ICOLD EUROPEAN CLUB

Working Group on Uplift Pressures under Concrete Dams

FINAL REPORT

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2004

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Uplift Pressures under Concrete Dams - Final Report

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ABSTRACT: The report illustrates the results of the activities carried out by the European Working Group. Four subjects were selected by the Group: "Regulatory Rules and/or Normal Practice" adopted in different Countries; "Analysis of Measurement Data", referring to recent relevant studies; "Numerical Modelling" to compute uplift pressures; "Clearing Of Drainage Systems".

1 INTRODUCTION

The Working Group started its activities in 1995. It is composed of 10 members, representing the following Countries: Italy, France, Spain, Great Britain, Switzerland, Germany, Sweden, Austria.
Examining the different aspects of potential interest, the Group decided to concentrate its work on the following topics:

- Regulatory rules (or, in their absence, rules commonly applied in normal practice) adopted in different Countries to take into account uplift pressures in dam design and safety assessments.
- Analysis and assessment of measured uplift pressures at a significant number of existing dams, looking for information and indications about the influence of the various factors affecting uplift pressures.
- Numerical modelling for the evaluation of uplift pressures in the dam body and foundation, discussing the capabilities, the limits and the difficulties in the use of the available numerical approaches, and highlighting interesting results of recent research studies.
- Techniques for clearing drainage systems, making reference to the experience of large dam owners and to the information available in technical literature.

2 REGULATORY RULES

Through the co-operation of experts from different Countries, information was collected about regulatory rules adopted in different Countries to take into account the uplift pressures in dam safety assessments.
When directly available, ancillary information on related aspects (such as drainage systems, uplift monitoring, etc.) was also collected.
Some Countries have no regulatory rules specifically addressed to uplift pressures. In these cases, information was sought about rules commonly applied in “normal practice”.
An exhaustive inventory of available regulatory requirements or applied practices was beyond the scope of work of the Group; instead it was aimed to gather sufficient information to enable useful comparisons to be made and to evaluate the compatibility of the different approaches.
The investigation was restricted to European Countries, as shown in the following table: (RR: Regulatory Rules; NP: Normal Practice):

- Italy	RR
- Spain	RR
- Portugal	RR
- Germany	RR
- Norway	RR
- Finland	RR
- Great Britain	NP
- France	NP
- Switzerland	NP
- Sweden	NP
- Austria	NP

A full report on the information gathered for each Country is given in Appendix 1.
Some synthetic comments are given below.

As far as regulatory rules are concerned, it should be noted first that their degree of detail is variable from Country to Country, but in most cases they are not very definite and rigid.

In some cases (Spain) there is only an indication to take uplift pressures into consideration, without any further directive or constraint.

In general the more detailed guidance relates to gravity, hollow-gravity and buttress dams. For other dam types, uplift pressures are not among the load factors to be considered or are left completely as the designer's responsibility.

The most specific rules can be found in the Italian Regulations (Technical Rules, 1982), where a linear or bi-linear distribution of uplift pressure is prescribed (for dams without or with drainage system), with headwater and tailwater pressures at the dam heel and toe, and with a maximum allowed reduction of uplift at the drainage line (this maximum allowed reduction corresponds to the reduction commonly adopted in normal practice). The uplift reduction can only be adopted if the diameter and the spacing of the drains comply with regulatory limit values.

In other Regulations (Germany, Portugal) analyses by means of numerical hydraulic models are envisaged, at least for the foundations of major dams.

In some cases (Spain, Portugal) an abnormal increase of uplift pressures should be evaluated, in addition to normal operating conditions.

In the most recent Regulations (Spain, 1996; Portugal, 1993) the word "uplift" is replaced by the more general term "pore pressures". In Portuguese Regulations a study of the mechanical effects of the water in terms of effective stresses is explicitly required.

Referring to normal practice, flow-net analyses and the use of conventional linear/bi-linear distribution are the most commonly applied approaches. Reduction factors ranging between 0.25 and 0.6 are normally adopted to take into account drain effectiveness.

3 ANALYSIS OF UPLIFT MEASUREMENT DATA

The Group recognised the value in preparing a summary of results (information, indications and, if possible, conclusions) derived from the analysis of measured uplift pressures at existing dams taking into account the effects of the main factors influencing the uplift pressure distribution.

Among the influencing factors, the following were included: foundation characteristics; effects of grout curtains; drains and other methods to control and limit uplift pressures; response of uplift to headwater variations; uplift in exceptional loading conditions (flood, earthquake).

It was also interesting to investigate the correlation between estimated uplift pressures using currently accepted methods and actual measured uplift pressures.

The Group considered it appropriate to concentrate the analysis of this subject on gravity dams.

Through information available to the members of the Group, a literature survey and personal contacts, important recent studies on this subject were identified. These studies were carried out by EDF, the Swiss National Committee of Large Dams, and EPRI (USA) who promoted two studies.

Two studies (EDF, EPRI) were specifically addressed to gravity dams. The other two (Swiss National Committee, EPRI) also included other types of dams, but most of the results were still relevant to gravity dams.

The Group reviewed such studies highlighting the findings considered of main interest.

The common motivation behind all these studies is the acknowledgement that design assumptions about the effect of drains, grout curtains, cut-offs, and other methods of controlling and limiting uplift pressures, have never been fully validated.

This becomes of particular interest in the safety re-assessment of existing dams, where many questions and differences of opinion arise as to uplift assumptions. Many dams would require modification to meet updated safety standards. A better understanding of the interaction of structural features and uplift pressure distribution can contribute to avoid unnecessary modifications.

3.1 *EXAMINED STUDIES*

3.1.1 *Study carried out by EDF (France)*

This study (Ref. 1), concluded in 1995, examined the uplift pressures measured in the foundation of 31 EDF gravity dams (260 measurement points, in total).

The effects of the drainage systems, grout curtains, stress levels and headwater variations on uplift pressures in the foundation and at the dam-foundation interface were investigated. The chronological records of the measured uplift pressures were processed by means of statistical techniques (multiple linear regressive analyses) to distinguish the components associated with different external factors (hydrostatic load, ambient temperature, time), and dimensionless/normalised coefficients were used to analyse and compare the measurement data.

3.1.2 *Study carried out by the Swiss Committee of Large Dams*

This study (Ref. 2) was carried out by a working group of the Swiss National Committee set up in 1986, and was concluded in 1992.
Approximately 70 dams (38 arch dams, 25 gravity dams, 3 arch-gravity dams, 4 buttress dams) were investigated with respect to geology and foundation treatment, examining the measurement data that were available for about 70% of the investigated dams.
The study was expanded to include theoretical principles and details about measurement techniques used in Switzerland.

3.1.3 *Study carried out by EPRI (USA)*

The Project "Uplift Pressures Under Concrete Dams" (Ref. 3) was promoted by EPRI to determine if existing records of uplift pressure readings could provide a reasonable basis for evaluating current analytical methods of estimating uplift pressure distribution, and to examine the influence of several factors on uplift pressures.
Among the factors of interest were: influence of dam foundation; effectiveness and reliability of grout curtains and drainage systems; uplift pressures within the dam; effects of rapid changes in head water or tailwater levels; uplift pressures in exceptional loading conditions.
The study examined a large amount of records of uplift pressure data at existing concrete dams. To this aim, a comprehensive questionnaire was prepared and sent to more than 100 organisations, in USA and foreign countries, obtaining responses from 63 of them.
Foreign contacts were primarily made through the various ICOLD National Committees. Foreign responses therefore represented a number of organisations within those countries.
Many agencies, companies, states, municipalities and other organisations contributed to the project. The study also had access to data and findings produced by similar research studies undertaken by the US Army Corps of Engineers and Edison Electric Institute.
Altogether, data were collected for 225 dams. The Project also reviewed a considerable amount of published data regarding uplift at existing dams.
Adequate information for analysis and interpretation was obtained for 148 dams (89 US dams, 59 foreign dams), of which 130 were gravity dams. Consequently, the conclusions and recommendations were limited to concrete gravity dams on rock foundations.
Following the preliminary review of data for all dams, the project was split into the following phases:

- Classification of dams which provided good data for the aim of the study;
- Detailed study of the data for each dam and each parameter;
- Study of the interrelationships of multiple parameters;
- Development of trends, conclusions, recommendations.

Much of the site data provided were incomplete because construction and foundation information was inadequate. As a result, the study was unable to validate assumptions about uplift with a high degree of confidence. However, it was possible to arrive at some interesting conclusions to direct the path of subsequent research.

3.1.4 *Study carried out by EPRI(USA)*

The Project "Uplift Pressures, Shear Strengths and Tensile Strengths for Stability Analysis of Concrete Gravity Dams" (Ref. 4) was developed in 1989-1992, after the conclusion of the previous EPRI Project, to examine some aspects of the subject in more detail.
In addition to uplift pressures, the Project also aimed to establish ranges of shear and tensile strengths and cohesion values for concrete-to-rock interfaces.
As far as uplift pressures are concerned the objectives of the three-year study were the following:

- Evaluate geological conditions, foundation treatment, and foundation drainage with respect to their influence on uplift;
- Evaluate drain clearing methods;
- Develop a rational approach for extrapolating measured uplift to design flood levels.

A comprehensive study of uplift pressures at existing gravity dams was undertaken to meet these objectives. Data from over 150 gravity dams was reviewed and 17 well-instrumented host dams were selected.
The selected dams were built between 1912 and 1974 and ranged from 30 to 170 m in height. A variety of sedimentary, metamorphic, and igneous rock foundations were represented.

3.2 *Main Results*

3.2.1 *Foundation Geology*

For the studies, particularly for those working on a large number of dams, it was difficult to obtain enough detailed data to examine in detail this aspect. Such difficulty is understandable considering that, as discussed by Terzaghi as early as 1925, “minor geological details” (defined as “features that can be predicted neither from the results of careful investigations of a dam site nor by means of a reasonable amount of test boring”) can have a critical impact on uplift pressures.
Case studies examined in Study 3.1.4 confirmed that the uplift pressures are controlled by the rock mass discontinuities, that are several orders of magnitude more permeable than the intact rock. Therefore in Study 3.1.4, after a brief discussion about the predictable types of discontinuities in different foundation rock-types, it was demonstrated by means of suitable examples that uplift pressures are mainly influenced by the following factors: the variability of the joint apertures and the degree of interconnection of the joints in a joint network; the different permeability along or across shear zones or faults (the material of the central core of a shear zone is usually relatively impervious so that flow perpendicular to the shear zone is restricted; the zones of broken rock on either side are permeable so that water can flow freely parallel to the shear zone).
Specific relationships between geological features and measured uplift pressures could not be established in any of the studies reviewed.

3.2.2 *Grout Curtain*

Foundation grout curtains are installed to seal pores, joints and interstices in the foundation rock and thereby reduce seepage.
In old dams, shallow concrete walls or cut-offs were often constructed near the heel of the dam, to prevent high uplift pressure from being transmitted along any large, open joint near the surface. In modern dams, grout curtains serve the same purpose. While it is agreed that a well-constructed grout curtain can reduce the amount of seepage through a dam foundation, the influence of the curtain on uplift pressures is still a topic of debate, and this is confirmed by the results of the studies.
Most of the dams examined in the four studies had grout curtain consisting of single or multiple lines of grouting holes, with very variable depths (from 10% dam height to several times dam height).
In Study 3.1.2 a quantitative statement concerning the reducing effect of grout curtains on uplift was not possible, for the relatively modest amount of data available.
The large amount of data examined in 3.1.3 (70% of the 148 dams investigated had grout curtains) pointed out very variable situations. These ranged from excellent examples of grout curtain effectiveness to situations where the grout curtain had a negligible effect upon uplift. Significant examples were also found of initially effective grout curtains later requiring remedial work. Consequently it was concluded that grout curtains can be effective in reducing uplift but, in the absence of instrumentation to continuously prove that effectiveness, it is not prudent to rely upon the curtain for significant uplift reduction.
A similar conclusion can be derived also from the results presented in Study 3.1.1. The effect of grout curtain could not be identified (probably shaded by the prevailing effect of drainage), thus confirming that it was not an important effect.
Even more definite indications came from study 3.1.4. Case studies from the selected 17 host dams and from published literature showed that single line grout curtains have no significant effect on uplift pressures. Six of the host dams had enough data to evaluate the influence of a single line grout curtain, and in no case could a measurable effect on uplift be attributed to the grout curtain.

3.2.3 *Drainage*

The results of all the four studies confirm that drainage is the single most effective mean of reducing uplift pressure, providing a direct highly permeable path between the water bearing discontinuities and the tailwater.
In Study 3.1.2, this was ascertained qualitatively by examining the profiles of uplift pressures along the dam-foundation interface. For gravity dams, normalised uplift profiles showed relatively low dispersion and a clear break directly behind the drainage line, while for arch dams a much greater dispersion resulted and the pressure decrease had a more uniform tendency (Fig. 1). A reason for this different type of behaviour could be that for most of the gravity dams the drains were located immediately downstream of the grout curtain, while for the arch dams they were located in the downstream part of the dam.
In Study 3.1.2 a quantitative statement about the influence of drainage systems on uplift was not

judged to be meaningful, because the monitoring data were relatively sparse and not based on standard measurement principles.

In all the numerous cases examined in Studies 3.1.3 and 3.1.4, some measurable degree of drainage effectiveness was found, and in a good number of examples the installation of drains produced dramatic benefits.

A global quantitative evaluation of uplift reduction produced by drainage is provided in Study 3.1.1. An isobar contour lines map was derived from all the available data, and compared with a corresponding theoretical map computed for a perfectly drained condition (see Fig.2). From this comparison, the measured uplift pressures downstream of the drainage line were higher than the corresponding theoretical values. In this area uplift pressures of about 30% of reservoir water level were observed. Furthermore, the isobar contour-lines of the measured values propagate downstream more than the theoretical ones, and that was interpreted as a symptom of an equivalent horizontal permeability of the foundations larger than the vertical one.

The above stated conclusions apply to the most common drainage method, consisting of a line of drains, drilled from the drainage gallery. The holes are usually vertical, but they are sometimes inclined to intersect more foundation joints.

In most cases such conventional configuration was found to be effective and adequate (Studies 3.1.1-3.1.4). At times, however, the drains did not intersect all the geological discontinuities. Localised regions of high uplift pressures could be reduced by drilling drains specifically designed to intersect the discontinuities causing the high pressures.

Different conclusions can be derived for other types of drains, such as "box drains".

Longitudinal box drains, sometimes arranged in several rows, were sometimes used in old dams to drain the concrete-rock interface area. They were constructed by laying a line of half-round culverts or similar "boxes" on the foundation rock just before the first lift of concrete was poured, and they were connected to tailwater for the release of the collected water. It is now known that very often the water flow is concentrated along the rock joints rather than at the concrete-rock interface and consequently box drains have only a limited ability to reduce uplift. In later constructions their use was abandoned. They are difficult or impossible to access and to keep clean and free draining. Many of them are known to have lost their effectiveness and the effectiveness of many more is unknown. Box drains at three dams were, however, inspected with a borehole TV camera, in Study 3.1.4, and they were found to be open and clean after as much as 60 years of service.

Galleries directly located on rock are also sometimes used; they are, in effect, large and truly open box drains, but, as box drains, may be not effective in reducing uplift in depth within the foundation.

Construction of a drainage tunnel in the rock beneath the dam was also reported in a few cases (Ref. 3, Ref. 5). It can be a very effective but relatively expensive method to drain the rock formations and reduce uplift pressures.

3.2.4 *Response to headwater variation*

This aspect was examined in Studies 3.1.1, 3.1.3 and 3.1.4.

Considering the whole set of results obtained in the three studies, it can be concluded that the practice of assuming that uplift pressures vary linearly with headwater is not confirmed by actual measured uplift behaviour.

All the 3 studies identified non-linear variations of uplift with headwater.

In Study 3.1.3 the examination of 35 cases with good uplift-headwater information demonstrated that increase in uplift pressure was not proportional to the rise in reservoir level, but was somewhat less. A rationale for this was found in the progressive closure of joints and other natural flow paths in the rock mass, that can be produced by increased compressive stresses produced by increased reservoir levels.

This observation would tend therefore to support the validity and logic of extrapolating uplift pressures relative to head water level as a reasonable conservative approach, at least up to the reservoir level where no tension exists at the dam heel.

Study 3.1.4 also concluded that uplift pressures do not always vary linearly with changes in headwater level, but uplift data from the host dams showed non linear variations with increasing gradients of uplift pressures at increasing reservoir levels; that is, uplift pressures did not increase in the same proportion as reservoir level, but more.

This behaviour was also associated to the variations of the permeability of a dam foundation when the joints in the foundation rock deform as the reservoir level changes. This aspect was also investigated and confirmed by means of simple theoretical analyses, based on a finite element model, to compute the stresses in the foundation resulting from dead weight and hydrostatic load.

Based on the data from the host dams and the results of the theoretical analysis it appeared that only small aperture joints deform sufficiently to give rise to

non-linear uplift response. Large aperture joints will probably not deform enough under the stress changes caused by headwater variations to create noticeable non-linearity.
It also appeared that grouting may stiffen joints sufficiently to prevent tapering of joints and the resulting non-linear uplift. None of the host dams which had extensive consolidation grouting showed non-linear uplift. Dams which would be expected to have non-linear uplift would consequently be those with tight, ungrouted joints and large variations in reservoir level.
In Study 3.1.1 both increasing and decreasing gradients of uplift pressures , for increasing reservoir level, were observed. In some rare cases, both the situations were observed at the same dam.
From all these results it can be concluded that uplift pressures can exhibit significant non-linearity in their response to headwater, characterised by increasing or decreasing gradients with reservoir level, depending on how the rock mass discontinuities are influenced by the stresses induced by the dam-reservoir system.
Study 3.1.1 also investigated a possible direct relationship between measured uplift pressures and the mean state of stress induced in the foundation. The results are summarised in Fig. 3 and showed some tendency (higher uplift pressures for smaller mean stress in the foundation) but the large dispersion of the data indicated that such possible relationship is not a major factor.

3.2.5 *Rate of Uplift Response*

The rate of uplift response is also an important aspect, because it is sometimes argued that a dam may not experience high uplift during a flood because the flood will be of such short duration that uplift will not have time to respond before the reservoir level returns to normal values.
This aspect was examined in detail only in Study 3.1.4. Frequent uplift readings were taken at six of the host dams. The interval between uplift readings varied from a few minutes at two dams to a few days at the other host dams.
Without exception, the data collected and examined in Study 3.1.4 showed no significant time lag between changes in headwater level and changes in uplift pressures.
The simple conceptual model illustrated in Fig. 4 was also used to support the conclusion derived from measured data. This model demonstrated that time lag would be expected in highly deformable but relatively impervious foundations. These two requirements are contradictory and it is therefore unlikely that significant time lag exists in rock foundations.
Reviewing some of the occurrences of time lag reported in literature and in Study 3.1.3, Study 3.1.4 pointed out that they do not correspond to actual time lag in the uplift response, but they can probably be attributed to a misinterpretation of variations in uplift due to seasonal temperature variations or to the delayed response of open standpipe piezometers.
Open standpipe piezometers require that the water flows into the pipe raising the elevation of the water surface before an increase in pressure is registered. The time required for this flow depends on the permeability of the foundation and the magnitude of the pressure change, and it can result in the illusion of a time lag.
For this reason open standpipes are not suitable for monitoring the effects of rapid changes in reservoir elevation at dams with low permeability foundations.

3.2.6 *Seasonal Uplift Variations*

The expansion and contraction of the concrete, resulting from seasonal air temperature variations, change the load distribution on the foundation and can consequently change the joint aperture and the uplift pressure distribution.
This aspect was investigated in detail in the Study 3.1.4 by considering examples from published literature and data from the host dams and by theoretical finite elements analyses. The theoretical analyses showed that in winter the vertical stresses near the heel is less compressive than in summer and the load that was originally at the heel is transferred downstream. As a result, the foundation behaves like a tapered joint and the uplift pressures increase.
The analysis of uplift data also confirmed that seasonal temperature variations can significantly influence uplift, with higher uplift pressures during cold weather. Temperature changes can also influence the degree of non-linearity of the uplift response to headwater fluctuations.
In some cases the variation in uplift pressure due to temperature changes can combine with the variation due to changes in reservoir level in a way which, if not recognised, can be misinterpreted as time lag.

3.2.7 *Uplift in Exceptional Loading Conditions*

Uplift response to seismic activity was examined only in Study 3.1.3. The few owners that reported on this issue, reported minor or no change in uplift pressures due to earthquakes. These limited findings

support the common approach of not considering increased uplift pressures during a seismic event.
As far as uplift pressures during flood condition are concerned, the possibility and the need of referring to measured uplift pressures to estimate the uplift for design flood headwater levels resulted from the analyses carried out in Study 3.1.4.
Basing the estimate on measured pressures is essential because the actual uplift pressures can vary substantially from common assumptions used in the design, as pointed out by the previously described results.
However, it must be underlined that measured uplift pressures can exhibit high spatial variability. The pressures measured at rather close piezometers can be significantly different.
Therefore, an extensive monitoring network is necessary to derive reliable uplift values for safety assessments from measured data, considering also that for gravity dams the safety assessments have to be carried out for independent blocks.
As far as the monitoring is concerned, in addition to the number and the location of the instruments, the type of measurement devices should also be considered. In addition to the comment expressed in para.3.2.5 about the use of open standpipe piezometers, it is to be remarked that the widespread practice to measure uplift pressures temporarily capping the pressure relief holes and enabling the pressures to build up and be measured is not a good practice.
Possible variations of the measured relationship between external loads and uplift pressures must also be taken into account. In addition to possible slow and progressive variations (drifts), also the possibility of sudden variations related to the reaching of unusual or exceptional reservoir levels must be evaluated. Slow drifts can be associated to a slow variation in time of the permeabilities of the foundation (increases or decreases of the permeabilities). Sudden and strong variations can be induced by the opening of rock discontinuities when the state of stress exceeds threshold values. This latter condition is less probable for gravity dams, compared to arch-gravity dams, for the lower stress levels transmitted to the foundation.
The extrapolation of measured uplift to higher water levels must therefore be based on a comprehensive understanding of the uplift under normal operating conditions and a thorough understanding of how reservoir level, foundation geology, and drainage affect the uplift pressures.
This is necessary to derive reasonable and conservative extrapolations of the measured behaviour.

3.2.8 *Uplift in Dam Body*

The monitoring of uplift pressures within the body of concrete dam is rare. It is carried out only in exceptional cases and very little information on this subject is available in technical literature.
In the large amount of information collected in Study 3.1.3 for more than 200 dams, meaningful data on uplift pressures within the dam body was available for only five dams. In these five cases measured pressures varied widely, from about 5% up to about 50% of reservoir head. These limited findings support the usual practice of giving primary attention to uplift at the concrete-foundation interface and in the foundation.
A comprehensive monitoring of uplift pressures in the concrete of a buttress dam is reported in “San Giacomo Dam: Results derived from the improvement of the uplift monitoring” (Ref. 5). Seven automatic piezometers were installed in the concrete mass. No uplift pressures were measured by piezometers placed at a distance of a few metres from the upstream face, confirming the widespread opinion that an effective hydraulic connection with the reservoir load can rarely be established in a sound concrete.
Different solutions are used to mitigate high pressures in dam body. These include: sealing the upstream face, controlled grouting within the dam mass, installation of drains. High leakage is usually the initiating reason for taking these remedial measures.

4 TECHNIQUES FOR NUMERICAL MODELLING OF UPLIFT PRESSURE

4.1 *Introductory Notes*

In this section a summary of the applied techniques for the numerical modelling of uplift pressures is given, distinguishing between methods appropriate to professional practice and methods more in the field of highly specialised engineering services or applied research.
In the description emphasis is given to the capabilities, limits, difficulties of use, etc. of the different methods, rather than to a detailed description of the methods themselves.
The numerical modelling of the flow of water through low permeability media (rock, concrete) with discontinuity surfaces (rock joints, cracks, rock-concrete interface, lift joints, etc.) can

undoubtedly be considered a difficult task. It is generally difficult, or impossible, to have a complete knowledge of such discontinuities and of their behaviour under different loading conditions, taking into account that the water flow along each surface is affected by a combination of several factors (i.e. location, aperture, surfaces roughness, contact area, curvature, infilling materials, laminar or turbulent flow, steady or transient state, etc.).
In addition, the strong influence of the foundation treatments (grout curtains, cut-offs, drainage systems, etc.) cannot be neglected. From the numerical modelling point of view, they are “artificially induced difficulties” in a problem which is already complicated.
On the other hand the strong influence of uplift pressures on basic safety assessments, such as the sliding safety for gravity dams, is well known. The subject was debated as far as back the beginning of the century (Levy rule, Rankine criteria).
The problem of the numerical modelling was approached from the beginning using two basic schemes: the flow of water along cracks or other discontinuity surfaces, or the flow of water through the materials (considered as porous).
Starting from initial simplified approaches and “closed form” solutions, more and more comprehensive and complicated numerical modelling possibilities became available, allowing the representation in the models of an increasing number of factors influencing the actual problem. The increased completeness of the models has of course a direct correspondence with the amount of input needed.
The available numerical approaches can be ranked according to their degree of complexity:

- Uncoupled analysis of the filtration state along selected surfaces.
- Coupled fluid-structure analysis, with linear stress-strain relationship for dam and foundation materials (poro-elastic approach).
- Coupled non-linear fluid-structure analyses, where the non-linear behaviour is concentrated on selected surfaces of particular interest (concrete-rock interface, important cracks in dam body or joints in the foundation).
- Coupled non-linear fluid-structure analyses, where the non-linear behaviour is applied to the whole modelled volumes.

This is of course only one possible classification, and it must be underlined that the degree of complexity of an analysis is influenced by several complementary factors: steady or transient state, type of non linear constitutive models used to characterise hydraulic and mechanical behaviours and their coupling, etc.
In the following, comments are made on the applied numerical approaches, subdividing them in two sections:

- Methods to evaluate the uplift pressures distribution along selected discontinuity surfaces (typically: cracks or lift joints in the dam body, dam-foundation contact surface);
- Methods to evaluate the uplift pressure distribution in the whole dam body and foundation considered as permeable media, typically referring to significant dam-foundation vertical sections.

4.2 *Computation Of Uplift Pressures On Selected Surfaces*

4.2.1 *Uncoupled Analyses*

Cracks in the concrete, contact surface between dam and foundation, lift joints in the construction, etc., identify preferential surfaces for the flow of water and for the consequent development of uplift pressures.
Analytical solutions have been available for a long time by means of charts, for a laminar flow in steady state conditions along a horizontal surface interested by drainage holes (dam-foundation interface, lift joints in concrete body). The charts in Fig. 5 (Ref. 6) show the uplift pressure at the drainage line as a function of the basic geometric parameters : distance of the drains from the upstream edge, spacing and radius of the drainage holes.
A more comprehensive and detailed solution to this problem can be obtained by means of simple finite element analyses, which can be used by most engineers operating in this field. Such analyses provide the description of the uplift pressures on the whole surface. In Fig. 6 (Ref. 7) typical results of this kind of analysis are shown.

4.2.2 *Coupled fluid- structure analyses*

The methods previously described neglect the influence of the continuous deformability on uplift pressures. The uplift pressures are, in most cases, governed by the hydraulic and mechanical properties of the discontinuities (joints, cracks, etc.) and by their coupling.
Theoretical models of the water circulation along joints are available. Their application in reliable numerical schemes is however limited to those cases

where the actual problem is characterised by a limited number of important and well-known discontinuity surfaces.
In such cases the continuous media (dam and foundation) are modelled as impervious, the selected important discontinuity surfaces are explicitly modelled with “joint elements”, and the permeability of the joints can be related to the state of stress and deformation.
The permeability of the joints is usually related to the cube of the joint aperture.
The cubic law derives from the theoretical solution of the laminar flow of an incompressible viscous fluid between two parallel surfaces, where the mean flow velocities are linearly related to the hydraulic gradients “i”, and the flow rate “Q” per unit width is expressed by : Q = (gb3/12ν)i. A number of experiments has been developed to adapt the cubic law to more realistic conditions of natural joints and fractures. Extensive studies of water flow through joints in rock provided several flow laws and their ranges of validity, pointing out that, in addition to the crack aperture “b”, the roughness of the crack surface “k” control the flow. Different flow laws are summarised in Fig. 7 (Ref. 11), together with the expressions and the parameters to use to relate velocities “v” and gradients “i” (v = - kiα)
An interesting analysis of this kind is illustrated in Ref. 8. The analysis was aimed at the evaluation of uplift pressures along the dam-foundation contact surface, for three headwater levels. The computational model and some results are shown in Fig. 8. The results underlined that the variation of joint aperture has a significant influence on the uplift pressures. The computed pressures, when compared with a linear distribution, were found to be higher for the max headwater level, and smaller for lower headwater levels.
Another interesting approach is the F.E.S. model (Fissured, Elastic, Saturated Rock Mass) proposed by G Lombardi (Ref. 9, 10), which enables the analysis of the variations in pore pressures in a rock mass foundation taking into account the effective state of fissuration.
Analyses of this kind require rather complex computational software and good experience in numerical modelling, combined of course with good experience in the definition of the input parameters.

4.2.3 *Applied Research Studies*

Applied research studies were recently promoted by EPRI and carried out by the University of Colorado (Ref. 11, 12), to quantify how crack properties, drain dimensions and water head influence uplift pressure distribution within cracks in concrete dam.
An experimental tests programme was carried out and numerical analyses were performed.
A first set of experimental tests was conducted on a single artificial fracture of about 1.5x0.5 m consisting of two parallel concrete slabs with uniform spacing. The effects of various crack parameters on crack flow and permeability were investigated. A first phase of these tests validated the cubic law and a literature review pointed out that this law can be considered valid over aperture range from 0.005 mm to few centimetres. Subsequent tests quantified the effect of the small scale surface roughness, the macro-roughness and the channelling around contact areas, and analytical expressions were derived to adapt the basic cubic law to the test results.
A second set of experiments was conducted on concrete cracks created by tensile splitting, investigating the effect of normal stress acting on the fracture.
Finally, full scale flow experiments were conducted on a large physical crack model (3 x 2 m), investigating different combinations of crack entrance heads, crack apertures, drain location and drain diameter.
In parallel with the experimental tests, an “ad-hoc” finite element computer programme was developed and extensive parametric numerical analyses were conducted.
The main results of this applied research study showed that:

- A fracture experiences its greatest non-recoverable decrease in transmittivity during its first loading cycle. Cyclic loading tends to increase the fracture stiffness and cause loading-closure relationship to become increasingly elastic.
- The normal stress-transmittivity relationship is highly non-linear. The effect of an incremental change in normal stress decreases with increasing normal stress.
- A model relating the cube root of transmittivity to the logarithm of normal stress can reliably describe the hydraulic response of a fracture to changes in normal stress.
- For small crack apertures (less than 0,4 mm) all drain sizes are essentially equally effective. For larger apertures (to 1 mm) uplift pressures are reduced significantly only by drains larger than 2.5 cm in diameter.
- For drains spaced at 1,7 m, the increase of drain effectiveness with drain diameter is higher for drain diameters ranging between 2,5 and 8 cm.

Further increases in drain diameter beyond 8 cm do not change drain effectiveness very much.

- The drains become less effective under turbulent conditions. Such conditions can be induced by increased crack apertures or increased flow gradients. A model which does not allow for turbulent flow behaviour always underestimates uplift.

4.3 *Computation Of Pore Pressures In Continuous Media*

Numerical approaches derived from soil mechanics are also used to compute pore pressures, modelling the actual problem as a porous media problem.

The filtration process through porous media has well established and consolidated analysis techniques, available through many computer programmes. They can help in the study and understanding of the uplift problem, even if the representation of jointed media as porous media requires the use of rather critical equivalence criteria for the definition of reliable input data.

4.3.1 *Uncoupled Analyses*

Classical "flow-net" analyses are used to evaluate the pore pressure distribution, typically referring to significant vertical sections of the dam-foundation system, neglecting the coupling between the hydraulic and the mechanical behaviour.

Generally the concrete dam body is considered impervious in these analyses, and the determination of the flow-net is restricted to the foundation. The resulting pore pressures are then transferred, as input data, to the structural safety assessments.

Generally the analyses are carried out in steady state conditions. Transient analyses are used only for predictive or interpretative models of observed behaviours.

Non-homogeneous foundations can be modelled assigning different permeability values to different portions of the model and the commonly available computer programmes enable the modelling of orthotropic permeabilities.

These modelling capabilities do not require specialised numerical experience. The engineering experience to properly define the permeability values adopting adequate equivalence criteria is more important.

Also the presence of grout curtains and drainage systems can be introduced in these models. This requires the use of more refined modelling techniques, such as "special elements" with assigned hydraulic boundary conditions to model the drains. Correspondingly, more specialised computer programmes (or more powerful general purpose programmes) and adequate numerical modelling experience are required.

However, the critical point remains the evaluation of the permeability parameters of the materials, not in their "undisturbed state" but when subjected to the load factors generated by the dam.

4.3.2 *Coupled Analyses . Poro-elasticity and Poro-plasticity*

The poroelastic/poroplastic methods are based on the effective stress approach, where the deformability and the strength of the materials are governed by the effective stresses. Considering the very limited porosity of the concrete and rock foundations, their stiffness cannot be considered negligible compared to that of the water, as assumed in soil mechanics.

Therefore, only a certain percentage of the total stress is transformed into pore water pressures (Biot coefficient).

As in any porous media approach, key parameters are the permeability values. Different permeability values can be assigned to different areas/elements and to different directions of flow.

A further coupling between hydraulic and mechanical behaviour can be included by relating the permeability to strain state indices (such as: void index, total deformations, plastic deformations, etc.).

Interesting applications of the poro-plastic approach are reported in "Poro-plastic analysis of concrete dams and their foundations" (Ref. 13). This paper describes the analyses carried out for two existing dams (a concrete dam and a stone-masonry dam) affected by anomalous behaviour.

The dams and the foundations were modelled as porous media, with an elasto-plastic mechanical constitutive model (the cracking phenomena in the concrete were therefore represented by equivalent plastic deformations) and the permeabilities were related to the plastic deformations. The coupling between the hydraulic behaviour and the deformations was therefore determined by the arising and the propagation of the irreversible deformations.

The initial permeabilities assigned to the body of the stone-masonry dam (10-7 horizontally, 10-8 vertically) increased with plastic deformations, up to values of 10-4. In the maximum water level condition a significant cracking state resulted in the

lower upstream part of the dam, and this governed the water flow through the dam body.
The uplift pressures at the dam-foundation interface showed some variation from the linear distribution, mainly at the downstream part (Fig. 9).
Such coupled analyses are highly specialised engineering services. They require powerful and specialised computer programmes and significant skill in numerical analyses.
As a general comment on these analyses, it is believed that the full saturation hypothesis and the steady state response to applied loads are critical points. They can offer an unrealistic representation of the actual conditions in those zones that are not interested by significant plastic deformations. Very rarely the hydraulic equilibrium with the reservoir load can be established in a normal concrete in sound conditions, even for slow variation of the hydrostatic load (Ref. 14). This is only possible in particular cases, such as very poor or deteriorated concrete or old stone masonry dams with a large void index.

4.3.3 *Applied researches. Analyses of unsaturated conditions*

The introduction of the partial saturation condition in transient coupled analyses makes the numerical analyses very complex and difficult.
Powerful computer programmes and, above all, great experience in non linear numerical modelling are required to carry out properly a computation affected by convergence problems due to the strong non linearities involved. These analyses belong therefore to the applied research field.
On the other hand, they can provide useful information about the influence of aspects not represented in simpler approaches.
The results of analyses where the partial saturation condition was taken into account (for example, Ref. 15) demonstrate the high hydraulic inertia of the dam body, that is a high resistance to water flow. In the numerical analyses described in the above Ref. 15, after 40 years of constant reservoir level the phreatic surface reaches a very limited penetration in the dam body (less than 20 % at the dam base, see Fig. 10).
This is due to the low permeability of a sound concrete, and to its strong reduction with the saturation degree (Ref. 16). Partial saturation conditions in the dam body correspond therefore to extremely low permeability. Localised voids, such as the longitudinal dam tunnels, also induce the presence of an unsaturated area around them, with the above stated effects on the permeability.

5 DRAIN MAINTENANCE AND CLEANING

5.1 *Calcium Carbonate Deposits*

Drains often become partially blocked with deposits. Calcium carbonate deposits are the most common and important cause of drain clogging.
The calcium carbonate is deposited in foundation drains through a three-step chemical reaction. The calcium carbonate is dissolved from a source, transported in solution, and then redeposited in the drains. The dissolution process begins when water absorbs carbon dioxide. The carbon dioxide may be absorbed directly from the air, or it may be picked up by groundwater percolating through soil (most soil is enriched with carbon dioxide as a result of organic decay). The high water pressure at the bottom of the reservoir can cause dissolved carbon dioxide gas to combine with the water to form carbonic acid (this reaction is pressure sensitive). As more carbonic acid forms, the pH of the solution decreases (i.e., acidity increases). A solution with a pH of less than 8.2 can dissolve calcium carbonate (from dam concrete, grout curtain, foundation rock) forming calcium bicarbonate. The calcium bicarbonate is then transported in solution as the water flows into the foundation. But calcium bicarbonate is unstable and if the water pressure decrease (as it is when water flows into the drains), this causes the reaction to reverse and the dissolved calcium carbonate is deposited in the drains. The rate and magnitude of the pressure decrease affects the density of the deposits and the amount of the precipitate which forms.
The time a drain takes to block depends on a large number of parameters: soluble minerals in foundation and concrete, flow rate, pH, etc.
The hardness of a deposit can vary, from rather soft to so hard that removal can be accomplished only by drilling. Soft deposits will generally harden over time.
The character of a deposit often changes with location. Drains may be severely blocked in one portion of the dam and completely free of deposits in another. A single drain can have deposits which range from soft to hard throughout all or some of its length.
Borehole camera inspections were carried out in EPRI Study 3.1.4 (Ref. 4). They showed that drains were often free of deposits at depth. That is, calcium carbonate deposits appeared to be confined to the

upper part (some metres) of the drains, when the pressure of the water is lowest.
Drain clogging is usually detected by visual observations (qualitative indicators), and/or by the examination of uplift and leakage monitoring data (quantitative indicators); a gradual increase in uplift pressure, accompanied by a decrease in leakage from drains, is most probably an indication of drains becoming clogged.
Experiences and information available to the members of the Group pointed out that drain cleaning is seldom performed according to pre-defined maintenance program. Information collected from dam owners indicated a wide range of time intervals between drain cleaning: from "annual" to "never".
Usually more attention is given to the condition of the drains in foundation, rather than those in dam body.
Often drain cleaning is planned and carried out as part of a general maintenance project.
In many cases it is imposed by the results of a re-assessment of the safety conditions indicating the strong influence of the drainage systems on the "safety-factors" and asking for their effectiveness to be proved or for their improvement (when the existing drainage systems are not adequate). With regard to this aspect, it must be noted that rigid regulatory rules can conflict with technically reasonable approaches. This happens, for example, when the possibility of taking into account the drain effectiveness in relieving uplift pressures is by law constrained to rigid limit values of drain diameter and spacing.
The techniques in use for drain cleaning vary, depending on the seriousness of the clogging conditions. On this matter, the information collected by the Group confirmed the findings of the review carried out in EPRI Study 3.1.4 (Ref. 4).
Focusing on methods which have met some measure of success in removing carbonate deposits, they can be classified as follows.

5.1.1 *Inexpensive Techniques*

They can be performed by site personnel using readily available equipment.
They include:

- Rodding: The deposits are pierced with metal rod. After the deposits have been broken up, the drains are usually flushed with water to remove residual matter.
- Flushing: Drains are first filled with water and then flushed to remove loose deposits and free-floating particles. Water and debris are blown out of the hole using a pressure of about 0.1 kg/cm2 per metre of drain depth. Flushing is the preferred method for removing iron bacteria deposits. Neither increases in flow nor decreases in uplift have been associated with this method, when used on calcium carbonate deposits. In reservoirs of fairly acid waters, flushing reservoir water continuously through the box drains was found to be effective in preventing the build up of deposited materials in this particularly delicate type of drain.
- Soaking: Filling a drain with reservoir water - more acid than drain water - and letting it acting to soften and dissolve deposits. The softened deposits can then be removed by flushing. Experiences carried out in USA pointed out that further study is needed to fully determine the effectiveness of soaking. It is possible that drain soaking with reservoir water, incorporated into a routine maintenance program, could reduce the need for more expensive clearing techniques.

These inexpensive techniques can of course be used only when the deposits are not too thick or too hard, because their effectiveness is limited.

5.1.2 *Moderately expensive techniques*

They include:

- Mechanical abrasion: Mechanical abraders (rotating rods equipped with various types of abrasive heads) can provide visually cleaner drains, and some increases in drain flows have also been observed, but definite reductions in uplift have not been identified.
- High pressure water blasting: Deposits are removed by pumping pressurised water through a hose equipped with a specially designed nozzle. Water blasting uses water pressures up to 2000 kg/cm2, depending on the hardness of the deposits. It has been tested at several concrete dams but it can be considered a still experimental technique. Although routinely used in other industries, there is no common consensus at this time regarding the pressure and flow values needed for drain cleaning.

5.1.3 *Expensive techniques*

They require special equipment and skilled operators. These are appropriate for drain rehabilitation but are probably too expensive to be used for routine maintenance.

They include:

- Ultra High-Pressure Water Blasting. Ultra High-Pressure water blasting equipment consists of a high pressure pump and a hose fitted with cutting heads having typically two or four nozzles which act to direct thin streams of high-pressure water toward the wall of the drain. Both fixed and rotating cutting heads are available. It has been reported that when the rotating head is used, deposits on drain walls can be removed in only one pass through the drain, while the use of the non-rotating head resulted in “stripping” the drain walls. Ultra high-Pressure water blasting equipment is large and may not fit inside a dam gallery.
- Redrilling. This is the only method that has been clearly shown to reduce uplift. Usually drains are drilled out to their original diameter, but available data indicate that redrilling the drains to larger than original diameters is most successful. It must be considered that investigations documented in technical literature demonstrated that calcium carbonate deposits form not only on the walls of drains but can also form a bit beyond them, inside joints and fractures intersected by the drain walls. It must also be observed that in a number of dams (particularly in old dams) drain holes are rather irregular, due to problems in construction. Where the line of the drain is very irregular, it can be easier and more cost effective to redrill the drain holes in completely new locations, rather then attempting to redrill the existing ones. Experiences in the use of flexible drilling equipment were not successful where the drains were very irregular.

5.2 *Iron Bacteria Deposits*

Less commonly than calcium carbonate deposits, iron bacteria deposits can also be found in drains at concrete dams. The deposits may be confined to the drains, although they are sometimes present in the gutters and on the gallery floor.

The deposits are typically slimy or gelatinous, rust-coloured and sometimes smell of hydrogen sulphide gas.

Iron bacteria grow in iron-rich water having a pH between 6.0 and 7.6 and a conductivity between +200 and +320 mV. Iron carried out by groundwater can accumulate in the sediments at the bottom of a reservoir and may, over time, provide an environment suitable for iron bacteria growth. The iron-rich water at the bottom of the reservoir may eventually percolate through the foundation and into the foundation drains. Sometimes drains are partially lined with iron pipe which can also increase the iron concentration in the water.

The iron bacteria deposits are soft enough to be removed by washing with low pressure water.

6 REFERENCES

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THE EUROPEAN WORKING GROUP

The European Working Group on “Uplift Pressures Under Concrete Dams” is composed by the following members :

Ruggeri (Chairman)	Italy
Poupart	France
Amberg	Switzerland
Rodriguez Gonzalez	Spain
Gomez Laa (until June 1996)	Spain
Rubin de Celix (since June 1996)	Spain
Beak (until October 1997)	Great Britain
Sandilands (since October 1997)	Great Britain
Stephan (since June 1996)	Germany
Bettzieche (since June 1998)	Germany
Wiberg (since June 1998)	Sweden
Wagner (since June 1998)	Austria

APPENDIX 1

REGULATORY RULES AND NORMAL PRACTICE

ITALY

Regulations

Regulatory rules concerning uplift pressures are given in the "Dam Regulation" D.P.R. 1 Nov. 1959 n. 1363 (Regulation for the design, construction and operation of dams) and D.M. LL.PP. 24 Mar. 1982 (Technical Rules for the design and construction of dams)

Regulatory Rules For Uplift Pressures In Safety Assessments

Hydrostatic pressures have to be taken into account in the safety evaluation against sliding for gravity, buttress and multi-arch dams. The safety evaluation against sliding is based on the ratio T/N (T: resultant of forces parallel to the sliding surface, N: resultant of forces normal to sliding surface).
Uplift pressure are not considered as load factors for arch dams.

Gravity dams

For gravity dams the sliding assessment has to be executed for the base section and for any horizontal section along the dam body using the following rules for uplift pressures.
Uplift pressures distribution along a horizontal section trough the dam is assumed to vary linearly from full reservoir pressure at the upstream heel to zero or tailwater head at the downstream toe, if drains are not present.
For drains to be taken into account, they must comply with the following rules:
- spacing not greater than 2.5 meters,
- diameter not less than 200 millimetres in the foundation and not less than 120 millimetre within the dam.

When such drains are included in the dam and in the foundation, uplift pressure distribution will vary linearly from full reservoir pressure at the upstream heel to maximum pressure that can occur at drains line and, from this value, to zero or tailwater head at the downstream toe. In any case, uplift pressure at the line of the drains should not be lower than 0.35 times the difference between upstream and downstream water head plus the tailwater head.

Buttress dam

The rules for buttress dams are determined by the ratio between buttress centerline spacing and minimum buttress thickness (or the sum of thicknesses when the buttresses have internal cavities). When this ratio is between 2 and 4 along, at least, 2/3 of the height, the rules for gravity dams should be used but uplift has to be determined considering pressure acting just below the upstream head of the buttress and assuming a zero value of pressure along its downstream bound.
When the ratio is less than 2, the rules for gravity dams should be used.

Multi-arch dams

Uplift pressures in buttresses are determined using the same distribution as for gravity dams, but the extent of loaded area should have a length in upstream-downstream direction equal to twice the thickness of the buttresses.

Constructive And Surveillance Regulatory Requirements

Structural and constructive regulatory requirements are provided for galleries and drains. Within the dam, near the upstream toe and along its whole length, a gallery has to be included. The gallery has to be practicable. It should constitute the upper end for the foundation drainage and the lower end for the formed drains in the dam body.
Uplift pressures should be measured during dam operation. For this purpose piezometers should be installed along the dam galleries. A technique often followed in dams where piezometers are not installed is temporarily capping pressure relief holes and fitting a pressure meter to measure uplift. Normally these holes would be left with a valve open to prevent build up of pressure. The valves are closed from time to time enabling the pressure to build up and be measured. To avoid significant uplift forces, only a limited number of relief holes are capped, and not concurrently.

GREAT BRITAIN

Regulations

The United Kingdom has no Regulatory rules on uplift or values of drains.

Normal Practice For Uplift Pressures In Sliding Assessments

The practice is that the Construction Engineer responsible for the design and construction of the dam decides on the degree of relief which he will provide within his design and constructs the drainage system in accordance with this. The operator of the dam is responsible for ensuring that the factors of safety against uplift assumed by the designer are not exceeded.
There have been few concrete dams designed in Britain in recent years. Discussions with designers suggest that the practice on those dams which have been designed, involve carrying out flow net analysis on the base of the dam using finite element techniques to determine the uplift pressures and installing conventional cut-offs and drains.

Gravity Dams

The assumptions made of the uplift pressures acting beneath gravity dams depend on whether there is provision for pressure relief through drains.
Typical assumptions are the following.
Where there is no provision for drainage, the uplift pressure at the dam heel is assumed to be 0.66-1.0 the headwater, and a linear variation is assumed for this value to the tailwater head at the dam toe.
Where there is provision for pressure relief through drains: uplift pressure at the line of the drains may be assumed to be reduced to between 0.25 and 0.5 of the difference in pressure between the upstream and downstream faces.

Buttress Dams

In general buttress dams are not considered at risk as much as gravity dams. Any uplift pressure will be relieved to the sides of each buttress.

Measurement Of Uplift Pressures

A number of method have been adopted::
- Dipping vertical standpipes from the gallery or downstream toe, using a dip meter.
- Piezometers installed in vertical or inclined holes into the dam or foundations.
- Temporarily capping pressure relief holes and fitting a pressure meter to measure uplift. Normally these holes would be left with a valve open to prevent build up of pressure. The valves are closed from time to time enabling the pressure to build up and be measured.

S W E D E N

Regulations

There are no regulatory rules in Sweden concerning uplift pressures under concrete dams. New rules common to the members of the Swedish Power Association are currently being developed.

Normal Practice For Uplift Pressures In Sliding Assessments

Various assumptions for uplift pressures ha been used in the past. Large pressure reduction have often been made for drainage systems that have been difficult to maintain.
Today more of a common practice can be found.
The following normal practice is found in guidelines adopted by Vattenfall. The assumptions are used for sliding and overturning analysis. If design assumptions are verified, reductions in pore pressures can be made.

Gravity dams

If no drainage system exists, a linear distribution of uplift pressures is assumed between the full reservoir head (at the dam heel) and the tailwater head (at the dam toe).
Inspection galleries, which serve as pressure relief, are often incorporated in gravity dams. If drainage is drilled in such gallery, the uplift pressure at both faces of the gallery may be reduced to 30 % of the

difference between pressures at the upstream heel and downstream toe, added to the downstream head. The normal drainage holes spacing is 1.5-2 m, and the hole diameter is larger than 50 mm.
Drains which are drilled from a galley very close to the bottom of the dam may be assumed to reduce the uplift pressure to 50% of the difference between upstream and downstream heads added to the downstream value.
The drainage system must be inspected periodically, and redrilled if necessary. It is advised that the effectiveness of the drainage system be measured when installed or redrilled.
Reduction in uplift pressure due to a grout curtain is normally not accepted. If regrouting of the curtain is part of the maintenance plan, uplift pressure may be reduced by 50% as above.

FINLAND

Regulations

Finland has no regulatory rules concerning uplift which are specific to dams. Reference is made to a Structural Code of Practice which includes a section on external water pressure and pore pressure.

Regulatory Rules For Uplift Pressures In Safety Assessments

The Structural Code of Practice requires a linear reduction of uplift pressures from the upstream face to the downstream face.
The Dam Safety Code of Practice gives minimum values for factors of safety against overturning and sliding.
The Code of Practice does permit allowance to be made for any drains. Galleries, drains and pressure relief wells are used in the construction and allowance for these in the design is made.

NORWAY

Regulations

Some rules concerning uplift pressures are given for gravity dams in the Dam Regulation, 1990.

Regulatory Rules For Uplift Pressures In Safety Assessments

The 1990 Rules require that, wherever it is of importance for the stability of the dam, uplift pressures should be taken into consideration.
The uplift pressures may be taken as being equal to the pressure on the upstream side, decreasing linearly through the cross section, to the pressure on the downstream side.
For gravity dams, full water pressure has to be assumed upstream of the neutral axis of the cross section, and from there, decreasing linearly to the pressure on the downstream side.
If the dam is constructed with a drainage system, reduced internal water pressure values may be used.

SPAIN

Regulations

General regulatory rules concerning uplift pressures are given in the recent "Technical Regulation for Safety of Dams and Reservoirs", March 1996, that updates the previous "Instructions for the Design, Construction and Operation of Large Dams", March 1967.
In the new Regulation no specific technical indications are given (they are left to the responsibilities of the dam designer and dam owner), but basic safety criteria are defined to prevent and limit the potential risk to dams.

Regulatory Rules And Normal Practice For Uplift Pressures In Safety Assessments

In the Regulation the term "uplift" it is not used and it is replaced by the more general term "pore pressures".
The effects of pore pressures must be taken into account in the safety assessments for the three types of loading conditions (Normal, Abnormal, Extreme). These loading conditions must be defined by the Designer, according to general directions given in the Regulation.
In the Abnormal Conditions an abnormal increase of the pore pressures must be considered.
In the design the evaluation of pore pressures must be adequately justified, and corresponding preventing actions (drains, etc.) have to be adopted.
Pore pressures must be measured during dam operation, and if the measured values are higher than the design assumptions corrective actions must be taken.
The current normal practice, derived from the more detailed technical directions of the 1967 Regulation, is based on flow-net analyses to determine the distribution of pore pressures. Considering the unavoidable uncertainties that affect such evaluation, empirical rules are also used when adequate works for the reduction of uplift pressures (drains, grout curtains, etc.) are carried out and accurate measurement devices are used.

P O R T U G A L

Regulations

General Regulatory rules concerning uplift pressures are given in the "Regulation for the Design of Dams", n. 846/93, September 1993.

Regulatory Rules For Uplift Pressures In Safety Assessments

In safety assessments the effects of pore pressures must be considered taking into account the water flow through material pores, joints or cracks, mainly in the foundation, and the associated actions (mass forces, surface forces, volumetric changes).
In the analysis of the foundations of important dams, 2D or 3D numerical hydraulic models must be used for the evaluation of the water flow and pressure gradients. Mechanical effects of the water must be studied in terms of effective stresses. These effects are considered using mass forces proportional to gradients.
For stability analyses, mass forces can be replaced by surface forces, considering the uplift forces acting on the concrete/foundation interface and taking into account the effect of the drainage system.
For gravity dams or thick arch dams, the uplift pressures at the drainage line must be about 1/3 of the upstream hydrostatic pressure.
Limit values for leakage from foundation drains are also given in the Regulation

A U S T R I A

Regulations

There are no regulatory rules in Austria concerning uplift pressures under concrete dams.

Normal Practice For Uplift Pressures In Safety Assessments

Different assumptions for uplift pressures distribution were used in the past, and standard approaches cannot be identified.
Nevertheless, as time passed, certain common practice appeared.
For gravity dams a triangular distribution of uplift pressures is generally assumed, with 85% of the water head at the upstream heel, linearly decreasing to zero (or tailwater head) at the downstream toe.
For arch dams, uplift pressures decreasing linearly from 25% of water head (upstream heel) to zero (downstream toe) are often assumed under the highest dam sections. Under lower dam sections (toward the abutments) the same assumptions as for gravity dams is used.
However, the assumptions on uplift pressures are also related to the assumptions on shear strength parameters used in stability calculations.

G E R M A N Y

Regulations

In Germany the assessment of dam safety is based upon German DIN standards.
Federal state laws prescribe the use of these standards.
Basic standards for dams are DIN-19700 part 10 (Dam plants – General specifications) and part 11 (Dam plants–Dams) and DIN-19702 (Stability of solid structures in water engineering).
In the former GDR the TGL-regulations (TGL-technical standards) are still valid during a transition period. TGL 21239, part 2: “Dam plants – Dams – Technical demands for design and construction of gravity dams” is the valid regulation for gravity dams founded on solid rock.
In addition, the guideline 242 “Calculation methods for gravity dams” of the DVWK (German Association for Water Resources and Land Improvement) describes methods to determine the permeability of the subsurface rock considering open fractures, and the finite element method (FEM) as a method to calculate pore water pressures.

Regulatory Rules On Uplift Pressures In Safety Assessments

In stability analyses according to DIN-19700 (part 11) uplift and pore water pressure are calculated as a load. These loads are combined with “states of abutment” (state of the subsurface rock). The “states of abutment” are distinguished by the effectiveness of intervention measures (e.g. grout curtain or drainage) which are effective in state A, partly effective in state B, and ineffective in state C. Safety factors are reduced form A to C.
According to DIN-19702, uplift and pore water pressure forces can be determined by:
simple assumptions, for example linear reduction along the base.
flow-net analysis, if ground water flow is influenced by special constructions.
If a crack opens in a gravity dam, full hydrostatic water pressure (uplift) is assumed over the area of the crack. The pore water pressure may be assumed to decrease linearly to zero from the end of the crack to the downstream toe of the dam.

In mass concrete and masonry dams, a crack may not be longer than half of the cross section. In masonry dams horizontal cracks on the upstream side are allowed only for extreme load cases (e.g. earthquakes).
In practice, increased water pressure within cracks is not used with seismic loads.
According to TGI 21239 uplift and pore water pressure can be reduced if grout curtain or drainage system, or both, are provided. “States of abutment” are not used.
The hydraulic gradient between upstream toe, grout curtain and drainage must not be larger than 10.
1 m is recommended as the minimum distance between drain holes.
The minimum size of drains is 200 mm diameter.

Measurement Of Uplift Pressures

If the stability analyses are based on the presence of a grout curtain and/or drainage systems, their effectiveness has to be continuously monitored by means of appropriate measuring devices. The amount of seepage has also to be monitored continuously.

F R A N C E

Regulations

There are no regulatory rules concerning uplift pressures in French Regulation.

Normal Practice For Uplift Pressures In Safety Assessments

For dams without drains, a linear distribution of uplift pressures is used (from full reservoir head to tailwater head).
When drains are installed, the uplift pressures are normally assumed between 50% and 66% of the difference between upstream and downstream water head plus the downstream water head.
The uplift pressures are usually measured by piezometers to verify the design assumptions.
The design of drainage system is achieved with standard practice commonly adopted, described in the CIGB bulletin n. 88.
The geology of the site, and more accurately the sizes and the importance of faults are taken into account to design the spacing and the tilting of the drainage boreholes. The grout curtain is also taken into account.
The seepage path and the uplift depend on the location of grout and drainage curtains. It is not considered a good practice to have a drainage curtain as deep as the grout curtain because of the reduction of the seepage path and the increase of the hydraulic gradients. The distance between drainage and grout curtain and the depth are designed to comply with these constraints.
During the dam construction, and particularly during the excavation works, the improved knowledge of the rock quality may lead to adaptation of the drainage design in order to enhance performance (reducing uplifts as much as possible).
A good practice is to keep a sufficient space between two holes to have the opportunity to bore additional holes.

SWITZERLAND

Regulations

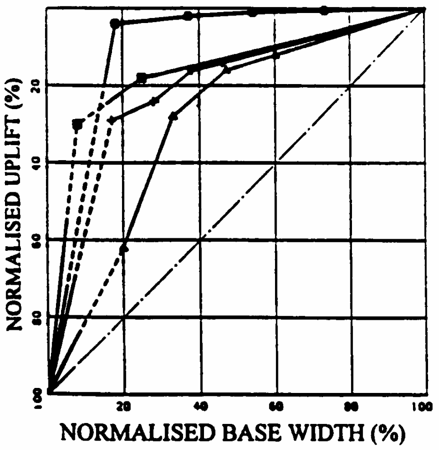
There are no regulatory rules concerning uplift pressures in Swiss Regulation.

Normal Practice For Uplift Pressures In Safety Assessments

Uplift pressures are usually taken into account for the stability assessment and the safety assessment against sliding for gravity dams.
The distribution of uplift pressures along the base section is assumed to vary linearly from full reservoir pressure at the upstream heel to zero or tailwater head at the downstream toe.
Where a properly functioning drainage system exists, the pressure relief is taken into account by reducing the uplift pressures at the upstream heel. The reduction factors currently varies between 0.75 and 0.8.

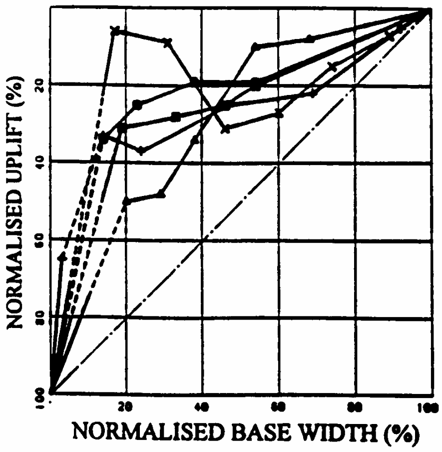
SWISS GRAVITY DAMS

HEIGHT > 100 m



□ GR. DIXENCE
○ ALBIGNA
▲ SPITALLAMM
\+ OBERAAR

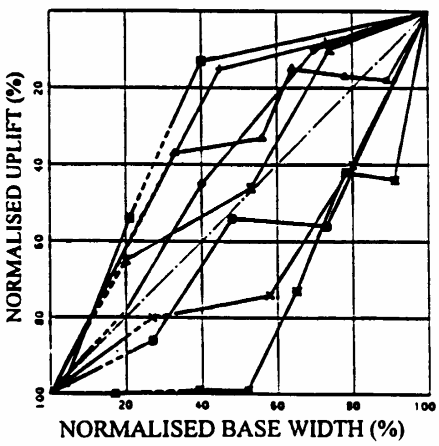
HEIGHT < 100 m



□ RAETERICHSB
○ MAPRAGG
▲ ROBIEI
\+ BAERENBURG
× GRIES

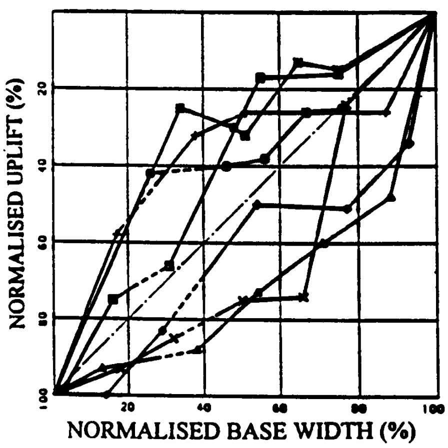
SWISS ARCH DAMS

HEIGHT > 150 m



□ MAUVOISIN
○ CONTRA
⧗ CONTRA
▲ LUZZONE
× EMOSSON
\+ ZEUZIER
✱ CURNERA
◆ ZERVREILA

HEIGHT < 150 m



□ MOIRY
▲ GIGERWALD
\+ LIMMERN
○ PUNT DAL GALL
× NALPS
◆ SANTA MARIA
✱ CAVAGNOLI

Fig. 1 – Normalised uplift profiles (Ref. 2)

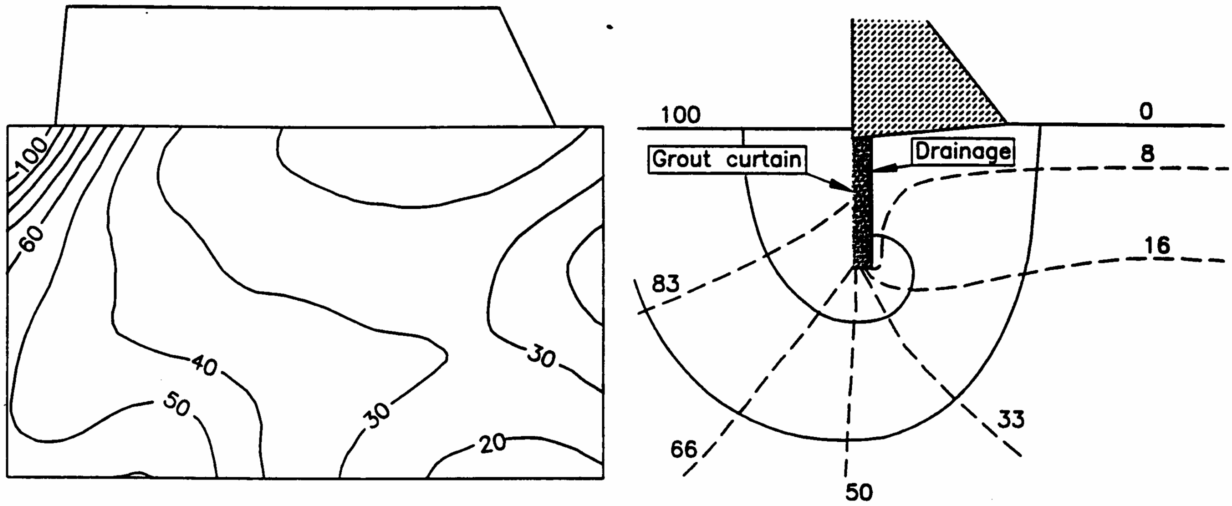


Fig. 2 – Measured and theoretical isobar contour-lines (drained foundation) (Ref. 1)

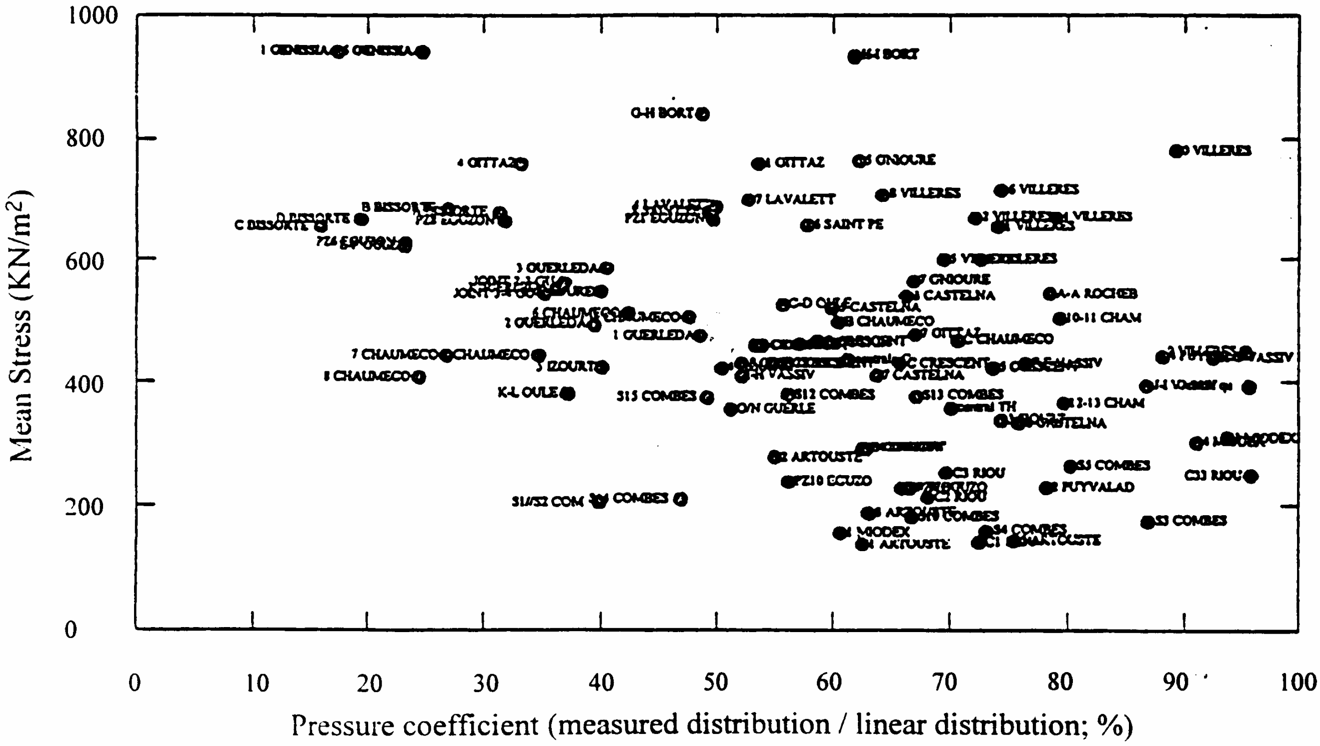


Fig. 3 – Relationship between uplifht and mean stress in the foundation (Ref. 1)

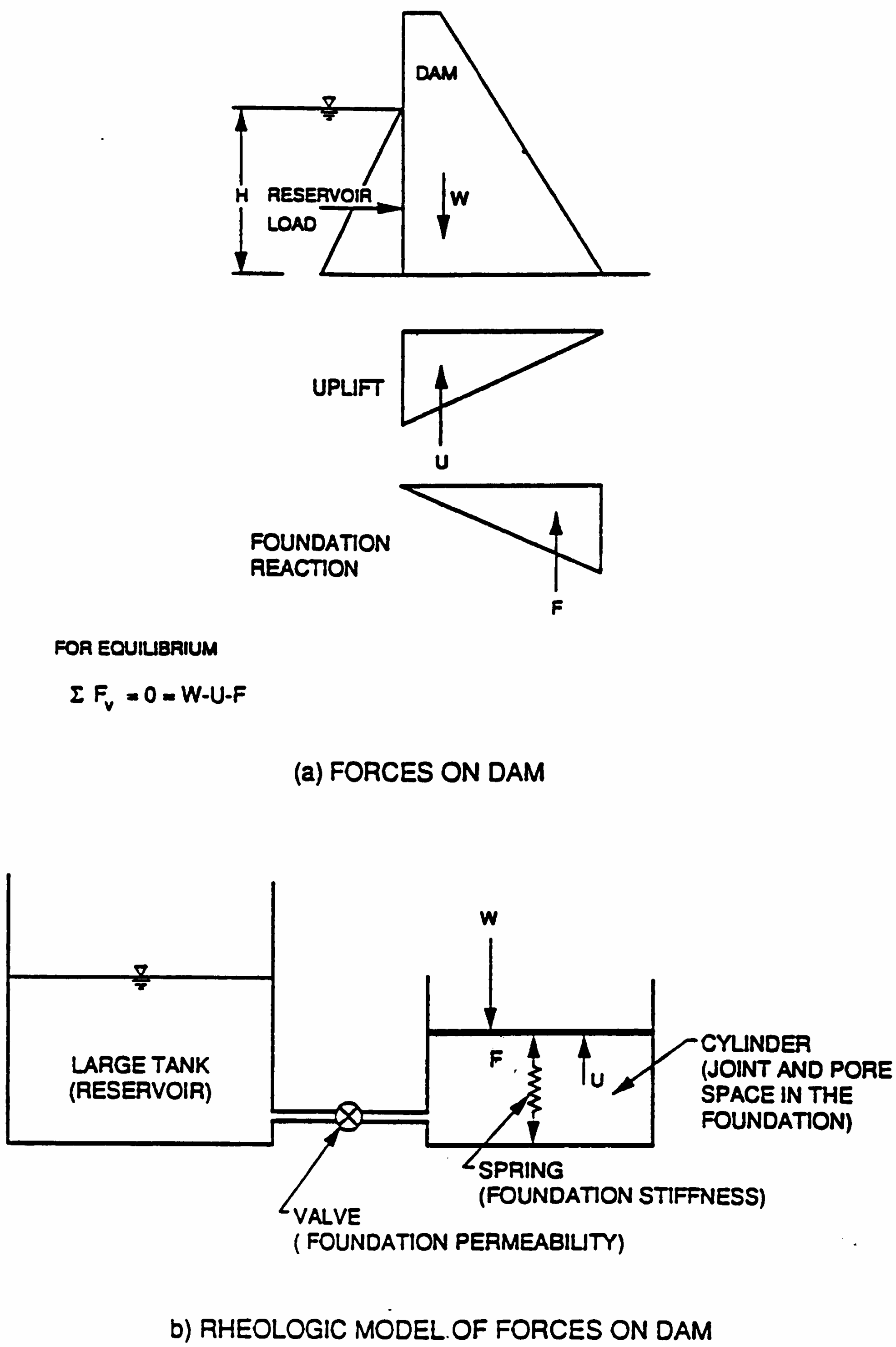


Fig. 4 – Model of flow in a deformable dam foundation (Ref. 4)

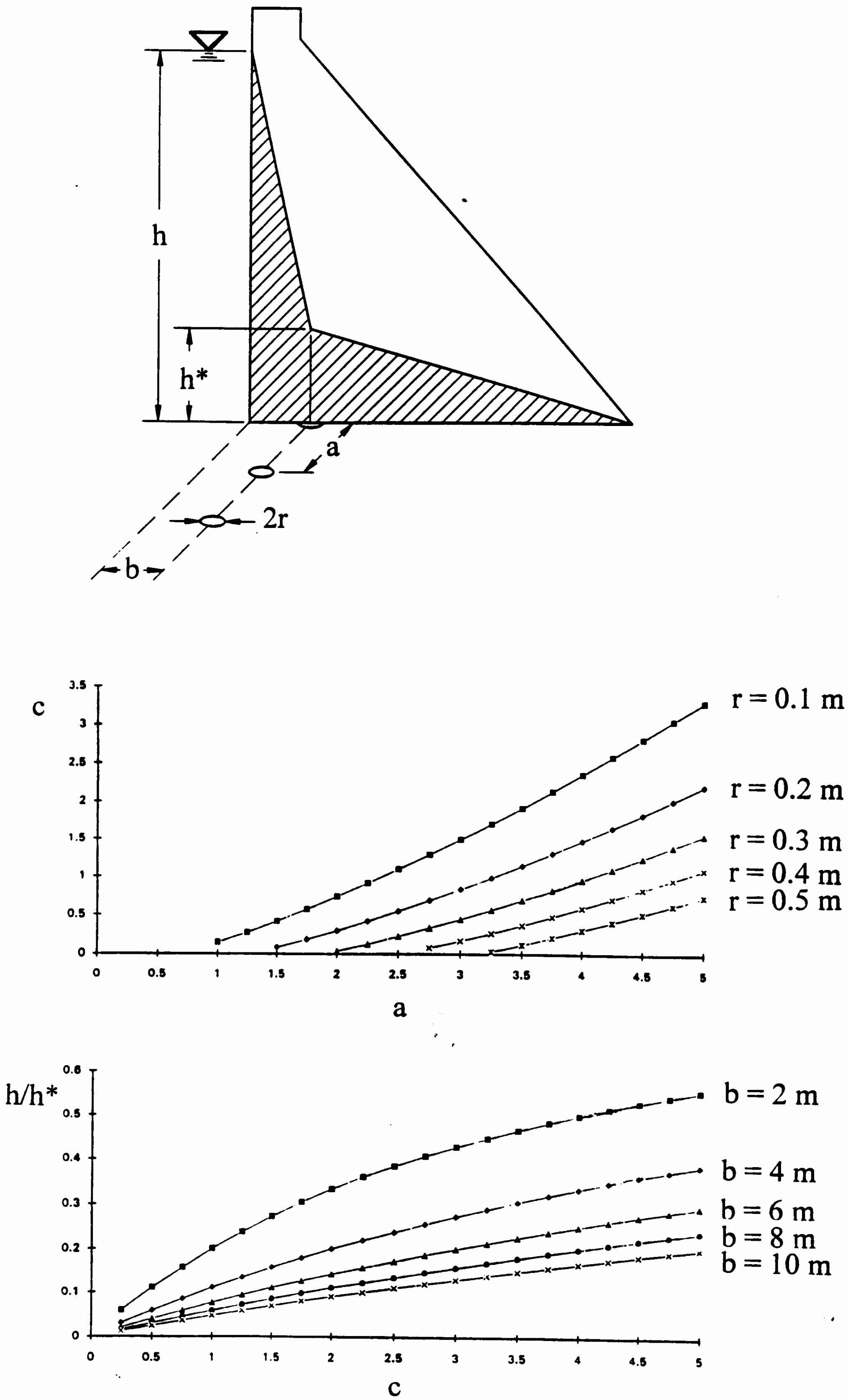


Fig. 5 – Uncoupled analysis – Charts (Ref. 6)

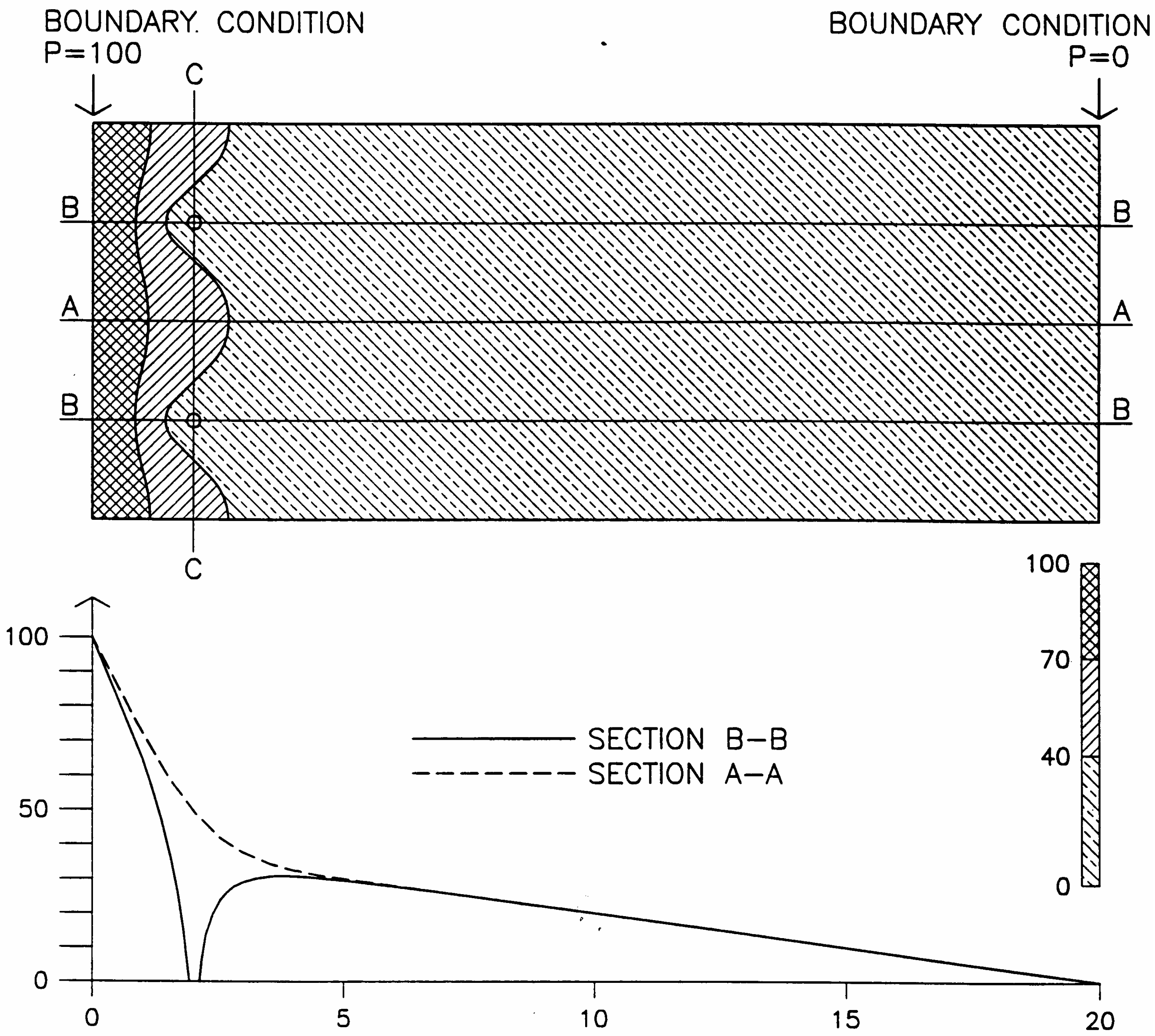
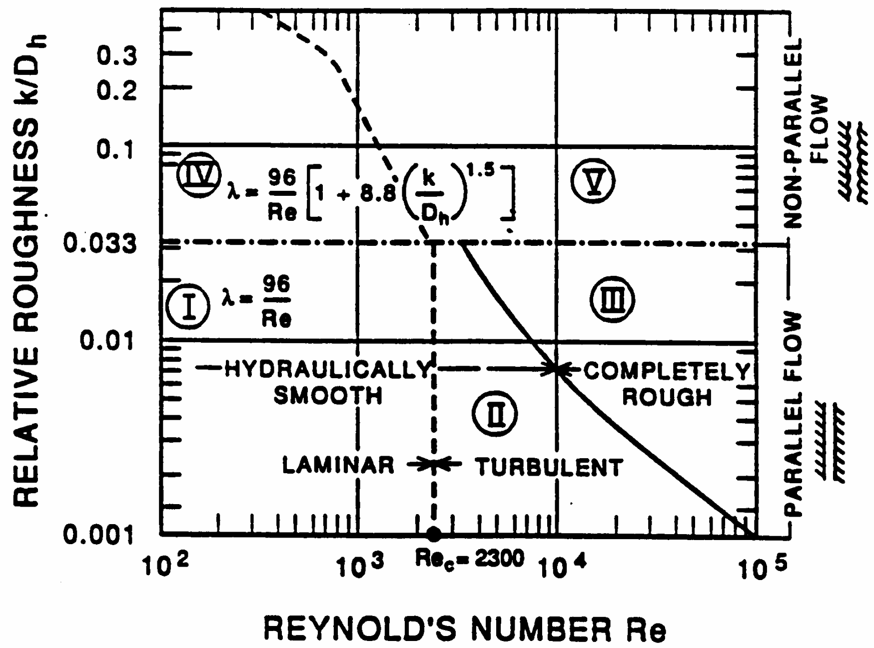


Fig. 6 – Uncoupled F. E. analysis (Ref. 7)



------ LIMIT LAMINAR - TURBULENT FLOW

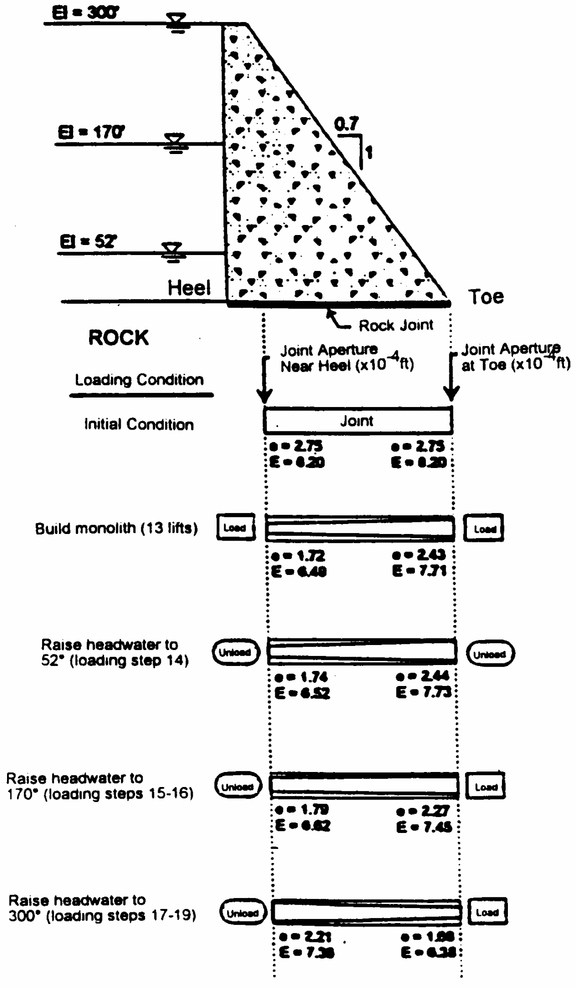
—·—·—·— LIMIT PARALLEL - NON PARALLEL FLOW

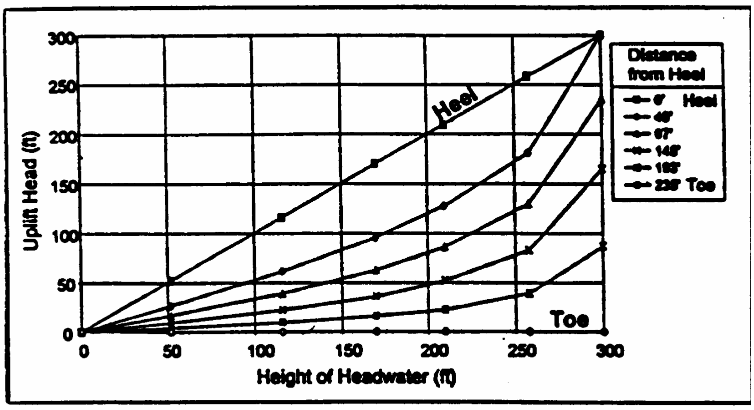
——— LIMIT HYDRAULICALLY SMOOTH - COMPLETLEY ROUGH FLOW REGIMES

λ FRICTION COEFFICIENT ASSOCIATED WITH LOSSES OF ENERGY IN THE CRACK

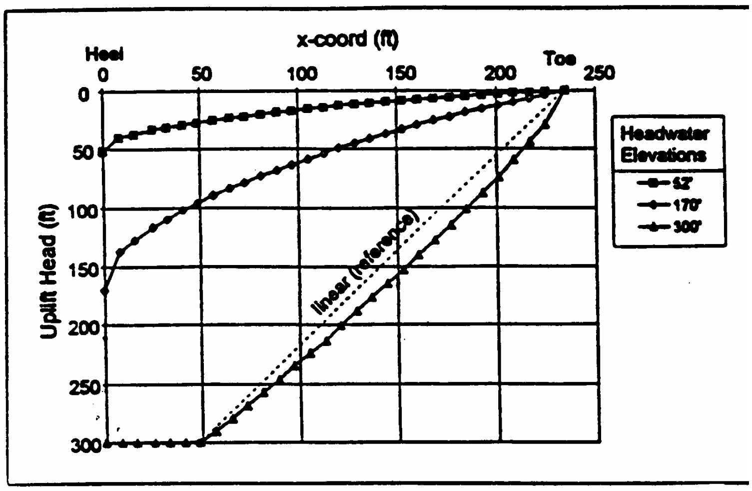
Hydraulic zone*	Hydraulic Conductivity, K	Exponent, α	Flow condition
I	$\frac{g.b^2}{12\nu}$	1.0	Laminar
II	$\frac{1}{b}\left[\frac{g}{0.079}\left(\frac{2}{\nu}\right)^{0.25}\cdot b^3\right]^{\frac{4}{7}}$	4/7	Turbulent
III	$4\sqrt{g}\,log\left[\frac{3.7}{k/D_h}\right]\sqrt{b}$	0.5	Turbulent
IV	$\frac{gb^2}{12\nu[1+8.8(k/D_h)^{1.5}]}$	1.0	Laminar
V	$4\sqrt{g}\,log\left[\frac{1.9}{k/D_h}\right]\sqrt{b}$	0.5	Turbulent

Fig. 7 – Flow laws (Ref. 11)





Variation of uplift head at six locations along the joint whit headwater elevation



Variation of uplift head along a single joint whit three headwater elevations

Fig. 8 – Coupled fluid-sturcture analysiys (Ref. 8)

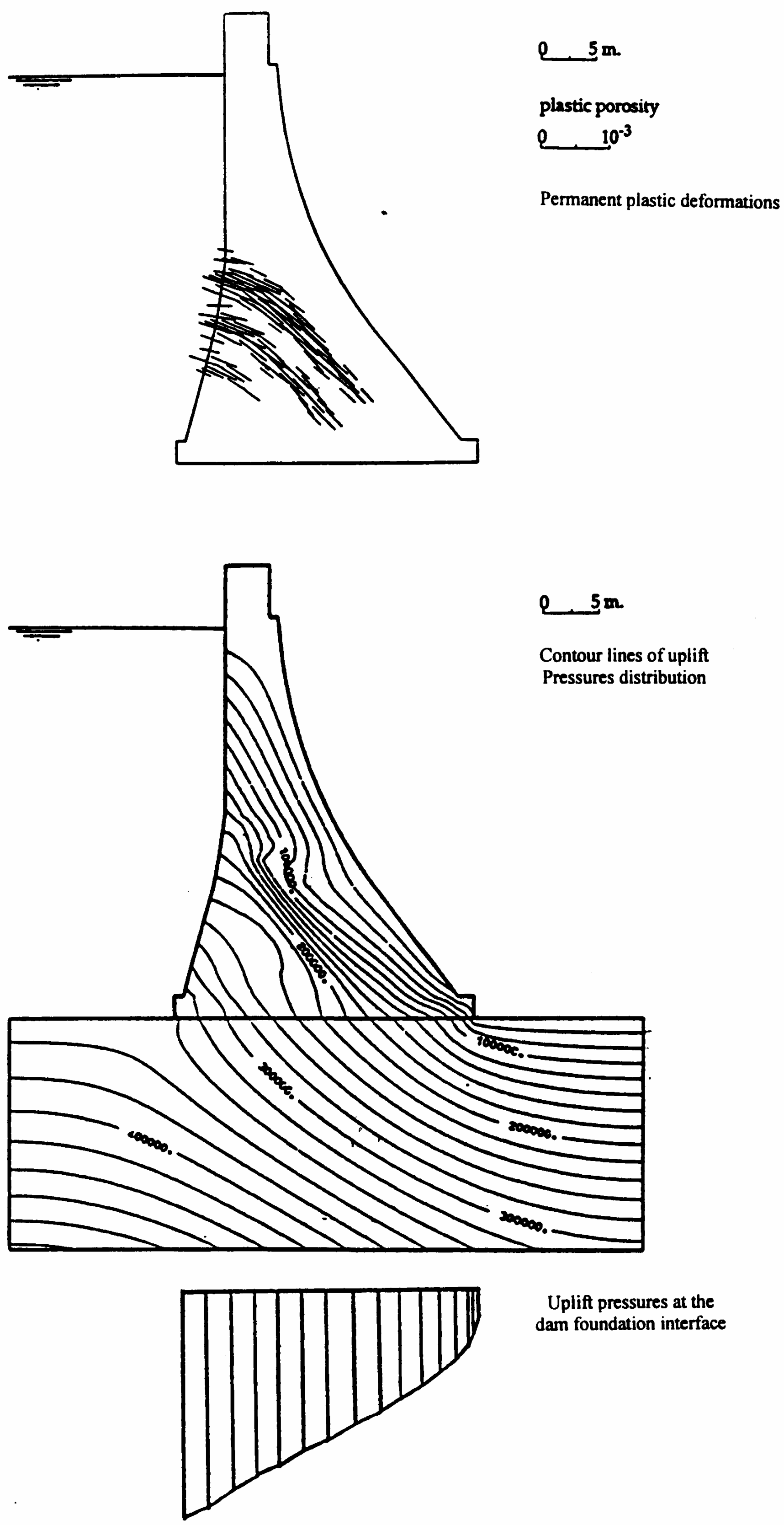
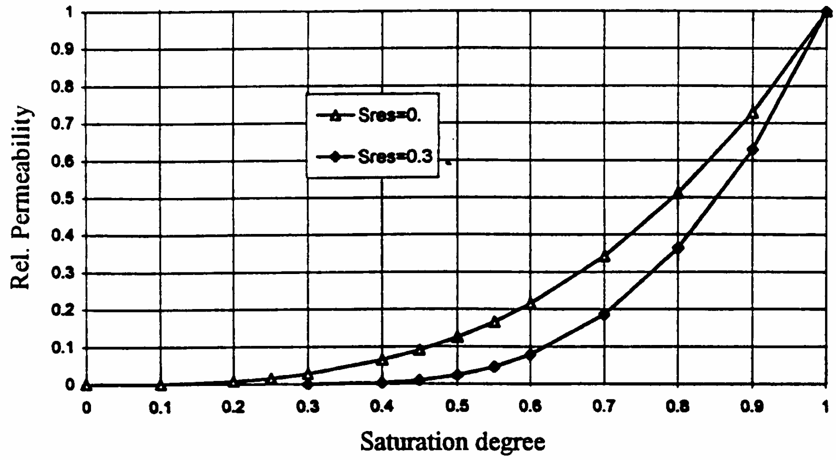
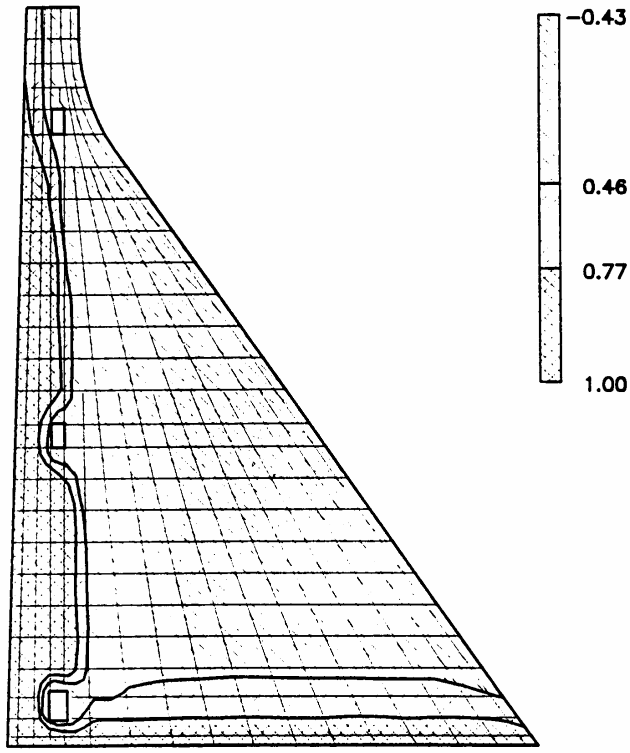


Fig. 9 – Poro–plastic analysis (Ref. 13)





Saturation condition in the dam body

Fig. 10 – Analysis of Unsatured Conditions (Ref. 15)