

Sealing of Massive Water Inflows through Karst by Grouting: Principles and Practice

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ABSTRACT

Dams and quarries sited in karstic limestone may often experience a sudden and severe increase in underflow or inflow as a result of washout of soft karstic residue. This paper provides a step-by-step guide as to how to approach dealing with such a potentially catastrophic event. Guidance is given on certain basic construction issues, and summaries of three recent case histories are provided to illustrate the approach.

1. INTRODUCTION

Dams must often be founded on karst, and limestone quarries invariably encounter karstic features. Both dams and quarries create relatively abrupt and unnatural hydraulic gradients; a dam creates a gradient by virtue of the height of water it impounds, whereas a quarry creates a hydraulic gradient by forming an open space beneath the natural piezometric surface. Whereas it is typical to install a grout curtain under a new dam, such permeation or fissure grouting operations cannot hope to comprehensively treat a karstic rock mass to a degree that seepage under long term service conditions may not – eventually – result in channels being opened through features in the karst filled with residual clay or other erosional or weathering materials. This long term deterioration is superimposed on any short term disturbance to the karstic terrain actually created by the activities involved in constructing dams or mining quarries, such as blasting, excavation, and the alteration of piezometric conditions. Grout curtains in virgin karst thus have a finite effective life – the length of which depends on the rock mass characteristics, the intensity and quality of any grouting conducted, and the prevailing hydraulic gradients. Unfortunately, this life cannot be precisely predicted.

Deep quarries in limestone terrains are, ideally, located and engineered to avoid known karstic horizons, but such foresight and precision is often not attainable in

construction practice and therefore it is typical for aggregate suppliers to quarry – with the best environmental intents and technical practices – and to assume that they will have to pump out water infiltration to some extent, at some time. Given the highly competitive economics of the aggregate industry, it is not feasible to think that a quarry owner will invest in a potentially massive and sophisticated preemptive grout curtain around its property if it is much more economical and cost effective short term for that owner to simply pump out the inflow which can reasonably be expected or calculated to occur during the – hopefully – long service life of the investment. In this regard, some property owners have mineral extraction rights extending several hundreds of years into the future. It may also be observed that for obvious reasons quarries are often in juxtaposition to major transportation arteries such as rivers (which define the hydraulic head which may subsequently be exerted on the quarry floor) or railroads (which may have been created to convey the mined product). Especially in the Ordovician terrain east of the Mississippi River, such quarries may well be in close proximity to urban areas, including major highways. The potential exists, therefore, for activities *in* the quarries to affect structures *outside* of the quarries.

The potential for significant problems to arise relating to technical challenges, operational cost effectiveness, and potential damage to the surrounding environment, is even more acute in the case of existing dams built on karstic terrain. Although it is conventional wisdom to state that dams in the U.S. were invariably built on “good” sites, since the country was so large, and engineers always had the “walk away” solution of relocating the structure elsewhere, this view can be quickly discredited. A significant percentage of the United States’ large dams – identified in a 2002 study by *Hydropower and Dams* as being 6724 in number, were – had to be – founded on sites with less than perfect geology. The magnificent vision of the Tennessee Valley Authority could not have been realized if an embargo had been placed on sites with limestone bedrock. Construction of the great U.S. Army Corps of Engineers’ and private utility structures of later decades in Indiana, Tennessee, Missouri, Georgia, and Alabama in particular would also have been denied if fears over karstic response had overridden the contemporary social and economic needs of the community.

An unexpected massive and sudden inflow into a deep, operational quarry – apart from potentially endangering operators’ lives – could render the quarry unworkable (due to submergence of its assets), and/or unprofitable (due to costs of pumping). Such inflows could also have the potential to alter the local groundwater conditions and so possibly accelerate or trigger sinkhole activity within or beyond the property’s boundaries. Massive inflow through karstic features under an existing dam could well create a dam safety situation (if the overlying or adjacent structures were adversely affected) or could cause severe financial consequence if lake levels could not be maintained: in this case power generation, flood regulation, and/or recreational impacts would be felt.

The problem of providing a long term security to dams on karst has been assiduously addressed by many Federal and private owners for over 80 years. In particular, in

recent years, major rehabilitations have been funded to a number of large and vital existing structures owned by the U.S. Army Corps of Engineers, including Beaver Dam, AR; Walter F. George Dam, GA; and Mississenewa Dam, IN. All have been protected by “positive” concrete cut off walls – overlapping large diameter piles in the case of Beaver Dam (Bruce and Dugnani, 1996), diaphragm walls in the latter instances. A notable exception to this pattern has been the repair of the foundations of Patoka Lake Dam, IN, where a relatively innovative grout curtain (Dreese et al., 2003) was selected on overwhelming cost reasons over a concrete wall. Similarly, the recent karst related seepage problem of a major TVA structure was also resolved by the use of contemporary grouting principles (Bruce et al., 1998). Dams, founded on bedrock containing potentially erodible material, can develop an increasingly severe and sudden problem at any time in their useful life. Bruce and Gillon (2003) describe the performance of a 70-year old dam in New Zealand where the recent erosion of clay infilling in fissures resulted in the need for a very precise remedial grouting operation. It is also known that certain dams in Ordovician terrain south of the Mason-Dixon line are recording massive under seepage which is either being “managed”, or is being closely monitored with the confidence that the seepage constitutes no imminent threat to life, structure, or profitability. In contrast, and for wholly understandable and rational reasons, major quarry owners have been more reactive than proactive, in the sense that major investments in water cut offs have been initiated only when a major “break in” has occurred (Bruce et al., 2001) or when overwhelming environmental pressure has demanded it.

This paper addresses the actions that may be taken when the particular flow velocity and/or volume reaches a level that simply demands action must be taken. Such interludes are typically highly stressful for all parties, especially given the consequences of “failure”. They invariably present a technical scenario which is extremely challenging to resolve.

2. FUNDAMENTAL ELEMENTS OF SOLUTIONS TO CATASTROPHIC FLOWS

As noted above, long term seepage control in dams is increasingly being provided by installing concrete cut offs. Such a task requires intense engineering, sophisticated construction, and high levels of funding. It may take many years for such a remediation process to be completed. This paper focuses instead on the short term response to emergency conditions which can be afforded by grouting. The following 8-step sequence reflects the three fundamental stages in the implementation of a successful remedial grouting operation:

- Exploration and situation assessment.
- Responsive execution.
- Verification of performance.

Sudden, significant and obvious changes to the preexisting structural and hydrological regimes characterize a karst related flow event. Flow or seepage rates

may increase substantially – by an order of magnitude or more, the flow may be discolored, new seepage entry and exit points develop (e.g., “eddies” and “boils”), piezometric surfaces drop, and/or surface manifestations will occur in the form of depressions in embankments and sinkholes in overlying overburden.

At such times, normal facility operations are interrupted or suspended, and depending on the severity of the situation, a fundamental structural safety issue may be declared and a wide range of technical, operational, managerial, financial, and statutory bodies may become involved. Time will be of the essence in order that resolution is achieved as quickly and cost effectively as possible, and that any safety-related issue is correctly and firmly managed. The following steps reflect the approach the author has adopted over the course of several such events.

- Step 1. Appoint a Project Manager to act as a coordinator of the short term emergency and the subsequent longer term remediation efforts. This Manager should be from the ranks of the facility owner, and should have long and direct experience with the construction and operation of the site and with the modus operandi of the ownership. The Manager should be divorced from his prior routine duties as far as possible, and should be fully empowered to seek further assistance, both from internal resources and external consultants. A separate “mission control” room should be established for his use, wherein all data are collected and analyzed and all technical meetings are held. Every meeting should be formally minuted.
- Step 2. Evaluate exactly what the situation is, via analysis of all available data sources, but at this time paying special attention to memorializing verbal accounts from actual witnesses. Such accounts can be of great benefit in subsequent analysis, but their value depends on their accuracy and completeness, both of which will rapidly recede with time.
- Step 3. Implement all necessary short term measures which legally, administratively, or practically have to be taken. From the technical viewpoint, this may include installing additional, simple instrumentation (to help quantify the issue, e.g., structural movement monitoring, flow measurements); increasing the frequency of reading existing instrumentation; site inspection; relocating equipment that is threatened by inundation; installing extra pumping capacity; or even (in the case of a dam) quick reduction in reservoir level. These actions help to create a baseline, mitigate the impact, identify if the situation is deteriorating further, and so help the Project Manager determine the level of imminent danger.
- Step 4. Design and conduct a focused program of site investigation, the purpose of which will be to establish the exact path of the flow (typically it is in a massive conduit as opposed to in a widely dissipated “delta”), its rate and velocity, and the nature of the rock around the conduit. (If the conduit is

found to be in a zone surrounded by other clay-filled karstic features which have not, as yet, been “flushed out”, this will represent a severe problem during subsequent remediation and service as illustrated in Section 4 below.) This study will permit a remedial design to be conducted, and priced. It will also highlight if the flow has the potential to create further distress to overlying or adjacent structures. During this time, the reading instrumentation schedule of Step 3 is maintained.

The site investigation should comprise the following two tasks, which are complimentary:

- Desk study: review all relevant construction records; historical performance data; instrumentation data; regional, local, and site geology; climatic and seismic records; aerial photographs; personal recollections; and published technical papers.
- Field study: install investigation holes by the fastest and most economical method to try to physically locate the conduit. This should be done as far “upstream” as possible. These holes can then be instrumented to provide ongoing data on groundwater levels, chemistry, temperature and pH, or can be used for various types of geophysical testing, e.g., seismic tomography, or can in fact be used as grout holes in the subsequent remediation. Other types of geophysical testing such as Ground Penetrating Radar, Spontaneous Potential, Electrical Resistivity (Dipole-Dipole or Wenner Schlumberger), and magnetic or gravimetric surveys can be conducted. Dye testing if properly and thoughtfully conducted, can be extremely useful (Bruce and Gillon, 2003).

It may happen that despite the best of efforts and intentions, the exact source or path of the flow cannot quickly be determined with accuracy. Perseverance is essential: the subsequent steps should not be commenced until closure on Step 4 is satisfactorily concluded.

- Step 5. Assuming the situation is to be positively rectified, as opposed to merely being monitored and/or managed by other means (e.g., ongoing pumping from the quarry floor), the Project Manager and his advisors develop the design for remediation. At this stage, input from specialty contractors and other specialists should be sought, and the technical literature reviewed for case histories of similar nature. It is essential that the design clearly identifies the “measure of success” of the project in terms of, for example, the residual flow rate. It is common to find that few contractors will have faced such a severe problem before, and unfortunately, most will tend to underestimate the difficulty of the remediation. Considerable amounts of time and money have been lost by initially employing local contractors in haste, using simple and conventional methods which are later proved to be wholly inadequate. It is also usually the case that such contractors have

been hired on a “cost plus” or “time and materials” basis and so may not be highly motivated to achieve a quick and definitive solution, even if they did possess the technological resources.

- Step 6. With the design and budget approved, the contractor is hired. This should be done on the basis of “Best value” as opposed to “Low bid”, although the two are occasionally the same. Emphasis should be placed on the experience, expertise, and work plan of the Contractor, as opposed to his price. Engaging the “wrong” contractor will certainly lead to disappointment and dispute over schedule, performance, and cost, and indeed inappropriate construction methods may worsen the situation and make further remediation attempts even more challenging.
- Step 7. Execute the work. During this phase, all data relating to the contractor’s operations (e.g., drilling, water testing, and grouting records, and progress) and to the impact on the overall structure/bedrock system (e.g., flow characteristics, piezometric levels, structural movements, changes in groundwater chemistry, temperature, etc.) must be collected and studied in real time by the Project Manager and his team, in “mission control”. Only in this responsive, integrated fashion can the effect and effectiveness of the work be revealed progressively, and a sound engineering basis created upon which to instruct changes to the program if required (e.g., need for additional or deeper holes; different grout mixes; etc.). Such data are also invaluable in the ongoing process of reevaluating the soundness of the design (Step 5). This step is in place until the remediation has been completed and a short term (e.g., 7 days) confirmation period has successfully elapsed. A fully comprehensive “as built” report covering all the relevant data from Steps 1 through 7 should be prepared as soon after the remediation as practical.
- Step 8. Long term monitoring. Many – if not all – the piezometers and other monitoring devices installed beforehand should still be functional at this point. The Project Manager must establish a regular schedule for reading these instrumentation sources, analyzing their data, and for conducting any relevant revised site or structural inspections. A database must be established, together with a well defined series of protocols to follow if certain instrumentation triggering and threshold levels are reached, or if any significant flow or pressure aberrations should reoccur. These protocols should include details of the responsible person(s) to be notified, and appropriate emergency response plans.

It must be stated that the most effective a grout curtain in karst will ever be is immediately after its construction. In service, as the full hydraulic gradient is being placed on the curtain (i.e., the lake level is restored, or the quarry is pumped dry) pockets of ungrouted and/or ungroutable weathered material will be exposed to pressures which may prove sufficient, over time, to cause such pockets to “blow out”.

This will occur despite the very best efforts of the design and construction teams. However, there is no predictive capacity as to how severe this increase in residual permeability will be, or how fast it will develop. Clearly, such deterioration will depend on the nature of the karst (i.e., how much erodible material remains), the applied hydraulic gradient and the length of time over which it acts. Illustration of this is found in Section 4, below.

3. BASIC CONSTRUCTION CONSIDERATIONS FOR GROUDED CUTOFFS

Definition of the Measure of Success. Pragmatically, a restoration of the condition status quo ante is a sensible goal. Occasionally, betterment can be achieved, but often it is found not cost effective or even necessary to attempt such relative improvement. In addition to clearly stating what the post treatment, residual flow should be, other project specific goals, if applicable, should be precisely set, e.g., attaining certain key piezometric levels, structural movement thresholds, longevity of the curtain and so on.

Drilling. Since much will already be known in precise geological terms about the lithology and structure of the rock mass, and since it is generally the goal only to locate and fill major conduits (as opposed to treating microfissures), the drilling should be conducted with the most cost effective method available – provided always that it is compatible with maintaining the security of overlying or adjacent structures. This usually means using a direct circulation down-the-hole hammer (Bruce, 2003), powered by compressed air which will help greatly in “cleaning out” clay from karstic features. Holes should be drilled at least 150 mm in diameter to permit the later installation of grouting-related pipework. Depending on the rock mass structure, holes may be most effectively inclined 10 to 15° off vertical. At least two rows of holes are necessary, for geological and operational reasons, with the holes in each row not spaced more than 3 m apart on centers. It is essential to log carefully the drilling conditions encountered in each hole, so that a simplified geological profile can be established, identifying, as a minimum the locations and extents of

- Overburden,
- Hard massive rock,
- Fissured rock,
- Very weathered rock,
- Clay infilled karst, and
- Voided karst.

During the drilling of each hole, the exit point of the flow must be continuously monitored to determine if the conduit has been influenced: volume and/or color changes or the presence of compressed air are critical observations. Any interconnections between holes must be accurately recorded (depth and distance).

Grouting Materials. In the cases of fast, large volume flows in very large conduits, conventional “slurry” grouts (High Mobility Grouts: HMG (Chuaqui and Bruce, 2003)), even when thoughtfully formulated, will simply be washed away, perhaps even causing an environmental problem downstream of the curtain. Similarly, the potential benefits of highly sophisticated – and expensive – chemical grouts (Bruce et al., 1997) are rarely exploitable since they lack the short term gelling and strength characteristics to mechanically resist the hydrodynamic forces in the conduit. In contrast, the author has experienced success using either Low Mobility Grouts (LMG) (Cadden et al., 2000) in lower head, low velocity conditions, and hot bitumen (together with HMG and LMG) in particularly adverse conditions. Various additives and admixtures including accelerators, antiwashout agents, viscosifiers and even polypropylene fibers are used to “tailor” both LMG and HMG grout suites to the precise project requirements.

Grout Injection and Sequencing. It is common to find all, or most, of the flow channeled into one or a small number of well defined conduits, although very soft, potentially erodible, or fissured rock conditions may still exist in the surrounding bedrock. The basic principle is to allow the flow to continue in these conduits, while treatment continues of the rock mass (through which water is not yet flowing) around the conduits. Depending on the nature of the rock mass, this “preemptive” treatment can be conducted by conventional open hole “staging methods”, or by the MPSP system (Bruce and Gallevresi, 1988) – both of which use families of HMG – or by using LMG in upstage, end of casing applications. Again, observation of the flow outlet point is essential at all times, together with an ongoing assessment of any changes to piezometers and other instrument readings. Typically little benefit in terms of flow or pressure reduction is found at this time, even though it is absolutely essential to conduct this work at this juncture (i.e., at a time when the water flow rate in this part of the final grout curtain is minimal).

The last, and most critical and dramatic phase of the grouting program is to then put the “plug” in the conduit, given that the surrounding rock mass has been “repaired” against the danger of internal erosion when the curtain is functioning. When dealing with flows of 150,000 L/min or more, and head differentials of over 30 m, cement based grouts – even those of high rheology and accelerated hydration – cannot be relied upon to resist the hydrodynamic situation in the conduit. In such extreme conditions, the use of hot bitumen, in conjunction with the simultaneous injection of HMG and/or LMG has proved to be a most reliable solution.

Bitumen has been used in projects around the world for decades, but it is only within the last few years that full engineering value has been extracted from its extraordinary potential. In short, the hot bitumen encounters the flow which quickly removes the heat from the material (injected at temperatures of 200°C and over). The material begins to gel and congeal and thus, when pumped at sufficiently high rates, will begin to overwhelm the flow. The simultaneous upstream injection of LMG or HMG causes these materials to be drawn against the cooling, but still relatively hot bitumen mass leading to a “flash set” of the cement based grouts in the conduit. This

multi-material plug continues to form as injection continues. Eventually, the conduit is (temporarily) plugged with the gradually cooling (and shrinking) bitumen plug. At this point, further rapid injection of HMG and LMG is continued to create upstream of this temporary plug, the “final” plug which will eventually resist the hydraulic gradient applied to the temporary plug. Failure to conduct sufficient HMG and LMG grouting at this time will simply ensure failure of the operation since the temporary bitumen plug will continue to cool and shrink and so permit the water to exploit the growing gap between conduit boundary and bitumen. The plugging operation is continued without interruption until completion: unless bitumen is pumped continuously down through the specially installed pipework at high temperatures, the system will “freeze” prematurely and the conduit will not be plugged.

The organization and management of the plugging operation is an exercise in detail and logic, and must involve the skills, input, and cooperation of all parties. Clear field leadership is essential.

4. RECENT EXAMPLES

4.1 Tims Ford Dam, TN (Bruce et al., 1998)

This 460-m-long embankment dam has as its right abutment a ridge largely comprising karstic limestone. Seepage through this ridge had increased steadily since first impounding in 1971 until by the mid 1990s it had reached over 29,000 L/min at full pool. A study of construction and geological data from historical records directed an intense site investigation via drill holes, permeability testing, and dye testing. This identified the depth and extent of the karstic zone. The goal was to reduce the flow to less than 4000 L/min at maximum pool.

Fortunately, the reservoir could be drawn down to a level which minimized flow, and so permitted the treatment of the mass with a suite of LMG and HMG mixes, creating a curtain through the rim about 240 m long and as deep as 37 m. The karstic bedrock was between 23 and 36 m beneath the surface. Final closure to the curtain was obtained during the period the lake had to be raised again. Over 1550 m³ of LMG and 605 m³ of HMG were injected, into a total of 250 holes.

At full pool level, the leak was recorded as having been reduced to about 950 L/min. It has remained at this rate since the end of the treatment in early 1998.

4.2 Quarry in West Virginia (Bruce et al., 2001)

An inflow of about 140,000 L/min suddenly developed into the floor of this fully operational quarry, originating in a river about 450 m away. The head differential was over 50 m. Remediation had to be undertaken since a) the quarry was an integral part of a major commercial organization, having long term aggregate supply contracts to satisfy, and b) it would have been prohibitively expensive to pump on an ongoing basis.

Desk studies were supplemented by programs of geophysical testing (fracture trace analyses, EM surveys and dipole-dipole) and exploratory drilling. These holes were sampled for water chemistry, pH, and temperature. The result was that the likely flow path was identified, being, at its most intense, over 15 m wide and at two elevations (18 to 30 m down; and as deep as 60 m). However, other karstic features, as yet not transmitting water, were found over a far larger lateral and vertical extent. Following an assessment of the viability of other options, a two-line grout curtain was designed, about 340 m long, 70 m deep, and within 20 m of the river bank.

The work was conducted in several successive phases, each driven by the analysis of the results of its predecessor. Locally, the curtain was thickened or regouted, in response to the developing picture. “Success”, i.e., the reduction to a total inflow of about 30,000 L/min, was achieved, temporarily, on several occasions, only for the integrity of the curtain to be compromised as a result of clay filled karsts being blown out under rising gradients. Eventually, however, success was achieved – inflow from the river was virtually eliminated under a differential head of 43 m. This project required the injection of 6465 m³ of HMG, 1649 m³ of LMG, and 4724 m³ of hot bitumen.

4.3 Quarry in Missouri

A virtually identical problem was encountered in Missouri three years after the project described in Section 4.2 above. The same generic approach to assessing the problem and designing and executing the solution was adopted. Extensive use was made of Electrical Resistivity and Spontaneous Potential geophysical exploration, dye testing, aerial photography and piezometric observations. The velocity of the underground flow reached about 25 m per minute. In this case, the river created a maximum differential head of about 90 m on the base of the quarry, and the maximum recorded inflow was about 120,000 L/min.

A multi-row grout curtain 77 m long was constructed to a maximum depth of 107 m. Intensive treatment of the incipient karstic features was first and systematically conducted to improve the ground around and under the location of the main conduit, found to be about 70 to 84 m down and 18 m wide. The major difference in the geology with the previous case was that the boundaries of the conduit were found to be relatively competent. As a consequence, the actual formation of the final plug – although it took several weeks to plan, organize, and prepare – took barely 48 hours. The result was total elimination of the flow and full restoration of piezometric levels upstream of the curtain. The overall curtain involved the injection into about 77 holes of approximately 1650 m³ of LMG, 2830 m³ of HMG, and 165 m³ of hot bitumen.

The relatively competent nature of the bedrock around the conduit permitted straightforward stage grouting procedures to be used with the HMG in the pretreatment phase of the operation, as opposed to the MPSP system necessary for the

similar phase of treatment in the much less competent rock mass found in the West Virginia project.

5. FINAL OBSERVATIONS

Space restrictions prevent full descriptions being given to the three case histories summarized above. The reader should be cautioned from believing that these projects were anything other than extremely stressful for all the participants, demanding the highest levels of technology, administrative, engineering, management skills, and attention to detail. There is an old adage that “you find out about people in adversity”. The development of a sudden and major flow into or under a major engineering structure founded on or in karstic limestone presents serious adversity in various forms to all concerned. It is hoped that this paper will in general provide comfort, confidence, and guidance to those who are faced with such events. In particular, it may form the basis for contingency plans or protocols that could be developed (and hopefully “left on the shelf”!) by managers of major facilities founded in karstic limestone terrain.

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