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RESEARCH REPORT

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ON INTERNAL EROSION IN EMBANKMENT DAMS

A literature survey of the phenomenon and the prospect to model it numerically

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Cover illustration: sinkhole on the tailings dam at Zinkgruvan in Sweden (picture courtesy of SWECO).

PREFACE

The work presented in this research report has been carried out as a collaboration project at Luleå University of Technology between the research divisions:

- Mining and Geotechnical Engineering, Department of Civil, Mining and Environmental Engineering
- Fluid Mechanics, Department of Applied Physics and Mechanical Engineering.

This research project has been financially supported by SVC (Swedish Hydropower Centre).

Many people have assisted us during the work with this literature survey. We would like to especially thank Prof. Sven Knutsson and Prof. Håkan Gustavsson for helpful suggestions during this project.

Luleå, October 2008

Hans Mattsson, J. Gunnar I. Hellström and T. Staffan Lundström

ABSTRACT

The main objective with this literature survey is to elucidate the state of the art of internal erosion in embankment dams in order to be able to formulate a research program for numerical modelling of internal erosion in a physically sound manner. Since these processes normally are localised to specific zones in a dam, the ordinary continuum approach frequently utilised in soil modelling will not, by itself, be successful. The plan of the research group is therefore to treat internal erosion numerically as a type of localisation and describe the constitutive behaviour with micromechanical models in localised zones. In the next step, the internal erosion model developed will be implemented in a mathematical consistent fashion in a continuum model, based on e.g. the finite element method. In such software, ordinary computations of stresses, strains, deformations and pore pressures in an embankment dam can be performed; results which possible lead to conclusions about the initiation of internal erosion processes. When internal erosion is initiated, the micromechanical models will describe these processes in localised zones.

It was decided to restrict the literature survey to areas important for the research direction defined above. The chosen areas are thus: numerical modelling of embankment dams, internal erosion processes, embankment dam failures and accidents, filter design, micromechanical models of hydro dynamical loads on single particles and system of particles as well as micromechanical models for friction between single grains and system of grains. The most essential findings for each area, in the context of the research, will be described below.

There exists much experience in the field numerical modelling of embankment dams which is exemplified by the large amount of published papers on the subject. However, the processes of internal erosion have not been modelled in a general manner, as outlined in the research idea above, in any of the papers cited in this literature survey.

There are three main internal erosion processes that can initiate piping: backward erosion, concentrated leak and suffusion. Piping can occur in the embankment, through the foundation and from the embankment into the foundation. Embankment dams are normally constructed in zones of different materials. Different compressibilities of the various zones might lead to internal redistribution of stresses and uneven settlements, which might cause cracks or ”soft zones” where internal erosion can be initiated. Cracks can also occur later on by hydraulic fracturing. All of that described above are examples of mechanical processes that are theoretically possible to model numerically. To receive information about potential internal erosion problems in a dam at an early stage,

geophysical methods are a promising alternative or complement to numerical methods. However, much research and development remain before the results from geophysical methods utilised for permanent monitoring and surveillance of dam structures are completely trustful.

It is often very difficult to determine the exact reason for a dam accident or a failure since the processes involved have a tendency to destroy evidence which might have existed. Statistical data, however, shows that failure by overtopping and piping are the most common modes of failure while failure by slides is less common. Many piping failures occur very fast, leaving a short time for proper actions.

The safety of large embankment dams is strongly dependent on the reliability of the performance of their critical filters. Existing filter design criteria are in most cases empirically derived, implying that a lot of knowledge can be gained by taking a mechanical approach to the problem. More research would also be desirable in the field of ageing effects of dams and uncertainties of core/filter appearance during and after accidents, incidents etc. For this, numerical modelling also seems to be a promising approach.

Continuum formulations of flow through porous media often results in equations consisting of a few unknown parameters such as the permeability. The physical background of such parameters can often be traced to the detailed flow in the pores and it is therefore in place to study the flow on this level, as well. The forces on individual particles have been exploited for certain geometries and for a number of flow conditions. We however need to investigate further higher Reynolds number flows, more complex geometries and instationary conditions.

The forces from the micromechanical models must be balanced by gravity and forces that emanates from particle interactions for the dam to be stable. As a first, and most simple, criteria the size distribution of the particles being subjected to hydrodynamic pressure is compared to the pore size distribution of the porous medium. Hence the particles will move if they are small enough independent on magnitude of the force on it. In reality, however, the hydraulic forces must exceed particle interaction forces and or gravitational forces keeping the particles in place at normal conditions. The forces are strongly dependent on the size of the particles and therefore dependent on different phenomena such as cohesion, adhesion and static friction.

In the final part of the literature survey a concept for numerical modelling of internal erosion is presented based on ideas that have emerged from this work. The concept involves as well mathematical developments in order to formulate a micromechanical model for internal erosion as laboratory tests. The theoretical work and the practical laboratory work should be performed simultaneously and in an interactive manner. A software containing a model that simulates internal erosion could be useful for: an increase of the knowledge about internal erosion processes, evaluating the risk for dam incidents caused by internal erosion, estimating the time for progression of internal erosion and piping, studying self-healing of leaks, changes in filter behaviour subsequent to particle accumulation and ageing effects in dams, analysing the amount of instrumentation needed in a dam and the proper location for monitoring and surveillance as well as designing dams to mention a few examples. It is therefore apparent that the route suggested has a high potential to become a tool for future improvements of dam safety.

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1. INTRODUCTION

Internal erosion in embankment dams is not a completely understood phenomenon. Dam failures, accidents and deterioration of dams because of ageing effects do take place as a result of internal erosion. Even if dams generally should be considered as safe constructions, a large part of the present knowledge about problems associated with internal erosion is a result of studies of former dam incidents. This circumstance is almost inevitable since internal erosion processes are very complex and take place inside the embankment or foundation making the erosion transparency limited until it has progressed enough to be visible or detected by measurements. Instrumentation inside a dam could perhaps detect erosion processes at an early stage but at the same time internal erosion has a tendency to concentrate adjacent to different kinds of installations, so it is not desirable to have more installations than necessary.

There has been much progress in the field of numerical modelling of advanced geotechnical problems during the last decades. To the authors´ knowledge, the processes of internal erosion in embankment dams have, however, still not been incorporated successfully in such numerical models. A possible explanation for this could be that soil normally is interpreted as a continuum in commercial numerical software, while the analysis of formation of cracks, softened zones and pipes as a result of the erosion process, almost certainly need some additional approach based on e.g. localisation. It is, of course, also important to have sufficient knowledge of the fundamental mechanical behaviour of internal erosion to be able to model it in a proper way.

As a part of the initiative from the Swedish Hydropower Centre (SVC: Svenskt Vattenkraftscentrum) to secure and develop knowledge and competence in Sweden within the hydropower engineering field, our research group is focusing on dam safety issues. The overall objective for future research in the group is to develop a numerical model that can simulate the processes of internal erosion in a physically sound fashion. In addition, the model should be implemented in a software code in order to be applicable in research and for practical purposes. As a first step, the software should be validated by comparing the current knowledge of the mechanical responses from the internal erosion processes with results from simulations in order to determine how successful the tool is; and if the result is positive the software can be used in more general cases. Beyond the obvious fact that it is of considerable value to increase the understanding about the internal erosion processes, there are many potential advantages with a software containing a model that simulates internal erosion, it can for instant be utilized for: design purposes, evaluating the risk for accidents and failures involving internal erosion, analysing the time for development of internal erosion and piping, studying the mechanisms associated with ageing of embankment dams, investigating the self-healing phenomenon and the potential remaining damage to the structure, examining the quantity of instrumentation needed as well as suitable locations for them. Naturally, such software should also be able to model ordinary variables dealt with in numerical geotechnical modelling software, like: stresses, strains, pore pressures, deformations etc. To comprehend the physical processes involved in internal erosion, laboratory tests must be performed and the result should be a base for the formulation of mathematical criteria.

This literature survey is a result of the research group's ambition to understand the state of the art of internal erosion and numerical modelling of the phenomenon before the final research programme is formulated. In Section 8, some ideas are presented, that have emerged from this literature survey, about how to actually carry out the numerical modelling project of internal erosion.

2. NUMERICAL MODELLING OF EMBANKMENT DAMS

Numerical modelling can be a useful tool in geotechnical engineering since a wide variety of problems is theoretically possible to model. Embankment dams, that geometrically are quite simple structures, are especially attractive to model numerically in order to get an early notion of potential hazardous processes, not visible on the outside but that can be in progress inside the dam. The main problem with numerical modelling (e.g. noticed by Fell *et al.*, 2005) is that those performing such analyses normally need specialist knowledge in a wide range of technical subjects. At the same time, numerical software is within reach for almost anyone with a computer. Not seldom, commercial software are extremely user-friendly which makes it easy to produce results, even wrong ones, often presented in the form of colourful plots. Potts and Zdravkovi´c (1999, 2001) list the knowledge desirable to be able to perform useful geotechnical finite element analyses. ”Firstly, a sound understanding of soil mechanics and finite element theory is required. Secondly, an in-depth understanding and appreciation of the limitations of the various constitutive models that are currently available is needed. Lastly, users must be fully conversant with the manner in which the software they are using works. Unfortunately, it is not easy for a geotechnical engineer to gain all these skills, as it is very rare for all of them to be part of a single undergraduate or postgraduate degree course. It is perhaps, therefore, not surprising that many engineers, who carry out such analyses and/or use the results from such analyses, are not aware of the potential restrictions and pitfalls involved.” The books by Potts and Zdravkovi´c (1999, 2001) and Muir Wood (2004) are examples of very good texts on the subject ”numerical modelling in geotechnical engineering”; the books contain many valuable general advises about how numerical modelling should be performed.

In practical modelling of dams, there will probably always exist uncertainties whether the dam will have the same characteristic and behaviour after construction that was intended at the design stage or not. This is certainly a challenge for a user of numerical software which obviously besides detailed understanding of the computer program and its mathematical basis must have engineering judgement and practical experience; since if the model is not representative for the actual dam the outcome of the computation must be questioned. It is not realistic to believe, that a precise prediction of the performance of the dam is always possible and with that in mind a user should avoid to draw to “high-flying” conclusions. Examples of insecurities, to deal with in embankment dam engineering are discussed in e.g. Milligan (2003). Especially, the high probability of segregation at handling and placement of some common types of core and filter materials should be paid attention in studies of internal erosion. It is likely that erosion processes could be concentrated to or initiated at zones with segregated material. The problem of segregation is described further in Section 5.1 in connection to filter design.

The Finite Element Method (FEM) has been used to estimate stresses and deformations in embankments for about 30 years, so there exists much experience in this field. In the literature many papers can be found that contain result from finite element analyses on embankment dams. Such papers, of special interest in the context of internal erosion, are e.g. Sherard (1986), Ng and Small (1999), Zhang and Du (1997), Day *et al.* (1998), Sharif *et al.* (2001) and Huang (1996). However, the processes of internal erosion have not been modelled in a general manner in any of the papers the authors´ studied in this literature survey.

When modelling internal erosion it can be convenient to start from a fluid mechanic point of view, which creates possibilities regarding the solution of the fluid flow through porous media, and the modelling can be performed in numerous ways as outlined by Bear (1972). It can, for instance, be convenient to apply Hodographic methods in which the boundaries of the fluid including the free surface within the dam are transformed to form simpler geometries. Although the flow in it-self can be treated with explicit expressions such as Darcy's law non-homogeneous and anisotropic permeabilities and the naturally formed free surface often makes realistic modelling with analytical tools complex. One possible way to model the flow through embankment dams is instead to solve for the momentum equations for two phases and then add an advection equation to deal with the motion of the surface formed between the phases, the air and the water, for instance. To deal with the sharp transition in viscosity and density a modified Heaviside-step function can be introduced, see Sharif *et al.* (2001). Adaptive FEM analysis of free surface flow is also applicable, Sheng-Hong (1996). Another method is to combine spectral analysis with a Laplace transform in order to model the transport of material through for instance the riprap, Wörman and Xu (2001). A combination of the finite element method with a stochastic approach and the FEM part is used to solve the mechanical behaviour of the earth structure while the stochastic method is used for the probabilistic part of the earth modelling, Mellah *et al.* (2000). Numerical procedures based solely on the finite element method are also common. Such a technique can, for instance, be applied to hydraulic fracturing in the core of earth and rock-fill dams. The finite element procedure makes then use of special joint elements that allow fluid flow and fracture to be modelled allowing the progress of pore pressure in the core of a dam to be simulated, Ng and Small (1999). The Finite Volume Method can be used to solve the flow through the porous material. With such approaches the main flow features are resolved in a better way Elforsk (1999) and Elforsk (2005).

2.1. Continuum models

Flow through porous media, which embankment dams can be considered as, is generally modelled by a continuum approach implying that the flow in the pores is averaged and that averaged quantities such as the permeability are introduced.

The flow through porous materials is often described by Darcy's law that gives a linear relationship between the flow rate and the pressure. The law was originally derived from experiment Bear (1972), Scheidegger (1972) and Dullien (1992) but Tucker III and Dessenberger (1994) have shown that the law can be derived from the Navier-Stokes equations. The conditions are that inertia and long-range viscous effects are negligible of which the former is generally a too crude assumption for at least the final stages of internal erosion. Let us, however, for the moment stick to the stated assumptions and write Darcy's law in its general form:

$$v_i = -\frac{K_{ij}}{\mu} p_{,j} \tag{2.1}$$

where v_i is the superficial velocity, K_{ij} the permeability tensor, μ the viscosity and p the pressure. An obvious interpretation of (2.1) is that the permeability can vary as a function of direction. It is also important to notice that:

$$u_i = \frac{v_i}{\phi} \tag{2.2}$$

where u_i is the average velocity in the pores and ϕ the porosity. In this rigorous form of Darcy's law the permeability is solely dependent on the detail geometry of the porous material and has the unit m^2. In the literature describing embankment dams it is also common to give the permeability the unit *m/s*, conductivity or velocity, Fell *et al.* (2005). This implies that the viscosity and the gravitational constant are incorporated in the actual permeability value. The advantage of this approach is that a velocity-like component is explicitly given. The draw-back is that the actual value is dependent on the conditions for the measurements.

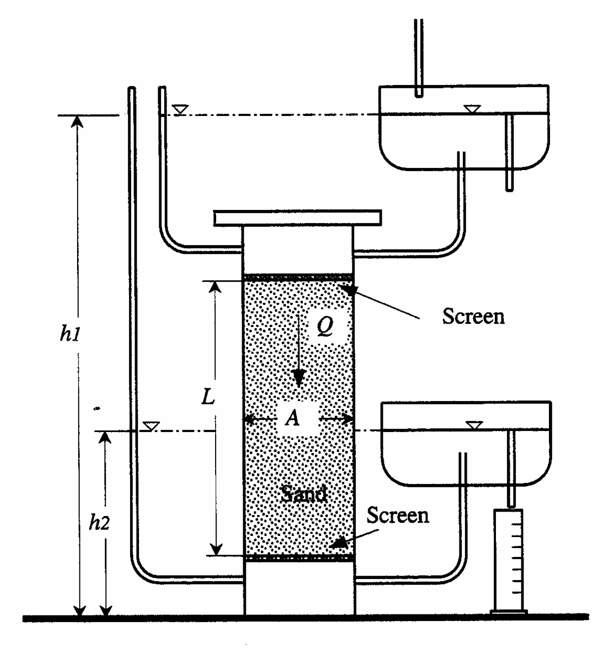

Figure 2.1 Set-up by Darcy.

The permeability of a porous material is generally obtained from measurements although numerous models often based on the micro geometry of the porous media have been derived. The models are developed for special cases and measurements need to be carried out in order to calibrate the models Gebart (1992). Many methods for the experimental assessment of the permeability tensor have been proposed following the experiments by Darcy; Kershaw (1972), Adams *et al.* (1988), Pikulik *et al.* (1991), Parnas and Salem (1993), Parnas *et al.* (1995), Young and Wu (1995), Gebart and Lidström (1996), Lekakou *et al.* (1996), Weitzenböck *et al.* (1998), Lundström *et al.* (1999) and Lundström *et al.* (2000). Most of these methods are based on either of two fundamentally different principles: (i) parallel flow, by which the liquid is made to flow in a controlled direction through the porous sample (the Darcy experiment, see Figure 2.1); and (ii) radial flow, by which the liquid is injected at a "point" and flows freely in all directions (usually in a plane). Either of these principles may be applied in a variety of ways, e.g. by driving the liquid by a constant flow rate or a constant pressure drop. The advantage of the radial flow techniques is that a full permeability tensor, including principal directions, can be obtained in a single experiment Ahn *et al.* (1995) and Weitzenböck *et al.* (1998). The disadvantage is that they are based on tracking a flow front, i.e., the flow must be unsaturated, with the risk of including various transient effects, such as capillary action and void formation at the wetting flow front. Parallel flow measurements, on the other hand, can be carried out under steady state conditions Lundström *et al.* (2000) and edge effects can be avoided

Lundström *et al.* (2002). Hence there are by all means several ways to determine the overall permeability of a porous material.

As Reynolds number (*Re*) increases over 1-10, the value is depending on material properties such as porosity and packing (quadratic, hexagonal or other), it is well-known that experimental data starts to deviate from Darcy's law. The overall effect is that the pressure drop becomes higher than what are predicted by equations 2.1-2 due to inertia and turbulence and an additional term is introduced in the so called Forchheimer equation

$$-\frac{1}{\mu}K\frac{\Delta p}{L}=\frac{Q}{A}+b\left(\frac{Q}{A}\right)^{m} \tag{2.3}$$

where b is a property of the porous media and m is a measure of the influence of fluid inertia. The equation was later modified by Ergun by fittings to experimental data according to

$$\frac{\Delta p}{L}g=150\frac{(1-\varepsilon)^{2}}{\varepsilon^{3}}\frac{\mu\frac{Q}{A}}{D_{p}^{\;2}}+1.75\frac{(1-\varepsilon)}{\varepsilon^{3}}\frac{\rho\left(\frac{Q}{A}\right)^{2}}{D_{p}} \tag{2.4}$$

where ε is the fractional void volume in the bed and D_p is the effective diameter of particles Ergun (1952). The Ergun equation does not have any micromechanical basis and the geometrical parameters shaping the form of the equation are therefore unknown. Recent studies indicate that the parameters of the Ergun equation should be modified for certain geometries Papathanassiou *et al.* (2001) and Nemec and Levec (2005) which was also confirmed by a study by Hellström and Lundström (2006).

3. INTERNAL EROSION PROCESSES

The soil fractions that are considered as most susceptible to erosion are relatively uniform coarse silt and fine sand. Cohesive soils as clays are more resistant to erosion as long as the chemical bonds are not destroyed, Srbulov (1988). It seems like some core materials of glacial origin can be particularly susceptible to internal erosion. Glacial till is of special interest since it is a common core material in Swedish dams. Erosion susceptibility of glacial till is described in e.g. Foster *et al.* (2000a), Ravaska (1997), Norstedt and Nilsson (1997) as well as Sherard (1979) and discussed further in Section 4.1.

According to Fell *et al.* (2005), in their excellent textbook about geotechnical engineering of dams, four conditions must exist for internal erosion and piping to occur. These are: ”1) there must be a seepage flow path and a source of water; 2) there must be erodible material within the flow path and this material must be carried by the seepage flow; 3) there must be an unprotected exit, from which the eroded material may escape and 4) for a pipe to form, the material being piped, or the material directly above, must be able to form and support ´roof˝ for the pipe”.

In a central core earth and rockfill dam there are mainly three processes (Fell *et al.*, 2005) which can initiate piping: backward erosion, concentrated leak and suffusion. Backward erosion is initiated at the exit point of seepage and the erosion is gradually progressing backward forming a pipe. Concentrated leak initiates a crack or a soft zone emanating from the source of water to an exit point. Erosion gradually continues along the walls of the erosion hole intensifying the concentrated leak. Suffusion is the process where the fine particles of the soil wash out or erode through the voids formed by the coarser particles. This can be prevented if the soil has a well graded particle size distribution with sufficiently small voids. Soils are called internally instable if suffusion takes place and internally stable if particles are not eroding under seepage flow.

Piping can occur in the embankment, through the foundation and from the embankment into the foundation. Conceptual models for development of failure by piping for these three cases are presented in Figure 3.1 to Figure 3.3. In addition, a failure path diagram for failure by piping through the embankment is shown in Figure 3.4. Similar failure path diagrams for failure by piping through the foundation and from the embankment into the foundation can be found in Fell *et al.* (2005) or Foster (1999). External erosion problems are described in e.g. Skoglund and Solvik (1995).

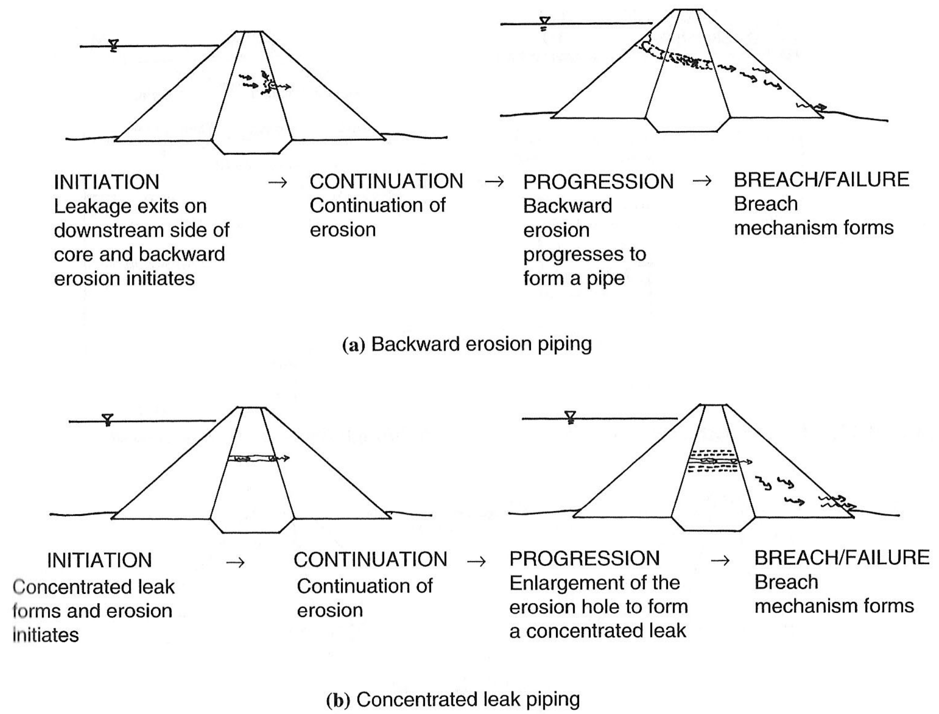

Figure 3.1 Conceptual model for development of failure by piping in the embankment: (a) backward erosion and (b) concentrated leak (Foster, 1999).

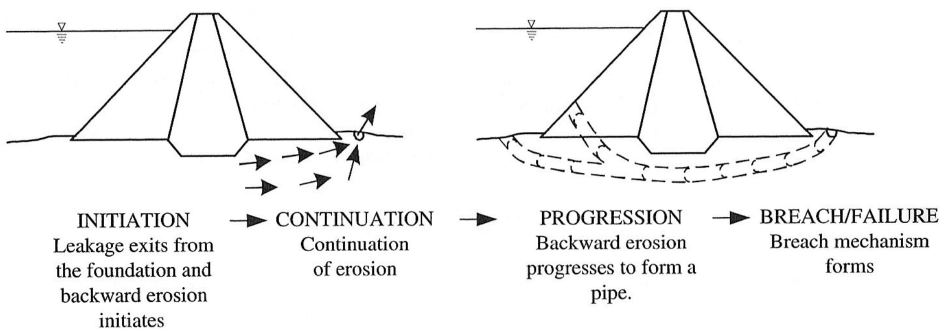

Figure 3.2 Conceptual model for development of failure by piping in the foundation (Foster, 1999).

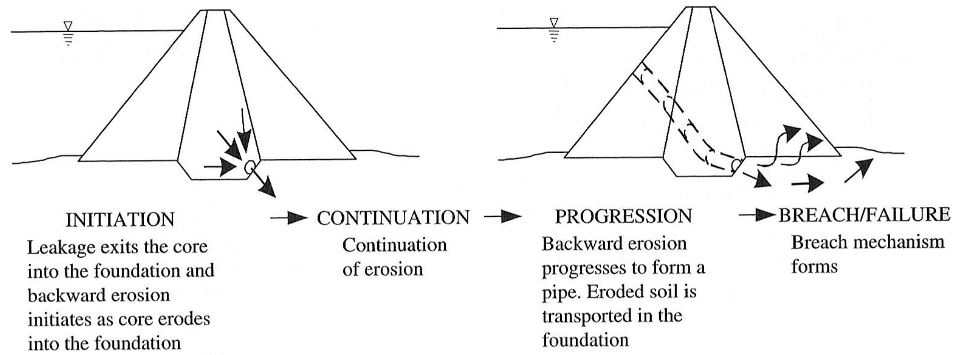

Figure 3.3 Conceptual model for development of failure by piping from the embankment into the foundation (Foster, 1999).

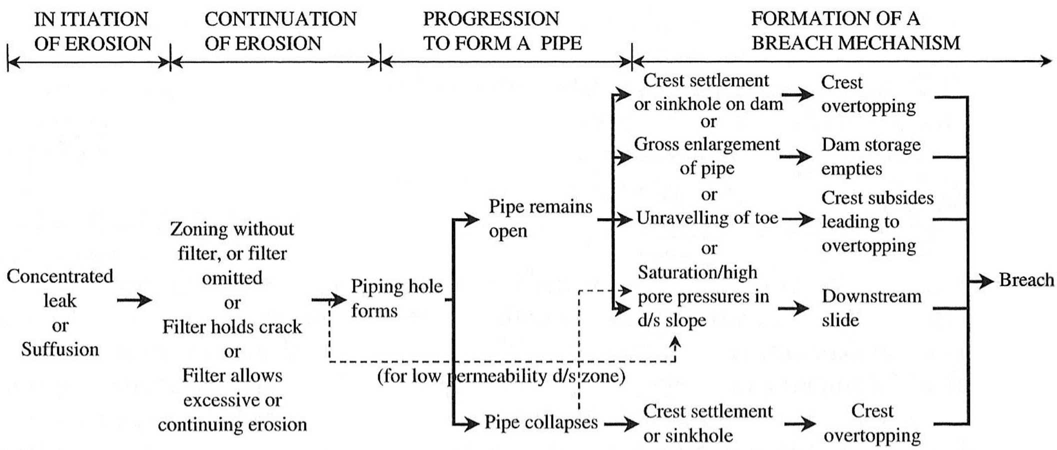

Figure 3.4 Failure path diagram for failure by piping through the embankment (Foster, 1999).

3.1. Uneven settlements and hydraulic fracturing

Embankment dams are subjected to deformations as compression, extension and shear distortion. Normally, embankment dams are constructed in zones of different materials. Each zone has an important function for the structure as a whole. Different compressibilities of the various zones might lead to internal redistribution of stresses compared to the distribution in a homogeneous embankment. Deformations in a dam might lead to cracks or "soft zones" where internal erosion can be initiated. Fill materials have in general low tensile strength, so cracking can result from even very small tensile stresses. As will be seen below, there is often various types of uneven settlement that is the main cause for cracking. Even if the deformations are of too low magnitude and character as to cause cracking immediately, cracking may occur later on by hydraulic fracturing. Drying and shrinkage can also cause low internal embankment pressures, Sherard (1986).

In Kjærnsli *et al.* (1992), results from measurements of displacements and stresses in dams are presented for some case studies. All the dams included have been sufficiently well instrumented and monitored to give a full account of developments within the dam during construction, the simultaneous or subsequent impoundment as well as during operation. Measurements in dams with central earth core point out that the greatest deformations, and the corresponding redistribution of stresses, take place during the construction of the dam and not during the first filling of the reservoir. Impoundment is, however, the main reason for transverse displacement of the core. Longitudinal displacements tend to be directed towards the centre of the valley bottom implying that horizontal compressional stresses increase in the central regions and decrease towards the abutments. Even if most of the settlements arise during the construction phase of a dam, settlements will continue for many years after the dam is completed. To maintain freeboard during the operation period, the crest of the dam is commonly built a little bit higher than the design level (a camber is provided). Detailed information about construction settlement and post construction settlement magnitudes in embankment materials can be found in e.g. Fell *et al.* (2005).

Figure 3.5 to 3.10 present several schematic examples of differential settlement induced cracks in the embankment: transverse differential settlement cracks, longitudinal differential settlement cracks, internal differential settlement cracks and potential cracks between the dam fill and the spillway wall. If internal erosion through the core zone is initiated by a crack in the transversal direction, it may create a problem to the safety of the dam. Longitudinal cracks oriented in a foremost vertical direction in a dam, do not have as obvious effect on leakage as transverse cracks. However, in some cases, longitudinal cracks can be the route of leakage if these are connected with transverse cracks. Causes of cracking in embankment dams can be further studied in e.g. Kim *et al.* (2004), Fell *et al.* (2005), Zhang and Du (1997), Ng and Small (1999), Sherard (1986) as well as in Lo and Kaniaru (1990).

In Figure 3.5 two common types of transverse differential settlement cracks are shown. The cracks are located at the dam crest. Since the deformations are directed towards the centre of the valley bottom, horizontal compression stresses increase in the middle regions and decrease closer to the abutments, where extension and cracking might occur. Obviously, larger changes in the abutment slope increase the risk for tensile stresses and cracking in areas above the points where the variations occur, as in Figure 3.5b.

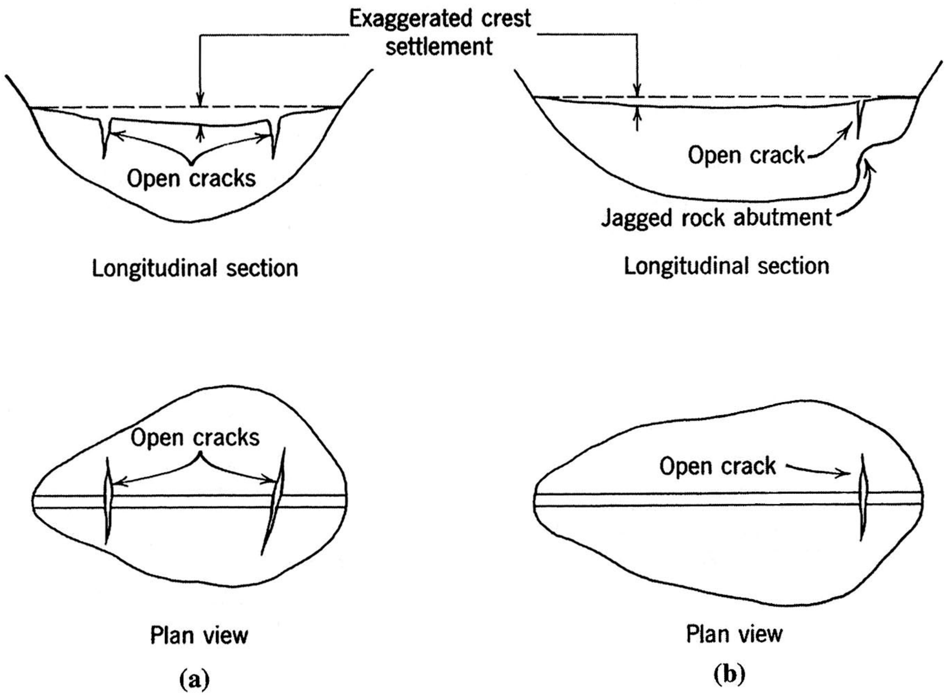

Figure 3.5 Ordinary transverse differential settlement cracks.

The core in Figure 3.6 is less compressible than the rest of the embankment and cracks develop at the boundary between the core and the rest of the fills. Further, in this situation, the weight of the fills on each side of the core is partly transferred to the stiffer core, thus increasing the stresses of the core.

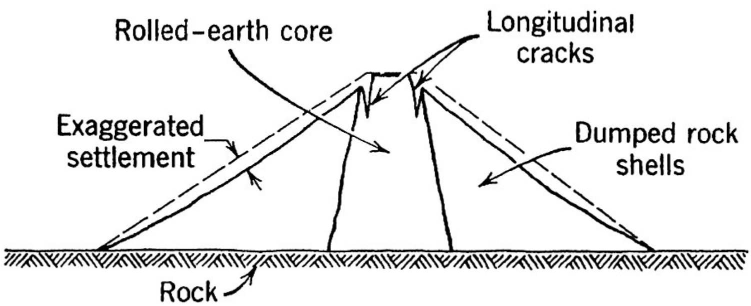

Figure 3.6 Longitudinal differential settlement cracks between embankment zones (Sherard *et al.*, 1963).

Seepage through a permeable natural foundation can be reduced by constructing a cutoff with low permeability through the permeable material. When the depth of the permeable soil is relatively small, a cutoff trench can be excavated through the permeable layer and filled with relatively incompressible rolled earth, as in Figure 3.7. The natural foundation on both sides of the cutoff trench is more compressible than the rolled earth fill. Therefore the settlements will be larger at the upstream and downstream slopes than at the dam crest region and cracks as shown in Figure 3.7 might appear.

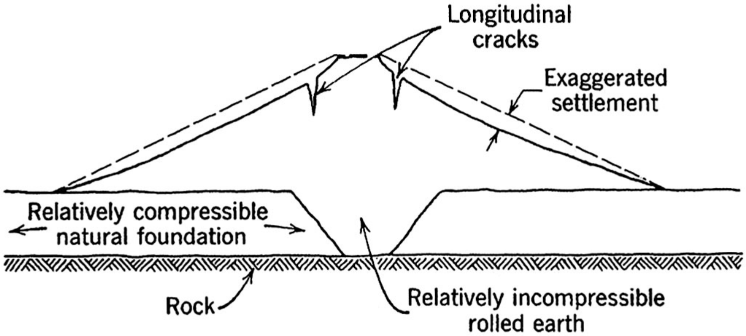

Figure 3.7 Longitudinal differential settlement cracks over foundation discontinuities (Sherard *et al.*, 1963).

Figure 3.8 presents three cases for which internal differential settlement cracks can arise. In the example in Figure 3.8a the core is less stiff than the fills on each side. The result of this is that the core settles more than the other materials and the weight of the core are partially transferred to the adjacent zones. The core is thus "hanging" on the more rigid zones, a phenomenon called arching, and as a consequence of this the vertical stress on horizontal planes in the core will be reduced compared to homogeneous embankment conditions. An increase in water pressure can lead to a fracture, as the water pressure lower the effective stresses in the core even further. If the vertical effective stress becomes low enough for cracks to develop, cracks on the upstream face of the core might initiate concentrated leak while cracks on the downstream face of the core might initiate backward erosion. The case in Figure 3.8b displays tension cracks above a very compressible clay lens in the foundation. Apparently, it is important to perform careful geotechnical investigations for such a case. Figure 3.8c shows tension cracks in the soil near the top of a concrete diaphragm wall illustrating that for dam applications, concrete is often too rigid. When the soil mass surrounding the wall settles, additional load will be transferred to the wall by negative skin friction. If the wall is rigid, this can also cause crushing of the wall and penetration of the wall into the dam fill. One way to overcome this problem is to use plastic concrete that is easier to deform (Fell *et al.*, 2005).

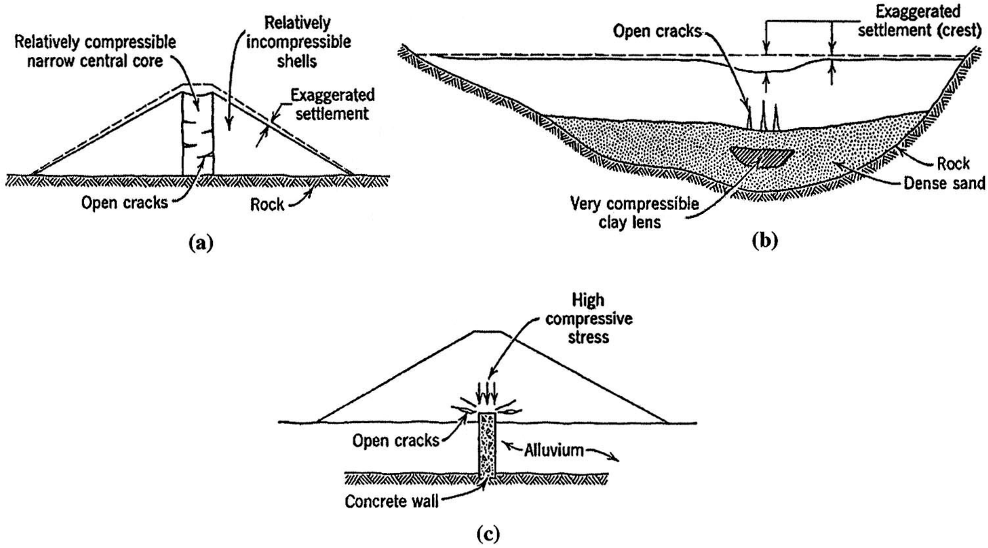

Figure 3.8 Typical internal differential settlement cracks in embankments (Sherard *et al.*, 1963).

The interface between earthfill and concrete structures is often a potential region for cracking and internal erosion processes. In Figure 3.9 two similar situations are demonstrated, where settlement of the dam fill can cause a crack to develop between the spillway retaining wall and the embankment fill.

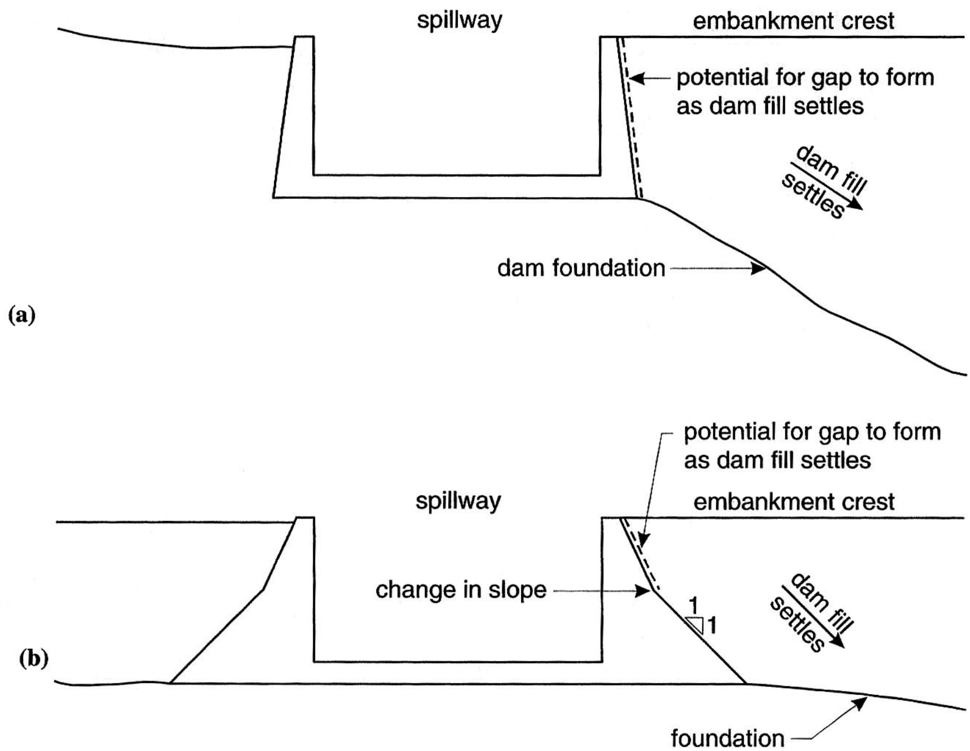

Figure 3.9 Situations where a crack might form between the dam fill and the spillway wall: (a) steep foundation adjacent to spillway wall; (b) change in slope of the retaining wall (Fell *et al.*, 2005).

Differential settlement cracks can expand and develop further by the hydraulic fracturing process. If differential settlements have lead to "soft zones" rather than cracks, hydraulic fracturing might later on form cracks in such "soft zones". Hydraulic fracturing is the process when water pressure splits a soil with a low permeability, which might happen if the minor total principal stress at any point within the soil becomes equal to the pore water pressure at the same level (then the minor effective principal stress reduces to zero). Fracturing may proceed continuously following roughly the plane of the minor principal stress. For hydraulic fracturing to take place, the adjacent material must be sufficiently impervious so that water can wedge its way along opening cracks more rapidly than its permeates the pores. Therefore, hydraulic fracturing occurs only in rather impervious materials, such as a dam core of clay or moraine (Sherard, 1986) and not in coarse pervious materials such as gravel and stone.

In a central core earth and rockfill dam, internal stress redistribution might takes place in both directions, i.e., by transverse arching of the core between the upstream and downstream shells (Figure 3.8a) and longitudinally between dam sections of different height (Figures 3.5 and 3.10). A critical zone for potential leaks is over the abutments, where longitudinal stretching of the dam and hydraulic fracturing are most probable. The internal embankment stress on the transverse planes marked with a dotted line in Figure 3.10 can be expected to be low and a leak on either of these planes is particularly likely.

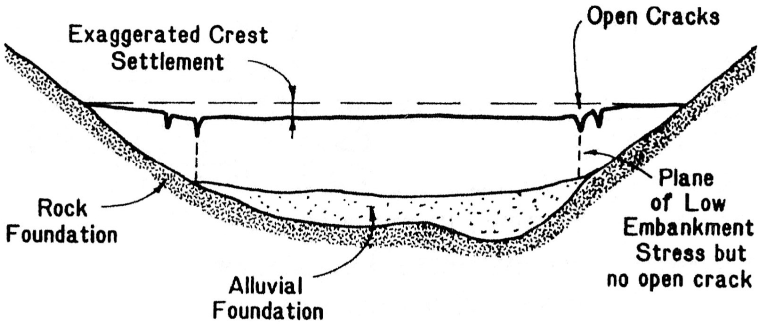

Figure 3.10 Planes where hydraulic fracturing is likely to occur (Sherard, 1986)

For a high central core dam, with rockfill or gravel shells, the conditions for stress transfer and hydraulic fracturing are commonly more severe than in relatively low homogenous dams. The reason for this is that the impervious section, the core, is relatively thinner and the total settlements are greater (Sherard, 1986). It should be noticed that a thin vertical core is more sensitive to fracture than a sloping core where the weight of the upstream rockfill shell helps to keep the core in compression.

Zhang and Du (1997) studied the effects of abutment slopes on the performance of high central core earth and rockfill dams by three-dimensional nonlinear finite element modelling. It was e.g. concluded that: ”(1) As a result of the longitudinal soil arching, the steeper an abutment slope is, the more susceptible to hydraulic fracturing the core will be. (2) Large changes in abutment slopes will result in large tensile stresses in the core contact areas and therefore should be avoided. (3) Application of a 5 m thick high-plasticity clay in the core contact areas may markedly decrease the tensile stresses near the interface, and reduce the risk of contact cracking because of its good strain hardening performance after yielding. (4) The compaction of the core is shown most effectively in improving the stress-strain conditions in the core. The risk of hydraulic fracturing can be greatly alleviated if the gradation and compaction of the core material are properly carried out.”

It was for a long time not completely accepted in the profession that concentrated leaks can develop through dams by hydraulic fracturing. This could be because hydraulic fractures cannot be visually observed since they are inside the dam, erosion processes destroys evidence of them and different interested parties, normally, have different vested interests in the result of dam incident investigations (Sherard, 1986). In the paper by Sherard (1986) evidence is gathered to support the conclusion that concentrated leaks by hydraulic fracturing occur frequently without being noticed, even in dams not exposed to large differential settlement. The evidence is presented in the following main categories: ”(1) concentrated leaks which appear soon after the first reservoir filling through well-constructed homogeneous dams with no internal filter or drain, sometimes leading to breaching failure; (2) erosive leaks through central core dams with inadequate filters, usually occurring soon after the first relatively rapid reservoir filling; (3) records of well constructed and designed central core dams in which nearly full reservoir pressure was measured in piezometers at the downstream face of the core, and in which exploratory borings in the dam made to study this condition showed conclusively that near horizontal, water-filled cracks existed in the core; (4) FEM calculations showing that only relatively small differential settlements will easily create stress conditions in most dams under ordinary conditions which will allow hydraulic fracturing to occur; and (5) the discovery of ´wet seams´ inside impervious dam sections, with water content higher than can be accounted for reasonably by any other mechanism than the entry of water into an open crack.” Different scenarios that might take place in an embankment dam after initial leaks have developed by hydraulic fracturing are also thoroughly described in the paper by Sherard (1986). Notice, however, that the mechanical background to hydraulic fracturing implies that the initiation of cracks by hydraulic fracturing has nothing to do with the erodibility of the embankment material and that the development of initial leaks is not limited to dams with inadequate filters.

In numerical software, it is obviously vital to properly model hydraulic fracturing since this phenomenon might initiate the erosion processes. Lo and Kaniaru (1990) derived an analytical expression for prediction of the hydraulic fracturing pressure

$$u_f = C\cos\phi + (1+\sin\phi)\sigma_t/2 + (1+\sin\phi)\sigma_H \tag{3.1}$$

where u_f is the hydraulic fracturing pressure, σ_t is the tensile strength, σ_H the total minor principal stress, C is the cohesion and ϕ is the angle of internal friction. The expression in equation 3.1 was also evaluated against experimental hydraulic fracturing tests. It was found that the hydraulic fracturing pressure depends on the type of test performed and good agreement against the theoretical result was obtained only for one type of test; saturated-consolidated hydraulic fracturing tests. In numerical modelling, constitutive relations have to be general, therefore equation 3.1 should not be implemented without modifications; it might, for instance, be appropriate to try to reformulate the expression as a function of effective stresses instead of total stresses.

3.2. Methods to examine and measure internal erosion

The statistics of dam failures and accidents reveal that it is essential to establish systems for monitoring and surveillance of dams. Fell *et al.* (2005) states, "it is apparent that many accidents would have become failures if they had not been detected by monitoring and surveillance and some action taken". The amount of monitoring and surveillance that is necessary depends on a number of factors, such as: the consequences of a failure, the type of dam and its size, known deficiencies, the age of the dam, the reservoir level etc. (Fell *et al.*, 2005). It is especially important to have sufficient inspection during first filling of the reservoir and when the reservoir level is in the vicinity of historic high levels. Some common warning signs during piping incidents in embankment dams are listed in Fell *et al.* (2005) as: increase in leakage, muddy leakage, sinkholes, settlements, cracking, whirlpool in reservoir, increase in pore pressure and sand boils. According to Fell *et al.* (2005), the most important detection methods for identification of deterioration of embankment dams are, in order of efficiency: inspection by trained observers, seepage flow measurements, vertical and horizontal surface displacement measurements, pore pressure measurements, reservoir water level and rainfall measurements (as these influence the other factors) as well as internal deformation measurements.

Regular visual inspections are most important in a dam surveillance program. Historically, damage has mainly been detected by direct observation. This is also the case for Swedish dams, see Nilsson (1995). By visual inspection of the dam and its surrounding, deteriorations that are not easily detected by instruments (fissures, leakages, dam spots etc.) might be found. The main drawback with visual inspection to detect internal erosion is that the erosion process might have progressed much too far when it becomes visual.

Seepage flow information, by visual surveillance or measurements, is one of the best indicators of the effectiveness of the impervious elements of a dam. If the seepage water contains dispersed soil particles, which can be noticed by discolouration of the water, immediate attention is required since it might indicate an internal erosion process. To measure the quantity of seepage, special seepage collection and measuring systems can be built in to a dam, see e.g. Fell *et al.* (2005) or Kjærnsli *et al.* (1992).

Results from surface displacement measurements of the dam embankment can be a useful indication for developing problems that might be associated with internal erosion, such as:

local larger settlements, slope instability etc. In order to measure horizontal as well as vertical displacements a network of markers is installed on the surface of the dam. To determine the coordinates of the markers by triangulation reference pillars are placed on firm ground on the sides of the dam, Kjærnsli *et al.* (1992).

Measurements of pore pressures in the embankment as well as the foundation of a dam can provide important information for stability analyses, detect potential ”blow-up” conditions in the foundation and observe unusual seepage pressures. Different types of instruments can be utilised to measure this pressures (Fell *et al.*, 2005), as: observation wells, Casagrande type piezometers, hydraulic piezometers, pneumatic piezometers and vibrating wire piezometers. Much engineering skill is needed to select appropriate types of instruments and design where to install them to obtain an overall picture of the pore pressures. Normally, it is not practically or economically feasible to install the instruments so closely together that the pore pressure measurements can be expected to capture local deviations or impairments in the dam, Kjærnsli *et al.* (1992). In that case, pore pressure measurements will obviously not be a reliable precursor to internal erosion and piping.

Internal displacements and deformations are often monitored on larger embankment dams. This can give information about long-term deformations as well as probable arching effects and extensional strains that might cause cracking followed by internal erosion. An overview of the types of instruments that might be used can be found in e.g. Fell *et al.* (2005) or Kjærnsli *et al.* (1992). Vertical displacements can be measured by installing a telescopic tube system with cross-arm anchors or magnetic markers. Lateral movements can be measured by inclinometers. An inclinometer can be installed at any inclination. Displacements resulting from extension can be measured by extensometers, which normally are installed horizontally in the longitudinal direction of the dam.

Installation of instruments for monitoring and surveillance into the dam core results in a potential weakness, since internal erosion and piping might initiate along the instruments installed. The mechanisms for internal erosion are similar as for piping along a conduit, discussed in Fell *et al.* (2005). From internal erosion and piping perspectives it is more dangerous to make penetrations through the core in the transverse direction. In order to form an opinion if instruments should be installed in the core or not, the value of the information gathered must be weighted against the risk to damage the core. Some senior dam engineers, as the authors in the reference Fell *et al.* (2005), seems to be generally reluctant to bring any potential weaknesses into a dam core.

In Fell *et al.* (2003) the capability of standard methods of surveillance and monitoring to detect internal erosion and piping is discussed. There point of view in the matter might be best summarized by the following quotation from the paper: "Many dam owners rely heavily on surveillance, and monitoring of pore pressures and seepage to warn of potential internal erosion and piping problems for many older dams which were not provided with filters designed and constructed to control internal erosion. It has been the authors perception that many may have placed an over reliance on monitoring and surveillance rather than carrying out remedial works, given the potential for rapid failures, and the likelihood that some (or most) modes of initiation of internal erosion may not be easily detectable. Detection may however be possible once erosion has progressed, provided the monitoring is well designed, and read frequently enough."

A potential problem that should be addressed in this context is that standard instruments for monitoring and surveillance normally have a shorter life length than the dam structure itself. If instruments cease to function, they must be replaced, which is a process that might cause damage to the dam during drilling and backfill grouting operations or the dam owner must go on without the information.

An interesting complement to standard detection methods is geophysical methods, e.g. temperature measurements, resistivity measurements, streaming potential measurements, acoustic methods, seismic methods and electromagnetic methods. Many geophysical methods can be classified as non-destructive (or just cause minor intrusions) and, when that is not the case, existing standpipes might be possible to utilise for the measurements. Geophysical methods have the theoretical potential to register small changes in the seepage flow rate, globally, along the entire dam and thus detect internal erosion at an early stage. The essential objective for the measurements is to detect changes and the absolute accuracy becomes less important as compared to the relative accuracy. In some cases, geophysical methods are most suitable for regular measurements since seasonal variations can be considered then, single measurements at investigations might be difficult to evaluate if too many of the variables are unknown or uncertain (Johansson, 1997). Further, it is advantageous to use different geophysical methods together as far as possible in order to improve the quality and reliability of the evaluation, Johansson *et al.* (2004).

Still much research and development are needed to be able to trust on the results from geophysical methods if they are to be utilised for permanent monitoring and surveillance of dam structures, Elforsk (2001). For example, it is essential to understand more about the accuracy of which geophysical measuring techniques can be expected to capture measurable parameters related to internal erosion (e.g. density, seepage flow, hydraulic conductivity, temperature, seismic velocity, dielectricity and resistivity) and how such measurable parameters are affected by the mechanical processes of internal erosion, see e.g. Johansson (1997) and Elforsk (2001).

Notice, when internal erosion processes can be successfully modelled numerically and the behaviour is included in general software, such software might be utilised to study how sensitive measurable parameters in geophysical methods are to internal erosion processes in a dam structure. In general, it seems like numerical modelling is a great complement to geophysical methods in order to detect internal erosion at an early stage. A vision can even be to use measurements and numerical tools in-situ for direct control of the dam safety.

4. EMBANKMENT DAM FAILURES AND ACCIDENTS

It is often difficult to precisely determine the cause to a dam accident or failure. Several types of processes might be involved in an accident and multiple modes in a failure. Internal erosion has as previously mentioned a tendency to destroy evidence of initial leaks which might have existed. As Sherard (1986) noticed, ”another common problem is that different interested parties have different vested interests in the outcome of the investigations. It is natural that the designer desires to show that the design was not at fault, that the engineer who controlled the construction wants to show that the construction supervision was done carefully, and that the owner wants to establish that someone is clearly responsible, etc.” This might obstruct the investigation process and lead to that definite conclusions, agreed upon by all parties, and are not reached. However, Peck (1980) states that a failure of a dam is seldom the consequence of a single shortcoming. ”Usually there is at least one other defect or deficiency, and the failure occur where two or more coincide. This inference supports the principle of designing to provide defence in depth, the ´belt and suspenders´ principle long advocated by Arthur Casagrande. It postulates that if any defensive element in the dam or its foundation should fail to serve its function, there must be one or more additional defensive measures to take its place.” Still more research in the fields of internal erosion processes as well as dam monitoring and surveillance might provide the knowledge necessary for a better understanding of the reasons for dam incidents.

A statistical analysis of embankment dam failures and accidents is thoroughly presented in Foster *et al.* (2000a), based on a large database of embankment dam incidents as well as a population database for embankment dams. The study is specifically focusing on internal erosion and slope stability problems. The result provides useful insights into the factors which contribute to dam incidents.

In Table 4.1 overall statistics of failure for different failure modes is presented. The historical average frequency of failure is estimated to be 1.2 % over the life of the dam. Failure by overtopping and piping are the most common modes of failure while failure by slides is less common. In the case of piping failure, the incidence of piping through the embankment is two times higher than piping through the foundation and twenty times higher than piping from the embankment into the foundation. Downstream slide failures are more common than upstream ones. Most of the failures occur during operation of the dam. Further, it was noticed in Foster *et al.* (2000a) that about half of all piping failures through the embankment are associated with the presence of conduits. The different modes of piping associated with conduits are: piping into the conduit, along and above the conduit or out of the conduit (Fell *et al.*, 2005).

Foster *et al.* (2000a) studied the factors affecting the frequency of incidents of: piping through the embankment, piping through the foundation, piping from the embankment into the foundation, downstream sliding and upstream sliding. All types of incidents were examined with regard to dam characteristics as e.g. dam zoning, year of construction, dam heights, filters, core soil types, compaction, location of conduits, foundation geology and foundation cutoff. The paper contains a great number of important observations and is highly recommended for further reading.

In Foster *et al.* (2000b) the statistical data presented in Foster *et al.* (2000a) is utilized to estimate the relative likelihood of failure of embankment dams by internal erosion and piping. The method described in Foster *et al.* (2000b) is called the University of New South Wales (UNSW) method. In the method ”the likelihood of failure of a dam by piping is estimated by adjusting the historical frequency of piping failure by weighting factors which take into account the dam zoning, filters, age of the dam, core soil types, compaction, foundation geology, dam performance, and monitoring and surveillance”. The UNSW method ”is only suitable for preliminary assessments, as a ranking method for portfolio risk assessments to identify which dams to prioritise for more detailed studies”. The sequence of events leading to failure by piping can be divided into: initiation, continuation, progression and breach. In the UNSW method it is not possible to assess what influence each of the weighting factors has on the different events. It is recommended in Foster *et al.* (2000b) that an event tree method is used to obtain a better understanding of how each of the factors influences initiation, continuation, progression and breach. Use of event trees to estimate the probability of failure of embankment dams by internal erosion and piping is discussed in Foster and Fell (2000). The recommendation is to compare results from event tree analyses with results from the UNSW method to find out if there are large differences. If this is the case, the reasons to it should be examined further.

The time to develop failure and the warning signs which may be evident prior to failure are important factors on assessing whether actions to prevent failure are possible or what warning time will be available for evacuation of the population at risk downstream of the dam. In Fell *et al.* (2003) ”a method is presented for the approximate estimation of the time for progression of internal erosion and piping, and development of a breach leading to failure in embankment dams and their foundations. The method accounts for the nature of the soils in the dam core, the foundation, and the materials in the downstream zone of the dam. Guidance is also provided on the detectability of internal erosion and piping, taking account of the mechanism of initiation, continuation, and progression to form a breach, for internal erosion and piping in the embankment, the foundation and from the embankment to foundation”. The development of piping is often a very fast process, leaving not much time for proper actions. The case studies of piping failures and accidents included in Foster *et al.* (2000b) revealed that ”in the majority of failures, breaching of the dam occurred within 12 h from initial visual indication of piping developing, and in many cases this took less than 6 h”. About two thirds of the failures occurred on first filling or in the first 5 years of operation. For the piping accidents it was found that ”the emergency situation often lasted several days, with piping reaching a limiting condition, allowing sufficient time to draw the reservoir down or carry out remedial works to prevent breaching”.

Fell *et al.* (2003) noticed ”that in most cases it has not been possible to identify the time of initiation of erosion, and the first signs of erosion tend to be at the progression phase, with a concentrated, often muddy leak”. The problem to detect that internal erosion has been initiated relates to the typical processes of initiation: backward erosion, concentrated leak and suffusion. Fell *et al.* (2003) points out that backward erosion ”would not necessarily be expected to be preceded by large increases in seepage during the time the erosion is gradually working back from the downstream exit point. When the erosion has progressed to within a short distance of the reservoir/foundation interface, it breaks through rapidly or very rapidly”. Concentrated leak piping ”could be expected to initiate very rapidly or rapidly once the reservoir level reaches the critical level at which erosion begins in cracks or high permeability zones; or the critical level at which hydraulic fracture initiates”. Suffusion ”is likely to be a more slowly developing process, accompanied by more gradual increases in seepage, and changes in pore pressure with time”. Further, it is much to learn about dam failures and accidents by reading case histories, see e.g. Vogel (2004), Watts *et al.* (2002) and Von Thun (1996).

Table 4.1 Overall failure statistics for large embankment dams up to 1986, excluding dams constructed in Japan pre-1930 and in China (Foster *et al.*, 2000a).

	No. of cases		% failures (where known)		Average frequency of failure ($\times 10^{-3}$)	
Mode of failure	All failures	Failures in operation	All failures	Failures in operation	All failures	Failures in operation
Overtopping and appurtenant						
Overtopping	46	40	35.9	34.2	4.1	3.6
Spillway–gate	16	15	12.5	12.8	1.4	1.3
Subtotal	62	55	48.4	47.0	5.5	4.9
Piping						
Through embankment	39	38	30.5	32.5	3.5	3.4
Through foundation	19	18	14.8	15.4	1.7	1.6
From embankment into foundation	2	2	1.6	1.7	0.18	0.18
Subtotal	59	57	46.1	48.7	5.3	5.1
Slides						
Downstream	6	4	4.7	3.4	0.54	0.36
Upstream	1	1	0.8	0.9	0.09	0.09
Subtotal	7	5	5.5	4.3	0.63	0.45
Earthquake–liquefaction	2	2	1.6	1.7	0.18	0.18
Unknown mode	8	7				
Total no. of failures	136	124			12.2 (1.2%)	11.1 (1.1%)
Total no. of failures where mode of failure known	128	117				
No. of embankment dams	11 192	11 192				

Note: Subtotals and totals do not necessarily sum to 100%, as some failures were classified as multiple modes of failure.

4.1. Internal erosion susceptibility of glacial till

Foster *et al.* (2000a) have studied if the geological origin of the core has any influence on the incidence of piping. It was concluded ”that dams with core materials of glacial origin have experienced more piping accidents (i.e., initiation of piping) than those built from other materials but have experienced fewer failures”. In many of the incidents have sinkholes appeared on the crests and slopes of the embankment dams. Milligan (1976) discusses geotechnical aspects of glacial tills and states that ”because of the dense and impermeable nature of many tills, the term ´till´ is generally considered to represent an ´ideal´ material for the foundations of heavy structures and for rolled earth fill as in dams. The heterogeneous nature of till deposits in situ and their wide variation in engineering properties, make till, on occasion, far from a ´ideal´ material.” According to Sherard (1979) has the troubles occurred in dams comprised of remarkably similar coarse, broadly graded soils that are frequently of glacial origin, within the typical gradation shown in Figure 4.1. Milligan (2003) mentions that ”the characteristic feature of these glacial tills is that they are reasonably similar in gradation throughout the world, but the character of the fines content depends on the source rock”. Case studies of dam incidents could be found in e.g. Milligan (2003), Norstedt and Nilsson (1997) and Sherard (1986). Many completely successful dam cores have been constructed of coarse grained, broadly graded glacial soils. Problems seem to develop only under unusual circumstances.

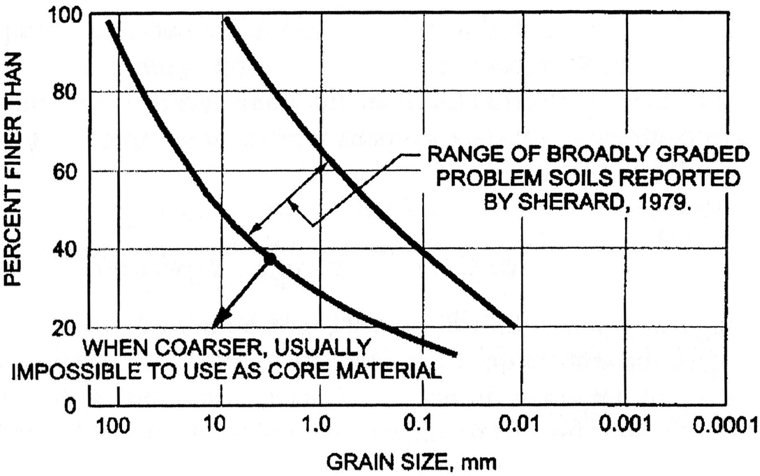

Figure 4.1 Problem soils according to Sherard (Milligan, 2003).

A number of explanations to the internal erosion susceptibility of glacial till can be found in the literature. Sherard (1979) states that the problems ”can be explained by the fact that the broadly graded coarse soils are internally unstable". The self filtering of the base soil does not work satisfactory and the fine particles will erode through the coarse particles. The general process leading to this type of incidents was described by Sherard (1979) as follows: "a concentrated leak developed through the core which caused a type of internal erosion in which the soil fines are eroded selectively and carried out of the core, leaving the coarse sand and gravel particles behind to act as a pervious drain. The volume of fine material eroded was larger than the volume of the void spaces between the coarser soil particles causing progressive collapse of the material above the initial leakage channel, which action finally reached the dam surface as manifested by the sinkhole or crater." Observe that this implies that the coarser particles are floating in a matrix of fines, which is common for glacial moraine (Sherard and Dunnigan, 1989), and the fines are carrying the embankment compressive stresses. In addition, it was noticed by Sherard (1979) that "probably sinkholes in the downstream shell are more hazardous than in the upstream shell if the dam has a central core, because the sinkhole could remove lateral support for the core allowing it to move locally downstream under the water pressure". Milligan (2003) means that "many tills, except when badly gap-graded, are highly stable to seepage in situ" and explains that "internal erosion (piping) of broadly graded till core materials may result more often as a consequence of segregation during placement". Foster *et al.* (2000a) writes that "it is possible that these glacial soils are more erodible because their fine silt and clay size fractions are finely ground rock, rather than more common clay minerals". Ravaska (1997) obtained test results that pointed out that "the higher the specific surface area is the better the capability of the till to resist erosion". Milligan (2003) states that "most of the incidents, not exclusively, apparently occur in tills of lower plasticity". The term plasticity is a measure of the ability of a soil to undergo unrecoverable deformation without cracking and is therefore also important for the ability of the soil to resist internal erosion.

5. FILTER DESIGN

It is widely accepted that the safety of large embankment dams is mainly dependent on the reliability of the performance of their critical filters. Filters have two basic functions: prevent erosion of soil particles from the soil they are protecting (named the base soil) and allow drainage of seepage water. A summary of the chronological development of filter design criteria is found in ICOLD (1994). In Fell *et al.* (2005) a brief review of available methods for designing filters is presented. Fell *et al.* (2005) also recommend a modified method for designing critical filters with flow normal to the filter; the method is based on Sherard and Dunnigan (1985, 1989) as well as USDA-SCS (1994).

In general, filter design criteria are worked out with the base soils grouped under various categories (e.g. four categories) based on their fines content, and a different filtering criterion is given for each base soil category. The quantity D_{15F} (particle size of the filter material for which 15 % by weight is finer) is supposed to reflect the particle capture ability of the filter material. The basic idea is to design the particle size distribution of the filter so that the voids in the filter are sufficient small to prevent erosion of the base soil and the void size in the filter is controlled by the finer particles, for which D_{15F} is a measure (Sherard *et al.*, 1984a). In most filter design methods, the quantity D_{85B} (particle size of the base material for which 85 % by weight is finer) represents the particle size and the erosion parameter of the base soil. No sufficiently adequate physical explanation for the use of D_{85B} was found in the literature during the work with this literature survey (see e.g. Sherard *et al.*, 1984a as well as 1984b and Sherard & Dunnigan, 1989). As an example of a filtering criterion, it is prescribed in the recommended method in Fell *et al.* (2005) that for base soils of sand and gravel with less than 15 % material finer than 0.075 mm, the filter quantity D_{15F} must be smaller or equal to $4 \times D_{85B}$ (valid for base soil after regrading). The filter should be permeable enough for the seepage flow to pass through it without building up high pore pressures. This is normally accounted for in filter design by prescribing a smallest allowed value of the quotient D_{15F}/D_{15B} (e.g. $D_{15F}/D_{15B} > 4$ or 5 ensures a much higher permeability of the filter than of the base soil) and restrict the amount of fines in the filter. Low fines content is also desirable in order to avoid cracks in the filter. Internal instability or suffusion in the filter material should be avoided, a number of different criteria to handle this are proposed based on the analysis of the grain size distribution, see e.g. Fell *et al.* (2005). Segregation at placement of the filter material should be prevented, e.g. by specifying a maximum particle size. The issue of segregation is of high importance in the context of internal erosion, and discussed further in Section 5.1.

The current filter design criteria are mostly empirical, based on statistical correlations of experimental observations. These criteria are constantly being questioned and revised as long as new experimental data becomes available. Inherent in the design methods are for example the assumptions that percent fines content reflect the erodibility of the base soil, D_{85B} reflects the erosion parameters of the base soil and D_{15F} reflects the particle capture capacity of the filter material. According to Reddi and Kakuturu (2004) have these assumptions not been validated thus far. On the other hand, Sherard and Dunnigan (1989) express themselves like this: ”the filter boundary D_{15F} determined in the no erosion filter (NEF) test for each given impervious soil is a unique value, which can be considered a fundamental property of the soil in the same sense that the Atterberg limits or the effective shear strength envelope parameters are fundamental properties of impervious soil”. Anyway, it feels like more research in the field of filter design would be valuable since it is so important, and then with a physical sound mechanical approach instead of constantly applying empirical methods. A recent alternative filter criterion is proposed by Indraratna *et al.* (2007), based on a mathematical procedure to determine the controlling constriction (opening) size. The main idea is that it is the constriction sizes rather than the particle sizes that influence filtration. Constriction sizes have been studied before by e.g. Kenney *et al.* (1985).

If filters in existing dams do not satisfy the modern filter criteria there is always a potential of internal erosion. The filter performance mainly affects the continuation of erosion. There are many dams built with filters not meeting the accepted filter criteria. To exemplify, a study of case histories of piping incidents, discussed in Foster and Fell (2001), pointed out that the dams included in the study were ”generally constructed from the 1960s to 1970s, which coincides with a period when there was a trend away from the use of uniformly graded multiple filters and towards the use of a single filter of substantial width and broad gradation”. The filters in the study had wide gradings and low proportions of sand sizes, which can make them susceptible to segregation and potentially internally unstable. Dams with filters that do not fulfil modern design criteria rarely fail but damage to the dam in the form of sinkholes and large leakage is rather common. Foster and Fell (2001) presents a method for assessing embankment dam filters that do not satisfy design criteria. ”The method can be used to determine whether filters that are coarser than required by modern filter design criteria will eventually seal or experience continuing erosion leading to possible failure of the dam in the event piping initiates.” The method is empirical, based on laboratory test results and characteristics of dams that have experienced piping incidents. An interesting outcome of this study is that it seems like it is a considerable margin between no-erosion and continuing erosion criteria for most soils; except for fine base soils where the margin is much less (Fell *et al.*, 2005). This might help to explain why self-healing so frequently occur at dam incidents. Empirical filter design criteria do not normally take into account the variability in the particle size distributions of the base soil and filter. Mínguez *et al.* (2006) present a new general method for assessing the probability of fulfilling any empirical filter design criteria considering base soil and filter heterogeneity by means of first-order reliability methods. The method can be used by engineers to estimate the safety of existing filters in terms of probability of fulfilling their design criteria.

More research would be desirable in the field of ageing effects of dams and uncertainties of core/filter appearance after accidents, incidents etc. This is a difficult research subject since it is not easy to "look inside" a dam without damage it; detection with installations and geophysical methods might not be sufficient so the authors´ of this literature survey suggest numerical modelling as a complementary approach. Results from the no erosion filter tests reported by Sherard and Dunnigan (1989) gave interesting information about the soil behaviour at the interface between the base specimen and the filter. This result might at least partly reflect the real behaviour in a dam. The laboratory test setup of the no erosion filter test is shown in Figure 5.1. Series of this type of tests were made and a filter boundary size D^b_{15F} was found between successful and unsuccessful tests for each base soil in the study, where "successful" was defined as no visible erosion of the walls of the preformed hole in the base specimen and "unsuccessful" when the erosion was visible.

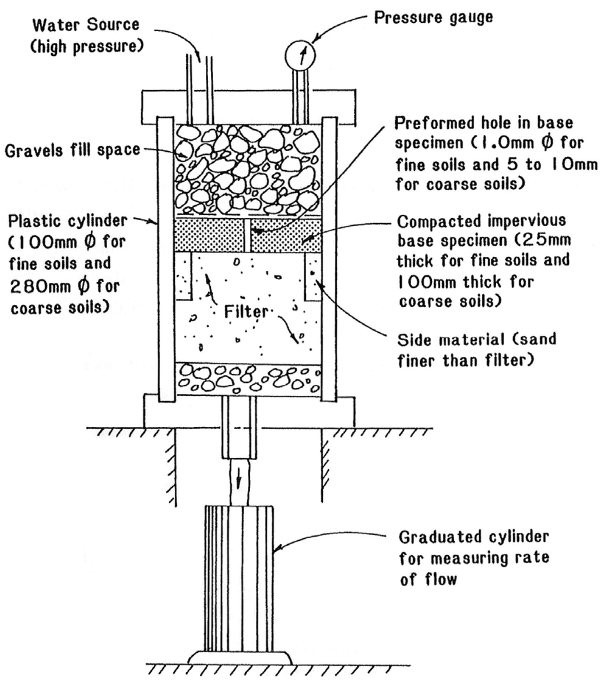

Figure 5.1 The no erosion filter test (Sherard & Dunnigan, 1989).

The condition observed at the end of successful tests on fine silt or clay as base soil (that have more than 85 % by weight of particles finer than the 0.075 mm sieve) is presented in Figure 5.2. The seal of the filter did not take place only at the small area at the end of the preformed leakage hole. The water flowing in the leakage hole turned 90° at the filter face and progressively eroded out radially from the hole, creating a thin slot and sealing the filter face over the entire diameter. In unsuccessful tests, for filters with D_{15F} sizes up to two to five times larger than D^{b}_{15F}, essentially the same thing happened; a thin slot appeared and the filter face was finally sealed, however, the preformed hole was enlarged since eroded material from the walls of the initial leakage channel was needed for the self healing.

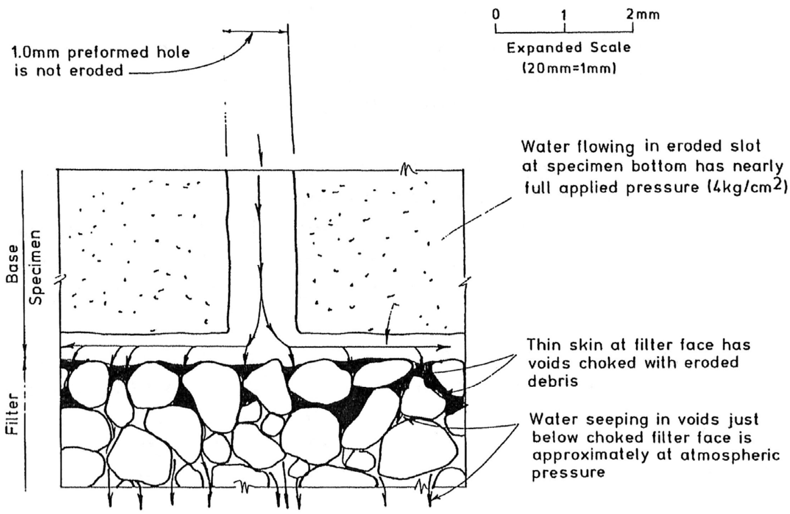

Figure 5.2 Condition at end of successful test on fine silts and clays (Sherard & Dunnigan, 1989).

The condition at the end observed on silty and clayey sands with low fine contents (15 % or less by weight finer than the 0.075 mm sieve) as base soil in a successful no erosion filter test is fundamentally different from the behaviour of the clays and silts previously discussed, see Figure 5.3. For the sandy soils with low contents of fines no eroded slot developed at the bottom of the base specimen. It was believed that the main reason for this was that the coarser particles are in contact and form a granular skeleton with high intergranular friction forces that resist movement of particles by erosion. In the unsuccessful tests on silty and clayey sands a large crater immediately developed through the base specimen.

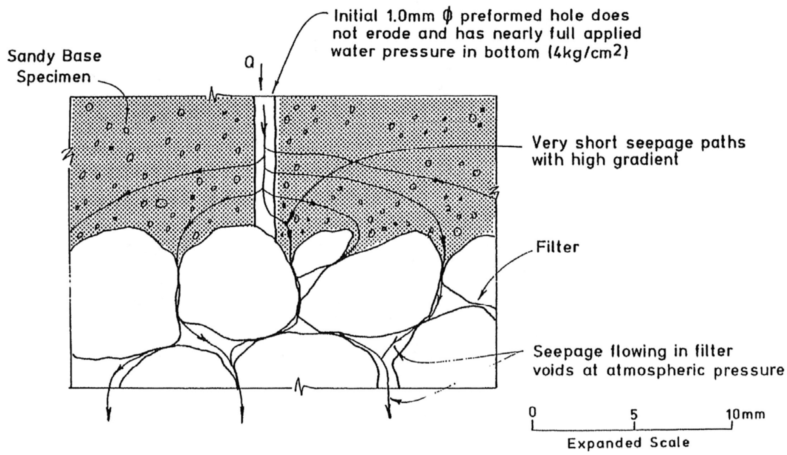

Figure 5.3 Condition at end of successful test on silty and clayey sands with low fine contents (Sherard & Dunnigan, 1989).

Reddi and Kakuturu (2004) proposed a mathematical method to analyse the self healing nature of concentrated leaks at core-filter interfaces. The relative degree of self healing of various filters for a given base soil can be studied with the method. An experimental method was also developed to determine the erosion properties of the base soil and obtain input values to the parameters of the mathematical model. It was noticed that the entire particle size distribution of the filter governs particle accumulation at the interface between the base soil and the filter. "It is suggested that a wide range of particle accumulation shapes (ranging from a rapid reduction in particle accumulation to a relatively uniform particle accumulation) is possible for soils when only D_{15F} of the filter is specified."

5.1. Segregation of filter materials

It has been noticed that dams with good filter performance have filters with characteristics that would tend to make them less susceptible to segregation, Foster and Fell (2001). Segregation is the process that causes separation of a graded soil material into finer and coarser zones. It might occur at placement of material by downslope discharge during dumping and spreading operations. The larger particles have a tendency to accumulate at the bottom of the slope. All broadly graded materials that do not have sufficient fines to provide some degree of cohesion are difficult to place without segregation. If a segregated zone of sufficient coarseness arises in the filter there is a risk for loss of fines from the core by internal erosion. Since it is the large soil particles that are most sensitive to segregation, it is especially important to restrict the maximum particle size included in a filter. Milligan (2003) warns that the issue of segregation is a critical uncertainty, which is generally poorly recognized and dealt with in dam engineering. In many projects the problem with segregation is handled by a declarative statement in the specifications telling that "materials must be placed without any segregation taking place". This is, however, in many practical cases very difficult to accomplish and the specifications thereby places an obligation on the contractor that cannot be met.

Kenney and Westland (1993) performed rotary-drum segregation tests to study the process of segregation in different granular soils, including granular filter materials. It was found that: 1) all dry soils consisting of sands and gravels will readily segregate; 2) the patterns of segregation are repeatable in repeated tests and the amount of segregation might be predictable; 3) water effectively prevents segregation of finer soils (e.g. sandy soils) while it has no influence on the segregation behaviour for coarser soils (e.g. gravels). Water in unsaturated soils reduces segregation because the sorting mechanism is suppressed by the increased effective stresses caused by the capillary tensions acting at points of contact between wetted particles. Soils with a high content of small particles are most strongly influenced by these effects of capillary stresses. In Figure 5.4, an approximate gradation limit for wetting effects is presented. Further, Kenney and Westland (1993) carried out tests at different water content and the results showed that the ability of water to inhibit segregation was not changed even at water contents equal to one half of the "field capacity" values.

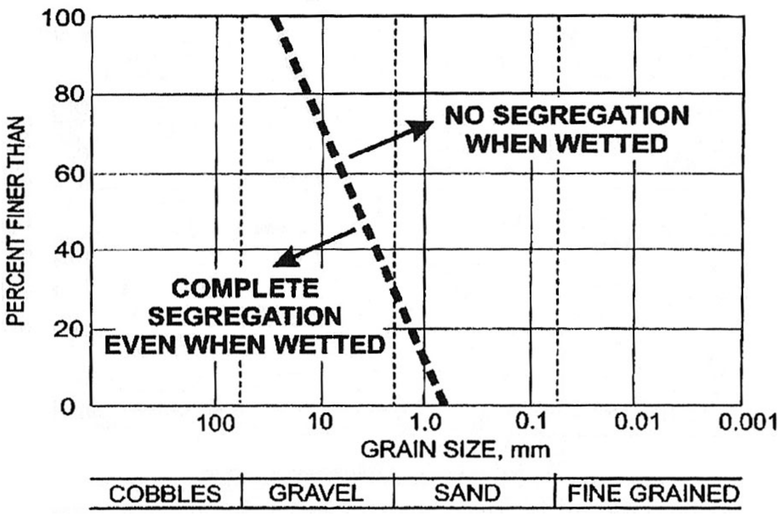

Figure 5.4 Approximate gradation limit for preventing segregation by wetting (Milligan, 2003).

Milligan (2003) concludes that ”much of the uncertainty related to appropriate filter design and placement may be largely resolved by specifying the use of wide, narrowly graded, sand-rich filters, of a gradation finer than the limit suggested in Figure 5.4 and placed wet in thin lifts. Then the specification clause that ´segregation will not be permitted´ is a practical reality.”

6. MICROMECHANICAL MODELS OF HYDRO DYNAMICAL LOADS ON SINGLE PARTICLES AND SYSTEM OF PARTICLES

Continuum formulations of flow through porous media often results in equations consisting of a few unknown parameters such as the permeability. The physical background of such parameters can often be traced to the detailed flow in the pores and it is therefore in place to study the flow on this level, as well. To start with we can scrutinize models for single particles. The viscous drag force on a single sphere in an infinite fluid at $Re < 1$ is, for instance, described by Stokes law:

$$f = 6\pi R \mu v \tag{6.1}$$

where R is the radius of the sphere and v the velocity of it. This expression was early extended by Oseen to be valid for Re up to about 10 according to Lamb (1932):

$$f = 6\pi R \mu v \left(1 + \frac{3}{16} \mathrm{Re} \right) \tag{6.2}$$

and later on by Happel and Brenner (1957) to be valid for the case of a sphere moving through a tube with diameter D to be:

$$f = 6\pi R \mu v \left(1 + 2.1 \frac{R}{D} \right). \tag{6.3}$$

The drag force has also been derived for particles with other geometries than spherical. For slender rods the drag force per unit length along the rod is, Emersleben (1925):

$$f = 4\pi \mu v \; . \tag{6.4}$$

While the following expression has been derived for flow perpendicular to the rod, Lamb (1932):

$$f = \frac{4\pi}{2 - \ln(\mathrm{Re})} \mu v \; . \tag{6.5}$$

The obvious conclusion is that the drag on a particle is linearly dependent on the averaged velocity as long as *Re* is less than unity. The importance of velocity then increases to 2nd order.

For very dense systems and at low *Re* the flow follows Darcy's law and the porous media can be characterised by their permeability through measurements or by the usage of permeability models based on, for instance, the detailed rod geometry: cf. Speilman and Goren (1968), Bear (1972), Scheidegger (1972), Sangani and Acrivos (1982), Jackson and James (1986), Dullien (1992), Wang *et al.* (1994) and Higdon and Ford (1996). Generally permeability models may be divided into the following models, phenomenological, conduit flow (Kozeny-Carman for instance), empirical, network, deterministic (directly from Navier-Stokes) and models based on flow around submerged objects (models proposed by Brenner and Brinkman, for instance) Dullien (1992).

The permeability is often represented by the Kozeny-Carman relation, according to:

$$K = \frac{1}{B}\frac{\varepsilon^3}{S^2(1-\varepsilon)^2} \tag{6.6}$$

where ε is the porosity, B a constant and S the specific particle surface. The Kozeny-Carman relation is based on a tube-like representation of a porous medium. The constant B has to be determined experimentally, but has never been determined explicitly, Williams *et al.* (1974). The Kozeny-Carman relation assumes an isotropic porous medium. For rods other equations have been derived that distinguish between the permeability along and perpendicular to the direction of the rods, according to:

$$K_{\parallel} = \frac{8}{c}\frac{\varepsilon^3}{(1-\varepsilon)^2}R^2 \tag{6.7}$$

$$K_{\perp} = C\left(\sqrt{\frac{\varepsilon_{\min}}{\varepsilon}} - 1\right)^{5/2} R^2 \tag{6.8}$$

where R is the radius of the rods and c and C are constants depending on the arrangement of the rods while ε_{min} denotes the minimum pore volume for this arrangement. Also in this case forces on individual particles can be calculated. Mei and Auriault (1991) derived, for instance, the average drag acting on a particle in a porous medium with fore-aft symmetry at small Reynolds number to be:

$$f = \mu v\left(k_0 + k_2 \,\mathrm{Re}^2\right) \tag{6.9}$$

for a two-dimensional array. Here k_0 and k_2 are non-dimensional constants to be determined which is, for instance, done for both ordered and random arrays of spheres by Hill *et al.* (2001). To summarize the forces on individual particles have been exploited for certain geometries and for a number of flow conditions. We however need to investigate further higher Reynolds number flows, more complex geometries and instationary conditions.

7. MICROMECHANICAL MODELS FOR FRICTION BETWEEN SINGLE GRAINS AND SYSTEM OF GRAINS FOR NON-COHESIVE SOILS

The forces outlined in the previous chapter must be balanced by gravity and forces that emanates from particle interactions for the dam to be stable. As a first, and most simple, criteria the size distribution of the particles being subjected to hydrodynamic pressure is compared to the pore size distribution of the porous medium. Hence the particles will move if they are small enough independent on magnitude of the force on it. In reality, however, the hydraulic forces must exceed particle interaction forces and or gravitational forces keeping the particles in place at normal conditions. The force due to gravitation is given from:

$$\overline{F}_{g_z} = mg\overline{n}_z \tag{7.1}$$

if gravity, g is directed along the z-axis and m is the mass of the particle. The type of the dominating particle interaction force is dependent on the typical size of the particles. For very small particles, around one micron and smaller cohesion dominates. For larger particles up to fractions of millimetres adhesion becomes the major mechanism for particle interaction while for even larger particles, that is in the range of one millimetre and larger friction forces are major mechanisms for the particle interaction. The static frictional force that is directly related to the normal forces on the particle is directed in the opposite direction to the applied forces and has a magnitude according to:

$$0 \leq F_{f_z} \leq \mu_s N \tag{7.2}$$

as long as the static friction factor, μ_s is isotropic and can be treated as a scalar. Once the drag force on the particle becomes higher than the expression on the right-hand side of this expression the magnitude of the friction force can be expressed as

$$F_{f_z} = \mu_k N \tag{7.3}$$

where the kinetic friction factor, μ_k often is smaller than μ_s. The friction factor in this form is usually determined by experiments.

8. CONCEPT FOR NUMERICAL MODELLING OF INTERNAL EROSION

In recent years, numerical methods have been used to an increasing degree for solving more complicated problems in geotechnical engineering. A wide range of problems have been analysed with the finite element method (FEM), the finite difference method (FDM) and more recently the boundary element method (BEM). By utilizing e.g. the finite element method, based on a continuum approach, stresses, strains, deformations and pore pressures in an embankment dam can be theoretically analysed and it might be possible to draw conclusions from that information about the initiation of internal erosion processes. An approach worth considering, when initiation is established, is to interpret internal erosion numerically as a type of localisation and describe the constitutive behaviour with micromechanical models in localised zones. The mathematical description of the strain localisation phenomenon is nowadays an intensive research area (see e.g. Adachi *et al.*, 1997) and, hopefully, models for strain localisation could give valuable input to the formulation of internal erosion localisation.

The failure of a soil material is usually accompanied by formation of narrow zones with highly localised deformation. The development of such localised deformation zones might cause significant stress redistribution and strength reduction by softening, which can lead to a progressively developing slip line that induce failure of the entire soil structure. A typical example in geotechnical engineering is the progressive development of the slip surface in a slope stability failure. With conventional finite element techniques is it difficult to capture deformation that is localised to narrow zones. If a very dense element mesh is used, it would be theoretically possible to capture the localised deformation. However, this is normally not practicable due to large computational costs. In addition, is the conventional FEM technique not suitable for describing the kinematics at localised failure and the constitutive laws included might not be representative for the material in the localised zone. There exist a lot of different strategies for numerical modelling of localisation, Tano (2001). In e.g. discrete crack formulations the shear bands are following the boundaries of the elements and a remeshing algorithm is necessary, when the path is not known in advance, in order to be able to get cracks in the proper directions. In formulations based on element-embedded discontinuities, e.g. the inner softening band method described by Tano (2001), remeshing is normally avoided and relatively large elements can be used.

The choice of a suitable strain localisation model for reformulation in to an internal erosion localisation model might not be an easy task. For example Tano (2001) write the following sentences about strain localisation models. "Many methods have been developed and their popularity is switching continuously. No method seems to lead to correct results for all types of localisation analyses. Some of the methods are mathematically elegant but may be impractical in large scale modelling due to the requirement of very dense finite element meshes."

One way forward is to develop a numerical model for internal erosion processes based on micromechanical soil properties and implement it mathematically as a type of localisation in e.g. finite element software. Besides the ordinary application for analysis of deformation and stability in geotechnical engineering projects, such a software could be useful for: increasing the knowledge about internal erosion processes, evaluating the risk for dam incidents caused by internal erosion, estimating the time for progression of internal erosion and piping, studying self-healing of leaks, changes in filter behaviour subsequent to particle accumulation and ageing effects in dams, analysing the amount of instrumentation needed in a dam and the proper location for monitoring and surveillance, designing dams etc. In Figure 8.1, numerical modelling of backward erosion treated as a localisation in the finite element method is illustrated.

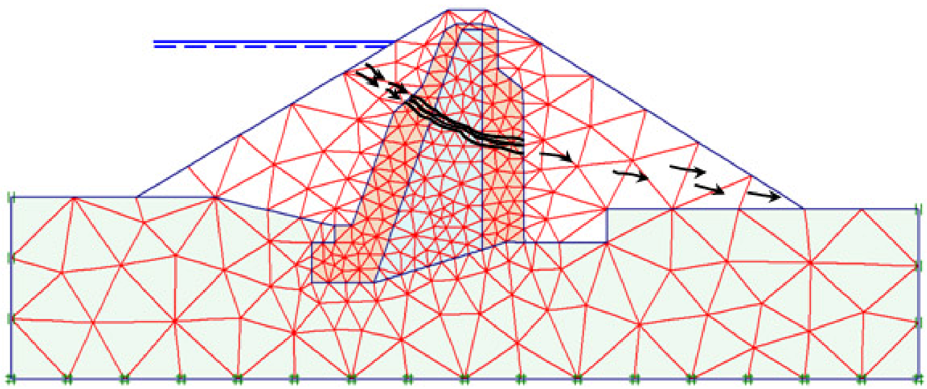

Figure 8.1 Numerical modelling of internal erosion.

A fundamental understanding of the physical processes involved in internal erosion is required to be able to successfully formulate models for the constitutive behaviour in localised zones. Theoretical formulations of a model for internal erosion should be complemented by performing laboratory tests. In the theoretical formulations, the following matters should be paid attention to:

- Performing a literature survey of strain localisation models that could be useful as a mathematical base for micromechanical internal erosion localisation models.

- Examining which physical quantities that are of importance and should be included in the internal erosion model.
- Formulating a micromechanical model for internal erosion (or utilising an existing, see Section 6 and 7).

It is of considerable importance that the localisation model, the micromechanical model and the finite element model are mathematically consistent with each other. Otherwise, the whole exercise of developing models that should be used for practical purposes is utterly pointless. Appropriate laboratory tests on internal erosion must be designed and tests performed. The test result is expected to give useful information about internal erosion processes as well as important physical quantities. It is desirable that foremost measurable quantities are included as parameters in the models. Internal erosion models should be based on test results and the type of tests needed is influenced of the type of models chosen, so it is very important that the theoretical work and the practical laboratory work are performed simultaneously and interactive.

When the internal erosion model is developed and included in software it is essential to evaluate its behaviour. Since dams are complex structures it is not advisable to evaluate the model directly on a dam; some simpler structure would be preferable. The evaluation of the model may be carried out in two simplified steps before real dams are studied. First, laboratory tests performed, according to the discussion above, could be numerically simulated and the numerical result could be compared to the laboratory test result. It would be advantageous if this kind of evaluation could be performed continuously, if possible, during the development of the model. As a second step, small scale internal erosion tests ought to be performed on model dams. These tests will be numerically simulated and numerical and laboratory test results compared. An example of a small scale test on a model dam at Vattenfall Research and Development in Älvkarleby is shown in Figure 8.2.

Figure 8.2 Model test at Älvkarleby.

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