down to 30 in both slopes; it should reach 65 m into left abutment and 130 in into the right, respectively.

#### 3.8.3.2 Hydrogeological Situation

The hydrogeological setting is of particular importance because karsticity appears in very different forms, and some of them are less difficult to deal with. A meticulous study discovered that such a setting existed at Panix. Thus, the scope of the grouting work could be reduced to a minimum—the grout curtain was abandoned and the consolidation grouting was replaced by contact grouting. Figure 3.41 shows (a) the deep groutholes carried out for investigation to complement the investigation confirming the knowledge concerning the permeability (Series A and K) and to treat the few slight intensely karstified areas (Series PIL and PIR) and (b) the shallow holes for contact grouting to seal the contact along the inspection gallery (Series G and H).

According to Fig. 3.42 two stratigraphic limestone units are involved. The lower one is forming the bottom of the valley and the right abutment and adjacent slope. It showed karstic features: Springs appeared group-wise at several sections at the toe of the slope; dye tests carried out in boreholes disclosed hydraulic connections between the reservoir area and several of these springs (Fig. 3.43). In contrast to this, extensive permeability testing by means of WPT's and the mostly excellent state of the core samples proved the rock practically tight, except in a shallow surface parallel seam with a few local slightly permeable spots.

It had to be inferred from all observations that the karstification of this limestone still remained in an early stage. While the rock itself is tight, only a very few karstic features have evolved—presumably channels of several millimetres up to a few centimetres in width, most of them filled with a cohesive loam. In spite of this comparatively favourable situation, both the state of the joints encountered in the right abutment-tunnel and the deep elevation of the lowermost springs gave reason to assume that some karstic openings could cross the dam site also far inside the abutment and at greater depths.

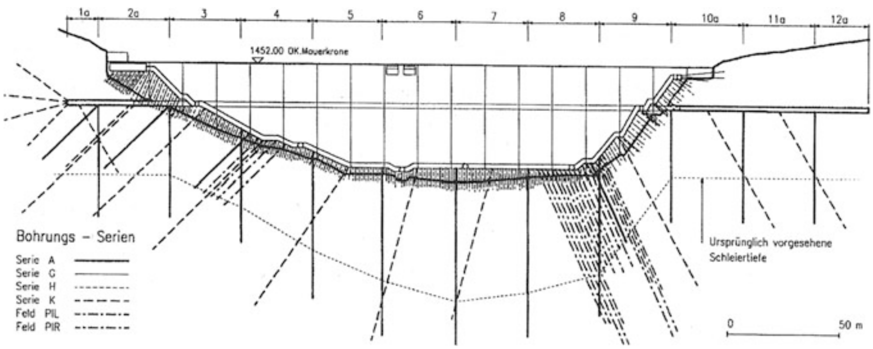



**Fig. 3.41** Panix Dam. Original area of grout and boreholes curtain eventually carried out: Series A and K—drilled for investigation, Series G and H—contact grouting along the inspection gallery, PIL and PIR—local test grouting on the left and right slopes, respectively

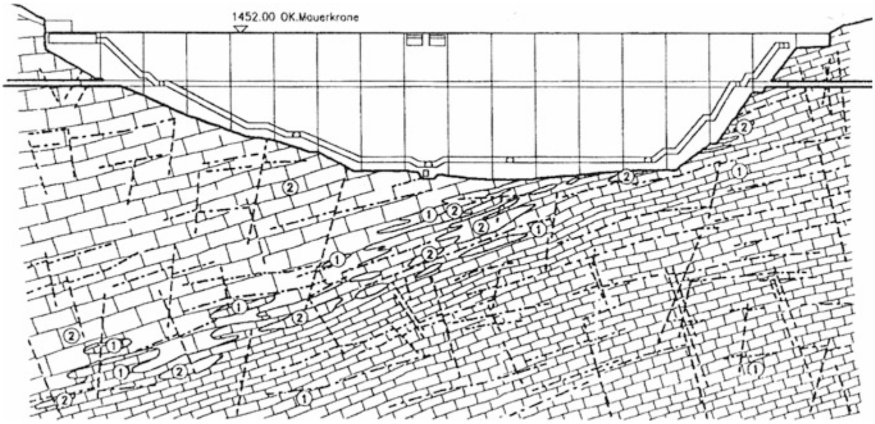



**Fig. 3.42** Panix Dam. Stratigraphical units („Quintnerkalk“, „Korallenkalk“)

The dam site is located directly upstream of a gorge. This and the abruptly falling valley provide for a steep gradient. This produced an unfavourable hydraulic situation as the relief is located in a comparatively deep position. Thus, the originally designed rather shallow grout curtain would have not substantially lengthened the flow lines through the foundation. A grout curtain, if considered necessary, had to reach much deeper and had to cover a much larger area also into the abutments. Moreover, in view of the rareness and the small size of the karst channels, for the groutholes a very close spacing (order: 0.5 m) would have been required while the portion of unproductive grouthole meters easily exceeded 98%—economically, leaving a very small portion of he productive holes.

### 3.8.3.3 Concept

In dam construction, the safety always has to be the decisive factor, particularly in case of a karstic underground. However, it is not reasonable either always to schematically treat the rock beforehand once it appears to be karstic—as frequently done. When relevant information seems to indicate that the karstification did not produce already large and extended cavities but just a few small voids, then it is required to carefully analyse all components of the geological situation. At Panix, such an examination justified a considerable modification. The eventually executed drillings are shown in Fig. 3.41; the treatment comprised the following components:

- The consolidation grouting was abandoned because the grout takes were presumed not to reach substantial amounts, say $\geq 50$ kg/m. It could not be expected that such takes together with the wide spacing of the groutholes could improve both the imperviousness and the strength which originally was already

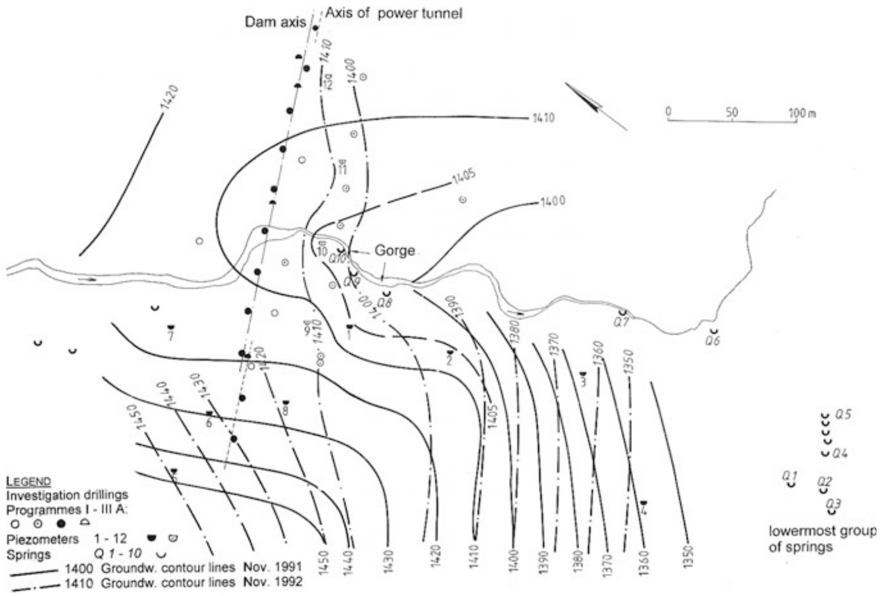



**Fig. 3.43** Panix Dam. Groundwater contour lines—before and after impoundment (1991/1992), location of drill-holes for investigation—Programs 1-III.A, piezometers 1–12, springs (Q1–Q10)

high enough not to produce sizeable settlements. Therefore, this consolidation grouting was replaced by contact grouting.

- The contact grouting was aimed at sealing the constructional joint between rock and concrete along the foundation of the darn and the uppermost portion of the rock, possibly affected by the excavation. This grouting work was carried out along one row and in two subsequent series with a final spacing of 1.0 m, except in a few short sections with a 0.5 m spacing. The groutholes were drilled from the gallery through the contact joint and, at first, 1 m into the rock. This first stage of the holes was grouted with a packer fastened inside the concrete (Stage 1a). Later the holes were re-drilled and deepened down to 3 m below the contact. This second stage, 2 m in depth, was grouted with the packer fastened at 1 m below the contact (Stage 2). Finally, the first stage was grouted again in order to control the success of the first grouting (Stage lb). The results demonstrate a successful progressive sealing. The order of the takes and the successive decrease from the first to the second—and section-wise third—series (G, H and I, respectively) apply to all stages:

| | | | |
|---|---|---|---|
| Stage la | G−411 | H−225 | I−36kg/in |
| Stage 2 | G−138 | H−I07 | I−41kg/in |
| Stage lb | G−<10 | H−<10 | I−<10kg/m |

The rather high takes of the first stage indicate considerable voids placed along the contact and required to be sealed; medium–sized takes of the second stage reflect narrow and short joints, partly as a result of the loosening due to excavation.
The curtain grouting was abandoned. In view of the large area strictly to be covered, the extremely uneconomical portion of unproductive grouthole meters and the low degree of karstification, it appeared to be prudent not to carry out a schematic grouting.

#### 3.8.3.4 Further Control Drilling, Test Grouting and Final Results

In order to obtain still more information about the permeability and the type of the karstification further check holes were drilled and tested (‘K-holes’); in addition, test grouting was carried out in the middle of the left slope and around the toe of the right slope (‘PlL-holes’ and ‘PlR-holes’, respectively). The check holes and the test grouting on the left slope fully confirmed the idea of a rock which is practically impervious, except a shallow surface seam of a slightly permeable rock. The test grouting at the right slope was placed right at the toe because this section was not too far from the springs mentioned above. This test grouting in some grouthole stages yielded WPT-results between 30 and 40 LU and absorbed substantial amounts of grout (maximum—940 kg/m), i.e. the holes encountered and grouted karstic features. The 17 holes of this PIR-test grouting have been grouted in three series; the decrease of the takes confirms the progressive sealing:

| | | | |
|---|---|---|---|
| Grouting | PIR/I−204 kg/in | PIR/II−89 kg/in | PIR/III−38 kg/m; |
| WPT | PIR/I−2.5 LU | PIR/II−0.38 LU | PIR/III−0.19 LU |

This concept of replacing the consolidation grouting by contact grouting and of abandoning the curtain grouting also comprised the installation of an improved piezometer system aimed at recognizing as soon and as near as possible any particular leakage and under-proportional decrease of the uplift. If such leakages had happened, they would have been treated locally. The results demonstrate that the modified concept was appropriate:

- The seepage losses do not reach a sizeable order of magnitude (<1.0 l/s).
- The piezometers placed directly downstream of the dam (P8–P12) indicate very little reaction; the piezometer P13 is located upstream of the dam at the right slope (Fig. 3.44).

The piezometers installed along the gallery inside the dam registered very little reaction because the contact between dam and rock of foundation was successfully sealed (Fig. 3.45).

The quantities of water flowing out of the springs depends on the precipitation rather than on the reservoir (Fig. 3.46).

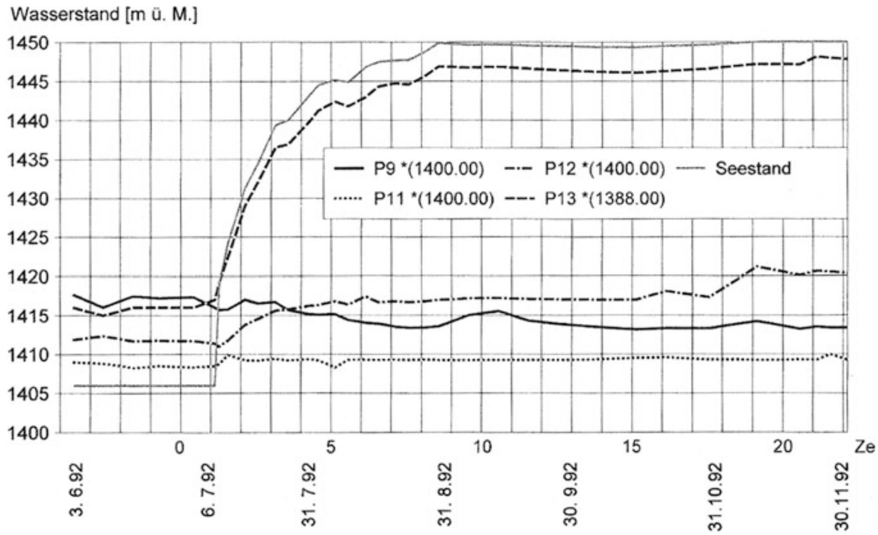



**Fig. 3.44** Panix Dam. Hydrographs of representative piezometers: P9, P11, P12 downstream of the dam, P13 upstream of the dam

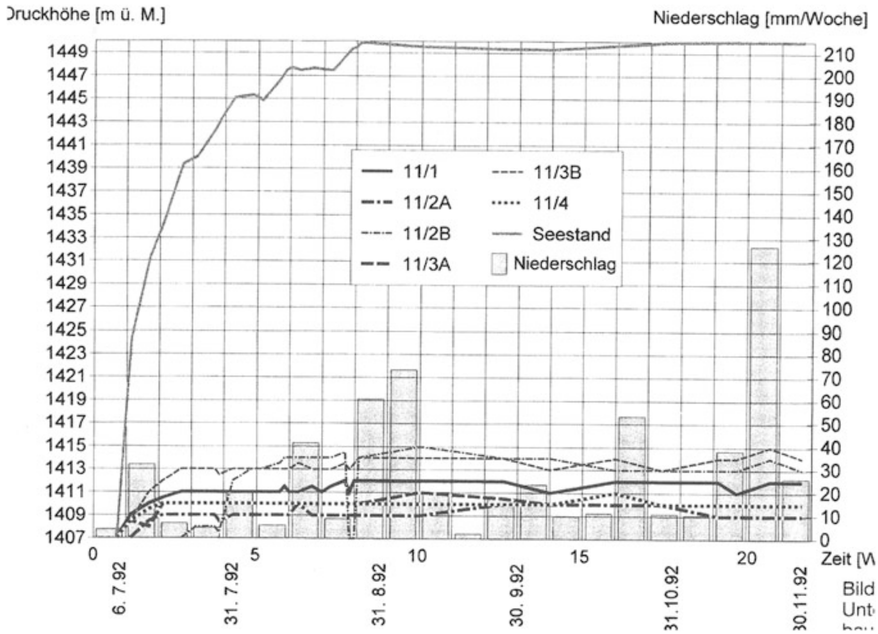



**Fig. 3.45** Panix Dam. Hydraulic behaviour inside the dam—very little reaction due to rain

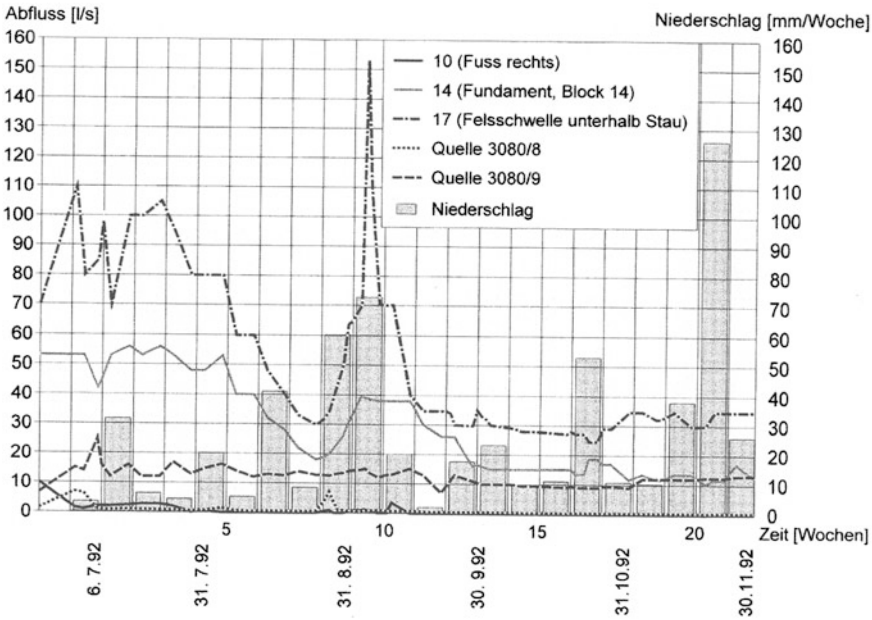



**Fig. 3.46** Panix Dam. Quantities of water out of springs depend on rain falls

## *3.8.4 Mujib Dam*

### 3.8.4.1 Introduction

The 67 m high Mujib Dam (Jordan) was built between 1999 and 2003. The underground of the valley floor and the moderately inclined slopes consist of various types of limestone.

Limestone is a soluble rock, and karstification in this rock is ubiquitous. Karstified limestone always is a strong engineering geological challenge necessitating precise geological understanding of the reason(s) and the type of karstification.

In an early phase of the project the rock mass appeared to be rather impervious. The permeability tests yielded low Lugeon values and the grout takes were small. These results indicated a tight rock, practically without substantial karstification. Nevertheless, since karstic features could occur, a grout curtain was installed. Figure 3.47 shows the layout of the curtain at the right hand side.

Until autumn 2004 the reservoir was filled-up just to 38 m above foundation level and 177 m.a.s.l. due to the dry summer season. No noticeable seepage was observed, i.e. the presumed imperviousness of the bedrock seemed to be confirmed. However, in a night of heavy rainfalls, filling the reservoir to the top (194 m.a.s.l.), sudden seepage developed, surging at the right dam toe and slope until the morning

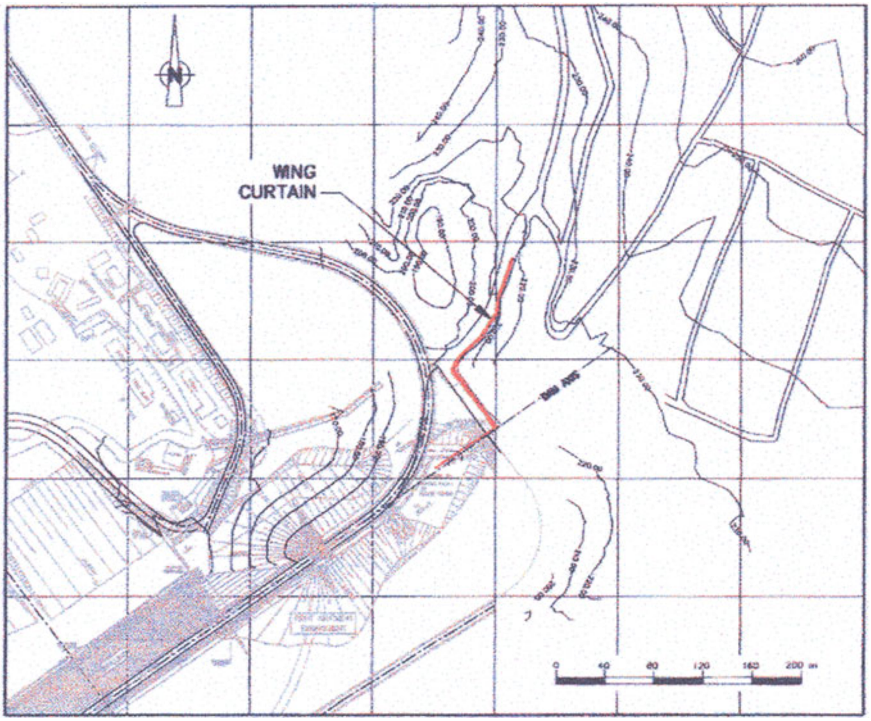



**Fig. 3.47** Mujib Dam. Layout of the grout curtain at the right abutment; in *red*: Wing Curtain

hours with already approximately 140 l/s. It increased during the following 2 weeks up to 240 l/s and then remained about constant. Both, the prompt beginning of the seepage and its subsequent increase suggested a twofold cause:

- The original grout curtain did not reach far enough into the right abutment, hereby allowing to bypass the curtain while the left hand side remained tight.
- The subsequent increase of the seepage indicated erosion of infillings out of formerly plugged karst channels crossing the grout curtain (Fig. 3.48).

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

- The rule of the thumb recommends that a grout curtain inside the abutment should reach up to the point where the rising groundwater table reaches the reservoir level. This rule is not applicable when the position of the ground water table is unknown and, hence, the length of the curtain is determined by estimate. Too long an extension implies overdoing; by contrast, if the curtain is too short seepage will bypass it. If the latter case happens, the curtain has to be extended. It depends on the whole setting whether the first or the second approach is preferable. The conditions at this site favoured the second one. The required length of the additional section could be tested out since the reservoir was

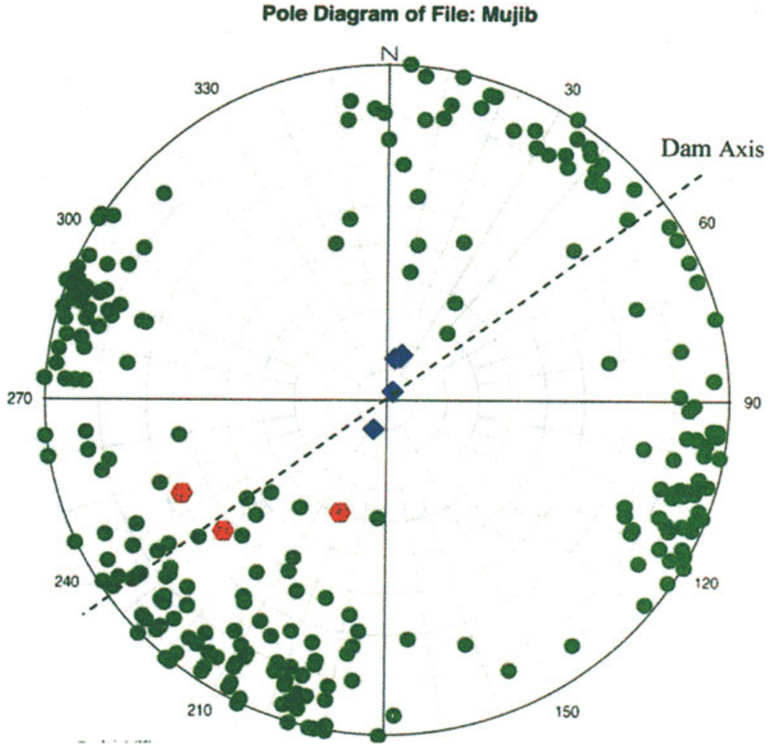



**Fig. 3.48** Mujib Dam. Density of joints; *bold line* dam axis: *green* joints, *blue* beddings, *red* faults

impounded. Thus, the lengthening of the curtain yielded an optimal solution—no longer than required—thus, it was really appropriate.

- Those karst channels filled with debris existing inside the Phase 1 grout-curtain were not groutable and remained ungrouted. First the higher pressure of the fully impounded reservoir was capable of removing the erodible fillings, hereby opening the channels for further grouting. Supposing the existence of filled karst channels was known beforehand, it would have been expedient; to postpone the grouting work until the fillings would have been washed out. However, this is only a theoretical consideration because nobody would have agreed to a proposal to impound first and grout later.

The following presentation of the testing and grouting results prove the appropriateness of this diagnosis and both grouting campaigns. This paper focuses in particular on the groutability of the karst channels: The fillings of debris made them impervious but as the increasing water pressure during the sudden filling of the reservoir exceeded the tolerable hydraulic gradient of these saturated materials regressive erosion washed them away facilitating grouting. No turbid water was observed surging from the newly developed '*springs*'. This suggests flushing of sandier material from karstic joints where the washed-out material must not necessarily have reached the open air but could have filled open cavities on its way downstream.

Besides the grout curtain installed along the gallery and adjacent sections, Fig. 3.47 shows also the additional sections grouted in Phase 2—the WING-Curtain.

#### 3.8.4.2 Phase 1 Grouting

*Consolidation Grouting*

Consolidation grouting is aimed to seal joints and artificial fractures due to excavation in the surface-near reach. This was done all along the dam foundation prior to the placement of the embankment of the dam. Vertical groutholes have been drilled from the surface of the rock along 2 or 3 lines at each side of the grouting gallery with a distance of 3 m in between the 21 successive sections. The lowermost section is located at El. 160 m.a.s.l., the uppermost at 217 m.a.s.l. Grouting was carried out in three series, The P- and S-holes are 5 m in depth, T-holes 3 m, respectively.

About 86% of all the holes had grout takes less than 20 kg/m, 12.2% of the holes had grout takes >20 <100 kg/m while two holes in sections at El. 172 and 217 m.a.s.l. absorbed 1883 and 860.2 kg/m, proving again the high irregularity of karstified limestone. These very large grout takes revealed for the first time that karstic voids exist below the maximum storage level. Since the final pressures reached only 3 bar—i.e. hydrofracturing was not involved—karstic voids had been filled.

These 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 reached 1.5 m below the contact into the foundation and were placed along two lines in the U/S and D/S-walls of the gallery. At the upstream side 46 holes were drilled, actually every 1.5 m, the downstream side received only 33 holes because occasionally the spacing was enlarged to 3 m. The final grouting pressure reached 3 bar.

The main results are as follows:

- The average grout takes were 23.5 kg/m for the U/S-holes and 29.9 kg/m for the D/S-holes.
- The maximum grout takes reached 437.9 kg/m for the U/S-holes and 446.9 kg/m for the D/S-holes.

- 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 borehole was not deducted, while in Phase 2 the given quantities are net quantities, hole-fillings subtracted.
- 80.5% of the U/S-holes and 83.3% of the D/S-holes took <20 kg/m.
- 6.5% of the U/S-holes and 12.9% of the D/S-holes took >50 <200 kg/m.
- 2.2% of the U/S-holes and 3.2% of the D/S-holes took >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 done through inclined boreholes, as mapping revealed a predominance of near to vertical joints. Holes were drilled from the 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 drilled the space splitting way with the depth according to the findings and with the maximum depth of 50 m.

Water pressure tests (WPT) have been carried out in P-, T-, Q- and CH-holes, not in S-holes. The WPT-results are given as $Q_{10}'$- and $Q_{10}*$-values (Fig. 3.49), where $Q_{10}'$ gives the original permeability and $Q_{10}*$, the larger one due to erosion or hydrofracturing. The statistical evaluation of the results shown in Figs. 3.50 and 3.51 reveals the permeability, groutability and success as follows:

- About 80% of the rock were originally almost tight—$Q_{WPT} \leq 1$ LU[1] while approx. 20% absorbed some water and were also groutable.
- The permeable and groutable voids existed down to a depth of approx. 30 m (Fig. 3.51).
- After the T-series only one significant test section of permeability was found, indicating a good seal had been achieved.
- The differences between $Q_{10}'$- and $Q_{10}*$-values are small, which indicates a low deformability and erodibility of the rock mass under the applied pressures.
- 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 partly could be responsible for this fact by hydrofracturing. On the other hand, high quantities, as observed during Phase 2 grouting always happened pressure-less. The high pressure in Phase 1 only would be test pressures after the real grout take, not really the grouting pressure.

[1] 1 LU = 1 l/min × m × 10 bar.

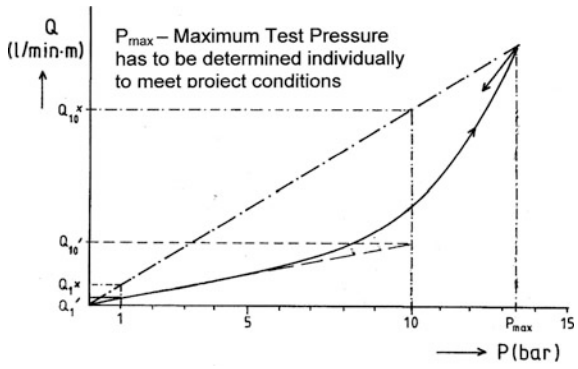



**Fig. 3.49** Definition of WPT parameters $Q_{10}'$ (original permeability) and $Q_{10}^{*}$ (provoked permeability)

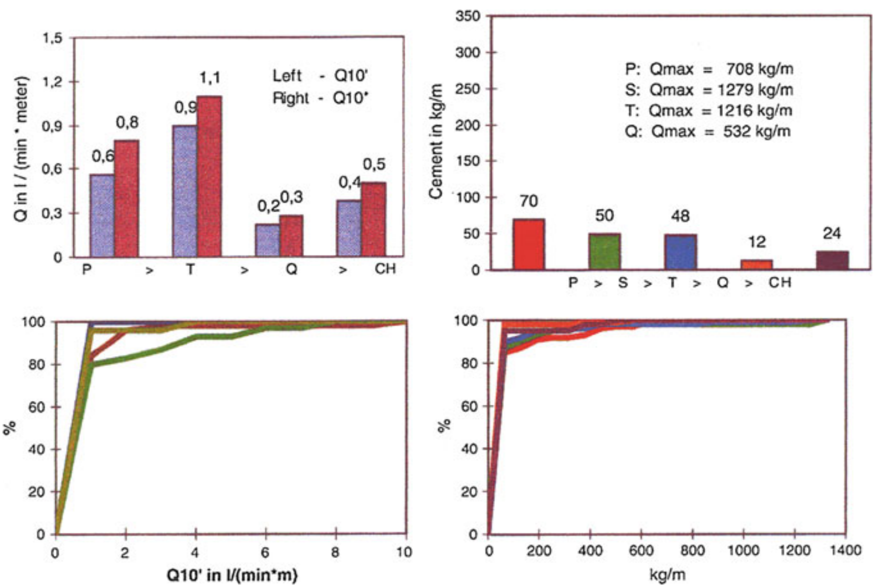



**Fig. 3.50** Mujib Dam. Mean values and frequency distribution of WPT-results (*left*) and grout takes (*right*) obtained in Section GG of Phase 1

The grouting work has been carried out in four subsequent series: P, S, T and Q; in some places check holes (CH) have been executed. Their first stages began between 2 and 5 m where grouting pressures were applied between 4 and 8 bar. The holes ended between 30 and 50 m and the final grouting pressures ranged between 12 and 15 bar. The number of holes and number of grouthole stages are in series P—11 holes with 75 stages, S—12 holes with 70 stages, T—22 holes with 120 stages, Q—24 holes with 122 stages and CH—8 holes with 42 stages.

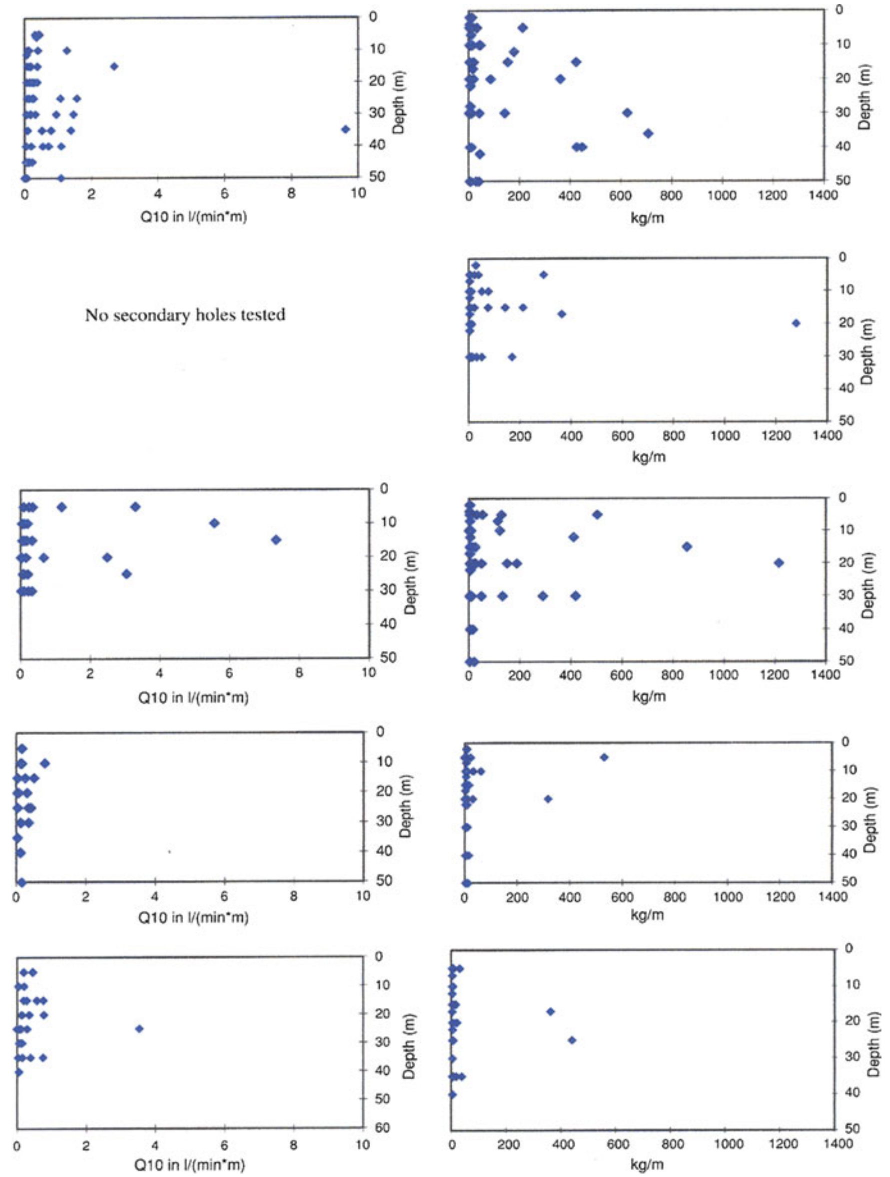



**Fig. 3.51** Depth of WPT-results (*left*) and grout takes (*right*), obtained in Series P, S, T, Q and CH (from *top* to *bottom*) in Series GG of Phase 1

The maximum grout takes reached 708 kg/m in P-holes, 1279 kg/m in S-holes, 1216 kg/m in T-holes and 532 kg/m in Q-holes. Permeable voids absorbing sizeable amounts of grout were encountered down do a depth of 30 m. About 80% of the grout stages were within (almost) impervious rock, while only the rest hit permeable and groutable voids. This difference illustrated in Figs. 3.50 and 3.51 is typical for karstic limestone and is discussed later on together with the results of Phase 2 Grouting. Fan grouting into the right abutment did not render noteworthy peculiarities.

#### 3.8.4.3 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 in Phase 1. However, it soon it became apparent that large 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. This fact gave reason to a new regular grouting system in three subsequent series—P, S, and T. The holes were inclined as in Phase 1. Grouting was done in downward steps, ending at 30 m in series P and S, and at 50 m in series T.

The top stages were grouted with pressures between 3.3 and 6.3 bar, the bottom stages with pressures between 12 and 14.2 bar, where in nearly all of the stages the grout entered at low pressures. 27 P-holes with 123 stages, 12 S-holes with 43 stages and 28 T-holes with 80 stages were drilled and grouted. The maximum grout takes reached 12,792 kg/m in P-holes, 3467 kg/m in S-holes, and 162 kg/m in T-holes. Q- and CH-holes have not been drilled. The results of the statistical evaluation are graphically presented in Figs. 3.52 and 3.53. In Phase 2, all the

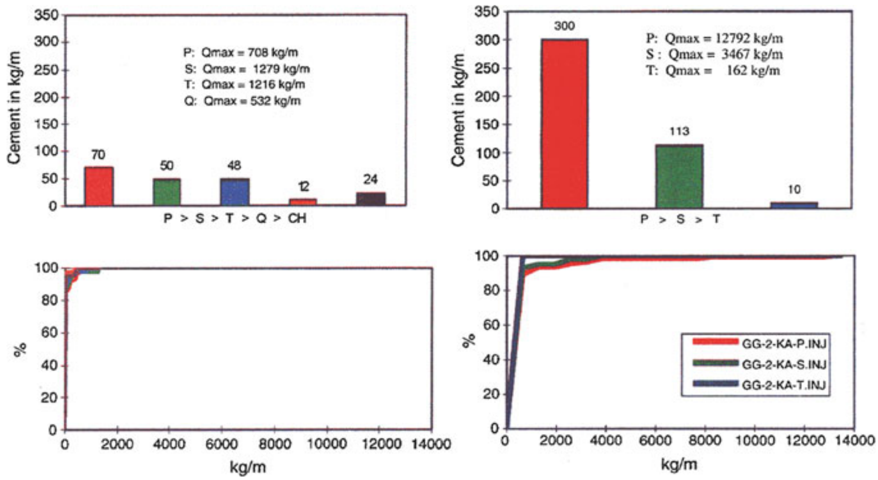



**Fig. 3.52** Mujib Dam. Mean values and frequency distribution of grout takes obtained in Section GG of Phase 1 (*left*) and Phase 2 (*right*)

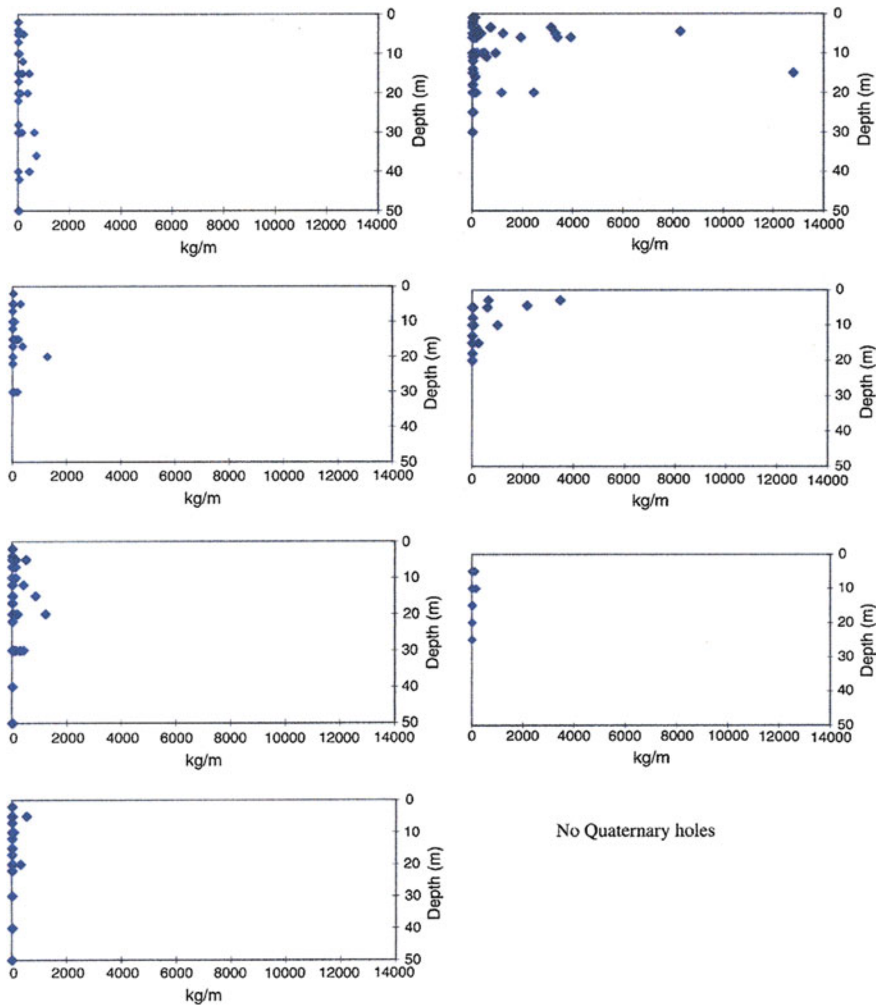



**Fig. 3.53** Mujib Dam. Depth of grout takes in Series P, S, T and Q (from *top* to *bottom*) in Section GG of Phase 1 (*left*) and Phase 2 (*right*)

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 a section of open air till the fan of Phase 1, then in a somewhat odd configuration along the cut slope in D/S direction and further in a right bend uphill. This configuration was necessary in view of the

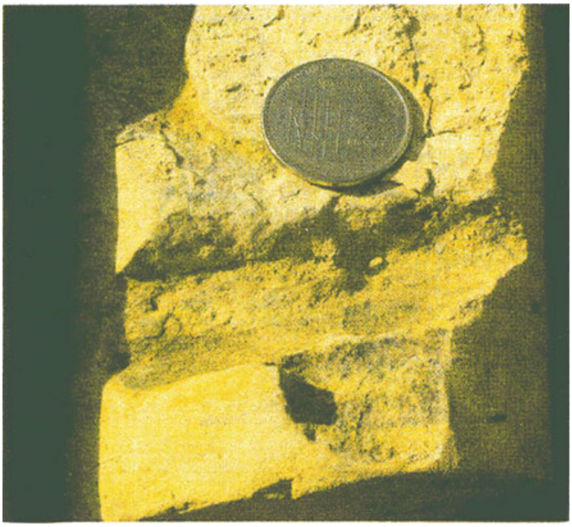


**Fig. 3.54** Mujib Dam. Linear karst channel in NX-core, Wing section, right abutment

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 meters in terrace material which would have been extremely difficult to stabilize (Fig. 3.47).

This section encountered voids which had either originally been open or had been cleared of debris during the sudden filling of the reservoir. Figure 3.54 shows a linear karst channel. Such a feature can only be grouted when directly encountered or when intersecting with a fissure crossed by another drill hole.

Both wing sections had the same typical groutability of karstic limestone which is very well expressed by the graphical presentation of their results (Figs. 3.55 and 3.56):

- The majority of the grout holes encountered tight rock—about 60% in Wing-Section 1 and 75% in Wing-Section 2. Vice versa, groutable rock was encountered in about 40% in Wing-Section 1 and 25% in Wing-Section 2.
- 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.
- This contrast applies in particular to Wing-Section 2 where the rock had evidently longer and wider karstic joints which took even huge quantities. Not all of them were filled with the primary holes already: the depth distribution reveals that a few single ones remained open and were still grouted with the secondary and tertiary holes.
- The depth distribution indicates that the permeable and groutable voids were restricted to the upper 35 m below ground surface at elevation 165 to 180 m.a.s.l.
- The significant grout takes filled really open voids and did not originate from fracturing of latent discontinuities because they took place at rather low pressures.

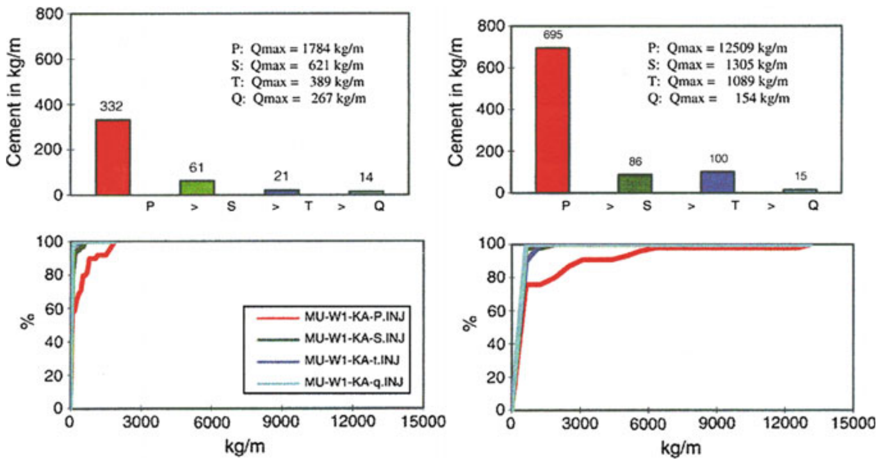



**Fig. 3.55** Mujib Dam. Mean values and frequency distribution of grout takes in Section W1 (*left*) and W2 (*right*), Phase 2

*Comparison of Phases 1 and 2 Grouting Carried Out from the Gallery*

The results of GG1 and GG2 are compared with each other in Figs. 3.52 and 3.53. The comparison demonstrates that the groutability of the rock changed drastically between these two phases:

- The large grout takes during Phase 2 are below 177 m.a.s.l. which was already under reservoir pressure before the sudden filling of the reservoir.
- The large individual grout takes indicate sizable voids, extending to depths of approx. 25 m.
- The average grout takes in Series P have quadrupled—from 69.3 kg/m in Series 1 to 300.3 kg/m in Phase 2.
- As a result, for 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;
  10.1 kg/m = 3.4% in Series T.

Phase 1 encountered 20% of groutable stages, well corresponding with the WPT-results of the Q- and CH-holes: The mostly narrow and only occasionally a

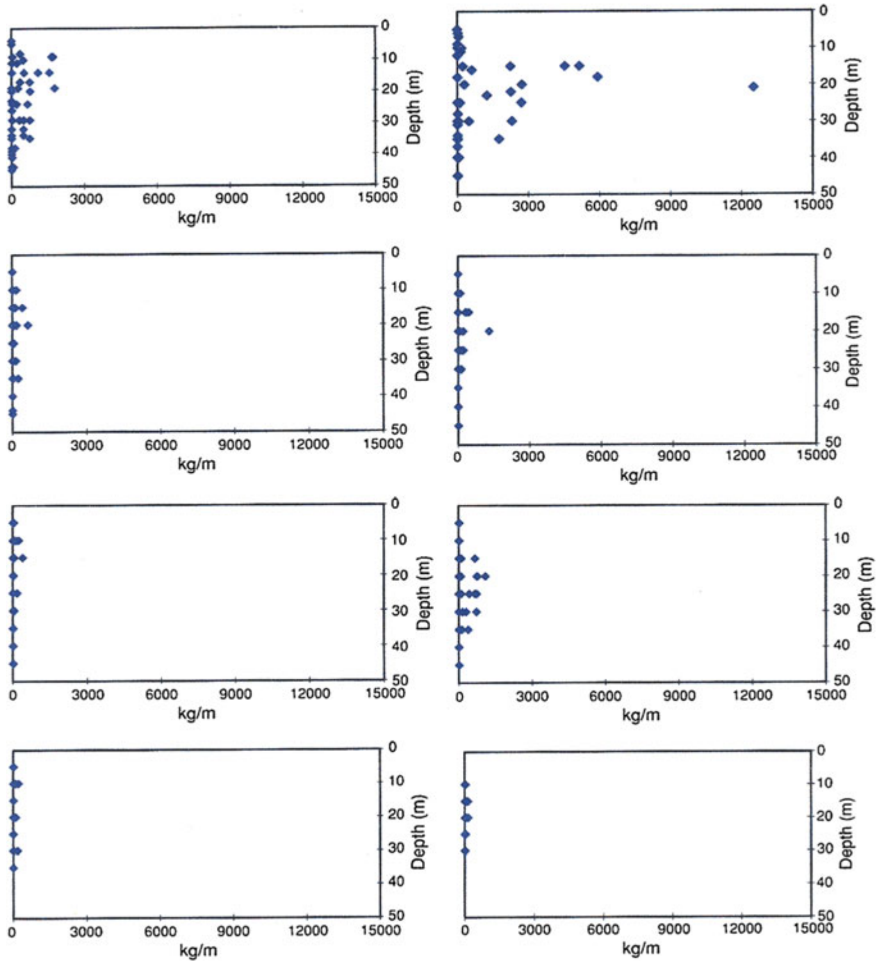



**Fig. 3.56** Mujib Dam. Depth of grout takes in Series P, S, T and Q (from *top* to *bottom*), Section W1 (*left*) and W2 (*right*), Phase 2

little wider voids were successfully sealed. By contrast, Phase 2 found only 10% of groutable voids which, however, absorbed even huge quantities. They became accessible due to the erosion during the sudden filling of the reservoir ensuing higher hydrostatic pressure. That process is schematically illustrated in Fig. 3.57.

Holes PL 534 and Q 534,75 allow a convincing comparison which confirm how the processes occurred in Phases 1 and 2 (Fig. 3.58): With a distance of 0.75 m in between, the holes are directly neighboring. The two sections absorbing 625 and 426 kg/m in Phase 1 were tight in Phase 2. Vice versa: the sections taking 4.8 and 4.2 kg/m in Phase 1 could absorb even huge amounts in Phase 2 because now the

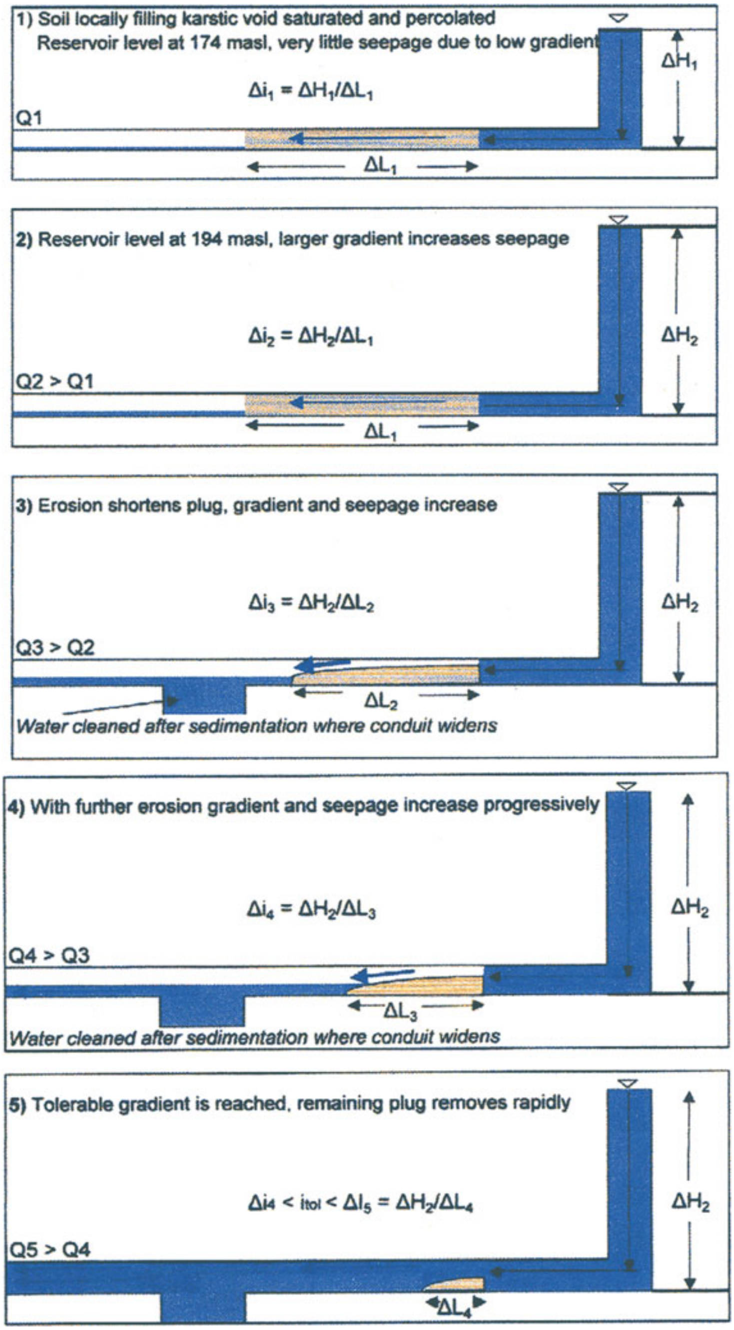



**Fig. 3.57** Mujib Dam. Erosion of fillings from plugged karst channels at higher hydraulic gradient

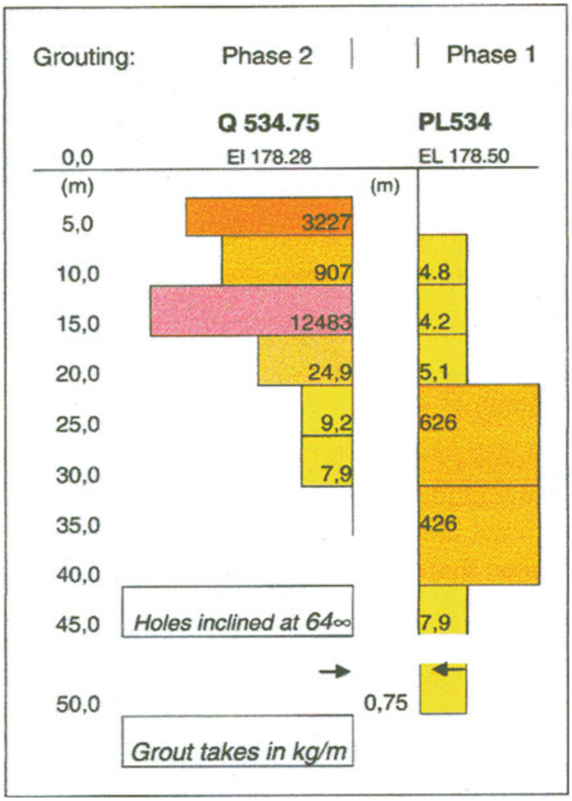



**Fig. 3.58** Mujib Dam. Originally plugged channels absorbed large grout quantities in adjacent hole (75 cm distance) after erosion of infillings

karst channels intersecting the holes were groutable since the fillings had been eroded which formerly clogged their entrances.

In detail we have to envisage the following process: A karstic fissure is completely filled up to a certain length with soil. The apparent or real cohesion of this material is strong enough to withstand the low pressure caused by the partially filled reservoir but not strong enough any more if the reservoir level rises. Its enlarged pressure reduces the cohesion and grain by grain is pushed out until the complete plug is removed. Depending on the type of the soil and the length of this section this may take minutes or hours or months.

*Insufficient Grouting Pressure?*

It was questioned whether the apparently insufficient grout takes of Phase 1 were possibly caused by too low a grouting pressure. First of all, it is to remembered that different processes occur:

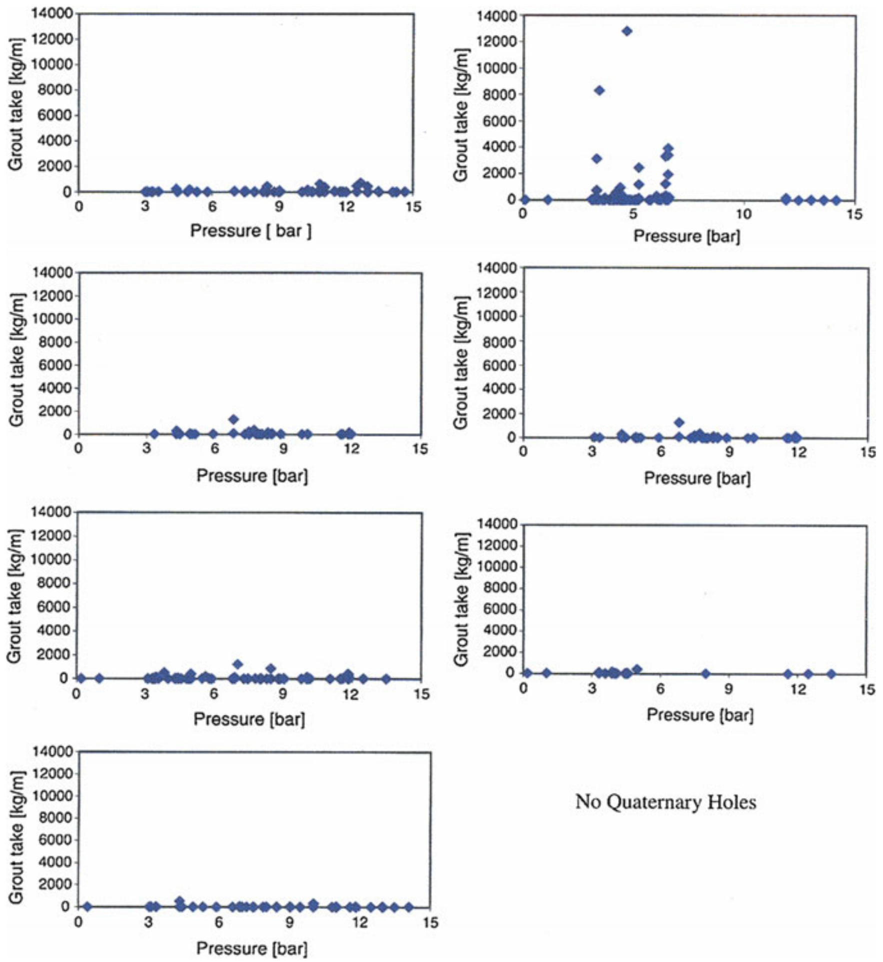



**Fig. 3.59** Mujib Dam. Relationship between grouting pressure and grout takes in Series P, S, T and Q (from *top* to *bottom*) in Section GG. Phases 1 (*left*) and (*right*)

- Grouting of real voids begins at low pressures which increase progressively with the gradual filling. The process is completed when the grout take stops since the pressure suddenly rises faster.
- If latent discontinuities intersect the hole instead of open voids, the grouting pressure rises and grouting begins after the pressure became high enough to fracture the plane.

The facts displayed in Figs. 3.59 and 3.60 disprove that suspicion:

- The large grout takes in Phase 1 took place at small to moderate pressures—$P_{max} \geq 3 \leq 13$ bar.

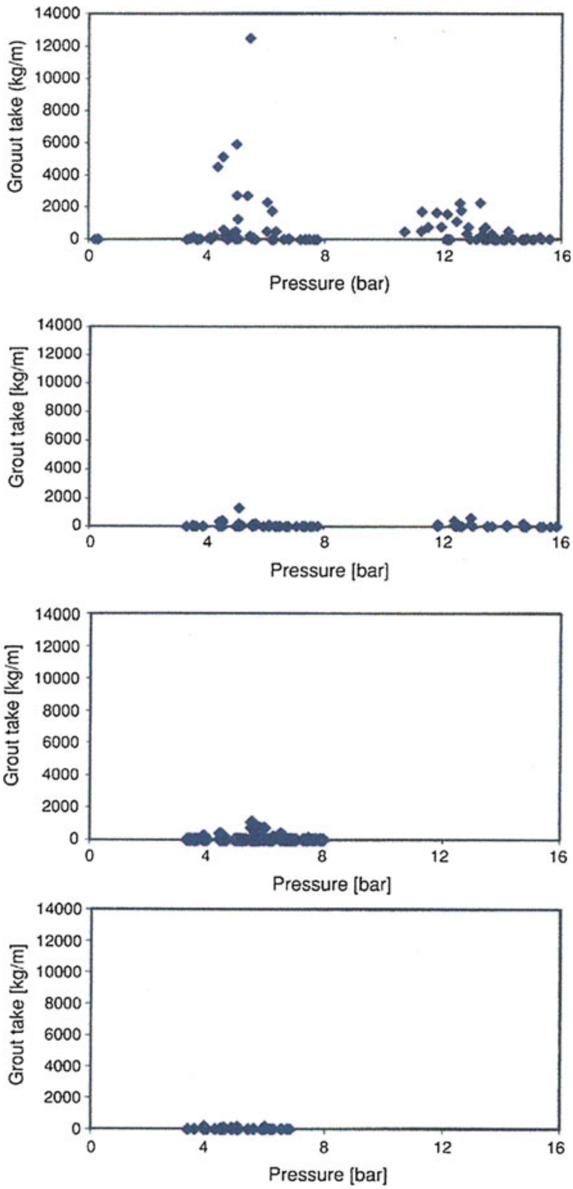



**Fig. 3.60** Mujib Dam. Relationship between grouting pressure and grout takes in Series P, S, T and Q (from *top* to *bottom*) in Sections W1 and W2 together, Phase 2

- The very large grout takes in Phase 2 took place at small pressures—$P_{max} \geq 3 \leq 6$ bar.

All these grout takes were caused by filling of existing voids. No grout was absorbed any more at the higher pressures applied—$\geq 13$ bar—which proves that the grouting pressures were actually high enough. The rather small pressures needed in Phase 2 really confirm that the erosion of the infillings out of the formerly clogged karst channels created open voids which caused the much larger grout takes.

The groutability implies a further aspect: Short and narrow joints absorb little quantities of grout. Successive series yield small mean values with little reduction from P- to Q-holes, as obtained with Phase 1 grouting. Phase 2 grouting encountered extended and open voids which caused much larger grout takes in the P-holes combined with a considerable reduction of the subsequent series. The difference between Phase 1 and Phase 2 illustrates the decisive importance of the geometry of the water conducting paths on the grouting results and the scope of the grouting work: wider, longer and intersecting joints (or karst channels) allow a wider spacing of the holes and less series.

#### 3.8.4.4 Final Considerations

A possible karstification makes limestone a hazardous foundation. At first glance that seemed not to be a particular issue for the Mujib-Dam, because during consolidation and contact grouting only rather small karstic features were encountered and successfully filled. The first curtain grouting confirmed these results because the grouting requirements seemed to be low and the first-partial-impoundment produced very little seepage losses. These encouraging results were hasty because the stronger hydraulic gradient caused by the full reservoir was able to initiate the erosion of the semi-pervious but un-groutable fillings deposited long ago in karstic channels. The channels became permeable and groutable and could be filled with the second grouting campaign. Its positive result was not at all predictable and it could not be taken for granted that this work would succeed as it really did. All the more one can be satisfied with the final result of the entire treatment.

The wing sections lengthen the grout curtain into the right abutment. The permeable and groutable zones along these sections reach down to a depth of 35 m below ground surface at El 165–180 m.a.s.l. That zone was partly not under water during the first partial impoundment reaching up to El. 177 m.a.s.l. Some of their potentially water conducting fissures became first inundated with the complete filling. The leakage began simultaneously with that complete filling because it was already observed at the following morning. Thus, at least some of the voids were open and groutable beforehand.

The first leakage—about 125 l/s—began already during the night of the complete filling. During the following 14 days the amount of the leakage doubled. The first portion flowed through the already open voids of the wing sections. The second portion seeped through the increasingly eroded karst channels below the right slope of the dam—possibly also through some eroded voids inside the wing sections.

#### 3.8.4.5 Lessons Learned

Karstification produces different types of Karsticity which here applies to both abutments. 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 next pressure step causes hydraulic fracturing of latent planes, or erodes debris out of the fissure. Thus, as different processes yield similar results, the problem of possibly existing 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 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 karst fissure could occur even after some 50 years. Are such developments recognizable early enough? At least to some extent: If erosion of infillings occurs, the hydrostatic situation along the dam axis may also change. Continuous recording of the uplift pressure enables to recognize such changes. Details are dealt with in the following chapter.

### 3.8.5 Conclusions for Dam Sites in Karstic Limestone

Grouting of karstic limestone for hydro projects gained growing importance. This impermeabilization is comparatively expensive and time-consuming. Therefore, it is expedient to do everything possible which can help to keep the expenses still in an acceptable scope. The projects presented allow to draw the following conclusions.

The size and purpose of the project in relation to the geological situation form an important aspect. A distinct or heavy karstification requires a large-scale treatment which would not be warranted for small projects which, vice versa, with an untreated foundation only permit a temporary impoundment. Dams for other purposes, necessitating a permanent filling, can be built only when the great expenses for the treatment are shared with a large storage volume, otherwise the treatment causes too a high a burden on the unit price per cubic meter of the storage volume.

In karstic limestone the groutholes primarily serve to detect the voids. For a limestone of an initial karstification, very few small openings, the use of a schematic treatment paradoxically leads to the situation that the expenditure becomes more useless the higher it should be: More holes are required but less voids are met; the portion of the productive drill meters decreases accordingly. The least karstified limestone requires the closest grouthole spacing possible, otherwise the very few voids remain undiscovered. An exaggeration makes the methodological and economical dilemma evident: A limestone without voids would need a continuous perforation by means of groutholes to ascertain that no voids exist! Of course, the concept of making the grouthole pattern closer and closer ends to be reasonable at a certain spacing. Thus the question arises up to which closest spacing the groutholes have to be condensed in order to meet all substantial karst channels. Vice versa: how many channels and voids of which sizes can be missed?

These questions seem to support the concept of a tight spacing (>0.5 m). However, even then there is no guarantee to meet all voids. Instead, it is considered prudent not to apply to close a spacing of the groutholes ($\geq 1.0$ m) but to complete this grout curtain by a rather tight net of piezometers, also towards upstream. It can be objected, of course, that it is also warranted to choose an even closer spacing—or the opposite. In principle, either one can hardly be contradicted. However, this objection addresses the heart of the dilemma of grouting karstic limestone. For the unavoidable lack of specific knowledge, nobody knows the result beforehand.

Provided, the karstification is still in an early stage and the hydrogeological situation is favourable, grouting work eventually can be reduced. Possible leakage spots can be treated specifically later on. In many cases a substantial part of the work will not be required and considerable savings can be achieved. The example of the Panix Dam demonstrates that this concept is possible in principle. This approach requires sufficient investigation in order to reliably assess whether the openings possibly left untreated will not cause hazardous leakages. It is clear that without such an investigation a schematic treatment cannot be avoided. Of course, it is easier and regrettably often done, readily to decide in favour of a schematic grouting program rather than to examine meticulously all relevant aspects in order to find out whether the work is really needed or whether it is justified to wait and see. Nevertheless, in view of the usually great expenditures required for a schematic grouting program, we should be under the obligation to examine the chances for more economical solutions.

Sometimes a grouting program was extended because during the first filling of the reservoir a linear uplift reduction was observed. From this it was inferred that a functioning grout curtain was not yet achieved and it was decided to carry out further grouting. For a foundation consisting of karstic limestone this conclusion can be hasty:

- In case of a geological setting combining ungroutable hair-cracks to very fine joints and open cavities, grouting is required and may be successful, however, an over-proportional reduction of the uplift is not achievable even with additional grouting. It needs large grout takes to fill the cavities completely.

However, the permeability due to hair-cracks and fine joints remains and this causes the linear uplift reduction.

- A limestone without cavities but owning just hair-cracks and very fine joints also produces a linear uplift reduction because they are not groutable either; such a setting characterizes a practically impervious rock which, if representative, needs no sealing.

In respect to the manifold interdependence between the geological setting of the voids, the grout takes and the uplift reduction, finally it is referred to the classification A–E, established for the geological conditions of the foundation at Pueblo Viejo, which has been observed also at other dam sites—of course, in compliance with their respective conditions {examples: Atatürk-Dam (Turkey); Zimapan-Dam (Mexico)}.

The depth and the length of the grout curtain, which depend (a) on the permeability distribution with depth and (b) on the sideways inclination of the groundwater table, should be known prior to the beginning of the construction, otherwise the expenses cannot be determined precisely and early enough. Then, it is unclear whether additional work might be required to finish the impermeabilization; it likewise remains unknown whether the purpose of the project justifies the expenses or from which extension of the work onwards the required treatment becomes too expensive for the purpose. This claim is as self-evident as it is difficult to be fulfilled. In permeable limestone the groundwater table rises sideways with little inclination only: the more permeable the rock the less inclined the groundwater table. It is difficult and very often impossible to explore this during the feasibility study due the lack of money and time. Nevertheless, the example of too long a grout curtain at Pueblo Viejo demonstrates the importance of this hydrogeological research. Considerable investigation had been done, however, later it became obvious that this was not enough because neither the inclination of the groundwater table deeper inside the mountain was sufficiently known nor the directions of the groundwater flow.

## 3.9 Conclusions Drawn from These Examples

The comparative presentation of these examples of grouting programs result in many apparently inconsistent observations:

- The water absorption frequently did not correspond with the grout takes: The combination of low WPT-values and large cement takes do not fit together, high WPT-values and small grout takes apparently also contradict each other.
- Sometimes it was impossible to reduce substantially the permeability in spite of the injection of large amounts of grout.
- It also happened that very little seepage occurs although the original WPT-values were high but grout takes were low.

- It is understandable that a grout curtain of small takes cannot reduce the hydraulic head over proportionately; by contrast it frequently happened that no over-proportionate head reduction took place in spite of high grout takes.

These observations raise questions: What relationship exists between Lugeon-values and the average permeability of a large rock mass, and what permeability is required to allow an effective impermeabilization by ordinary cement suspension grouting? It is useless to grout a tight rock. Unless a rock has a minimum permeability—groutable water paths grouting is neither required nor possible

A further inconsistent result is this:

- In some projects the grout takes reach considerable quantities even in the last series of an already close grouthole pattern, possibly ranging at $\geq 300$ kg/m. Such quantities of the latter series should actually indicate that there remained a considerable volume of voids. Nevertheless, because of the already tight spacing of the holes and the previous decrease of the grout takes it was certainly appropriate to finish this grouting program.
- In other projects—and in contrast to the above—further grouting series are sometimes requested although the primary (and secondary) holes absorbed only small quantities—say: 30 and 20 kg/m, respectively.

This is a clear contradiction because the quantities of the cement injected reflect the available volume of voids, provided the course of the grouting has not been determined by hydrofracturing. Voids of similar sizes, shapes amid volumes should absorb similar grout takes, irrespective of their different origins. It would not be consistent to finish the grouting work in spite of the large takes in the final series while in other projects further grouting series were requested although their primary holes took very little grout. To acknowledge this contradiction means nothing else than to realize that the various rocks are not equally groutable but the final result of a grouting program differs from case to case, leaving a smaller or greater residual permeability. If projects characterized by large grout takes are performing well, it becomes questionable whether those with very small grout takes were really required.

The permeability around the borehole can substantially differ from the permeability of the entire dam site which is controlled by the hydrogeological regime. In cases where the geological setting causes an anisotropic permeability with a smaller permeability between upstream and downstream, the hydrogeological regime is more important in answering the question whether and how much to seal the foundation rather than the results of WPT's usually just carried out along the axis of the dam. Once the hydrogeological regime requires a sealing of the foundation, the appropriate technology—diaphragm or grout curtain—has to be worked out. The given groutability has also to be examined because a successful execution of a grouting program presupposes that the rock is groutable.

If this examination proves a rock to be groutable and if grouting also is justified for economic reasons, then all grouting parameters (grouting pressure, W/C-ratio, spacing of groutholes, direction of grouting) have to be adapted to the geotechnic conditions of the project site.

# Chapter 4
# Permeability Testing by Means of Water Pressure Tests

The permeability of the foundation at dam sites is usually explored by means of water pressure tests (WPT), also called Lugeon-tests. This part of the paper focuses on the most important topics of the execution, evaluation, interpretation and application of the results.

## 4.1 Execution

### *4.1.1 Packer*

The test procedure is illustrated in Fig. 4.1. The type of the packer is important because leakage around the packer falsifies the result. Wherever possible, a single packer should be used as this diminishes the leakage. In heavily jointed rock longer packer sleeves can minimize this leakage. When a pneumatic packer is used, the pressure applied to inflate and fasten the packer has to be much higher than the maximum test pressure, otherwise the packer sleeve is compressed and the water escapes through the gap out of the section being tested—producing the impression of a much higher permeability.

### *4.1.2 Automatic Recording*

Automatic recording of both pressure and quantity is to be preferred. However, as this is somewhat sophisticated, one has to be sure that the devices are functioning well, otherwise there will be a source of considerable errors. Where the local technological standard makes the automatic recording doubtful, manual recording, using simple manometers and flow-meters is preferred because it is less expensive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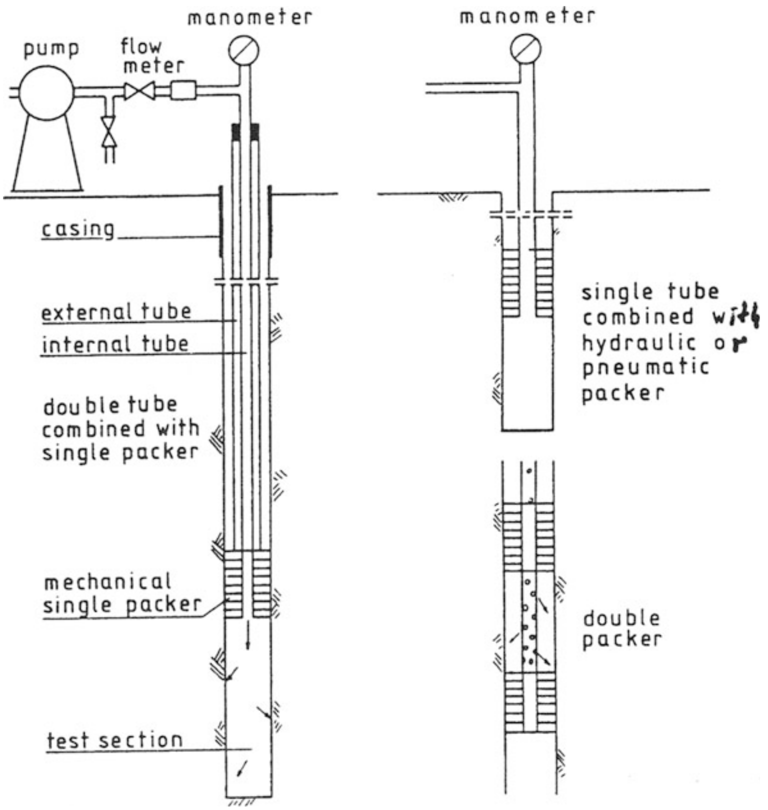



**Fig. 4.1** Layout of water pressure test (WPT) considering different packer systems

and more reliable. Rough working conditions impair the precision of the devices and it is advisable to calibrate the instruments repeatedly.

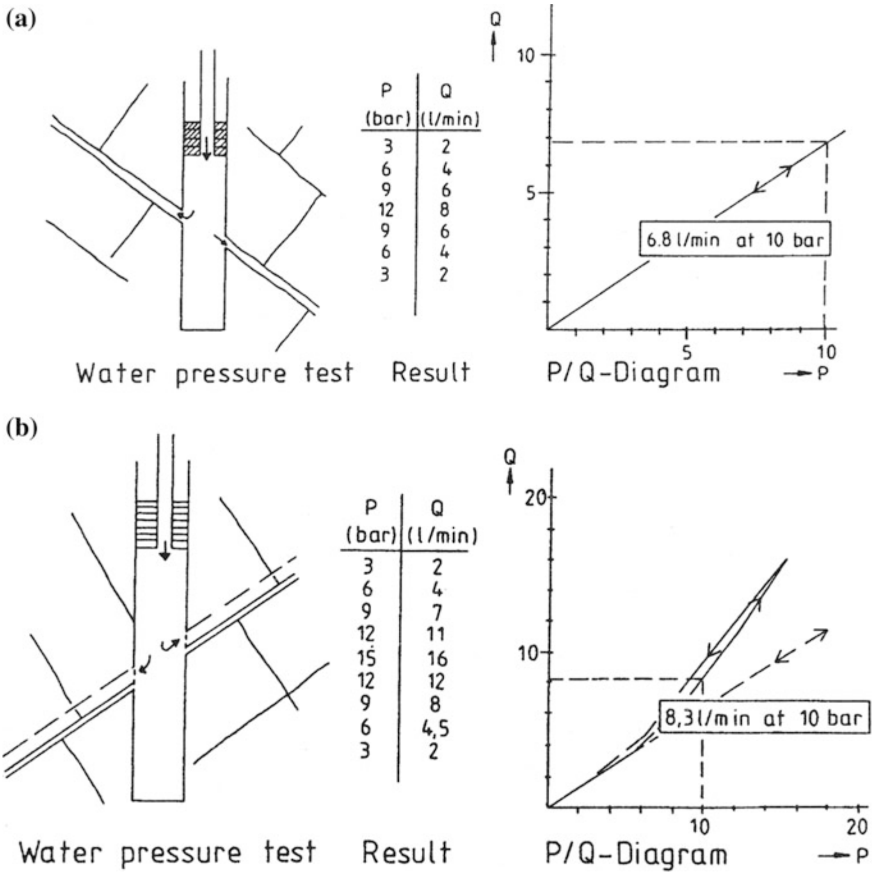
### *4.1.3 Maximum Test Pressure*

The maximum test pressure should be at least 20% higher than the hydraulic head later imposed by the reservoir. It should also be high enough—at least in several tests—to examine the critical pressure which initiates hydrofracturing because knowledge of this property is required for the definition of the grouting pressure.

In case of dams higher than 100 m it is not sufficient to apply a maximum pressure of 10 bar only, just in order to meet the reference pressure of the Lugeon-criterion. The greater hydraulic head provided by the reservoir can induce a larger permeability by hydrofracturing; thus it has to be examined whether this potential condition becomes effective. In cases of dams lower than 100 m the maximum pressure need not always be as much as 10 bar, except those WPT's aimed at analyzing the critical pressure.

### *4.1.4 Increasing and Decreasing Pressure Steps*

WPT's are usually carried out by applying increasing and decreasing pressure steps to examine whether the pressure-dependent changes occur proportionately—or not—and to find out the reasons for this. Figure 4.2a shows a test with a linear P/Q-relationship, i.e. without hydrofracturing. Figure 4.2b gives an example for an over-proportionate increase of Q caused by a dilation of existing water paths.




| P (bar) | Q (l/min) |
|---|---|
| 3 | 2 |
| 6 | 4 |
| 9 | 6 |
| 12 | 8 |
| 9 | 6 |
| 6 | 4 |
| 3 | 2 |

| P (bar) | Q (l/min) |
|---|---|
| 3 | 2 |
| 6 | 4 |
| 9 | 7 |
| 12 | 11 |
| 15 | 16 |
| 12 | 12 |
| 9 | 8 |
| 6 | 4,5 |
| 3 | 2 |

**Fig. 4.2** **a** WPT, increasing and decreasing pressure steps, P/Q-diagram indicating a linear relationship. **b** WPT, increasing and decreasing pressure steps, P/Q-diagram indicating an over-proportionate relationship caused by dilation

Besides these there are other P/Q-relationships; everyone has a distinct geo-mechanic reason. Whenever this has to be analyzed, a sequence of pressure steps is important. In fact, whether and how many steps are chosen should depend on the purpose of the testing program:

WPT's aimed at exploring only the natural permeability need one pressure step only. In order to measure the original permeability, i.e. not influenced by dilation of fissures or hydrofracturing of latent discontinuities, a low pressure should be applied—say 1 or 2 bar. Such tests are adequate for grouting programs when WPT's are carried out to control the progressive sealing of the subsequent grouting series and to find the relationship between water and cement takes. The one-step-test is appropriate because it provides the required information at low costs as testing time is minimized.

Although from the academic point of view this is highly appreciated, within a grouting program a systematic testing carried out in all subsequent series is not required. It is however helpful to do this at least in the last series at the end of the grouting work: to allow the dam engineer to make a much better assessment of the sealing achieved.

WPT's aimed at investigating the dilation and hydrofracturing behavior and defining the appropriate grouting pressure require a sequence of pressure steps. Houlsby suggested five pressure increments (a–b–c–b–c). Such a sequence produces a useful result. This suggestion is expedient but should not be understood as a general rule. The number and order of the pressure steps have to be chosen for the individual requirements of the projects. This type of WPT's is applied in feasibility studies. In defining the absolute scale of these pressure steps the maximum pressure should not only exceed the hydraulic head provided by the reservoir but should also reach the critical pressures causing dilation or hydrofracturing. This is particularly important for the definition of the adequate grouting pressure.

## 4.2 Evaluation

### *4.2.1 Pressure Correction*

Because of the hydrostatic head, the eventual groundwater back pressure and the head losses due to skin friction, the pressure indicated by the manometer is lower than the effective pressure in the borehole section being tested (Fig. 4.3). Heitfeld introduced a '*pressure correction*' to find the actual pressure, subsequently improved by Ewert. Kutzner argues in favor of this correction. Houlsby again recently denied the need for such a correction.

The meaningful discrepancy between the pressure recorded at the top of the borehole ($P_M$) and the effective pressure ($P_T$) becomes obvious in P/Q-diagrams published by Foyo. Representative examples are shown in Fig. 4.4. Without pressure correction the apparently effective pressure increases. Unless the effective

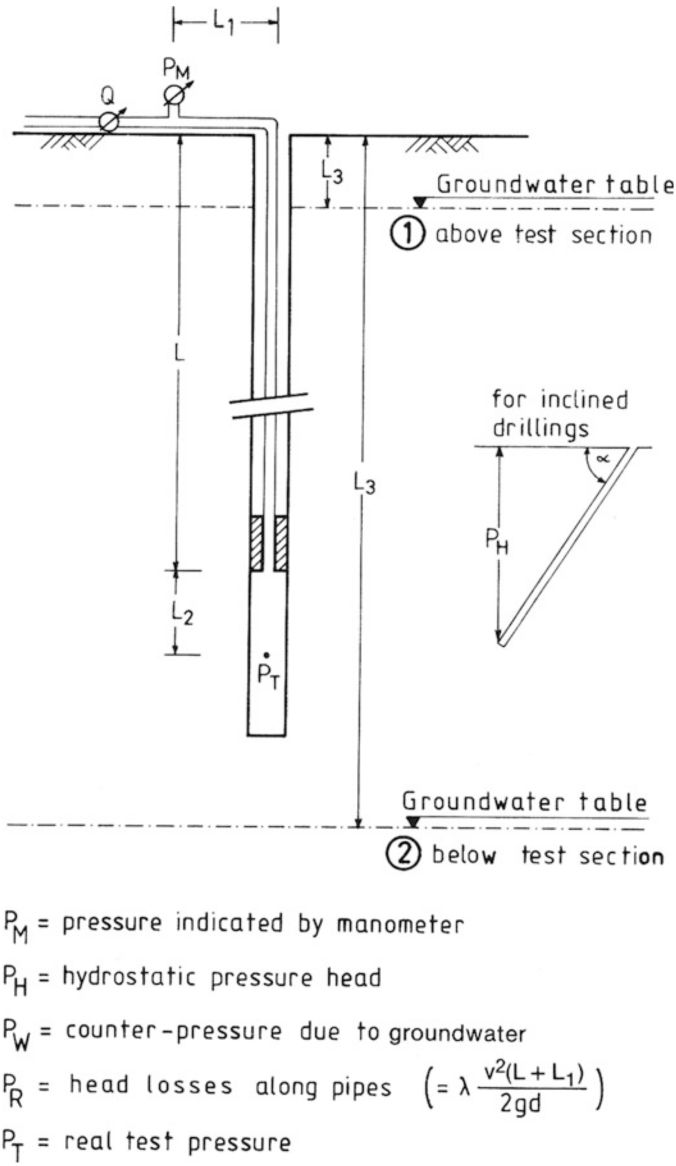



**Fig. 4.3** WPT, pressure correction for finding the effective pressure

pressures are directly measured—still exceptional cases—the pressure correction should be done whenever the given conditions cause substantial discrepancies between $P_M$ and $P_T$ and the purpose requires a rather precise determination of the actual effective pressure.

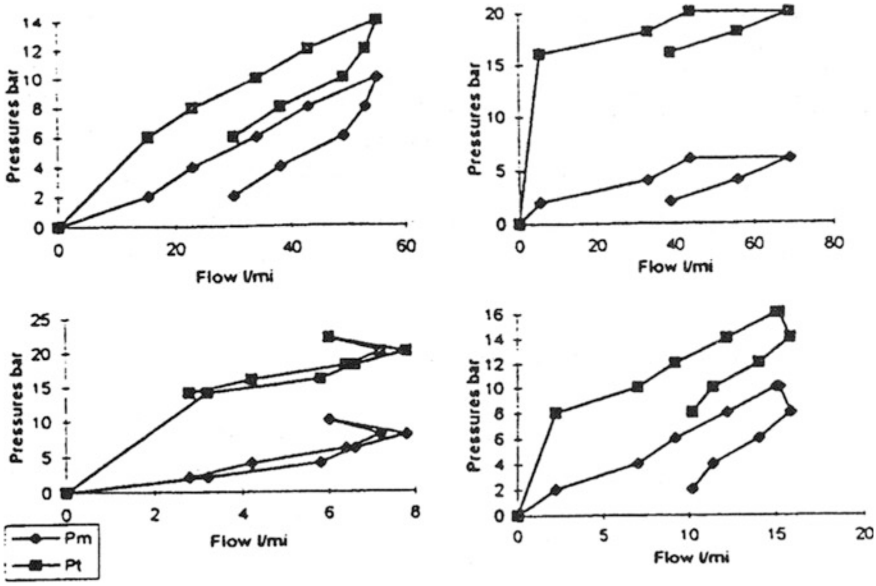



**Fig. 4.4** Effect of pressure correction in P/Q-diagrams, after FOYO

The pressure correction involves two factors which are sometimes negligible, the groundwater back pressure (4.1: $P_W$) and the head losses due to skin friction (4.2: $P_R$):

$$P_T = P_M + P_H - P_W - P_R \tag{4.1}$$

$$P_T = P_M + P_H - P_R \tag{4.2}$$

The groundwater back pressure is not effective when the borehole stage being investigated lies above the groundwater table. The groundwater back pressure is not effective if the rock is absolutely tight and no absorption takes place as the water paths do not intersect the borehole, e.g. the water paths and the stage of the borehole tested belong to separate hydraulic regimes. The situation becomes complicated when latent discontinuities are cracked during higher pressure steps:

* opened paths end without being connected with original water paths, thus the groundwater back pressure remains ineffective
* opened paths are connected and the back pressure becomes effective

The head losses due to skin friction are negligible in cases of low water flows and short pipes. If the permeability has to be explored just qualitatively—the rock is too permeable or the rock is tight enough—the head losses are not important and can be disregarded.

We acknowledge more and more the importance of the appropriate pressure in both testing and grouting because softer rocks have to be treated more frequently and hydrofracturing begins at pressures below 5 bar. To avoid the negative results of hydrofracturing, grouting pressures should not be too high. Provided (a) the borehole section treated has a deep position, (b) the groundwater back pressure is lacking and (c) the head losses due to friction do not reach a sizeable magnitude, the hydrostatic head can easily increase the effective pressure above the critical pressure level (Fig. 4.4). If the importance of the pressure correction remains disregarded, the intended low pressure indicated by the manometer is presumed to be effective while the actual pressure is much higher. In a testing program, this falsifies our assessment of the original permeability. In a grouting program the hydrostatic head produces an even higher effective pressure due to the greater density of the grout suspension, this forces hydrofracturing and re-filling of opened discontinuities.

Hydrofracturing does not begin at a pressure of 1 bar, not even in soft rock. Therefore, Houlsby and Ewert proposed this pressure step to find out the original permeability. Thus we have to be sure that the effective pressure is actually 1 bar, or we have to be sure that the absorption rate does really belong to this pressure but not to higher or lower pressures. Thus, it is necessary to know the effective pressure precisely, which means we need to ensure the pressure indicated by the manometer is correct otherwise our assessment is misleading.

The scope of this contribution does not allow space to discuss individual examples. The importance of the pressure correction is considered evident, particularly in view of the definition of the appropriate grouting pressure for grouting of rock types susceptible to hydrofracturing. This latter aspect is essential because the author is more and more convinced that WPT's are more helpful to examine the deformation behavior and the groutability rather than to investigate the permeability.

### *4.2.2 Graphical Presentation*

Cambefort introduced P/Q-Diagrams to present graphically the results of WPT's. His diagrams were not very clear and conclusions were difficult to draw. This might have been the reason for the reluctance of other authors to present their results in similar ways or to prefer other forms of graphical presentations. The author has used P/Q-diagrams since 1969, gaining the positive experience that P/Q-diagrams are a valuable tool to present the information graphically and to interpret the results. In fact, the ambiguity of the Cambefort diagrams disclosed more about failures in the execution or evaluation of the WPT's than about the correctness of the P/Q diagrams. Provided a test is properly executed and evaluated it yields a diagram which is useful not only to characterize the permeability and the deformability of the rock but also to compare rock types in this respect.

The combination of P/Q-diagrams, geological features and grouting results is really informative and also proves the usefulness of these graphs. Meanwhile, P/Q-diagrams have found wide acceptance and are increasingly applied. Other authors evidently also share the experience that this presentation allows a fast and reliable interpretation of the hydrogeological situation, particularly when it is combined with the geological profile.

Houlsby comments quite critically on the P/Q-diagram: “Why risk the inconsistencies of graphing when it is quite superfluous?” He prefers bars to plot the results (Fig. 4.5): “bars are used to convey pictorially relative magnitudes, but once the concept is understood, the process of deciding which Lugeon value to report for the test can be done entirely mentally”.

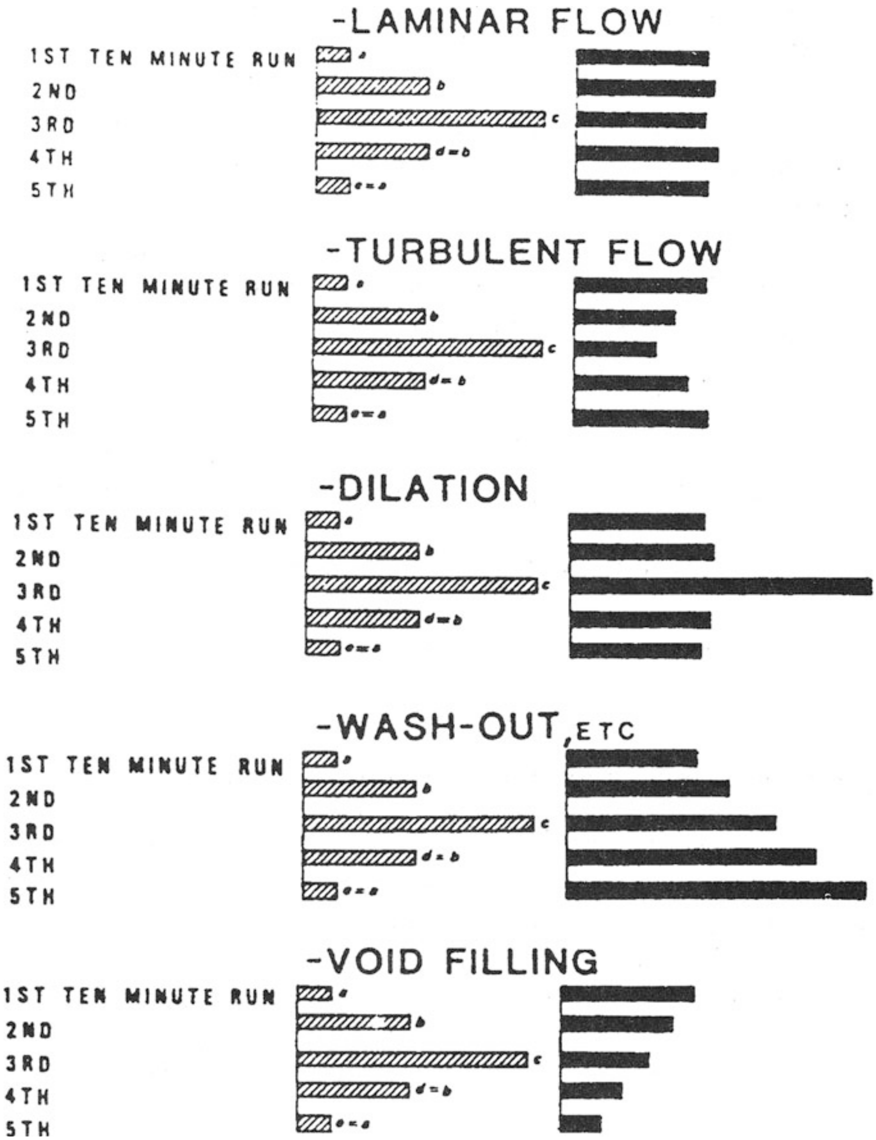



Fig. 4.5 Evaluation and graphical presentation of WPT proposed by Houlsby

Further details are unnecessary because anybody may use the type of presentation he considers the most appropriate. Nevertheless, it should be mentioned that the presentation preferred by Houlsby has its inconsistencies as well: It is less suitable for comparisons of programs, it does not give the absolute scale and is not easy to be instantly interpreted; furthermore, it does not disclose so clearly the deformation behavior, as an important result of the test. Last but not least, the presentation in bars, in fact, is nothing else but the reverse form of a P/Q-diagram—the data are the same.

### *4.2.3 WPT-Values Proposed*

Proportionate P/Q-relationships form a minority; the other ones prevail. Their absorption rates at 10 bar are not just ten times the absorption at 1 bar. If hydrofracturing occurs below 10 bar, the Lugeon criterion related to 10 bar reflects already an enlarged permeability. In describing the original permeability, the effect of hydrofracturing should be eliminated. On the other hand, it is useful to grasp the influence of hydrofracturing—or other modifications. Therefore, the author uses not only the conventional Lugeon criterion but the following WPT-values illustrated in Fig. 4.6:

- $Q_1$ absorption rate at a pressure of 1 bar reflecting the true permeability, i.e. without being influenced by dilation of existing fissures or hydrofracturing of latent discontinuities;
- $Q_{10}$ absorption rate at a pressure of 10 bar, linearly extrapolated from Q1 in order to eliminate possible influences caused by dilation or hydrofracturing, and to permit the comparison with conventional Lugeon-values obtained in other programs.
- $Q_1$* absorption rate at the individual maximum pressure applied in a given program linearly reduced to the absorption rate at a pressure of 1 bar, including eventual influences caused by dilation or hydrofracturing;

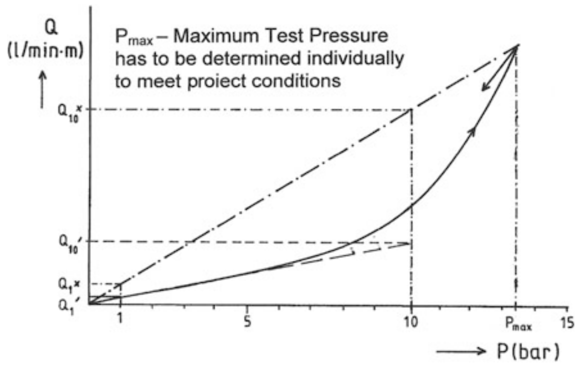



**Fig. 4.6** WPT-values proposed, definition of WPT parameters $Q_{10}'$ (original permeability) and $Q_{10}$* (provoked permeability)

- $Q_{10}$* absorption rate linearly extrapolated from Q1* to a pressure of 10 bar.

Differences between $Q_{1/10}$ and $Q^*_{1/10}$ respectively, indicate whether—and how much—dilation or hydrofracturing occurred.

### *4.2.4 Computerized Evaluation*

An investigation program usually comprises a great number of WPT's. The pressure correction and the graphing require a lot of work. A computerized evaluation of testing and grouting data combined with a direct presentation of the diagrams is helpful to save time. The results are instantly available and can be interpreted—particularly important in all programs—where the results of the foregoing tests and grouting determine the subsequent work. Sometimes WPT's are carried out systematically also in grouting programs, particularly in the later series. Then, the WPT-values should also be evaluated statistically like the grouting data by using the computerized evaluation. Further details are presented in connection with the evaluation of grouting data. Figure 4.7 presents an example of such an evaluation.

## 4.3 Hydrofracturing Caused and Indicated by WPT's

The relationship P/Q is mostly non-linear but over- or under-proportionate. The over-proportionate increases occur very often—sometimes in combination with a slow or sudden pressure drop. It originates from '*hydrofracturing*'. this general term comprises various courses of a WPT or a grouting process:

⇒ too high a pressure causes a splitting of latent discontinuities,
⇒ too high a pressure causes the dilation of existing fissures,
⇒ too fast a water flow causes the erosion of materials filling open fissures.

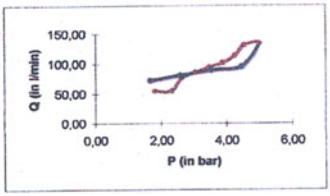
### *4.3.1 Splitting and Dilation*

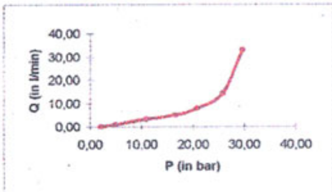
All rock types suffer dilation or hydrofracturing provided the pressure is high enough to exceed the critical level. This '*critical pressure*' ranges between: $\leq 2$ bar and $\geq 60$ bar and is controlled by the local conditions. The analysis of approximately 60 different rock types confirmed that critical pressures can be examined by means of WPT's, provided the test is skillfully carried out and evaluated. The general validity of this method has been established and it is helpful in defining the grouting pressure.

**Project:** Siah Bishe
**Work:** W/P-Tests
**Series:** Sia-NWP-all.WPT
**Hole No.:** Various holes
Coordinates (x, y, z): 49697,00 79605,00 1879,00
Type of drilling: with core-recovery
Inclination: 90°
Diameter of hole: 76 mm
Internal diameter of pipe: 25.4 mm
Rock: Shale etc
Roughness pipe: 0.01 mm
Distance Hole-Manometer: 3 m

*Factors:*
*W.L.* – *Groundwater level,*
$P_M$ – *Test pressure at top of Hole,*
$P_T$ – *Effective test pressure,*
$Q$ – *Water take in l/min,*
$Q_{1/10}{}^{\#}$ – *for 1 bar and 10 bar, related to absorption at 1 bar,*
$Q_{1/10}{}^{*}$ – *for 1 bar and 10 bar, related to true absorption at $P_{max}$*

| Hole | W.L. in m | Depth in m | $P_M$ in bar | $P_T$ in bar | Q in l/min | $Q_{1/10\,bar}{}'$ in l/(min*m) | $Q_{1/10\,bar}{}^{*}$ in l/(min*m) |
|---|---|---|---|---|---|---|---|
| | | 15 to 20 | 0,40 | 1,81 | 54,60 | 6,05 | |
| | | | 0,90 | 2,31 | 54,60 | | |
| | | | 1,40 | 2,63 | 77,20 | | |
| | | | 1,90 | 3,05 | 86,40 | | |
| | | | 2,40 | 3,46 | 94,60 | | |
| | | | 2,90 | 3,88 | 102,20 | | |
| | | | 3,40 | 4,25 | 114,00 | | |
| | | | 3,90 | 4,54 | 130,60 | | |
| | | | 4,40 | 4,99 | 134,00 | | 5,37 |
| | | | 3,40 | 4,45 | 96,00 | | |
| | | | 2,40 | 3,51 | 90,00 | **6,05** | **5,37** |
| | | | 1,40 | 2,59 | 81,60 | **60,50** | **53,68** |
| | | | 0,40 | 1,67 | 72,60 | **Type:** | **6b** |
| | 1,00 | 27,8 to 28,8 | 2.00 | 2.10 | 0,00 | 0.00 | |
| | | | 4,90 | 5.00 | 1,00 | | |
| | | | 10,80 | 10,90 | 3,20 | | |
| | | | 16,50 | 16,60 | 4,95 | | |
| | | | 20,70 | 20,80 | 7,80 | | |
| | | | 25,80 | 25,86 | 14,40 | | |
| | | | 29,80 | 29,74 | 33,10 | | 1,11 |
| | | | | | | **0,00** | **1,11** |
| | | | | | | **0,00** | **11,13** |
| | | | | | | **Type:** | **3** |
| NPS-17 | 1,00 | 17,9 to 19,9 | 2,00 | 2,10 | 0,00 | 0,00 | |
| | | | 4,00 | 4,10 | 0,00 | | |
| | | | 10,00 | 10,10 | 0,00 | | |
| | | | 15,00 | 15,10 | 0,00 | | |
| | | | 20,00 | 20,10 | 0,00 | | |
| | | | 24,00 | 24,10 | 0,00 | | |
| | | | 24,80 | 24,82 | 28,00 | | |
| | | | 29,00 | 28,94 | 41,00 | | |
| | | | 33,00 | 32,27 | 105,00 | | 1,63 |
| | | | | | | **0,00** | **1,63** |
| | | | | | | **0,00** | **16,27** |
| | | | | | | **Type:** | **3** |



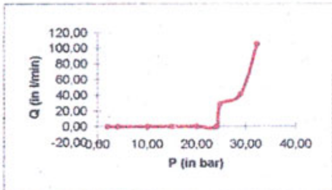




**Fig. 4.7** Computerized evaluation of WPT

The dilation of open fissures and the splitting of latent discontinuities are controlled by the relationship '*test or grouting pressure versus counter-pressure*'. The geological conditions determine whether the counter pressure is caused either by the weight of the overlying rock at the level of the borehole section treated or by the strength of the rock surrounding the borehole. Figure 4.8 illustrates the different conditions.

Weathering produces a loosened rock bond, the rock units are disconnected along the discontinuities, thus they can move relative to each other. Open joints ("fissures") cause an original permeability while bedding planes may still be latent and, hence, tight. These conditions exist in seams near the surface. The '*critical*'

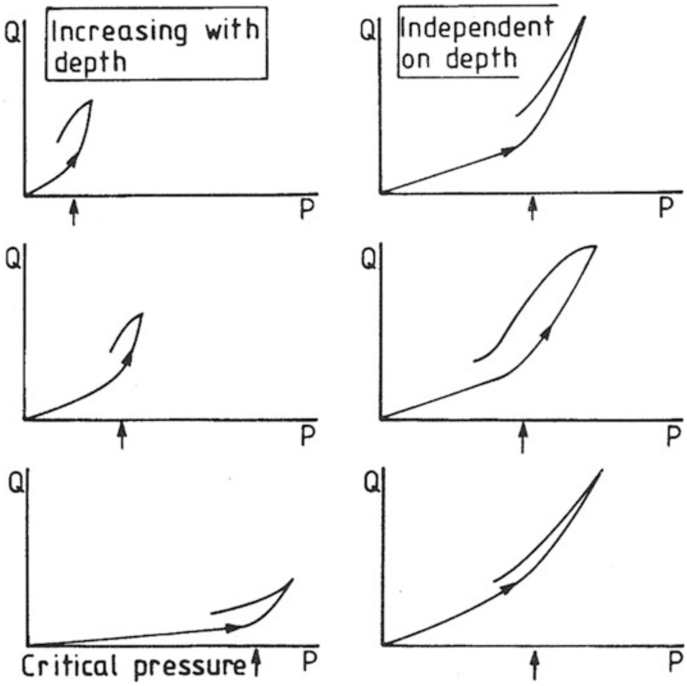



**Fig. 4.8** Critical pressures for different rock bond: disconnected—increasing with depth, connected—independent of depth

pressure causing the splitting of the latent beddings or the dilation of the fissures depends on the weight of the overlying rock. In such cases the critical pressure increases with depth. As the influence of weathering disappears with depth, the disconnected rock bond is usually limited to a shallow band of about 10 or 20 m thickness parallel to the surface, sometimes even less.

At a greater depth the rock bond is still connected; the discontinuities are still latent or only partly opened. Water paths have developed only locally. These cases form the overwhelming majority in all rock types. The critical pressure is controlled by the strength of the rock. It is largely independent of depth because the individual strength remains about the same. A further important factor is the angle of intersection between the boreholes and the most susceptible discontinuities. An intersection of 90° facilitates both the dilation and the splitting. Bedding planes have a particular susceptibility to hydrofracturing. As the strength differs widely, the critical pressures likewise show a wide range. In weak rocks very low pressures can cause hydrofracturing irrespective of the depth $\leq 5$ bar. For instance, in strong rocks high pressures are required to crack the rock even if it lies at a shallow level.

Examples of critical pressures for various rock types related to depth are given in Table 4.1. The results of this comparative analysis are summarized as follows:

**Table 4.1** Critical pressures for various rock types

| | | | |
|---|---|---|---|
| Aabach | Sandstone, Shale | 6 | S |
| Albarello | Gneiss | 5> | S |
| Almendra | Granite | 15> | S |
| Antrift | Sandstone, Siltstone | 3 | O |
| Cemadilla | Gneiss | 5–10 | S |
| Eglisau | Claystone | 12 | S |
| Eglisau | Marly Sandstone | 7 | S |
| Godae-e-Landar | Sandstone, Claystone | 7> | S |
| Haueda | Sandstone, Siltstone | 8> | S |
| Haune | Gypsum | 3> | O |
| Huites | Granite | 5> | S |
| Karun I | Limestone | 10> | S |
| Llosa del Caval | Limestone, Sandstone | 8 | S |
| Mingorria | Granite | 5> | S |
| Möhne | Shale, Sandstone | 7–11 | S |
| Panix | Limestone | 7–10 | S |
| Ponga | Slates | 6 | S |
| Prims | Slates | 3> | O |
| Prims | Slates | 10–12 | S |
| Peublo Viejo | Limestone | 10> | S |
| Riansares | Gypsum, Claystone | 5> | S |
| Tavera | Conglomerate | 5> | S |
| Twiste | Sandstone, Siltstone | 2> | O |
| Yuracmayo | Tuffites | 5> | S |

*S* controlled by strength, *O* controlled by overburden pressure

- The critical pressures increase with depth, if the rock bond is disconnected which is associated with a shallow seam; if the rock bond is still connected, the critical pressure remains about the same irrespective of depth.
- The critical pressures have a wide range—between $\leq 2$ bar and $\geq 60$ bar, the splitting of latent discontinuities mostly begins at pressures far below 20 bar, the dilation of existing fissures begins at even lower pressures.
- In uniform rock types the critical pressures are of the same order of magnitude; in non-uniform rock critical pressures cover a wide range of values irrespective of depth.
- Bedding planes, cleavage planes or planes caused by lamination are more susceptible to hydrofracturing than normal joints.
- An intersection angle of 90° between borehole and discontinuities facilitates hydrofracturing.

The influence of (a) the depth-dependent weight of overlying rock and (b) the depth-independent deformability on the critical pressure is of prime importance for the definition of the grouting pressure which is discussed later on. At present, this complexity is not fully recognized and acknowledged. If it was, appropriate grouting pressures would be used in more projects.

Splitting or dilation of discontinuities occurs under various conditions. Either one produces its own P/Q-relationship. This allows us to characterize such conditions by means of P/Q-diagrams.

Only in four of these 24 examples the critical pressures are controlled by overburden pressure, all the others by strength, and even then hydrofracturing occurs already at rather low pressures. This result does not confirm the pressures mainly applied in practical grouting work: there the pressures increase with depth because it is wrongly assumed that the overburden pressure is the decisive factor—as it increases with depth, the grouting pressure can go up as well. This approach causes a lot of overdoing as exemplified in Sect. 3.5.

### *4.3.2 Erosion and Clogging*

With increasing the test pressure, the water flows more quickly through the water paths. If it flows too quickly, erodible material filling the fissures ('*infillings*') is transported away. Provided the fissure has an open and free outlet, erosion ('*Wash-out*') takes place; if such an outlet is lacking, clogging occurs. Erosion causes a drastic over-proportionate increase of the water takes, often combined with a distinct pressure drop. In case of clogging the erodible material is transported towards a narrow spot of the fissure where a regressive sedimentation begins, this causes an under-proportionate increase of the water takes. Erosion as well as clogging produce their own significant P/Q-diagrams which are presented later on.

## 4.4 Classification and Interpretation of P/Q-Diagrams

Interpreting P/Q-diagrams allows the dam engineer to characterize the permeability and deformation behavior under both hydraulic load and grouting pressure. Thus, about 30 years ago the idea developed to use P/Q-diagrams for an analysis of the pressure-dependent permeability and deformation behavior. Several authors worked on such a classification: Cambefort, Ewert, Foyo, Houlsby, Klopp & Schimmer, Kutzner, Verfel and others. The different authors use different systems for their classification but, here only a few systems can be briefly discussed.

The possibility of such a classification implies the danger of overdoing; it is prudent not to classify too many types. A classification system should cover all possible cases of all rock types and it should be of general applicability. It should not be too sensitive, otherwise we run the risk of defining two or even more types for apparently different courses which, in fact, are based on similar conditions but appear to be different just because of slight variations of the influencing factors (execution, setting of water paths, deformability), We should always realize that the nature of all influencing factors limit the accuracy of the test.

Houlsby distinguishes between five different types of P/Q-relationship (Fig. 4.5):

- Proportionate increase of water absorption indicates laminar flow.
- Under-proportionate increase indicates turbulent flow.
- Over-proportionate increase indicates dilation of paths.
- Strong under-proportionate increase combined with pressure drop indicates erosion.
- Decreasing absorption throughout the test indicates void filling.

Kutzner stated that, according to his experience, also five types of P/Q-diagrams should be considered; he gives the same explanation: A—laminar flow, B—turbulent flow, C—elastic deformation, D—erosion or cracking, E—filling of isolated voids without outlets (Fig. 4.9).

In the author's view, both classifications are incomplete and not useful for practical purposes:

- Impermeable rock has not been considered. Under the condition of the test pressure usually applied, this type occurs frequently. Therefore, it is prudent to consider this as a special type. This WPT does not absorb water yielding the P/Q-diagram in Fig. 4.10.
- The distinction between laminar and turbulent flow is not correct from the hydraulic point of view and it is not recognizable for geological reasons. It is also somewhat useless for practical purposes because its possible influence is far beyond the accuracy of the test:

  * Kutzner explains his classification by referring to Rissler. He found in model tests that laminar flow takes place in fissures less than 0.13 mm in width while fissures wider than 0.4 mm show turbulent flow, both irrespective of pressure, which in effect means irrespective of velocity. This conflicts with the hydraulic laws because the change between laminar and turbulent flow depends on the velocity of flow. The water flows slowly during the low-pressure stages and faster during the high-pressure stages: the change between laminar and turbulent flow takes place within the same WPT, irrespective of the width. The observation made by Rissler, certainly correct for

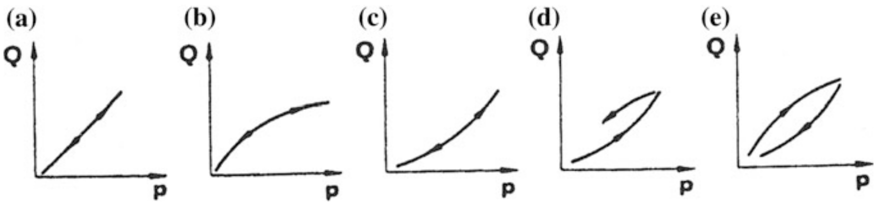



**Fig. 4.9** Classification of P/Q diagrams after Kutzner

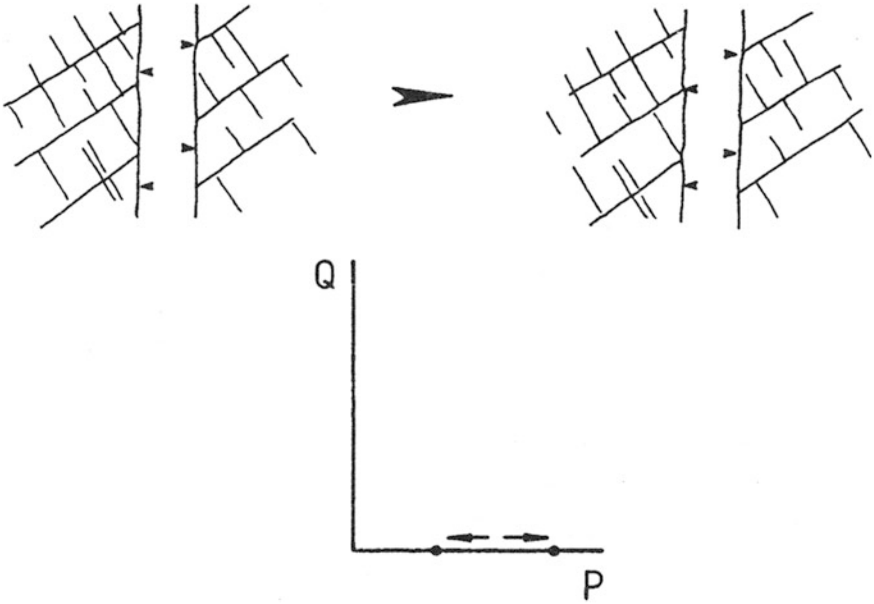



Fig. 4.10 Tight rock yielding a special P/Q-diagram

the conditions of his model tests, cannot simply be transferred to the other hydraulic regimes presented by the actual water paths whose conditions are completely unknown, not recognizable and permanently changing.

* Within the borehole section tested we may have just one or a few or even a multitude of paths and likewise a variety of diameters. The geometry of the paths changes behind the face of the borehole and beyond. All these different patterns have their individual hydraulic properties, of course—but which ones? Samples from core drillings do not disclose this, even if the core recovery reached 100%. For a conclusive answer, the results obtained by such a model test are actually not helpful.
* Changes between laminar and turbulent flow are most likely to occur in some tests and this influences the P/Q-relationship yielding graphs like those shown (Fig. 4.11: Group 4). This type is quite abundant. The deviation from the straight line apparently reflecting laminar flow can be slight or distinct but what does this actually disclose and what interpretation about geological features is permitted? A classification as a special type is only justified if this allows the dam engineer to specify the hydrogeological setting or a specific deformation. This, however, is not possible. It has been demonstrated that the results of WPT's are ambiguous because a variety of different settings can yield similar results. This means that WPT's in principle have a low accuracy, the accuracy decreases with larger water takes. Thus, only a distinction between graphs which are clearly different and which can be geologically explained is considered helpful for practical purposes.

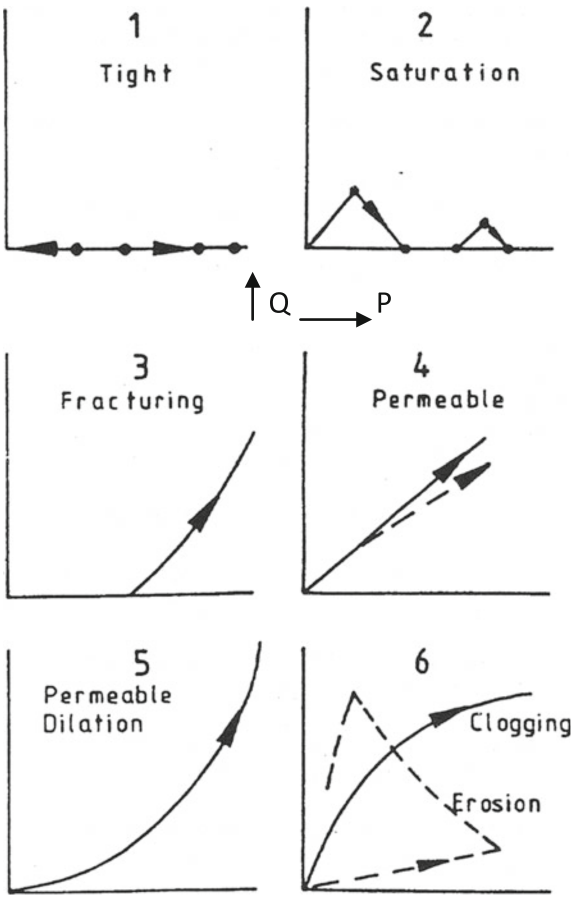



**Fig. 4.11** Classification of P/Q-diagrams according to geo-mechanical conditions

* Conclusively, the distinction between laminar and turbulent flow is useless for the purpose of practical assessments, Changes between both certainly occur but are of minor importance, they are not clearly identifiable and do not allow the dam engineer to classify a geologically well-defined group of individual properties.

When Ewert began to deal with this complex, he evaluated WPT's from programs carried out in 8 different rock types. This yielded the classification of 10 types of P/Q-diagrams. At that time his view was still influenced by the great variety of diagrams given by Cambefort, Later, it was learned that this number of graphs was probably excessive. In fact, while gaining more experience also with other rock types, one has to realize that in reality we have many types which are actually different. Moreover, working with only a few typical ones is more advantageous for practical reasons. With growing experience, it was understood that WPT results can be ambiguous and that the number of types to be distinguished depend decisively on the pressure being applied:

- If the pressure is high enough, hydraulic fracturing or dilation or erosion occurs always. Thus, under the condition of a sufficiently high pressure only one type of P/Q-diagram remains: an over-proportionate increase of the water take due to hydrofracturing, irrespective of all the transitional types in between. Consequently, the question of whether more or fewer types should be distinguished is only relative: It depends on the highest pressure appropriate for the geotechnical conditions of a given project site and for the purpose of a program (assessment of original permeability, assessment of deformability, assessment of permeability reduction by grouting).
- Different combinations of water conducting paths can yield similar water takes. The principle is straight forward: a few wide paths can absorb the same or a similar quantity of water as a multitude of narrow paths. The examples presented in Fig. 4.12 have been obtained by laboratory tests.

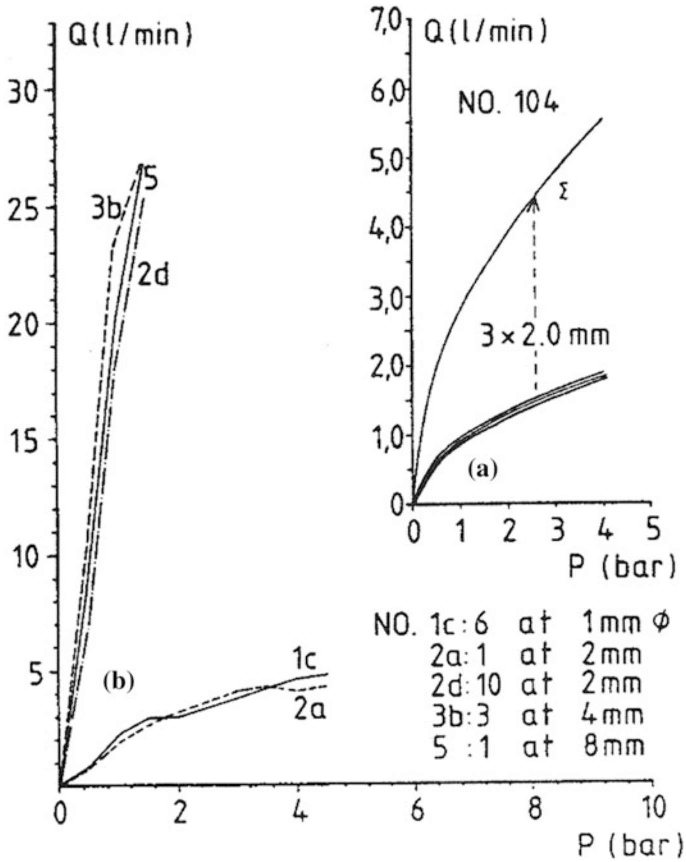



**Fig. 4.12** WPT using modified water paths: similar WPT-values result from many narrow paths and few wide paths

Considering all these results this classification was used and has been successfully applied:

⇒ impermeable rock,
⇒ rock of unchangeable permeability (P/Q—linear),
⇒ rock of changeable permeability due to deformation (P/Q—over-proportionate) and
⇒ rock of changeable permeability due to erosion (P/Q over-proportionate, pressure drop).

Subsequently the author continued analyzing WPT-data obtained in further rock types and this resulted in a more comprehensive view. It turned out to be prudent tracing back the variety of P/Q-diagrams to the '*geological mechanism*' controlling the course of the WPT. Conclusively, then the following classification was used which differentiates between 6 groups, either one comprising various similar forms always caused by the same mechanism shown in Fig. 4.11:

Group 1 tight rock, no absorption during highest pressure stages
Group 2 tight rock, insignificant absorption during one or another higher pressure step due to saturation of isolated voids
Group 3 tight rock, (over-proportionate increase of water take during higher pressure stages due to '*fracturing*' of latent discontinuities
Group 4 permeable rock, linear P/Q-relationship, slight deviations included (over- or under-proportionate)
Group 5 permeable rock, over-proportionate increase of water takes during higher pressure stages due to '*dilation*'
Group 6

(a) tight or permeable rock, after reaching critical pressure radical increase of water take together with pressure drop due to erosion,
(b) although a different form of diagram but principally a similar geological mechanism: distinct under-proportionate increase due to clogging; both courses indicate transport of erodible material.

It should be repeated that such a system is relative. While we would always obtain a P/Q-diagrams indicating hydrofracturing provided the test pressure was high enough, we get various diagrams because we apply maximum test pressures appropriate for the given conditions. The meaning of the relationship between strength, appropriate test pressure and number of types becomes clear by comparing different examples:

- In cases where rock of high strength is investigated and not too high a test pressure is applied, the tests yield only P/Q-diagrams belonging to group 1 or 3 (tight or proportionate increase, the latter with slight variations).
- In cases where rock of low strength is investigated and a high test pressure is applied, the tests yield P/Q-diagrams showing fracturing by dilation, cracking and erosion.

The question of an '*appropriate pressure*' will be discussed later. The comparison makes clear that the number of types obtained depends largely on the relationship between the factors mentioned above.

Some authors classify the P/Q-relationship without interpreting the diagrams. This makes the classification useless. As a P/Q-diagram reflects the individual geological mechanisms, the interpretation of the diagrams, following the above classification, is useful. It also serves to characterize not only the original permeability but also the water routing, the deformation behavior and, together with the results of test grouting, the groutability. Examples confirm that both the classification of the diagrams and the interpretation of their respective geological reasons allow a much better assessment of these rock properties. Thus, P/Q-diagrams yield a valuable typology describing the pattern of given rock types.

In summary it has been experienced that classified P/Q-diagrams, in combination with field observations and test grouting, are helpful to examine and to characterize the permeability in detail, the water routing, the deformation behavior and the groutability. The system proposed here comprises six groups related to the course of the WPT's which is determined by the geological mechanism. However, WPT-results in detail sometimes can be ambiguous and, furthermore, they only reflect the permeability of the rock surrounding the borehole which can differ substantially from the permeability of the whole rock mass. This latter permeability is controlled primarily by the hydrogeological regime rather than by the local water paths. Therefore, it is necessary to give further consideration to the question of whether, where and to what extent WPT's are useful.

## 4.5 Size and Shape of Water Paths and Absorption Rates, Ambiguity of WPT's

Head losses due to skin friction increase inversely with diameter and length, wide openings permit an almost pressure-less flow while narrow paths require very high pressures. Laboratory tests have been carried out to estimate the relationship between size and shape of natural paths and their flow capacity in terms of Lugeon-units. The water paths were modeled in such a way that they simulated actual geological paths as truly as possible. The details are described in Reference 8, the following examples give an idea:

- An individual circular path, 1 mm in diameter and 900 mm in length, causes a water absorption of less than 0.75 LU.
- An approximately rectangular fissure of $0.2 \times 20.8$ mm in width and 700 mm in length absorbs about the same quantity.

The findings of this laboratory testing permit a reverse relationship to be estimated between WPT-results and the shape and the size of actual paths.

Because of the different rheological characteristics (Bingham vs. Newtonian fluid) a grout mix needs a comparatively higher pressure to flow. Thus, to find the pressures required to initiate the flow of the suspension and to maintain it over a certain time the simulated paths were also grouted. The 0.2 mm path, for instance, needed a pressure of 12 bar, the 1 mm path only 2 bar. Some results of this laboratory testing are summarized in Table 4.2. As the simulated paths used for these tests have their specific hydraulic characteristics, the results cannot be transferred directly to actual paths of a similar shape and size. This, however, does not matter; it is essential to recognize the order of magnitude of the pressures required to grout wide or narrow paths and to relate these paths to the Lugeon-units.

The results of the above laboratory testing and field observation permit the following conclusion:

- When the absorption rate is very small—order of magnitude: 1 LU—the type of the water bearing paths around that borehole section can be appraised rather precisely: the conductivity is necessarily caused by only one or just a few very narrow paths; larger openings do not exist. Thus, low WPT-values can be interpreted as a feature of the hydrogeological setting. This is important to assess the groutability.
- A similar interpretation of high WPT-values is impossible; because the large water takes can be caused by one wide fissure or by a multitude of narrow ones. The absorbed quantity alone does not disclose the number and the size of the possible paths. The laboratory tests also confirmed that different settings can produce equal or similar WPT-values (Fig. 4.12). Thus, these results are ambiguous.
- Compared to a rock type with just a few but wide openings, another one with a multitude of narrow paths has a greater potential of head losses due to skin friction, thus a lower coefficient of permeability in spite of a similar WPT-value.
- While wide fissures can be grouted at very low pressures ($\leq 1$ bar), narrow paths require high pressures to be penetrated ($\geq 10$ bar). This is quite meaningful for the grouting process of dissimilar paths crossing the same bore hole stage and for the groutability.

**Table 4.2** Summarized results of laboratory tests to determine the relation between size of water paths and grouting pressure

| Shape of path | Diameter or width (mm) | Length (mm) | Area ($mm^2$) | Quantity at 4 bar (1/min) | Quantity extrapolated to 10 bar (LU) | Grouting pressure required (bar) |
|---|---|---|---|---|---|---|
| Circular | 1 mm dia | 900 | 0.79 | 0.3 | 0.8 | 6.3 |
| | 2 mm dia | 900 | 3.14 | 1.3 | 3.2 | 2.5 |
| | 3 mm dia | 900 | 7.06 | 2.6 | 6.5 | 1.2 |
| Rectangular | 0.2 × 20.8 | 700 | 4.16 | 0.3 | 0.8 | 12.0 |
| | 1.0 × 20.8 | 700 | 20.8 | 6.0 | 15.0 | 2.0 |

## 4.6 WPT-Values and Coefficient of Permeability ($k_f$)

We are facing a dilemma when we try to describe quantitatively the permeability of rock: For hydraulic computations we need the coefficient of permeability $k_f$ but the conversion of $Q_{WPT}$ into $k_f$ is impossible. Darcy's law presupposes a uniform and isotropic subsoil. A small section of rock as examined by a WPT usually does not meet this requirement. A certain homogeneity assumes a large rock section. For hydraulic reasons the conversion $Q_{WPT}/k_f$ value cannot yield reliable results:

- The basic formula used was established for porous soils, it does not meet the conditions of fissured rock because the volume of voids and the geometry of the paths is not at all comparable.
- The results of WPT's do not disclose whether the water is distributed through many small paths or through a few wide ones. Different settings have their own head losses due to friction which, in a WPT, are only partly effective because the water radially drains off:

  * The unequal friction causes unequal $k_f$ values, even in case of similar WPT-results.
  * The radially draining off in a WPT is hydraulically not comparable to a directed flow through the entire foundation, While the friction largely remains ineffective in a WPT the directed flow activates all the friction, thus, $k_f$ is necessarily smaller.

The comparison between converted $k_f$ values and $k_f$ values based on recorded seepage quantities proves the impossibility of a simple conversion (Fig. 4.13). The example of the Aabach Dam is particularly impressive: The average water take in WPT's ranges at about 35 LU, leading to a calculated $k_f$ value of $1 \times 10^{-5}$ m/s; the seepage, however, is less than 0.5 l/s which gives a $k_f$ value of $1 \times 10^{-7}$ m/s. The case of the Tavera Dam presents the opposite: $Q_{WPT}$ is less than 1 LU yielding $k_f = 0.7 \times 10^{-7}$ m/s, the seepage ranges at about 6 l/s yielding $k_f = 0.5 \times 10^{-6}$ m/s.

Further examples are available confirming the incomparability between the measured and the calculated $k_f$ value. All this proves that a simple conversion does not yield reliable results. Nevertheless, as the engineers need a $k_f$ value to assess the amount of underseepage, such a conversion is often done. Provided the under-seepage remains small enough, the actual $k_f$ value is usually not re-calculated. If the method appears to give a satisfactory value, the true situation is rarely analyzed and remains undiscovered.

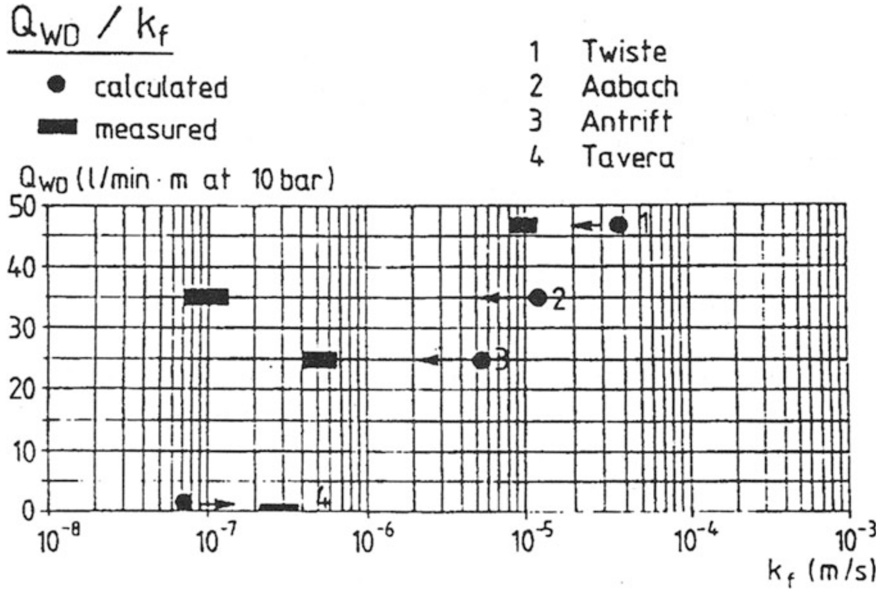



**Fig. 4.13** Comparison of calculated and measured $k_f$-values confirm the impossibility to convert WPT-values into $k_f$

## 4.7 Assessment of the Usefulness of WPT's

### *4.7.1 Permeability*

The Lugeon-Test is the only practical method to investigate the permeability of jointed rock. Due to the ambiguous nature of this test, the anisotropic distribution of water paths and to their steadily changing geometry, the coefficient of permeability $k_f$ cannot be determined quantitatively. Therefore, we still use $Q_{WPT}$-values to describe the permeability. But a critical analysis discloses that their usefulness is also limited.

*WPT's do not yield the coefficient of permeability ($k_f$)*

- A definite conclusion concerning the permeability is only possible in case of no water takes at all. Provided such results representatively reflect the state of the rock mass, the rock can be considered impervious.
- A practical conclusion is also possible when the water takes are extremely large, then a sealing is required and warranted, unless special features of the hydro-geological regime prove the foundation between upstream and downstream sufficiently impervious.

- While no water absorption proves the rock impervious, low or high Lugeon-values do not characterize quantitatively the permeability.
- A comparison of different rock types allows the dam engineer to establish a relative scale which, however, cannot be related to the coefficient of permeability. This can be calculated only when the seepage is recorded, i.e. after the filling of the reservoir.
- WPT's are valuable to prove the success of grouting, irrespective of the actual coefficients of permeability before and after grouting.

*While the test is useful to recognize*

→ impervious rock,
→ very permeable rock,
→ the (relative) success of grouting, and a
→ comparison of various rock types,

it does not yield a reliable coefficient of permeability. We should therefore consider several other factors to decide whether impermeabilization by means of grouting is needed as described later.

### 4.7.2 Deformability

Similar to a flat jack test, WPT's reveal the deformability of the rock by an over-proportionate increase of the water absorption. This increase can be slight or very distinct, thereby indicating a smaller or a greater deformation, respectively. During the decreasing pressure steps, the over-proportionate absorption may decrease again to the original amount indicating that an elastic deformation prevailed; vice versa, a final remaining deformation discloses that the deformation also comprises a plastic component. The P/Q-diagrams in Fig. 4.14 are examples of both types.

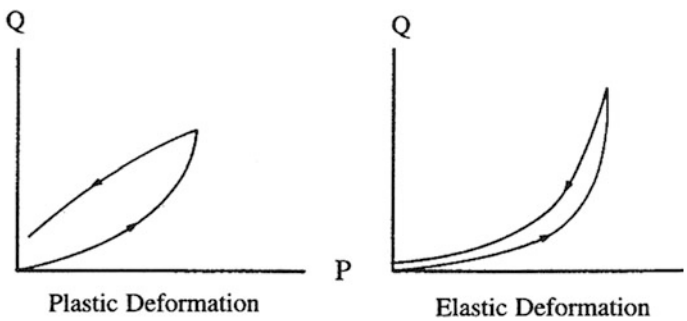



**Fig. 4.14** P/Q-diagrams indicating dilation of water paths

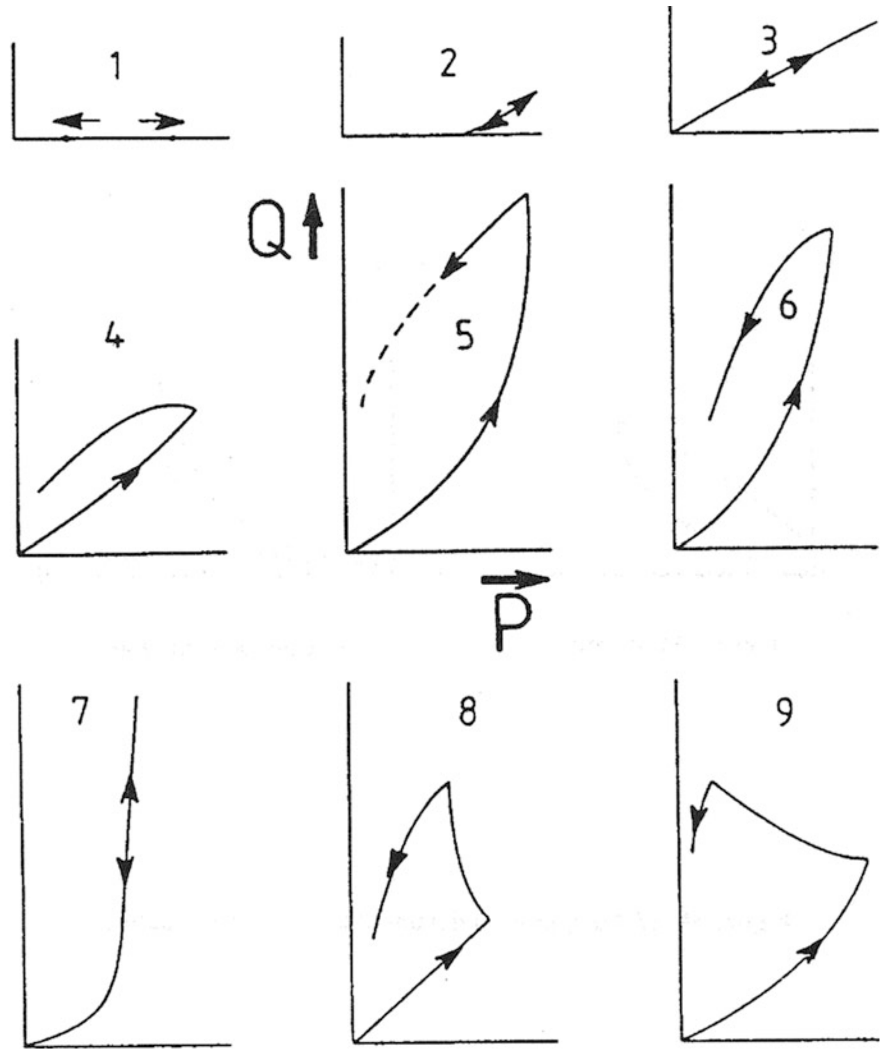



**Fig. 4.15** Selection of frequent P/Q-diagrams indicating different forms of deformation

Figure 4.15 displays a variety of frequent P/Q-diagrams either one caused by its own type of deformation. They confirm the usefulness of P/Q-diagrams to assess the deformation behavior. Thus, P/Q-diagrams are likewise an important tool to define the appropriate grouting pressure and help to assess the individual groutability. WPT's are now considered almost more important for the exanimation of the deformability than for the permeability. Details are presented later.

## 4.8 Typological Classification by Means of WPT's

In the author's experience P/Q-diagrams can be successfully used to understand better the hydrogeological setting in view of the geometry of the water paths, and the impact of the testing and grouting pressure on the rock, i.e., the deformation behavior. Vice versa, certain rock properties yield typical P/Q-diagrams identifying the rock type. This is helpful to examine the permeability and the individual groutability. If a rock mass were absolutely tight and all WPT's showed no water absorption at all, only P/Q-diagrams of Group 1 in Fig. 4.11 would be obtained and grouting would not be required. On the contrary, if all test hole stages met open fissures and the rock had a very high strength, WPT would yield P/Q-diagrams of Group 4 in Fig. 4.11: The rock is permeable and groutable and a grouting program would have to be carried out.

Of course, neither the first nor the second case will ever occur in such a pure form. Nevertheless, comparative studies reveal that rock types are characterized by a typical individual combination of a few P/Q-diagrams. This combination specifically characterizes the permeability and the deformation behavior of this particular rock type. Comparative studies revealed that rock types can be identified—and classified—by their typical P/Q-diagrams. Thus, the idea was to establish a typological classification. Meanwhile approximately 60 test programs have been evaluated in this respect. The results confirmed the possibility of such a classification which contributes to a better assessment of the relevant characteristics. This typological classification will be demonstrated by the following examples; the discussion disregards the eventual need for contact grouting and refers to curtain grouting only.

### *4.8.1 Massive Conglomerate with Intercalated Siltstone Beds*

The rock at the Tavera Dam is largely impervious but includes very few zones of little permeability caused by master joints with sandy infillings. Due to the low strength the critical pressure range, independent of depth, between 5 and 10 bar (Fig. 3.2). WPT-results and the state of the outcrop confirmed each other.

Such a rock needs no treatment because the overall permeability of the whole foundation is very small and grouting could not improve the state of the rock any further. The few permeable zones are not groutable while high-pressure grouting could only produce groutstone pillows intercalated along the hydrofractured siltstone beds.

### 4.8.2 Well Bedded Sandstone-Siltstone-Alternations

The rock at Antrift, Haune and Twiste Dams is permeable due to steeply inclined extended master joints; small joints are closed, horizontal latent bedding planes are covered with mica and possess little tensile strength making this rock highly susceptible to hydrofracturing along the beddings. Two types of P/Q-diagrams prevail:

- Fig. 4.16a indicates originally permeable rock. Once the very low critical pressures ($\leq 3$ bar) are reached the latent bedding planes are hydrofractured producing an over-proportionate increase of the water take.
- Figure 4.16b shows a borehole stage without connected open fissures but closed joints and beddings; this rock is impervious. Low pressures initiate hydrofracturing and the rock becomes permeable along the fractured bedding planes.

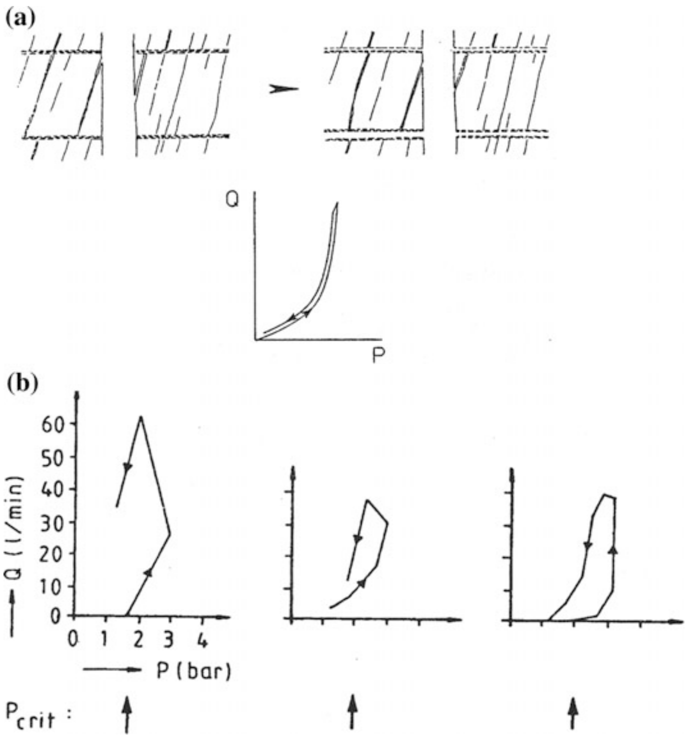



**Fig. 4.16** **a** Typological classification of P/Q-diagrams: sandstone-siltstone with pronounced bedding planes prone to hydrofracturing, permeable rock section is fractured along the bedding after reaching the critical pressure. **b** Typological classification of P/Q-diagrams: sandstone-siltstone with pronounced bedding planes prone to hydrofracturing, tight rock section is fractured along the bedding after reaching critical pressure

Such a foundation is very permeable. The distinct fissility along the bedding planes causes a poor groutability. Very low grouting pressures should be used to minimize hydrofracturing. With low pressure grouting only the wide master joints can be grouted, all smaller paths necessarily remain unsealed leaving a considerable residual permeability. Other means are required to reduce this unless the conditions of the project provide for a low hydraulic gradient which may lead to an acceptable under-seepage.

### 4.8.3 Carboniferous Shale with Intercalated Sandstones

The rock at the Möhne Dam is characteristic of the permeability and the hydrofracturing behavior of this rock type (Fig. 3.17). Some borehole stages were tight, others absorbed little water. Hydrofracturing occurred in 83% of all sections with critical pressures between 6 and 10 bar. The critical pressures were completely independent of the depth. In such a case the height of the dam and the hydraulic gradient determine whether a treatment is necessary. High grouting pressures were not appropriate, thus a rather close spacing of the groutholes would have to be chosen.

### 4.8.4 Slightly Karstic Limestone

The rock at the Panix Dam yielded P/Q-diagrams displayed in Fig. 4.17. Successive WPT's produced hydrofracturing at similar pressures ranging between 10 and 15 bar irrespective of their depths. Such a rock needs no treatment unless a dam more than 100 m in height were envisaged and karstification more developed.

### 4.8.5 Competent Granite

The rock at Huites Dam showed an unexpected susceptibility to hydrofracturing. WPT's disclosed that this rock is practically impervious because 96% of the borehole stages were tight. The fissures met in the permeable stages (4%) were dilated at low pressures (2–5 bar). In 32% of all stages pressures between 8 and 16 bar initiated hydrofracturing caused by a laminated texture. The major part of all sections (60%) resisted the highest pressures applied without cracking. Figure 4.18 illustrates the situation. Such a rock is practically impervious and therefore needs no treatment even if the dam were higher.

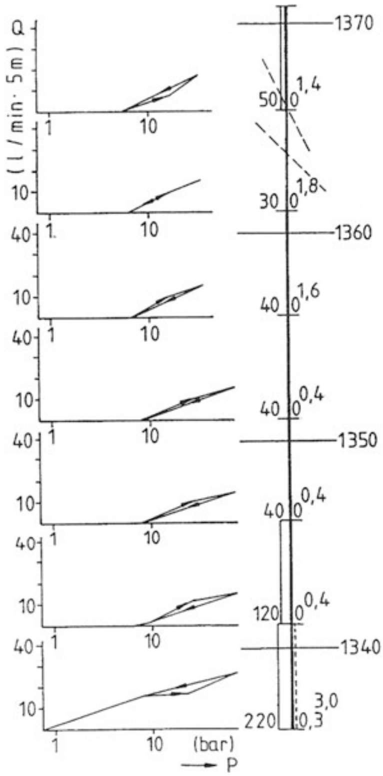



**Fig. 4.17** Typological classification of P/Q-diagrams. Limestone fractured at very similar pressure

## 4.8.6 *Miocene Molasse*

The rock at Eglisau Dam consisting of various layers of sandstone, claystone, marlstone and limestone (altogether named '*Molasse*') showed a different fracturing behavior. Figure 4.19 presents various P/Q-diagrams. While a very shallow zone is slightly to distinctly permeable, the rock below is impervious. The fracturing occurs independent of depth but the various rock types show different critical pressures: Some marlstone banks are cracked at pressures below 5 bar, several sandstones needed 5–7 bar, claystone fractured at approximately 12 bar. For such a geological condition a treatment should be envisaged only in the uppermost permeable zone, the rock further below needs no improvement nor could that be achieved.

## 4.8.7 *Well Bedded Quartzite*

The rock studied at a project site in Taiwan is either tight or little permeable. Due to the high strength neither dilation of existing paths nor hydrofracturing of the

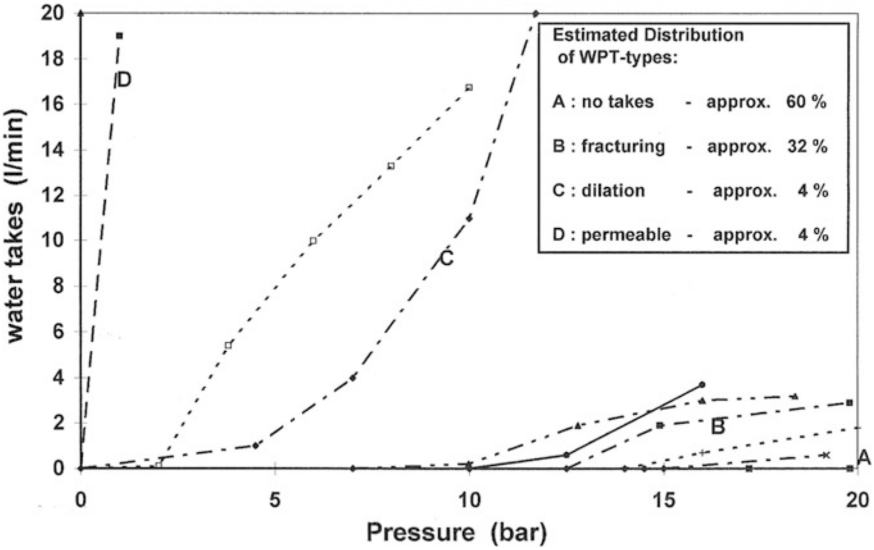



**Fig. 4.18** Typological classification of P/Q-diagrams. Competent granite fractured at medium-sized pressures due to laminated texture

bedding planes took place. Figure 4.20 shows the P/Q-diagrams reflecting this favorable situation. Such a rock needs no treatment although very high grouting pressures could be applied.

### *4.8.8 Massive Competent Limestone*

The rock at the Pueblo Viejo Dam is practically tight except at the left abutment where relaxation provided for a moderate permeability. The limestone includes karstic cavities, smaller and very large ones exist side by side; most of them are filled with volcanic ashes. This rock is characterized by the P/Q-diagrams displayed in Fig. 4.21: (a) impermeable, (b) slightly permeable and (c) hydrofracturing at very high pressures ($\geq 30$ bar) (Fig. 4.22).

Such a rock confronts the engineer with peculiar conditions: The impervious rock itself strictly needs no treatment, but the permeable section under the abutment requires conventional grouting and the karstic cavities have to be filled. If one had to reckon with many cavities of smaller widths and extension possibly traversing the foundation from upstream towards downstream then it would be necessary to choose a close grout hole spacing and to apply high pressure grouting in order to find access to the cavities by an intentional hydrofracturing.

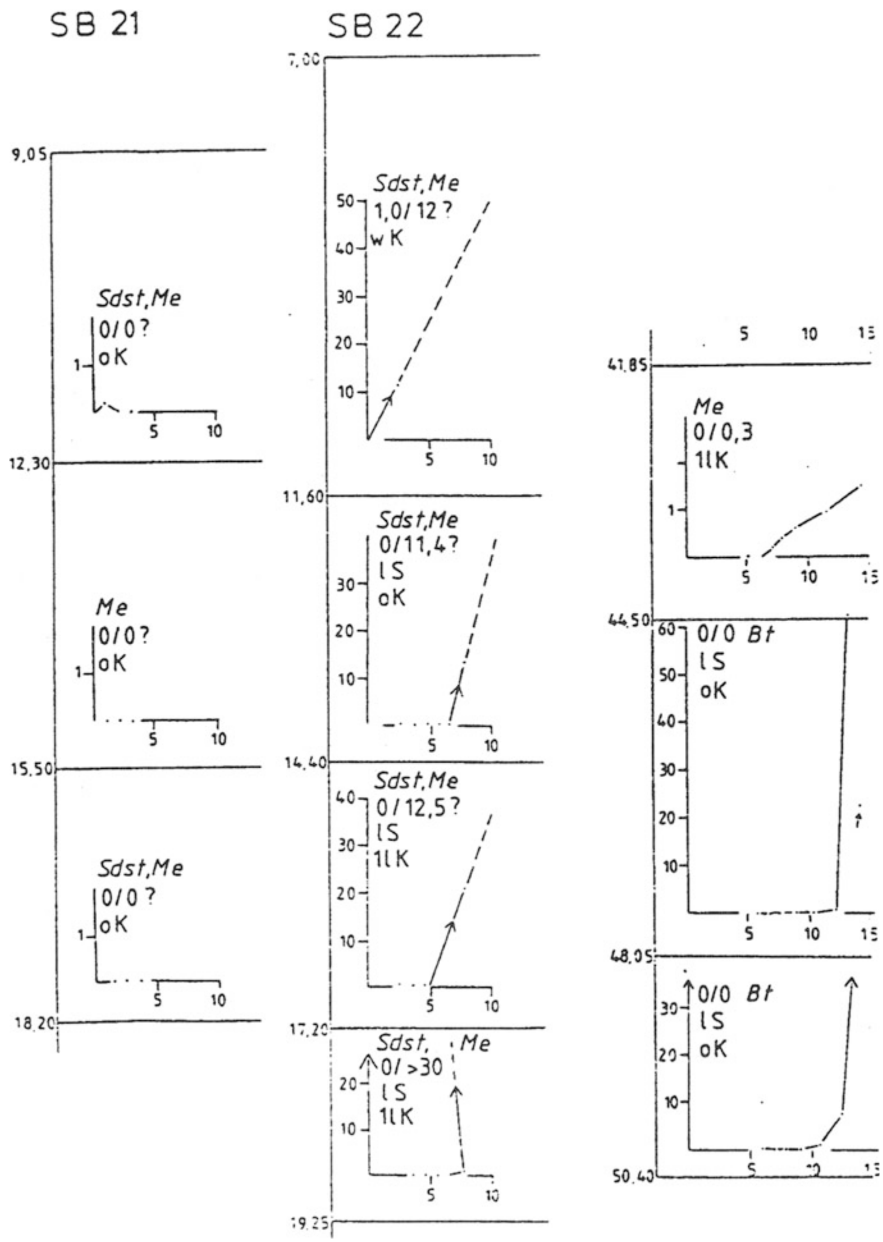



**Fig. 4.19** Typological classification of P/Q-diagrams. Well bedded sandstone, claystone, marlstone and limestone susceptible to fracturing at different critical pressures

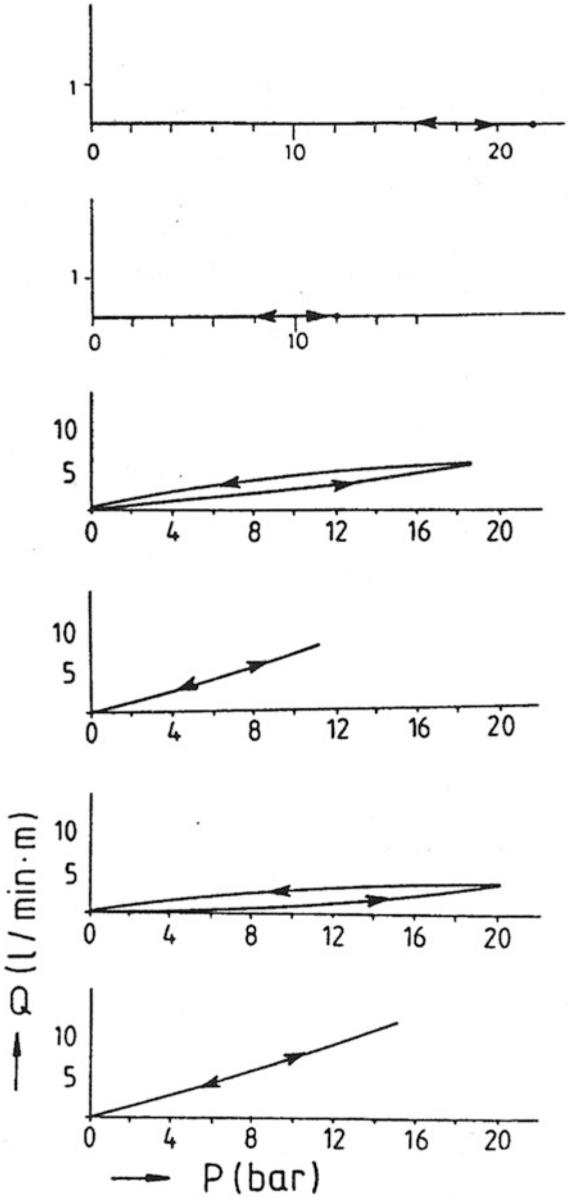



**Fig. 4.20** Typological classification of P/Q-diagrams: well bedded hard quartzite, light to slightly permeable, without fracturing

## 4.9 Impermeabilization Criteria Based on WPT-Values

The velocity of the percolating water (erosion) and the amount of under-seepage (water losses) are calculated on the basis of Darcy's law which requires a coefficient of permeability ($k_f$). The methodological problem of the conversion $Q_{WPT}$ to $k_f$ has

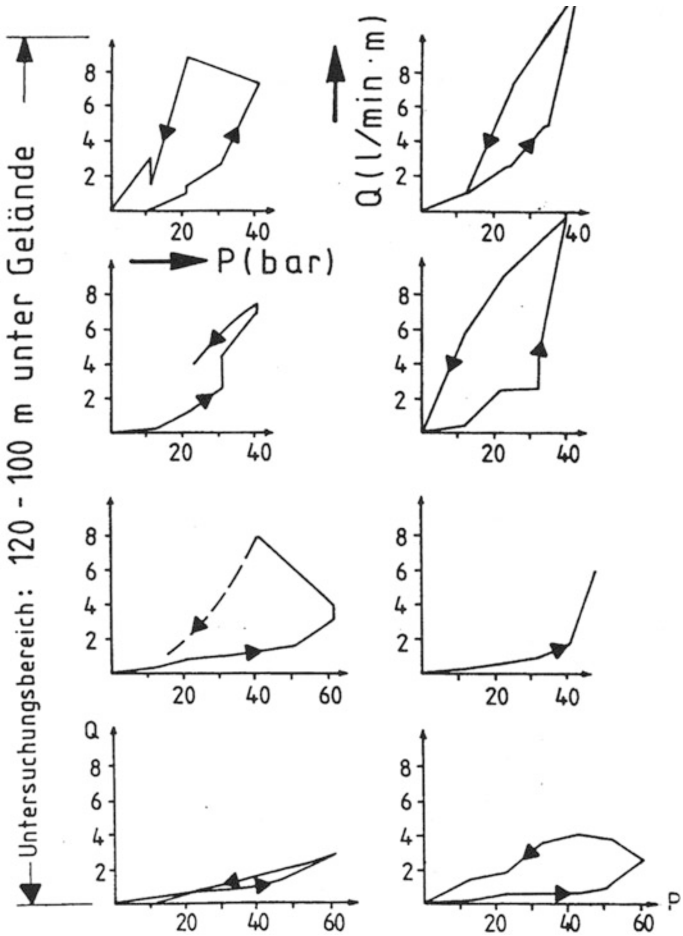



**Fig. 4.21** Typological classification of P/Q-diagrams: massive limestone, hard, fracturing occurs at very high pressures

already been discussed. In view of these difficulties and being unable to establish a $k_f$-value for given cases we still use the WPT-value as the basis for the decision on a grouting program.

Lugeon (1933), introducing the WPT, concluded that the rock is practically impermeable if the absorption is less than 1 l/min/m at 10 bar. This was understood in such a way that in all opposite cases the rock had to be sealed. Several authors formulated their criteria which mostly followed Lugeon or differed only slightly. Altogether they shared the philosophy that rock can be sealed up to the residual permeability being the order of the criteria. The absorption rate of 1 LU was the widely acknowledged limit of the tolerable rock permeability, in case of higher WPT-values a grouting program was considered necessary. In some projects less conservative criteria have been applied, but, quite often the '1-*Lugeon-rule*' dominates. And it is applied even where a favorable hydrogeological regime makes sealing unnecessary.

The application of this absolute absorption rate as the basis for the decision raises a principal question. Low WPT-values justify the conclusion that the rock is (almost) impermeable, but how permeable is the rock in case of medium or high WPT-values, or from which WPT-values onwards do we have to reckon with too high a permeability? Dam sites of negligible seepage are known although the WPT values were quite high. Thus, in case of high WPT-values the situation is not at all clear.

Modified values for the acceptable permeability have been proposed: Heitfeld argued that the type of the dam as well as the geological setting should be considered. Houlsby proposed even more differentiated impermeabilization criteria also taking into consideration the value of the water. Nevertheless, both Heitfeld and Houlsby—and many other authors—still proposed for certain cases very stringent WPT-values (1-2 LU). This inherently presupposed that all rock types are similarly groutable which really means that the permeability can be equally reduced to that value. This conception disregards that rock types have their individual groutabilities which often do not permit a reduction of the permeability by cement suspension grouting.

Almost 30 years ago we began to recognize that rock masses have their own groutability which should be considered in order to improve the success and the economy of the treatment. This groutability is closely connected with (a) the size and the shape of the fissures as they determine the grouting pressure required for penetration and (b) the strength of the rock mass as this controls the hydrofracturing behavior and limits the maximum grouting pressure.

Furthermore, the hydrogeological regime is also important and should be taken into account. Considering the existence of an individual groutability and the influence of the hydrogeological condition of the dam site the authors used to apply the following classification

- Absorption rates smaller than 5 LU describe a practically ungroutable and sufficiently impervious rock type; the water conductivity is necessarily caused by a few narrow paths inevitably producing a low $k_f$-value.
- WPT values above 20 LU indicate rock types which may be groutable (in many cases the practical groutability begins at higher values): this rock should be sealed by grouting before impounding the reservoir, unless a favorable hydrogeological regime provides for a sufficiently low overall permeability of the foundation between upstream and the downstream relief—or a test grouting disclosed that the rock is not groutable.
- WPT values ranging in between are considered questionable cases. It is not definite neither is it sure that the permeability is low enough nor that grouting is required and helpful. The decision on the appropriate measures depends on the local conditions:

* In conditions where the seepage paths between upstream and downstream are short, the rock tends to be erodible, the hydrogeological regime is unfavorable and a grouting gallery does not exist (making a later treatment impossible) a grout curtain would have to be installed prior to the impoundment.
* In different conditions (a gallery exists, long seepage paths, resistant rock. favorable hydrogeological regime) the decision on grouting can be postponed until the actual seepage indicates the true permeability. This approach does not bear any risk but implies the chance of reducing the scope of the treatment—or abandoning it completely.

The LU-values mentioned above have to reflect representatively the permeability of the whole rock mass. WPT's carried out in a rock of a very variable permeability—karstic limestone, for instance—yield an untypical mean value which does not really characterize the true permeability but gives a wrong impression, as in the following example. A mean value of 100 readings is 4.2 LU and this apparently indicates a low permeability. In fact, there are 95 values of 1.5 LU and 5 values of 55 LU, which identify a nearly impervious rock containing a few but considerable openings which, however, may cause unacceptable seepage losses.

Ewert presented these criteria for the first time in 1981. Meanwhile other colleagues came to similar views: Kutzner proposed a similar classification taking the limits at <5 (to 10) LU for the sufficiently impervious rock and >25 (to 30) LU for permeable and groutable rock while WPT values in between cover cases which need particular investigations. Such an investigation has to consider, above all, all components of the hydrogeological regime; several examples presented above demonstrate that the careful consideration of the hydrogeological regime is a promising approach in minimizing the costs. The essential hydrogeological components experienced in several projects are described in Chap. 7.

## 4.10 Directions of Unequal Permeability

It was the original purpose of Lugeon's WPT to examine the permeability of the rock around the borehole section tested. This task remains. In addition, certain geological settings allow to obtain further valuable information: Rock types have often not a similar permeability in all directions, particularly when the rock is well stratified or possesses parallel joint sets.

For certain purposes it might be useful to determine the more and/or the less permeable directions: For instance, which direction yields the greatest uplift reduction? Figure 5.1 shows an example elaborated at the Aabach Dam: WPT's were carried out in groups of three boreholes: Type A—parallel to the bedding, Type M—diagonally to the bedding, Type C—at right angles to the beddings.

The results confirmed the expectation: WPT's in A-holes–pressure reduction towards M-and B-holes, WPT's in B-holes–pressure reduction towards M-and A-holes.

These tests showed convincingly that the flow across the beddings yielded a much larger pressure reduction and thus contributed to the decision to abandon the grout curtain. This was already substantiated in detail in Sect. 3.6, Fig. 3.29 shows how the differing geological setting along the control gallery determines the uplift reduction towards downstream.

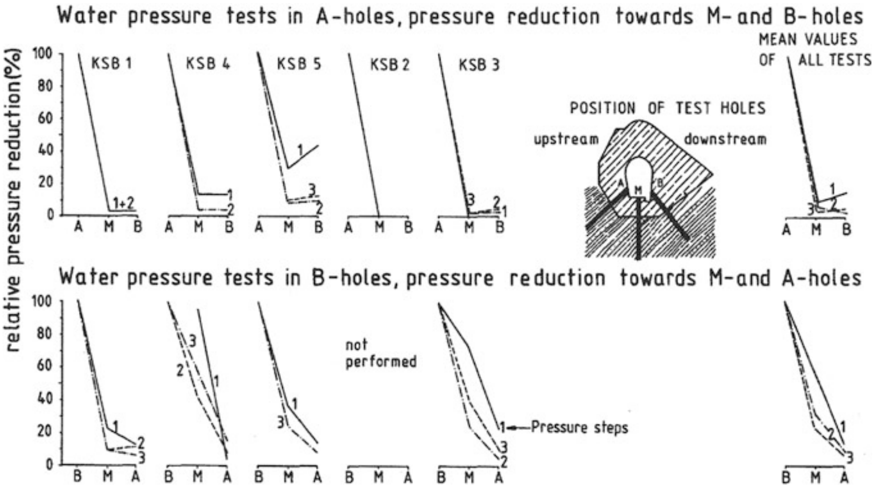



Fig. 4.22 Different directions of WPT to examine different uplift reduction

# Chapter 5
# Hydrofracturing of Latent Discontinuities in Rock and Implications for Successful and Economical Execution of Grouting

## 5.1 Introduction

During the injection of grouting materials into rock, open paths are dilated or closed discontinuities are pressed open once the grouting pressure exceeds the resistance of the rock surrounding the borehole. The dilation of existing paths is usually called '*Hydrojacking*', the cracking of latent discontinuities '*Hydrofracturing*'. Both occurrences are momentous, particularly in the sphere of dam construction: a larger permeability of the rock mass is pretended often causing unnecessary impermeabilization or too extensive grouting measures.

Cases of hydrofracturing occur frequently, largely due to the practice used worldwide to increase the grouting pressure with depth. This causes hydrofracturing and backfilling of latent discontinuities particularly in deeper sections. Such grouting wastes time and money, which could be avoided if it were considered that latent discontinuities in many rock types are hydrofractured at rather low pressures already even in greater depths.

About 30 years ago the author emphasized the negative consequences of hydrofracturing for the technical and economical success of grouting programs. The examples available at that time comprised the few rock types only, here presented already in Chap. 3. Given the fact that this complexity even today is not duly considered, it is time to continue the discussion also because since then a multitude of further projects became available providing important knowledge on the hydrofracturing behaviour of almost all rock types. It confirms the former conclusions and proves their universal validity.

Now data sets from permeability tests and grouting carried out worldwide in approximately 80 projects are available for this discussion. Some of the sets are fragmentary, others are complete. The treasure of data is so extensive that the scope required for a completely comprehensive evaluation of all these projects reaches far beyond the available space. The number of projects considered for this contribution and the scope of evaluation has to be restricted, nevertheless a sizable number of



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projects has been evaluated to allow a comparative analysis. This selection comprises all rock types. Due to the limited space this contribution deals exclusively with the dilation of existing water conducting paths and, above all, with the fracturing behaviour of latent discontinuities; all other aspects necessarily remain disregarded although they were worth to be discussed as well. The results of the evaluation confirm the necessity of this analysis because the conclusions are really momentous. They hopefully contribute to improve inadequate grouting concepts, reduce bad investment and lower too great expectations in the effectiveness of grouting measures.

It is often assumed that the overburden pressure, i.e. the weight of the overlying rock determines the resistance of the rock surrounding the borehole against deformation and fracturing. If so, the resistance would increase with depth and a stronger resistance would allow that the grouting pressure could increase with depth accordingly without dilating open fissures of fracturing latent discontinuities. However, the foregoing chapters presented examples which do not confirm that concept, but indicate that it is often not the overburden pressure causing the resistance against fracturing but it is the strength of the rock around the borehole section treated. This strength as the decisive factor does not increase with depth, provided the type of the rock remains about the same. As long as the strength does not change with depth, the critical pressures determining fracturing remain the same as well, irrespective of the increasing depth. Hence it follows that with increasing depth the likewise increasing grouting pressure exceeds the strength of the rock and fractures latent discontinuities.

## 5.2 Problem

Since about 70 years the permeability of rock masses is usually determined by means of WPT's introduced by Maurice Lugeon. Water is pressed in ascending and descending pressure steps into isolated sections of boreholes. The amount of water absorbed serves as a measure describing the permeability. In reality the tests do not determine the permeability of the entire rock mass but only the absorption capacity of the rock surrounding the hole.

Number and height of pressure steps are selected in accordance with the conditions of a project. The amount of absorption describing the permeability is given for a reference pressure for which LUGEON used 10 bar. The LUGEON-values (or WPT-values) are given in LUGEON-units (LU) defined as liter per meter and minute at a pressure of 10 bar (l/min m 10 bar, or l/min$^{-1}$ m$^{-1}$10 bar$^{-1}$; 1 l/min m 10 bar = 1 LU). This unit is still largely in use but sometimes individually defined reference pressures are applied to meet particular project conditions.

Using a reference pressure of 10 bar, LUGEON concluded that rock would be practically tight if at this pressure less water is absorbed than 1 LU. This quantity became the worldwide accepted LUGEON-criterion. Unfortunately, it inherently implies a problem: WPT-results of less than 1 LU identify certainly a tight rock, and hence it will be decided to install a grout curtain. And it is done about everywhere. However, if WPT-results are a little larger than 1 LU is that rock already permeable? And from which quantity of water absorption—LU-value—onwards is the rock so permeable that grouting is really required and warranted? The decision to grout depends on several factors. All should be considered otherwise the LU-criterion remains the only one. If so, that criterion might function as a trap. Practical examples often show sizeable overdoing.

Hydrojacking, i.e. the dilation (or widening) of existing paths begins in rock types of little strength at pressures smaller than 10 bar. It causes an over-proportionate increase of the water take already before the reference pressure is reached. Under such conditions the LUGEON-value referring to 10 bar pretends a larger permeability compared to the original one.

This discrepancy impairs our assessment. In rock types of great strength, the dilation of existing paths begins at pressures above 10 bar, thus the absorption rate at the reference pressure reflects still the original permeability.

Hydrofracturing splits latent discontinuities producing a fissure as soon as the testing or grouting pressure reaches the critical pressure, different from case to case. It causes a much larger effect of pretence: the latent planes absorb no water during the low-pressure steps but large amounts after fracturing. Figure 2.1 exemplifies both cases. It is obvious that the wrong assessment of the original permeability has considerable consequences.

Hydrojacking and hydrofracturing are particularly effective in grouting work where even higher pressures are usually applied. The results of dilation and fracturing have to be valued differently:

- Hydrojacking and backfilling of dilated joints by grouting usually yields a better impermeabilization because the grout travels farther, more excess water is pressed out and a better adhesion along the contact between groutstone and rock is achieved. Thus, dilation yields a positive effect unless too much grout is needed before the maximum grouting pressure desired is reached.
- Hydrofracturing usually produces too large grout takes without yielding a better impermeabilization. Sometimes even huge quantities of grout are injected without achieving any effective sealing. Fracturing is particularly momentous if the grouting pressure increases with depth because it exceeds the critical pressure even in rather strong rock types. Therefore, there exists a close interrelation between the fracturing behaviour and the optimal grouting pressure. Therefore, it is expedient examining individually that behaviour to define the adequate grouting pressure. Considering the negative effect of hydrofracturing, it is time now to improve our knowledge and take it into account much better than done in the past.

Hydrojacking becomes obvious by the over-proportionate increases of the water takes in WPT's, particularly during their higher-pressure steps. Widening of open joints is a continuous process as illustrated by the P/Q-diagram shown in Fig. 5.1, left. By contrast, hydrofracturing of latent discontinuities occurs when the test or grouting pressure becomes high enough to exceed the critical pressure (Fig. 5.1, right).

## 5.3 Basis of Examination

If a borehole section is tested and grouted the fracturing processes can be recognized particularly well because the grout enters and fills the discontinuity pressed open. Figure 3.10 shows a rather spectacular example caused by peculiar conditions ruling the curtain grouting at the Twiste Dam. The details are presented in Sect. 3.2. The conditions for grouting were difficult:

- The beddings lie horizontally, the rock is heavily jointed and the main joints are vertical.
- Inclined holes were extremely unstable, vertical holes were required to minimize permanent clogging.
- The bedding planes between sand- and siltstone are covered with mica, thus they are extremely susceptible to fracturing. In the uppermost zone of the curtain the latent bedding planes were pressed open and backfilled at a pressure of 2 bar already. The P/Q-diagram shown in Fig. 3.10 revealed the low pressure initiating the fracturing.

These extremely small critical pressures were unknown beforehand. Therefore, the following concept was used to install the grout curtain: Initially all groutholes were drilled through the alluvial sediments just into the rock. These sections were furnished with casings. The packer was fastened always at the end of the casing, i.e. quite close to the surface. Downwards grouting was carried out. Hence, while the packer remained in its position, the consecutive sections were drilled and successively grouted, with re-drilling in between. However, the second and the third section received almost no grout while the first one was fractured and grouted repeatedly because the counter-pressure was the smallest directly below the packer.

The examples displayed in Figs. 3.10a, b and 5.1 prove that dilation and fracturing processes are expressed by P/Q-diagrams, thus they can be examined by WPT's provided multiple pressure steps are carried out, single pressure steps do not disclose such developments.

The accuracy of WPT's, i.e. their P/Q-diagrams grows with the number of pressure steps as the examples shown in Fig. 5.2 confirm. As many steps as possible are desirable, but for economic reasons their number must be limited (Fig. 5.2a, b). The large number of pressure steps applied in the test shown in Fig. 5.2c is particularly informative but can be realized in exceptional cases only.

The tests evaluated for this analysis contain 2 or 3 pressure steps at most—1–5–10 bar, for instance. Hence, the critical pressure can be ascertained only approximately. Therefore, the pressures given in tables and figures describe the order of magnitude, mostly not the precise results.

The P/Q-diagram shown in Fig. 5.2c comes from the investigation holes drilled for the project Las Portas. The graph presents a tight sequence of ascending and descending pressure steps. Figure 5.3 shows all the 6 WPT's carried out in that hole. In these graphs the descending pressures steps have been arbitrarily eliminated, leaving only the relation P/Q of the ascending steps. Five tests show

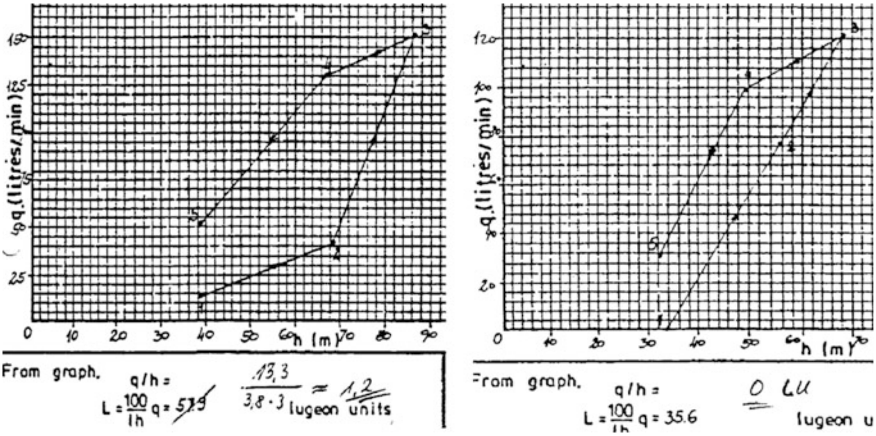
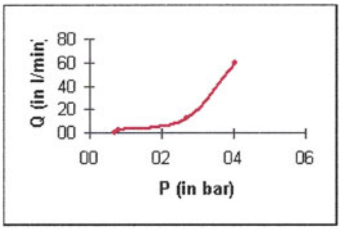



**Fig. 5.1** P/Q-diagrams illustrating dilation of existing path (*left*) and fracturing of latent plane (*right*)

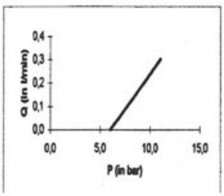
| Hole | W.L. *in m* | Depth *in m* | $P_M$ *in bar* | $P_T$ *in bar* | Q *in l/min* | $Q_{1/10\ bar}'$ *in l/(min*m)* | $Q_{1/10\ bar}^*$ *in l/(min*m)* |
|---|---|---|---|---|---|---|---|
| S-2a | 7 | 5 to 10 | 00 | 01 | 00 | | |
| | | | 0,1 | 0,8 | 2,5 | 0,6 | |
| | | | 2,0 | 2,7 | 11,9 | | |
| | | | 3,5 | 4,0 | 60,0 | | 3,0 |
| | | | | | | **0,6** | |
| | | | | | | **6,4** | **29,8** |

**(a)**




| | | | | |
|---|---|---|---|---|
| 5,0 | 6,1 | 0,0 | 0,0 | |
| 10,0 | 11,1 | 0,3 | | 0,0 |
| 5,0 | 6,1 | 0,0 | | |
| | | | 0,0 | 0,0 |
| | | | 0,0 | 0,1 |

**(b)**




| | | | | |
|---|---|---|---|---|
| 00 | 00 | 00 | | |
| 1,0 | 1,1 | 0,0 | 0,0 | |
| 3,0 | 3,1 | 0,0 | | |
| 6,0 | 6,1 | 0,0 | | |
| 7,0 | 7,1 | 0,1 | | |
| 8,0 | 8,1 | 0,1 | | |
| 9,0 | 9,1 | 0,0 | | |
| 10,0 | 10,1 | 0,1 | | |
| 14,0 | 14,1 | 0,1 | | |
| 15,0 | 15,1 | 0,2 | | |
| 16,0 | 16,1 | 0,3 | | |
| 18,0 | 18,1 | 0,8 | | |
| 23,0 | 23,1 | 0,8 | | 0,0 |
| 20,0 | 20,1 | 0,6 | | |
| 18,0 | 18,1 | 0,3 | | |
| 16,0 | 16,1 | 0,1 | | |
| | | | 0,0 | |
| | | | 0,0 | 0,1 |

**(c)**

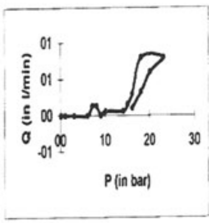



**Fig. 5.2** Number of pressure steps determine accuracy of tests and their P/Q-diagrams

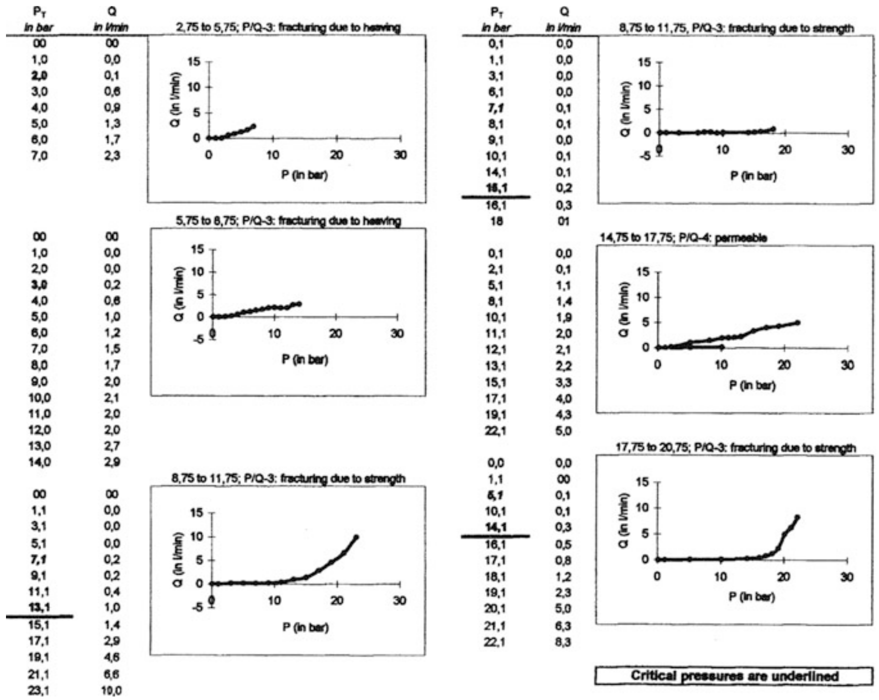



**Fig. 5.3** Sequence of P/Q-diagrams originating from WPTs in Hole S-1 drilled at Las Portas

fracturing: In the two tests carried out up to a depth of approx. 9 m the critical pressures of 2 and 3 bar were controlled by heaving.

The fracturing of the deeper tests was controlled by the strength of the rock—with the interesting phenomenon that fracturing took place in two subsequent phases. The initial fracturing took place at 5 and 7 bar with a very small splitting yielding a very little water absorption only, the main fracturing occurred at critical pressures between 13 and 15 bar, producing a larger over-proportional increase of the water take. The same scale used for all diagrams reveals that the intensity of fracturing varies while the critical pressures do not increase with depth. The tight sequence of pressure steps discloses this gradual fracturing. The tests carried out in other rock types presumably undergo a similar gradual cracking which mostly remains undetected because of the few pressure steps usually applied.

All the tests analysed including those presented in Figs. 5.2 and 5.3 have been evaluated by using a special software (GRT2002[1]): After entering P and Q (pressure and quantity) for each step of a test, GRT2002 calculates the absorption parameters $Q_1/Q_{10}$ and $Q_1^*/Q_{10}^*$, respectively and plots the P/Q-Diagram.

[1]GRT2002 is a still unpublished software established by Ewert to (a) evaluate the data obtained in WPT's and grouting and (b) to plot their results.

Contrarily to the conventional LUGEON-criterion these parameters permit assessing not only the permeability but also the deformation behaviour.

## 5.4 Classification of P/Q-Diagrams

The various types of P/Q-diagrams reflect different courses of the tests and, hence, individual geotechnical conditions. The types have been repeatedly classified. The author presented such a system at first in 1979, which subsequently has been adapted to the growing knowledge during the following years. This analysis considers the classification system established in 1992 and successfully used since then; it is introduced in Fig. 4.11 and here finally presented in Fig. 5.4. The P/Q-diagrams given in Fig. 5.3 exemplify two types of this classification.

## 5.5 Factors Ruling Hydrofracturing and Hydrojacking

The rock bond, the state of the discontinuities and the strength properties control whether hydrofracturing or hydrojacking occur:

- In a surface near seam weathering reduced the strength along the discontinuities and disconnected the rock bond, i.e. the rock units bordered by joints, bedding

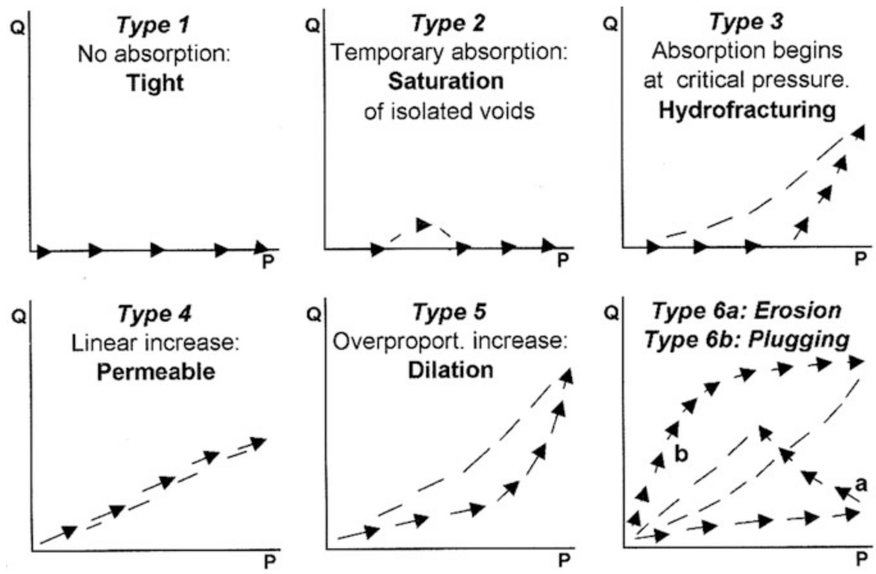



**Fig. 5.4** Classification system of P/Q-diagrams identify geotechnical conditions

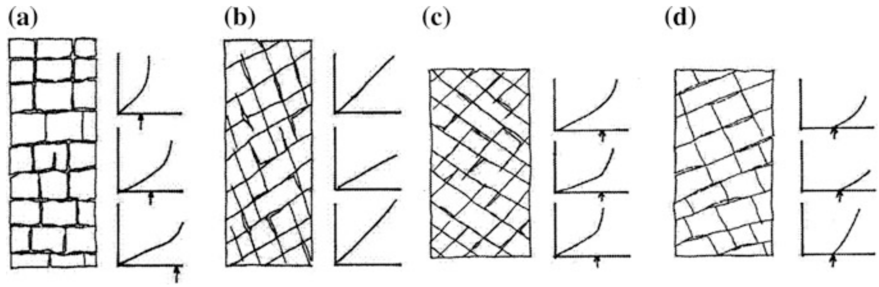



**Fig. 5.5** Factors ruling hydrojacking and hydrofracturing depend on the state of the rock bond; from **a** (*left*) to **d** (*right*)

planes and cleavage planes are separated from each other. The separation increases with growing disintegration making the rock units movable against each other. The test—or grouting—pressure initiating a heaving of the rock units above a largely disconnected bedding plane, for instance, depends on the weight of the overlying rock causing the overburden pressure. That pressure increases with depth (Fig. 5.5a). Sometimes fresh and solid rock begins already at a depth of a few meters—or even decimetres, sometimes weathering extends over several tens of meters. In most cases the noticeable influence of weathering on the strength ends between ten and twenty meters.

- Further below, with decreasing intensity of weathering the overburden pressure does not determine the critical pressure any more since the rock bond is still connected. Different courses occur depending on the types of the discontinuities available, the degree of separation and the strength of the rock around the section of the borehole being tested or grouted:

  * If water-bearing paths exist and the rock has great strength, the water takes increase proportionately with pressure (Fig. 5.5b).
  * If water-bearing paths exist and the rock has little strength, the water takes increase over-proportionately because hydrojacking takes place dilating the paths. As that strength remains about the same and does not increase with depth, the critical pressure does not increase either; it merely varies within a certain range determined by local factors and independent of the depth. This setting yields P/Q-diagrams exemplified in Fig. 5.5c
  * If all discontinuities are still closed, the most susceptible plane is fractured first as soon as the critical pressure reaches and exceeds the tensile strength across that plane. Since that strength remains about the same the critical pressures do not increase with depth either but vary as well. Figure 5.5d illustrates this situation.

Hydrojacking widens existing paths, thus the ruling factor is the compressibility of the rock bordering the paths. By contrast, hydrofracturing depends on the tensile strength across the discontinuities, the compressibility of the rock and the relation between the orientation of the borehole and the orientation of the most susceptible discontinuities. Their various types have a different susceptibility to fracturing: bedded rocks and slates are particularly prone but also metamorphic rocks possessing a pronounced parallel texture of their crystals.

In order to comply with the main theme of this contribution, the analysis focuses primarily on the hydrofracturing behaviour of latent discontinuities. While this analysis considers only the various rock types involved; the influence of further geological features (orientation and number of joint sets, relaxation etc.) and details regarding the technology of drilling necessarily remain disregarded.

## 5.6 Data Evaluated

A total of 2272 WPT's carried out in 448 boreholes from 20 dam sites located in Spain has been evaluated. The resulting P/Q-diagrams have been classified using the systems given in Figs. 5.4 and 5.5:

Type 1 No absorption, tight
Type 2 Temporary absorption, saturation of isolated voids, accessible from borehole
Type 3 Absorption begins at critical pressure
Type 4 Linear increase, permeable
Type 5 Over-proportional increase, dilation
Type 6 (a) Erosion of infillings, (b) Plugging with removable material

Together with the results obtained for $P_{max}$, $Q_{max}$, $Q_{min}$ and $P_{crit}$, the portion of the various types has been determined and listed in Table 5.1.

The relative portions of the Types 1–6 have been figured out and graphically presented in form of circle diagrams. For Types 3, 5 and 6a (hydrofracturing, hydrojacking and washing out) the relation between critical pressure and depth has been plotted in form of point diagrams. $P_{crit}$ received always the same scale to make the graphs comparable. Several diagrams contain a few points only—either due to a small number of WPT's or to the little shares of Types 3, 5 and 6. Both, the circle diagram and the point diagram form one Fig. 5.6, exemplified here. For each case brief comments are added to the diagrams to focus on the results of the evaluation; further geological and technological details remain disregarded.

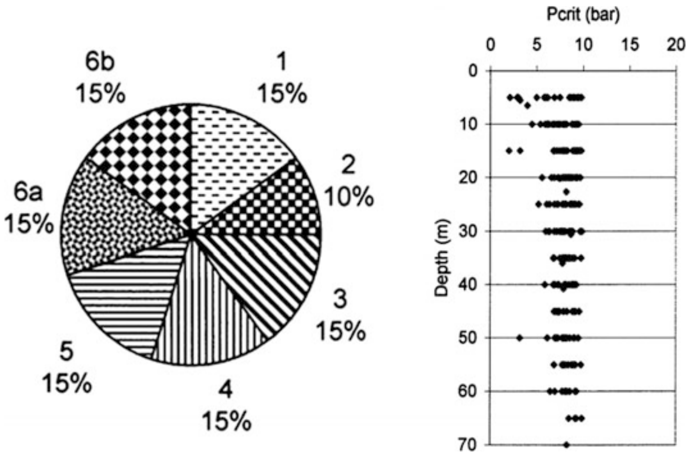



**Fig. 5.6** Method and results of evaluation. *Left* Shares of Diagram-Types. *Right* Distribution of critical pressures with depth

## 5.7 Results

### *5.7.1 Aixola*

The data of 17 WPT's come from a program aimed at investigating the foundation of a small rock fill dam: it consists of Flysch of Upper Cretaceous: thick sandstone banks (1–1.5 m) are intercalated with thin marlstone and limestone banks, locally karstic. The beddings are steeply inclined towards the reservoir (70°).

WPT's were carried out down to a depth of 20 m. Large water takes restricted the maximum test pressure to 4.2 bar. P/Q-diagrams of Type 5 prevail: dilation (Table 5.1; Fig. 5.7). Pcrit increases with depth down to 10 m; within this zone the courses of hydrojacking depend on the pressure of the overlying rock. P/Q-Type 3 occurred only once—fracturing took place due to insufficient tensile strength.

### *5.7.2 Albarello*

The subsoil of the arch dam, 50 m in height, consists of granite and granite-gneiss. WPT's were carried out systematically during the investigation as well as during the curtain grouting. The results of all 922 tests are considered. The tests pressures ranged mostly between 8 and 12 bar; those few tests taking much water reached small pressures only. All P/Q-Types were observed, except Type 2 (Table 5.1; Fig. 5.8). Almost one third of all tests absorbed no water; fracturing and dilation took place in another third, with critical pressures between 6 and 12 bar.

**Table 5.1** Basic results

| Project | Rock types | Holes | | Evaluation WPT | | | | Portion of P/Q-Types | | | | | | | | | | | | | | $P_{crit}$ (bar) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Max. | | | | | 1 | | 2 | | 3 | | 4 | | 5 | | 6a | | 6b | | Due to | Due to |
| | | Num | Depth | Num | $P_{max}$ | $Q_{max}$ | $Q_{min}$ | n | % | n | % | n | % | n | % | n | % | n | % | n | % | Heaving | Strength |
| AIX | Mst, Tst, Sst, Kst | 6 | 20 | 17 | 4.2 | >100 | 0 | 0 | 0 | 0 | 0 | 1 | 5.9 | 7 | 41.2 | 9 | 53 | 0 | 0 | 0 | 0 | > 1 < 2 | >3.7 |
| Alb | Granite, Gneiss | 122 | 65 | 922 | 12.3 | >100 | 0 | 292 | 31.7 | 0 | 0 | 201 | 22 | 293 | 31.8 | 104 | 11 | 5 | 0.5 | 27 | 3 | > 2 < 5 | > 6 < 12 |
| ALM | Granite | 11 | 83 | 137 | 26.4 | >100 | 0 | 26 | 19 | 0 | 0 | 21 | 15 | 73 | 53.3 | 13 | 9.5 | 1 | 0.7 | 3 | 2 | > 1 < 5 | > 5 < 15 (<25) |
| ARG | Tst, Sst + Mst | 13 | 79 | 78 | 15.5 | 22.9 | 0 | 3 | 3.8 | 0 | 0 | 0 | 0 | 68 | 87.2 | 6 | 7.7 | 0 | 0 | 1 | 1 | <5 | > 7 < 12 |
| ARR | Kst, Sst, Tst | 9 | 30 | 44 | 5.4 | 55.2 | 0.1 | 0 | 0 | 0 | 0 | 0 | 0 | 15. | 34.1 | 25 | 57 | 3 | 6.8 | 1 | 2 | > 1 < 3 | > 4 < 5 |
| BEN | Mst, Tst, Sst | 15 | 65 | 63 | 13.2 | 17.4 | 0 | 13 | 20.6 | 4 | 6 | 6 | 9.5 | 32 | 50.8 | 6 | 9.5 | 0 | 0 | 2 | 3 | 5.2 | > 6 < 9 |
| BUR | Granite | 11 | 40 | 48 | 10.1 | 20.3 | 0.5 | 0 | 0 | 0 | 0 | 0 | 0 | 25 | 52.1 | 6 | 13 | 7 | 15 | 10 | 21 | > 3 < 6 | > 5 < 9 |
| CAN | Quarzite, Schists | 9 | 53 | 69 | 15.0 | 13.3 | 0 | 3 | 4.2 | 0 | 0 | 8 | 11 | 54 | 76.1 | 4 | 5.6 | 0 | 0 | 0 | 0 | > 2 > 5 | > 3 < 7 |
| CER | Gneiss | 11 | 65.5 | 86 | 11.7 | 213 | 0 | 20 | 23.3 | 0 | 0 | 8 | 9.3 | 54 | 62.8 | 4 | 4.7 | 0 | 0 | 0 | 0 | > 3 < 5 | > 6 < 10 |
| COS | Schists | 68 | 62 | 130 | 11.0 | 7.4 | 4.1 | 0 | 0 | 0 | 0 | 0 | 0 | 111 | 85.4 | 19 | 15 | 0 | 0 | 0 | 0 | > 2 < 4 | > 4 < 7 |
| DONA | Mst, Tst, Ust | 10 | 25 | 23 | 10.8 | 5.4 | 0.4 | 0 | 0 | 0 | 0 | 0 | 0 | 20 | 87.0 | 0 | 0 | 0 | 0 | 3 | 13 | | |
| EDRA | Granite | 10 | 20 | 19 | 10.9 | 43.3 | 0 | 1 | 5.3 | 0 | 0 | 0 | 0 | 9 | 47.4 | 5 | 26 | 0 | 0 | 4 | 21 | > 1 < 4 | |
| EUL | Schists | 72 | 32 | 321 | 11.2 | 46.9 | 0 | 142 | 44.2 | 0 | 0 | 39 | 12 | 115 | 35.8 | 21 | 6.5 | 2 | 0.6 | 2 | 1 | 2.5 | > 6.5 < 8 |
| LAR | Tst, Ust, Sst, Kst | 3 | 25 | 11 | 4.5 | 26.5 | 7 | 0 | 0 | 0 | 0 | 0 | 0 | 11 | 100 | 0 | 0 | 0 | 0 | 0 | 0 | | |
| LLO | Kst, Sst, Tst, Kgl | 7 | 100 | 68 | 13.1 | 74 | 0 | 6 | 8.8 | 1 | 2 | 6 | 8.8 | 43 | 63.2 | 10 | 15 | 1 | 1.5 | 1 | 2 | > 3 < 5 | |
| MIN | Granite | 10 | 43.5 | 42 | 13.2 | 0.3 | 0 | 20 | 47.6 | 0 | 0 | 8 | 19 | 10 | 23.8 | 4 | 9.5 | 0 | 0 | 0 | 0 | | > 5 < 10 |
| PAL | Dol, Kst | 11 | 50 | 33 | 12.4 | 4.1 | 0 | 1 | 3 | 1 | 3 | 3 | 9.1 | 22 | 66.7 | 3 | 9.1 | 3 | 9.1 | 0 | 0 | | > 8 < 12 |
| PON | Slates | 14 | 33 | 47 | 11.9 | 8.8 | 0.4 | 0 | 0 | 0 | 0 | 0 | 0 | 22 | 46.8 | 23 | 49 | 2 | 4.3 | 0 | 0 | > 3 < 5 | > 6 < 10 |
| POR | Schists, Quarzite | 2 | 21 | 10 | 24.0 | 1.4 | 0 | 0 | 0 | 0 | 0 | 7 | 70 | 3 | 30 | 0 | 0 | 0 | 0 | 0 | 0 | > 2 < 4 | > 10 < 16 |

(continued)

**Table 5.1** (continued)

| Project | Rock types | Holes | | Evaluation WPT | | | | Portion of P/Q-Types | | | | | | | | | | | | | | $P_{crit}$ (bar) | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Max. | | | | | 1 | | 2 | | 3 | | 4 | | 5 | | 6a | | 6b | | Due to | Due to |
| | | Num | Depth | Num | $P_{max}$ | $Q_{max}$ | $Q_{min}$ | n | % | n | % | n | % | n | % | n | % | n | % | n | % | Heaving | Strength |
| RIA | Gips, Tst | 11 | 63.5 | 61 | 11.2 | 0.4 | 0 | 7 | 11.5 | 3 | 5 | 13 | 21 | 27 | 44.3 | 11 | 18 | 0 | 0 | 0 | 0 | 5.6 + 6.1 | > 5 < 10 |
| VAD | Schists | 5 | 55 | 11 | 12.0 | 8.4 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 5 | 45.5 | 6 | 55 | 0 | 0 | 0 | 0 | 3.5 | > 5 < 10 |
| VIL | Schists | 14 | 25 | 27 | 11.2 | 82 | 2.1 | 0 | 0 | 0 | 0 | 0 | 0 | 17 | 63 | 6 | 22 | 0 | 0 | | 15 | > 2 < 4 | > 3 < 5 |
| All | | 444 | 100 | 2267 | 24 | >100 | 0 | 534 | 23.5 | 9 | 0.4 | 321 | 14.1 | 1036 | 45.7 | 285 | 12.7 | 24 | 1.1 | 58 | 2.6 | | |

*Dol*—Dolomite, *Gy*—Gypsum, *Kgl*—Conglomerate, *Kst*—Limestone, *Mst*—Marlstone, *Sst*—Sandstone, *Tst*—Claystone, *Ust*—Siltstone

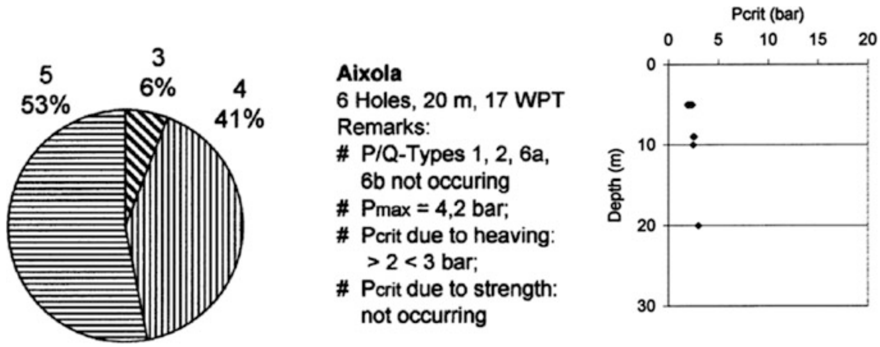



**Fig. 5.7** Aixola. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

### *5.7.3 Almendra*

The project comprises a combined gravity and arch dam, 180 m in height. The subsoil consists of massive granite, often slightly metamorphic with parallel orientation of crystals. More than thousand WPT's schematically executed in the groutholes shall be analyzed later on; for this contribution 137 WPT's carried out in 11 investigation holes have been evaluated.

Approximately half of the tests recorded a proportionate relationship between pressure and water take. Almost on fourth of the tests showed dilation or fracturing. The critical pressures ranged between 5 and 13.5 bar. Their scattering is considerable but does not increase with depth. One fifth of the tests absorbed no water though high test pressures up to 25 bar were applied. The results are presented in Table 5.1 and Fig. 5.9.

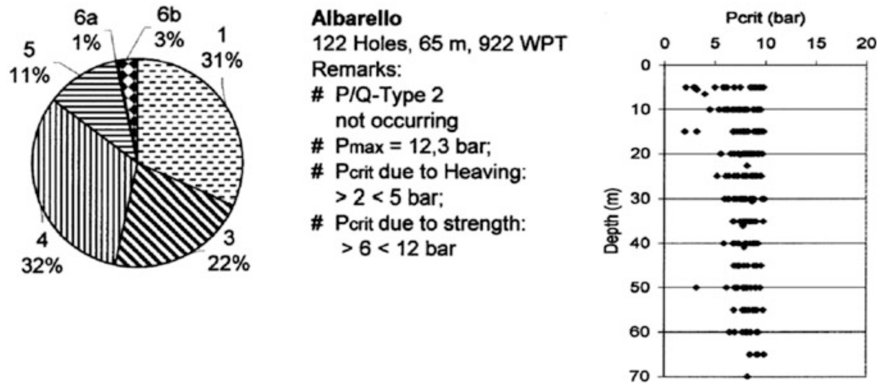



**Fig. 5.8** Albarello. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

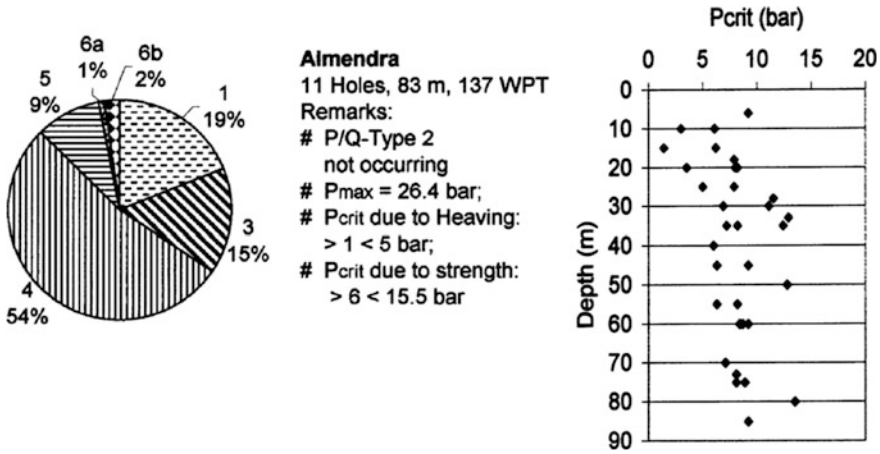



**Fig. 5.9** Almendra. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

## 5.7.4 Argoza

Approximately half of the tests recorded a proportionate relationship between pressure and water take. Almost one fourth of the tests showed dilation or fracturing. The critical pressures ranged between 5 and 13.5 bar. Their scattering is considerable but does not increase with depth. One fifth of the tests absorbed no water though high test pressures up to 25 bar were applied. The results are presented in Table 5.1 and Fig. 5.10.

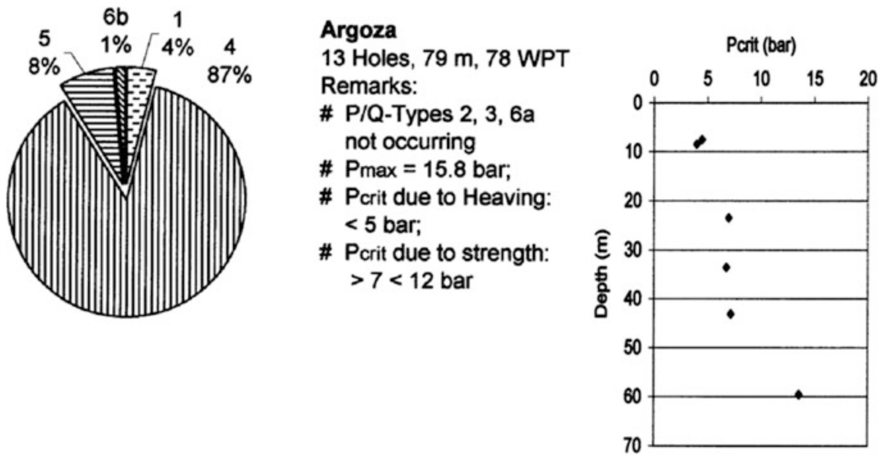



**Fig. 5.10** Argoza. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

At the site projected for a dam of 60 m in height, the underground consists of an alternating sequence of thick quarzitic sandstone banks and thin layers of shale, locally changing into massive limestone (Wealdian).

The 78 WPT's evaluated were carried out within an investigation program for a dam site. The largest part of the borehole sections tested encountered open paths: 87% of all tests belong to Type 4—permeable, with proportionate relationship between pressure and water take. Three sections remained tight, the rock did not crack in spite of pressures between 10 and 15 bar—Type 1. In 6 sections dilation occurred beginning at pressures between 6.8 and 7.2 bar. Table 5.1 and Fig. 5.10 present the results.

### 5.7.5 *Arriaran*

44 WPT's were carried out in 9 holes up to 30 m deep to investigate the underground of a roller compacted concrete dam 50 m in height. A flysch-like sequence (Albian, Cenomanian) contains an alternation of sandstones, siltstones and marly limestone. Due to large water takes only test pressures up to 5.4 bar were achieved.

The results presented in Table 5.1 and Fig. 5.11 are summarized as follows:

* One third of the tests recorded a proportionate relation between pressure and water takes—Type 4.
* More than half of all tests recorded dilation—Type 5.
* Down to a depth of 15 m the critical pressures increased from 1 to 3 bar, i.e. dilation was determined by overburden pressure.
* Further below the dilations taking place at pressures between 4 and 5 bar were ruled by strength, i.e. independent of depth.
* Three tests noticed erosion of infillings occurring at pressures <3 bar—Type 6a

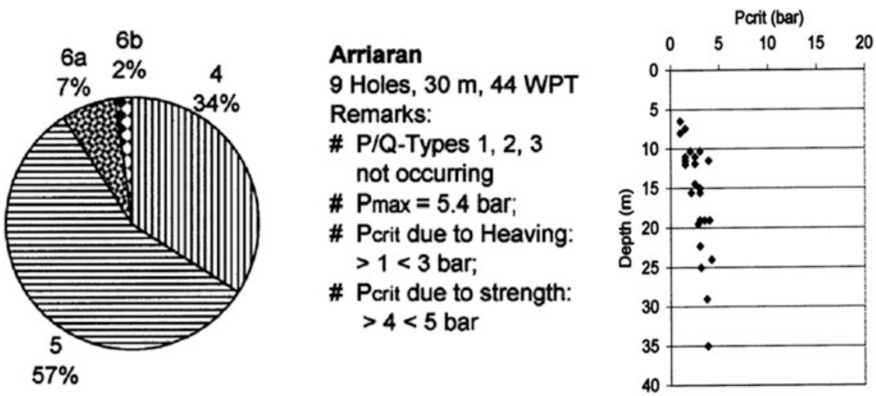



**Fig. 5.11** Arriaran. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 4 and 5 (*right*)

### 5.7.6 Benamor

In order to investigate two alternative dam sites, 63 WPT's were carried out in 15 holes down to 65 m in depth. Although the underground consists at both sites of an alternation of marly claystone, sandstone and sandy limestone (Tertiary), different permeabilities evolved as the results reveal: $Q_{max}$ and the shares of the P/Q-Types are different. The following summary compares the results of the dam sites (Table 5.1; Fig. 5.12):

* Tight borehole sections (P/Q-Type 1) reach 21% (BEN-1: 54.5%; BEN-9: 2.4%).
* Tight boreholes sections with saturation of isolated voids (P/Q-Type 2) are seldom—2% (BEN-1: 0%; BEN-9: 10%).
* 10% of the sections were fractured (P/Q-Type 3) (BEN-1: 18%; BEN-9: 4.9%).
* The share of permeable sections (P/Q-Type 4) reaches 50% (BEN-1: 18%; BEN-9: 68%).
* Dilation was recorded in 10% of the sections (P/Q-Type 5) (BEN-1: 9.1%; BEN-9: 9.8%).

All cases of dilation and fracturing were determined by strength. The critical pressures ranged between 5 and 10 bar and did not increase with depth. The relatively large share of P/Q-Type 2 (saturation) was probably caused by porous limestone.

### 5.7.7 Burdalo

The subsoil of the rock fill dam consists of granodiorites. The 48 WPT's were carried out in 11 holes of 40 m in depth drilled to investigate the foundation. One

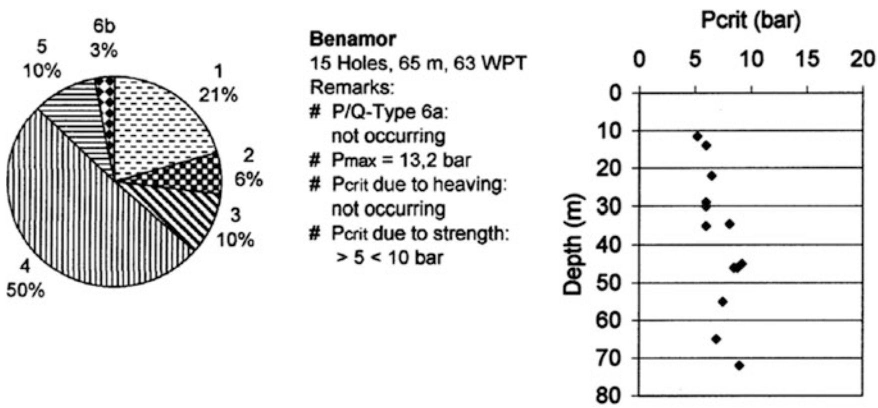



**Fig. 5.12** Benamor. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

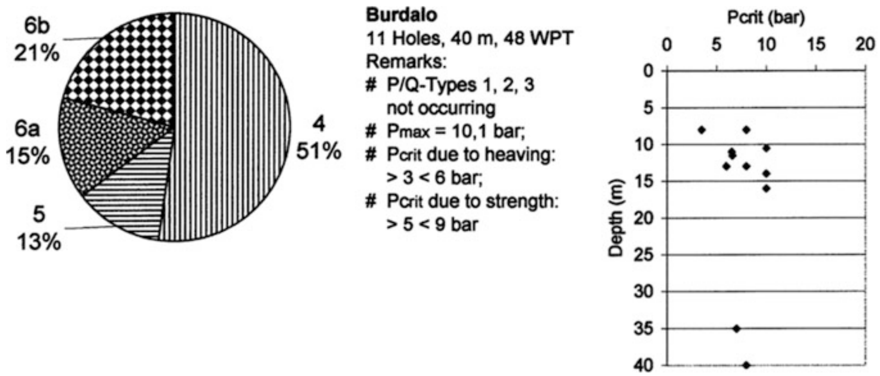



**Fig. 5.13** Burdalo. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

exceptional test yielded 20.3 LU, all other tests absorbed less than 3 LU. The tests pressures ranged at 10 bar, a smaller pressure was applied in the uppermost section.

Impervious rock was not encountered, all sections absorbed water. Half of the tests produced P/Q-Type 4, with a proportionate relationship between pressure and water take. The other half of the tests comprises at equal shares P/Q-Type 5 (dilation), P/Q-Type 6a (erosion) and P/Q-type 6b (clogging). Down to 10 m the dilation processes were determined by the overburden pressure, further below by strength. The results are listed in Table 4.1 and graphically presented in Table 5.1 and Fig. 5.13.

### *5.7.8 El Canal*

In order to investigate the dam site 9 holes were drilled up to 53 m in depth. The subsoil consists of quartzites and slates (Silurian). 44 WPT's were carried out with pressures between 3 and 5 bar in the upper sections and with pressures between 10 and 15 bar further below.

The results presented in Table 5.1 and Fig. 5.14, are summarized as follows:

* 16% of all test sections were originally tight (P/Q-Type 1), a few remained but most were fractured (P/Q-Type 3).
* Three thirds of the test sections were permeable but only a few absorbed substantial amounts. They belong to P/Q-Type 4.
* A few sections were dilated (P/Q-Type 5).

All critical pressures dilating existing paths or cracking latent discontinuities were small and did not increase with depth.

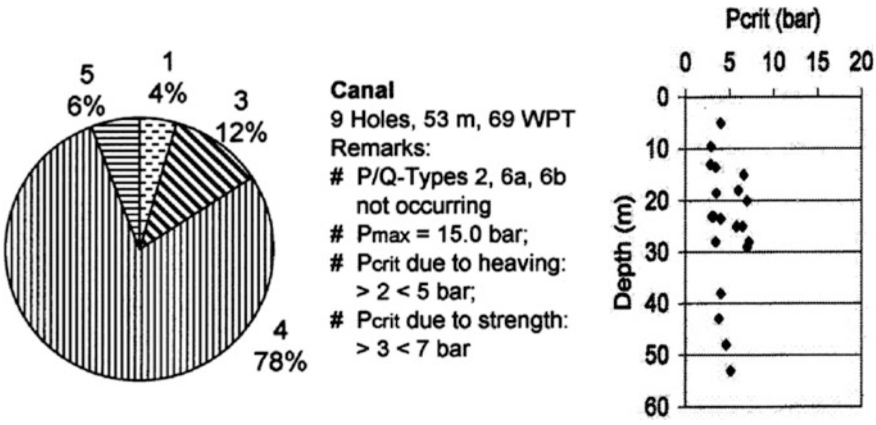



**Fig. 5.14** El Canal. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

## *5.7.9 Cernadilla*

The gravity dam, 45 m in height, was founded on Gneiss (Cambrian). This analysis considers 86 WPT's carried out in 11 investigation holes up to 63.5 m in depth. The following summary is illustrated in Table 5.1 and Fig. 5.15:

* In several zones intense weathering reaches to a depth of approximately 30 m, thus permeable rock was encountered in about 63% of all tests—P/Q-Type 4. Large water takes often kept the achievable test pressures small—$P_{max}$: $\leq 1$ bar, $Q_{max} \geq 100$ LU.
* In some of the permeable tests sections the paths were dilated at pressures between 3 and 5 bar—P/Q-Type 5.

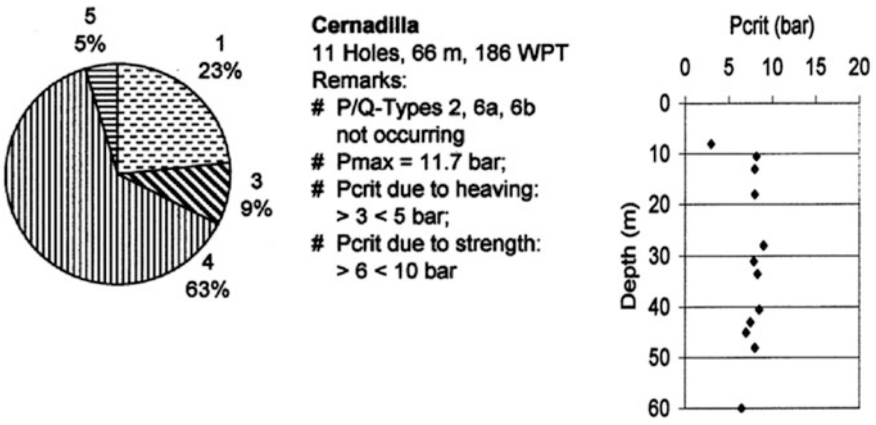



**Fig. 5.15** Cernadilla. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

* Besides there were impermeable zones: almost a quarter of the tests were tight and did not fracture, even at pressures between 9 and 12 bar—P/Q-Type 1.
* The impervious rock included test sections cracking at pressures between 6 and 10 bar.
* The critical pressures were scattered but did not increase with depth.

### *5.7.10 San Cosmade*

The subsoil of the gravity dam, 17 m in height, consists of slates (Devonian). Data are available from

* 10 WPT's carried out in 7 investigation holes, 10 m deep, and
* 120 WPT's executed in 61 groutholes, 52 m deep.

The boreholes run parallel to the vertically dipping schists which influenced the results of the WPT's. The absorption rates $Q_{max} > 100$ LU and $Q_{min} = 2$ LU indicate a significant permeability. All test sections absorbed water but only two types of P/Q-diagrams occurred: 85% of the tests remained unaltered (P/Q-Type 4), 15% of the tests were dilated with critical pressures beginning at pressures of approx. 2 bar already (P/Q-Type 5). In the tests carried out near to the surface hydrojacking was controlled by heaving, further below by strength. The results are listed in Table 5.1 and graphically presented in Fig. 5.16.

### *5.7.11 Doña Ana and Laredo*

Both dam sites (Doña Ana: gravity-dam, 23 m in height; Laredo: surface-sealed rock-fill dam, 40 m in height) can be dealt together because similar rock types form their underground and the WPT-results are quite similar as well: a well bedded alternation of marlstone, sandstone and claystone (Lower Cretaceous).

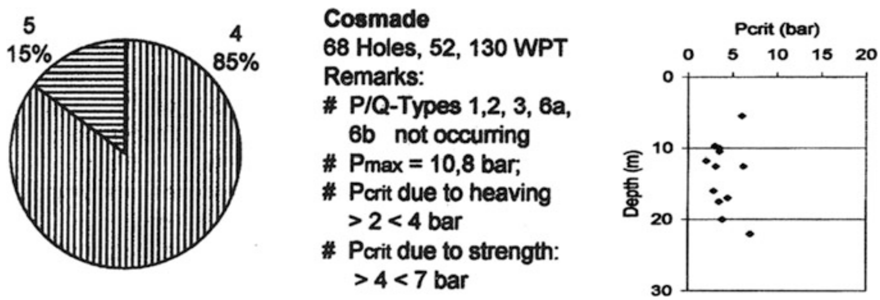



**Fig. 5.16** San Cosmade. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 4 and 5 (*right*)

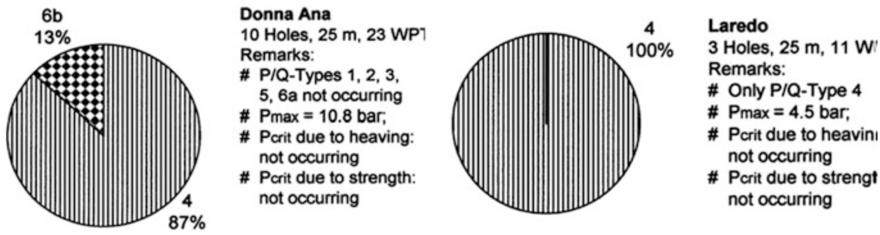



**Fig. 5.17** Doña Ana and Laredo. Portions of P/Q-Types 4 and 6b

The investigation programs included at Doña Ana 23 WPT's carried out in 10 holes, and at Laredo 11 WPT's executed in 3 holes, respectively. All test sections at Doña Ana belong to P/Q-Types 4 and 6b—permeable and clogging, at Laredo to P/Q-Type 4—permeable. The results reveal that the underground at Laredo is much more permeable (Table 5.1; Fig. 5.17).

### *5.7.12 Edrada*

The gravity dam, 24 m in height, was founded on granite. Data are available from 19 WPT's executed in 10 investigation holes up to 20 in depth. Test pressures between 4 and 10.9 bar were applied producing water takes between 43.3 and 0 LU. Table 5.1 and Fig. 5.18 present the results in detail:

* Impermeable rock (P/Q-Type 1) was encountered only once.
* 9 tests belong to P/Q-Type 4, their paths remained unaltered.
* 5 tests recorded dilation (P/Q-Type 5); the critical pressures increased with depth down to 20 m, i.e. hydrojacking was determined by the overburden pressure.
* 4 tests recording clogging (P/Q-Type 6b).

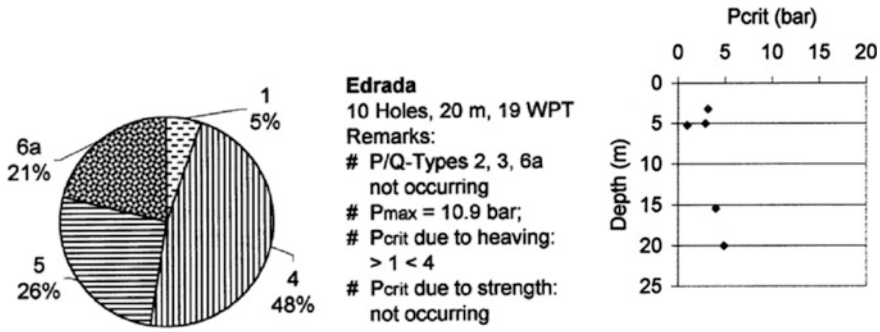



**Fig. 5.18** Edrada. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 4 and 5 (*right*)

### 5.7.13 Santa Eulalia

The data come from 321 WPT's systematically carried out before grouting in 72 holes. They were drilled from the gallery of the 75-high arch dam and reach to a depth of 32 m. The underground consists of metamorphic slates (Cambrian). The test pressures ranged between 9 and 11 bar. With the exception of a few test sections the rock was tight or just a little permeable. The following is deduced from Table 5.1 and Fig. 5.19:

* A considerable portion of the test sections remained impermeable up to the a maximum pressure of 11 bar (P/Q-Type 1) while in other zones hydrofracturing took place at critical pressures between 6.5 and 8 bar (P/Q-Type 3), thus, latent discontinuities susceptible to fracturing are evidently rare.
* About one third of the test sections was permeable and remained unaltered during higher pressure steps (P/Q-Type 4).
* Several further tests encountered also permeable rock but were dilated during higher pressure steps (P/Q-Type 5).
* The range of scattering of the critical pressures causing dilation or fracturing is rather small and does not increase with depth.

The data listed in Table 5.2 reveal exemplarily that water bearing paths obviously need a particular shape and a particular deformation behavior to be groutable: in the four test sections listed latent discontinuities were pressed open at pressures between 6 and 11 bar ($Q_{10}^* > Q_{10}'$). Only two remained accessible because subsequently they could be grouted at pressures between 15 and 20 bar, taking 50 and 100 kg/m, respectively. In the other sections the opened paths closed immediately after the tests and could not be pressured open for the access of grout despite the higher pressures between 20 and 25 bar.

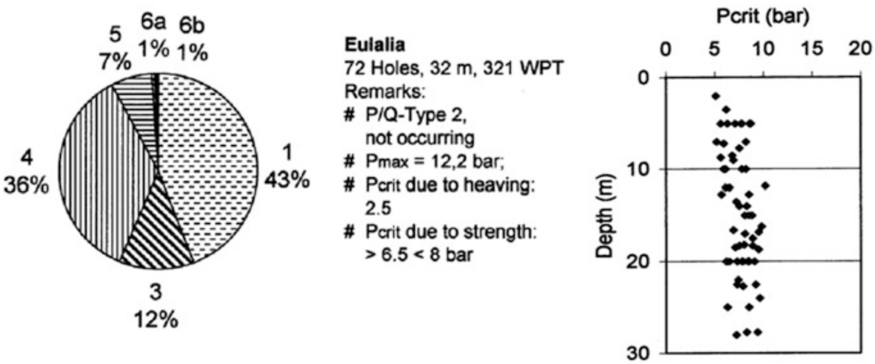



Fig. 5.19 Santa Eulalia. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

**Table 5.2** Santa Eulalia, comparison between water and grout takes (example)

| Hole Nr | Depths [m] | Cement [kg/m] | $P_{grt}$ [bar] | $P_{wd}$ max | Q-1′ … Q-10′… Q-1* Q-10* [1/(min*m)] | | | | Type | $P_{crit}$ [bar] |
|---|---|---|---|---|---|---|---|---|---|---|
| E-66 | 8.5 | 50.0 | 15.0 | 10.6 | 0.0 | 0.0 | 0.2 | 1.9 | 3 | > 6 > 11 |
| | 13.5 | 100.0 | 20.0 | 10.6 | 0.0 | 0.0 | 0.2 | 1.9 | 3 | > 6 > 11 |
| E-66 | 15.0 | 20.0 | 20.0 | 10.7 | 0.0 | 0.0 | 0.1 | 1.4 | 3 | > 6 > 10 |
| | 20.0 | 20.0 | 25.0 | 10.7 | 0.0 | 0.0 | 0.2 | 1.9 | 3 | > 6 > 10 |

### *5.7.14 La Llosa Del Caval*

The subsoil of the dam site projected for a gravity dam, 100 m in height, consists of an alternating sequence of conglomerates, sandstones, siltstones and marlstones. Data from 68WPT’s are available carried out in 7 holes up to 100 m deep.

A few times large water takes ($Q_{max}$ = 69 LU) were absorbed allowing small test pressures only. However, in most cases small water takes were recorded and the test pressures were correspondingly high—between 8 and 13.1 bar. The following conclusions are drawn from Table 5.1 and Fig. 5.20:

* The rock contains a small portion of impervious sections, remaining tight until the maximum pressures of 13 bar (P/Q-Type 1).
* About a similar portion is fractured at pressures between 5 and 13 bar; this fracturing is determined by the strength and does not increase with depth (P/Q-Type 3).
* 79% of the test sections were permeable already from the beginning. Its largest part recorded a proportionate increase of the water takes (P/Q-Type 4) while the paths of the others were dilated (P/Q-Type 5).

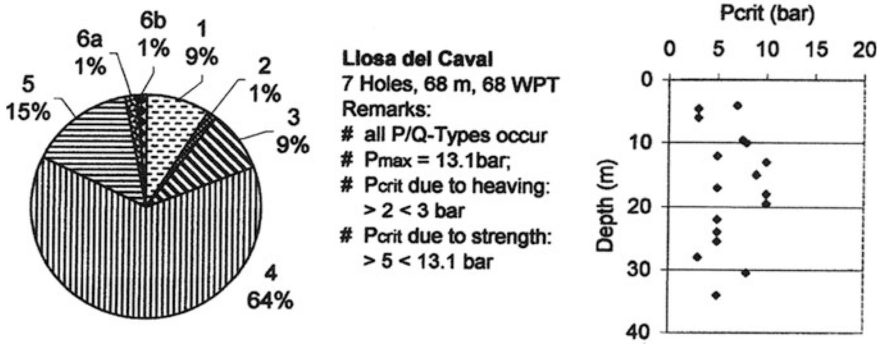



**Fig. 5.20** Llosa del Caval. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

### 5.7.15 *Mingorria*

The 60 m high arch dam was founded on granite. Data are available from 42 WPT's carried out in 11 holes drilled to a depth of 44 m. The results indicate impervious rock: $Q_{max}$ = 0.3 LU, $Q_{min}$ = 0 LU, $P_{max}$ = 13.2 bar. The state of the foundation is favorable (Table 5.1; Fig. 5.21). Summarized results:

* 20 test sections encountered impervious rock; they remained tight up to the maximum pressure (P/Q-Type 1).
* 8 test sections were originally tight but fractured at pressures between 6.5 and 8.5 bar due to insufficient strength (P/Q-Type 3). The critical pressures did not increase with depth.
* 10 test sections absorbed small amounts of water and remained unaltered also during the high pressure steps (P/Q-Type 4).
* The same pressures were able to widen the paths in 4 test sections, also because of insufficient strength (P/Q-Type 5).

### 5.7.16 *Palancia*

The subsoil of this possible dam site contains sedimentary deposits of various Mesozoic members, which means a variety of different rock types such as conglomerate, sandstone, claystone, limestone, dolomite and gypsum. The foundation was explored by means of 11 boreholes in which 33 WPT's were carried out. The overall permeability of the foundation is small: $Q_{max}$ = 4.1 LU, $Q_{min}$ = 0 LU, $P_{max}$ = 12.4 bar.

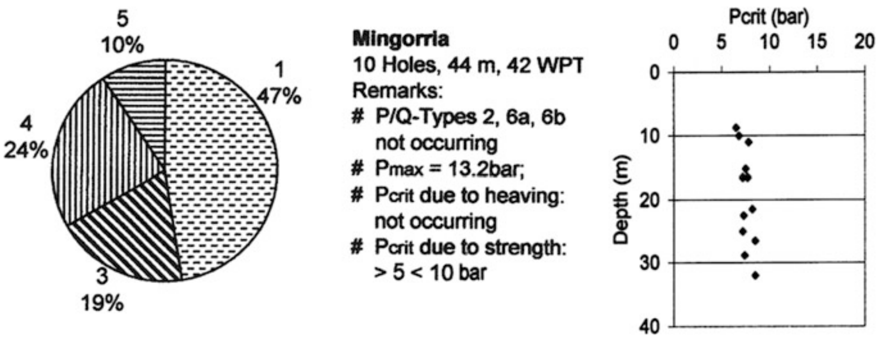



**Fig. 5.21** Mingorria. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

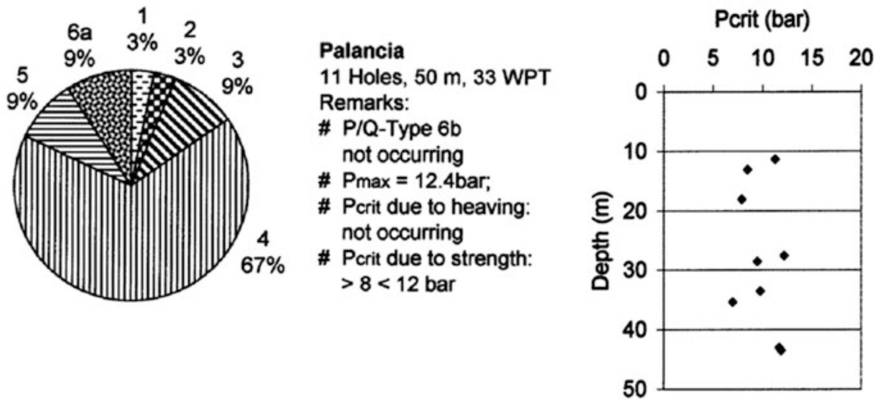



**Fig. 5.22** Palancia. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

Table 5.1 and Fig. 5.22 reveal that

* 2 test sections were impermeable (P/Q-Types 1 and 2);
* 22 test sections were permeable and absorbed water from the beginning, although just a little, with a proportionate relationship between pressure and quantity (P/Q-Type 4);
* 3 test sections absorbed water and their paths were dilated at pressures between 5 and 12 bar, these courses of hydrojacking were determined by strength (P/Q-Type 5);
* 3 test sections were originally tight but fractured at pressures between 6 and 11 bar, which also occurred due to strength (P/Q-Type 3);
* 3 test sections took originally some water and were washed out at pressures of approx. 6 bar (P/Q-Type 6a);

All critical pressures were scattered but did not increase with depth.

### *5.7.17 Ponga*

The project envisages a 20 m high gravity dam to be built on tightly folded slates (Carboniferous). The subsoil had been explored by 14 boreholes up to 33 m in depth, including 47 WPT's. According to the main features—$Q_{max}$ = 8.8 LU, $Q_{min}$ = 0.4 LU, $P_{max}$ = 11.9 bar—the foundation seems not to be very permeable.

The results listed in Table 5.1 and illustrated in Fig. 5.23 are summarized as follows:

* Originally impermeable tests sections were lacking, i.e. all sections absorbed water from the beginning.

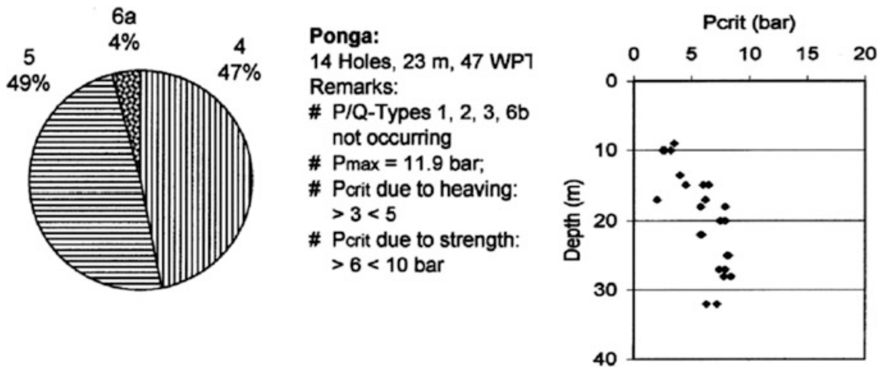



**Fig. 5.23** Ponga. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 4 and 5 (*right*)

* In almost half of the tests the water takes grew proportionately with pressure (P/Q-Type 4); the paths in the test sections of the other half were dilated (P/Q-Type 5). Down to 18 m depth the critical pressures increased from 3 to bar up to 5 bar because they were determined by the overburden pressure; further below the critical pressures ranged between 6 and 10 bar but without increasing with depth. Figure 5.23 demonstrates convincingly that different behavior.
* In 3 test sections paths with infillings were encountered which were eroded (P/Q-Type 6a);

### *5.7.18 Las Portas*

The arch dam is approximately 120 in height and was founded on schists and quartzite. The available information comprises the detailed grout takes of the entire curtain but only the data of 10 WPT's carried out in 2 investigation holes. In spite of that scarceness, the example is worth to be considered because the WPT's have been carried out in a very particular way: instead of the few pressures steps usually applied, these tests comprised an extraordinary great number of pressure steps with maximum pressures up to 23 bar. These sequences disclose much better how the relationship evolves between pressure and water takes and which are the factors causing the various courses of WPT's, i.e. the types of the P/Q-types. Figure 5.3 presents 6 of these tests: as so many pressure steps were applied only the ascending steps are considered with their data and diagrams; the descending branches are disregarded. The data listed in Table 4.1 and illustrated in Fig. 5.24 are interpreted as follows:

* Due the high pressures applied in most tests the cases of hydrofracturing prevail (60% of P/Q-Type 3) and the cases of hydrojacking are still noticeable (20% of P/Q-Type 5).

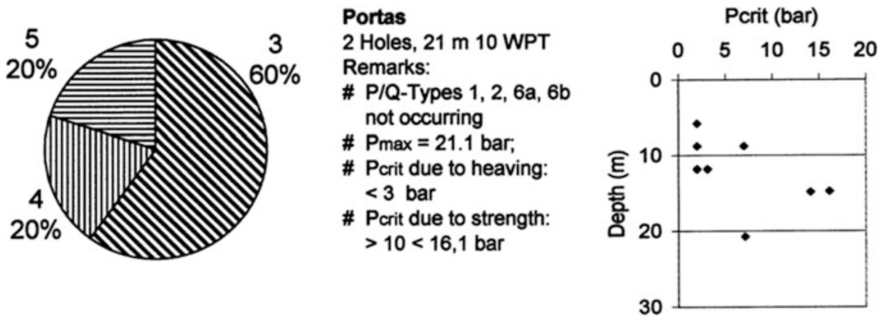



**Fig. 5.24** Las Portas. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

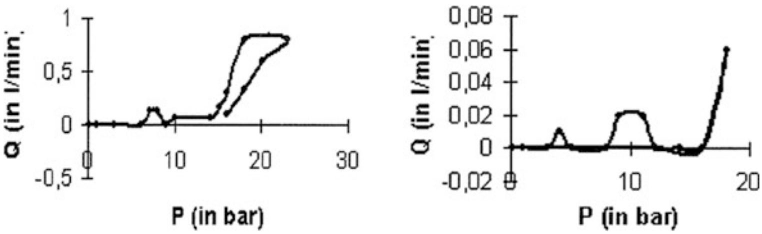



**Fig. 5.25** Las Portas. Diagrams of P/Q-Type 3. Indicating gradual beginning of hydrofracturing

* The critical pressures were controlled in the upper seam by the weight of the overlying rock and further below by strength.
* The remaining portion of the tests indicated open paths from the beginning which were not dilated during the higher pressures steps but preserved its geometry.
* The P/Q-diagrams displayed in Fig. 5.25 disclose that hydrofracturing is obviously often a gradual process.

## *5.7.19 Riansares*

The dam site projected was explored by 11 boreholes up to 71 m in depth encountering rather young marly-clayey sediments (Miocene and Pliocene); 61 WPT's were carried out. The maximum pressures reached 10.2–11.2 bar. The main features characterize a practically impervious rock: $Q_{max}$ = 0.4 LU and $Q_{min}$ = 0 LU. The detailed results are presented in Table 5.1 and illustrated in Fig. 5.26:

* In 7 test sections the rock remained tight up the maximum pressure (P/Q-Type 1).
* 3 test sections yielded P/Q-Type 2—saturation of isolated voids.

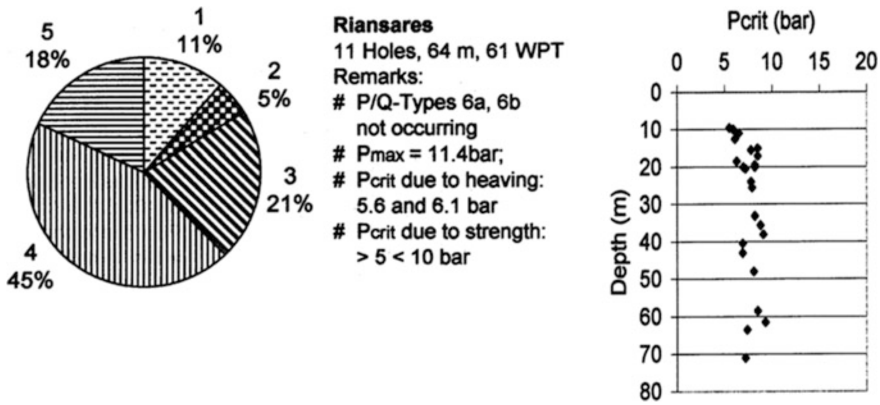



**Fig. 5.26** Riansares. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Types 3 and 5 (*right*)

* Hydrofracturing determined by strength occurred at pressures between 5.7 and 9 bar in 13 tests (P/Q-Type 3).
* 38 tests sections encountered permeable rock—a proportionate relation between pressure and water take was recorded in 27 of them (P/Q-Type 4), an over-proportionate relation due to hydrojacking occurred in 11 tests (P/Q-Type 5).
* The distribution of the critical pressures related to depth shows that up to a depth of 12 m the dilation of the existing paths is controlled by heaving while further below it occurs due to strength. The critical pressures of the first group increase with depth, those of the second one keep their order of magnitude.

### *5.7.20 Valdejudios*

The design envisages a gravity dam, 15–20 m in height. The subsoil consists of sandstone and claystone (Miocene, Pliocene) overlying slates and sandstone alternating with siltstone (Carboniferous). Five boreholes up to 55 m in depth were drilled to investigate the foundation; they include 11 WPT's. The permeability is small to moderate: $Q_{max}$ = 8.4, $Q_{min}$ = 0 LU, $P_{max}$ = 12 bar. According to Table 5.1 and Fig. 5.27 the 11 tests executed belong only to P/Q-types 4 and 5: permeable and dilation, the latter one with critical pressures of 3.5 bar caused by heaving and further below between 5 and 7 bar controlled by strength. The critical pressures do not increase with depth.

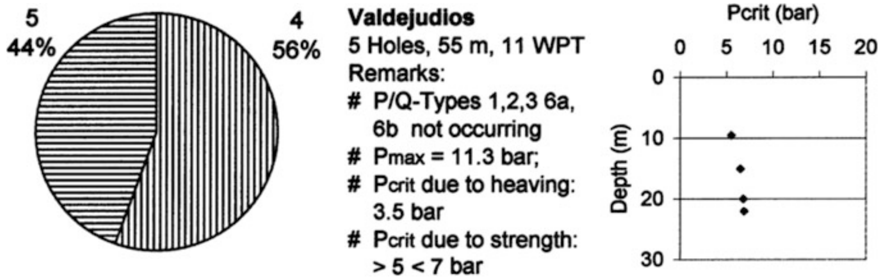



**Fig. 5.27** Valdejudios. Portion of P/Q-Types (*left*) and relation $P_{crit}$/Depth for Type 5 (*right*)

### *5.7.21 Results of WPT's Carried Out in Other Countries*

Critical evaluations reveal that WPT-results are not always correct. Deficient executions or inappropriate evaluations impair the accuracy of the results not too seldom. The dimension of such failures depends on the technological state of the company and, above all, the qualification and experience of the personnel doing the job. Results can be inexact in each country. In order to check whether the results obtained in the Spanish projects correspond well to those obtained elsewhere, the results of the WPT's used for this analysis are compared with the results of tests obtained in other countries. The comparison yields no discrepancy; thus they are obviously correct. For that comparison a large number of WPT's carried out in *31* '*international*' projects worldwide have been considered.

The ranges of scattering of the critical pressures obtained in a total of 47 projects are listed in Table 5.3; names and rock types of the international projects are printed in italic to identify them easily. It is evident that both groups yielded similar—or even equal—results, e.g. all results are representative.

### *5.7.22 Interpretation*

The results obtained with the analysis of the many examples in almost all possible rock types confirm the courses of hydrojacking and hydrofracturing postulated in Chap. 5. The interpretation of the results yields the following summary:

Hydrojacking of existing paths begins once the test pressure exceeds the overburden pressure (heaving) or is high enough to compress the rock along the open path. Widening of the paths causes an over-proportionate increase of the water takes which continues as long as the test pressure grows. The same applies to the grouting pressure.

The pressures initiating the dilation of the paths due to heaving increase with depth. They differ because:

**Table 5.3** Critical pressures causing hydrofracturing or hydrojacking identified in projects around the world, mainly in Spain

| Legend: | | |
|---|---|---|
| | >> X | Pressure steps up to Pmax |
| | oooo | Dilation due to heaving |
| | ***** | Dilation or Heaving due to strength |
| | *>> ? | Pcrit has not been reached |
| | >111 | only P/Q-Type 1 occur, Pcrit needs higher pressure |

| | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Critical pressures (bar) | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Project | Rock type | | | | | 5 | | | | | 10 | | | | | 15 | | | | | 20 | | | | | 25 | | | | | 30 | | |
| Aabach | Sst, Tst Slates | o | o | o | | | * | X | | | | | | | | | | | | | | | | | | | | | | | | | |
| Aixola | Tst, Sst, Mst | o | o | * | * | X | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Albarello | Granit, Gneiss | | o | o | o | o | * | * | * | * | * | * | * | X | | | | | | | > | 1 | 1 | 1 | | | | | | | | | |
| Almendra | Granit, Gneiss | o | o | o | o | * | * | * | * | * | * | * | * | * | * | | | | | | | | | | | | | X | | | | | |
| Antrift | Sst, Tst | | | o | o | o | x | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Argoza | Tst, Sst, Mst | | | o | o | o | | | * | * | * | * | * | | | | X | | | | > | 1 | 1 | 1 | | | | | | | | | |
| Arriaran | Tst, Sst, Mst | | o | o | | * | X | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Bashe | Norit | | | | | | | | * | * | * | * | | | | | | | | | | | | | | X | | | | | | | |
| Benamor | Tst, Sst, Mst | | | | | | * | * | * | * | | | | | X | | | | | | | | | | | | | | | | | | |
| Burdalo | Granit | | | | o | o | o | * | * | * | | X | | | | | | | | | | | | | | | | | | | | | |
| Canal | Quarzit, Slates | | o | o | * | * | * | * | | | | | | | | | X | | | | | | | | | | | | | | | | |
| Cernadilla | Gneis | | | | o | o | * | * | | | | | X | | | | | | | | | | | | | | | | | | | | |
| Cosmade | Schists | | | o | | | * | * | * | * | * | X | | | | | | | | | | | | | | | | | | | | | |
| Dona Ana | Tst, Mst, Sst | o | o | o | o | o | | | | | | X | | | | | | | | | | | | | | | | | | | | | |
| Edrada | Granit | o | o | o | o | o | | | | | | X | | | | | | | | | | | | | | | | | | | | | |
| Egliau | Tst, Sst | | | | | | | | | | | * | * | | X | | | | | | | | | | | | | | | | | | |
| Eulallia | Schists | | | o | | | | | * | * | | | X | | | | | | | | | | | | | | | | | | | | |
| Gallo | Porphyr | | | | | | | | * | * | * | * | | | X | | | | | | | | | | | | | | | | | | |
| Godar | Sst, Tst, Kgl | | | | | | | * | * | * | * | * | * | | | | | | | | X | | | | | | | | | | | | |
| Haueda | Tst, Gyps | | | | | | | | | * | * | | X | | | | | | | | | | | | | | | | | | | | |
| Haune | Sst, Ust | | o | | * | * | X | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Haxter G. | Kst | | o | o | | * | * | X | | | | | | | | | | | | | | | | | | | | | | | | | |
| Huites | Granit | | o | o | | | | | | | * | * | * | * | * | * | * | | | | X | | | | | | | | | | | | |
| Jala | Kst | o | | | | | | | * | * | X | | | | | | | | | | | | | | | | | | | | | | |
| Karun | Kst | | | | | | | | * | * | * | * | * | X | | | | | | | | | | | | | | | | | | | |
| Korek | Kst | | o | o | | X | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Llosa | Sst | | | | o | o | | | | | | | | | | | | | | X | | | | | | | | | | | | | |
| Margarite | Gabbro | | | | | | | | | | | | | | | | | | | | | | | | | | | | | | >> | | X |
| Mingorria | Granit | | | | | | | | * | * | * | | | | X | | | | | | | | | | | | | | | | | | |
| Möhne | Schists | | | | | | * | * | * | * | * | | X | | | | | | | | | | | | | | | | | | | | |
| Moragahak. | Gneiss | | | | | | | | | | | | | * | * | * | * | * | * | | X | | | | | | | | | | | | |
| Ohra | Porphyr | | | | | | | | | * | * | * | * | * | * | * | * | | | | | | | | | X | | | | | | | |
| Palencia | Dol, Kst | | | | | | | | | * | * | * | * | X | | | | | | | | | | | | | | | | | | | |
| Panix | Kst | | | | | | | | | | * | * | | | | | | | X | | | | | | | | | | | | | | |
| Ponga | Slates | | | | o | o | | * | * | * | * | | X | | | | | | | | | | | | | | | | | | | | |
| Portas | Schists, Quazt | | | | o | | | | | | | * | * | * | * | * | | | | | | | | | | X | | | | | | | |
| Prims | Slates | | | o | o | | | | | | | | | | | | | | | | | * | * | | X | | | | | | | | |
| Pueblo viejo | Kst | | | | | | | | | | | | | | | | | | | | | * | * | * | * | * | * | * | * | * | >> | | X |
| Riansares | Tst, Gyps | | | | | | * | * | * | * | * | | X | | | | | | | | | | | | | | | | | | | | |
| Rupacha | Kst | | | o | * | X | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Tavera | Kgl, sst, Ust. | | | | | | * | * | * | * | * | * | | | | X | | | | | | | | | | | | | | | | | |
| Tous | Kst, Dol | | | | | | | | * | * | * | X | | | | | | | | | | | | | | | | | | | | | |
| Twiste | Sst, Ust | | o | o | * | * | X | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Tumaya | Kst | | o | | X | | | | | | | | | | | | | | | | | | | | | | | | | | | | |
| Up. Gotw. | Kst, Ust | | | | | | | o | | | | | * | * | * | | | | | | X | | | | | | | | | | | | |
| Villagudin | Schists | | | o | o | * | * | | | | | | X | | | | | | | | | | | | | | | | | | | | |
| Yuracmayo | Tuffite | | | | | | * | * | * | * | | | | X | | | | | | | | | | | | | | | | | | | |

- The borehole intersects the open paths prone to dilation at different depths.
- The depth of the zones of weathering differs between a few meters and many tens of meters.

In both cases the overburden pressure differs accordingly.

Underneath the zone of weathering begins the solid rock bond. There the overburden pressure is not effective any more but is replaced by the deformability of the rock along the water bearing paths. The water take begins to increase over-proportionately once the critical pressure is reached. This increase continues as long as the test pressure grows. But as the strength remains about the same with depth the critical pressures behave accordingly and differ only within an individual range of scattering.

Hydrofracturing of latent discontinuities takes place once the critical pressure is achieved being able to overcome the tensile strength across the planes and the compressibility of the rock. All types of discontinuities are not equally susceptible to fracturing. Particularly bedding planes originating from a change of the sediments are prone to cracking but also cleavage planes or parallel texture of planar crystals as mica, for instance.

Multiple pressure-steps tests are needed (Figs. 5.2c and 5.3 for instance) to recognize as exactly as possible the true courses of fracturing and the order of the critical pressure. For economical reasons it is usually impossible to carry out too many ascending and descending pressure steps. Instead we have to be content with a few pressure steps. This leads inevitably to a certain inaccuracy. In assessing the results of these analysis, that deficiency has to be taken into account.

This valuation focuses on hydrofracturing due to strength. The many examples presented prove that in un-weathered rock owning a connected bond the critical pressures do not increase with depth but vary within an individual range of scattering.

That individual scattering has geo-'*logical*' and technological reasons:

- The latent discontinuities found in the successive sections of the borehole are not completely identical but differ somehow with respect to their geometry and tensile strength.
- The pressures recorded in the working reports are not always the actually effective ones:

  * Many tests were carried out several tens of years ago when the pressure was usually recorded at the top of the borehole and not yet directly within the section tested as it later became possible. The effective pressure differs from the one at the top of the borehole because the hydrostatic head caused by the column of water inside the pipe as well as the counter-pressures possibly caused by groundwater and head losses due to friction are effective as well. These factors enlarge or reduce the pressures recorded at the top of the hole, thus a '*pressure correction*' is required to find out the effective one. Such corrections have been made in the evaluation used for this analysis. However, the parameters available are probably not always correct because many programs were carried out several

decades ago when the state of the art was less advanced and little attention was paid to the difference between pressure recorded at the surface and pressure effective in the borehole section.
* The latent discontinuities are tight prior to fracturing. Thus, at the beginning of the test the true pressure depends just on the pressure at the top of the hole and the static head, the counter-pressure is not yet effective.
Thus, a systematic failure might be involved whenever the test pressures reported referring to the top of the borehole differ from the effective ones. However, that failure is irrelevant for the purpose of this analysis because it merely makes the range of scattering somewhat smaller.

The ranges of scattering appearing in the various programs are different. These differences are caused by one—or more—of the following factors:

* The different angle between borehole and discontinuities;
* The different inclination of the slopes;
* The different intensities of relaxation underneath the slopes;
* The different heterogeneities of the rock types etc.

All these differences cannot be identified as such because of the possible failures mentioned above. Moreover, much more pressure steps and a precise execution and evaluation were required to recognize separately the factors and quantify their impact. Nevertheless: although the deficiencies impair the results—particularly the relation between the orientation of the discontinuities and the borehole—but they are evidently not that great as they could change the order of magnitude of the critical pressures, i.e. the influence of these factors is relatively small.

In Table 5.4 the critical pressures causing hydrofracturing due to strength have been classified separately using the criteria '*rock type*' and '*sharing of P/Q-Types*'.

#### 5.7.22.1 Rock Type

Un-weathered granites usually have larger compression strength and modulus of elasticity than young sedimentary rocks. Consequently, granites should have substantially higher critical pressures as bedded sandstones or limestone, for instance. Surprisingly that is not true. Perhaps granites and similar strong rocks show a wider range of scattering, however the order of magnitude of the critical pressures observed in all the rock types analyzed differs just a little: latent discontinuities are fractured also in granites at pressures between 5 and 10 bar as they are in slates or young sediments.

Nevertheless, within the same order of magnitude appear certain differences due to their geological conditions:

* The low critical pressures between $> 3 < 7$ and $> 3 < 5$ bar, respectively observed in the programs CAN and VIL correspond to their rock types—cleavable slates.

**Table 5.4** Grouping of projects considering their rock types and their portions of P/Q-Diagram-Types 1, 2 and 3, abbreviations for rock types are given in Table 5.1

Abbreviations for rock types are given in Table 4.1

| Assigned to | Project | Rock type | $P_{crit}$ (bar) due to | | Further cases of fracturing require for P/Q-1 and P/Q-2, $P_{crit} > P_{max}$ | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Heaving | Strength | P/Q-1 | P/Q-2 | P/Q-3 | $P_{max}$ |
| *Criterion "Rock type"* | | | | | | | | |
| Group A | BUR | Granite | > 3 < 6 | > 5 < 9 | 0 | 0 | 0 | 10.1 |
| | EDRA | Granite | > 1 < 4 | | 1 | 0 | 0 | 10.9 |
| | MIN | Granite | | > 5 < 10 | 20 | 0 | 8 | 13.2 |
| | ALB | Granite, Gneiss | > 2 < 5 | > 6 < 12 | 292 | 0 | 201 | 12.3 |
| | ALM | Granite, Gneiss | > 1 < 5 | > 5 < 15 (<25) | 26 | 0 | 21 | 26.4 |
| | CER | Granite | > 3 < 5 | > 6 < 10 | 20 | 0 | 8 | 11.7 |
| Group B | EUL | Schists | 2.5 | > 6.5 < 8 | 142 | 0 | 39 | 11.2 |
| | CAN | Quarzite, Schists | > 2 < 5 | > 3 < 7 | 3 | 0 | 8 | 15 |
| | COS | Schists | > 2 < 4 | > 5 < 9 | 0 | 0 | 0 | 11 |
| | PON | Slate | > 3 < 5 | > 6 < 10 | 0 | 0 | 0 | 11.9 |
| | VAD | Schists | | > 5 < 10 | 0 | 0 | 0 | 12 |
| | VIL | Schists | > 2 < 4 | > 3 < 5 | 0 | 0 | 0 | 11.2 |
| Group C | AIX | Mst, Tst, Sst, Kst | > 1 < 2 | >3.7 | 0 | 0 | 1 | 4.2 |
| | ARG | Tst, Sst, Mst | <5 | > 7 < 12 | 3 | 0 | 0 | 15.50 |
| | ARR | Kst, Sst, Tst | > 1 < 3 | > 4 < 5 | 0 | 0 | 0 | 5.4 |
| | BEN | Mst, Sst, Tst | | > 6 < 10 | 13 | 4 | 6 | 13.2 |
| | DONA | Mst, Sst, Ust | | | 0 | 0 | 0 | 10.8 |
| | LAR | Tst, Ust, Sst, Kst | | | 0 | 0 | o | 4.5 |
| | LLO | Kst, Sst, Tst, Kgl | > 3 < 5 | | 6 | 1 | 6 | 13.1 |
| | PAL | Dol, Kst | | > 8 < 12 | 1 | 1 | 3 | 12.4 |
| | RIA | Gy, Tst | | > 5 < 10 | 7 | 3 | 13 | 11.2 |

(continued)

**Table 5.4** (continued)

Abbreviations for rock types are given in Table 4.1

| Assigned to | Project | Rock type | $P_{crit}$ (bar) due to | | Further cases of fracturing require for P/Q-1 and P/Q-2, $P_{crit} > P_{max}$ | | | |
|---|---|---|---|---|---|---|---|---|
| | | | Heaving | Strength | P/Q-1 | P/Q-2 | P/Q-3 | $P_{max}$ |
| *Criterion "Portion of P/Q-Types"* | | | | | | | | |
| Group I | BUR | Granite | > 3 < 8 | > 5 < 9 | 0 | 0 | 0 | 10.1 |
| | COS | Schists | > 2 < 4 | > 5 < 9 | 0 | 0 | 0 | 11.00 |
| | PON | Slate | > 3 < 4 | > 6 < 10 | 0 | 0 | 0 | 11.9 |
| | VIL | Schists | > 2 < 4 | > 3 < 5 | 0 | 0 | 0 | 11.2 |
| | ARR | Kst, Sst, Tst | > 1 < 3 | > 4 < 5 | 0 | 0 | 0 | 5.4 |
| | DONA | Mst, Tst, Ust | | | 0 | 0 | 0 | 10.8 |
| | LAR | Tst, Ust, Sst, Kst | | | 0 | 0 | 0 | 4.5 |
| Group II | EDRA | Granite | > 1 < 4 | | 1 | 0 | 0 | 10.90 |
| | ARG | Tst, Mst, Mst | <5 | > 7 < 12 | 3 | 0 | 0 | 15.50 |
| Group III | ALM | Granite | > 1 < 5 | > 5 < 15 (25) | 26 | 0 | 21 | 26.4 |
| | MIN | Granite | | > 5 < 10 | 20 | 0 | 8 | 13.20 |
| | ALB | Granite, Gneiss | > 1 < 5 | > 5 < 12 | 292 | 0 | 201 | 12.3 |
| | CER | Gneiss | > 3 < 5 | > 8 < 10 | 20 | 0 | 9 | 11.7 |
| | EUL | Schists | 2.5 | > 6.5 < 8 | 142 | 0 | 39 | 11.2 |
| | CAN | Quarzite, Schists | > 2 < 5 | > 3 < 7 | 3 | 0 | 8 | 15 |
| | BEN-1 | Mest, Tst, Ssst | | > 6 < 9 | 12 | 0 | 4 | 13.2 |
| | BNE-9 | Mest, Tst, Ssst | | > 5 < 10 | 1 | 4 | 2 | 13 |
| | LLO | Kst, Sst, Tst, Kgl | > 3 < 5 | | 6 | 2 | 6 | 13.1 |
| | PAL | Dol, Kst | | > 8 < 12 | 1 | 2 | 3 | 12.4 |
| | RIA | Gy, Tst | | > 5 > 10 | 7 | 3 | 13 | 11.20 |

* By contrast, the thick banks of dolomites and limestone found in the program PAL yielded higher critical pressures (> 8 < 12 bar) obviously reflecting the larger strength.

#### 5.7.22.2 Shares of P/Q-Types 1–3

With respect to the shares of P/Q-Types 1–3 it has to be distinguished among three groups (Table 5.1):

* The programs without P/Q-Types 1–3 constitute the first group which contains only P/Q-Types 4–6: all test sections were originally permeable. It is noteworthy that all programs carried out in slates belong to this group.
* The second group comprises two programs whose test sections remained impermeable: P/Q-Type 1. Sections with fracturing are lacking—either the critical pressures were still higher than the maximum pressures applied (10.5 and 15.5 bar) or latent discontinuities do not exist in these sections.
* The third group consists of programs containing P/Q-Types 1 and 3. The explanation given before applies analogously: either susceptible planes are rare or the maximum test pressure was not always high enough to exceed the critical one. This reason would extend the range of scattering.

### *5.7.23 Summary*

The critical pressures causing hydrofracturing have been assigned to the following 7 groups between 5 and 30 bar each of them forming an interval of 5 bar. Those three cases with $P_{crit} > P_{max}$ are disregarded. The shares are distributed as follows:

$$
\begin{aligned}
P_{crit} &= <5\,\text{bar}:\ 17.8\,\% \\
P_{crit} &= 5-10\,\text{bar}:\ 64.4\,\% \\
P_{crit} &= 10-15\,\text{bar}:\ 11.1\,\% \\
P_{crit} &= 15-20\,\text{bar}:\ 0.0\,\% \\
P_{crit} &= 20-25\,\text{bar}:\ 2.2\,\% \\
P_{crit} &= 25-30\,\text{bar}:\ 2.2\,\% \\
P_{crit} &= >30\,\text{bar}:\ 2.2\,\%
\end{aligned}
$$

Although this distribution may not apply in the same relationship to all rock types existing worldwide, its tendency can be considered appropriate. Conclusively, approximately 82% of all cases of hydrofracturing begin at critical pressures smaller than 10 bar and 93% of all cases are fractured at pressures smaller than 15 bar. Rock types with larger strengths of their rock bonds are evidently very rare.

## 5.8 Mechanism of Hydrofracturing Processes

This analysis demonstrates that in about 80% of all rock types the critical pressures due to strength vary between 3 and 10 bar. They are scattered in an individual range but do not increase with depth. At first glance, the pressures seem to be surprisingly low because many of the rock types analyzed are quite strong. However, the decisive factor is not the strength of the rock itself but the strength of the rock bond, particularly the tensile strength across the discontinuities. This strength is evidently not larger in strong but cleavable rock as it is in well-bedded alternative sequences such as sand-, silt- or claystone. By contrast, massive rocks without such planes are not susceptible—massive limestone or granites without a prevailing texture, for instance.

The susceptibility to hydrofracturing and its extent depends on the following factors:

* The availability of latent discontinuities;
* The angle of intersection between borehole and susceptible plane;
* The drilling technique;
* The stress field at the entrance of a susceptible plane;
* The stiffness of the rock bordering the plane being fractured;
* The development of pressure during fracturing and subsequent grouting.

### *5.8.1 Type of Latent Discontinuities Susceptible to Fracturing*

Particularly susceptible to fracturing are bedding and cleavage planes or intercalated layers of cleavable minerals, mica above all. The parallel texture of crystals in metamorphic rock makes the rock also susceptible to fracturing—as in gneisses and quartzites.

### *5.8.2 Angle of Intersection Between Borehole and Susceptible Plane*

The intersection between borehole and plane at a right angle makes fracturing easier, i.e. more likely to occur—vertical holes and horizontal beddings, for instance. By contrast, intersection at acute angles impedes fracturing.

### *5.8.3 Drilling Technique*

The various drilling techniques in use produce either a smooth or an uneven wall of the borehole: rotary drilling combined with diamond bits tend to yield a rather smooth surface, percussion drilling combined with TC-bits produce rough surfaces. If softer rock containing weak bedding planes is intersected, the drilling bit cuts a notch into the wall along such planes.

Its depth and width depend on the type of drilling: small in rotary/diamond-drilling, wide in percussion/TC-drilling.

### 5.8.4 Hydraulic Jack

The susceptibility to fracturing increases with the depth and the width of that notch because deeper and wider wedges carved into the wall allow a larger volume of grout to enter. Provided the other factors are equivalent, that plane is fractured first allowing the largest wedge to be carved into the wall. The grout conveys the pressure into the wedge and begins to act as a hydraulic jack. Its effect increases while progressive fracturing opens further sections allowing more grout to enter. The effect of the injected water or grout as a hydraulic jack increases with the growing contact area.

### 5.8.5 Stress Field at the Entrance of a Susceptible Plane

The stress field acting at the wall of the borehole section tested or grouted shall not be discussed here in detail. Presenting the general conditions is hardly useful, because each relationship between the orientation of the discontinuities and the direction of the borehole has to be considered individually. Since we are confronted with a large variety of different situations the scope of this distribution does not allow discussing individual cases. In general, fracturing begins—and continues—once the stress breaking up the plane overcomes the tensile strength across the plane. Fracturing begins gradually and is then a continuous process also supported by the stiffness of the rock.

### 5.8.6 Self-Induced Chain Reaction Due to Stiffness of Rock Bordering the Plane Being Fractured

As soon as the notch begins to be pressured open and the first drop of water or grout penetrates the wedge, the strength of the rock against bending becomes effective. It prevents the adjacent rock from getting bended—or even ruptured—and to keep its form. The bending strength and the effect of the water or grout pressed into the wedge acting as a hydraulic jack cause a continuation of the splitting along the plane. Water or grout are necessarily forced to follow otherwise a vacuum were formed between its front and that part of the plane still being connected. Thus, the fracturing of a latent plane continues as a ‘self-induced’ chain reaction, provided the grouting pressure was high enough to initiate fracturing and remains high enough not to interrupt that process. The layers or flat pillows of groutstone deposited along discontinuities pressured open confirm that the mechanism takes place this way. The P/Q-diagram given in Fig. 5.3 suggests with its numerous pressure steps that hydrofracturing is presumably a gradual process.

### *5.8.7 Development of Pressure During Fracturing and Subsequent Grouting*

Figure 3.27a exemplifies the changes of the pressure occurring during and after the moment of fracturing. The graph shows the course of a grouting test:

* First phase: No grout take took place at the low pressure of 2 bar.
* Second phase: A latent bedding plane fractured suddenly once the critical pressure of 6.5 bar was reached. The absorption began and the pressure dropped to 4.5 bar over approx. 10 min. Then the pressure dropped slowly and reached 3 bar when grouting was stopped arbitrarily after another 85 min.

The decrease of the pressure during the grouting process complies with the working method of a hydraulic jack (growing contact area) and indicates a steady filling of the fissure pressured open just in front of the grout.

That behavior of the grouting pressure does not necessarily occur in natural water paths. There pressure and flow velocity usually decrease with growing distance from the hole and cement grains are deposited obstructing the paths where the velocity falls short of the critical one initiating sedimentation. This clogging causes the pressure to go up. Whether that increase is a temporary one washing the clog away depends on the local conditions. Usually sedimentation, obstruction, pressure increase, erosion and pressure drop occur repeatedly, with the next pressure peak always a bit higher—and so on until the maximum grouting pressure defined is reached (Fig. 3.27b).

## 5.9 Results of Grouting Programs Using Conventional Grouting Pressures

As mentioned at the outset and illustrated in Fig. 5.28, in accordance with the conventional concept for curtain grouting the grouting pressure should increase with depth. Different increase rates are foreseen for soft and hard rock, respectively. Moreover, American and European grouting people recommended different rates.

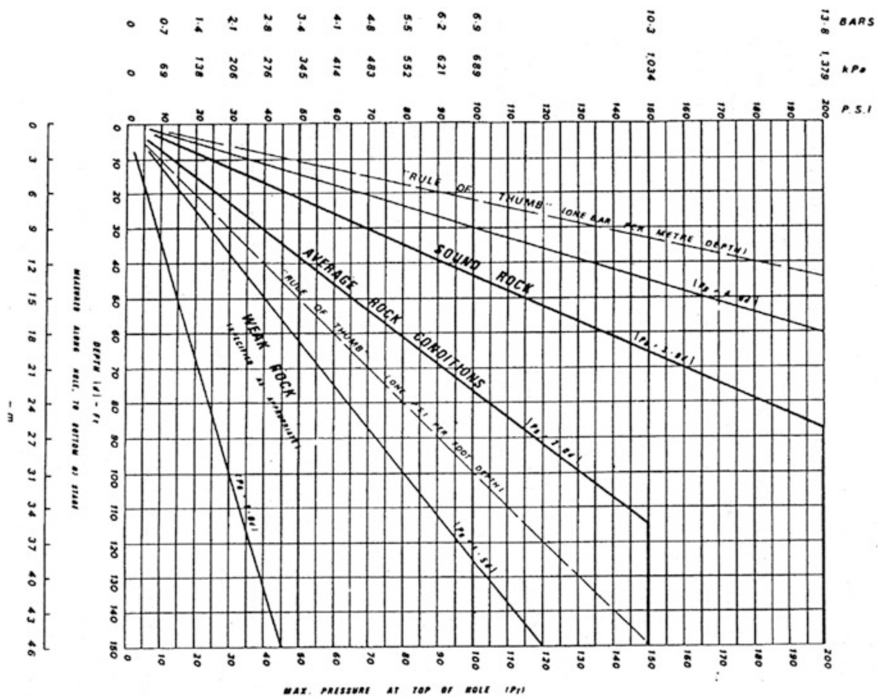


**Fig. 5.28** Increase of grouting pressure with depth, different increase rates for soft and hard rock types according to Houlsby ($x$—pressure, *above* given in psi (kpa and bar, *below* given in psi for the top of the hole), $y$—depth, given in feet, and m)

The author has repeatedly argued and substantiated that this concept is not appropriate because it does not meet the properties of rocks.

The concept of such an increasing pressure compromises between two contrary requisites:

* To avoid dilation due to heaving the grouting pressure shall be small near to the surface where the overburden pressure is low.
* To achieve a wide spreading of the grout suspension injected, higher grouting pressure should be applied in the deeper zones.

According to Fig. 5.28 in strong rock the pressure is allowed to increase at a rate of 1 bar/1 m depth, which means that a pressure of 20 bar can be applied at a depth of 20 m. The analysis presented here demonstrates that that rule does not meet the rock mechanical requirements: the critical pressures are mostly much smaller than the grouting pressures defined by the conventional concept. Moreover, contrarily to the former assumption the critical pressures do not increase with depth because underneath the zone of weathering they are not determined by the overburden pressure but by the strength of the rock bond. This strength depends primarily on the tensile strength across the latent discontinuities. Therefore, even rock types possessing a large modulus of elasticity can be fractured along latent discontinuities at relatively low pressures.

Using grouting pressures increasing with depth results for deeper zones in higher grouting pressures, of course. At first glance, it seems to be consistent applying staggered depths for the groutholes of the subsequent series as displayed in Fig. 5.29 because higher grouting pressures produce a farther travel of the suspension. However, if latent discontinuities susceptible to fracturing exist that approach is not appropriate and the grouting pressures should not increase with depth: they would soon exceed the critical pressures fracturing those planes while that farther travel expected would not be achieved; thus, the holes of the later series should be drilled to the same final depth.

The idea that the overburden pressure determined the hydrojacking and hydrofracturing behavior of deeper zones was wrong from the very beginning: If the wall of a borehole is compressed the deformation of the rock does not depend on the weight of the overlying rock but on the strength of the rock around the wall.

The reduction of the pressure around a borehole occurs principally similar to the reduction of the pressure around a tunnel: As long as the rock is solid or slightly weathered only and the rock bond is largely connected, the load of the rock rests on the roof forming an arch which diverts the pressure around the tunnel into the ground. If weathering reduced the rock bond technical support is required to keep the roof stable. If the rock bond is completely disturbed, the support has to carry the entire overburden pressure. Under extreme conditions the rock collapses completely resulting in an open shaft up to the surface.

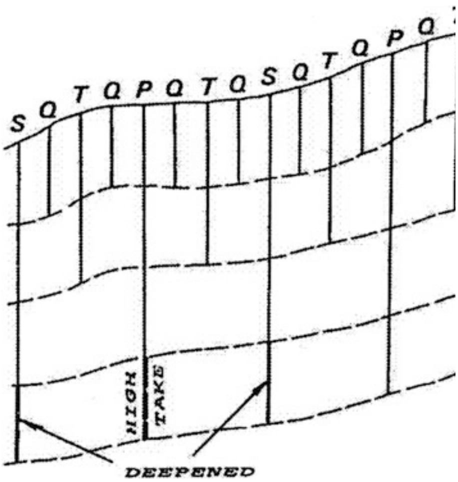



**Fig. 5.29** Staggered depth for grout holes of subsequent series

The examples Tavera and Almendra illustrated in Sects. 3.1 and 3.5 already the effect of too high a grouting pressure; they are not singular cases but occur frequently:

- Within the curtain grouting executed at the Almendra Dam the grout takes were very small when low grouting pressures were used. This corresponds very well with the reduced permeability of this rock type. By contrast, grouting pressures >20 bar yielded very large grout takes even in tight rock—evidently because of fracturing.
- The behavior recorded at the Tavera Dam was quite similar. WPT's reached up to 12 LU, but grout takes at pressures <10 bar were very small—just sufficient to backfill the hole; on the contrary, pressures $\geq 20$ produced sizeable grout takes in spite of the nearly tight rock, expressed by WPT-results between 0 and 2 LU.

## 5.10 Changes of Absorption Capacity Due to Hydrojacking and Hydrofracturing

Instead of the conventional Lugeon-values the author prefers to use the absorption parameters $Q_1'/Q_{10}'$ and $Q_1^*/Q_{10}^*$ because they permit recognizing the deformation of water-conducting paths either by hydrojacking or hydrofracturing. Comparing for both parameters ($Q_1'/Q_{10}'$ and $Q_1^*/Q_{10}^*$) the mean values for the average takes and the summation lines indicating the relative frequency distribution, permits to recognize very clearly the effect of hydrojacking or hydrofracturing.

These parameters permit a better assessment as the following example demonstrates:

* For the WPT-results carried out in 6 boreholes Fig. 5.30 shows the mean values of the absorption parameters $Q_{10}'$ and $Q_{10}^*$.
* Figure 5.31 gives for these parameters the relative frequency distribution in form of summation lines.

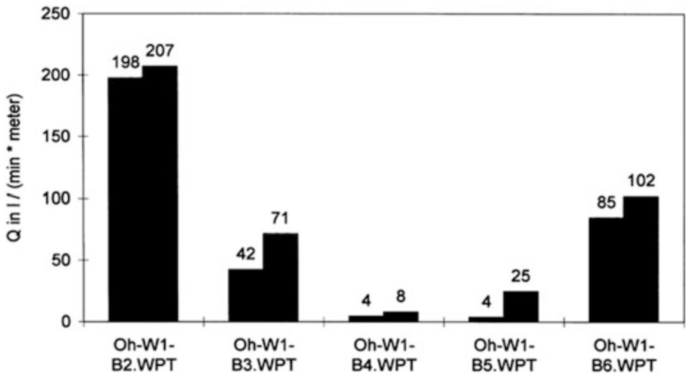



**Fig. 5.30** Differences between mean values of $Q_{10}'$ (*left*) and $Q_{10}^*$ (*right*) for WPTs carried out in single holes characterizing a section of individual permeability

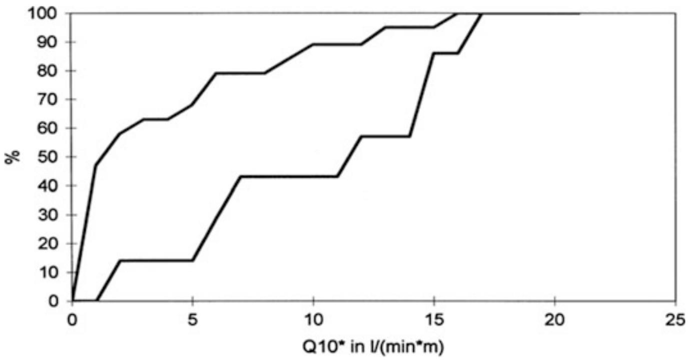



**Fig. 5.31** Relative frequency distribution for the WPT-results obtained in Hole B 5, differences between $Q_{10}'$ (*left*) and $Q_{10}^*$ (*right*)

The results obtained in all boreholes differ considerably from each other because the section around each hole has its own hydrogeological setting and mechanical behavior. The scope of deformation due to strength is different from case to case:

* In Hole Oh-W1-B2 the relative difference between the average takes of $Q_{10}'$ and $Q_{10}^*$ is rather small: 198 versus 207 LU.
* By contrast, Hole Oh-W1-B5 shows a large difference between the average takes of $Q_{10}'$ and $Q_{10}^*$: 4 versus 25 LU.
* These discrepancies are convincingly expressed also by the different position of the summation lines for $Q_{10}'$ and $Q_{10}^*$, respectively.

## 5.11 Comparison Between Critical Pressures Occurring in WPT's and Grouting

This contribution analyses the critical pressures determined by means of WPT's. It has been stated already that their critical pressures refer to the effective ones obtained either by direct measurement or by pressure correction. The question arises whether the critical pressures occurring in grouting share the same level in spite of the different properties of water and grout. As far as known this complex has not been investigated by direct measurements in the borehole section treated. Such investigations are methodologically difficult: Lab tests are hardly helpful because the results are not transferable to the practical work. So far the grouting pressures are recorded at the top of the borehole and the effective pressures are estimated by adding the weight of the grout suspension depending on the Water/Cement-Ratio between the top of the hole and the depth of the section treated. For the time being the critical pressures causing hydrofracturing of latent discontinuities in WPT's and in grouting can be compared only indirectly.

For the WPT's the effective pressures have been considered. The grouting pressures are taken as recorded at the surface because a pressure correction is impossible. These pressures are taken into account also because in a worldwide scale the grouting pressures are mostly recorded at the top of the borehole.

The two cases of Tavera and Almendra exemplarily presented in Figs. 3.1–3.6 and 3.20–3.24, respectively, standing for many others demonstrate that hydrofracturing occurs also in grouting—and quite often. Their data do not really disclose whether the critical pressures share the same order as in WPT's. It seems at first glance that those observed in grouting were larger: the critical pressures identified in WPT's ranged at Almendra between 5 and 15 bar and at Tavera between 5 and 11 bar, respectively, in both projects the large grout takes took place at pressures larger than 20 bar.

However, also opposite cases occur: At Haune dam the pressures causing fracturing recorded at the top of the hole seemed to be smaller in grouting compared to WPT's. There an extremely sensitive hydrofracturing behavior was experienced: the latent beddings planes fractured already at effective pressures between 2 and 3 bar. In view of that behavior similar pressures were applied also in grouting. They opened the bedding planes and filled them with grout. Just the larger density of the suspension was enough to crack the beddings.

The following reflections clear up whether the critical pressures cracking the latent discontinuities in WPT's and grouting are principally different or share the same order in spite of the dissimilar properties of water and grout:

- In the initial phase of testing or grouting when water or grout fill an impermeable borehole section the pressure begins to increase. As long as fracturing does not occur water or grout are not flowing, thus their different flow properties are not effective yet. In both cases the critical pressures cracking the latent planes should be about the same because the strength of the rock bond is the determining factor while the flow properties are still irrelevant. Equal pressures at the top of the hole yield higher effective pressures in grouting because of the larger density of the suspension ($\gamma_{\text{Suspension}} \geq 1 \leq 1.6$ g/cm$^3$; $\gamma_{\text{Water}} = 1$ g/cm$^3$). Conclusively, with respect to the pressure at the top of the hole, fracturing in grouting should start at a lower pressure already compared to a WPT.
- The density of the suspension depends on the water/cement ratio; the smaller the W/C-ratio the larger the density:

  * W/C = 2 yields $\gamma_{\text{Suspension}} = 1.29$ g/cm$^3$;
  * W/C = 1.5 yields $\gamma_{\text{Suspension}} = 1.37$ g/cm$^3$;
  * W/C = 1 yields $\gamma_{\text{Suspension}} = 1.52$ g/cm$^3$;
  * W/C = 0.7 yields $\gamma_{\text{Suspension}} = 1.67$ g/cm$^3$;

- The effect of the larger density increase with the depth of the section:

  * Using a suspension of W/C = 0.7, a depth of 10 m causes additional pressure of 1.7 bar;
  * Using a suspension of W/C = 0.7, a depth of 50 m causes additional pressure of 8.5 bar.

The depth of the sections considered in Almendra and Tavera ranged between 50 and 80 m, those in Haune only between 5 and 10 m.

- Simultaneously with the fracturing water or grout penetrate the new fissure, hence flowing begins and the different properties become effective. For water the head losses due to friction at the entrance and along the path are smaller—and, vice versa: higher pressures are needed to move the suspension because its flow activate larger head losses.
- In testing and grouting fracturing itself should occur at approximately equal pressures, however the pressures are first recognized and registered after the beginning of the flow, i.e. when the different flow conditions for water and grout have adjusted already and are not identical any more. Then the higher pressure required for the flow of the suspension, also including the head losses due to friction, become effective and are recorded.

In cases like Almendra and Tavera the discrepancy between the possibly similar pressures initiating fracturing and the dissimilar ones being recorded during the flow gives the impression that fracturing needs higher critical pressures in grouting rather than in testing. Cases like Haune apparently indicate the opposite. Under the conditions that in WPT's the critical pressures refer to the effective pressures while in grouting the critical pressures are related to the pressures recorded at the top of the borehole, we have to distinguish among the following groups:

* If the grouting pressure gains more due to density than it loses due to friction, the critical pressures observed in grouting seem to be larger than in WPT's.
* If the grouting pressure loses more due to friction than it gains due to density, the critical pressures seem to be smaller in grouting than in WPT's.
* If head losses and gains compensate each other, the critical pressures in grouting and testing are similar.

The increase due to density depends (a) on the W/C-ratio and (b) on the depth of the section treated. The head losses due to friction are determined by the flow properties of the suspension and the geometry of the paths: the narrower the paths and the larger the viscosity the higher the pressure to initiate the penetration of the paths and keep the suspension flowing to reach sufficient spreading of the grout.

## 5.12 Hydraulic Fundamentals of Testing and Grouting Processes

In testing and grouting of open paths, pressures to overcome the head losses due to friction are needed to penetrate them and to maintain the flow. The height of the pressure required depends on the size and the shape of the paths, the roughness of the walls bordering the paths and the flow properties of the materials injected. The narrower the paths and the larger the cohesion of the material the higher the pressure required—and vice versa. The injection of water and grout develops differently: the flow of water can be endless; the flow of grout tapers off as soon as the path is clogged due to sedimentation—as intended to seal the paths. Since the LUGEON-criterion plays such an important role the author carried out lab tests to find out for various sizes and shapes those paths being capable of absorbing that specific amount of water defined as LUGEON-unit and to quantify the pressure required to grout them.

The tests revealed that a planar shaped conduit of 0.3 mm in width and 20.8 mm in width yielded water takes between 0.9 and 1.3 LU, depending on the length of the path: 300 and 900 mm, respectively. A grouting pressure of 12 bar was needed to penetrate that path using a suspension of W/C-ratio = 1. The flow through that path stopped once that the pressure dropped below 9 bar.

With growing width of the paths the absorption capacity increases yielding larger LUGEON-values while the required grouting pressures decrease accordingly. Other widths and shapes yield different results, of course. Each real fissure possesses its specific relation between pressure required versus geometry and conductivity. The lab tests confirmed what had to be expected for the relation between width, conductivity and pressure required. Moreover—and more important—the lab tests identified the geometry of those paths absorbing the water take of 1 LU which is tiny indeed; it also quantified the pressure required to grout such paths. The results are in conformity with the fundamentals of hydraulics as calculations confirmed. The respective details are presented in Chap. 2.

## 5.13 Consequences

### *5.13.1 Hydrofracturing and Individual Groutability*

To overcome the head losses due to friction high grouting pressures are needed to grout narrow paths. The grouting pressure required increases inversely proportional to the width of the paths. The cohesion of the suspension is an effective property too: thicker suspensions need higher pressures—and vice versa. Figure 5.32 summarizes the results of the lab tests introduced above:

* The grouting pressure required to penetrate the path and keep the grout flowing increases with decreasing width of path.
* Critical pressure fracturing latent planes is determined by individual strength.

The relationship between the geometry of the path, the grouting pressure required and the hydrofracturing behavior necessarily leads to the conclusion that each rock type possesses its own individual groutability. It originates from the interplay of:

* the critical pressures ($P_{crit}$),
* the grouting pressures required ($P_{req}$), and
* the actual grouting pressure ($P_{grt}$).

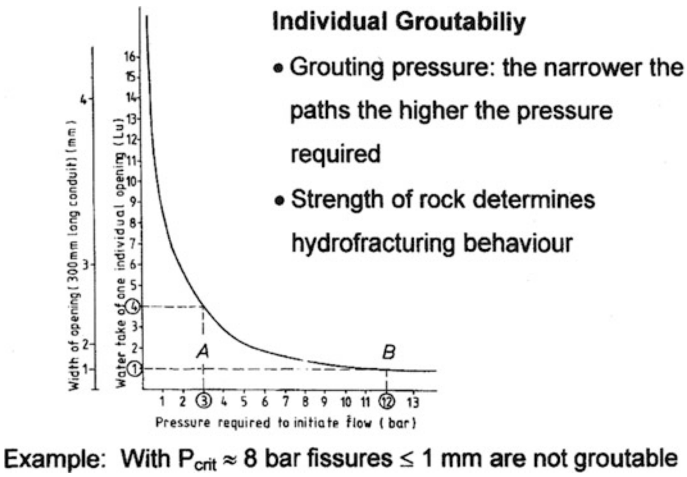



**Fig. 5.32** Relationship between width and grouting pressure determined on geologically defined models

Since each of these factors differs, we have a great variety of individual conditions.

The examples illustrated in Fig. 5.32 demonstrate the consequences for a rock mass containing latent planes fracturing at $P_{crit} \approx 8$ bar:

- The fissure of 2.5 mm in width and 300 mm in length and absorbing a water take of 4 LU can be grouted because the grouting pressure required is smaller than the critical pressure; Case A: $P_{req} < P_{crit}$.
- The fissure of 1 mm in width, 20.8 mm in breadth, 300 mm in length and absorbing a water take of 1 LU cannot be grouted because the grouting pressure required is higher than the critical pressure; Case B: $P_{req} > P_{crit}$.

The grouting program carried out at the Twiste Dam introduced in Sect. 3.2 exemplifies convincingly the significance of the relation $P_{req} > P_{crit}$: Even middle-sized fissures requiring grouting pressures in the order of 4 bar were not accessible any more since fracturing took place already at critical pressures between 2 and 3 bar. That very poor groutability made it impossible to reduce the WPT-values as desired and to accept finally a residual permeability in the order of 30 LU. Figure 5.33 sketches the condition: fine paths remain ungrouted if $P_{req} > P_{grt} > P_{crit.}$

If latent discontinuities exist in the section being grouted the practical consequences are as follows:

- Rock types owning substantial strength causing relatively high critical pressures ($\geq 1$ bar), paths smaller than 1 mm are not groutable.
- Rock types of little strength causing relatively low critical pressures ($\leq 10$ bar), paths smaller than 2 mm are not groutable either.

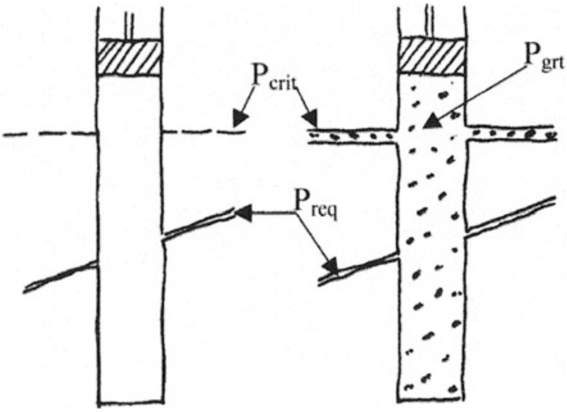



**Fig. 5.33** Open paths remain ungrouted if latent planes are fractured before

- At first glance one could assume that the LUGEON-value itself permits already an assessment whether a grouthole section is groutable—in the sense that
  - * small values indicating narrow paths need high grouting pressures and, thus can be treated only if latent planes prone to fracturing do not exist, while
  - * large values indicating wide fissures are groutable at low pressures already, even if latent planes exist.

The first conclusion is correct, the second one not because whenever the large LUGEON-values do not originate from one wide fissure but from many narrow paths encountered within the same borehole section they are not groutable if the relation $P_{crit} < P_{req}$ applies. Exactly that condition rules the grouting work in many rock types.

In most projects it is still custom to decide on the execution of a grouting program on the basis of Lugeon-values: if they exceed 1 LU that rock will be grouted. Of course, large grout takes are injected if sufficiently high grouting pressures are applied. They seem to prove the necessity and the successful execution of the work. That success is taken for granted because the results are usually not analyzed with thoroughness and a certain skepticism. That concept implicitly postulates that everything desired is achievable. However, that assumption is hasty and often not justified. Large grout takes themselves do not confirm already the necessity of a grouting program and its successful execution but often rather the contrary because hydrofracturing also produces such results. In view of the great variety of different conditions of all the rock types we are dealing with, we should conclude that an optimal result (most impermeabilization at least costs) can only be achieved if the relevant properties of a given rock form the basis for the design, the execution and the expected result of the grouting work.

### 5.13.2 Hydrofracturing Restricts Width and Depth of Groutable Paths

At first glance grouting of narrow paths seems not to be an important issue: they cause WPT-values on the order of one or a few LU-units only. They produce a sufficiently small permeability which usually does not warrant expensive grouting measures. By contrast, large WPT-values can originate from different situations:

- The water can be absorbed by a few wide fissures causing a large permeability, such rock types are groutable.
- The water can be absorbed from many narrow paths. Despite sizable water takes the permeability of an extended rock section is usually small: The permeability depends primarily on the size and shape of the individual paths. The narrower the paths, the smaller the permeability; the content of voids is comparatively meaningless. The reliance on the size is comparable to the situation of porous soils: the big pores in gravel produce a much larger permeability than the very small ones in clay although the latter one may have a much greater void content.

Large WPT-values originating from a multitude of narrow paths pose a problem: high grouting pressures are required but they are not achievable if latent planes are fractured before at lower pressures. Thus, under the condition of $P_{crit} < P_{req}$ narrow paths cannot be grouted if such planes exist. The critical pressures are often quite low as exemplified above for the case of the Twiste Dam. In such cases even middle-sized fissures are not groutable any more.

In addition to the critical pressures and the grouting pressures required, the W/C-ratio is also effective. It determines the weight of the suspension ($\gamma$), which increases proportionately with the content of cement, i.e. inversely with the W/C-ratio. The weight of the suspension ($\gamma$) and the depth of the section being grouted (H) determine the static head ($P_H = \gamma \times H$), which enlarges the effective pressure ($P_{eff}$). Further factors are

* $P_W$—the counter-pressure due to groundwater, if available above the borehole section treated, and
* $P_F$—the head losses due to friction if the suspension is flowing.

In most cases practiced worldwide the grouting pressure is recorded at the top of the borehole ($P_M$). Thus, the equation introduced in Fig. 4.3 applies for the effective pressure: $P_{eff} = P_M + P_H - P_W - P_R$.

Table 5.5 adds up the effective pressures $P_{eff}$, separately for the suspensions of W/C = 2; W/C = 1.5; W/C = 1; W/C = 0.7; the two other factors vary for $P_M$ between 0 and 20 bar and for $P_H$ between 10 and 100 m. Counter-pressure due to groundwater ($P_W$) and head losses due to friction ($P_F$) remain disregarded because it is assumed that the borehole section treated is located above the groundwater table and does not intersect open paths but latent planes only.

**Table 5.5** Factors determining effective pressures: pressure at the top, weight of suspension and depth

| Suspension | | Borehole | Pressure at top of borehole | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| W/C | γ (g/cm$^3$) | Depth (m) | 0 (bar) | 2.5 (bar) | 5 (bar) | 7.5 (bar) | 10 (bar) | 12.5 (bar) | 15 (bar) | 17.5 (bar) | 20 (bar) |
| **2** | **1.29** | **10** | 1.3 | 3.8 | 6.3 | 8.8 | 11.3 | 13.8 | 16.3 | 18.8 | 21.3 |
| | | 20 | 2.5 | 5.0 | 7.5 | 10.0 | 12.5 | 15.0 | 17 5 | 20.0 | 22.5 |
| | | 30 | 3.9 | 6.4 | 8.9 | 11.4 | 13.9 | 16.4 | 18.9 | 21.4 | 23.9 |
| | | 40 | 5.1 | 7.6 | 10.1 | 12.6 | 15.1 | 17.6 | 20.1 | 22.6 | 25.1 |
| | | 50 | 6.4 | 8.9 | 11.4 | 13.9 | 16.4 | 18.9 | 21.4 | 23.9 | 26.4 |
| | | 60 | 7.7 | 10.2 | 12.7 | 15.2 | 17.7 | 20.2 | 22.7 | 25.2 | 27.7 |
| | | 70 | 9.0 | 11.5 | 14.0 | 16.5 | 19.0 | 21.5 | 24.0 | 26.5 | 29.0 |
| | | 80 | 10.3 | 12.8 | 15.3 | 17.8 | 20.3 | 22.8 | 25.3 | 27.8 | 30.3 |
| | | 90 | 11.6 | 14.1 | 16.6 | 19.1 | 21.6 | 24.1 | 26.6 | 29.1 | 31.6 |
| | | 100 | 12.9 | 15.4 | 17.9 | 20.4 | 22.9 | 25.4 | 27.9 | 30.4 | 32.9 |
| **1.5** | **1.37** | **10** | 1.4 | 3.9 | 6.4 | 8.9 | 11.4 | 13.9 | 16.4 | 18.9 | 21.4 |
| | | 20 | 2.7 | 5.2 | 7.7 | 10.2 | 12.7 | 15.2 | 17.7 | 20.2 | 22.7 |
| | | 30 | 4.1 | 6.6 | 9.1 | 11.6 | 14.1 | 16.6 | 19.1 | 21.6 | 24.1 |
| | | 40 | 5.5 | 8.0 | 10.5 | 13.0 | 15.5 | 18.0 | 20.5 | 23.0 | 25.5 |
| | | 50 | 6.9 | 9.4 | 11.9 | 14.4 | 16.9 | 19.4 | 21.9 | 24.4 | 26.9 |
| | | 60 | 8.2 | 10.7 | 13.2 | 15.7 | 18.2 | 20.7 | 23.2 | 25.7 | 28.2 |
| | | 70 | 9.6 | 12.1 | 14.6 | 17.1 | 19.6 | 22.1 | 24.6 | 27.1 | 29.6 |
| | | 80 | 11.0 | 13.5 | 16.0 | 18.5 | 21.0 | 23.5 | 26.0 | 28.5 | 31.0 |
| | | 90 | 12.3 | 14.8 | 17.3 | 19.8 | 22.3 | 24.8 | 27.3 | 29.8 | 32.3 |
| | | 100 | 13.7 | 16.2 | 18.7 | 21.2 | 23.7 | 26.2 | 28.7 | 31.2 | 33.7 |

(continued)

**Table 5.5** (continued)

| Suspension | | Borehole | Pressure at top of borehole | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| W/C | $\gamma$ (g/cm$^3$) | Depth (m) | 0 (bar) | 2.5 (bar) | 5 (bar) | 7.5 (bar) | 10 (bar) | 12.5 (bar) | 15 (bar) | 17.5 (bar) | 20 (bar) |
| **1.0** | **1.52** | **10** | 1.5 | 4.0 | 6.5 | 9.0 | 11.5 | 14.0 | 16.5 | 19.0 | 21.5 |
| | | 20 | 3.0 | 5.2 | 8.0 | 10.5 | 13.0 | 15.5 | 18.0 | 20.5 | 23.0 |
| | | 30 | 4.6 | 6.6 | 9.6 | 12.1 | 14.6 | 17.1 | 19.6 | 22.1 | 24.6 |
| | | 40 | 6.1 | 8.6 | 11.1 | 13.6 | 16.1 | 18.6 | 21.1 | 23.6 | 26.1 |
| | | 50 | 7.6 | 10.1 | 12.6 | 15.1 | 17.6 | 20.1 | 22.6 | 25.1 | 27.6 |
| | | 60 | 9.1 | 11.6 | 14.1 | 16.6 | 19.1 | 21.6 | 24.1 | 26.6 | 29.1 |
| | | 70 | 10.6 | 13.1 | 15.6 | 18.1 | 20.6 | 23.1 | 25.6 | 28.1 | 30.6 |
| | | 80 | 12.2 | 14.7 | 17.2 | 19.7 | 22.2 | 24.7 | 27.2 | 29.7 | 32.2 |
| | | 90 | 13.7 | 16.2 | 18.7 | 21.2 | 23.7 | 26.2 | 28.7 | 31.2 | 33.7 |
| | | 100 | 15.2 | 17.7 | 20.2 | 22.7 | 25.2 | 27.7 | 30.2 | 32.7 | 35.2 |
| **0.7** | **1.67** | **10** | 1.7 | 4.2 | 6.7 | 9.2 | 11.7 | 14.2 | 16.7 | 19.2 | 21.7 |
| | | 20 | 3.3 | 5.2 | 8.3 | 10.8 | 13.3 | 15.8 | 18.3 | 20.8 | 23.3 |
| | | 30 | 5.0 | 6.6 | 10.0 | 12.5 | 15.0 | 17.5 | 20.0 | 22.5 | 25.0 |
| | | 40 | 6.7 | 9.2 | 11.7 | 14.2 | 16.7 | 19.2 | 21.7 | 24.2 | 26.7 |
| | | 50 | 8.4 | 10.9 | 13.4 | 15.9 | 18.4 | 20.9 | 23.4 | 25.9 | 28.4 |
| | | 60 | 10.0 | 12.5 | 15.0 | 17.5 | 20.0 | 22.5 | 25.0 | 27.5 | 30.0 |
| | | 70 | 11.7 | 14.2 | 16.7 | 19.2 | 21.7 | 24.2 | 26.7 | 29.2 | 31.7 |
| | | 80 | 13.4 | 15.9 | 18.4 | 20.9 | 23.4 | 25.9 | 28.4 | 30.9 | 33.4 |
| | | 90 | 15.0 | 17.5 | 20.0 | 22.5 | 25.0 | 27.5 | 30.0 | 32.5 | 35.0 |
| | | 100 | 16.7 | 19.2 | 21.7 | 24.2 | 26.7 | 29.2 | 31.7 | 34.2 | 36.7 |

The results demonstrate that effective pressures depend not only on the pressure at the top of the hole but also on the W/C-ratio and on the depth. Even without any pressure at the top ($P_M = 0$), a W/C-ratio of 2, i.e. $\gamma = 1.29$ g/cm$^2$, cause effective pressures at a depth of 10 m: $P_{eff} = 1.29$ bar and at 100 m: $P_{eff} = 12.9$ bar, respectively. For all sections of identical depths $P_{eff}$ increases proportionately with $P_M$ and $\gamma$.

The effective pressures listed in Table 5.5 lead to important conclusions:

- Narrow paths necessitating relatively high pressures ($P_{req}$) cannot be grouted if the borehole section contains discontinuities susceptible to fracturing at relatively low critical pressures ($P_{crit}$). Example: planes fracturing at $P_{crit} \leq 10$ bar make grouting of paths with $P_{req} \geq 10$ bar impossible: $P_{req} \geq 10$ bar $\geq P_{crit}$, even if WPT's yielded large LUGEON-values.
- If discontinuities susceptible to fracturing are available, the interplay between the factors W/C, $P_{req}$ and $P_{crit}$ restricts the achievable depth for grouting of narrow path—or: the achievable depth decreases the lower $P_{crit}$, the larger $P_M$ and the smaller W/C.
- The latent planes in approximately 80% of the rock types analyzed fracture at $P_{crit} \leq 10$ bar. The achievable depths of grouting narrow—or even middle-sized paths—related to $P_M$ and W/C are marked in Table 5.5. Further below only wider fissures can be grouted.

### *5.13.3 Hydrofracturing Restricts Grouting Pressure*

The results of this analysis conflicts with the conventional methodology recommending an increase of the grouting pressure with depth. The increasing grouting pressure soon reaches and exceeds the critical pressure. Below that depth the unfavorable condition $P_{grt} > P_{crit}$ is fulfilled producing the negative consequences of hydrofracturing demonstrated with the various examples presented above and exemplarily illustrated in Figs. 3.20–3.25. The results of such inadequate grouting measures prove that too high a grouting pressure defined in accordance with the conventional rules causes hydrofracturing. In order to avoid or to minimize those negative consequences the grouting pressure should be restricted whenever the rock possesses discontinuities susceptible to fracturing. Thus, the grouting pressure should not increase with depth if susceptible rock types are treated.

The meaning of the data listed in Table 5.5 requires rather the opposite: In order to compensate the effect of the higher density of the suspension on the grouting pressure, proportionately growing with depth, the pressure provided by the pump and recorded at the top of the hole should progressively decrease for the grouting work carried out in deeper zones to keep the favorable condition $P_{grt} < P_{crit}$ fulfilled.

### 5.13.4 Hydrofracturing Causes Economical Disadvantages

The cases presented before show exemplarily that the re-filling of fractured planes causes greater expenses, which impair considerably the economic efficiency of a grouting program. Two extreme cases shall be compared to demonstrate the economical consequences.

**In Case A** exist only a few grouthole sections with latent discontinuities prone to fracturing while the overwhelming majority encounters permeable rock which has to be grouted. Therefore, the expenses required for the transport to the site, the installation and maintenance of the equipment and the drilling work are not avoidable because the grouting work is required. One has to accept that portion of the total costs needed for the re-filling of the few planes pressured open. Nevertheless, one should try to minimize even the consequences of those few cases since they also cause additional costs as the example of the two lowermost grout sections of hole KB-E 37 in Fig. 3.20 shows: The grout take of 1040 kg/m result in a total amount of 7280 kg absorbed from the hole between 75 and 82 m. With an average grouting velocity of 6 kg/min a total of 20 h was required to grout these two sections which yielded extra costs of 2000 US$ simply for these two sections (unit price: 100/h).

**In Case B** the grouting program includes 50,000 current meters of groutholes. It is assumed that 70% encounter rock of a little permeability taking on the average 100 kg/m, while 30% intersect impervious rock containing latent discontinuities. They are cracked and opened because of too high a grouting pressure and subsequently filled taking 1000 kg/m. Considering a grouting velocity of 6 kg/min yields the following costs:

* For 70%: 35,000 m × 100 kg/m = 3500 t: 6 kg/min = 9722 grouting hours × 100 US$ = 972,200 US$
* For 30%: 15,000 m × 1000 kg/m = 15,000 t; 6 kg/min = 41,666 grouting hours × 100 US$ = 4,166,600 US$

Since the costs of all grouting hours amount to 5,138,800 US$, the portion of 81% of them arise for the filling of discontinuities pressured open. The working hours and the costs for the period needed to keep the equipment installed and maintained increase accordingly. All these expenses can be saved if fracturing is avoided. Figures 3.20–3.25 confirm that the quantities taken into account for Case B are realistic; grouting programs of that scale are not seldom, often they have even a larger size.

Whenever a grouting gallery is lacking, a longer grouting time due to fracturing leads to an extension of the grouting program. This is a momentous disadvantage because it postpones also the construction of the dam since the grout curtain has to be installed from the ground surface prior to the erection of the dam, which has to wait until the grouting work is finished. The completion of the dam is delayed and the project is put into operation months or even years later. The economical consequences can really be serious and may jeopardize the efficiency of the project.

Considering the many—even large—dam projects designed and completed without gallery it is prudent to pay attention to a further aspect of this complexity: Given a certain susceptibility to hydrofracturing its negative consequences for the technical and economical success of a grouting program are larger when the grouting work is executed from the ground surface. Grouting out of a gallery, i.e. against the load of the dam increases the overburden pressure, which reduces the intensity of hydrofracturing in spite of somewhat larger grouting pressures applied.

## 5.14 Conclusions

The results of this analysis are summarized as follows:

1. The various types of discontinuities have a different deformability. Hydrojacking occurs preferably in regular joints, hydrofracturing in latent discontinuities owning a pronounced fissility due to bedding, cleavage or laminar texture of planar crystals.
2. Hydrojacking enlarges the grout takes. As long as the consumption of cement and the distance of the travel can be kept in reasonable limits, grouting of dilated joints is advantageous because it leads to a better impermeabilization.

Within the surface-near zone of weathered rock the widening of open joints is caused by heaving of the overlying rock, i.e. the overburden pressure is the decisive factor. In that zone the pressures widening the joints increase with depth.

Further below, where the rock bond is still connected, the dilation of open joints depends on the deformability of the rock surrounding the borehole. Down there, the pressures beginning to widen the joints are scattering within an individual range. These ranges do not increase with depth.

Weathering ends at different depths: the weathered rock can reach down to a depth of several tens of meters or ends after a few meters already.

3. Hydrofracturing of latent discontinuities should be avoided—or minimized at least. Hydrofracturing causes very large grout takes and extends the grouting time, often enormously. With the exception of a few cases, the re-filling of originally latent planes does not reduce the permeability of the rock mass: the water seeps through the joints left open and around the groutstone pillows deposited along the formerly closed planes. The great efforts and expenses required for such measures are spent in vain.

The critical pressures cracking the latent planes are scattering within an individual range but do not increase with depth.

Hydrofracturing on purpose is expedient in case of parallelism between boreholes and open joints: access to them can be accomplished only by fracturing the latent planes.

4. Grouting pressure: As the critical pressure causing fracturing determined by strength does not increase with depth, the grouting pressure should not increase either whenever the rock possesses latent discontinuities as is the usual case.

The custom of increasing the grouting pressure proportionately with depth is not in harmony with the conditions of most rock types; all rules of thumb concerning the grouting pressure should not applied any more.

The appropriate grouting pressures have to be determined individually from case to case by means of WPT's and taking into account the critical pressure of susceptible planes as well as the density of the grout, the depth of the zone being treated and the counter-pressure due to groundwater. Test grouting is required to confirm that all relevant factors are adequately considered.

5. Staggered depths to be applied for groutholes of subsequent series are not expedient if the grouting pressure does not increase with depth. The holes of all series should reach to the same depth.
6. Pressures $\geq 10$ bar are required to penetrate and to grout those narrow paths yielding water takes in the order of 2 LU. The impermeabilization of these paths by means of cement suspension grouting is impossible if latent planes fracturing already at lowers pressures exist in the same borehole section. According to the result of this analysis that condition applies to the majority of all cases.

The unfavorable relationship between the high pressures required to grout narrow paths and the comparatively low critical pressures fracturing latent discontinuities identified for the majority of rock types has serious consequences: narrow paths cannot be grouted because before the high pressure is reached required to penetrate and to grout these paths, latent planes are fractured at a lower pressure.

Large water takes do not necessarily indicate groutable rock: high grouting pressures are also required if these water takes result from a multitude of narrow paths found in the same section; they cannot be grouted if latent planes fracturing at lower pressures are also available.

7. Restriction of depths: If latent discontinuities exist narrow paths can be grouted only to limited depths. The achievable depth varies: it decreases with growing grouting pressure imposed at the top of the borehole and increasing weight of the suspension.

**A Final Statement**

It is rather easy to press large quantities of grout into the rock; that needs simply a sufficiently high pressure. Unfortunately, just a large cement consumption is seen as a confirmation for both the necessity and the correct execution of a grouting program. However, a meticulous and skeptical analysis discloses that a large number of grouting programs were not carried out appropriately: on the way to the high grouting pressures designed latent planes are fractured before which subsequently absorb large grout takes. Such a process wastes lots of money and time. This analysis demonstrates that latent discontinuities fracture already at much lower pressures than realized so far. The due consideration of that fracturing behavior should help to avoid its harmful consequences.

# Chapter 6
# Groutability and Grouting of Rock

## 6.1 General Information

Once WPT's prove that the foundation is too permeable, it has to be analyzed whether a treatment is required. For this analysis, a hydraulic model should be used whenever justified. This analysis first must examine whether the foundation setting comprises favorable hydrogeological components, possibly making a treatment unnecessary. If they are lacking, the area to be treated and the appropriate technology must be defined. Questions which should be asked are:

- Is the rock groutable or is a diaphragm technologically and economically the better alternative?
- Is there a sufficiently impervious zone already in a shallow position to permit a connected curtain, or do we have to put up with a hanging curtain which is hydraulically much less favorable as shown in Fig. 6.1.

Joint fillings, usually not groutable but possibly erodible, are also important in this respect. A well-functioning grout curtain increases the hydraulic gradient. Erodible joint fillings may not resist such higher gradients, if they were washed away the permeability would increase again. Hence, grouting of fissures filled with erodible materials may not be expedient, other methods might be better.

- The grouting practice as reflected by papers and reports can be summarized as follows: All types of grouting at most sites are carried out rather schematically.
- Curtain grouting is done in subsequent series; the next series is chosen once the grout takes of the foregoing one are still considered to be too high. This often causes a very tight spacing of the groutholes, sometimes as small as 0.5 m.

Recommendations concerning the lower and the upper limit of grout takes warranting a grouting program have not been established. The prevailing concept seems to be: the higher the grout takes the better—unless the absorption is not extra-ordinarily high. Several authors suggest as a minimum 40–50 kg/m, others



F.-K. Ewert and U. Hungsberg, *Rock Grouting at Dam Sites*, Professional Practice in Earth Sciences, DOI 10.1007/978-3-319-64036-5_6

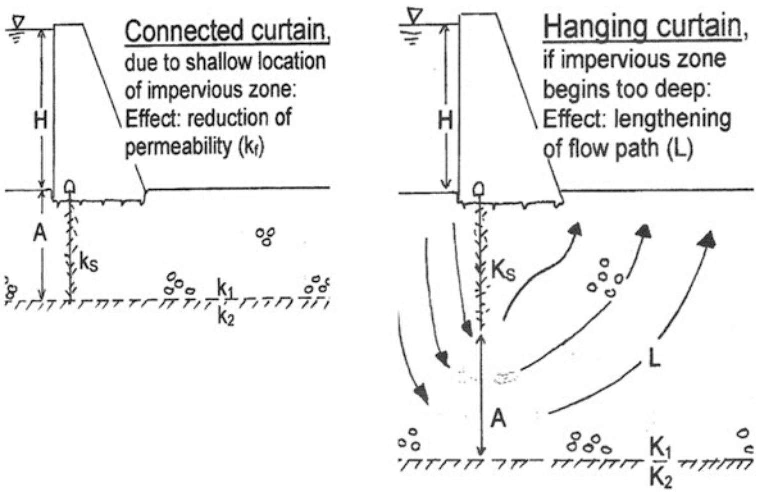



**Fig. 6.1** Connected curtain (*left*), hanging curtain (*right*) depending on depth of less permeable zone: $k_1 > k_2 \approx k_S$

continue even if the grout takes do not reach 10 kg/m. An upper limit has not been defined either.

Groutstone layers shown in control core drillings are considered confirming the need and the success of the program which is often not true. The grouting pressures increase with depth and different increase rates are being used. Water–cement-ratios are set differently whereas a tendency for thicker suspensions (1:3 down to 1:0.7, by weight) can be observed.

## 6.2 Hydraulic Characteristics of Water Paths

The course of the grouting process is controlled by the hydraulic characteristics of the water paths and the properties of the grout mix. Both together determine the grouting pressure required to penetrate the paths and to maintain the flow of the grout mix to reach a certain extension. There hardly exist two completely identical water paths within the same grouthole section treated, all intersecting paths are most likely of dissimilar width and shape. They have dissimilar hydraulic characteristics, which means that unequal pressures are required to initiate and to maintain the flow; hydraulic calculations as well as laboratory testing confirm this. The summary given in Table 5.2 shows a wide range of grouting pressures required for the same W/C-ratio: the 0.2 mm-path needs 12 bar, 1 mm-path only 2 bar; even finer paths need higher pressures, and wider ones can be grouted (almost) pressure-less. As different paths need different grouting pressures, unequal paths are not grouted simultaneously but one after the other at increasing pressure levels—the wide paths first, the finer ones afterwards everyone at their own pressure level. Such a pressure-dependent stage-wise grouting inherently throws up an important uncertainty:

- The finer paths can only be grouted if they remained accessible during the grouting of the wide openings. This is quite unlikely because they tend to be sealed superficially.
- The finer paths can only be grouted if the grouting pressure can reach the required level without provoking hydrofracturing of latent discontinuities already at a lower pressure level.
- The interaction between the different grouting pressures required for different paths and the different hydrofracturing behavior causes the individual groutability.

Rock types have their individual joint spacing; hence, the various paths have different lengths. A longer path needs a comparatively higher pressure to be grouted since it activates higher head losses due to friction—and vice versa. Figure 6.2 shows this qualitatively. The geometry of the paths also influences the extension of the flow: longer paths allow a farther travel of the grout provided the pressure is high enough to maintain the flow. This likewise permits a wider spacing of the groutholes—and vice versa.

## 6.3 Water/Cement-Ratio

In the past—and sometimes even today—we had different views about the appropriate W/C-ratio. Some favored thin suspensions (W/C > 5), others preferred thicker ones (W/C < 2). It was assumed that thinner suspensions penetrate finer paths more easily. This concept is now being changed. In fact, the penetrable width depends primarily on the size of the cement grains rather than on the water content of the slurry. This has been proved by petrological investigations by means of thin sections and microscope: rocks grouted with thin suspensions do not show grout-stone layers in finer joints than rocks grouted with thicker ones. Other important aspects advocating lower W/C-ratios are the grouting time and the strength of the groutstone:

- Using lower W/C-ratios needs more time to pump the amount of cement required to fill the groutable voids. Pumping thin suspensions makes the grouting work more expensive.
- More bleeding water must drain off to achieve a higher strength of the groutstone.

Stable cement suspensions are favored by many grouting experts (Kutzner). Recently Lombardi and Deere recommended stable cement suspensions with water/cement (W/C)-ratios of approximately 0.7 (by weight) with superplasticizer

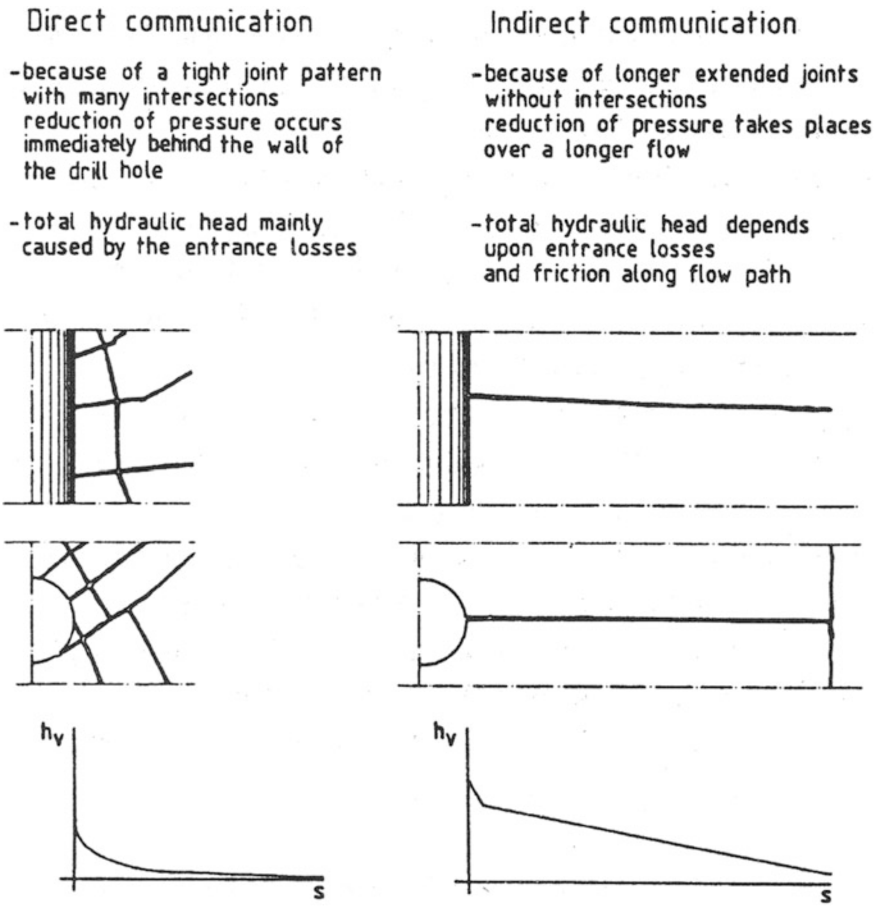



**Fig. 6.2** The relationship between the type of water routing around the borehole and grouting pressure (schematic illustration)

additives to reduce the cohesion and viscosity required to allow lower grouting pressures. They also propose the use of a single mix as this greatly simplifies the grouting procedure. This proposal is considered a progressive and helpful step.

We have been used to starting grouting with a somewhat higher W/C-ratio (2.0–1.0) and, after having injected a certain quantity, to change to a lower one (0.7). The reasons for this procedure are two-fold:

- The grout should reach a certain extension in finer paths which is easier to achieve with a suspension of lower cohesion as less pressure is required.
- To avoid the grout from traveling too far in wide openings—hence, too high a cement consumption—the reduction of the W/C-ratio enlarges the cohesion causing an earlier plugging of the paths.

In the light of new technologies and research results it becomes questionable whether this is really appropriate. Meanwhile it seems to be necessary to reconsider the interaction between the following factors:

- Grouting of stable mixes of low cohesion by using superplasticizer additives enhances the penetrability already at lower grouting pressures.
- Within one grouthole section dissimilar water paths of an unequal penetrability exist, each one necessitating its individual grouting pressure.
- Latent discontinuities are already hydrofractured at very low pressures.

This complexity shall be discussed separately because it needs more space and time.

The grouting cement normally used possesses a Blaine number of 3500 $cm^2/g$, sometimes only a cement of a lower Blaine number is available. The 3500-cement yields stable mixes; thus it is suitable for ordinary grouting procedures. For extraordinary requirements, today we use cements of a much higher Blaine number, sometimes exceeding 10,000 $cm^2/g$. This improves the penetrability but such a cement is much more expensive, and it must be assessed, whether the somewhat better success of the grouting pays for the higher investment.

## 6.4 Execution and Grouthole Pattern

The execution is very like water pressure testing: instead of water a cement suspension is pumped into boreholes, stage by stage, usually of 5 m in length. The grouting technology comprises, above all, the grouthole spacing, the water/cement-ratio and the grouting pressure.

In downstage grouting the holes are alternatively drilled and grouted, re-drilled and drilled further down and grouted etc. (Fig. 6.3). In upstage grouting the hole is drilled down to the final depth and then grouted from bottom to top. The latter approach is less expensive, and thus is often preferred. However, certain geological conditions call for downstage grouting as it serves to first seal and stabilize the upper zone. To reach the best result, the grouting technology has to be adapted to the individual groutability. The best results mean: minimum borehole meters (wide spacing of groutholes), cement consumption and grouting time versus a maximum impermeabilization.

Open fissures transport the grout slurry almost pressure-less over long distances producing large grout takes. Too far a travel requires too much grout and grouting time which is not appreciated, in contrast to this, in narrow paths the grout does not travel far enough while the grouting pressure increases rapidly, little grout is absorbed necessitating a close spacing of the groutholes.

The location of the paths to be sealed are unknown. It is uncertain whether and where the groutholes meet the water paths. To compensate for this, curtain grouting is carried out in subsequent series: The distance between the groutholes will be

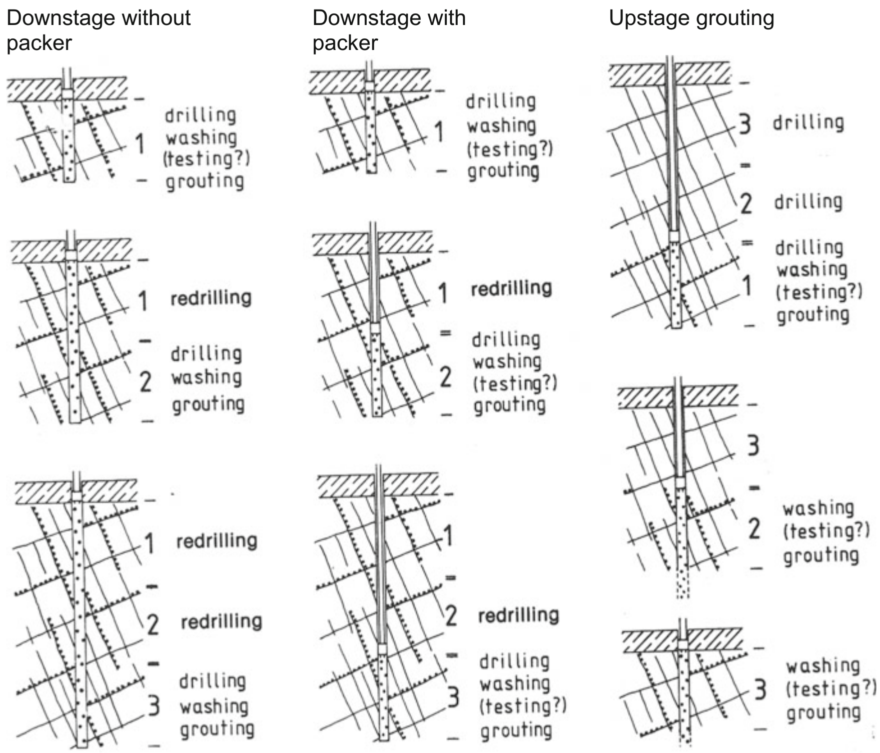



**Fig. 6.3** Illustration of downstage and upstage grouting methods

halved as long as a substantial quantity is still absorbed. Subsequent groutholes are called primary holes, secondary holes etc—or series A, B etc. The reason for this splitting is the need for an overlapping sealing to be achieved in spite of the unknown location of the openings. The reduced takes of subsequent groutholes indicate that the paths have been sealed successfully.

If grouting is not impaired by hydrofracturing, the order of the grout takes and the gradual decrease between the subsequent series depends on the geometry of the paths; the following quantities refer to the mean values (order of magnitude):

- Primary (P-) holes encountering wide and extended fissures score large grout takes (500 or even 1000 kg/m). The secondary (*S*-) holes meet the fissures already filled, their grout takes are much smaller. The gradual decrease from P- to S-holes is considerable (example P-holes—100%, S-holes—50%). The further series behave accordingly.
- By contrast, in the case of not-too-extended fissures P-holes produce moderate to small takes (200 kg/m) and the S-holes show only little reduction, perhaps 150 kg/m. If the fissures are much shorter than the distance between P- and S-holes (a few decimeters versus several meters) the S-holes meet still untreated

rock, hence, their grout takes show no reduction at all but can be even larger; the reduction of grout takes first begins with the tertiary (T-) holes or even later. A typical succession of mean values might be this: P—100 kg/m, S—110 kg/m, T—80 kg/m, Q—40 kg/m.

The old concept regarding the grouting pressure recommends increasing the pressure with depth. In that context, it is often used not to drill all the groutholes to the final depth but to use staggered stage depths for the groutholes of the subsequent series, exemplified in Fig. 5.28 already. The reason for this seems to be quite clear. As the higher pressure applied in deeper zones forces the grout to travel farther away, even a wider grouthole spacing achieved an overlapping sealing. The authors advocate a different concept for defining the grouting pressure which also implies consequences for the depth of the groutholes.

Hydrofracturing disturbs the grouting process heavily: The fracturing of latent discontinuities interrupts the increase of the grouting pressure. The fracture plane begins to absorb the grout, the pressure usually drops and grouting of the narrow paths is interrupted resulting in an incomplete sealing. The gradual decrease of the grout takes of the subsequent series is often interrupted too and the later series produce again larger grout takes—Fig. 3.8 presents an example. Such an occurrence is usually a clear indication that hydrofracturing is effective. These grout takes do not reflect natural fissures but artificial voids created by too high a pressure. As soon as this is noticed the current grouting process should be stopped to drill control holes with core recovery. It has been repeatedly experienced that the various sets of discontinuities are unequally prone to hydrofracturing. To increase the probability of encountering groutstone layers intercalated in fractured planes the holes should intersect this set as steeply as possible.

## 6.5 Evaluation of Data to Control the Grouting Process

All individual grout takes should be recorded and immediately evaluated statistically. The individual takes can be plotted in bars along the grouthole (Fig. 6.4) or listed in tables, separately for all the subsequent series (Table 6.1). Whenever the grout-hole stages were also water pressure tested, it is useful to compare grout takes and Lugeon-values because this yields valuable conclusions concerning the geometry of the water paths and the groutability.

The statistical evaluation allows to assess the course of the grouting work already before we drill core samples: Do we apply the appropriate grouting technology which produces a progressive sealing of the original voids or is the result impaired by inadequate grouting parameters? This evaluation should comprise both the mean values and the frequency distribution. The mean values allow a fast impression of whether the grout takes of the subsequent series are decreasing which testifies a progressive sealing, in particular when complemented by the frequency distribution (Fig. 6.5). Summation curves for the relative frequency distribution

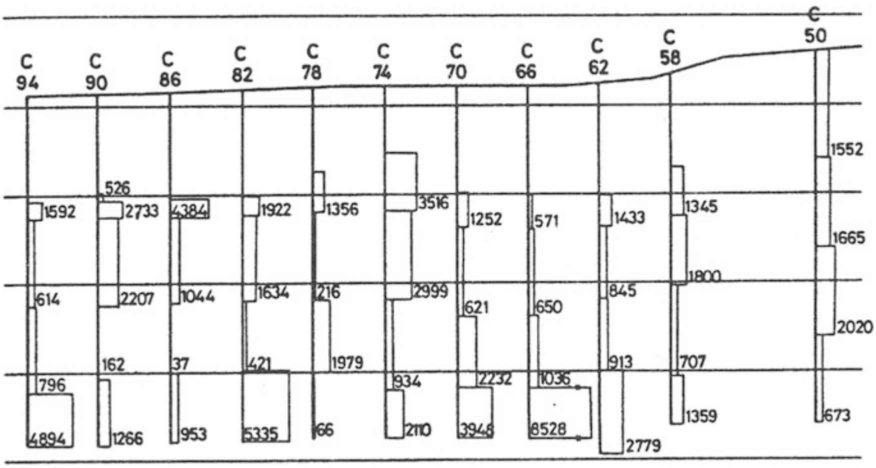



**Fig. 6.4** Individual grout takes plotted in *bars* along groutholes

show quite clearly whether subsequent series have smaller grout takes, Fig. 3.11 serves as an example. In contrast to this, approximately identical curves reflect a technological failure usually caused by hydrofracturing, as shown in Figs. 3.11 and 3.31, respectively.

The histograms presented before in Fig. 6.6 also give the relative frequency distributions although the reduction accomplished by grouting of successive series is expressed less clearly. The classification of grout quantities used in this example is based upon a '*Grout take classification*' proposed by Deere, in Weaver. Whether the first or the second form is used depends on the requirements of the project. A computer program has been developed to allow a fast statistical evaluation of grouting and testing data. We often must process large numbers of data, furthermore it is necessary to complete the evaluation mentioned above by adding the absolute frequency distribution.

The frequency distribution, i.e. the number of grouthole stages of all series should be known to make sure that the results reflect appropriately the permeability and the groutability of the given rock type inclusive the decrease of the grout takes. The examples displayed in Fig. 6.7a, b are typical for karstic limestone: within mostly tight rock the voids share a little volume only but need to be grouted—and quite often with enormous quantities of grout.

Analysis of selected segments of all quantities absorbed: Sometimes, the overwhelming majority of all grouthole stages absorbed small to moderate grout takes while a very few ones took extremely large quantities—say: 98% off all stages had <300 kg/m versus 2% had >2000 kg/m. It is then expedient to draw an additional graph only selecting the segment <500 kg/m because this allows a more exact assessment; Fig. 6.8 gives an example.

Relationship between individual takes and depth: The permeability decreases with depth or remains about the same—important for the function of the curtain:

**Table 6.1** Recorded individual grout takes can be listed in tables

| Grout takes in kg/5 m, bentonite and sand included | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Depth | −5 | −10 | −15 | −20 | −25 | −30 | −35 | −40 | −45 | −50 | −55 | −60 |
| Pressure | 2 | 3 | 5 | 6 | 7 | 8 | 9 | 10 | 11 | 12 | 13 | 15 |
| B-3 | 0 | 0 | 9 | 5 | 6 | 10 | 5 | 16 | 17 | 15 | 0 | 595 |
| B-9 | 45 | 13 | 9 | 6 | 7 | 25 | 0 | 5 | 5 | 7 | 5 | 35 |
| B-15 | 0 | 15 | 10 | 18 | 0 | 0 | 7 | 0 | 0 | 13 | 11 | 20 |
| B-21 | 6 | 7 | 14 | 9 | 10 | 305 | 305 | 0 | 9 | 0 | 7 | 24 |
| B-27 | 9 | 13 | 0 | 6 | 9 | 13 | 0 | 0 | 9 | 19 | 0 | 19 |
| B-33 | 9 | 33 | 16 | 21 | 0 | 0 | 12 | 0 | 10 | 0 | 102 | 17 |
| B-39 | 0 | 11 | 13 | 6 | 6 | 7 | 0 | 7 | 26 | 16 | 0 | 41 |
| B-45 | 26 | 11 | 7 | 12 | 7 | 9 | 7 | 7 | 22 | 32 | 7 | 8 |
| B-51 | 24 | 14 | 16 | 0 | 0 | 0 | 32 | 0 | 6 | 200 | 0 | 18 |
| B-57 | 9 | 17 | 613 | 15 | 380 | 0 | 380 | 23 | 58 | 58 | 69 | 45 |
| B-63 | 0 | 0 | 0 | 0 | 0 | 35 | 0 | 0 | 39 | 77 | 106 | 12 |
| B-69 | 0 | 0 | 0 | 0 | 0 | 0 | 0 | 52 | 22 | 188 | 9 | 29 |
| B-75 | 38 | 24 | 65 | 0 | 34 | 2083 | 85 | 12 | 6 | 0 | 12 | 611 |
| B-81 | 44 | 0 | 0 | 854 | 9 | 11 | 26 | 7 | 0 | 15 | 212 | 212 |
| B-87 | 66 | 892 | 892 | 0 | 0 | 30 | 0 | 7 | 10 | 10 | 0 | 12 |
| B-93 | 19 | 19 | 19 | 19 | 0 | 0 | 0 | 0 | 22,363 | 24 | 107 | 221 |

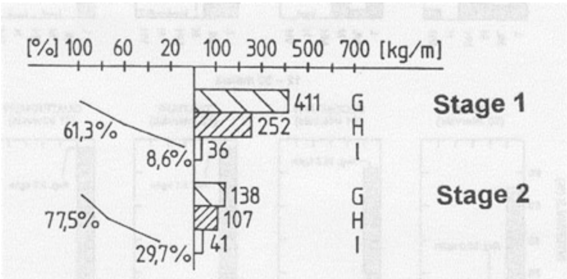



**Fig. 6.5** Decreasing mean values indicate progressive sealing

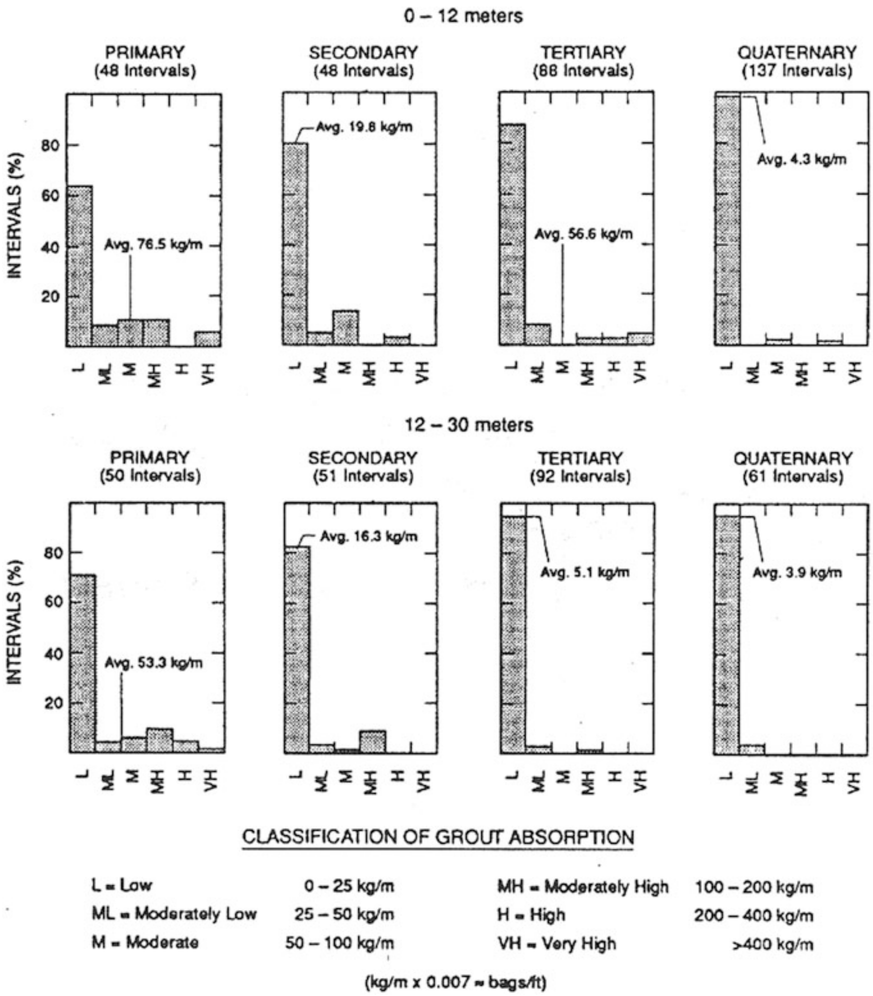



**Fig. 6.6** Frequency distribution of grout takes based upon grout take classification proposed by Deere

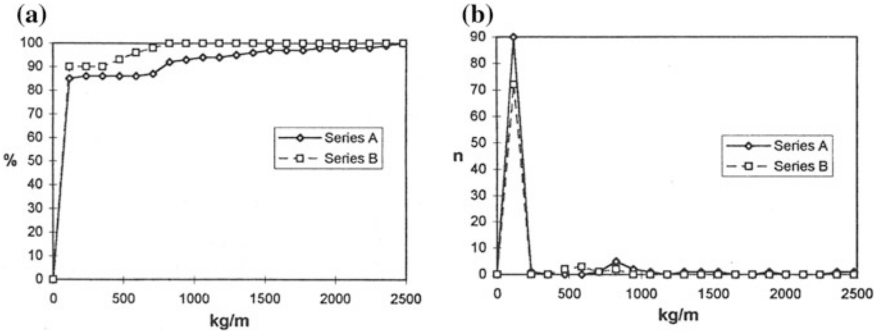



**Fig. 6.7** **a** Statistical evaluation of grout data obtained in Karstic limestone; the relative frequency distribution of all grout takes. **b** Absolute frequency distribution of all grout takes

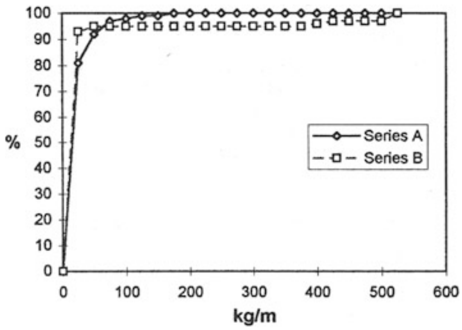



**Fig. 6.8** Statistical evaluation of grout data obtained in karstic limestone

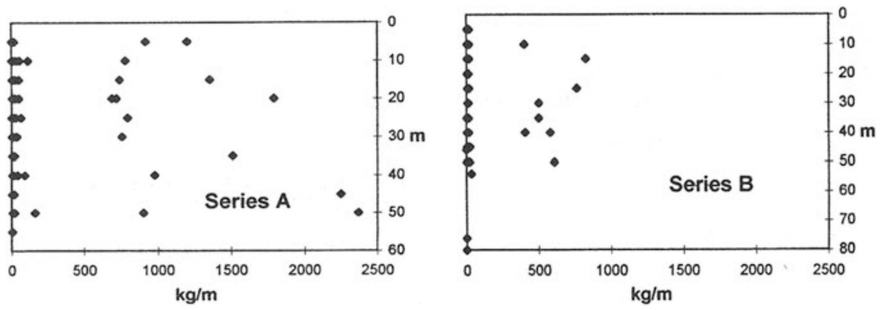



**Fig. 6.9** Distribution of individual grout takes related to depth; *A* = primary *B* = secondary

can it reach an impervious zone yielding the much more effective *'connected curtain'*, or it ends far above resulting in a less effective *'hanging curtain'*. Thus, it is very helpful to examine this relationship presented in Fig. 6.9.

The same data have been used to draw the respective graphs for the relative and for the absolute frequency distribution of all data (Fig. 6.7a, b), for the segment of grout takes <500 kg/m (Fig. 6.8) and for their distribution related to depth (Fig. 6.9a, b).

## 6.6 Relationship Between WPT-Values and Grout Takes

It has been frequently pointed out that the WPT-values can hardly be correlated with the grout takes. An attempt to explain the discrepancy was lacking. Nevertheless, today this discrepancy is explainable—and this discussion yields important conclusions. We should remember that moderate or large WPT-values can originate either from only one or just a few wide fissures or from a multitude of very narrow paths—haircracks, for instance, inaccessible to cement suspensions. Furthermore, we should consider the negative consequences of hydrofracturing occurring at pressures higher than 10 bar—the reference pressure of the Lugeon-criterion: A tight rock absorbs no water at 10 bar but if it were fractured at 15 bar it would take a lot of grout. Thus, considering both factors enables to explain the discrepancies.

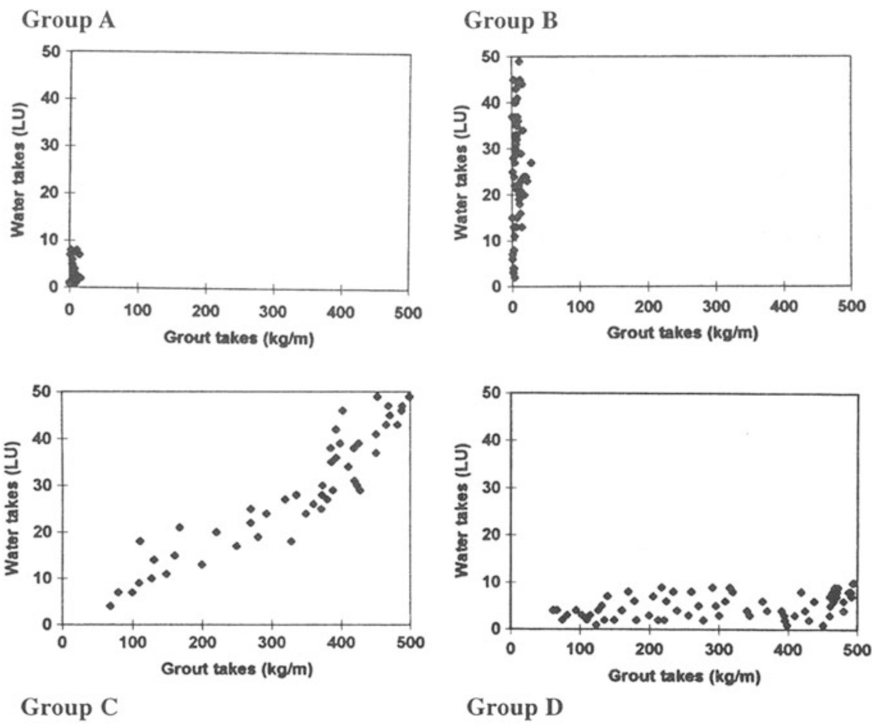



**Fig. 6.10** Grouping of different relationships between WPT-results and grout takes

In principle, four combinations occurring in practical grouting programs have to be distinguished (Fig. 6.10):

- **Group A**: Little WPT-values and little grout takes (order of magnitude: <5 LU, <40 kg/m); it is evident that the rock is impervious or little permeable only, it is inaccessible to both water and grout. Grouting was not required nor would it be possible using an appropriate grouting pressure.
- **Group B**: Large WPT-values and little grout takes (order of magnitude: <20 LU, <40 kg/m); the considerable water absorption is caused by a multitude of narrow paths. Cases of Group B are not groutable using a conventional cement and even micro-cement does not yield a much better result. If the hydrogeological regime requires sealing measures, other means would have to be applied. Since the permeability originates from very narrow paths, the permeability of the entire rock-mass is usually not very high and a treatment is most likely not required.
- **Group C**: Large WPT-values and moderate to very large grout takes (order of magnitude: >30 LU, 50 kg/m up to even more than 1000 kg/m); medium to wide openings allows the absorption of substantial or even very large quantities of grout. Grouting is required, possibly and warranted.
- **Group D**: Little WPT-values and large grout takes (order of magnitude: <5 LU, >100 kg/m up to even more than 1000 kg/m); cases belonging to Group D represent a sufficiently impervious rock, the high grout takes originate from hydrofracturing. The filling of dilated paths or opened latent discontinuities hardly produce a further sealing while it causes considerable expense.

These groups frequently observed in practice should be understood as average situations, transitional cases occur.

## 6.7 Hydrofracturing Caused and Indicated by Grouting

Enormous grout takes of several tons per meter were recently reported from a grouting program carried out in shale. This was surprising because shale is mostly impervious. The discrepancy between expectation and reality was allegedly caused by caves. As a very high grouting pressure was applied, it seemed much more likely that those huge grout takes resulted from an intense hydrofracturing. The exemplary grouting programs, as discussed previously in this paper, showed that hydrofracturing is quite frequent; many cases are not spectacular but quite a few yielded bad results.

Like in WPT's, too high a pressure causes the dilation of fissures and the splitting of latent discontinuities also in grouting. It occurs either due to the lifting of the overlying rock or due to the deformation of the insufficiently hard rock surrounding the borehole. The decisive role of the rock bond has been discussed already. The fracturing behavior is essential for the definition of an appropriate grouting pressure. In displacement grouting, fracturing is helpful as it results in a better prestressing.

In penetration grouting it is undesired because the backfilling of cracked discontinuities needs more grouting time and cement. This can be very expensive, particularly in grouting programs carried out in rock types of low strength. Moreover, a repeated intercalation of groutstone layers can lead to a harmful heaving (Example in Fig. 3.8). Bedding or similar planes are particularly prone to hydrofracturing. The filling of dilated fissures requires also more grout, however, this is rather positive because it improves the sealing effect due to a better adhesion while the increase of the cement consumption is comparatively meaningless.

Dilation or splitting of discontinuities can be analyzed by means of WPT's. Groutstone layers deposited in opened discontinuities indicate hydrofracturing too. The example shown in Fig. 3.10 demonstrates this convincingly. Closed bedding planes were pressed open and subsequently filled with grout at the very low pressure of 2–3 bar already. WPT's revealed the low critical pressures very clearly (Fig. 3.7). The statistical analysis gives evidence already whether hydrofracturing probably occurred. Later, drill cores disclosed the form of an intercalated groutstone layer and the state of the contact between groutstone and the fractured discontinuity.

The critical pressures listed in Table 5.1, obtained in many programs carried out in many different rock types, lead to the following conclusions:

- WPT's are a reliable tool to find out the critical pressure which initiates hydrofracturing.
- The form of an intercalated groutstone layer together with the unweathered state of the adjoining surface of the fractured plane confirm that this results from the refilling.

The analysis disclosed that many grouting programs allegedly successfully executed were neither necessary nor helpful but expensive. Originally, the intercalation of groutstone layers was supposedly proving the success. However, detailed analysis discovered that they indicated hydrofracturing instead of a sealing of true paths. Since the grouting process was controlled by fracturing it was not successful at all.

It has been observed that narrow fissures remained ungrouted while latent discontinuities have been opened and subsequently grouted. This occurs because:

- The various types of discontinuities do not have a similar susceptibility to fracturing, bedding planes are particularly sensitive. Moreover, the strength is not the same in all directions and while all existing planes necessarily have different orientations, only one set is intersected at approximately right angles which facilitates fracturing.
- Wide fissures need a low grouting pressure; finer ones require high pressures. If latent discontinuities are fractured at comparatively low pressures they will be grouted first and the narrow paths mostly remain ungrouted—either their entrances are already sealed or the required pressure levels are not reached in the meantime.

The critical pressures in both WPT and grouting can be different, even in the same rock. It depends on the W/C-ratio and the state of the discontinuities whether hydrofracturing occurs easier in WPT or in grouting. Lombardi & Deere, advocating for low W/C-ratios, argued that heavier suspensions imply *'less risk of hydrofracturing'*. Actually, the situation is more complex and will be discussed later on.

## 6.8 Orientation of Groutstone Layers Intercalated Due to Hydrofracturing

Rock types have their individual fabric of discontinuities often comprising not only joints but also bedding or cleavage planes. The discontinuities, even in the same rock, differ in all relevant respects:

- Accessibility—they are still latent or partially or completely open.
- Susceptibility to dilation—controlled by the compressibility of the rock and by the angle of intersection with the groutholes.
- Susceptibility to hydrofracturing—also controlled by the compressibility of the rock, by the angle of the intersection with the groutholes and by the tensile strength across the latent planes.

As the effect of dilation is comparatively meaningless, further discussion focuses on hydrofracturing. All rock types have their own susceptibility to hydrofracturing and it depends on the grouting pressure and on the angle of intersection whether this becomes effective; an intersection at right angles eases hydrofracturing. Provided, the conditions for hydrofracturing are fulfilled, they do not apply equally for all the available discontinuities but first for the most suitable set. Thus, such groutstone layers tend to be intercalated along the approximately parallel planes of the preferred set; the groutstone '*pillows*' have a nearly parallel orientation, Fig. 3.32 presents one of many examples.

The distance between two parallel pillows depends on the length of the grouthole stage treated. Within the same stage, hydrofracturing uses the weakest plane first, hence it occurs preferably once or—if ever—only a very few times producing just one or very few groutstone layers along the 5 m-long grouthole stage. The pillows are deposited a few meters away from each other. They are largely isolated and an overlapping curtain is quite unlikely to be achieved.

## 6.9 Individual Groutability

The decision in favor of a grouting program is often done schematically on the basis of WPT-results. Neither favorable components of the hydrogeological setting nor the possible differences between permeability and groutability are duly considered.

When WPT's apparently prove a rock permeable, it is supposed to be groutable. This is not necessarily true. Within the last 40 years it became evident that rocks have their individual groutability. The best result of a grouting program—greatest sealing at least costs—can only be achieved when this groutability is considered, particularly, when weak rock is involved. Sometimes even permeable rock types possess a poor groutability. In this case, intense grouting possibly makes no sense.

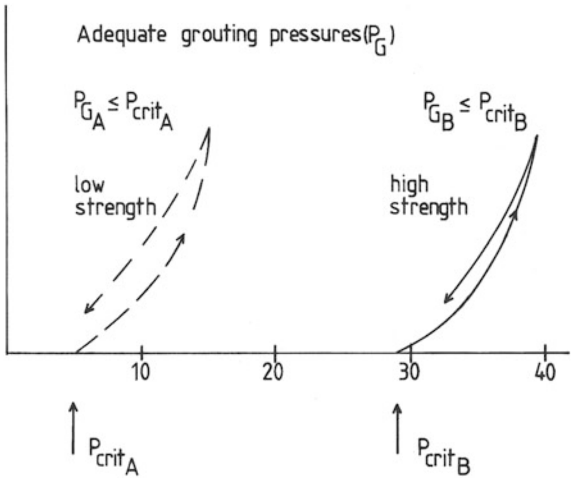
The individual groutability has two reasons:

- Besides the rheological properties of the grout mix, the size, shape and roughness of the paths to be grouted determine the grouting pressure required to penetrate the paths and to reach a reasonable extension of the grout travel.
- Rock bond and strength determine the hydrofracturing behavior of the rock.

A high pressure is needed to grout narrow paths—and vice versa. In wide fissures the grout mix flows already (almost) without any pressure. The meaning of this complexity becomes evident when the required grouting pressures are related to the critical pressures which cause hydrofracturing as the following opposite cases presented in Fig. 6.11 exemplify:

- Supposed a hard rock required a pressure of 30 bar to be fractured, narrow paths can be grouted because the grouting pressure required for penetration exemplify—say: 20 bar—is attainable without fracturing.
- By contrast, in rock types of low tensile strength latent discontinuities are fractured already at pressures between 3 or 5 bar. Provided the grouthole intersects both wide and thin fissures, grouting usually begins at low pressures say—2 bar—just enough to penetrate the wide fissures. The narrow paths remain ungrouted because they need higher grouting pressures which are not attainable: These pressure levels can never be reached because the susceptible

**Fig. 6.11** Relationship between required grouting pressure and critical pressure causing hydrofracturing of latent discontinuities



discontinuities cracked before, causing a pressure drop. This mechanism was experienced very clearly in the grouting work carried out at the Twiste dam.

The different flow characteristics of all the various paths and the different strength properties of all rock types lead to the conclusion that rocks have their individual groutability. This also means that the original permeability can often not be reduced to the desired degree. In soft rocks, only the wide openings can be grouted; in hard rocks also narrow paths are groutable. A poor groutability leaves a considerable residual permeability which normally cannot be reduced any more by further cement suspension grouting. The individual groutability should influence the decision on a grouting program. Provided a sufficiently high grouting pressure is applied it is always possible to press grout into the rock. However, a grouting program aimed at sealing natural voids is only warranted if actual water paths are grouted instead of hydrofracturing the rock—except the rare condition that the set of discontinuities susceptible to fracturing has the orientation of the grout curtain, an exceptional case, of course.

Medium-ranged WPT-values do not prove that the permeability of a given rock exceeds a critical order. WPT-results as such do not disclose the groutability, Fig. 6.12 explains this by comparing examples. It is assumed that both rock types

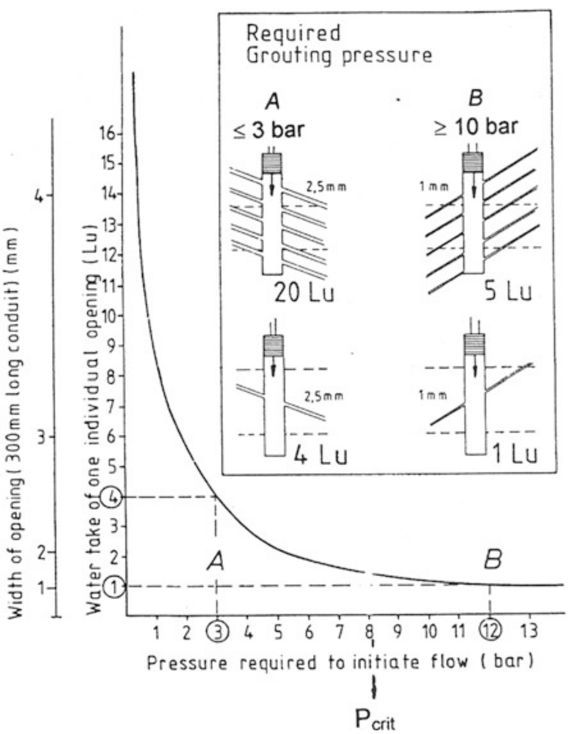



**Fig. 6.12** Ambiguity of WPT results with respect to groutability and grouting pressure

have latent bedding planes, susceptible to hydrofracturing at similar critical pressures around 8 bar, while their hydrogeological setting is different.

**Case A** The rock has one wide fissure per stage, yielding a WPT-value of $\sim$4 LU; five fissures produce $\sim$20 LU. The initial grouting pressure ranges at 3 bar. This fissure can be grouted before the critical pressure is reached.

**Case B** The rock has one very narrow path—say 1 mm in width producing $\approx$1 LU; five of these paths lead to $\approx$5 LU. A grouting pressure of 12 bar is required to start the penetration. These paths cannot be grouted because the latent bedding planes are already fractured before this pressure is reached.

Low WPT-values allow a rather reliable assessment because they indicate narrow paths which need high grouting pressures. In contrast to this, large WPT values do not disclose the size of the paths, the groutability and grouting pressure required cannot be quantified either. Therefore, it is required to assess the specific groutability for given cases by means of a test grouting program.

## 6.10 Grouting Pressure

The pressure is certainly a decisive factor to accomplish the desired result of a grouting program:

- In penetration grouting aimed at filling natural voids - as in curtain grouting, for instance—hydrofracturing in principle is not required in soft rock because it causes too great a cement consumption. Hence, it should be avoided unless a controlled fracturing is required to provide for the access between groutholes and open fissures.
- In displacement grouting fracturing is intended in order to achieve the prestressing of the rock mass required to improve strength.

Both the purpose of the grouting program and the rock properties should be duly considered because many examples of penetration grouting demonstrate that hydrofracturing badly impaired the grouting success or even damaged the rock.

### *6.10.1 High Pressure Grouting*

Several authors are not afraid of the negative consequences of high pressure grouting often occurring but recommend fracturing to improve the groutability. As an example of such a position, it is referred to Nonveiller "...*Whether or not hydraulic fracturing is desirable for grouting is still a controversial issue in the profession. Hydraulic fracturing caused by high injection pressures increases the* *groutability of the formation and the reach of the injected grout, because new interconnections in the net of fissures are created. Thus the efficiency of grouting is increased. The permeability of the formation is reduced to a greater distance from the injection hole, the spacing of holes can be increased and an economy is achieved by reducing drilling costs. This is an advantage over the alternative to achieve the same reduced average permeability with a lower injection pressure and a closer hole spacing...* ".

Of course, this concept is the right one for replacement grouting, particularly in hard rock. For penetration grouting this concept is not appropriate, particularly for soft rocks and for rock types of little natural permeability:

- Substantial amounts of grout are required to refill the '*artificial*' voids caused by hydrofracturing. More grout is often needed to fill them rather than to seal the natural water paths. An analysis of many grouting programs showed that the negative effect of fracturing on the technical and economic success of rock grouting is far more frequent than often presumed.
- Due to the unequal susceptibility of the various types of discontinuities, the most susceptible type—particularly bedding planes—tend to be fractured first while narrow paths remain ungrouted, leaving a considerable residual permeability.
- If the planes of the same set rule this hydrofracturing, it is impossible to achieve an overlapping sealing: Grouting produces parallel groutstone layers located quite far from each other.
- If a grouting program is determined by hydrofracturing the grout takes of successive series do not decrease—or not enough. As the reason for this is undiscovered additional groutholes of a further series are set in between eventually resulting in a very close spacing of the groutholes. In soft rocks, comparatively high grouting pressures cause a deformation of the rock around a borehole being grouted. The pressure and, hence, the deformation usually reach the maximum at the end of the grouting process. If neighboring holes are getting too close to each other—say less than 0.7 m or so—the deformation may affect the adjacent rock sealed previously. It may loosen the contact between groutstone and rock along a grouted fissure thereby inducing a new permeability. The phenomenon of a new permeability created by too high a grouting pressure and too close a grout hole pattern has been repeatedly experienced quite impressively. It must be inferred that each rock type can only be sealed to an individual optimum—overdoing can deteriorate the state of the rock again.

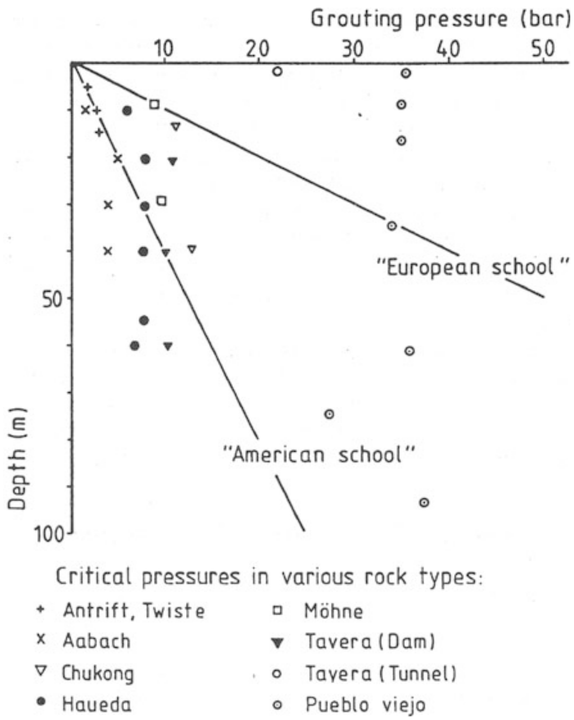
It is recommended to check very carefully whether hydrofracturing is helpful or should be avoided. In most cases of penetration grouting, is certainly not advantageous.

### 6.10.2 Concept of the Depth-Dependent Grouting Pressure

Many authors recently followed the old concept of increasing the grouting pressure with depth. This still widely applied concept is based on a compromise between two opposite demands:

- The grouting pressure should be as high as possible because high pressures yield a wider spreading of the grout slurry which saves drilling meters as it allows a wider spacing of the groutholes.
- High pressures can initiate hydrofracturing of latent discontinuities which results in an uneconomic cement consumption and carries the risk of affecting the rock while an overlapping grout curtain is usually not achieved. Different rates are used to increase the grouting pressure with depth. According to Fig. 6.13 the '*European school*' advocates for a higher rate (1 bar/m), the *'American school'* prefers a lower one (0.25 bar/m). The compromise assumes that the hydrofracturing behavior of a rock mass is controlled by the overburden pressure. In order, not to exceed the critical pressure the grouting pressure is intentionally kept below that level, thus, it increases with depth to avoid hydrofracturing—or to minimize its consequences, at least.

**Fig. 6.13** Different increase rates of grouting pressures



The appropriate rate is often controversially discussed. This, however, is not at all the most important issue. In fact, this is the mechanical reason of hydrofracturing. The overburden pressure is only effective within the weathered rock near to the surface, further below the hydrofracturing is determined by the strength of the rock around the borehole. As this does not increase with depth, the critical pressure remains about the same irrespective of depth. This condition prevails in un-weathered rock with a connected rock bond; Table 5.1 summarizes the critical pressures observed in numerous frequent rock masses. Conclusively, the concept of a depth-dependent increase of the grouting pressure is not appropriate for most cases.

Depth-dependent grouting pressures do not avoid hydrofracturing but promote it: The exemplary projects discussed already demonstrate that grouting pressures based on that concept frequently exceed the critical pressures. The author suggests using a concept considering both the purpose of the grouting program and the strength of the rock masses.

### *6.10.3 Influence of Geological Conditions on the Initial Grouting Pressure*

The borehole stage to be treated may be completely tight or the rock may have wide or small fissures, respectively. This influences the effective grouting pressure ($P_{eff}$) decisively:

In case of a tight stage (Fig. 6.14a) the effective pressure of the grouting process amounts to

$$P_{eff} = P_M + P_H$$

with $P_M$—pressure recorded at the top of the grouthole (surface),
and $P_H$—additional pressure due to depth of section grouted.

$P_{eff}$ increases with the density of the grout, thus, considerable pressures become effective in deep sections if low W/C-ratios are used. The effective pressures in grouting are much higher than in water pressure testing. Provided planes susceptible to hydrofracturing intersect the borehole this condition causes hydrofracturing despite a grouting pressure at the surface ($P_M$) intentionally kept below the critical pressure. (Example: $P_M$ = 10 bar, a 100-m-deep section results in $P_{eff}$ = 20 bar in WPT versus $P_{eff}$ = 25 bar in grouting (with $\gamma_{Grout}$ = 1.5).

In case of a permeable rock head losses due to friction ($P_F$) become effective yielding to

$$P_{eff} = P_M + P_H - P_F$$

Wide fissures activate little head losses, usually smaller than $P_H$, small fissures cause large ones, often exceeding $P_H$. Thus, the effective pressure can be quite

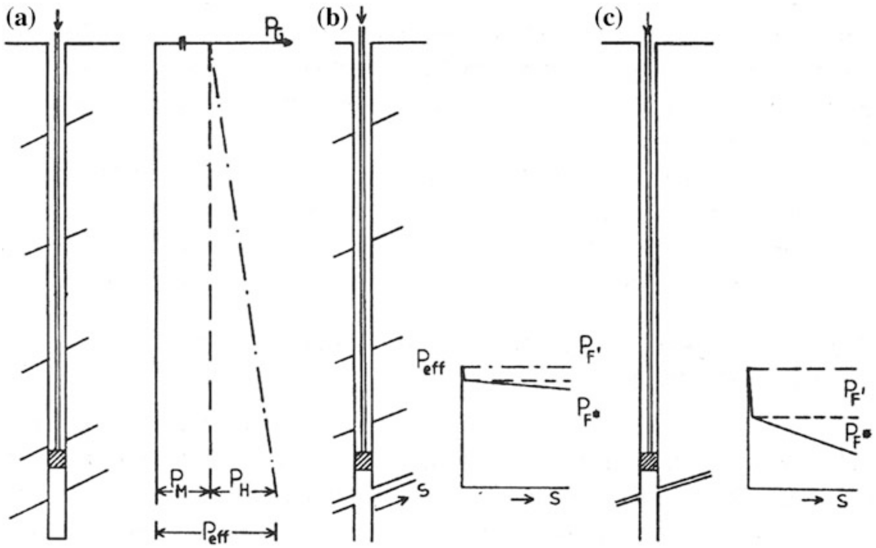



**Fig. 6.14** Relationship between state of discontinuities and grouting pressures: **a** Joints intersecting the groutholes stage are closed. **b** Wide fissures intersect the grouthole stage. **c** Narrow fissures intersect the grouthole stage. $P_M$—Pressure at the top of the hole. $P_H$—Static head caused by suspension. $P_{eff}$—Effective grouting pressure. $P_F$—Head losses due to friction at the entrance of the fissure (s). $P_F$*—Head losses due to friction at the entrance of the fissure(s)

different. Whether the greater static head provided by the higher density of the suspension becomes fully effective or whether it is compensated by the higher head losses occurring in the flow of the grout, depends on the state of the fissures (Fig. 6.14b, c).

This latter one must be considered in defining the maximum grouting pressure using P/Q-diagrams. The actual condition cannot be determined, of course. Nevertheless, by considering the state of jointing and the results of both WPT's and test grouting, it is possible to achieve an assessment of reasonable reliability.

### 6.10.4 *Influence of Geological Conditions on the Course of the Pressure During Grouting*

Penetrating the fissures causes head losses due to friction at the entrance. Further head losses occur along the fissure depending on both the width and length between the entrance and the spot where the fissure branches off: narrow and long paths activate large head losses—and vice versa (see Fig. 6.2). These components—besides those caused by the properties of the grout—determine the grouting pressure required to begin and to maintain the grouting process.

During grouting, the pressures increase in various forms, either one characterizing its own course indicated by typical grouting paths curves depicted in Fig. 6.15 (reduced version of Fig. 4.19)

- Wide and practically endless fissures—karstic channels, for instance—absorb nearly pressureless enormous grout quantities; special measures may be needed to plug the openings which often fail (grouting paths a + b).
- For open fissures (5 mm in width and several meters in length) only low pressures are
- required to start the penetration and keep the grout flowing (grouting path c)
- Hydrofracturing of latent discontinuities in not too hard a rock produces a peculiar course, recognizable by automatic recording. Soon after fracturing at the individual critical pressure level the pressure decreases steadily while the grouting process continues. Unless grouting is not intentionally stopped, any quantity is absorbed (grouting path d). This produces intercalated groutstone cushions, sometimes up to many decimeters in thickness and many tens of meters in length. A distinct pressure drop has been repeatedly observed; core samples confirmed this mechanism which has the following reasons:

  (1) The steadily intruding grout forms a cushion heaving the overlying rock—or compressing the rock. The growing cushion works as a hydraulic jack covering an increasing area of the opened plane which causes the pressure to decrease accordingly.
  (2) The intruding cushion extends the fracturing ahead of the wedge. As the rock has a certain strength against bending, splitting precedes the front of the grout causing there suppression.

- Short and narrow fissures cause a fast increase of the grouting pressure (grouting path e). In not too weak a rock, latent discontinuities need higher pressures to be cracked followed by a moderate pressure drop, the new pressure build-up occurs slowly (grouting path f).
- Hydrofracturing of latent discontinuities in hard rock occurs mostly without pressure drop. The pressure built-up usually continues, the grout takes are correspondingly low (grouting paths g–i).
- Fissures of smaller width and extension need a higher initial pressure; the first local sedimentation produces an increase of the pressure, possibly able to split the first plug away causing a little pressure drop; after a little time of further grouting a new plugging causes another increase of the grouting pressure—maybe again followed by the next pressure drop; drop and built-up of grouting pressure proceed until the maximum grouting pressure is reached. Figure 6.15 shows no respective grouting path).

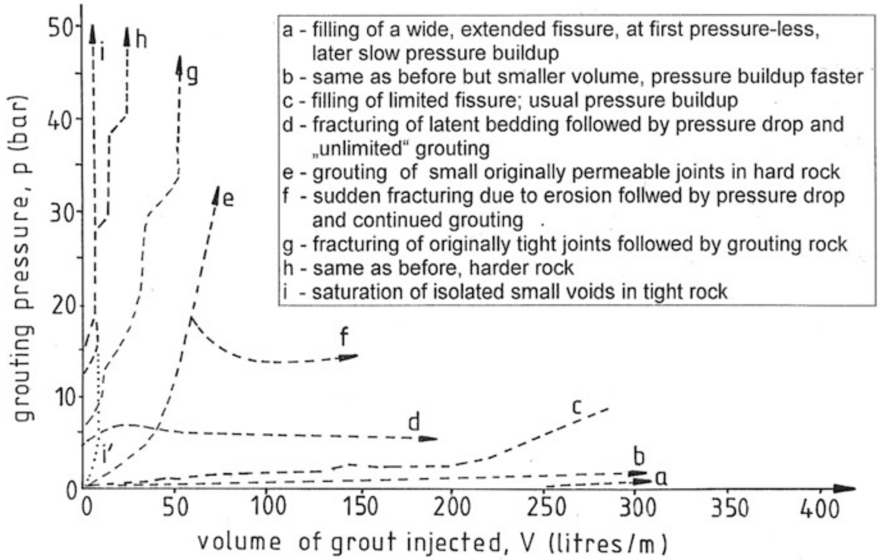



**Fig. 6.15** (Reduced version of Fig. 4.19) Grouting path curves indicating different courses of grouting caused by different settings of discontinuities

## 6.10.5 *Influence of Rheological Properties of Suspension on Grouting Pressure*

Provided a fissure is not too small for the cement grains suspended in a cement suspension, a Bingham fluid, needs a certain initial pressure to surmount the shear resistance. According to Fig. 6.16 the shear stress required depends on the shear gradient (D) and the viscosity (ŋ):

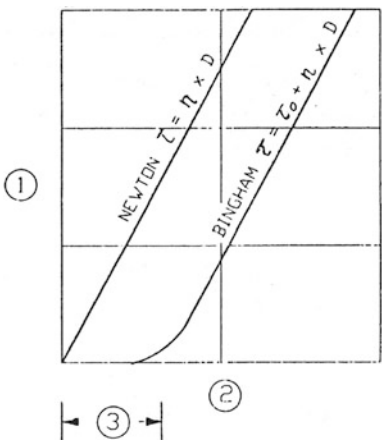



**Fig. 6.16** Typical flow curves, after KUTZNER *1* Shear gradient D (l/s) *2* Shear stress τ *3* Flow limit $\tau_0$

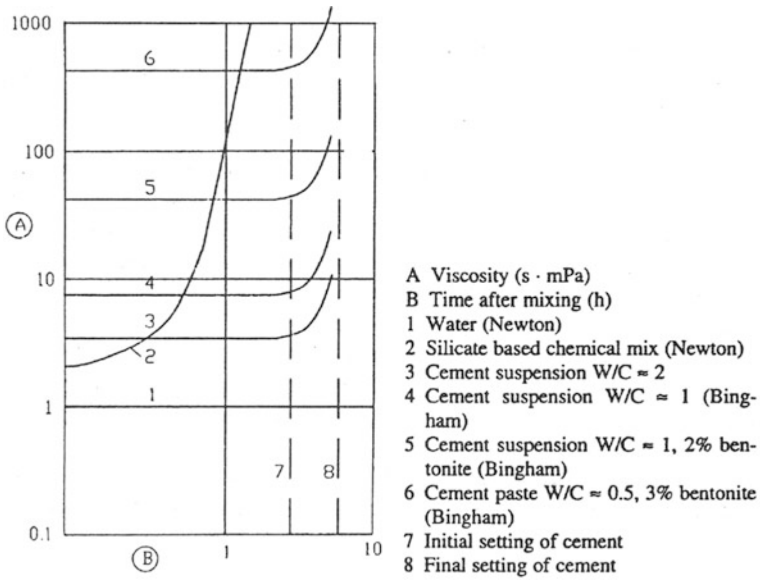



**Fig. 6.17** Viscosity development of grouting materials, after Kutzner

$$\tau = \tau_0 + \eta \times D_2$$

with the flow limit 0. The flow limit decreases with the W/C-ratio (Fig. 6.17), thus, higher pressures are needed for thicker suspensions. Additives can be used to lower the viscosity which reduces the grouting pressure. The hardening of the grout slurry means that the viscosity intentionally increases with time. This is principally desired although it limits the time for the grouting process.

The viscosity and the flow limit are essential properties for practical purposes because they influence:

- the mixing process (time required for mixing),
- the pumpability (pressure required to pump the mix through the pipeline) and
- the penetrability (extension of the grout in pores or fissures), which means the suitability of the various grouting materials. The viscosity and the flow limit are determined by using different devices as described by Kutzner.

### 6.10.6 Grouting Pressure Appropriate for Various Geological Conditions

Different grouting pressures should be used to meet the individual geological settings. Of course, all the grouthole stages treated in the same program will comprise different settings and it is impossible to adapt the technology individually to each stage. For practical reasons, it is necessary to use a generalized formula which has to satisfy those stages of the whole set which own the most sensitive conditions regarding safety. Overdoing in some stages can easily be put up with rather than to leave too many other stages incompletely treated. On the other hand, it is likewise not justified either to treat all stages very intensely simply because this would be adequate for just a few ones. The following examples primarily focusing on dam foundation grouting shall exemplify the principles preferred in defining the grouting pressure.

#### 6.10.6.1 Practically Tight Rock of High Strength

If core samples, outcropping rock and WPT show no water absorptions in most stages and very little ones in several others—say 5 LU (order of magnitude)—the rock is considered practically tight and, hence, needs no penetration grouting. This applies particularly to rocks of a close jointing and/or little strength and to dam types producing not too steep a hydraulic gradient.

#### 6.10.6.2 Practically Tight Rock, Latent Discontinuities Susceptible to Hydrofracturing

A practically tight rock may have latent discontinuities which are susceptible to hydrofracturing despite the high strength of the rock itself. It depends on their orientation whether fracturing becomes effective or not:

- They are not fractured under impoundment condition if they lie horizontally or if they are flatly inclined since the vertically acting hydraulic head compresses the discontinuities.
- In contrast, vertical or steeply inclined discontinuities may be pressed open while the reservoir is filled. The relationship between critical pressure and maximum acting pressure caused by the reservoir must be calculated if the latter one were larger and if the orientation were unfavorable, the under-seepage could increase over-proportionately with the rising reservoir level.

To reduce the negative consequences of large water losses it might be required to deliberately fracture these discontinuities to intercalate as many parallel groutstone cushions as possible, with the aim of reducing the plastic component of the deformation. If this could be accomplished, at least partly, the underseepage would tend to increase more linearly because the opening of the planes was restricted to a mainly elastic deformation. In this case replacement grouting prestressing the rock would have to be applied using grouting pressures above the critical level as illustrated in Fig. 6.18.

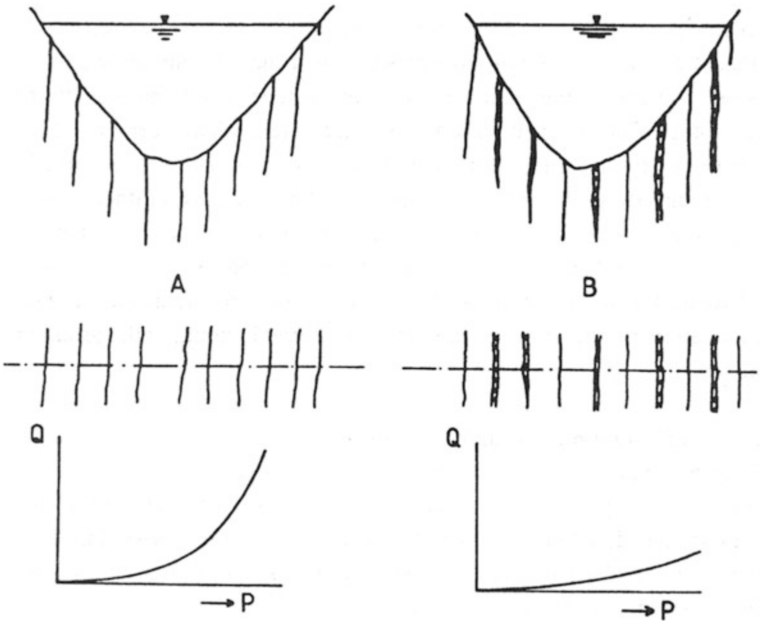



**Fig. 6.18** Groutstone pillows intercalated along vertical beddings, crossing the dam axis and being prone to hydrofracturing, may diminish fracturing and thus the over-proportionate increase of seepage

#### 6.10.6.3 Permeable Rock, Dilation of Existing Paths

A permeable rock of considerable strength and not being prone to fracturing tolerates high grouting pressures. They should be applied to utilize the dilation of the existing paths because this produces a better sealing. Thus, the grouting pressure should exceed the critical level as shown in Fig. 6.19. The adequate grouting pressure should be determined in accordance with the conditions of given cases, that means it has to be figured out in how much the grouting pressure should exceed the critical pressure.

#### 6.10.6.4 Permeable Rock, Hydrofracturing of Latent Discontinuities Controlled by Overburden Pressure

If core samples, outcropping rock and WPT's prove the rock to be permeable, a grouting pressure below the critical level should be used as sketched in Fig. 6.20. Since the rock is disconnected the critical pressure increases with depth. The respective pressures determined by WPT's should guide the depth-related grouting pressures. The permissible grouting pressure will be low, particularly in the zone near to the surface.

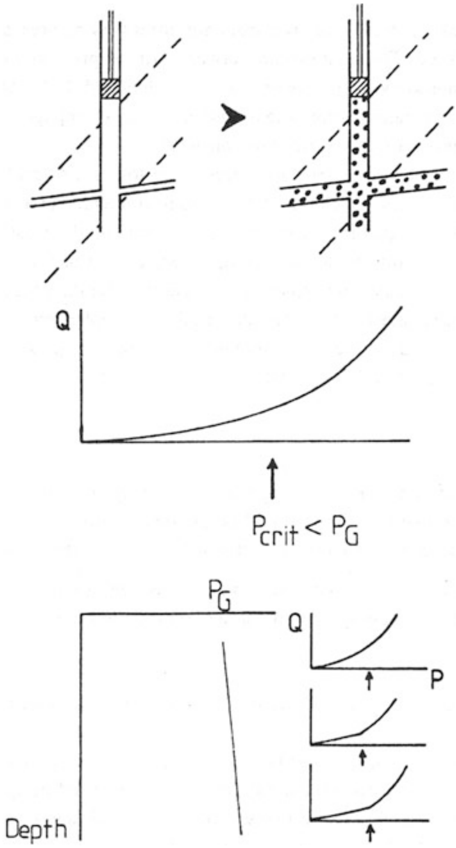


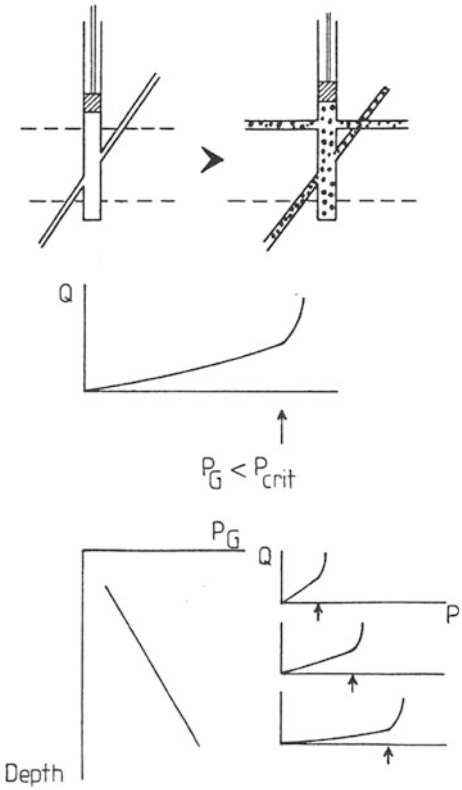

**Fig. 6.19** Definition of grouting pressures to be related to geological conditions. To use dilation of fissures in permeable hard rock for better sealing, grouting pressure should exceed the critical pressure and may increase with depth slightly without provoking hydrofracturing of latent discontinuities

There, only wide fissures can be grouted requiring low pressures due to their small head losses activated by friction. With depth, the grouting pressures can increase, thus, narrower paths can be sealed, leaving a smaller residual permeability. Grout suspensions of low viscosity should be preferred. Heavier suspensions imply the risk of a more intense fracturing.

#### 6.10.6.5 Permeable Rock, Hydrofracturing of Latent Discontinuities Controlled by Strength

The grouting pressure should be also kept below the critical level. Here the critical pressure does not increase with depth; hence, the grouting pressure should remain roughly uniform with depth. It must be determined individually. It depends on the strength before hydrofracturing begins, only low pressures should be allowed or

**Fig. 6.20** Definition of grouting pressures to be related to geological conditions. To avoid hydrofracturing of latent discontinuities controlled by overburden pressure, grouting pressure should not reach the critical pressure which increases with depth, thus grouting pressure can increase accordingly




whether high pressures can be permitted, respectively (Fig. 6.21). The groutability differs accordingly. A much better sealing can be accomplished in the latter case. In some rocks the critical pressures share the same order, in others they are scattered within a certain range. This behavior should be considered.

## 6.11 Test Grouting

A substantial grout consumption, a certain reduction of the takes through the subsequent series and groutstone layers deposited along presumed fissures seem to confirm the need, for the right execution and the success of a grouting program. This is not necessarily true. First an over-proportionate reduction of the uplift across the grout curtain demonstrates its effectiveness. Piezometers installed at either side of the curtain are needed to recognize this. On the other hand, the decision on a grouting program must be done beforehand, particularly if a gallery is not available. Thus, it is required to investigate whether the injected grout mix is not pressed into

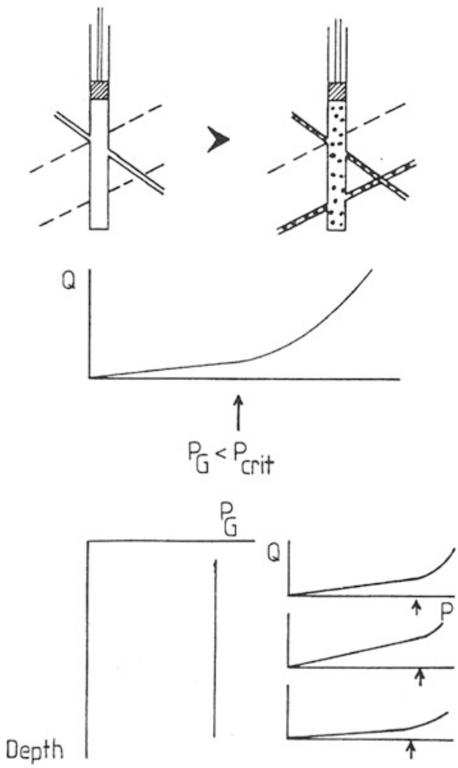



**Fig. 6.21** Definition of grouting pressures to be related to geological conditions. To avoid hydrofracturing of latent discontinuities controlled by strength, grouting pressure should not reach the critical pressure which does not increase with depth, thus, grouting pressure should not increase either

fractured rock but sealing real water paths. This must be explored with a test grouting program.

If one could believe that rocks are always groutable and the permeability could be reduced in principle by grouting to the desired degree, test grouting was not necessarily required. This situation is changing. It is increasingly acknowledged that grouting programs should be adapted to the individual conditions. Permanent adaptations will always be required during the course of the grouting work; however, the most important sealing has to be achieved at the outset. In view of the possible difference between permeability and groutability, and knowing that the permeability can be reduced by grouting only to a certain individual degree, a test grouting program is absolutely necessary.

If a project area comprises sections of essentially different conditions, we should carry out specific test grouting in each section. This test grouting should analyze the given groutability to find out which grouting technology serves best to achieve the most effective sealing at least cost. The technology comprises the relevant factors as follows:

* The best direction of the groutholes related to the orientation of the joints.
* The tendency of the groutholes to deviate.

* The adequate W/C-ratio including additives.
* The appropriate grouting pressure considering both hydrofracturing and groutability.
* The grouthole spacing.
* The number of series.
* The upwards or downwards direction of grouting.
* The type and the extent of control drillings.
* Only the most important and not self-explanatory aspects shall be briefly discussed.

The direction of groutholes in relation to the orientation of the joints is important because an intersection with the fissures to be grouted at approximately right angles facilitates penetration. But as it likewise supports hydrofracturing, an optimization is required. This also must consider the stability of the boreholes against collapsing which is influenced by the inclination of the hole, the type of drilling, the degree of jointing and the extent of weathering.

The deviation of the groutholes controls the depth of the still overlapping curtain. It ends where the deviation exceeds the lateral spreading of the grout. The deviation depends mainly on the fabric of discontinuities, on the type of drilling and on the angle of intersection.

The hydrofracturing behavior can be examined by means of WPT's. P/Q-diagrams disclose the critical pressures and the type of fracturing. The appropriate grouting pressure can be inferred from these diagrams too. The degree of jointing shown by the core samples and the relationship between water and grout takes give information on the groutability. In accordance with Fig. 6.10, the combination of intense jointing, large WPT-values and small grout takes indicate a poor groutability.

The grouthole spacing is defined by considering the decrease of the grout takes and WPT-values through three or four subsequent series. For the last series, the spacing usually varies between 3 and 1 m. If the last series has still large grout takes, a further series is chosen. This can be risky because the stress field caused by too close a grouthole spacing may loosen the rock bond again inducing a new permeability. Thus, there is always an optimum sealing determined by the individual strength.

Downwards grouting is appropriate when wide fissures appear at the surface and a grout travel towards the surface is likely to occur. An initial shallow grouting at comparatively low pressures, eventually with a closer spacing, provides for a sealing and stabilization of the upper seam. Further down a higher grouting pressure is eventually permissible.

Finally, the result of the test grouting should permit to complete both the technical specifications and the bill of quantity.

## 6.12 Type of Water Paths, Grouthole Pattern and Grout Takes

Rock types of extended and wide fissures cause large grout takes, particularly in the first series. For this case a multiple-row-curtain with a grouthole pattern between 2 and 3 m in each row is appropriate. Rock types with many but narrow paths need small grout takes already in the first series. Usually a one-row-curtain is appropriate—unless the rock is erodible calling for a wider curtain which lowers the hydraulic gradient. Small grout takes should be compensated for by a closer spacing of groutholes (order of magnitude: 1 m): however, this works only to a certain extent because too close a grout hole pattern can make the rock permeable again, as already mentioned before.

A grouting program carried out in rock types of wide and extended fissures is more expensive, of course. It yields a significant relative reduction in permeability, provided penetration grouting of natural voids can be realized and too far a grout travel can be avoided. In contrast to this, it is less expensive to grout rock types of narrow and short paths; the attainable relative reduction in permeability is smaller.

The relationship between the size of the paths, the order of the grout takes and the attainable reduction in permeability inherently leads to a limit where grouting is not warranted anymore because the reduction becomes too small and is not worth the expense. The question regarding the appropriate boundary must arise. Answers can only be given for individual cases. Nevertheless, it is suggested that in penetration grouting the spacing of the groutholes should not be less than—say—0.8 m and the grout takes should not be less than 50 kg/m, otherwise the grouting program is not considered economical and reasonable anymore. Here it is important to remember that grouting programs in rock types of very narrow paths are usually not required as such a hydrogeological setting usually causes a small original permeability.

While the geometry of the paths roughly determines both the unit price per 1 $m^2$ of the grout curtain and the relative reduction in permeability, the effect on the seepage, however, is different. The less expensive connected curtain in a rock of narrow paths is hydraulically much more effective than the more expensive hanging curtain in a rock of wide fissures.

## 6.13 Final Remarks

It was often assumed that '*grouting is more an art than engineering*'. To describe the way grouting was done in the past, this aphorism was probably not so far off the mark. But in general, the situation has already improved and this should continue. Rock grouting will never reach the accuracy being typical for the construction work in civil engineering—simply because some relevant features of the rock remain always undiscovered. But we must distinguish between properties which qualitatively characterize the grouting behavior of a given rock mass and those which decide quantitatively on detailed results.

For instance, the very distinct fissility of a well bedded sequence making the rock susceptible to hydrofracturing is a qualitative property because it requires a low grouting pressure. This should be determined beforehand because it restricts the achievable degree of impermeabilization and, thus, determines the final result. Of course, this approach presupposes acknowledgement that each rock only permits its own degree of impermeabilization, exclusively depending on the properties involved instead of our desire. Acknowledging this, also means to recognize that rock of an adverse groutability may conserve a still substantial permeability. Other means have to be applied if a further reduction should be required.

It is (almost) always possible to press large quantities of grout into the rock. It was quoted at the outset already that recently several tons per meter were injected into a claystone giving reason to presume that it had large cavities. The actual reason was quite simple: much too high a grouting pressure was applied. Such a program is characterized more correctly by the other admittedly somewhat cynical aphorism '*grouting is often an invitation for companies to make money*'. Fortunately, this stands for exceptional cases.

Finally, it is concluded that in rock grouting two essential components of our methods should be improved:

High W/C-ratios should not be used any more. Modern additives permit to achieve the favorable rheological characteristics of the grout also for efficient suspensions. Using ultrafine cements in rock grouting yields a certain additional sealing, although less than mostly expected. Using these cements can be recommended provided the expenses increase only slightly. Conventional grouting cements (Blaine's number 3500 $cm^2/g$) penetrate and seal paths up to 0.3 mm, supposed the rock is strong enough to resist the grouting pressures required for such paths. In such cases, only permeabilities of a tolerable order remain. It should be acknowledged that rock types cannot be grouted to such an extent that the same residual permeability remains. This dissimilarity is caused by the individual groutability which only permits their own degree of sealing. Under unfavorable conditions substantial permeabilities remain.

It is still customary to increase the grouting pressure with depth. This is appropriate only when the critical pressure depends on the overburden pressure which is usually limited to a shallow seam where weathering caused a loosened rock bond reducing the tensile strength of the latent planes. Further below the hydrofracturing is determined by the strength of the rock mass. It behaves largely independently of depth; thus the critical pressures do not increase either. Therefore, the grouting pressures should be guided by both the specific properties of the rock and the purpose of the treatment. In defining the grouting pressure, it should be considered that a grout mix of a high density causes a higher pressure, thus hydrofracturing of latent planes is more likely to be initiated.

# Chapter 7
# Hydrogeological Regime Around Dams and Reservoirs

## 7.1 Introduction

The water seeping from the reservoir underneath the dam and through the abutments produces uplift, causes seepage losses and may initiate erosion. These components are of prime importance for the safety and the economy of the dam. The detrimental effect of seepage and erosion increases with the permeability of the foundation. A high permeability causes a lowering of the groundwater level which is not favorable either:

- The grout curtain has to reach further into the abutments
- Water flows out of the reservoir either directly through the abutments towards the downstream slope or towards adjacent valleys, in both cases necessitating constructional measures.

We really face a discrepancy between the concepts realized in practice and a potential of improvements seldom utilized. The usual concept applied for the impermeabilization of the foundation follows the Lugeon-criterion. It calls for sealing measures, if water pressures tests (WPT) introduced by Lugeon exceed a tolerable water take which is mostly 1 l/min $\times$ m at a pressure of 10 bar (=1 Lugeon, i.e. 1 LU). Lugeon concluded correctly that the rock forming the foundation of a dam would be tight if the tests absorbed less water. The successors deduced the opposite: water takes larger than 1 LU indicate permeability, thus foundations have to be sealed, usually carried out by grouting. This concept is still largely applied. It results in many grouting programs, which are overdone or even unnecessary. The reason is simple: The Lugeon-value is merely a measure of the absorption capacity of the rock around the borehole section tested but does often not reflect the hydraulic situation of the entire foundation. Other factors are relevant too and further features are involved including a potential of considerable savings; little efforts are needed to use them.



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The terms '*permeable*' or '*impermeable*' as well as '*pervious*' or '*impervious*' have a relative meaning because rocks or soils are (almost) never completely impervious. Here these terms are to be understood for the purpose of dam construction: if the foundation lets pass too much water it is considered permeable—and vice versa. A foundation whose coefficient of permeability is smaller than $1 \times 10^{-7}$ m/s is practically '*sufficiently impervious*' and needs no treatment unless peculiar conditions exist.

## 7.2 The Position and Inclination of the Groundwater Table

In assessing the overall-permeability of the slopes bordering a future dam site it is helpful to investigate the position of the groundwater table, because its inclination informs about the permeability. Such an investigation is required if there is reason to expect a deep position of the groundwater table. The effect of the hydrogeological regime on the impermeabilization measures applies not only to the dam site itself but also to the abutments and often even to the neighboring mountains. Regrettably, it is not always possible to install a net of piezometers when an alpine relief complicates the access to the spots desired where boreholes should be drilled to install piezometers.

It causes often severe consequences if permeability of the rocks forming the surrounding area and the position of the groundwater table are not investigated. It would be helpful if the position of the groundwater table could at least roughly be determined because its inclination informs already about the overall permeability—and gives an idea whether grouting measures are required.

Unless peculiar climatic, geologic or geomorphologic conditions cause the groundwater table to fluctuate far below the bottom of the valley, the groundwater flows towards the river feeding it. The gradient required initiating that flow, thus, the inclination of the groundwater table depends on the relationship between the amount of water infiltrating the rock and the permeability of the subsoil. That inclination characterizes the overall permeability of the foundation often more precisely than the results of WPT's: steeply inclined groundwater tables usually indicate rock of little permeability—and vice versa. In consequence of this relationship it is prudent to install a net of holes for groundwater observation covering not only the dam site but also the surrounding area.

The groundwater table under the adjacent slopes influences the lateral extension of the grout curtain. In general, the curtain should extend up to the spot where the groundwater table reaches and exceeds the reservoir level (Fig. 7.1). A slowly rising groundwater table from a deep position causes unfavorable consequences as

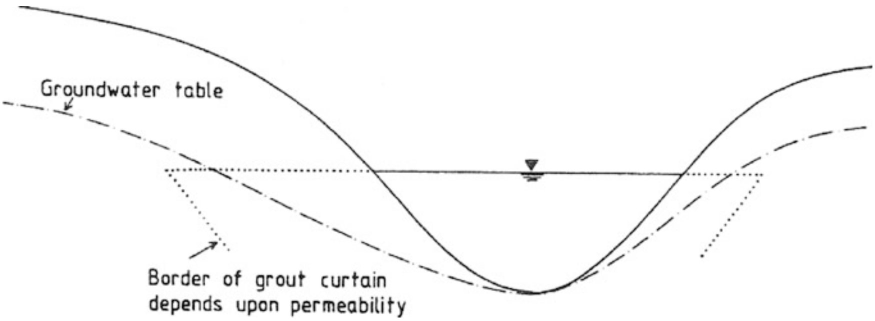



**Fig. 7.1** Relationship between the inclination of the groundwater table and the lateral extension of the grout curtain

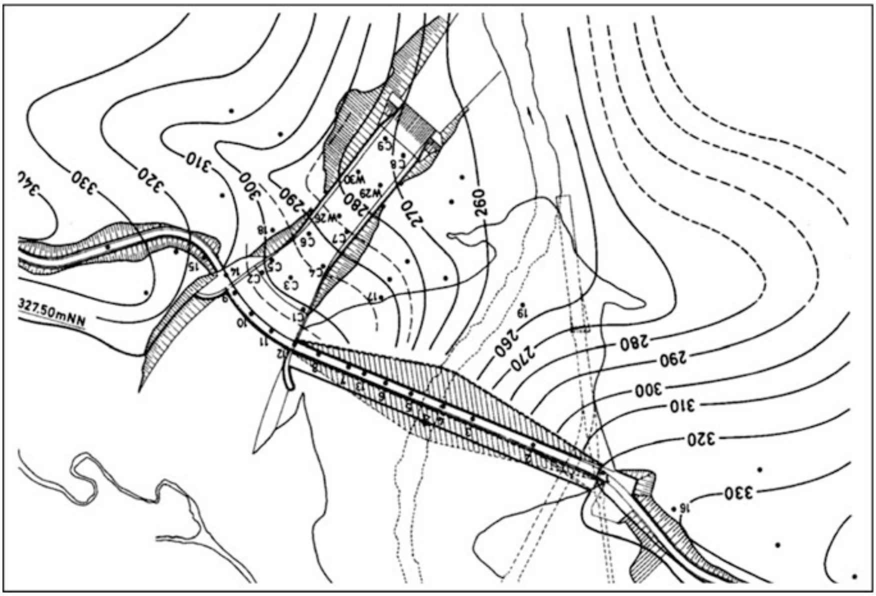



**Fig. 7.2** Tavera dam. Groundwater contours show a steeply inclined groundwater table hereby confirming the imperviousness of the rock

introduced below. Whether these consequences occur and to which extent depends on hydrogeological factors—some produce a favorable effect, others an unfavorable one. The example of the Tavera Dam demonstrates the usefulness of an early investigation of the hydrogeological regime shown in Fig. 7.2.

### 7.2.1 Steep Inclination of the Groundwater Table Indicates Impervious Rock

WPT-results confirmed the imperviousness of the rock forming the foundation of the Tavera Dam. They supported the decision to abandon the grout curtain originally designed. That decision was substantially deduced because:

- of the steep inclination of the groundwater table, which directly follows the ground surface with little distance in between, and that
- the position of the groundwater table encountered before filling of the reservoir was identical with the one afterwards.

### 7.2.2 Flat Inclination of the Groundwater Table Indicates Permeable Rock

The investigation of the groundwater table in the area surrounding the future dam site should be a must, particularly whenever the groundwater table rises into the abutments at a very flat angle. A deep position of the groundwater table may cause either unnecessary investment for overdoing or additional counter-measures to keep serious problems under control becoming obvious during the filling of the reservoir. The following conditions caused by the hydrogeological regime can occur:

(1) In case of a flat gradient of the groundwater slope and a high reservoir level the grout curtain has to extend far into the mountains. Extended tunnels are required to provide for access needed for the installation of the lateral section of the grout curtain. Without knowing the position of the groundwater table beforehand, the tunnels and the grout curtain may be too long or too short or may still be lacking.
(2) In case of a deep position of the groundwater table and an insufficient extension of the grout curtain, with the rising reservoir level the water will flow out of the reservoir into the abutment, surround the grout curtain and flow to the slope downstream of the dam—or even farther away—where it may cause erosion.
(3) A deep position of the groundwater table causes a further problem: the water stored in the reservoir with its rising level begins to form a barrier impeding the groundwater to flow towards the river but diverts the water also towards downstream—increasing the negative consequences already mentioned even further.
(4) A deep position of the groundwater table in connection with a neighboring valley causes the most serious problem possible: water escapes out of the reservoir and flows towards the other valley where it flows out of the slope.

## 7.3 Seepage and Uplift at the Dam Site

Again: WPT's only explore the permeability of the rock surrounding the borehole. This can differ from the permeability of the whole foundation between the upstream and the downstream side of the dam. In the case of the anisotropic rock sketched in Fig. 2.2, WPT-results do not reflect the permeability of that foundation which is controlled by the low permeability of the intercalated siltstone layers; a sealing would not be necessary if the siltstone were impervious enough. Therefore, it is prudent to investigate whether the hydrogeological regime comprises less permeable components to be utilized. The appropriate depth of the treatment is another important issue: until now the depth of the curtain has been frequently related to the height of the dam. Often a depth equivalent to the height of the dam was chosen. This is rather schematic and hence not appropriate any more. Also in this respect the hydrogeological regime may offer favorable components. Thus, as to the hydrogeological setting the following topics are to be discussed:

- Are there impervious intercalations between upstream and downstream controlling the seepage?
- Is there an impervious base at a shallow depth allowing the installation of a connected curtain?
- If only a hanging curtain is possible: what is the effect of this curtain on the length of the seepage paths? In this instance it has to be considered that the distances between the upstream side of the dam and the downstream relief increase towards the abutments.
- Is the natural groundwater level high enough to form a hydraulic barrier against water flowing out of the reservoir?

### *7.3.1 Influence of Geological Features Causing an Anistropical Permeability*

The possibility of an anisotropic permeability due to less pervious intercalations as shown in Fig. 2.2 is well known. However, such conditions mostly remain disregarded and the decision in favor of a treatment is based only on WPT-results. The following few examples demonstrate that such features can be successfully utilized. All geological features influencing or determining the permeability of the whole foundation between upstream and downstream which permit reducing the scope of the work should be considered.

#### 7.3.1.1 Alluvial Loam

Alluvial deposits in flat or moderate valleys often consist of fine grained material, such as clayey silt. This alluvial loam usually has a low permeability, probably less than $1 \times 10^{-6}$ m/s. By removing the topsoil layer and slightly compacting the loam upstream of the dam its permeability can be further reduced, and so it functions as an impervious blanket. For dams of a moderate height or for selected sections of such dams this blanket can substitute or complement the treatment of the rock.

A very convincing example was experienced at the Twiste Dam where the loam blanket upstream of the dam causes an over-proportionate head reduction (Fig. 7.3). Similar effects were observed also at other dams. Following this experience, the site for a further dam has been very carefully examined, hydraulic computations clearly showed the effect of the less pervious alluvial loam.

On the basis of positive experiences gained at various dam sites, the authors recommend utilizing the low permeability of alluvial loams. Until now too little attention has been paid to this issue. As the loam is often overlaying sandy gravel it

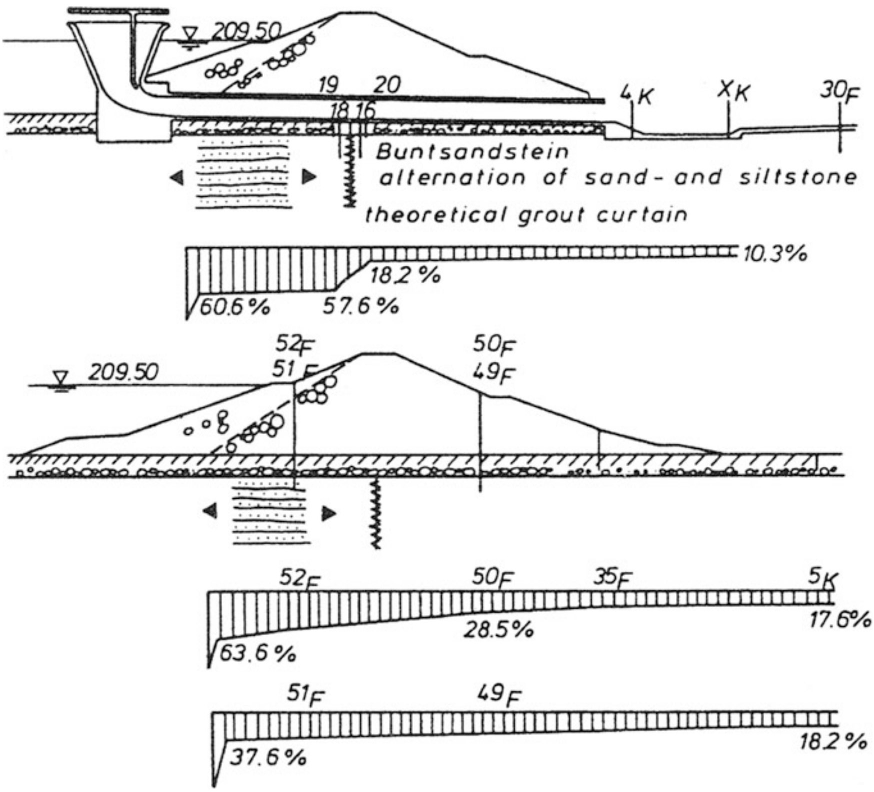



**Fig. 7.3** Twiste Dam. Over-proportionate head reduction across the alluvial loam upstream of the dam

has to be investigated whether the contact between the two is filtertight, otherwise too high a gradient could initiate erosion.

#### 7.3.1.2 Alternated Beddings

In considering zones of different permeabilities we usually envisage zones of a sizeable thickness, say >10 m. Figure 7.4 shows such a setting and its effect on the uplift reduction. At the Aabach Dam it has been the experience that even '*micro-features*' reduce the uplift substantially, and the effects of these features should be utilized. Details are presented in Chaps. 3.6 and 4.10.

The dam has been built on an alternating sequence of sandstone and siltstone—with the latter one prevailing. The outcropping rock and the core samples showed closed bedding planes; cross joints were partially open in the sandstone beds but closed in the siltstones. The beddings are thin (cm to mm), thus a separate differentiated permeability test for the sandstone and siltstone-beds, respectively, was impossible. Permeability testing yielded WPT-values of up to 60 LU while the grout takes of the test grouting indicated very small takes indicating a multitude of fine fissures which cause a poor groutability. From the state of the discontinuities it was inferred that the sandstone is quite permeable while the siltstone is almost impervious.

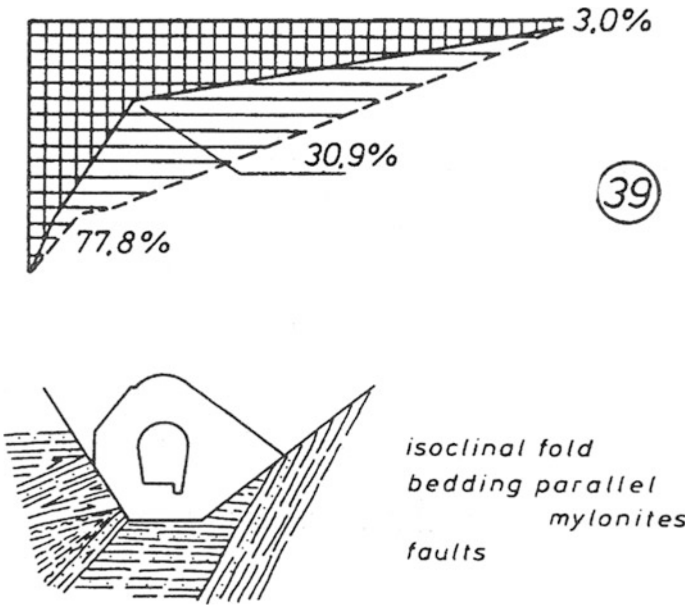



**Fig. 7.4** Aabach Dam Micro-features reduce uplift over-proportionately

Heavy isoclinal folding led to a grinding of the siltstone layers along the fold axis, turning this rock into a plastic dense mylonite, just several centimeters in thickness, which appeared rather impervious. The bedding strikes parallel to the axis of the dam, except one 30 m wide section in which the beddings strike across the dam. Thus, an anisotropic permeability of the rock mass between upstream and downstream had to be envisaged: along the sandstone beds—parallel to the strike—the rock is permeable, whereas across the siltstone beds the permeability is very much lower. Special tests confirmed the presumed '*anisotropic permeability*', Parallel to the bedding a high permeability caused by the sandstone-beds was found, vertical to the bedding the permeability was low due to siltstone-layers and mylonites which the water had to cross on its way downstream.

It was decided not to treat the foundation in spite of the high WPT-values. The negligible amount of seepage—less than 0.5 l/s (height of the dam: 50 m, length: 300 m)—proved that this decision was correct. Apart from this result the reduction of the hydraulic head is of particular interest: along the short flow just around the gallery an over-proportionate head loss takes place due to the repeated crossing of the less permeable mylonite layers while the head reduction along the flow parallel to the bedding is quite meaningless (Figs. 3.26, 3.27, 3.28 and 3.29).

#### 7.3.1.3 Fault Zones

A fault crosses diagonally the axis of the Antrift Dam separating fine-grained uncemented clastics of Miocene from sandstones of Lower Triassic (Fig. 7.5). The sandstone is very permeable. Thus, a diaphragm combined with a grout curtain was installed to minimize seepage. The treatment also covered the zone of the Miocene, although these deposits were considered less pervious. The long-term behavior now being examined over a period of more than 30 years shows that the permeability of

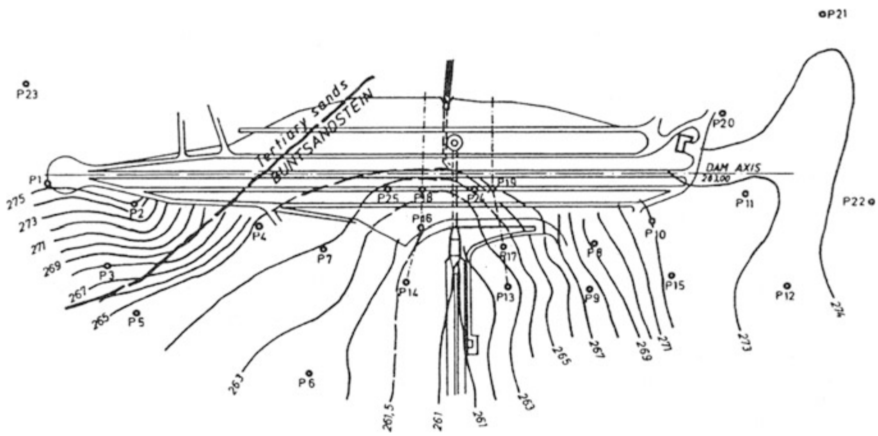



**Fig. 7.5** Antrift Dam. Influence of a fault on the reduction of uplift

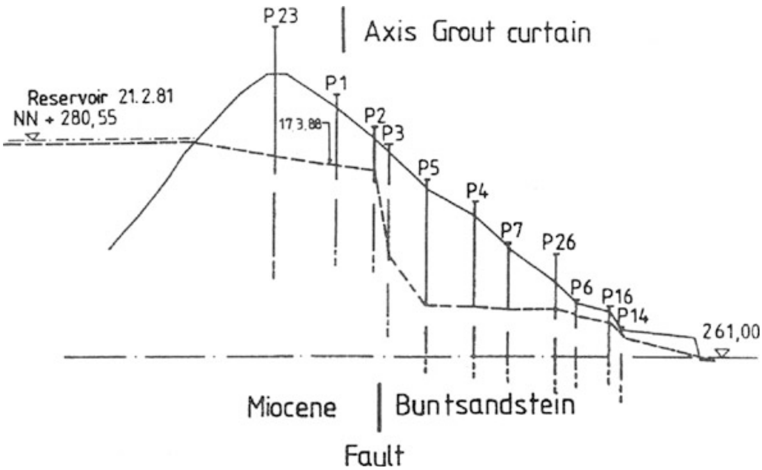



**Fig. 7.6** Antrift Dam. Influence of a fault on the reduction of uplift

the Miocene deposits, particularly along the fault zone, is much lower than anticipated and even lower than the grout curtain: the reduction of the uplift takes place predominantly across the fault and inside the Miocene while the curtain is not hydraulically stressed (Fig. 7.6). The practical consequence is clear. If this had been known more precisely beforehand, the treatment of this section could have been abandoned.

## 7.3.2 *Depth of Curtain; Connected or Hanging Curtain*

The concept of relating the depth of the curtain to the height of the dam should be replaced by a '*hydrogeological approach*'. This means examining whether the more effective '*connected*' curtain is possible or whether the geological setting only permits a '*hanging'* curtain:

- A connected curtain can be accomplished when the zone of lower permeability begins at a 'reachable' depth which depends on the deviation of the grout holes. Too great a deviation leaves ungrouted spots ('*windows*'). The deviation of groutholes, i.e. the danger of leaving untreated windows instead of achieving a completely overlapping curtain is controlled by both the type of the rock (strength, discontinuities) and the type of the drilling (vertical or inclined, core or percussion). Under unfavorable circumstances a tight curtain can be obtained only down to a depth of, say, 50 m. If the situation is favorable a depth of >100 is achievable, Fig. 6.1 illustrates the higher effectiveness of a connected curtain: the water has to flow either through the curtain and/or through the less permeable underground. The permeability of the curtain can be reduced from 100 to 1, provided the rock is not only permeable but also groutable (not necessarily the same!).

- A hanging curtain is required whenever the less permeable zone begins at such a greater depth that long grout holes do not produce an overlapping curtain but leave too many windows due to the deviation. This curtain functions as a wall penetrating into permeable rock, and the water flows under the toe of the curtain. As shown in Fig. 6.1 the water flows under the toe of the hanging curtain and through the untreated section of the rock thereby lengthening the flow lines. This also reduces the hydraulic gradient—but only to 50 or 30% and the amount of seepage diminishes accordingly.

The depth of the curtain should be related to the next underlying zone of '*sufficient impermeability*'. If such a zone lies near the surface, a shallow curtain is sufficient, irrespective of the height of the dam. Alternatively, where the less permeable zone begins only below the reachable depth of grouting, a less effective hanging curtain can be installed although more grouting work is required. The underlying rock has to be examined with respect to the position of an eventual boundary.

As the degree of deviation is strongly influenced by the type of drilling (core or percussion) it should be considered whether the more expensive core drilling, less prone to deviation, is the more economical solution because a more effective connected curtain can be obtained provided the impervious rock is not too deep.

The connected curtain reduces the uplift by reducing the flow of water traversing the curtain itself. The hanging curtain reduces the uplift by lengthening the flow lines around the toe of the curtain. While the effectiveness of the connected curtain depends on the achieved reduction of $k_f$, the hanging curtain works by this lengthening in relation to the length of the flow line between the upstream toe of the dam (100 head) and the relief at the downstream side (±0%). The shorter the hanging curtain the less effective—and vice versa. Too shallow a hanging curtain is practically ineffective.

### *7.3.3 Lateral Extension of the Grout Curtain in Relation to the Length of Flow Lines*

The length of the flow lines between upstream and downstream differ according to their position. The shortest flow line traverses the foundation within the lowest section of the valley, the lateral flow lines at either side becoming increasingly longer the further they run away from the center; the longest flow lines possible are crossing the abutments. The hydraulic gradient, inversely, is the greatest along the shortest flow line—and decreases towards the abutments. The effect of a hanging curtain of a given depth decreases accordingly. The relationship between the length of the flow lines and the depth of the curtain is an important factor in defining the lateral extension of the curtain, exemplified as follows:

- The shortest flow line under the base of a dam is typically 100 m. Without a curtain a 100 m high dam produces a hydraulic gradient of 1.0 A 100 m deep curtain trebles this flow line, reducing the hydraulic gradient to one third.
- A flow line traversing the abutment may reach a length of 400 m. Without a curtain this gives a hydraulic gradient of 0.25. The 100 m deep curtain lengthens the flow line to 600 m and reduces the hydraulic gradient to 0.16.

It is evident that in the outermost sections the influence of the curtain lessens and loses its function. Thus, it is prudent to install a hanging curtain only in that section of the dam where it really decreases substantially the hydraulic gradient. Too small a reduction would not re-pay the investment for the curtain.

The Twiste Dam gives an impressive example. The lateral flow lines are getting longer, particularly towards the right slope, and the hydraulic gradient decreases accordingly. The influence of the curtain disappears completely and approximately half of the curtain is not in use (Fig. 7.7). If this had been recognized and considered beforehand, the extension of the curtain could have been limited to that section of the dam where the curtain causes a significant lengthening of the flow lines. Such a possibility was not considered because this effect had been unknown in the past. Now, it is worth examining the hydrogeological situation and

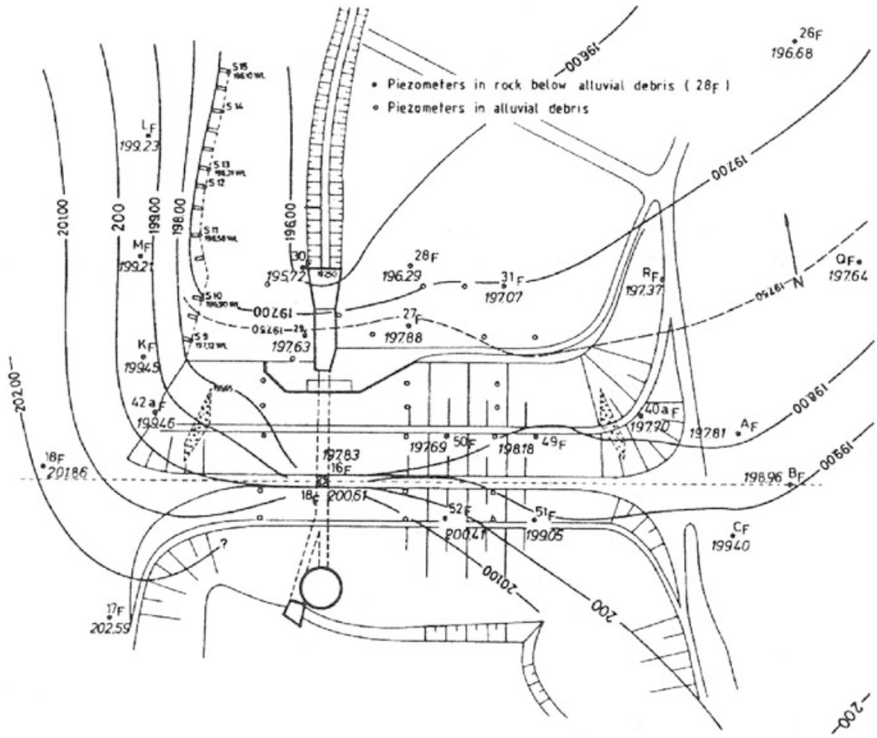



Fig. 7.7 Difference in reduction of uplift along the dam

establishing a hydraulic model in order to find out the benefits of this effect. Recently we have got positive results from new projects.

### *7.3.4 Inclination of Groundwater Level and Lateral Extension of a Grout Curtain*

Provided the geological setting allows the installation of a connected curtain, its lateral extension is determined by the inclination of the groundwater level. The zone to be sealed should reach up to the spot where the groundwater level exceeds the reservoir level (Fig. 7.1). During the feasibility study it is often impossible to investigate the groundwater level and its seasonal fluctuations. Hence, the position of the groundwater table mostly remains unknown. It is only with the impoundment that it becomes obvious whether the curtain has the correct extension.

When the curtain is too short, additional grouting is required implying over-proportionate costs. When the curtain proves too long, grouting was overdone. All this becomes particularly meaningful when the groundwater table has a flat inclination. In this latter case a long grout curtain together with access tunnels are required. We often have this situation in karstic limestone or other permeable rocks-jointed sandstones, for instance. It is expedient to examine the hydrogeological situation as early as possible to avoid the negative consequences of insufficient data. The following example demonstrates this complex.

The Pueblo-Viejo Dam (Guatemala) is situated in karstic limestone. Considerable hydrogeological investigations were carried out during the feasibility study. However, the situation of the groundwater table could be examined only in a relatively small area surrounding the dam site, because at that time it was impossible to install piezometers in the high surrounding mountains. Because of the distinct karstification (wide and extended fissures, cavities and caves) a systematic grouting had to be done. As too little was known about the inclination of the groundwater table, it was considered necessary to extend the grout curtain over 830 m into the left abutment and 420 m into the right (Figs. 3.33, 3.34). At that time another concept for the treatment of the rock would not have been justified. The final result seemed to be satisfactory. However, with further information a more economically favorable concept could have been realized because during the impoundment it became evident that a less extended grout curtain would have been sufficient:

- On the right side the original groundwater table was more inclined than anticipated. Moreover, the axis of the curtain is curved to reach the main fault more directly. Therefore, at the rear section of the curtain the normal groundwater flows from the hillside paradoxically from a downstream to an upstream direction against the curtain where it meets the seepage water coming from the reservoir. Groundwater and seepage water flow against each other, thus there is no hydraulic gradient across the curtain.

- The left side shows a similar situation. In the rear section (approx. 150 m) the natural groundwater table runs above the reservoir level. The natural groundwater also flows from the '*downstream*' hillside against the grout curtain. The groundwater and the seepage water are directed against each other. thus the hydraulic gradient across the curtain is likewise compensated. Within this section the grout curtain is not stressed either.

Both sections together amount to approx. 330 m. Provided the hydrogeological situation had been known better and hydraulically analyzed beforehand, the shorter grout curtain would have been sufficient.

## 7.4 Water-Tightness of the Reservoir

### *7.4.1 Deep Position of Groundwater Table*

A situation where there is a valley adjacent to the reservoir and a highly permeable rock in between, may cause a lowering of the groundwater table, possibly not reaching the reservoir level. A seepage line through the mountain will develop and water can flow out of the reservoir (Fig. 7.8). The reservoir may not be able to be filled up to the desired level—or indeed at all. Reservoirs located in karstic limestone are particularly vulnerable. The possibility of such a hydrogeological situation is generally acknowledged. However, a systematic investigation concerning the position of the groundwater level is often not carried out. Unless there are sufficiently clear indications that the groundwater is at a high level, the groundwater level of the surrounding area should be investigated beforehand.

The already mentioned Twiste Dam is an instructive example of this complex. The original design planned a grout curtain extending only 20 m into the abutments, other sealing measures were not foreseen. Investigations carried out to improve the grouting technology disclosed unfavorable hydrogeological conditions. In particular the groundwater table under the left slope was at a very low level due to its high permeability and the neighboring valley just 250 m away. The

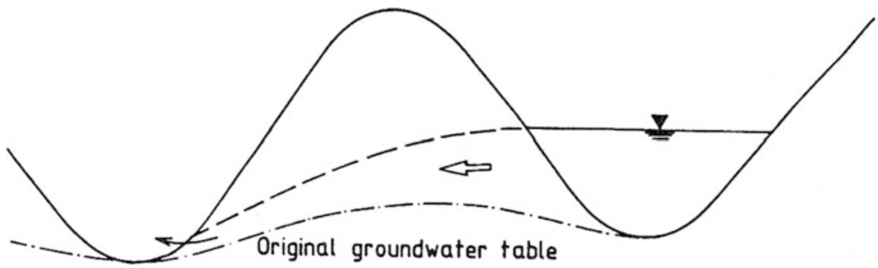



**Fig. 7.8** The deep position of the groundwater table causes a flow out of the reservoir towards the neighboring valley

hydrogeological situation at the right side was unfavorable too, although less serious. A hydraulic calculation showed that, without further measures, a considerable seepage would occur directly through both abutments of the dam towards downstream as well as through the left slope towards the adjacent valley. It had to be expected that the reservoir could never be filled completely. Thus, the slopes were sealed by means of an impervious embankment and the alluvial loam covering the bottom of the valley was compacted to lower the $k_f$-value. All impervious components together (dam, grout curtain, slope embankment and compacted alluvial loam) formed an impervious trough.

The results of the test filling program proved the necessity of the additional measures very clearly. The embankments on either side initially only reached up to the permanent reservoir level. When this blanket was temporarily overtopped and water infiltrated the slope, the expected seepage occurred. Springs appeared on both sides downstream of the dam and along the adjacent valley. After that, the embankment was enlarged up to the full reservoir level to avoid the infiltration of the slopes. The sealing measures again completely changed the groundwater regime of the surrounding area with considerable consequences for the project.

### 7.4.2 Impact of the Reservoir on the Groundwater

The high permeability of the rock in the surrounding area may cause a lowering of the groundwater table which may require slope sealing. This causes the groundwater level to react. As the groundwater is prevented from flowing into the river as before, it is stored behind the embankment and creates a rising groundwater level. At this point, the stored groundwater is diverted, flowing along the embankment, which functions as a hydraulic barrier, towards the dam where springs appear at the downstream slope.

The Twiste Dam, taken above as an example of the consequences of permeable reservoir slopes, conversely serves as an example for the impact of the reservoir on the geohydrological regime. Figure 7.9 shows the original direction of the ground-water flow, Fig. 7.10 the diverted flow.

A dam site situated in a very permeable sandstone-siltstone complex of poor groutability, with a low groundwater table and being susceptible to mechanical erosion, would usually be ruled out. But at the beginning of construction the unfavorable geology was not known and only uncovered step by step. The poor geotechnical conditions had to be compensated by construction. The conditions of high permeability in the surrounding rock was expressed for the first time at this dam. A low groundwater level required the slopes to be sealed. This diverted the stored groundwater producing the springs which were causing considerable concern about the safety.

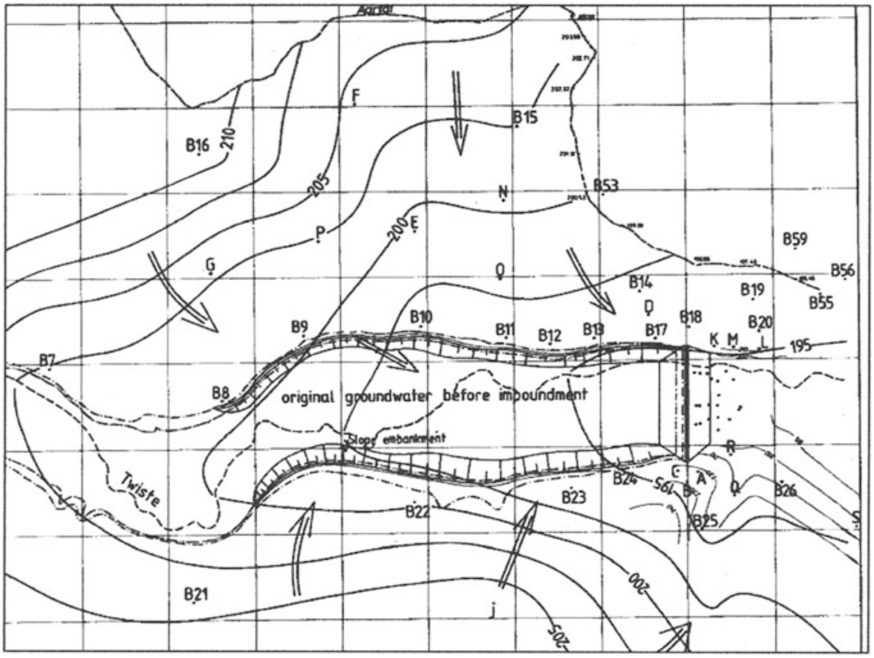



Fig. 7.9 Twiste Dam. The original groundwater table

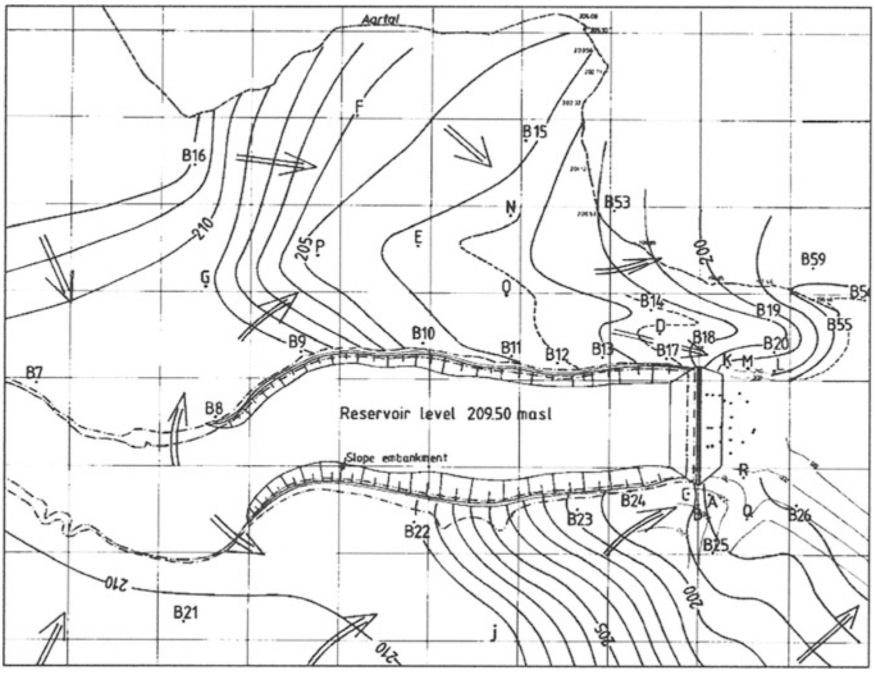



Fig. 7.10 Twiste Dam. Groundwater table after filling the reservoir

### 7.4.3 Precautionary Measures

When a second dam (Haune Dam) was built under very similar conditions all the required measurements could be planned from the beginning. Based on piezometer readings, the expected quantities of diverted groundwater were calculated taking into consideration various rainfall intensities and reservoir levels. The dam and the abutments received adequate filter layers; a close net of piezometers was used to observe the hydraulic behavior of the underground flow confirming the coincidence between actual development and design criteria.

The interaction between permeable rock, the sealing of slopes, the effect of the reservoir level on the direction of the groundwater flow and additional construction safety measures are summarized as follows:

- In permeable rock types the natural groundwater table can lie below the reservoir level. The reservoir forms a hydraulic barrier hindering the groundwater from the usual flow into the valley. The diverted groundwater flows towards the slope downstream of the dam. Springs are likely to occur at the toe of the dam.
- This change of hydrogeological regime has a considerable influence on the design of the dam as additional measures are required. Filter beds need a higher capacity and mechanical filter tightness is of particular importance as the amount of seepage water causes a permanent and considerable flow.

The slopes will require sealing—usually by means of impervious blankets—up to the full reservoir level. The length of this sealing depends on the individual situation of the groundwater and, of course, on the hydraulic head developed by the reservoir. It is appropriate to optimize the relationship between the factors '*length of sealing/reservoir head/expenditure*' by using model computations.

The examples presented should be understood as a proof of the principle that it is possible to build dams under the unfavorable geological conditions of a very permeable and erodible rock. The particular problems can be controlled by construction measures. However, in order to keep these measures within a reasonable scope the reservoir cannot be too high. Dams higher than 30 m (order of magnitude) are probably too expensive for constructional safety measures and are not worthwhile, unless there are special circumstances.

With regard to the methodology of the required hydrogeological investigation programs it is expedient to focus on two aspects:

- A system of piezometers is needed to monitor the hydrogeological regime, such as the configuration of and fluctuations in the groundwater table, and its possible consequence for the reservoir. Piezometers should be installed during the design phase before construction, so that the results can be used in the design. Unfortunately, this is not usually done.
- Using contour line maps to understand both the position of the groundwater table and the flow direction can be very helpful. Although the geologists

involved are specialized in engineering geology, in the author's experience, they are less used to work with this type of evaluation and presentation of data.

## 7.5 Hydraulic Computations

During the last decades of years, it has become common practice to calculate the groundwater flow in rock. lf such calculations are based on model tests simulating a percolated rock, their results will fit the observations. This coincidence is not surprising because the hydraulic characteristics of the laboratory model are known. It is much more difficult to establish a realistic model for actual cases because all the hydraulically relevant properties of a given rock mass are not easily understood. Nevertheless, examples have proved the usefulness of such a computation—provided the hydrogeological regime of the rock is not too complicated and is thus recognizable.

A realistic model has to consider the actual geometry and the permeability of the foundation including eventual layers of different permeabilities. The definition of such a model is usually very difficult or impossible. The main problem in establishing the model is to define generalized realistic parameters. The percolation through rock is determined by the hydraulic rules, thus, it should be possible to calculate it. But the question is whether the parameters describe the actual situation correctly enough. Where the parameters are not definable one should abandon the idea of computing the percolation behavior. This may apply for a rock which is too heterogeneous or insufficiently defined.

In order to define the model, the type of water routing should be known. Qualitatively, two different groups exist, and it needs an experienced geologist to assign the given rock types

- The fissures are quite regular, particularly in a near-surface seam. This applies to a regularly jointed sandstone, for instance. For such a circumstance, realistic parameters can be defined. These rock types are suitable for hydraulic computations.
- The discontinuities are not entirely open; approximately parallel fissures do not exist. The course, arrangement, width and shape of the water courses are irregular and not predictable; paths of different sizes exist next to each other. Core samples alone do not allow recognition of the characteristics, large sections of outcropping rock disclosing details are required for mapping.

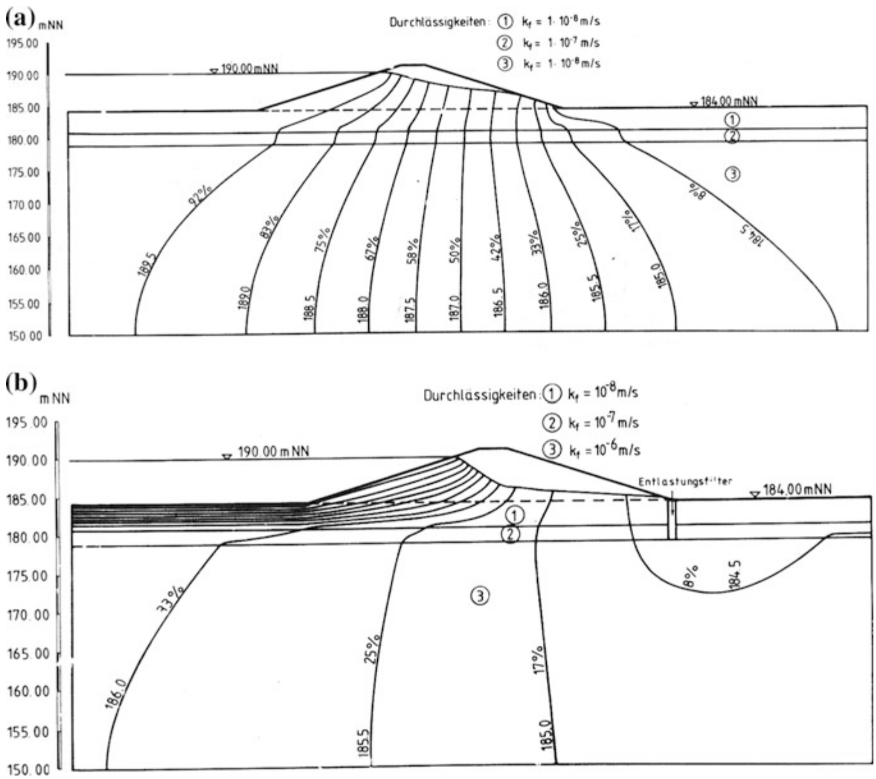
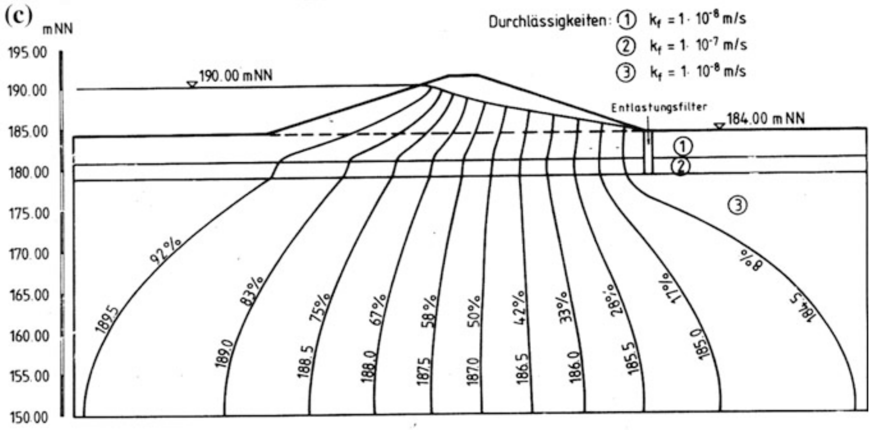



**Fig. 7.11** **a**–**f** Modelling of the under-seepage at the Sandwasser Dam under different settings

The example of the Sandwasser Dam (near Duderstadt, Germany) serves to demonstrate the usefulness of a model calculation for the under-seepage of a dam, including the impact of windows and the pressure relief caused by filters (Figs. 7.11a–c). It is unnecessary to comment on this example as the graphical presentations are self-evident. The second example deals with the Haune Dam (near Fulda, Germany). Here, a hydraulic computation was successfully applied to define the flow around the shoulder of the dam intended to select the right dimension for the filter layers (Fig. 7.12).

Hydraulic computations are considered to be a valuable method of examining the amount of seepage and the reduction of hydraulic head. It is possible to study the influence of the grout curtain and to optimize its depth and lateral extension.



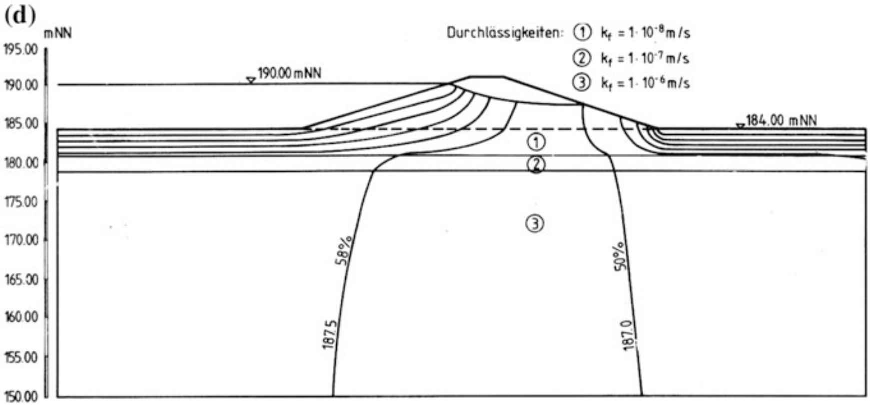




**Fig. 7.11** (continued)

The impact of permeable areas (‘*windows*’) within the upstream foundation of the dam on both the seepage and the head reduction can be estimated. The result of filter measures including the relief wells can be analyzed, and such measures can be optimized. Calculations are useful for estimating seepages around the shoulder of the dam through the abutment. The positive experiences justify such computations. The result, however, has to be understood strictly as a model. One has always to be aware of the possibility that the realistic situation might turn out to be different.

Judging from the effect of groundwater on the construction of tunnels, caverns, dams and on the effectiveness of impermeabilization measures, we need a more

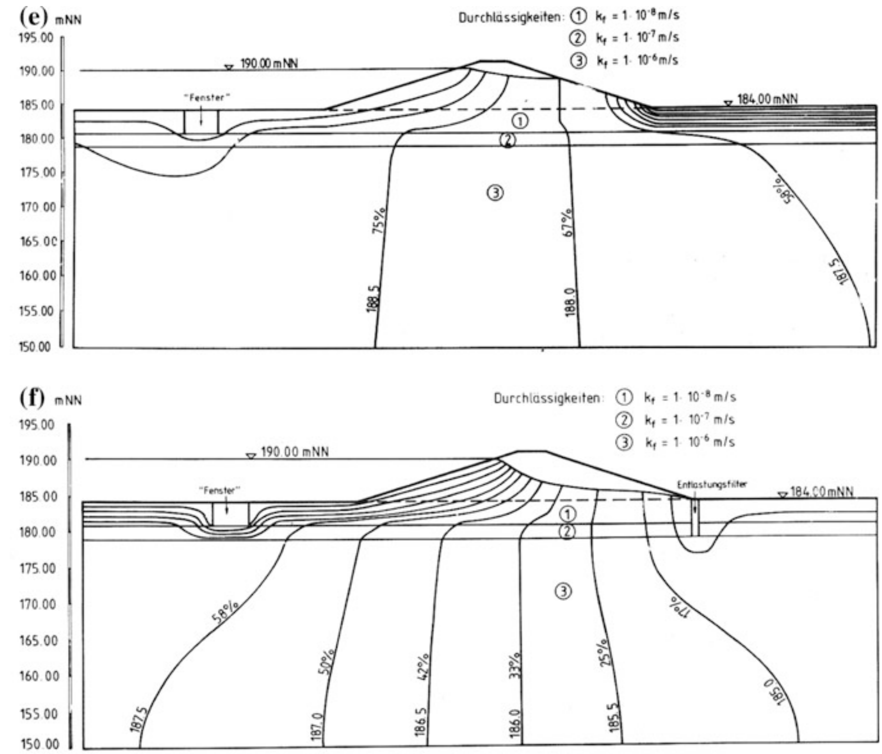



**Fig. 7.11** (continued)

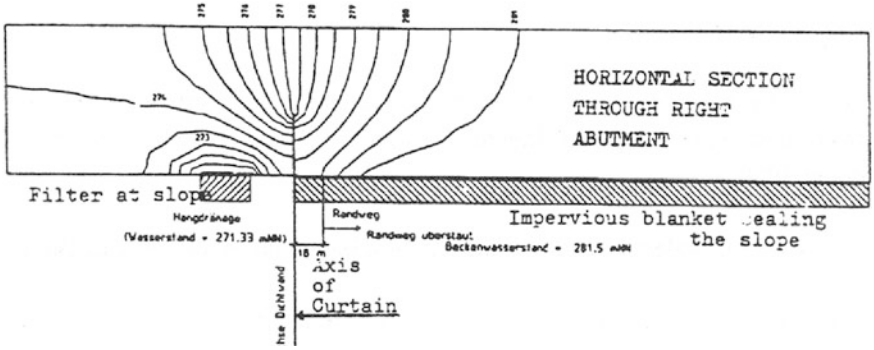



**Fig. 7.12** Modelling of seepage through right abutment at Haune Dam

precise knowledge of how water percolates through rock. We probably still lack the basic information to enable us to predict more accurately the behavior of rock masses as a result of their groundwater flows.

Considerable research work to improve fundamentals has still to be carried out. We should try to examine the various rock masses systematically to define their individual water routing. The results should be related to their petrographic and tectonic setting. It is very likely that a more systematic study will disclose typical characteristics of common validity.

# Chapter 8
# Doubts in GIN Principle Confirmed

## 8.1 Introduction

In rock grouting the custom has evolved of using individual concepts in order to comply with a given geological setting. A very high grouting pressure, for instance, may be adequate for case A but dangerous for case B. The diversity of nature calls for specifically-adapted concepts. This, however, is not always understood. Engineers used to well-defined standards and sometimes irritated by uncertainties occurring in rock grouting are inclined to consider standardized concepts as, for instance, the GIN principle proposed by Lombardi & Deere.

The GIN principle's decisive component is the concept of using a constant Grouting Intensity Number defined as GIN = $P \times V$ (equivalent to the grouting energy). The author in 1993 became acquainted with GIN at a dam site where it led to an excessive grout consumption without achieving adequate sealing. Too high a grouting pressure defined in accordance with the GIN principle caused an intense hydrofracturing. That consumption, and fundamental geological factors left unconsidered, made the usefulness of GIN questionable.

The very first impression was that GIN causes too much grouting in the case of narrow joints, while wide fissures are not reliably grouted. At the same time, it was clear that a method of such a tempting simplicity is attractive to engineers. Particular knowledge and experience to recognize the doubtful aspects of GIN are required, understandably better perceived by geologists. Grouting, after all, means treating a '*geological body*'. Grouting programs are frequently overdone or ineffective, either because of inadequate execution or because of too small a natural permeability. Unless at least a minimum of original voids is available, grouting is neither required nor possible. This remains unnoticed whenever grouting programs are not thoroughly evaluated or geologically interpreted and pairs of piezometers are lacking, notwithstanding the fact that the uplift may actually be reduced across the curtain to be recognized. The reservoir is impounded and a sufficiently small seepage is assumed to confirm the success of the curtain. However, only a small

 
F.-K. Ewert and U. Hungsberg, *Rock Grouting at Dam Sites*, Professional Practice in Earth Sciences, DOI 10.1007/978-3-319-64036-5_8

amount of seepage does not testify in itself that grouting reduced the permeability; it is quite possible instead that the underground would also have performed satisfactorily without any treatment.

Examining the data of several grouting programs confirmed the doubts. A summarized analysis has been published, and Lombardi has recently responded. The question arose of how to answer. A short summary will not convince supporters. In view of the serious implications, it is perhaps appropriate to discuss the substantial aspects calling for clarification. Lombardi's comment ignores fundamental characteristics of rock masses: this has to be substantiated. It is also necessary to address the critical components of GIN and to show how exemplary projects previously appreciated right now have not been quoted accordingly and interpreted correctly. Limited space here does not permit to reproduce the figures previously printed to which readers are referred.

## 8.2 Fundamental Characteristics

Large grout takes do not in themselves prove that grouting has been executed successfully, since it is rather easy to press (almost) any quantity of grout into the rock without achieving a positive result. Inadequate grouting can even affect the rock. Thus, a grouting program fails unless it is designed and carried out in accordance with the relevant properties of the rock. The importance of these qualities is usually emphasized, although details disclose that not everybody involved actually considers these factors.

### *8.2.1 Grouting Processes: Theoretical Analysis or Empirical Research*

Lombardi refers to a theoretical analysis he has made. Other engineers have made similar studies. The flow of cement suspensions through pipes is mathematically simulated and lab-tests are carried out to confirm the simulation. These results are used to draw conclusions for practical grouting programs. This approach is misleading because only the adjustable properties of the grout mix can be taken into account while all other essential factors are unknown and remain disregarded: the geometry of the paths as well as the width and wall roughness change permanently; infillings obstruct the voids; branching of paths change the hydraulic system; groundwater may dilute the suspension; latent discontinuities can fracture disturbing the hydraulic system; different strength properties of the rock influence both hydrofracturing and hydrojacking; and the angle of intersection between groutholes and fissures influences the grouting process.

It is stated that '*In general, grout will enter a number of joints at the same time*'. This conflicts with hydraulic laws and test results: narrow joints activate higher head losses due to friction than wider ones. The pressure needed to penetrate fissures increases inversely to their widths and directly with their lengths. As there hardly exist similar or even identical fissures within the same grouthole stage, the grout first enters the widest one; a second fissure can only be grouted if its entrance is not sealed already at the time the pressure gets high enough. The distribution of groutstone layers encountered in drill cores and excavation pits confirms the unequal grouting of dissimilar paths.

Given that there are unknown factors that cannot be grasped beforehand, how could a theoretical analysis based only on the properties of the grout simulate the grouting process appropriately? Lombardi himself expressed doubts: "*In any case the detailed knowledge of the geometry of the joint systems and of the mechanical characteristics of the joints themselves, is required to understand the grouting process. It is also necessary to specify the properties of the grout mix in order to obtain a given result.*" Thus, only an empirical study considering the factors, the properties of the grout, the head losses of the paths, the results of both water pressure tests and grouting as well as the geometry of '*groutstone intrusions*' reveal the grouting process. Investigating numerous grouting programs yielded results which have been useful already for practical purposes. If further programs were examined, the results might become more reliable, confirming each other. This approach makes it possible to understand how hydrofracturing and hydrojacking actually take place.

### 8.2.2 Hydrojacking and Hydrofracturing

Hydrojacking is commonly understood as a dilation of an existing fissure during the grouting process. Hydrofracturing is the cracking of latent, i.e. originally tight discontinuities. Despite these particular definitions Lombardi himself has mixed up both terms repeatedly. The grouting path curves (GPC) in Figs. 8.1 and 8.2 indicate hydrofracturing occurring at the critical pressure marked. In both cases GIN is zero at the moment of fracturing—but only for the very short moment of the fracturing itself—almost simultaneously begins the intrusion of the grout.

Hydrofracturing means pressing grout into a tight rock. Unless fracturing is required to gain access to fissures running parallel to the grouthole, it should be avoided. Hydrojacking instead can be useful, as it provides for a better sealing at a little more expense; under favorable conditions it does not enlarge the grout takes too much and a limitation is then unnecessary. Thus, there is no reason to be afraid of a certain amount of hydrojacking.

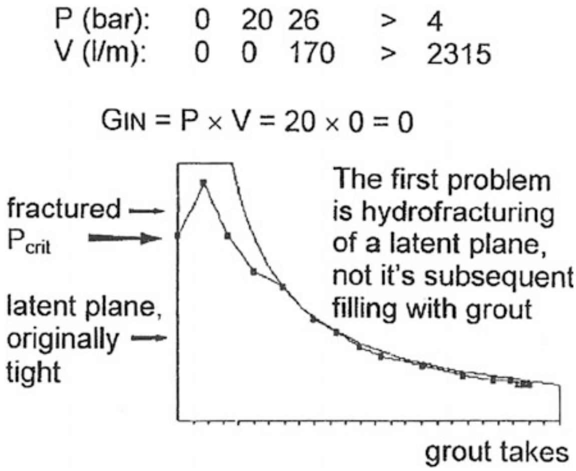



**Fig. 8.1** Grouting path curve (GPC) of Project D indicating hydrofracturing and subsequent filling

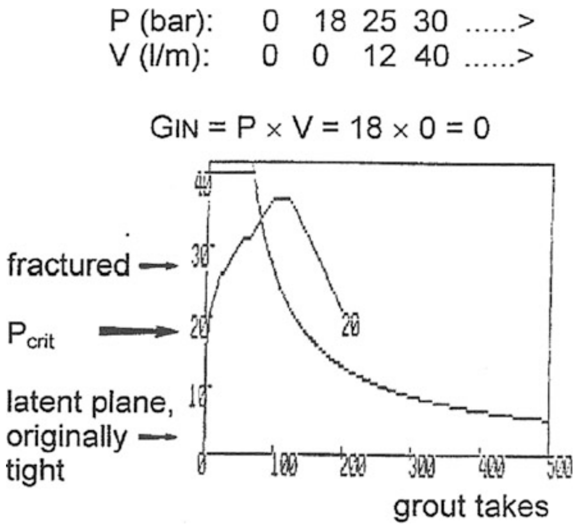



**Fig. 8.2** Grouting path curve (GPC) of Project C indicating hydrofracturing and subsequent filling

## *8.2.3 Hydrofracturing and Hydrojacking Related to Grouting Intensity*

Lombardi wrote: "*The risk of hydrojacking or hydrofracturing is highest where the grouting intensity is the greatest. This fact was clearly shown by the theoretical analysis of the grouting process*".

As illustrated in Fig. 8.3, hydrofracturing and hydrojacking have individual conditions which cannot be discussed here in detail. The grouting intensity occurring in hydrojacking differs from case to case. The situation of hydrofracturing in principle is the same at the very beginning and irrespective of different critical pressures, as the typical GPC shown in Figs. 8.1 and 8.2 demonstrates: At the beginning the grouthole stage is tight. While P is increasing V remains at zero. Latent discontinuities intersecting the grouthole are hydrofractured when the effective grouting pressure exceeds the critical pressure $P_{eff} > P_{crit}$. In the very moment of cracking grout has not yet been intruded and $V_0 = 0$. Thus, the grouting intensity is not at its greatest but at zero (Figs. 8.1 and 8.2):

$$\mathrm{GIN} = \mathrm{P} \times \mathrm{V} = \mathrm{P}_{crit} \times 0 = 0$$

However—and as mentioned before—the grouting intensity changes with the beginning of the intrusion and differs from case to case.

### *8.2.4 GIN and Grouting Pressure in Shallow Zones*

Lombardi stated that "*The situation is quite different in using the GIN method as at shallow depth the GIN value, and not the maximum pressure, is governing the maximum grouting process; therefore the limiting pressure can be chosen at quite high a level, even at ground level*". The contrary applies: The maximum pressure achievable determines fracturing and the beginning of the intrusion of grout.

We experienced frequently that tight bedding planes have been fractured in shallow zones at very low pressures already (4 bar, for instance), which subsequently absorbed huge quantities of grout.

This corresponds exactly with the conclusion drawn above. Figure 8.3 shows a combination of very small Lugeon values and large grout takes obtained in a nearly impervious granite which illustrates the possible consequences of using too a high grouting pressure in shallow zones. Consequently, the author cannot share Lombardi's recommendation.

It enlarges enormously the risk of hydrofracturing, possibly with serious consequences such as excessive grout consumption and plugging of filter layers in

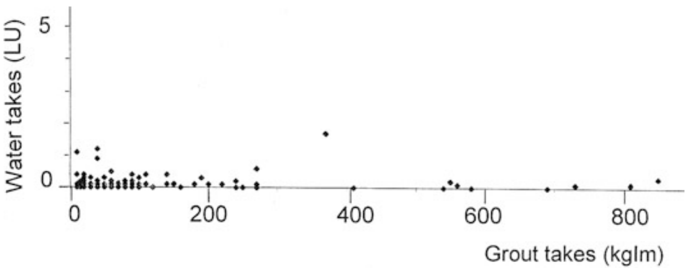



**Fig. 8.3** Even low grouting pressure provoked large grout takes due to hydrofracturing in impervious rock (Part of Fig. 3.23)

embankment dams. The conventional method of heaving a building by grouting confirms the effectiveness of hydrofracturing. In the latter case it is helpful, but in curtain grouting it produces a negative result.

### 8.2.5 Optimal Grouting Pressure Related to Depth

Lombardi points out that "*There is no doubt that the optimal grouting pressure for any grouting process depends, among other factors, on the compressive stresses in the rock mass as well as on the future water pressure*".

In competent rock it is not the compressive stress but the relationship between (testing or) grouting pressure and strength of the rock surrounding the borehole which rules hydrofracturing and hydrojacking. Rock types of low strength have low critical pressures even in very deep zones; these pressures do not increase with depth.

### 8.2.6 Optimal Grouting Pressure Related to Future Water Pressure

In this connection it is also stated that "*The grouting pressure has to be high enough in relation to the expected water pressure. It is better to hydrojack the rock at the time of grouting, than to have this occurring due to the water pressure, after impounding*".

The various types of discontinuities are not equally susceptible to hydrofracturing or hydrojacking, hence the orientation of the most susceptible set is essential and a concept to be established for the given conditions:

- The future water pressure has a compressing effect on latent horizontal planes. They will not be fractured but remain tight. The grouting pressure does not have to be related to the future water pressure.
- In contrast, vertical planes can be fractured or dilated causing enlarged seepages. A certain pre-stressed grouting using comparatively high grouting pressures might help to diminish this effect.
- If the rock mass contains horizontal latent planes prone to easy hydrofracturing and an inclined set of narrow joints, the planes may be cracked long before the grouting pressure reaches the level required to penetrate and hydrojack the joints. High grouting pressures can yield adverse results.

## 8.3 Critical Components of the GIN Principle

### 8.3.1 Grouting Intensity and GIN Limiting Curve

GIN means that the "*limiting value for the grouting pressure is not constant, but is progressively decreasing in function compared to the volume of the grout*". The 'Grouting Path Curve' (GPC) displayed in Fig. 8.4, here partly reproduced from Lombardi, does not indicate any hydrofracturing, so hydrojacking probably occurred only once.

Which were the negative consequences if hydrojacking would continue up to the maximum permissible pressure, disregarding both the GIN limiting curve, and the shadowed BCD-section Lombardi considered dangerous? Lombardi favors some

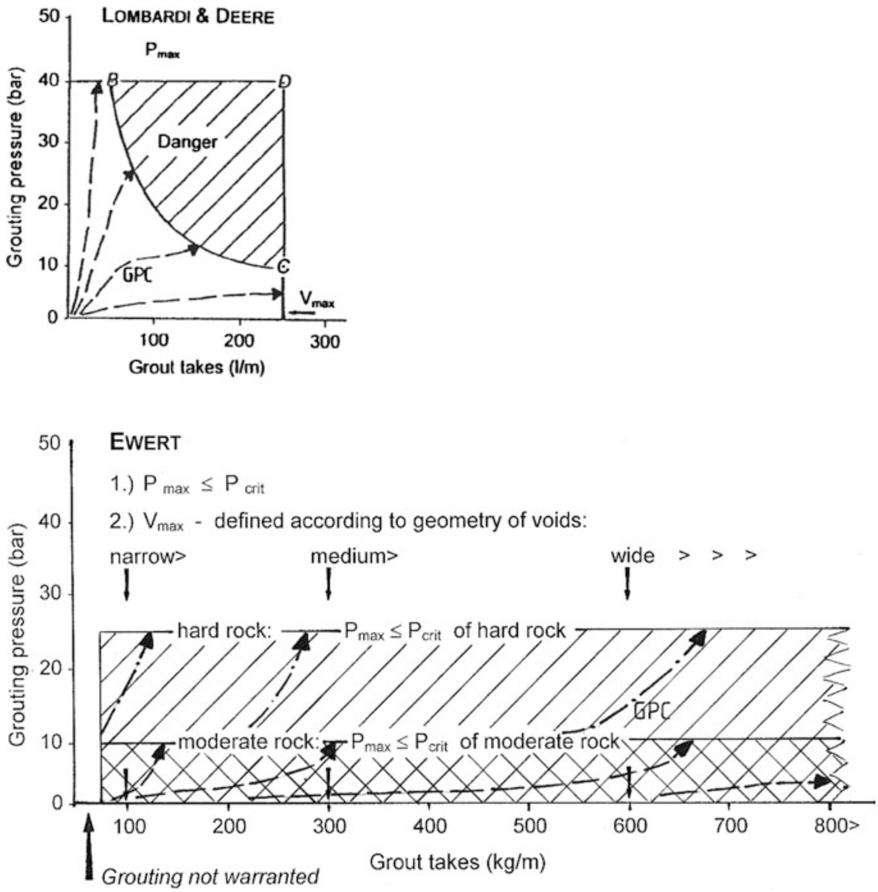



**Fig. 8.4** Comparison of grouting criteria for penetration grouting. *Top* Example for high grouting intensity of GIN = 2000, according to Lombardi & Deere. *Below* Grouting determined by individual facts: critical pressures, wide fissures absorb larger maximum grout volumes

hydrojacking since it improves the effectiveness of grouting. Why should this be dangerous now, provided $P_{max}$ does not exceed $P_{crit}$, and it does not enlarge too much the grout consumption?

On the contrary, the decreasing pressure is dangerous because

- It does not press out the bleeding water sufficiently.
- It does not provide for a tight adhesion between groutstone and rock.
- It does not remove or compress cohesion-less infillings.
- It does not create groutstone layers extended enough for large hydraulic gradients.

The decisive factors presented in Fig. 8.4 are not GIN, the grouting intensity number, but the geometry of the water paths along the voids—closed, partly or completely open—and the deformability of the rock along the borehole section treated—weight of the overlying rock due to disconnected rock bond, or strength of solid rock. Both, state and strength of the rock determine the size of the critical pressure—and hereby the amount of grout required to reach the maximum grouting pressure.

Using the same grouting intensity is the opposite of applying individual concepts as grouting practitioners prefer. They are convinced that different fissures need individual grouting intensities to be sealed tightly. More efforts are required to grout an open fissure of a 10 m wide extension rather than a short narrow joint. The latter one may be tightly filled with several tens of kg per meter of cement already, while a wide fissure absorbs possibly more than thousand kg per meter, sometimes even pressure-less. The few practical examples shown in Fig. 6.15 demonstrate the individuality of ruling conditions determining the grouting processes. Each rock around the borehole section treated has its own setting of influencing factors—and yields individual results.

Allegedly, GIN "*…reduces the risk of hydrofracturing*" and "*…provides additional flexibility in favor of the designer. They can, for example, increase the pressure, or the take, or both limits…..as long as the GIN-value is maintained*". It has been demonstrated already that GIN enlarges the risk of hydrofracturing. The basic formula $GIN = P \times V = const$ presupposes that either P or V can be increased—but not both.

It must be concluded that the GIN limiting curve exposes narrow joints to very high pressures implying the risk of overdoing by fracturing and—vice versa—leaves wide fissures insufficiently grouted. The first deficiency is only expensive; the second one can be dangerous.

### 8.3.2 *Definition of Grouting Pressure and Volume of Grout*

It is tradition to set a pressure limit for all grouthole stages belonging to the same rock zone and to stop grouting once this permissible pressure is reached. Such a pressure should be applied irrespective of the grout takes. This is a reasonable approach provided the maximum permissible pressure is defined in accordance with critical pressures: soft rock types permit only low grouting pressures; solid rock types enable high grouting pressures. Empirical research yielded an appropriate range between 5 and 30 bar. These grouting pressures are much smaller than those proposed by Lombardi & Deere which range between 15 and 50 bar, respectively. Since they are higher than most critical pressures, GIN in general favors hydrofracturing.

It is also custom to define individually the maximum volume of grout. It has been substantiated already that even the largest one proposed by Lombardi & Deere —300 1/m—is far too small. If wide fissures are grouted, much larger grout takes are needed to fill them, thereby forcing the pressure to increase. This is not always achievable: sometimes even very large takes—say 1000 kg/m—are not sufficient to reach the permissible pressure. In such a case additional groutholes are needed.

### 8.3.3 Water Pressure Tests and Groutability

Lombardi stated: "*Indeed, the actual groutability can be defined only by grouting tests and not by any water pressure test*". "*Abandoning the traditional water pressure tests*" is therefore a core aspects of the GIN method.

The grout takes alone do not characterize the groutability:

- Small grout takes seemingly indicate a low permeability. This is only true in combination with small Lugeon values. Very narrow joints are not groutable although a multitude of such joints causes a considerable permeability. The combination of large Lugeon values and small grout takes characterizes a permeable but ungroutable rock.
- Large grout takes do not necessarily indicate a permeable rock. Intense hydrofracturing frequently provokes large takes, irrespective of any permeability (Fig. 2.2).
- Large Lugeon values combined with large grout takes prove the rock permeable and groutable while small Lugeon values and large grout takes indicate a rock of little permeability but susceptible to fracturing.

Both WPT's and test grouting, are required to investigate the groutability. Moreover, WPT's became a useful tool to examine the hydrofracturing behavior and to define the appropriate grouting pressure. Thus, WPT's should not be abandoned: together with test grouting; they are very useful to determine the groutability.

## 8.4 Final Remarks

Within the frame of this presentation further projects cannot be dealt with. Lombardi & Deere concluded that the GIN principle has been successfully used in several projects. This, however, has not been confirmed. In some of those projects, where data evaluation was possible, it appeared that a grout curtain is either not effective or should not be installed.

The following criteria serve as an acknowledged basis in proving a grouting program successful:

- The grout takes of subsequent series decrease progressively.
- Groutstone layers are mostly deposited in originally open voids.
- WPT-values obtained after grouting are smaller.
- The uplift reduction across the curtain is over-proportionate.

Gaining this information requires a statistical evaluation (mean values, frequency distribution, distribution related to depth), controlling core drillings and piezometers located at either side of the curtain.

### *8.4.1 The GIN-Principle*

The GIN-principle introduced several decades ago was also a standardized concept. The decision for a grouting program might still be based on the Lugeon Criterion, however, during the execution of the work $P_{max}$ and $V_{max}$ serve as the decisive parameters. Meanwhile, specific research has shown that this concept does not improve grouting but, in reality signifies a setback: Whenever a rock is quite permeable due to wide fissures GIN implies the risk of an unreliable sealing because the quantities proposed for $V_{max}$ are much too small. In opposite cases, GIN causes considerable overdoing with a higher risk of hydrofracturing and its negative consequences because the pressures proposed for $P_{max}$ are by far much too high.

### *8.4.2 Concept of 'Grouting Guided by Facts'*

Standardized criteria take in consideration only one or, at best, a few fixed parameters but do not pay attention to all the other facts differing from site to site. Thus, a treatment using this concept is related in most cases to the same basis which, for safety reasons, is quite stringent. In case of good conditions this concept leads to considerable overdoing while an actual improvement is not necessarily achieved. Consequently, standardized criteria are not the adequate method to deal with widely differing settings.

In reality we are facing an endless variety of different facts: each site has its own geological setting causing its own groutability, its own hydrogeological regime, causing its own permeability of the entire foundation and its own type of dam providing for a specific hydraulic gradient. Considering the inappropriateness of standardized concepts and criteria we should prefer a concept in which the given facts guide the decision on a grouting program, its design and its execution.

The concept of '*grouting guided by given facts*' has been successfully applied in several projects already, although this very motto has not yet been used. In view of the limited space of this presentation the details of this concept cannot be outlined again. Most aspects have been discussed while presenting the grouting examples.

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