EXPERIMENTS ON THE RETENTION OF SOILS BY FILTERS

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by

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ABSTRACT

This thesis examines the possibility of a new filter design approach when non-intact conditions must be considered. Such conditions are mainly related to cohesive base soils when suffering any type of "piping". A series of tests has been performed where slurries of fine materials were passed through sands in such a way that an effective filter had to guarantee the retention of the particles. An experimental relationship between the permeability of the filter and the effective particle size to be retained, using the present and other published results has been found to be satisfactory at least for preliminary purposes. The suggested procedure is to consider the finer part of the base soil as the one to be retained by the filter. The effective particle size to be used in the proposed relationship must be determined from the soil-water interaction which will occur in the field. With this particle size, a filter is determined by its permeability.

A study of the influence of the water chemistry and of the concentration of the particles in suspension on the determination of the particle sizes of slurries has been carried out. It has been found that for materials finer than 30μ, having similar mineralogy to those used in these tests, flocculation increases with the increase of salt concentration. The effective particle size to be considered in the criterion proposed will depend on the base soil-water interactions. The determination of the size of slurry particles by sedimentation

methods has been found to be influenced by the concentration of the soil particles in suspension. A concentration of about 30 g dry soil/litre is recommended.

A series of tests using sands as base-soil in contact with sieves in place of filters has also been performed. It has been found that the coarse particles are those which really control filtration at the base-filter interface by self-filtering process and arching effects. It is suggested that the empirical filter design criterion proposed can be used in the same way for cohesionless soils when base and filter are in contact. The D_{85} base particle size seems to be reasonable in defining filtration characteristics.

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1. INTRODUCTION

The purpose of this research has been to study the problems associated with filter design particularly when dealing with fine base materials in non-intact situations.

The work has been divided into two parts: the first is concerned with a literature review of filters in general, their purpose and importance, associated mechanisms, design requirements, previous experimental work, design methods and criteria; the second part describes an experimental work where filters were tested for their suitability of retaining soil particles in suspension. A postulated relationship between size of particle retained and the permeability of the filter was verified. Since such a relationship allows a granular filter to be quantified as an equivalent sieve, a series of tests with sieves as filters and sands as base materials was carried out. The replacement of the granular filter by a sieve allows loss of material from the base soil to be observed with less uncertainty. The first part is related with Chapters 2 and 3 and the second part with Chapters 4, 5, 6 and 7.

2. FILTERS - GENERAL ASPECTS

2.1 Filter Purposes and Definitions

Filters and drains are required to provide security against damaging actions of seepage and groundwater in hydraulic structures such as wells and earth dams.

Drains are used to provide adequate boundary conditions for the pore water pressure control. Filters have the main purpose of retaining soil particles which are carried out through the soil by the percolating water. A filter can work in contact with a drain to guarantee that the finer material will not be washed through with consequent clogging of the drain or it can work as a drain itself.

In order to work efficiently a filter must obey the following requirements:

1. The internal void sizes of a filter must be sufficiently small to block the passage of most particles from the protected soil to establish stable interface after acceptable material migration with no further loss.

2. The amount of fine material should be limited to ensure the absence of clay particles in such a way that the filter is cohesionless.

3. It has been also postulated that the filter must be permeable enough to avoid hydraulic pressure build up to disrupt the filter and adjacent structures. Actually it is arguable if permeability criterion is necessary if the filter is not required to act as drain, provided overall head loss in the filter is not important.

Some terms are frequently used in relation to filters and they are defined in this work as:

(i) Base material - the soil which is in contact with the filter and which is protected by it against being washed away.

(ii) Filter - the layer of soil in contact with the base material with the functions already mentioned before.

(iii) D^F, D^B - diameter of filter and base particle soil respectively.

(iv) $D_X{}^F$, $D_X{}^B$ - particle soil diameter of the X% passing size of filter and base material respectively in a granulometric analysis; it will mean that X% in weight of the soil particles have diameter equal or smaller than D.

(v) Uniformity Coefficient (CU) - is the ratio between D_{60} and D_{10} particle sizes of the soil.

(vi) Uniform soil - material that has a very narrow range of grain sizes; the CU value of these soils is ≤ 2.

(vii) Graded soil - material that contains a broad range of grain sizes; the CU value of these soils is >2; a graded filter can also mean a filter system consisting of various layers, each zone obeying the filter requirements in relation to the adjacent one. The first meaning will be the one usually used in this work.

(viii) Gap-graded soil - poorly graded material where some soil particle sizes between the finer and coarser particles are missing.

Depending on the uniformity of the soil, different behaviour can be expected when used as filter material:

uniform filter materials possess the qualities of consistent permeability characteristics and little tendency to segregation itself; graded filter materials can produce a dense mixture with little tendency to erode or pipe but can suffer segregation.Because of these different characteristics some criteria for filters distinguish between uniform and graded materials.

A graded material may also have different types of grain size distribution curves as concave, convex, gap-graded, s-shaped or straight-line. Gap-graded and concave shaped curves will tend to segregate more than convex-shaped curves for the same range of particle sizes(*). These curves are shown in fig. 2.1 as an illustration.

Although the experimental work developed in this research is more concerned with filters working in embankment dams, the filter design methods and criteria developed up to now will be presented and are equally valid for any other type of structure.

When a filter is required in an earth dam, for example, depending on its position in the cross-section of the dam, it can have different purposes and must be designed accordingly. In a zoned dam with an internal impervious core and several upstream and downstream zones of increasingly pervious material, the transition from fine to coarse soil necessary to prevent piping is frequently achieved by the progressive zoning without any need for specially graded filter zones. In heterogeneous dams and dams with a thin impervious core, however, the difference between the impervious and pervious embankment zones is frequently so large

* as described by U.S.C.E. (1953).

that one or more layers of filter material with special gradation characteristics will be necessary.

As an illustration, the cross section of an earth dam with a central core with some possible required positions for filters is shown in fig. 2.2. The functions of these filters would be:

Filter 1 - Between the rip-rap on the upstream face and the upstream fill

This filter has the function of protecting the material of the upstream face against erosion due to wave action. Also it can protect the base material of the upstream face against piping during rapid drawdown. A filter in this position can usually be designed using the conventional rules existing which will be presented and discussed later.

Filter 2 - Between the upstream face and the core of the dam

Depending on the material of the core piping can occur during rapid drawdown when water flows from the core to the upstream face. In this case filter 2 must be designed to prevent the possibility of erosion of the material. Probably, for this situation to be a critical one the core would be composed of a coarse material and therefore, the filter may be designed using conventional filter criteria.

Filter 3 - Between the core and the downstream face of the dam

In this case the possibility of crack formation during construction and impounding must be considered depending on the material of the core. For a non-cohesive material the rapid collapse of a crack prevents internal erosion and instability and a filter can be designed on the assumption of an intact core. In this situation conventional filter criteria for cohesionless soils could be too conservative. Nevertheless, if a cohesive core is used, the criteria must be to prevent the passage of the eroded debris, as will be discussed further in Chapter 3 where existing filter criteria for cohesive soils in non-intact conditions will be presented. A filter in this position has also the function of preventing excess pore pressures in the downstream fill that could lead to slope instability of the fill.

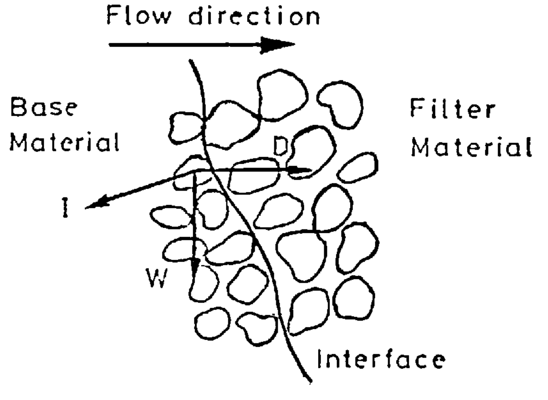
Filter 4 - Between the foundation and downstream fill of the dam

A filter in this situation can prevent the washing away of the soil foundation particles through the fill by the under-seepage water in a piping process. The conventional rules may be used in this case but also the possibility of heave must be checked and if so a weighted filter is necessary. Sherard et al (1963) discuss this problem and present a technique to design for it. Another function of this filter could be to prevent saturation of the downstream fill material or to accelerate the downward flow of water from the fill material at the end of construction if it is a fine graded soil.

As these examples show, the design of a particular filter will depend on whether the base material is a cohesive or a cohesionless soil and also if intact or non-intact conditions occur. Depending on this, the conventional rules and existing criteria, which were based mainly on laboratory tests in cohesionless base soils, could be too conservative or too dangerous.

2.2 Base-Filter Soil Interface

When a filter is designed to work for a specific base soil as a drain or as a filter itself, it must be able to avoid the migration of the base soil particles into it. At the interface with the filter, the forces that act in a base soil particle can be illustrated as:



where:

I = inter-particle force

W = weight of the particle

D = drag force due to the percolating water

The stability of the particle will be controlled primarily by the magnitude and direction of the resultant of these three forces which will depend on the nature of soil, arrangement of the particles and the flow itself. If the particle is in equilibrium under these forces, in principle, there is no need for a filter. If not, the filter must have a maximum continuous pore size such that the particle cannot pass through.

Real soils are not composed of only one particle size and in practice some loss of the finest base material at the interface will occur through the filter. Also at the same time of the progressive migration of the finest base particles, the coarser particles gradually block the larger entry pores into the filter up to a situation where no further movement can occur, creating a self-filtering process. As this process is gradual, the effectiveness of a filter will depend on the magnitude and acceptability of the particles movement and loss which must occur before stable conditions are established.

The self-filtering mechanism of the filter interface was observed in many laboratory filter tests and a great number of criteria for filter design are related to the coarser base particles. Some tests with sieves as filters performed by the author also showed the same type of mechanism. These tests will be presented and commented in Chapter 7.

It is known that the interparticle forces are very small in cohesionless soils. For cohesive soil however, they can be strong in relation to weight and hydraulic drag force although their nature is not yet completely understood; this subject will be discussed further in Chapter 5. Therefore, if intact conditions are expected the cohesionless soils are more susceptible to erosion than cohesive soils. On the other hand if a non-intact situation must be considered (e.g. cracks), other considerations arise. The inter-particle forces for cohesive base materials can become insignificant and segregation can destroy the self-filtering mechanism (Vaughan and Soares, 1979). As already mentioned before, it seems necessary to distinguish four situations with which filters are concerned: cohesionless or cohesive base soils and intact or non-intact situations. Each of these conditions will be presented in Chapter 3.

2.3 Mechanisms of Internal Erosion

When soil material is carried away from inside the earth structure (or its foundation), filters can be required to block the passage of this material, ensuring the internal stability of the structure. Piping, in the context of this work, is defined as any process where soil particles are carried away by the percolating water. The mechanisms associated with piping will be briefly discussed in this section to postulate better the requirements that filters should obey. Such mechanisms include the following:

2.3.1 Conventional Piping

This is defined as the process of formation of a "pipe" linking the upstream to downstream areas of a dam through its foundation or the embankment itself. This mechanism (also known as internal erosion) starts when leaking emerges from the base material increasing locally the hydraulic gradient and therefore the resulting increased velocity of water flow may be enough to accelerate erosion ticularly in non-cohesive materials. Once this happens par-
the hole so formed can progress in the upstream direction up to the formation of a complete hole ("pipe"). When it reaches the reservoir, the water velocity through the pipe

will increase greatly and can be enough to erode more material easily enlarging the "pipe" up to a complete failure of the structure.

As the formation of the "pipe" is a continuous process, it is necessary that the products of erosion are carried away completely or at least a significant amount of the smaller particles if segregation occurs. In cohesive materials the sides and the roof of the"pipe" may stay relatively stable but for cohesionless materials they may collapse if a certain size and shape of the pipe is attained. Also if segregation occurs with a large amount of the larger and heavier particles being retained on the floor of the "pipe", it can eventually stabilise with the velocity of the flow decreasing.

The provision of a suitable filter placed on the downstream side of the dam can prevent this mechanism by holding the first eroded particles. The design of the filter must be in accordance with the type of the base material as cohesive or cohesionless and also if dispersive conditions occur within the soil. The available filter criteria for such cases will be reviewed in the next chapter.

2.3.2 Erosion Associated with Dispersive Soils

Recently it has been reported that dispersion of clays was responsible for some failures of dams which were not well understood in the past.

Aitchison (1960) suggested that the mechanism for piping failures in earth dams could involve the dispersion of the clay fill material at the exit point of the percolating water leading to the progressive loss of fine particles in suspension. Aitchison et al (1964) studied the post-construction deflocculation as a contributory factor in the earth-dam failure. Also Aitchison and Wood (1965) studied about 60 piping failures in small earth-dams in Australia. The authors suggested that probably a post-construction deflocculation occurred in these dams. The fills were compacted in dry conditions and in these circumstances the soil would have the porosity characteristics of silty materials (aggregation condition of clay). Afterwards when after impounding they become wet, these silt-size aggregates would break down in small particles (clay-size) and could be carried away by the percolating water in a progressive process.

Also, Sherard et al (1972b) have also studied some piping failures in dams of the USA related with dispersive soils.

Much work has recently been done in identifying areas of the world where these soils exist and confirmed it as the cause of several dam failures. Also, it has been attempted to recognize these soils with more precision and to develop techniques to reduce the deflocculation and avoid the erosion that follows.

Sherard et al (1976a,b) have studied the problem of identification of these soils. Although some procedures have been adopted by them and others it is still not possible to be completely sure whether a soil is dispersive or not. It is not yet well understood why failure has occurred in several dams of dispersive soil while there are several other dams constructed of the same soil and apparently in the same way and conditions which have not failed.

Actually the clayey soil in many parts of the world present flocculation characteristics in such a manner that in an erosive process the clay particles will aggregate forming "clay flocs" with silt size. However there are some clays (dispersive) which present a different behaviour depending on some chemical effects and the "floc", even if so formed, can be broken down into smaller particles, clay size. The conditions in which dispersion occurs will be discussed in Chapter 5 during the analysis of the soil suspensions used in the present experimental work.

Such particles being very small can be easily eroded by the percolating water. It had been reported (Aitchison and Wood, 1965) that in a poorly compacted dam core they can even pass through the macro-spaces of the soil, suggesting that failure due to the erosion of the fill can happen even if cracks have not occured. On the other hand, Sherard and Decker (1976), concluded that dispersive damage process can only occur as the result of progressive erosion of a concentrated leak.

De Mello (1977) however points out that many cases of dam failures attributed to first-filling dispersion of clay really could have derived from excessively dry compaction in arid regions, from horizontal layering due to compaction gradients or from shrinkage cracks. He suggests that conventional chimney drains could have inhibited these failures.

These soils have already caused problems for many years in agriculture. The Soil Conservation Service (S.C.S.) of the United States developed a method to identify this type of soil which has been used for nearly 40 years. Dispersive clays cause problems in agriculture since the raindrops falling on the ground act as a dispersant and the smallest particles created are carried down by the percolating water and further deposited forming an impermeable layer that is not good for irrigation and drainage.

In general, it can be said that the main difference between dispersive and non-dispersive soils depends mainly on the nature of the ions in the absorbed water, the dispersive soils having a preponderance of sodium whereas the non-dispersive have calcium and magnesium. It will then depend on the soil-water chemical interactions and two possibilities can be considered for a soil to become dispersed in an earth dam:

- the soil can be susceptible to dispersion but can also stay stable when dry. After being placed in a dam with water percolating through it and therefore being saturated, dispersion can take place.
- the soil can be in a flocculated condition with the clay "floc" already formed but since the reservoir water is of the chemistry required to cause dispersion the "floc" will break down as the pore water is changed by the seepage water.

It has been found (Sherard and Decker, 1976) that common index tests to classify soils will not differentiate between dispersive and non-dispersive clays. Because of this, several tests have been recently developed or adapted for the identification of such soils (Sherard et al, 1976a,b).

Besides the use of adequate filters as a protective measure for such soils, precautions against deflocculation development can be taken by the use of calcium admixture in all parts of the core or in an upstream zone of the dam. In this way it will ensure that the water seeping through these zones is rich in bivalent ions in such a way that deflocculation is minimised. Actually with time such behaviour can not be guaranteed. Also construction measures can be adopted for defence against dispersive erosion and some are specified in Sherard and Decker, 1976.

The problem with such precautions is that they minimize the risk but do not ensure that dispersion does not take place. A filter can then work to control erosion of the soil if it occurs. The filter design for dispersive soils will be discussed in the next chapter when dealing with cohesive soil as base materials in non-intact conditions.

2.3.3 Erosion Associated with Cracks

A crack is defined as an opening in the fill and therefore it is likely to be a preferential flow path. Once a crack is formed, the soil from its walls can be more easily eroded depending on the flow velocity and the soil itself. The erosion process can be interrupted by the collapse of the crack walls, by segregation of the larger particles in the roof of the crack or by a suitable filter which is capable of retaining the eroded particles. Causes and mechanisms have been already extensively reviewed recently by Truscott (1977). Some comments will be made in this section about the behaviour of cracks when water flows through them.

Independently of whether the crack has been formed under wet or dry conditions, internal erosion will only happen when the water flows through the crack and its behaviour will involve the characteristics of stability of the crack itself, the erosion of its sides and the transport of the eroded debris. Depending on the type of soil, cohesionless or cohesive, in which the crack is formed, these characteristics have different influences on its behaviour.

Cohesionless Soils

In this type of soil the crack when flooded will collapse leading to the core of the dam behaving again as an intact material. Although a dry crack may remain stable because of the negative pore pressure developed in the soil, when it becomes wet this pressure is eliminated and since the soil has no drained tensile strength the walls and roof of the crack will collapse rapidly upwards, making a seal itself. Therefore, it can be said that in general cracks in cohesionless soils do not lead to danger to the dam, and the filter to be used, if necessary, must be designed accordingly to the approach adopted for these soils.

Cohesive Soil

On the other hand, due to the small tensile strength that the cohesive material presents,a crack can remain open even when flooded. Although the effective stress is zero when water flows through the crack, the small drained tensile strength can be sufficient to ensure the stability of the crack until it achieves a certain critical shape and size. Once the crack remains open its walls and roof will be subjected to an erosion process where soil particles are carried away by the water. The process of erosion of cohesive soils is not yet well understood and it will depend on the type of clay, water flow velocity and direction and the shape and size of the crack. Since all these factors controlling erosion are difficult to simulate in an experimental work, it is not possible to predict erosion with assurance. For this reason the safe procedure to be adopted is that erosion will always occur when cracks are formed and remain open.

The eroded debris which will be generated in these circumstances will depend on the clay-water chemistry. If flocculating conditions exist, the particles which will be released from the soil mass will be "clay flocs" in the silt range size and if the soil is a dispersive one, the particles will have individual clay size as already commented in the introduction to the dispersive soils in the last section.

Cracks in cohesive materials can close if the rate and magnitude of swelling of the soil exceeds the rate of erosion. This fact can occur when the crack has been formed under dry conditions exhibiting negative pore water pressures. When flooded the effective stress will be reduced and swelling will take place. If the crack has been formed by drained hydraulic fracture, probably the swelling of the soil has already occurred before the crack formation and will not contribute to its self-sealing. On the other hand the total stresses in the soil will tend to increase due to the swelling under decreasing effective stress. In this way swelling can reduce the cracking risk as has been commented by Vaughan (1976).

The transport of the eroded debris formed will be a function of the magnitude and direction of the flow velocity as well as of the size of the crack. For a horizontal flow for example, depending on these factors, segregation can occur or not, with, in the first case, only the smallest particles being carried away by the water while the coarse particles are deposited on the floor of the crack or the fine and coarse particles are carried away together with the possibility of self-filtering when arriving at the interface. Thus, the characteristics of flow influence directly the amount and type of segregation and therefore the filter design criteria to be adopted.

The filter design approach for such circumstances will also be reviewed in the next chapter when cohesive soils in non-intact conditions are considered.

2.4 Some Filter Design Requirements

The design of a filter involves in general, not only selecting its gradation but also many other factors such as thickness, breadth, permeability, grain shape, percentage of fines, construction techniques, etc. The main concern of this thesis is the grading of a filter since it controls the filter purposes in a first instance. This aspect will be explored in the next chapter. In this section a summary is made of the other aspects of filter design and a complete review has been given recently by Truscott (1977).

2.4.1 Filter Thickness

Usually the grain size criteria give the gradation of the filter material for a given cohesionless base material. The width of a filter is generally taken as a multiple of the size of the earth moving equipment used for construction. If the grain size is completely satisfied then only a nominal thickness is sufficient to account for the segregation and the possible settlement of the earth mass. On the other hand one of the functions of a filter as defined previously is to conduct the water collected to downstream without excessive pore water pressures developing in it. If it is desirable to reduce the loss of head due to percolation through the filter or if the required gradation of the filter material is not available,it is necessary to use "graded filters" composed of several layers. In any of these situations, thickness of a filter becomes an important parameter and requires careful consideration. It is worth mentioning that the grading criteria do not ensure adequate discharge capacity and the drains have to be designed as hydraulic conductors. Cedergren (1973) presents several examples of dams that had trouble associated with this type of problem and two of these are shown in fig. 2.3.

At the present time, the thickness of a filter is determined by one of the two approaches:

(i) Seepage analysis, which considers the width of a filter large enough to drain the seepage water without causing excessive seepage pressures.

(ii) Statistical analysis, which considers the random movement of the base particles in the filter process.

Seepage Analysis

Cedergren (1960) was the first to develop a rational procedure for the design of the filter thickness. He considered the hydraulic aspects of the problem based mainly on flow-net solutions. However, as pointed out by himself later on (Cedergren, 1973), the flow-net technique is rather cumbersome to apply in the filter itself because of its dimensions and a more practical procedure has been proposed and consists of:

1. Determination of the likely flow which the filter may be required to discharge using flow nets.
2. Estimation of the filter permeability. This can be done by laboratory tests (or empirical correlations) but proper account has to be taken of the stresses to which the filter will be subjected inside the embankment with corresponding changes in relative density.
3. Using Darcy's law and knowing the flow, permeability, length and head loss gradient, the thickness b of the filter can be estimated by a formula of the type:

$b = \frac{a}{ki}$ k = permeability
a = length
i = hydraulic gradient

4. The width calculated must be multiplied by a factor of safety to take into account the uncertainties which exist in steps 1 and 2. Also the factor of safety must take into account the possibility of cracking and then can be greater than 10. A series of values of flow measured in filters of cracked and intact earth dams has been presented by Truscott (1977) and can be used as a guide.

Vaughan (1977) presents a similar type of analysis but in this case a thickness for the filter is assumed and the permeability required can be determined.

The main difficulty in this method lies in the determination of the filter permeability. If the design width calculated as shown above is very large and uneconomic, a second filter of larger permeability can be provided keeping the first filter to the minimum size dictated by construction. This second filter must satisfy the piping criteria in relation to the first one.

However, as pointed out by Sherard et al (1963) and De Mello (1977), in general the design of filter thickness is based on previous experience and by the construction techniques to be used. Also the thickness adopted must take into account possible movements of the earth dam or its foundation. Sherard et al (1963) suggest minimum thickness of about 15 cm and 30 cm for horizontal drainage layers of sand and gravel respectively. For vertical or inclined filters used as chimney drains in homogeneous dams or immediately after central clay cores, a minimum horizontal width of 2.40 to 3.00 m is suggested by the same authors.

Statistical Analysis

Silveira (1965) developed a probabilistic analysis to compute the thickness of a filter considering the required thickness to stop the washing out of the base soil. Silveira's approach for filter design will be presented in Chapter 3. Thanikachalan et al (1974) also presented an investigation in which they considered the random movement of base particles and its clogging. In both cases, the design criterion sugges-ted, suffers from the simplified assumptions, with the soil idealized as being formed by spherical particles.

2.4.2 Material Characteristics

The most important characteristic of the material to be used as filter is that it must be and remain cohesionless at all times so that it will collapse if eventually a crack is opened which becomes full of water.

It is difficult to define a priori if a material that is non-cohesive can acquire cohesion. Since the amount of fines in a filter is generally kept to a minimum, the material is likely to be in the limit at which it does not show any cohesion or a very small one. However the existing methods of measuring the strength and plasticity of soils are not sensitive enough to detect the presence of the small amount of cohesion which could allow a crack to remain open in a filter material. Some simple tests have been suggested recently to verify if the material has some cohesion or not. They are briefly presented as following:

(i) Vaughan (1976) proposed a simple test where a compacted sample of moist filter is prepared using a standard compaction mould. The sample is placed in a tray and the tray is carefully filled with water. If the soil is cohesionless, it will collapse to its angle of repose and if not it retains its form. This test completely disregards the effects of compaction and consolidation of embankment loads and therefore gives only a rough idea of the behaviour of the material. Nevertheless it can be useful in practice, at least to provide the basis for a test in which these factors are considered.

(ii) Mantz (1975) discusses two tests to check if fine particles are cohesive or not. He notes that these tests must be carried out in water similar to the one to be used in practice, containing the same salts in the same concentration. In both tests the method is to split the material into groups of uniform size particles and test each size separately. By comparing the results it should be possible to determine the size at which the soil becomes cohesive. The first type of test consists of sedimenting the fractions into the appropriate water and measuring the packing coefficients C, the ratio of the volume of solids to the total sediment volume. The higher this value, the less cohesive the material. Mantz obtained a value of C equal to 0.5 for 16.5 microns particle size and considered these cohesionless. The other method, consists of measuring the angle of repose of the fractions α_R, defined as the angle made with the horizontal by the flat surface of a mass of solids after grain movement. The smaller α_R is, the less cohesive the material. The value measured is also dependent on the test method. Mantz sedimented the material into water in a 70 cc cylinder of 2.5 cm internal diameter. The cylinder was stoppered to exclude air, rotated through 90^o and the angle of repose measured. The following results were obtained for flaky particles using a slightly different method. It was considered that the 25.5 microns particle was cohesive but above 37.5 microns the material was non-cohesive in the water used during the test.

Particle Size (microns)	Angle of Repose Measured - α_R
25.5	65±6
37.5	38±3
46	39±3
56	42±4
68	41±4
79	42±4

On the other hand, although it could be apparent if a soil is cohesive or not it is not so clear under which conditions a non cohesive soil acquires cohesion. Some factors such as chemical effects, the amount and nature of the fines allowed in the soil can affect the cohesionless nature of the soils.

Chemical effects can cause a material to acquire cohesion by cementation (precipitation of iron oxides carried by the water, for example), weathering of the particles and by the formation of bonds. According to Truscott (1977) these bonds may be formed between soil particles by several mechanisms including the presence of bonding agents, secondary consolidation, compaction, organic activity, etc.

It is clear that the greater the amount of fines present in the filter material the more likely that it will become cohesive, although the principal factor will be the nature of these fines. Apparently there is considerable disagreement on the quantity of fines that can be tolerated in a filter zone (defined as the material passing the nr.200 B.S. sieve size). The amount of fines can influence both the cohesive and permeability characteristics of the filter material. Terzaghi and Peck (1967) insist that the filter should not contain any silt because this could increase very much the cohesion. Sherard et al (1963) and the Earth Manual (1974) suggest that the quantity of fines should not exceed 5%. On the other hand Vaughan et al (1975) found that a minimum of 5% was necessary to ensure no erosion of a crack in the Cow Green Embankment Dam and allowed up to 10% of fines. Washing of fines is generally accepted as a way of processing natural deposit materials to be used in filters.

Also the shape of the particles can play an important role in the behaviour of the filter as suggested by Vestad

(1976). The author reported the unsatisfactory behaviour of the filter zone at Viddalsvatn Dam where on impounding cracking and erosion occurred. He attributed the breakdown of the filter to arching because of the angular particle shape of the filter material. Angular particles (mainly from rock quarries) tend to form larger voids than rounded ones. This problem has been mentioned by Sherard et al (1963), Cedergren (1973) and De Mello (1977) and the only filter criteria available that recognize this fact are those presented by the USBR after Karpoff (1955). The particle should be approximately cubical and a commonly used criterion is to require that the maximum length of a particle be less than three times the minimum dimension. In many cases the material available should be processed to meet these requirements as well as the amount of fines, etc. Truscott (1977) discusses this matter in greater detail and lists the available types of material that have been used in practice.

Finally, materials for filters should be sound in such a way that their deterioration with time will be kept to a minimum under any climatic or chemical environment. Laboratory tests to determine that materials are satisfactory consist of the routine tests for soundness of concrete aggregate supplemented by special tests if necessary. More details are given in Sherard et al (1963) and Earth Manual (1974).

2.4.3 Other Design Considerations

Some points related to filter geometry and construction procedures are briefly commented in this section. It is not the objective of this work to go into detail in this matter.

Drainage layers and associated filters must extend along the entire longitudinal section of the dam at least to top water level in order to intercept any crack that may form through the core. Since cracks are more likely to occur near abutments (Sherard, 1972), special care must be taken in this area. The downstream filter zone must be located immediately downstream of the core or fill zone to be protected and made self healing. A vertical filter has some advantages over inclined filter and special care has to be taken during placement of this last type. However De Mello (1977) notes that in a high embankment a vertical, central filter zone can lead to upstream cracks and recommends the use of an upstream inclined filter.

The gradation of the downstream zone relative to the filter, in the case of a dam with a central core, must also meet the filter requirements. This is particularly critical when the downstream shell consists of rockfill with large voids. For example, Sherard (1972a) cites the case of Matahina Dam where piping occurred due to the washing out of the filter into the rockfill voids.

The most critical problem that can occur during placement of a filter is segregation of the material. To avoid this, it is recommended (Sherard et al, 1963, Earth

Manual, 1974) to limit the maximum particle size of a filter to 3 inches. It may also occur during vibrating compaction, with the finer material accumulating on top of the layer.

Techniques for placement and compaction of filter materials vary from case to case and some of the available ones are described in Sherard et al (1963) and the Earth Manual (1974). Because the filter and drainage zones are very important in relation to the safety of the embankment, it is essential that they are correctly constructed. For this, they must be properly specified and if this is done the construction control should be relatively easy. A list of factors to be controlled has been presented by Truscott (1977) as follows:

(1) Gradation should be checked regularly at source, on the fill and when placed;

(2) No contamination of filter zones by material from other zones or any other materials;

(3) Correct compaction by checking passes of compaction equipment, density tests and flooding of filters and observing settlements;

(4) No segregation of materials in the zone;

(5) Correct geometry of zones is maintained.

More specific descriptions of general procedures to be used are presented in the Earth Manual (1974), but it is clear that the designer should establish the type of control for the specific requirements of each case, as has been reported by many authors for different embankment dams.

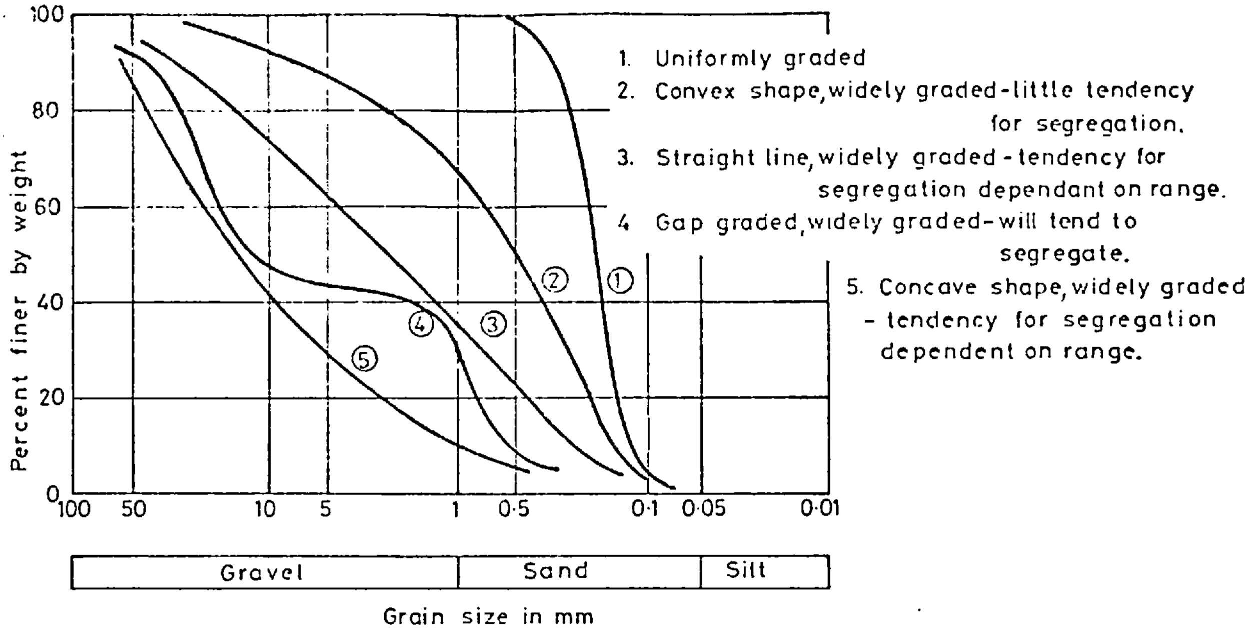


Fig. 2.1: Examples of general types of grain size curves (after U.S.C.E., 1953)

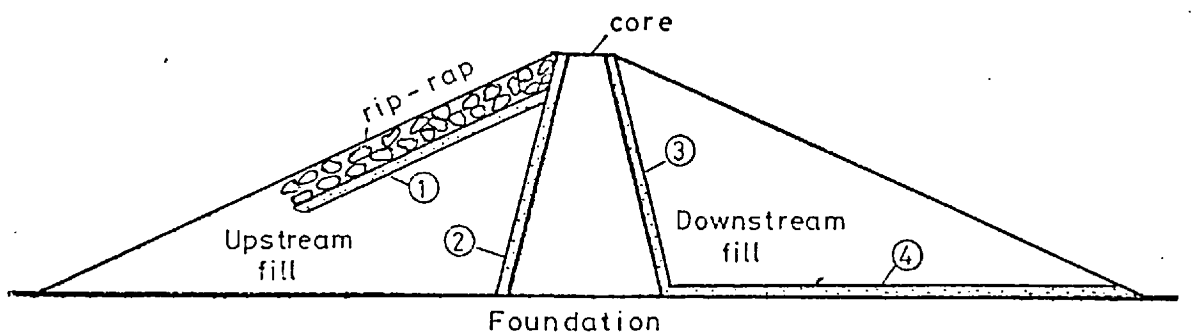


Fig. 2.2: Different positions of filter in an earth dam

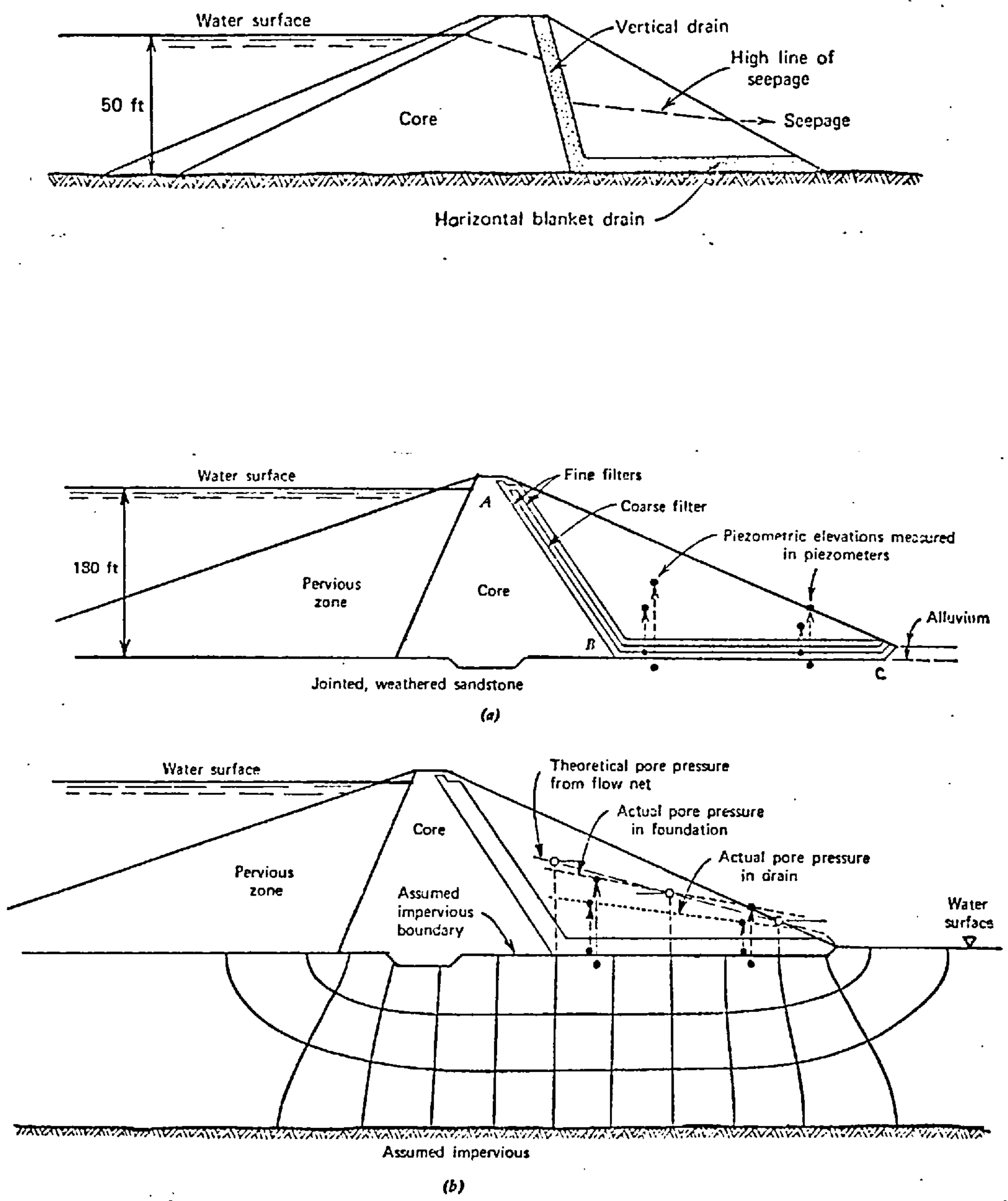


Fig. 2.3: Earth dams with internal drains of inadequate discharge capacity (after Cedergren, 1973)

3. PREVIOUS EXPERIMENTAL WORK, DESIGN METHODS AND CRITERIA

3.1 General Aspects

The design of adequate protective filters has become increasingly more important as earth and rock fill dams tend to be higher, longer and founded on more complex foundations. The design criteria used up to now have been based on experimental and theoretical work developed mainly for cohesionless soils and relating only typical grain sizes of base and filter materials. Although these criteria have been extrapolated for any type of base material, a few types of tests were developed specifically for cohesive soils, trying to simulate a more realistic mechanism of piping for these materials.

In this chapter existing design methods and criteria used for filters will be presented, resulting from experimental and theoretical work and others consisting of several tentatives of correlations using grain size ratios found from experimental data and observations. Soares (1977) has made a more complete review on the subject. Whenever possible, a more critical examination of the test results and analysis is made to try to establish the differences found by different investigators and to compare further with the author's experimental work and conclusions.

3.2 Cohesionless Material - Grain Size Distribution Analysis

The grain size distribution approach is expressed in the form of ratios which limit a certain size of the filter material in reference to some arbitrary size of the base material, in such a way that the filter prevents the migration of the fines into it and permits the movement of the percolating water without generation of high pore pressures, requirements already mentioned before in Chapter 2.

Some authors adopted different ratios to express filter criteria. The most used are:

(i) Stability ratio: $\dfrac{D_{15}^{F}}{D_{85}^{B}}$

(ii) Permeability ratio: $\dfrac{D_{15}^{F}}{D_{15}^{B}}$

(iii) Auxiliary ratio: $\dfrac{D_{50}^{F}}{D_{50}^{B}}$

Besides the different ratios adopted, the test procedure, equipment and materials, some experimental works published also differ in the following main points:

- different failure criterion for a filter or the authors have neglected to state which failure criterion was used;
- the hydraulic gradients in the base materials were varied; some authors used gradients similar to those occurring in practice and others used much greater gradients to eliminate time effects which are difficult to reproduce in laboratory tests;
- some authors have tapped or vibrated the apparatus during the tests and others did not or no mention of this was made.

(a) Terzaghi Criteria

The filter criterion based on grain size distribution was first developed by Terzaghi in 1922 based on his own experience and consideration of the geometry of soil particles. He specified that base and filter materials must satisfy the following requirements.

1. The 15 percent size of the filter material should not be more than four times as large as the 85 percent size of base material, e.g.

   $$D^F_{15}/D^B_{85} < 4$$

2. The 15 percent size of the filter material should be at least four times as large as the 15 percent size of base material, e.g.

   $$D^F_{15}/D^B_{15} > 4$$

He also recommends the use of laboratory filter tests as a check for these rules whenever possible.

The criterion 1 is concerned with preventing piping. According to this, it is expected that the 15 percent of the coarsest size of the base soil, prevented from entering the filter, will form a filter of its own by retaining the remaining 85 percent of base material. The self-filtering process was already expected to operate although no reason was given for this. The criterion 2 satisfies the requirement that the filter should be more pervious than that of the base soil to act as a drain.

Terzaghi has not made any distinction between uniform and graded soils as base and filter materials. It seems that to overcome this difficulty, a factor of safety was applied to his rules.

(b) Bertram Criteria

The first important experimental work on filters was done by Bertram in 1940 when additional information regarding filter design was found necessary. He also published the test procedure and results in a detailed way not always found in literature.

The filter tests consisted of permeability tests where soils were placed in test cylinders in two layers, one of the finer soil (base material) and another of the coarser soil (filter material). Water was then allowed to flow through the soils, passing from the base soil into the filter.

The main points in the general test procedure adopted by Bertram can be resumed as:

- All test samples were compacted to 70% relative density in such a way that the void ratio of all filters tested would be comparable.
- The hydraulic gradients used in the tests (6 to 8 and 18 to 20) were much greater than those found in the field, in order to compensate time effects in the tests performance.
- The coefficient of permeability of the base soil was determined at intervals during the progress of each test to establish qualitatively which type of failure occurred during a test and to obtain a measure of the rate of movement of the soil grains.

The author considered that a base-filter was unstable when either the fines from the base soil gradually clogged the pores of the coarse material or the entire finer soil was completely washed through the coarser one. He then used the following procedures to determine failure in the tests:

1. Permeability measurements of base soil during tests.
2. Measurement of the loss in weight of both soils which gave a quantitative index of the movement of the soil grains.
3. Determination of grain size distribution of both layers before and after test to find out the change which had taken place in the grading of the soils.
4. Visual observations.

The filter tests performed by Bertram can be divided into three series:

1st SERIES - They were considered preliminary tests where a natural sand of glacial origin was used as base and filter materials.

2nd SERIES - This consisted of filter tests using uniform sand for both base and filter materials. The tests can be divided into 3 types of base-filter combinations:

(i) Tests using round grained quartz sand as base and filter materials.

(ii) Tests using crushed quartz as base and filter materials.

The tests were similar to those of type (i) having the purpose of determining the 'shape factor'. The filter test results found were also similar to those with round particles.

(iii) Tests using combination of round grained quartz and crushed quartz materials.

The round and crushed materials were used as base and filter combination and vice versa.

In order to compare his test results with the criteria established by Terazaghi, Bertram found the critical ratios D^F_{15}/D^B_{85} and D^F_{15}/D^B_{15} for the filter tests at the limit of stability. These results are presented in figs. 3.1, 3.2 and 3.3. The following results were the minimum ratios achieved for each base-filter combination presented above:

(i) $D^F_{15}/D^B_{85} = 8.7$ ($\simeq 9$) and $D^F_{15}/D^B_{15} \doteq 10.7$ ($\simeq 11$)

(ii) the same results as tests type (i)

(iii) $D^F_{15}/D^B_{85} = 6.5$ ($\simeq 6$) and $D^F_{15}/D^B_{15} = 8.5$ ($\simeq 8$)

3rd SERIES - A limited number of tests was also performed by Bertram with natural sand in which graded bases were protected by graded and uniform filters.

Due to the limited number of tests done, the author assumed that the results did not allow definitive conclusions regarding the critical ratios for graded materials.

The main conclusions presented by the author based on the results of filter tests of uniform sands can be resumed as:

1. Minimum critical ratios: $\dfrac{D^F_{15}}{D^B_{85}} = 6.5$ and $\dfrac{D^F_{15}}{D^B_{15}} = 9.$
2. The critical ratios were practically independent of the shape of the soil grains.
3. The critical ratios were practically constant for the range of hydraulic gradients from six to twenty which was investigated.
4. Filter tests with downward and upward flow directions presented the same results.

During the analysis of Bertram's experimental work some points were raised which are:

- The critical ratios presented were concerned with the tests in the limit of stability. Actually the real ones could be between the results of these tests and those in which the filters have failed, resulting in slightly greater values.

- The maximum critical ratios found were those related to the finest base tested, which were $D^F_{15}/D^B_{85} = 11.5$ and $D^F_{15}/D^B_{15} = 15.0$, suggesting that these ratios could increase as the base materials become finer.

- When uniform base and filter materials were tested, having the same particle shape, the critical ratios could increase to $D^F_{15}/D^B_{85} = 9.0$ and $D^F_{15}/D^B_{15} = 11.0$. Actually the values 6 and 9 found for the same ratios were related to the tests where round base soil particles were tested with angular filter soil particles and vice-versa. Only one base size presented different results from the other tests and even so when the higher gradient 18 to 20 was used. It could be speculated that angular particles would give greater voids size and this could result in a finer filter material for a specific base with rounded particles.

(c) Newton's and Hurleys Criteria

A series of laboratory tests was conducted by Newton and Hurley with the purposes of determining the efficacy of natural bank gravels as filter materials. The small amount of information about these tests and filter criteria presented in this work, was obtained from U.S.C.E. (1953).

For the tests in which failure was detected the base materials were composed of graded gravelly sand with CU minimum 3.08 and maximum 4.77 and protected by fairly uniform filters composed of natural bank gravels. For all these failed combinations the ratio D^F_{15}/D^B_{85} was less than 4. The authors found that a failure criterion based on this ratio was not always applicable for very uniform filter grades and widely graded base materials. They suggested then the following stability criteria:

1. $D^F_{15}/D^B_{15} \leq 32$
2. $D^F_{15}/D^B_{50} \leq 15$

The low values of D^F_{15}/D^B_{85} found in these tests are not expected if self-filtering mechanism is believed to occur at the base-filter interface. The results also depend on the failure criteria used by the authors during the experiments.

The U.S.C.E. presented the test results by the ratios D^F_{15}/D^B_{15}, D^F_{15}/D^B_{50}, D^F_{50}/D^B_{50}, D^F_{15}/D^B_{85} and CU of both base and filter materials. Using these data and choosing arbitrarily D_{60} values for the filter in the fine, medium and coarse gravel particle size range, the granulometric particle size distributions of base and filter soils for three failure tests were estimated as shown in fig. 3.4. Although the base materials built are idealized, the real granulometric curves must be similar to those found in order to obey the ratios presented. It seems that the base materials were not well graded despite of their uniformity coefficient. The curves could be too concave in the upper part (in the region of particles with $D \geqslant D_{60}$) and this could be the reason for such low values of the ratio D^F_{15}/D^B_{85}. Also it seems that self-filtering is not only controlled by the 15 per cent coarser particles. A certain amount of the other particle sizes must exist to establish the equilibrium at the interface. This is expressed not only by the range of particle sizes contained in the grain size curve but also by the shape of the curve with respect to the distribution of grain size throughout the entire region. For the same CU values of the bases of the tests chosen, the ratios would increase for more than 6 if the granulometric curves were more well-graded.

This matter will be further discussed when the tests with sieves are presented in Chapter 7.

(d) U.S.C.E. (1941) Criteria

In 1941, the Waterways Experiment Station presented the results of an investigation of filter requirements for under drains. As part of this investigation a series of laboratory filter tests,consisting of permeability tests, was performed to determine the minimum grain size of filter materials required to prevent infiltration of fines into the filter material.

Three relatively uniform fine sands were used as base materials protected by a mixture of 80% concrete sand and 20% fine and medium gravel. The base-filter combination for each test was given by successively removing the fines from this filter material.

The general test procedure adopted during the tests was:

- The hydraulic gradient varied from 1 to 2.
- Head losses were observed through the filter layer.
- Tapping and vibration were also applied to the tests.

The authors found that visual observation furnished the most positive criterion for recording large movements of fine base material through the filter material. The following events were considered as failure for the filter tests:

1. Holes appeared in the top surface of the base soil.
2. The fine particles could be seen through the transparent walls of the permeameter moving downwards through the filter material.
3. The base soil particles were washed entirely through the filter material.

In some tests the base materials were stable until the permeameter was tapped, this being followed by either a complete washout of the base or a movement of fines into the filter. In other cases the base materials remained stable once the hole from an initial piping was filled. Filters were considered stable in both conditions. The authors reported failure in three base-filter combinations with the ratio D^F_{15}/D^B_{85} equal to 8.7, 6.9 and 5.9. The failure test which presented the lower ratio value 5.9 was repeated using a slightly higher gradient equal to 2 instead of 1.7 as before. The base-filter combination was stable this time although vibration caused the complete washout of the fines.

The authors achieved the following main conclusions regarding criteria and design for filters:

1. If the ratio D_{15}^{F}/D_{85}^{B} was less than 5, a fine material would not be washed through the filter material.
2. The grain-size distribution curves should be approximately parallel in order to minimize washing of the fine base material into the filter material.
3. Filter materials should be packed densely in order to reduce the possibility of any change in the gradation due to movement of fines.
4. A filter material is not more likely to fail when flow is in the upward direction than otherwise, unless the seepage pressure becomes sufficient to cause a quick condition of the filter.

In the analysis of test procedure presented by the authors, it could be speculated that the small thickness of base soil used during the tests (less than ½") could be, at least in part, the reason for detecting failure in some tests and also for a base-filter to become stable after an initial piping had been filled as reported in some tests. Lund (1949), as will be commented later, after performing a few tests with sands as base and sieves as filter materials concluded that at least a 2 in base soil thickness is required for a "natural filter" to be established and with less than that the whole mass of the sample would be unstable.

It must also be noted that the filter materials used in these experiments were graded ones, with CU minimum of 2.3 and maximum of 7.2. Any comparison with other criteria must take this into account.

(e) Providence District, CE (1942)

In 1942, the former Providence District presented a tentative filter design criteria, obtained from data of filter test results of various sources:

- Test performed at the Providence District laboratory.
- Tests by Newton and Hurley (1941).
- Tests by the Waterways Experiment Station (1941).
- Tests by Bertram (1940).

The result of this analysis was a filter design curve, fig. 3.5, in which the ratio D^F_{15}/D^B_{15} was plotted against the uniformity coefficient of the base. It was found that a reasonably well defined line of demarcation existed between the stable and unstable filter-base combinations. All the test data plotted were limited to materials of rounded, or semi-rounded particles but no distinction was made between uniform and graded materials.

(f) Lund's Experiments

In 1949, Lund carried out a series of filter tests to study the effects of varying the size and grading of the base, using uniform filters.

The tests consisted of permeability tests, in approximately the same way as Bertram. The author adopted the following main procedures:

- Both base and filter soils were placed under water.
- The filter permeability was measured and controlled during the tests.
- A gradient of 10 was used in all the tests.
- The apparatus was gently tapped.

The criteria used to determine the stability condition of the base and filter combination were:

1. The permeability changes of the filter during the tests.
2. Visual observations of migration of the base particles into the filter.

Adopting these two procedures, the test results were classified as:

STABLE - very small amount of material entered the filter, with no significant change in permeability throughout the test, even under conditions of gently tapping the apparatus.

UNSTABLE - after gentle tapping there occurred a significant drop in permeability and this event was almost always accompanied by a visual loss of the base material into the filter.

COMPLETELY UNSTABLE - the whole or greater part of the base was washed through the filter.

In general, Lund did not find any difficulty in classifying the test results and adopted the coarsest stable filter for each base as the critical filter size. The differences in grading between filters were kept small in such a way that the critical filter could be determined within narrow limits.

Three different series of tests were performed:

1st SERIES - Filter tests combining uniform filters (CU between 1.1 and 1.15) and uniform bases (CU = 1.05).

From the five base materials tested, the minimum critical ratio D^F_{15}/D^B_{85} obtained was 8.1 and the minimum ratio of the unstable tests was 9.8. The critical test results are shown in fig. 3.6.

2nd SERIES - Filter tests combining uniform filters and graded bases. The filters had the same uniformity coefficient range as in the first series tests. Seven base materials were tested, having the same D_{10} particle size and different uniformity coefficients (CU = 1.2; 1.5; 2; 3; 5; 6 and 7).

The minimum critical ratio D^F_{15}/D^B_{85} found in these tests had a value of 7.5 and the minimum ratio for the unstable tests was 8.6. The critical test results are presented in fig. 3.7.

3rd SERIES - Filter tests combining uniform filters having the same uniformity as in the tests presented before and graded bases. Four different materials with CU = 1.18; 1.4;2 and 3 were used as base soils.

The result of these tests indicated that the minimum critical ratio D^F_{15}/D^B_{85} was equal to 8.4 and the minimum ratio for the tests which were unstable, equal to 10. The tests of this series presented the same results as shown in fig. 3.8, e.g. the same filter materials defined the critical and unstable conditions for each base.

Lund achieved the following conclusions:

1. For uniform filters the ratio D^F_{15}/D^B_{85} was found practically constant for base materials having CU up to 7 and should be of order of 8. Terzaghi's recommendation of D^F_{15}/D^B_{85} equal to 4 was conservative although a factor of safety could be applied on the experimental value of 8 found.

2. The base coarser particles should be the ones that actually controlled filtration by a self-filtering process at the base-filter interface.

The second conclusion was also based on some tests using sieves as filters which will be presented in Chapter 7.

Although the unstable base-filter combinations as classified by the author involve a gentle tapping of the apparatus, it is believed that the unstable condition represents the unsafe filters.

A filter criterion based on the ratio D^F_{15}/D^B_{85} seems to be reasonably satisfactory as far as uniform filters are concerned. Unfortunately in the last series of tests the uniformity coefficient of the base was increased only up to 3. If it were shown that for more graded bases the critical filter would still be the same, the filtration process could be associated with the coarser particles by building up a natural filter behind them and retaining the subsequent particles.

If the critical ratio D^F_{15}/D^B_{85} could be considered constant, as it could from Lund's test results, no other ratio could be constant too. In fact, considering the grain-size distribution curves of both filter and base materials as approximately linear between the 85% and 10% the relationships can be developed.

$$\frac{D^F_{50}}{D^B_{50}} \simeq \frac{D^F_{15}}{D^B_{85}} (CU^B\ CU^F)^{0.7}$$

$$\frac{D^F_{15}}{D^B_{15}} \simeq \frac{D^F_{15}}{D^B_{15}} (CU^B)^{1.4}$$

showing for example that no single ratio based on those presented above could be developed if the critical ratio D^F_{15}/D^B_{85} is constant.

(g) U.S.C.E. - 1953

The Waterways Experiment Station (1953) published the results of an experimental work, carried out to determine the effectiveness of standard, Corps of Engineers, concrete sand and gravel as filter materials for a varied range of base materials.

In general, the test procedure followed the permeability tests procedure established by Bertram and the main points can be resumed as:-

- The gradient was raised in steps, usually from 0.5 to 1, 2, 4, 8 and 16 at various intervals of time, usually ½h. A maximum gradient of 26 was applied to one of the tests.
- Permeability measurements were made at intervals during the tests.
- After each test had finished, mechanical analyses were performed on the filter material to determine the amount of base material that had penetrated the filter.

The authors recognised that, although permeability measurements could be the best criterion to identify a base-filter combination, they were very difficult to obtain precisely, especially when the test was run over a long period of time, because of the presence of dissolved air in water. It was considered that the following procedures were more reliable and practical for determining the stability of the filter combination.

1. Visual observations:
2. Mechanical analysis of the filter material after the test.

A failure was detected when large portions of the base fell through the filter.

Three main series of tests were performed:

1st SERIES - Four tests were performed to determine the effectiveness of using a filter composed of standard concrete sand together with base materials consisting of very fine sands, a silt and a typical loess material which was used as a slurry.

No vibration was applied during these tests and all base-filter combinations were stable. The critical ratios were not determined and the silt and slurry presented the maximum ratios of D^F_{15}/D^B_{15} equal to 30 and 68 respectively and $D^F_{15}/D^B_{85} = 9$ for both. The standard concrete sand appeared to be a suitable filter material for fine-grained base materials.

2nd SERIES - In this series of tests, six different base materials were tested together with a filter consisting of standard concrete gravel aggregate No. 4 to 3/4 in size. Two of the base materials were standard concrete sands, the same used as filter in the series presented before. The uniformity coefficient of the bases varied from very uniform value 1.2 to a graded value of 6.1.

Vibration was always applied this time. The critical ratios were not determined for steady flow conditions since all the filter tests were stable. The authors concluded that the stable ratios for the finest base material $D^F_{15}/D^B_{85} = 6$ and $D^F_{15}/D^B_{15} = 8.5$ could be close to the limiting ratios based on observation and they were limiting ratios when vibration was applied. They also concluded that the No. 4 to 3/4 in gravel was a suitable filter for the standard concrete sand.

3rd SERIES - A few tests were performed to determine which mixtures of concrete sand and concrete gravel could be more suitable as a filter as far as inherent stability (resistance of the material to segregation and piping within itself) was concerned.

After steady flow condition, vibration was applied to the permeameter. The inherent stability was determined comparing the gradation curves before and after tests and permeability measurements were made during each test.

The results of these tests indicated that a filter consisting of a mixture of 70 per cent concrete sand and 30 per cent concrete gravel was the most suitable, regarding internal stability, permeability, and density characteristics. They also suggested that although the tests were not conclusive, they could help when selecting materials where similar types of gradations were available.

Besides the experimental work presented, the authors also compiled a series of other results which had already been published. Using these data and their filter test results, a series of plots was constructed in an attempt to correlate the data. These plots utilized various ratios of grain sizes of the filter and base materials, trying to find out some criteria which would accurately define the borderline between stable and unstable filter combinations. The compiled data used was from the following sources:-

- Tests by U.S. Corps of Engineers (1953)
- Tests by Newton and Hurley (1941)
- Tests by U.S.B.R. (1948)
- Tests by Waterways Experiment Station for Enid and Grenada Dams
- Tests by Waterways Experiment Station (1941)
- Tests by Bertram (1940).

Various types of correlations as D_{15}^{F}/D_{85}^{B} against CU of base, D_{15}^{F}/D_{85}^{B} against CU of filter, D_{15}^{F}/D_{15}^{B} against D_{15}^{F}/D_{85}^{B}, D_{15}^{F}/D_{85}^{B} against D_{50}^{F}/D_{50}^{B}, D_{15}^{F}/D_{85}^{B} against CU of the base were tried. The last one which had already been used by the Providence District and presented before, showed the best relationship. With the new data provided, the authors found the curves shown in fig. 3.9, more suitable in delimiting safe and unsafe zones. Curve I is related to the filter test results when steady flow condition occurred and curve II when vibration was applied. The design curve I obtained suggests that a limiting stability condition is reached when the ratio D_{15}^{F}/D_{15}^{B} approaches a value of approximately 40 when the base soil can have a CU $\geq$ 4. Below this value the critical ratio decreases with the uniformity coefficient suggesting that when the base material is of a relatively uniform grain size,the filter must be finer.

Based on the experimental work and empirical correlations of filter data calculated, the authors suggested the following filter criteria.

1. $D_{15}^{F}/D_{85}^{B} \leq 5$, but for very uniform materials (CU < 1.5) it may be increased to 6.

2. $D^F_{50}/D^B_{50} \leqslant 25$.
3. $D^F_{15}/D^B_{15} \leqslant 20$, but for widely graded base materials (CU > 4) it may be extended to 40.

They also achieved the following main conclusions besides those already presented together with the filter test results:

1. The usual filter criteria are probably too conservative for very fine-grained or cohesive soils.
2. Both graded and uniform materials are satisfactory for filters as regards holding a base material in place. Gap-graded materials and materials so widely graded that they tend to segregate during placement are not recommended.
3. If there is any doubt in the use of the criteria suggested, the best procedure is to perform laboratory tests as Terzaghi had already recommended.

It was also observed that the large discrepancies found in filter criteria proposed by many investigators are mainly due to the method of interpreting the test results.

(h) Leatherwood and Peterson

Leatherwood and Peterson (1954) have investigated the head-loss occurring at an interface between sands and gravels which represented the base and filter materials respectively. Unfortunately it was not possible to obtain data and more details of this experimental work. Based on this interfacial head loss they have proposed two rules for uniform and graded filters which are:

1. $D_{50}^{F}/D_{50}^{B} < 5.3$

2. $D_{15}^{F}/D_{85}^{B} < 4.1$

These criteria have the same limitations of those which do not consider uniform, graded, subrounded and crushed materials and it is specific for the base-filter combination tested.

(i) Karpoff

Karpoff (1955) presented a series of tests carried out for the U.S.B.R. in 1948. This experimental work had the purpose of determining suitable criteria for the design for both uniform grain-size filter and graded filter under severe field conditions.

The materials used in the tests were natural subrounded gravels, sands and silts and a few tests were also carried out with crushed rock filters. These materials were sieved into single fractions and from them the uniform grain-size filters and graded filters of different gradations were artifically prepared in the laboratory. Gradients of up to 45 were used.

Although not clearly stated it seems that Karpoff used the following procedures to define a failed test:

1. Visual observations.
2. Comparison between the grain-size distribution curves of the filter material before and after test.

Karpoff has performed two main series of tests, one with uniform filters and the other with graded filters, each one having a total of twelve tests.

The uniform filters were tested either with uniform bases (CU equal to 1.41 and 1.45) or with graded bases (CU equal to 7.56). The graded filters, having a CU minimum equal to 3.2 and maximum equal to 29.2, were only used with graded bases (CU equal to 7.3 and 23.8).

The test results were analysed using the ratio D^F_{50}/D^B_{50} for the uniform filter tests and the ratios D^F_{15}/D^B_{15} for the graded filter tests. The flow measured during each test was related to these ratios. The range of ratios permitting the greatest flow without noticeable effect on the stability of the filters was selected as the design criteria. The following rules were then presented by the author as a criteria for filters of subrounded particles:

1. For uniform grain-size filters

   $5 < D^F_{50}/D^B_{50} < 10.$

2. For graded filters

   $12 < D^F_{50}/D^B_{50} < 58.$

   $12 < D^F_{15}/D^B_{15} < 40.$

In addition to these rules it is also suggested that the gradation curves of the filter and base materials should be roughly parallel in the range of finer sizes.

A series of six tests using crushed limestone filter material was also carried out. The author concluded that although the number of tests performed was not enough to define a filter criteria, probably the criteria developed for designing graded filters from natural subrounded material was not applied for designing filters from crushed rock due to the high degree of angularity and large percentage of voids. It is then suggested that the following rules should be used as a guide when graded filters of angular particles must be used:

$$9 < D^F_{50}/D^B_{50} < 30$$

$$6 < D^F_{15}/D^B_{15} < 18$$

Although Karpoff had made a distinction between uniform and graded filters, the same did not occur with the base materials. The uniformity coefficient of both base and filter materials varied widely in each test combination and each graded filter tested had a different CU (minimum of 3.2 and maximum of 29.2). It could be speculated that if any difference is expected between uniform and graded filters, it could also be expected between graded materials having such different CU values. Also the different filters used with any one base material were widely spaced with respect to their particle size, making it difficult to evaluate critical ratio values with greater accuracy. The procedure adopted by the author to define the critical range of ratios as criteria for filters could be somewhat conservative since the decrease of flow measured during each test could be associated with some clogging of the base particles at the interface before the self-filtering had occurred without necessarily failure of the base-filter combination

(j) Davidenkoff's Experiments

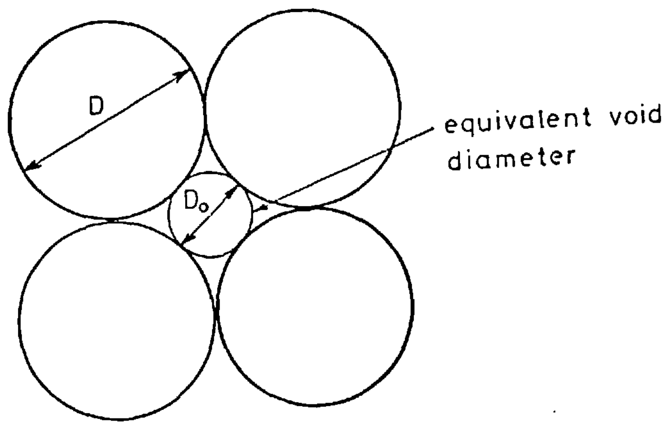
In 1955, Davidenkoff carried out an analytical study with the main purpose of developing a theory to be used as filter criterion for cohesive soils which will be presented later. Also in this work he included some experiments in non cohesive soils using plates with holes of different diameters put under granular soils, trying to simulate the behaviour of the base soil grains over the voids of the filter material.

He observed from these tests that dry granular sand grains of a given diameter can arch across a hole of much greater diameter without falling through and can also remain stable under a flow of water up to hydraulic gradients of about 10.

Davidenkoff using the results from the plate hole tests proposed one extrapolation to filter and base combinations to find a design criterion. He considered the filter particles as uniform spheres in a loose packing as:

D = grain size diameter

D_o = equivalent void diameter



For such packing, the equivalent void diameter would be $D_o = D/2.42$. The author found that for arching stability to be assured the hole diameter should not be more than 3.75 to 4.55 times the mean grain diameter. Taking into account these two considerations, he proposed that

$$D_m^F/D_m^B \leqslant 2.42 \times (3.75 \text{ to } 4.55)$$

$$D_m^F/D_m^B < 9.1 \text{ to } 11.0.$$

Davidenkoff reported a series of piping tests on uniform sized filters and bases (data and details of the tests not published) to check the above ratio and found that the proportion D^F_m/D^B_m was between 8.8 and 10.1 which agreed quite well with the values proposed. He suggested that for conditions other than vertical flow, as the tests were performed, in the downward direction, the relationship must be smaller.

These tests with plates give a better understanding of the arching effect which can exist at a base filter interface than a real rule for filter design. Although the piping tests confirmed the Davidenkoff's approach, it seems that the filter criterion proposed cannot be applied in a general way due to the idealized packing of the grains and uniformity of the soils. If the densest grain soils packing was considered, this ratio would increase to 24.3. Nevertheless, since the tests were performed with uniform sized materials, it could be said that:

$$D^F_{50}/D^B_{50} \simeq D^F_{15}/D^B_{15} \simeq D^F_{15}/D^B_{85} \simeq 8.8 \text{ to } 10.1$$

which agrees with the results found by Bertram.

(k) <u>Zweck and Davidenkoff</u>

In 1957, Zweck and Davidenkoff ran a series of tests on uniform sized filters and bases. The objective of this experimental work was to study the relation of the ratio D^F_{50}/D^B_{50} with the range of the base soil particles using different flow directions in each base-filter combination.

A total of nine different uniform sands and gravels were tested as base and filter materials and also a medium to coarse silt was used as base soil in some tests. Three series of tests were carried out according to the flow directions and for each base the coarser filter which ensured the stability of the base was determined. The results were presented in terms of the ratio D^F_{50}/D^B_{50}.

1st SERIES - Filter tests using downward vertical flow

In performing these tests the authors had in mind the results and conclusions achieved by Davidenkoff from his work which was already presented. Each base-filter combination was placed dry and also under water in the permeameter and a visible displacement of the base grains into the filter was used as a criterion for failure of the combination. In some tests a maximum gradient of 1 was applied to the samples.

2nd SERIES - Filter tests using upward vertical flow.

3rd SERIES - Filter tests using horizontal flow

The failure of a base filter combination was detected by the visible movement of the base particles into the filter material which was a function of the gradients (minimum of 0.5 and maximum of 2.2) applied to the tests.

The critical test results were tabulated by the author and by Kinstler (1970). The resultant conclusions of this experimental work can be summarised as follows:

1. For the case of downward flow direction:

- The critical ratio D^F_{50}/D^B_{50} was independent of the hydraulic gradient but it was dependent on the particle size range of the base grains. The criterion suggested by the U.S.B.R. after Karpoff (1955) that $D^F_{50}/D^B_{50} \leqslant 10$ for granular uniform filters is satisfactory for sands with grains size greater than 0.2 mm (medium sand). For finer sands and silts this ratio is too conservative and could be of the order of 19.

2. For the case of upward and horizontal flow direction:

- The critical ratio D^F_{50}/D^B_{50} decreases with the increase of the hydraulic gradient and increases with the decrease of the base soil particles. The U.S.B.R. criterion would give a factor of safety which is in many cases too high.

3. The largest critical ratio corresponding to the smallest base size (silt size) found in all tests was probably due to some cohesion.

It seems from these results that a filter criterion based on the ratio D^F_{50}/D^B_{50} is not the most practical one since it could vary with the range of the particles sizes involved. On the other hand, Lund (1949) from her test results concluded that the critical ratio D^F_{15}/D^B_{85} was a good criterion due to its constancy in the range of fine sands to gravel materials tested.

(1) <u>Sherard</u>

Sherard et al (1963) suggested some rules to be used as filter criteria, based on a study of the rules found by Bertram, U.S.B.R., the U.S. Corps of Engineers and also based on his own practical experience in filter design. He arrived at the following conclusions:

1. $D^F_{15}/D^B_{85} < 5$

2. $D^F_{15}/D^B_{15} > 5$

3. Grading curves of filter and base materials must be roughly parallel.
4. D_{15}^{F} >0.074 mm (filters should not contain more than 5% of fines passing the No. 200 sieve and the fines should be cohesionless).
5. The maximum particle size of the base material to be considered in applying filter rules must be 1 inch.

Sherard himself recognized that these rules are in general conservative and because of this could be applied for any type of soil. He also postulated that they are too conservative if any cohesion could be considered and for well-graded coarse soils with silty fines since these materials could behave as natural filters.

(m) Belyashevskii's Experiments

Belyashevskii et al (1972) performed a series of 13 tests using different compositions of uniform and graded filters (CU minimum equal to 1.5 and maximum 33.3) and river sand as base material (CU = 2.24). The aim of these tests was to observe the stability of the filter under fluctuating flow conditions that would develop under spillways.

The experiments were performed with the use of a large permeameter of 8 in diameter with flow perpendicular to the contact and also using a horizontal tube, percolation occurring along the contact plane.

According to the results presented by the authors, an important feature observed in their tests using pulsating flow, was the formation of segregation in the filter, with the finer particles moving in downstream direction and settling.

Nevertheless, they concluded that such behaviour is favourable to the stability of the filter assisting in a posterior self-filtering effect by clogging of the larger voids at the base-filter interface. Due to this, the graded filters (non-uniform) are considered superior to the uniform ones as far as stability is concerned and it is suggested that only clay and silt size particles must be washed out if their amount is superior to 5%.

From the test behaviour and results found, the authors also observed and suggested the following:

1. The existing rules for selecting suitable filters do not reflect the physical aspect of phenomena occurring in filters during fluctuating percolation.

2. The first layer of a graded filter under fluctuating-flow regime can be designed by using a criterion relating the ratio D^F_{50}/D^B_{50} with the uniformity coefficient D_{60}/D_{10} of the filter presented in a form of graph based on the test results. For selecting next filter layers if necessary, the criterion must be checked.

The number of tests carried out by the authors seems insufficient to ensure the proposed criterion since only two tests were considered as defining unstable situations. Other experiments using this approach are rarely found in the literature to compare with the one suggested. Thanikachalam et al (1974) used these experimental data to develop their own design procedure as is presented subsequently.

(n) <u>Thanikachalam et al</u>

Thanikachalam et al (1973, 1974) presented a filter design criteria based on correlations of base and filter particle size, using experimental data from about 60 tests done by U.S.C.E. (1941), Karpoff, Leatherwood and Peterson (1954), Dayaprakash and Gupta(1972) and Belyasheskii et al (1972).They found that the existing filter criteria and design methods do not adequately provide for the gradation of the filter material for a given base soil which ensures maximum permeability and at the same time, arrest piping of the base soil.

With these test results, the authors have tried some correlations relating D^B_{10} against D^F_X/D^B_X, D^B_{15} against D^F_X/D^B_X, and CU of filter against D^F_X/D^B_X where X was equal to 10, 15, 50 and 60 percent size. From this analysis they found that the size of the base material had a significant influence on the limiting ratios of grain size of filter and base materials, the ratios increasing with the decrease of the base size. Also the finer base soil requires well graded filter material.

Using the test results and classifying them as stable and unstable situations, according to each test procedure, the authors suggested two relationships to obtain the suitable filter for a given base soil. The two rules found and presented by the authors relate D^B_{10} against D^F_{10}/D^B_{10} and D^F_{10}/D^B_{10} against CU of the filter. The following equations define these rules:

$$\log_{10}(D^F_{10}/D^B_{10} - 3) = \frac{1.55}{\log_{10}(D^B_{10}-0.001)10^3}$$

$$CU^F = 0.915\, D^F_{10}/D^B_{10} - 4.575.$$

In this way, for a given D_{10}^{B} value, the ratio D_{10}^{F}/D_{10}^{B} can be specified and subsequently the D_{10}^{F} and D_{60}^{F} values of the filter.

Although the suggested method based on empirical correlations provides the gradation of the filter by two particle sizes, the demarcation lines between stable and unstable situations do not fit very well all the results used by the authors. Also the criteria used to define failed tests may vary with each test procedure from the different sources.

3.3 Cohesionless Material - Statistical Approach

Silveira (1965) developed a new probabilistic approach to filter design, considering the migration of the base particles into the filter.

The analysis is based on an examination of the probable path along which a particle of the base with equivalent diameter d' can migrate into the filter. Once the migration paths of any base particle are determined, the filter thickness can be designed in such a way that a "clean" filter material is guaranteed after the contaminated one.

The migration path depends on the size of the voids which the particle finds in its way, and on the assumption that the probability of a grain encountering a void of particular size is the same as the volumetric probability of occurrence of that void size. The probable migration distances of invading grains can then be determined.

Silveira developed a method to determine the void size distribution of a soil from its grain size distribution. In doing so he made the following assumptions:

(i) The grains are considered spherical.

(ii) The particles of the filter material are in a dense state.

(iii) The relative positions of the grains are purely random.

The relative positions of the grains adopted by the author are those shown in fig. 3.10 where d_i, d_j and d_k are generic diameters, with probabilities p_i, p_j and p_k of occurrence. The denser particle arrangement will give a void diameter $\bar{d}$ which will depend on the particle sizes involved. The method consists of transforming the continuous curve of grain-size distribution of the filter material into a discontinuous one, finding an average diameter value d for each range. The cumulative percentage of the grain-size curve is associated with the probability p of occurrence of each diameter found. The percentage of voids $\bar{p}$ corresponding to an arrangement of three particles of diameters d_i, d_j, d_k is given by:

$$\bar{p} = \frac{3!}{r_i!\ r_j!\ r_k!}\ p_i^{\ r_i}\ p_j^{\ r_j}\ p_k^{\ r_k}$$

where r_i, r_j and r_k are the number of times that d_i, d_j and d_k respectively occur in a group and are non-negative integers such that $r_i+r_j+r_k = 3$ and p_i, p_j and p_k are the percentages of filter sizes d_i, d_j and d_k. By ordering $\bar{d}$ in an increasing order, $\Sigma\bar{p}$ can be easily obtained and each pair $(\bar{d}, \Sigma\bar{p})$ represents a point of the void curve.

Silveira utilized graphical methods for obtaining the size $\bar{d}$ of each diameter, since a mathematical expression for that was not available due to its complexity. Atmatzidis (1973) gives a general expression to obtain the void radius by determining the radius of a circle tangent internally to three circles.

Having once determined the void size distribution, the problem of washing through reduces to a comparison of this curve with that of the grain size distribution of the base material.

The total distance S traversed by one particle in n voids is found by the author as:

$$S = \frac{\log(1-\bar{P})}{\log P} d_m$$

where:

- d_m is the average value of the diameters of the grains of the filter material which is considered an estimation of the distance traversed by the base particle in each proof.

- P is the probability that a base particle meets a filter void with diameter greater than its own in one trial or confrontation.

- $\bar{P} = (1-P^n)100$ is the probability that the particle will be stopped after n voids.

Curves of S against d' (base particle diameters) for values of confidence level of $\bar{P}$ per cent assumed can be drawn as shown in fig. 3.11 and they are a guide to the establishment of the filter thickness.

Atmatzides (1973) presented a modification of the probability function presented before by assuming that the encounter probability could be better related to the cross-sectional areas rather than the volumes of voids as performed by Silveira (1965).

The author also performed a series of laboratory experiments in order to develop a qualitative appreciation of the behaviour of sand migration into an openwork gravel and compared the observed results with a theoretical migration analysis on the same soils using the method presented by Silveira (1965) and that modified by himself. The author arrived at the following conclusions:

1. Other factors not considered in the theory such as hydraulic gradients and time affect the migration process. Higher hydraulic gradients resulted in longer migration paths. The time needed for a stable condition to occur was influenced by the original sand content of the gravel and by the mechanical composition of the sand.

2. The modified probability function suggested by the author predicted better the observed sand migration pattern than that presented by Silveira (1965).

De Mello (1977) presented extensive analysis of Silveira's theory as compared with the available existing comprehensive tests and concludes that this theoretical formulation, even considering that in principle it could be the most sound of all, was not satisfactory mainly because of the hypothesis on pore sizes distribution due to the oversimplified particle size, shape, distribution and packing assumed. Because of this he also concluded that this theory was not likely to be used confidently on any soil other than a very uniform, cohesionless soil with rounded particles only.

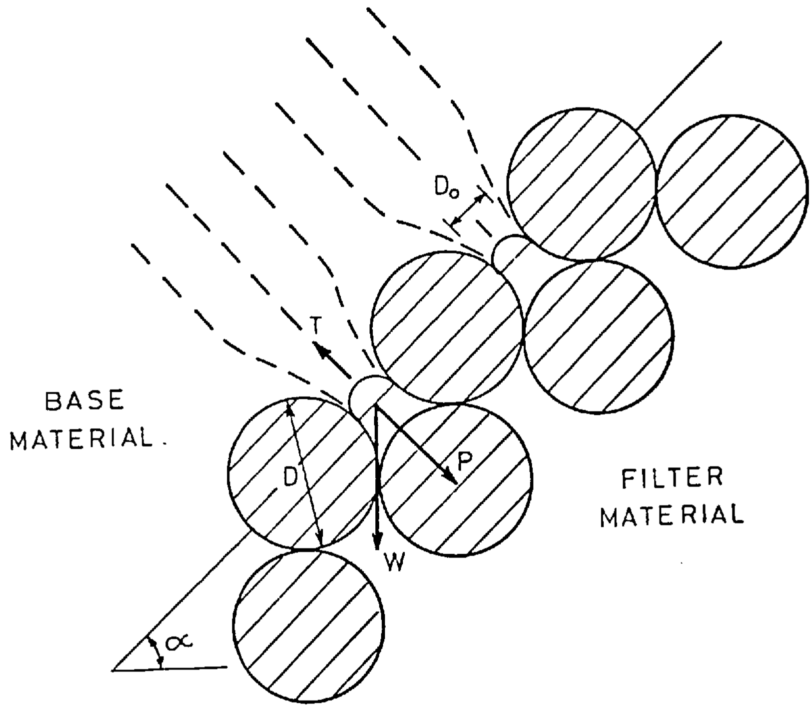
3.4 Cohesive Material - Intact Conditions

A filter located downstream of an embankment core will almost always be protecting a very fine soil from being carried away by the percolating water. If the same filter rules approach developed for cohesionless soils should be applied, the ratio between base and filter grain sizes would imply a very fine filter soil probably having cohesion. What has been found in practice is that a fine cohesive base material requires coarser filters than those proposed by many filter criteria. The cohesion developed among the grains ensures that the interparticle forces are strong enough, in most cases, to hold the grains together even when high hydraulic gradients are involved and despite the weight of such small particles. The following studies were carried out with the purpose of finding out a more realistic mechanism and establishing filter rules for such soils.

(a) Davidenkoff

The first filter design method for cohesive soils was proposed by Davidenkoff (1955). He suggested a mechanism for the behaviour of the interface between a cohesive base and a cohesionless filter, taking into account possible forces between clay particles.

The author proposed the particles arrangement as shown below.



where:

W = weigth of the base soil particle

T = tensile force acting around the base soil particle

P = seepage force acting at the base soil particle perpendicular to the interface.

By the equilibrium of these forces acting at a base soil particle at the interface and considering a required factor of safety in the direction of the seepage force, he proposed that the filter particle size diameter should be:

$$D_o = \frac{\text{limit surface of entrance} \times C_o}{F(\gamma_a.\cos\alpha + \gamma_w.i)}$$

and considering that the failure mechanism was related to a given "floc" size detachment of the base, as will be presented, he wrote the equation above as:

$$D = \frac{15 \times C_o}{F(\gamma_a \cos\alpha + \gamma_w . i)}$$

where

D = mean diameter of filter particles (D/D_o considered to be 2.42 due to the assumption of the particles arrangement)

D_o = corresponding void diameter size of the filter

C_o = tensile strength of base soil

F = factor of safety (proposed to be of the order of 3)

γ_a = submerged density of base soil

α = angle of filter bed with horizontal

γ_w = density of water

i = average hydraulic gradient.

To check the mechanism proposed and to support some assumptions made as to the size of the "clay floc", the author devised a series of experiments using four different mixtures of clay, silt and sand.

The set-up of the equipment used is shown in fig. 3.12. The tests consisted of constant head permeability tests and the water percolating through the soil was allowed to escape by a hole of fixed size made in the centre of the plate which supported the soil. These types of tests were also performed with non cohesive materials as presented in section 3.2(j).

In all cases "failure" was characterized by the detachment of a sample piece of clay with the approximate shape of a segment of sphere with a height of the order of 3.5 mm.

Davidenkoff (1955) also determined the "tensile strength" of the mixtures using traction rupture tests and by comparing the results with those from the expression shown before, he concluded that the values of C_o obtained in this way could be used in this expression to determine the D value.

The author also concluded from his studies that even using high gradients, for all practical purposes, any filter could be used for cohesive base materials even with very small "tensile strength".

Although the experimental results confirmed the failure mechanism proposed, they did not test the theory adequately and the following limitations can be pointed out:

- The idealized shape and packing of the particles of base and filter soils.
- The theory takes into account only very uniform filters.
- The average hydraulic gradient to be used is assumed to be the exit gradient which is difficult to predict.
- The "tensile strength" of soils is also difficult to predict.

(b) Zaslavsky and Kassif

Zaslavsky and Kassif (1965) proposed a mechanism of failure through piping in cohesive soils, similar to that presented by Davidenkoff (1955) which was discussed previously.

The proposed theory differs from Davidenoff's theory in the theoretical considerations of the forces acting in a base particle at the interface.

Assuming that the stability of a soil element at the surface of the cohesive soil, which could be a single particle or an aggregate of particles, was given by the equilibrium of the forces acting in the soil element, the factor of safety against piping could be defined as:

$$F_s = \frac{F_c}{F_g \cos\alpha + F_d}$$

where:

F_s = factor of safety

F_c = resultant of the cohesive force (resistant force corresponding to the force T presented in the previous section)

F_g = gravitational force } (driving forces corresponding to the forces W and P presented in the previous section)

F_d = drag force }

α = angle of filter bed with horizontal

These forces were determined as:

(1) The component of the gravitational force in the normal direction in relation to the interface ($F'_g = F_g \cos\alpha$) was determined from the submerged unit weight of the soil as:

$$F'_g = -V(G-1)(1-n)\gamma_w \cos \alpha$$

where:

V = volume of the element or aggregate

G = specific gravity of the solids

n = porosity

γ_w = unit weight of water

(2) The drag force was evaluated by considering that this force is proportional to the drag force acting macroscopically, but corrected by a shape factor a_1. In this way F_d was found to be:

$$F_d = -a_1 V \gamma_{wj}$$

j = component of hydraulic gradient normal to the interface

(3) The "tensile force" defined as the component of the resisting force, parallel to the driving forces and in opposite direction to them, was expressed as:

$$F_c = a_2 A \sigma_t$$

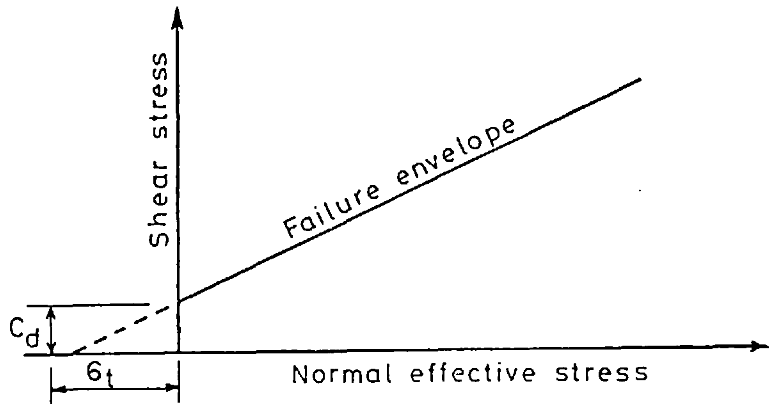
where:

a_2 = geometric coefficient

A = area of the projection of the particle on the plane normal to the direction of the driving forces

σ_t = tensile strength of the soil.

In order to estimate the value of σ_t the authors associated the "tensile strength" with the cohesion of the soil as illustrated schematically below:



The failure envelope represents the result of a drained test on a saturated cohesive soil in terms of effective stresses. The maximum possible tensile strength will be larger or of the same order of magnitude as the cohesion.

Considering these forces and substituting them into the equation of equilibrium, the factor of safety against piping was found to be:

$$F = \frac{a_3}{\bar{d}} \; \frac{a_2 \sigma_t}{-\gamma_w[(G-1)(1-n)\cos\alpha + a_1 . j]}$$

where:

$\frac{a_3}{\bar{d}} = \frac{A}{V}$, $\bar{d}$ being a mean diameter of the particles or the mean size of the aggregates and a_3 a geometrical coefficient.

This expression was also expressed as:

$$F = \frac{a \; \sigma_t}{\bar{d}\gamma_w[-a_1 j - (G-1)(1-n)\cos\alpha]}$$

where:

$$a = a_2 . a_3$$

For overconsolidated cohesive soils the value of $(a\sigma_t)$ is very large in comparison with the submerged unit weight and the equation above can be simplified, resulting in:

$$F = \frac{b\sigma_t}{\bar{d}\gamma_w j}$$

where:

$b = a/a_1$, a dimensionless soil constant of the order of unity, dependent on the mean size and geometry of the particles or aggregates washed out.

Kassif et al (1965), using the equation above, established a relationship between the hydraulic exit gradient j and the pore size of filter material shown in fig. 3.13 for a particular clay. Assuming σ_t constant for a certain range of washable sizes, straight lines can be obtained for different factors of safety. Since the critical hydraulic gradient j_c is estimated, the critical $1/d_c$ value may be determined directly. Under this gradient, the pore size of the filter material should be selected to the right of $1/d_c$ depending on the factor of safety adopted.

Kassif et al (1965) also presented a series of experiments performed which included permeameter tests with filter material and tests using plates with holes as carried out by Davidenkoff (1955) and presented earlier.

These tests had the purpose of determining the suitability of using crushed aggregates as filter materials for a compacted fat clay having 55-65% of the particles smaller than 5 microns and exhibiting high swelling properties.

In the first series of the experiments, seepage tests were performed on the compacted clay, 2.5 cm thick, placed in contact with the crushed aggregate inside transparent permeameters. Two types of procedure were adopted in doing these tests: firstly, the clay was completely confined in the permeameters, simulating a condition in which the clay was not allowed to swell, secondly, the clay was allowed to swell freely. The maximum average gradients reached in all tests was 50. The authors reported that no failure, either partial or complete,was observed in tests during two months and no clay particles were washed out into the filter materials even under high gradients.

As a result of these tests, fine crushed aggregates were reported to be used extensively and successfully in most cases in various structures in Israel as a filter material in contact with expansive clays.

A second series of tests was performed, similar to those performed by Davidenkoff (1955) and presented earlier. This time various different plates with different hole diameter were used, each plate having various uniform circular holes instead of only one hole as used by Davidenkoff. A clay layer of 3.0 cm thick was compacted to a predetermined density within the permeameter, saturated and allowed to swell under surcharges varying from 0.0 to 0.2 Kg/cm^2. A high hydraulic gradient (minimum of 10 and maximum of 1000) was applied by successive steps of 10, kept at the same value for two weeks. The time required for failure varied accordingly from several weeks to nearly eight months, depending on the surcharge.

After failure, the tensile strength of the material was estimated by the compressive strength obtained from laboratory vane and consolidated-undrained triaxial test with pore pressure measurements.

The actual critical exit gradients were estimated based on hydraulic considerations of seepage through the dome-shaped cavity (Zaslavsky and Kassif, 1965) using:

$$j_c = \frac{2q_{cr}}{\pi D^2 k}$$

where:

$$q_{cr} = \frac{\text{total discharge measured}}{\text{number of holes}}$$

D = diameter of hole

k = permeability of the material.

The gradients calculated in this way were much higher than those average measured gradients.

It was concluded that the exit gradients were independent of the hole diameters being only a function of clay properties and γ_w, meaning that the critical flow causing failure in the base soil was independent of the size of filter material.

The authors compared the theoretical analysis by the use of j_c into the graph presented in fig. 3.13, with the permeameter tests presented before. They found a reasonable correlation between the maximum filter size predicted and that from experimental tests.

The assumptions considered in this analysis seem to be more realistic than those done by Davidenkoff (1955). Although the theory has been confirmed by the experiments performed, it has in general the same limitations as those of Davidenkoff's since no reliable procedure has been developed for estimating exit hydraulic gradients and the value of σ_t is usually not known to any degree of accuracy.

(c) Wolski et al

Wolski et al (1970) presented results of an experimental work carried out on the Tresna Dam by the Soil Mechanics and Earth Structures Department of Warsaw Agricultural University.

The Tresna Dam, 38 m high, has a thin core of silty loam of volcanic origin and protective layers constructed of alluvial soils. The volcanic origin soils have the particular characteristic of their water content being above the optimum value (given by the Proctor Standard), showing a soft consistency when compacted and having a low shear strength. Due to this soft consistency the authors investigated the assumption that a core of this material would not present a great risk of cracking formation caused by arching.

During the construction of the dam, settlement measurements of up to 6 cm were observed between the upstream and downstream sides of the crest. Based on these observations, simplified model tests, simulating a "sudden fault" on the contact plane between the core and protective layer were performed to investigate the possibility of formation of cracks in the core. The water content of the base soils used in the tests, varied from $w = 1.0\ w_{opt}$ to $w = 1.7\ w_{opt}$. Although the model test results are only qualitative, they showed that probably only small local and negligible cracks could be developed in the Tresna Dam core, where the material had $w = 1.3\ w_{opt}$.

Two years after completion of the dam, two holes were made in the core and no crack was observed as the test results had predicted. The authors extrapolated this type of behaviour for all volcanic origin cohesive soils of soft consistency.

Another series of tests was carried out to observe the core-filter interface behaviour. Three different models were used, simulating base soil in contact with a plate having a hole of specific size, base soil in contact with the filter material in intact conditions, and base soil in contact with the filter material having a horizontal fault along it. The water content of the finer soil was also varied.

The model test results showed that piping in the contact zone between the base soil and filter had a rheological character of volume flow and that the soil in the disturbed zones was of a very soft consistency. Associating these disturbed zones to the "Bingham body" and considering the equilibrium of this "body" according to the Buckingham-Reiner equation, they determined that:

$$2\tau_o \ell = r \Delta p$$

where

τ_o = yield stress value (g/cm²)

The meaning of the other terms is shown in fig. 3.14.

The authors concluded from the model tests carried out that:

- Dam core of soft consistency is resistant to cracking caused by the arching process.
- The resistance of piping of such core dam depends on its 'rheological yield value' (presumably related to a viscous fluid behaviour because of the material consistency) as well as the pore size of the protective layer.

A filter design criteria was proposed in the form of a nomograph (fig. 3.15) which was developed by assuming that the piping resistance was given by the following condition:

$$i\gamma_w < 2(\tau_o/r)$$

where

$$i = \frac{H}{L} = \text{hydraulic gradient.}$$

In this way, since the shape of the core and the plasticity index I_p of the material are known, the filter material to protect it against piping is given by the D_{17} diameter size and the uniformity coefficient CU.

The criterion proposed although very simple to use, is restricted to soils similar to the one quoted and for the same conditions of placement. It also seems that the models used to simulate cracks in the core were over-simplified and the field observations of no cracks by the holes made in the embankment after completion, can be associated to cracks that have been sealed by self-filtering process since the filter size used seems to be rather conservative for cohesive soils. If this possibility exists, the segregation of the particles can also exist and the sealing of the cracks would not be guaranteed as a general rule. The non-intact situation should then be considered.

De Mello (1977) called attention to the fact that most of the tests performed in the laboratory in cohesive materials do not take into account many variables involved in relation to actual field conditions as compaction gradients, construction imperfections, loss of cohesion with time, etc. The construction procedure, according to De Mello (1977), has a marking effect on the interface formed, very different from the one prepared in the laboratory by compacting clay inside a metal cylinder. Therefore it is concluded that even if the laboratory tests performed do show some useful results particularly in relation to the relative influence of some of the factors involved, they only have a limited value in establishing criteria of filters for intact clays.

3.5 Cohesive Material - Non Intact Conditions

3.5.1 Introduction

The experimental published data dealing with cohesive soils in general are very few. If there is not a filter criterion for these soils, at least it is known that when intact conditions are guaranteed the usual granulometric distribution for cohesionless soils is conservative as already commented before. On the other hand when non intact conditions are susceptible to occurring such as due to cracking and dispersive soils, the tensile strength approach is not applicable. Once internal erosion occurs, erosion debris will be generated. If there is slow erosion, then segregation may occur and only the smallest particles will arrive at the filter. A filter to be effective, must be designed to prevent the passage of such particles.

It has been found that if the clay is non dispersive, the smallest particles generated in the erosion process will be "clay flocs", usually of medium silt size due to the flocculation of the soil. The process and conditions which are favourable to the occurrence of "clay flocs" will be presented later on. On the other hand, if the soil is a dispersive one, deflocculating conditions exist and the smallest particles will be actually the individual clay particle sizes, in such a way that they can only be retained by a cohesive filter. The dispersive soils have already been briefly discussed in the last chapter.

In general, the design procedure which has been adopted lately for non intact conditions, is to perform laboratory tests with clay particles in suspensions in water, passing through a filter.

3.5.2 Flocculation Conditions

In 1940 Cedergren designed filters by passing a slurry of clay through filters of various gradations. The coarsest filter which retained the clay particles was selected and performed satisfactorily in practice.

Vaughan et al (1970) performed some tests with non-dispersive boulder clay to design a "perfect filter" to the Cow Green Dam, 25 m high. As the possibility of cracks formation had to be taken into account, the filter design considered the eroded particles which had to be prevented. The likely chemistry of the water which would percolate through the core was first studied. The grading curve of the clay was obtained in two ways: one using a hydrometer and dispersant, following the British Standard sedimentation test procedure, and the other using the river water in the same way as the one before but with mechanical dispersion of the soil only, without chemical pre-treatment or dispersant. By the comparison of the two curves obtained, the flocculation conditions of the material could be detected. The material composed of the "clay flocs" was separated by sedimentation and passed as a dilute suspension in the same water through various gradations of filter material. The results of such tests are presented in fig. 3.16. The coarsest one to retain the flocculated material was selected as the filter for the embankment. It can be observed that when the 5% of fines passing in nr 200 sieve of the natural sand used in the tests, were eliminated, the sand did not retain the clay suspension any more and its permeability increased considerably. Also the permeability of the natural sand was similar to the uniform sand also used in the tests, suggesting that the effectiveness of the filter could be related to its permeability. The same behaviour was found when the author designed the Empingham dam filter which is presented later.

Kinstler (1970) presented the results of tests carried out by Naranjo (1970) to determine the effect of mixing time on the grading of boulder clay slurried in distilled water, London tap water, and distilled water containing 0.3% sodium hexametaphosphate. The particles in suspension were smaller than the nr.200 sieve size. The resulting material was passed through a filter of size 75-30 microns and each suspension contained all sizes less than a predetermined maximum. The results of the tests showed that suspensions of 1.2 micron maximum size or larger were retained by the filter.

Vaughan (1978) presented the design of the Empingham dam filter constructed between 1971 and 1975. The fill and the core were of Upper Lias clay. A vertical internal filter was provided and designed in the same way as the Cow Green dam filter quoted before. The flocculated clay considered to be typical for the design was also obtained by mechanical dispersion in the probable reservoir water and tested. The filter material used was a washed natural sand to which different amounts of silt were added. It was found that the addition of up to 5% silt decreased the permeability of the filter significantly and more than that had less effect. The addition of about 2.5% of silt was sufficient to make the filter effective in retaining the clay. The specifications for the filter required that it had to have about 5 to 15% of non-cohesive fines and to have a permeability of less than 10^{-4} m/sec. The natural unwashed sand used in tests presented these characteristics and was used as the filter dam. The gradations of the flocculated clay, of the washed sand and of the filter chosen are shown in fig. 3.17 and the test results in table 3.1.

Vestad (1976) published data on Vidalsvath Dam, constructed in 1971 where leakage and cracking have been detected. He has performed filter investigations in laboratory to test screened tunnel spoil material as an effective filter to prevent the transport of the particles from the core of glacial moraine. The test results were positive even if the moraine was not compacted and large quantities of water were allowed to pass with high gradients. In the experiments were also included artificial cracks, simulating field conditions. Details of these experiments were not given by the author. Although the laboratory test results were satisfactory, in practice the filter had failed. During impounding of the dam, cracking and erosion occurred and the author attributed the breakdown of the filter to arching of the angular particle shape of the filter material, not retaining the eroded fine particles. The author suggested that if the filter is of this type of material, having sharp-edged grains, the filter zone will not collapse as easily as it would in a filter of natural sand and gravel with rounded grains, when a crack is created.

Based on his experiences with Balderhead, Cow

Green and Empingham dams, Vaughan(1976) suggested a new approach when designing filters to protect cohesive soils in non-intact conditions. He found that a relationship between the medium "floc clay particle" to be retained and the permeability of the filter could exist based on the suspension test results for Cow Green, Empingham dams and on Naranjo's test results. He plotted the results in the way shown in fig. 3.18. The horizontal lines mean the permeability difference between the filters which retained and did not retain the considered clay floc.

Vaughan (1976) developed his approach by considering that if the coefficient of permeability varies with the square of the pore size and since the size of the floc which will pass through the pore size is expected to relate linearly with the pore space of the filter, the following relationship could be written:

$$k = A\, d^2$$

where:

k = permeability

A = constant to be determined

d = clay floc size.

The design criterion would be that once the floc size to be retained is determined, say by the method used in Cow Green filter tests presented before, the filter would be specified by its permeability.

The author has found that the floc size varies with the salt concentration of the water used in the sedimentation tests. Figure 3.19 shows the test results for several British clays. It can be seen that the floc size increases up to a soil concentration of 10 mEg/ℓ, becoming practically constant after that. He also suggested that the "floc size" varies with the concentration of the clay in water and he observed that it decreased with the increase of the amount of soil. A standard concentration of clay seems then to be desired in both sedimentation and filter tests.

To establish the relationship proposed as a criterion for filter design, more experimental data would be necessary and this was the main objective of this thesis as will be presented in the Chapters 4,5 and 6. Although the empirical relationship found was not exactly the one proposed, it seems that there is a similar type of relation between filter permeability and floc size.

3.5.3 Dispersive Conditions

There is not yet any design criteria for filters when dispersive soils are used as base material. There are a lot of uncertainties and unknowns in relation to this problem although they have been the subject of much interest lately.

The Australian investigators have concluded that the clay particles in these soils which are carried away in suspensions are so small that even a fine sand filter is not efficient to hold the particles.

Sherard et al (1972b) have speculated that a fine sand filter could be useful to prevent piping failure in dispersive fine grained soils. They supposed that the silt size particles and fine sands are also carried through the voids of a conventional sand filter, preventing the next smallest particles of the base material from doing so as well.

Another idea developed from Olson (1974) who carried out drained shear tests on clay minerals including both calcium sand and sodium montmorillonite, is that a large enough arch may form in a dispersed soil to span the maximum pore size in a normal filter material.

Sherard et al(1976a) have reported that laboratory filter tests using many different dispersive clays in which water is made to flow through a hole (1/4 to 1/2 in diameter) in a compacted clay specimen (6 to 9 in long) and discharged into a medium sand filter, showed satisfactory results. A hydraulic head of about 3 ft of water was used in the tests. They found that at the beginning of the tests, colloidal clay particles were washed through the filter without clogging it and after the first 10 to 25 min the water cleaned completely or the hole is sealed.

Bourdeaux and Imaizumi (1976) found similar results with tests using larger compacted clay specimens (2 by 2 by 2 ft) with a 1/4 in slot for the leakage channel.

There are,therefore, controversial opinions about the prevention of the eroded particles of a dispersive soil being washed away. Repulsive forces can develop between dispersed particles and if this happens the tensile strength approach will be destroyed. Also the self-filtering process may not be possible since a significant amount of the finest particles may be lost before the graded filter starts to build up and failure might have already occurred. Sherard and Decker (1976) also postulated that dispersive clay erosion damage can only occur as the result of progressive erosion of a concentrated leak and not as the result of gradual seepage of water through the pores of the clay mass. In this way, if a crack occurs for example, the risk of segregation of the largest particles must be considered. The smallest particle to be prevented by the filter would be the clay particle size, and if the filter must guarantee the retention of these particles, it would probably not be possible to use a cohesionless filter material.

3.6 Main Points

From the literature review of several filter design criteria the following points should be noted:

- The main difference in filter rules found by different authors could be due to the failure test criterion adopted by each one.
- A large number of tests already carried out with cohesionless materials are nevertheless probably sufficient to design base-filter combinations for such soils.
- When cohesive base material must be used, these rules can be overconservative due to the "tensile strength" of the soil. On the other hand, if non intact conditions must be considered, the approach first suggested by Vaughan (1976) seems to be

the more reliable.

- Rules of filter design for dispersive soils are not yet well established although it has been found from a few specific tests that sands could be used successfully as filters for these soils.

TABLE 3.1 Empingham Dam Filter Test Results

% fines added	permeability of the filter (m/s)	test results
0	4×10^{-4}	immediate failure
2.5	1.5×10^{-4}	filter operated after 5 hours with considerable build up of clay in it
5.0	7×10^{-5}	filter immediately effective, clay building up near filter surface
10.0	5×10^{-5}	filter immediately effective
15.0	3×10^{-5}	filter immediately effective

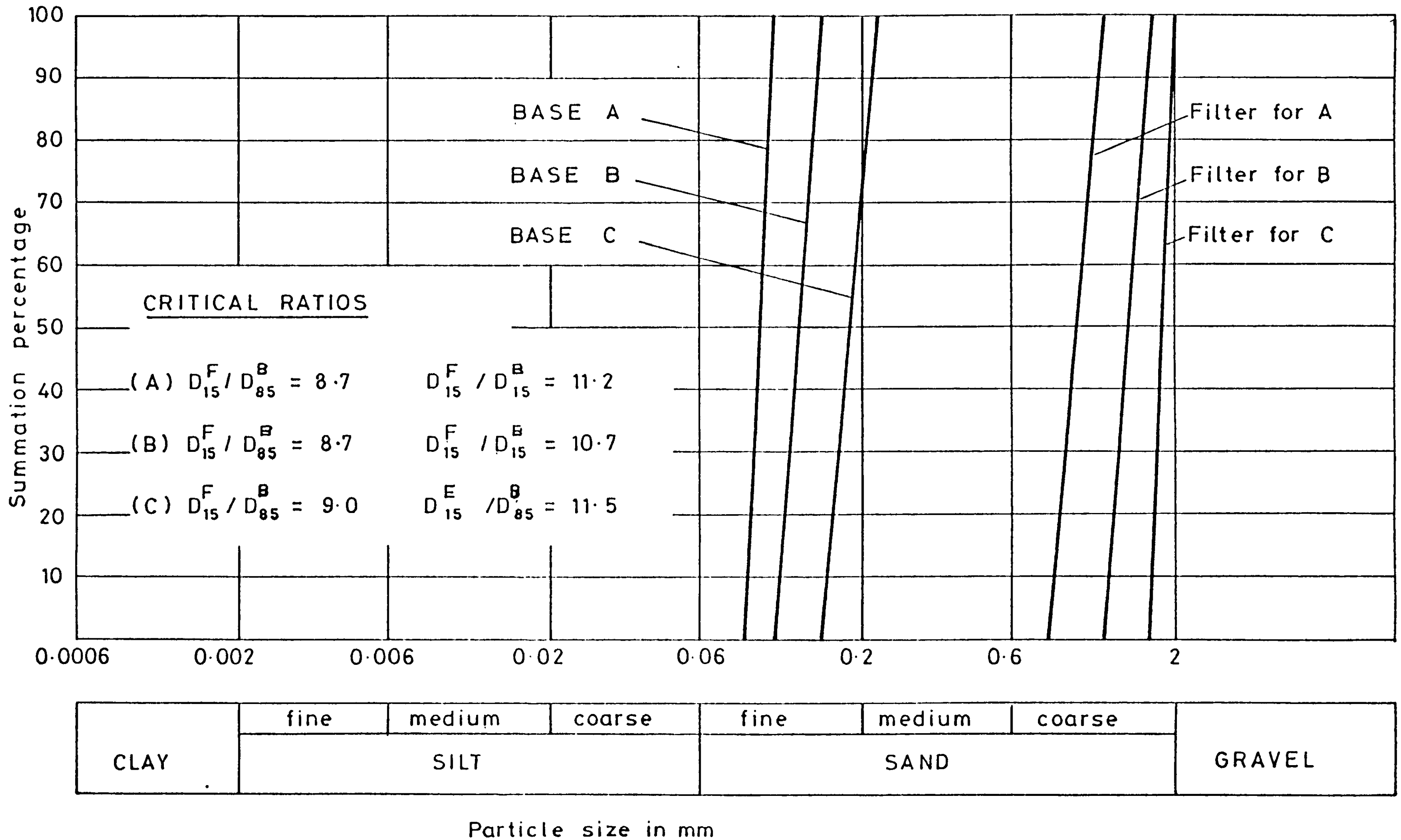


Fig. 3.1: Bertram's critical test results - 2nd Series (i)

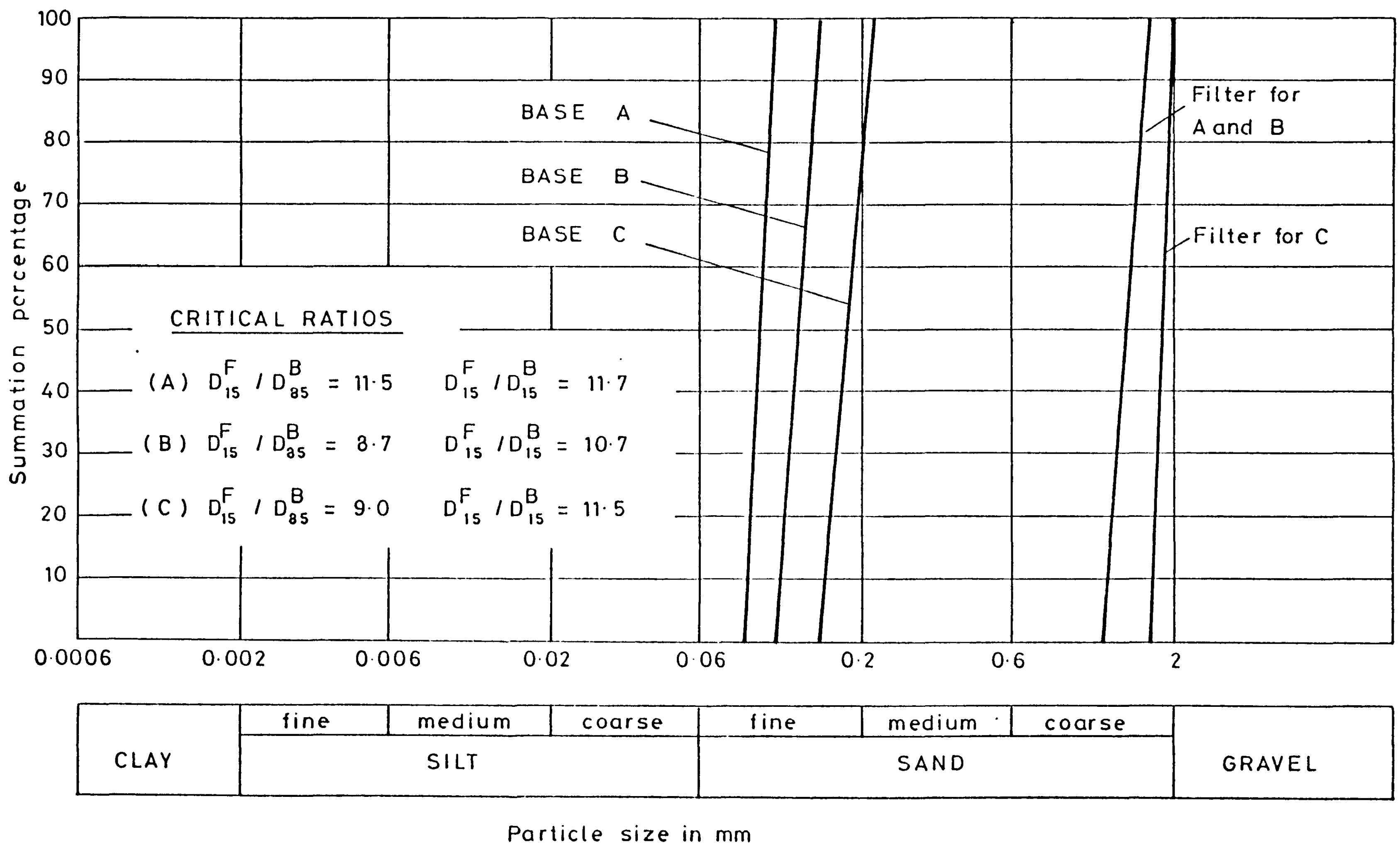


Fig. 3.2: Bertram's critical test results - 2nd Series (ii)

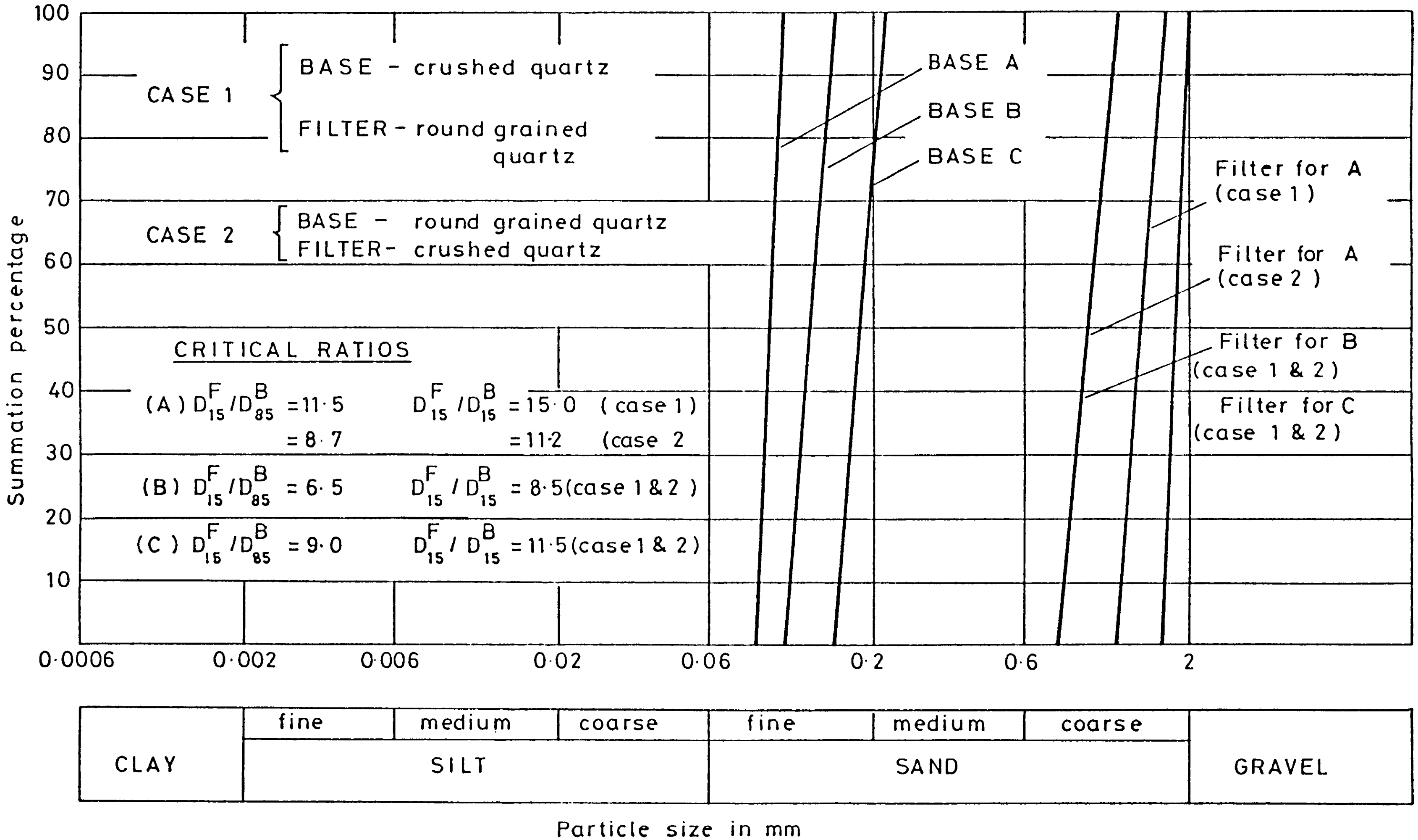


Fig. 3.3: Bertram's critical test results - 2nd Series (iii)

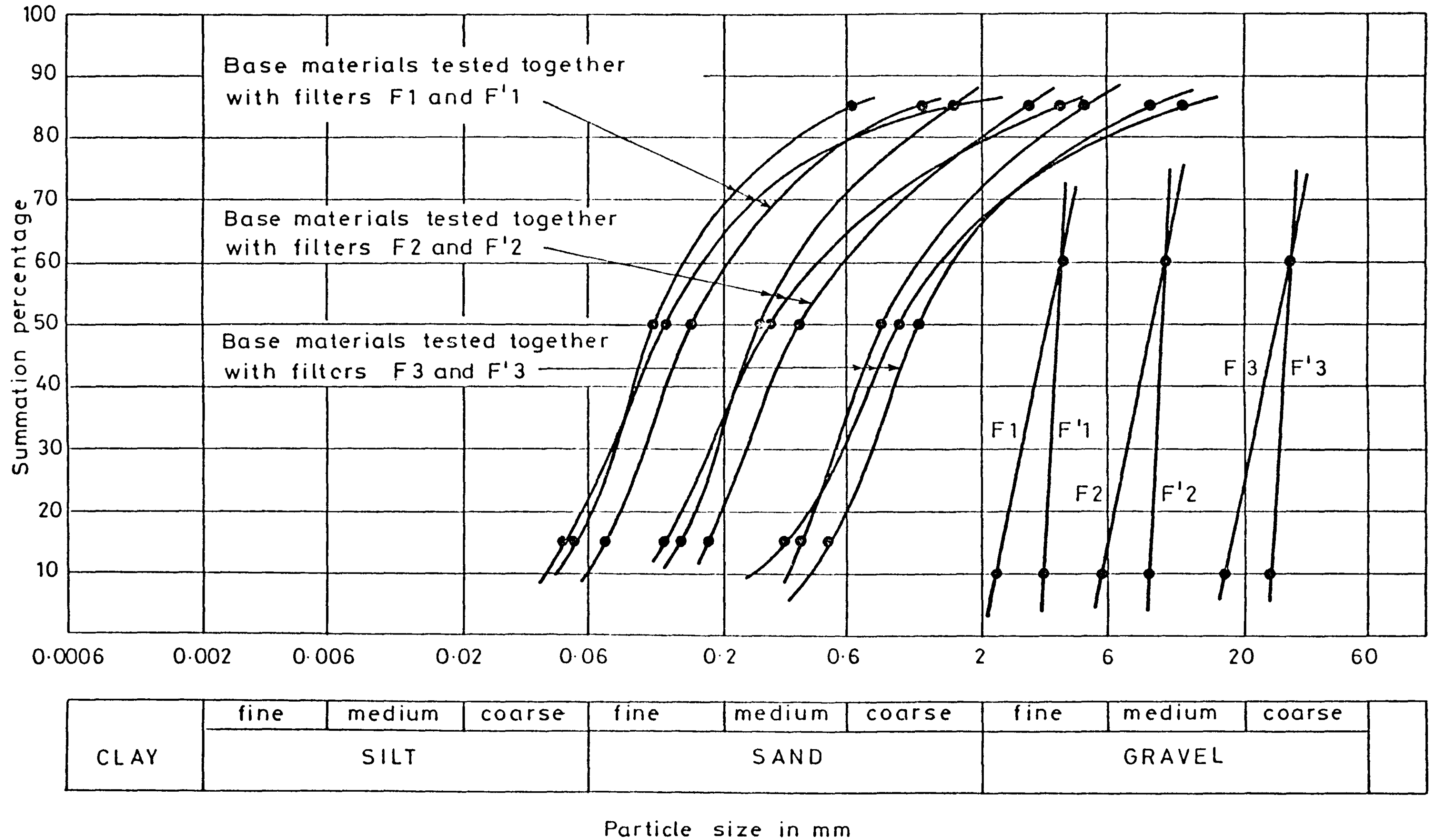


Fig. 3.4: Estimated base-filter combinations for Newton and Hurley's tests

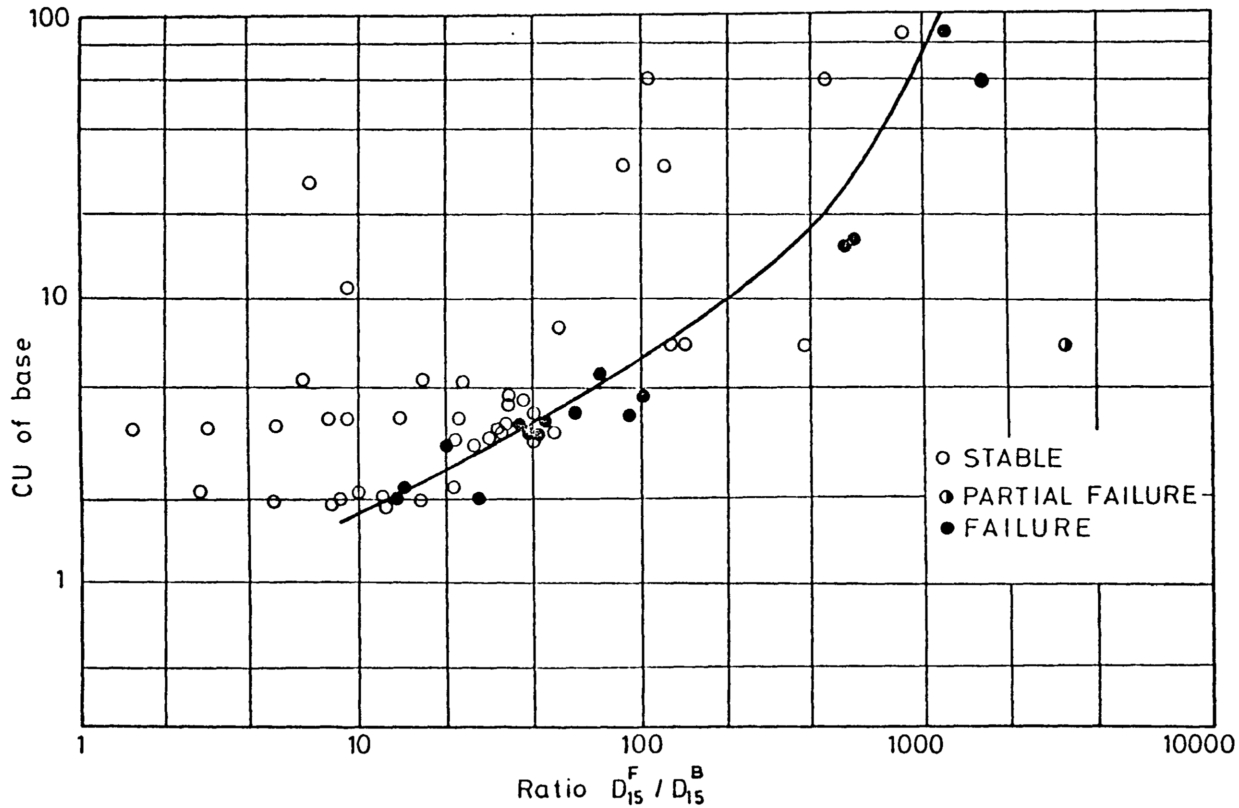


Fig. 3.5: D_{15}^{F}/D_{15}^{B} against CU of the base (after Providence District, 1942)

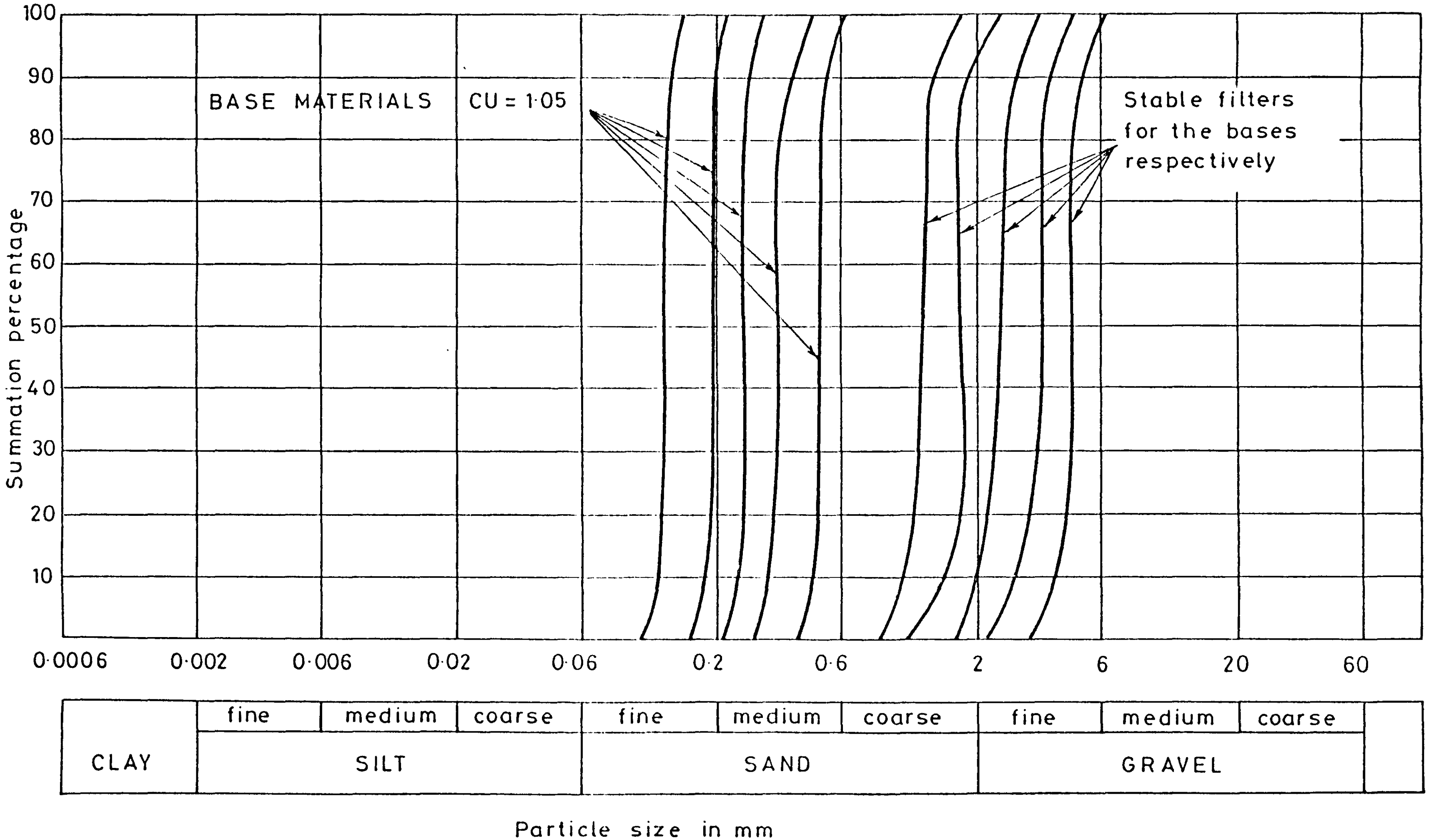


Fig. 3.6: Lund's critical test results - 1st Series

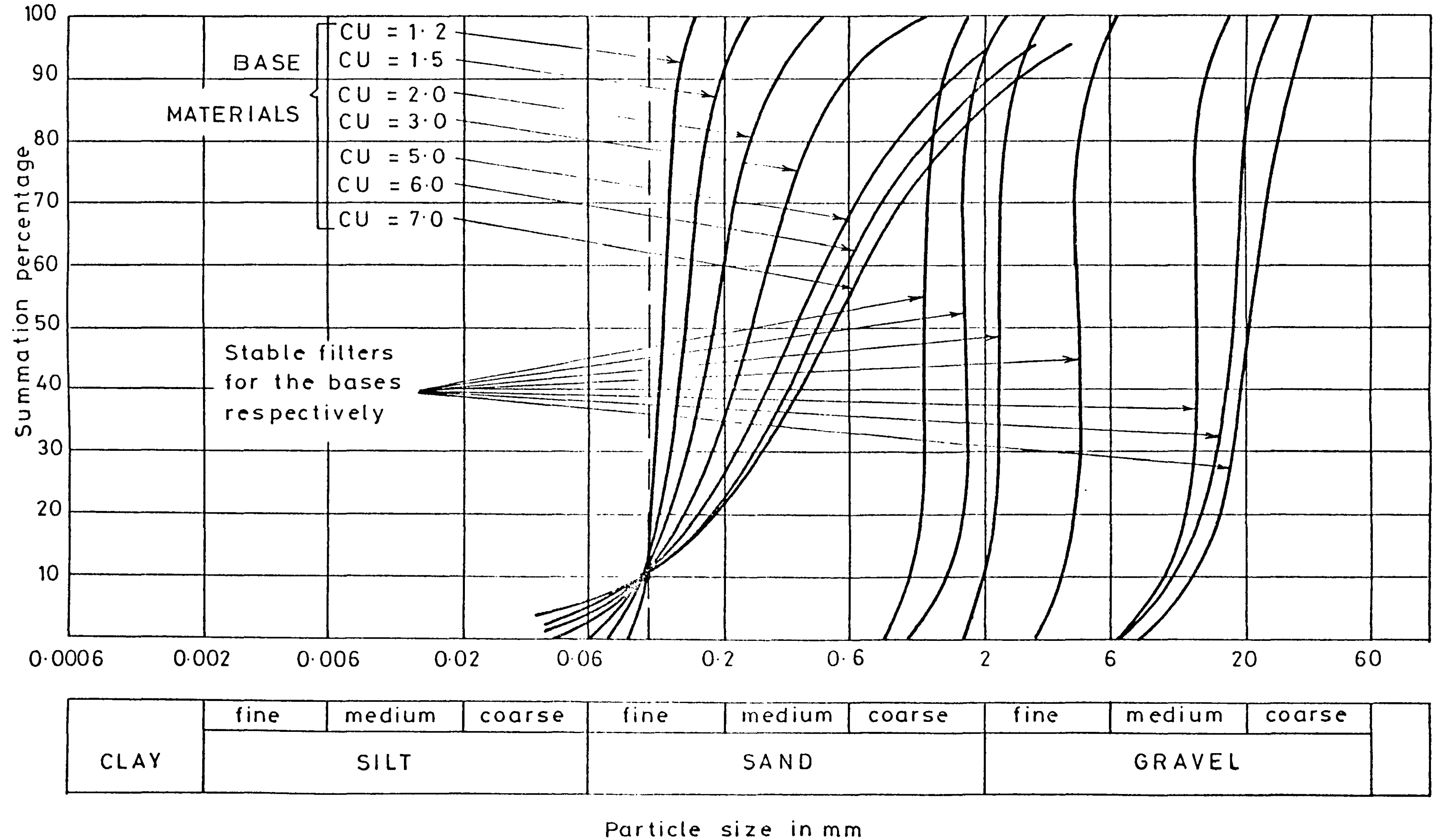


Fig. 3.7: Lund's critical test results - 2nd Series

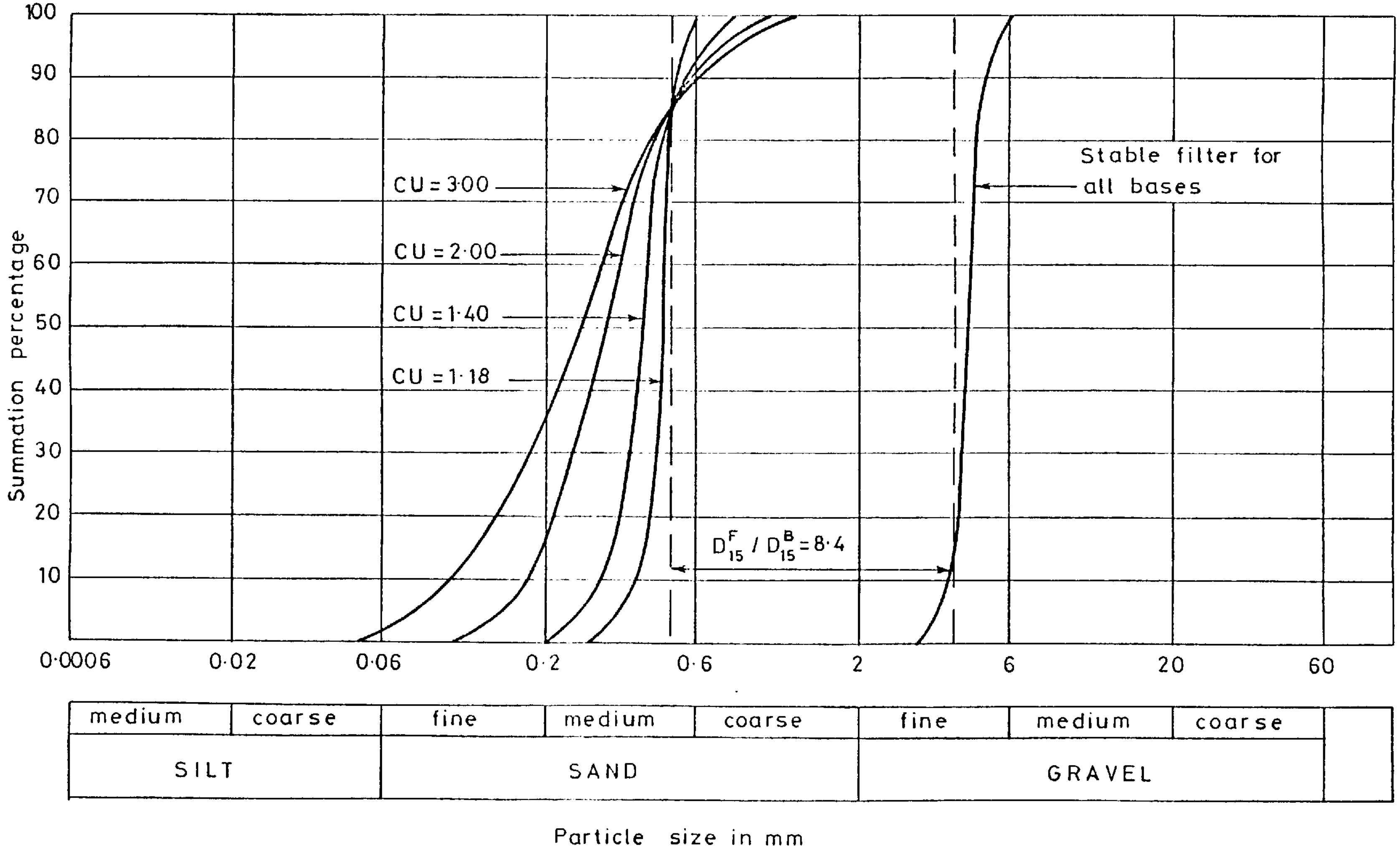


Fig. 3.8: Lund's critical test results - 3rd Series

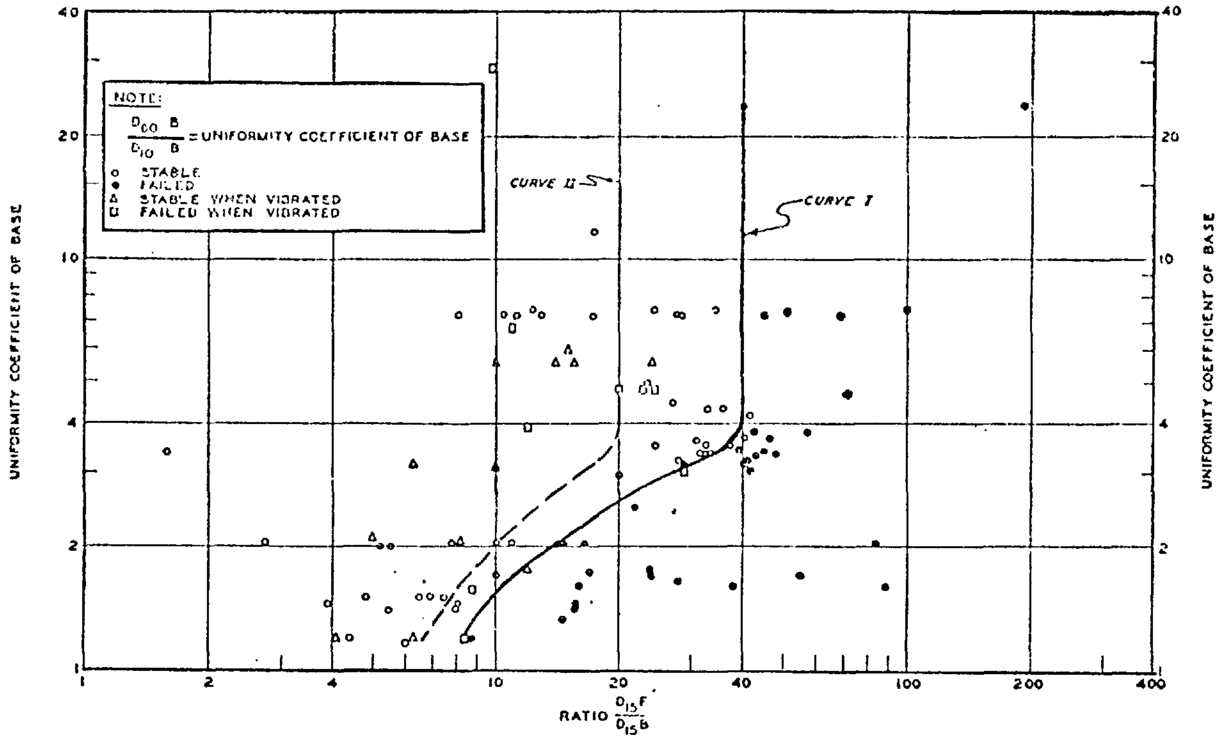


Fig. 3.9: Plot of filter ratio D^F_{15}/D^B_{15} against CU (base) (after U.S.C.E. 1953)

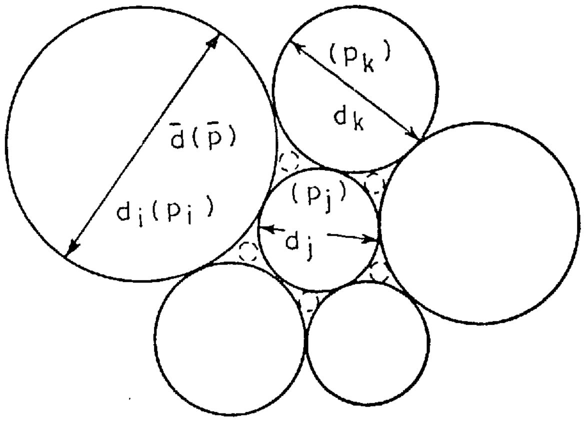


Fig. 3.10: Idealized filter grains packing (after Silveira 1965)

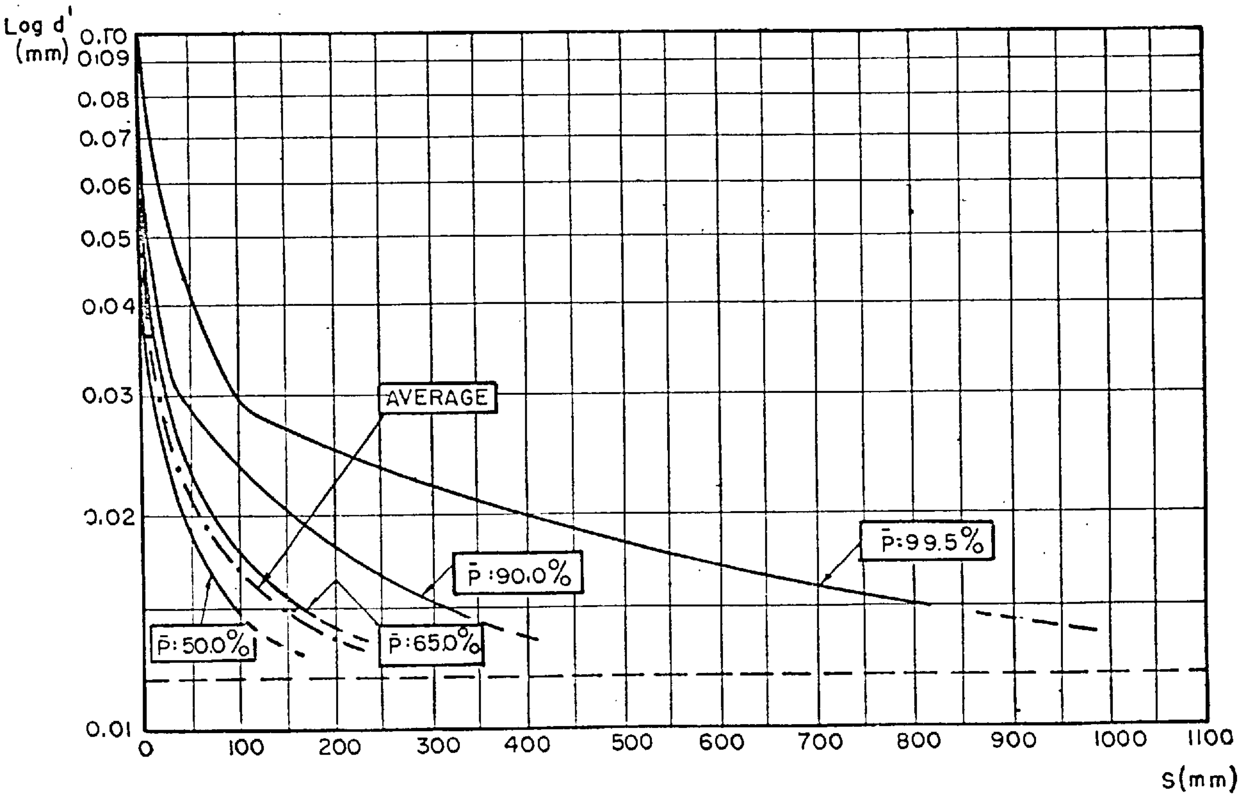


Fig. 3.11: Curves of S against d' for values of confidence level $\bar{P}$ per cent (after Silveira, 1965)

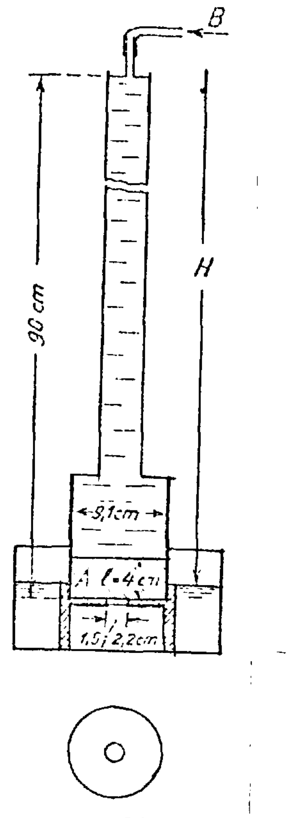


Fig. 3.12: Set up for filter tests
(after Davidenkoff, 1955)

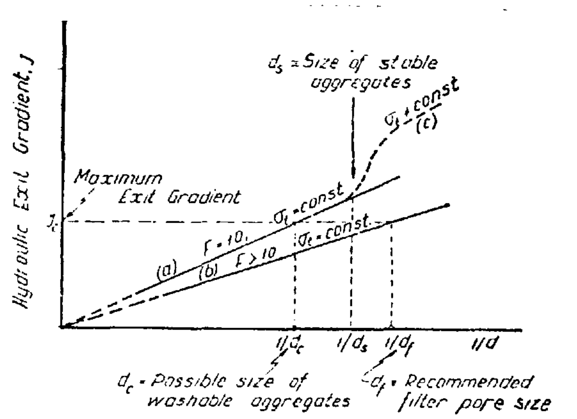


Fig. 3.13: Filter design method -
cohesive soils (after Kassif, 1965)

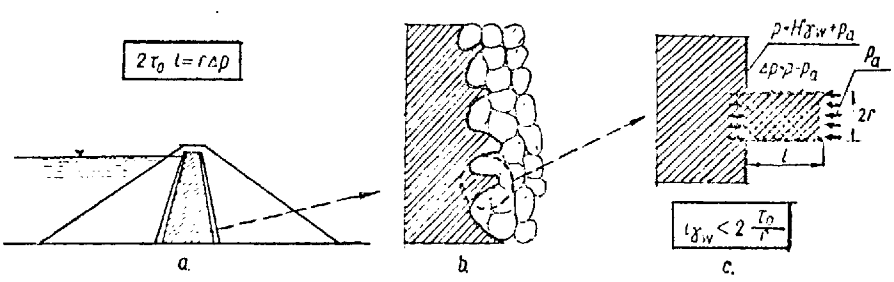


Fig. 3.14: Scheme for formulation of the criterion of piping resistance (after Wolski et al, 1970)

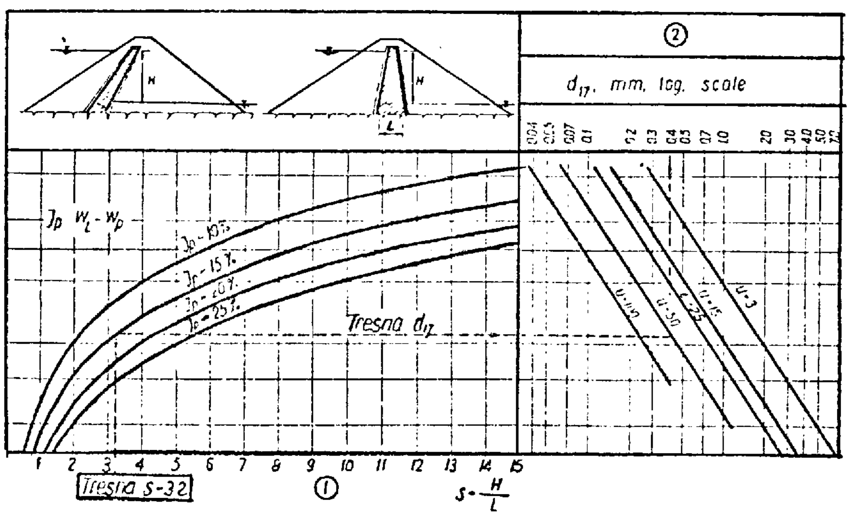


Fig. 3.15: Design filter procedure (after Wolski et al, 1970)

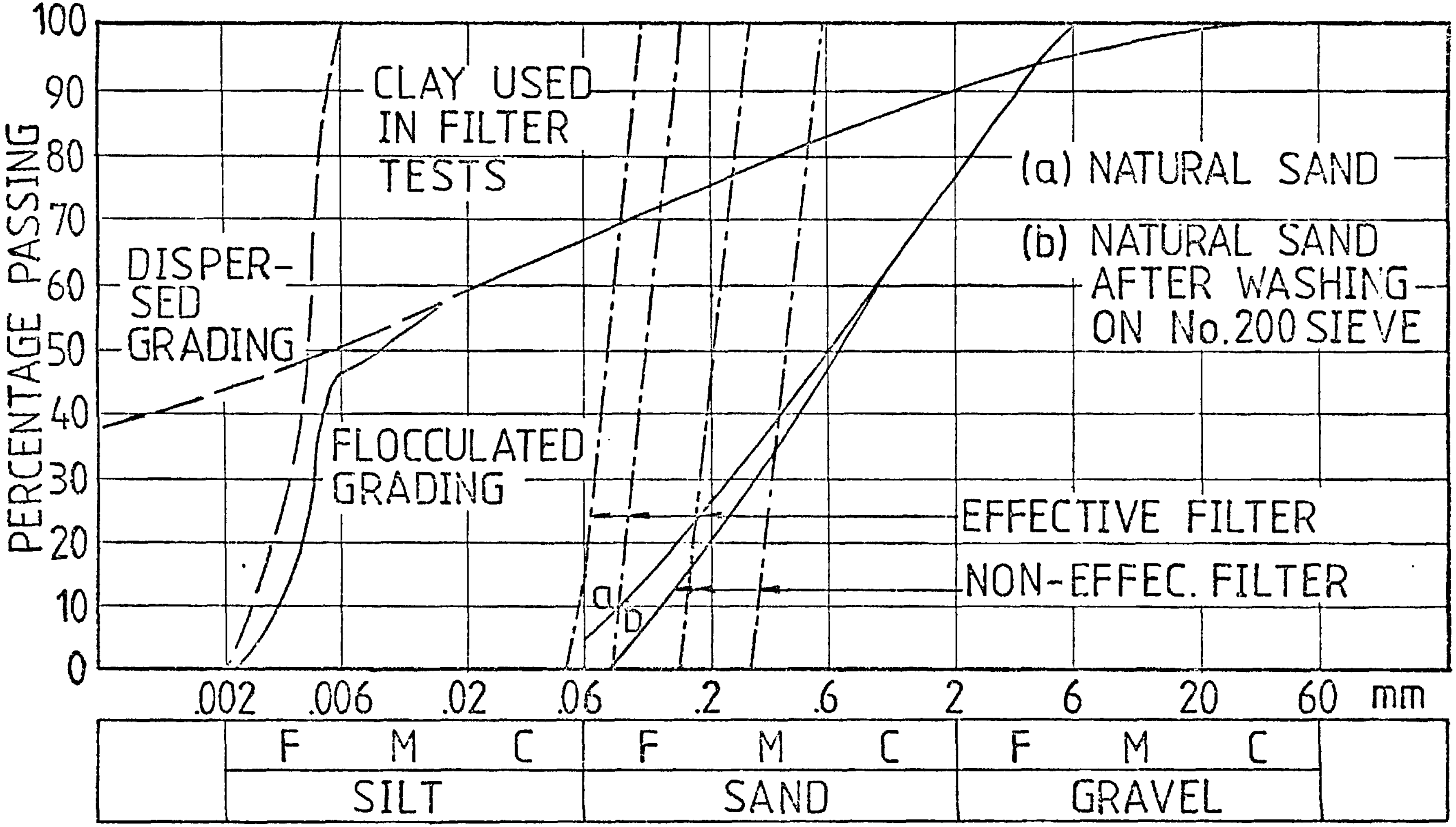


Fig. 3.16: Filter test results - Cow Green Dam (after Vaughan, 1976)

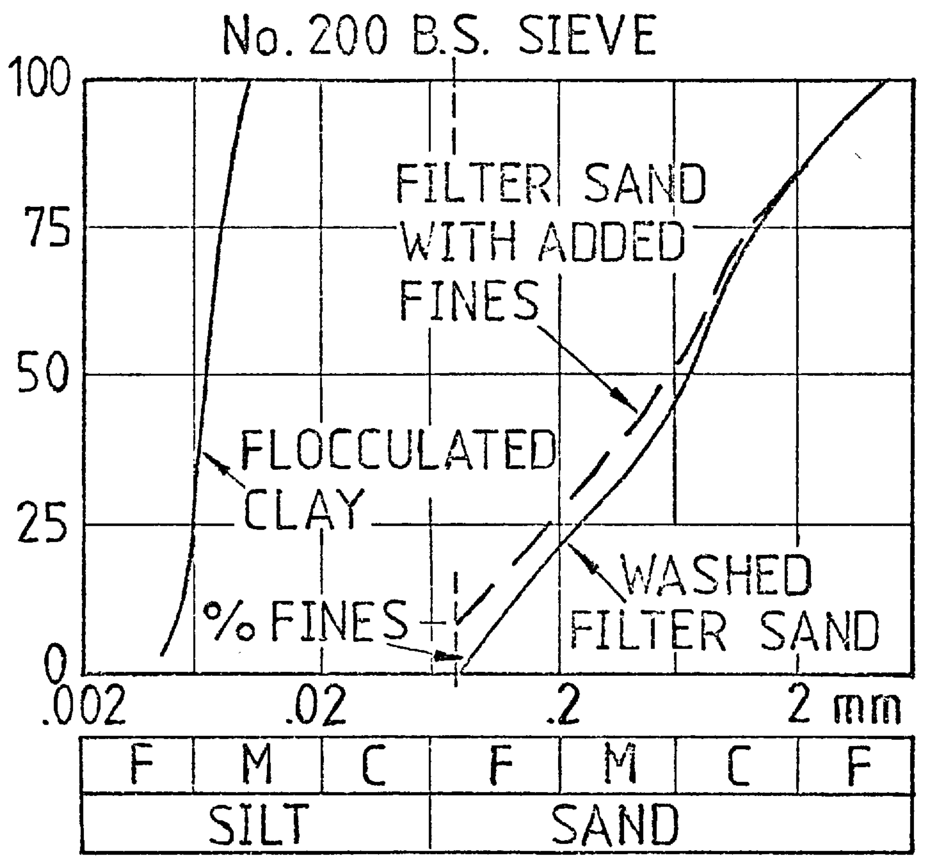


Fig. 3.17: Filter test results - Empingham Dam (after Vaughan, 1976)

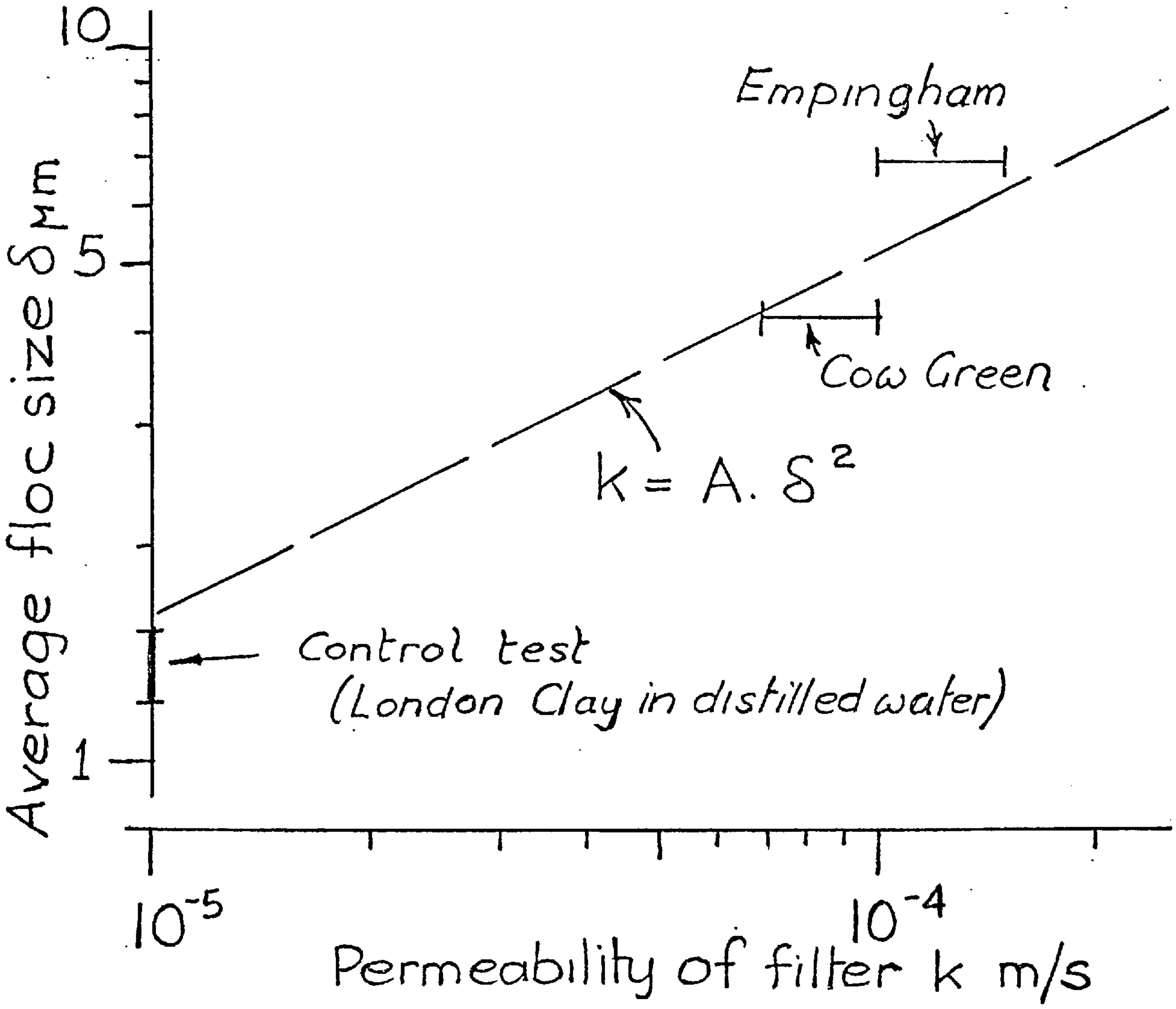


Fig. 3.18: Filter permeability against average floc size which will pass through filter (after Vaughan, 1976)

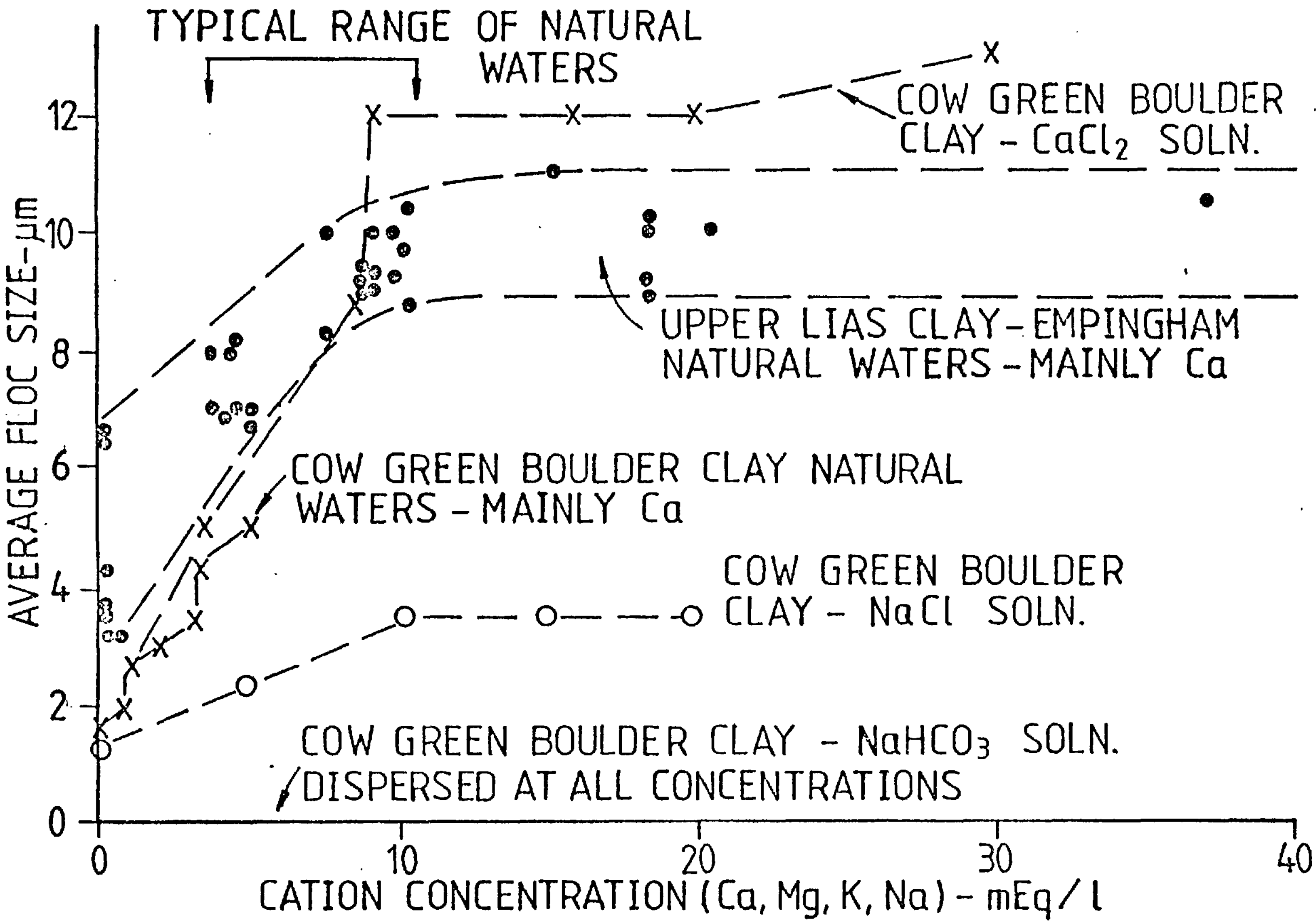


Fig. 3.19: Effect of salt concentration in water on average floc size (after Vaughan, 1976)

4. EXPERIMENTAL WORK - FILTER TESTS FOR NON-INTACT CONDITIONS

4.1 Introduction

As already commented in Chapter 3, the experimental work presented in this thesis, has the main purpose of understanding filter behaviour when special situations such as non-intact conditions arise, for example due to the development of cracks in the core of an embankment dam. The tests described in this chapter were carried out to try to verify the filter design approach suggested by Vaughan (1977) presented in Chapter 3. The test results will be considered in Chapter 6 during analysis of such an approach.

4.2 Filter Tests Considerations

For all the tests performed, three different material sizes were used as base soils. These materials, in the silt range size, were obtained from artificially prepared crushed silica, produced commercially by British Industrial Sand Ltd., with the size range varying in diameter from less than 1 μ to 60 μ. Aerodynamic and hydrocyclone classification processes were used to separate the materials and they were supplied by Dr P. Mantz of the Hydraulics section of the Civil Engineering Department of Imperial College. The grain size distribution curves of these materials are presented in fig. 4.1 as found by Mantz (1977) using pipette sedimentation analysis using glycerol-water mixtures. They differ from those found in this work because a different sedimentation test procedure was used. The gradings of the silts determined in this work are shown in fig. 4.2 and their determination will be discussed later on. The materials were called S1, S2 and S3 as indicated in fig. 4.2.

All the tests consisted of making slurries with the silts and passing them through sands, trying to simulate a filter preventing the fines which were being eroded by the water passing through an open crack in the base. A sketch of the equipment used in these experiments is shown in fig. 4.3.

All the slurries used in the tests had the same concentration of 30 gr dry soil/litre. The soil particle sizes have been found to vary with the amount of soil in solution, as will be commented in the next chapter. The slurries were prepared by mixing the silts with different types of water, according to each test, in a mechanical mixer for 5 minutes. After that, the solutions were put in a glass jar and remixed by turning the jar up and down when necessary.

The permeability of the filter materials was determined applying Darcy's law in falling head permeability test since most of the filters were at the fine and medium sand size. Initially the permeability of all sands available were determined using tap water for the sands of Series I tests and 'brine' water for the sands of Series II tests, to be used as a first guide to choose the slurry-filter combinations for each test. Before each filter test, the permeability of the sand was again determined, at this time using the same type of water as the slurry, since it varies with porosity of the material and also with the way of placement in the tube. Various tests were done with each sand so that the results if not precise, are at least consistent.

The filter test procedure consisted of:

1. The sand was chosen as a filter according to its permeability determined previously.
2. The soil was poured into the perspex apparatus tube above a 1.3 cm layer of gravel which was placed on a wire gauze to avoid the washing out of the sand itself. The sand was dry when uniform and wet when graded to allow a more homogeneous sample during placement.
3. The slurry was prepared by mixing each silt with the corresponding water for the test, with their gradings already determined as will be discussed in Chapter 5.
4. The same solution water was passed several times through the sand and after that a permeability test was performed.
5. The slurry was poured into the tube with water still about 5 cm above the filter surface to maintain the sand submerged. Care was taken to avoid turbulence at the filter surface but at the same time it was poured as quickly as possible to avoid sedimentation. It was observed that soil concentration in solution. only slightly changed at the region near the surface due to the mixing with the remaining water.
6. The solution was collected at the end of the tube on a dish and poured into the tube again in such a way that the gradient during testing was maintained more or less constant and equal to 8, the maximum obtained with the apparatus.

7. Visual observations were taken during the test and when possible the permeability of the filter was also observed.

The results of the tests were judged using the following criteria:

- Effective Filter - when the filter succeeded in the sense of retaining the slurry material.

If this condition occured, the water coming through the sand was completely or quite clean with the flow rate progressively decreasing and a 'skin' of the slurry material building up on and in the filter surface. At this time sedimentation began to occur due to the low permeability of the 'skin' of soil.

- Non-effective Filter - when the filter failed because it did not retain the slurry particles.

In this case, the water passing through the sand was completely 'dirty' all the time, the flow rate decreasing more or less depending on how much the fine particles had penetrated into the filter. For most of the cases, especially for uniform filters, the results were clearly in accordance with this criterion but some tests using graded filters showed difficulties in the interpretation of the results. Usually when the filter was effective the interface equilibrium was established in a few minutes. Nevertheless, some filters behaved as effectively only after the test had run for some time, due to the progressive clogging of the filter. To avoid a less subjective judgement of the results and a time dependence, when they were not immediately clear, they were considered as doubtful.

In general the test time varied from 10 min to 1.50 h depending on the results.

It must be emphasized that care was taken in always having a filter where the water velocity passing through it (for the gradient of more or less 8) was greater than the rate of settlement of the slurry particles, ensuring that there was no accumulation of fines on the filter unless it was arrested by filtration.

4.3 Filter Test Results

4.3.1 Uniform Filter Materials (Series I)

Sieve analyses were performed to determine the granulometric distribution curves of the sands available for use in this series of tests. They are presented in fig. 4.4. They were called A, B, U1, U2, U3, U4, U5 and U6. The sands presented uniform coefficient between 1.2 and 1.67 and they were considered as uniform according to the definitions presented in Chapter 2. Except for sand U6 all the materials had round particle shape.

The first tests were performed using 'town supply artificially softened water' in the sand permeability determinations and in the slurries of the three silts S1, S2 and S3. Mantz (1977) found that for this soil-water system, the surface repelling or attracting forces minimally affected the grades of the soils. The solids behaved as non-surface interacting. This point will be presented and discussed in the next chapter. Later, the sand permeabilities and filter test results were not considered since electrical conductivity and pH tests of this water showed that the supplier of the 'softened water' was no more effective and a large amount of salt (NaCl) was coming in solution and even in suspension (more or less 95 gr of salt/litre of water). As the particle sizes could not be reliable determined, it was decided to first investigate the chemistry water effects in the particle sizes, corresponding to the next chapter of this thesis. After that, the Series I of tests was repeated using the following three types of water:

- distilled and de-ionized water; pH = 7; σ = 2.0 μmhos/cm.
- town supply (hard) water; pH = 8, σ = 670-710 μmhos/cm.
- brine water, consisting of 35 g/litre of a commercial brand of NaCl such that the specific gravity was similar to that of sea water (1.025 g/cm^3); pH = 7.65; σ = 42.000 μmhos/cm.

Actually only silt S1 was used together with the three waters presented above for the filter tests of this series. For the other silts S2 and S3 it was concluded that the particle sizes were not significantly affected by the type of water used, as will be discussed in Chapter 5.

The slurries used in all tests were classified as:

S1D, S1H, S1B → slurries made by mixing silt S1 with distilled, 'hard' and 'brine' water respectively.

S2D, S2H, S2B → slurries made by mixing silt S2 in the same way as silt S1.

S3D, S3H, S3B → slurries made by mixing silt S3 in the same way as silts S1 and S2.

The test results of this series will be presented as:

TEST IS1 → filter tests of Series I using silt S1 as slurry material.

TEST IS1D → tests using slurry S1D

TEST IS1H → tests using slurry S1H

TEST IS1B → tests using slurry S1B

TEST IS2 → filter tests of Series I using silt S2 as slurry material

TEST IS2B → tests using slurry S2B

TEST IS3 → filter tests of Series I using silt S3 as slurry material

TEST IS3B → tests using slurry S3B

TEST IS1 RESULTS

The results of tests IS1D, IS1H, IS1B according to the soil-water systems listed above, are presented in figs. 4.5, 4.6 and 4.7 respectively. The permeability of each sand determined immediately before each test is also presented in these figures.

The clearer results were obtained in tests with 'hard' and 'brine' water. When distilled water was used in the slurries, the test results could not be easily interpreted due to the well graded distribution of the soil particles. The effective particle size retained by the sands became more difficult to predict and also the test results were not so clear, with the gradual clogging of the sands by the finer particles, changing the filter behaviour with time.

With 'hard' and 'brine' water the slurry material behaved more uniformly (CU = 1.8 and 1.15 respectively) due to the flocculation of the soil particles. The filter tests results could be better analysed and clogging of the sands with fines only occurred on a small scale.

Tests IS1D and IS1H had the same sands as effective and non-effective in retaining the slurry material, suggesting that the self-filtering is occurring at the interface which is controlled by the coarser particles. Also some clogging of the sands in test IS1D could have contributed to this result.

Sand U3 did not retain the slurry material of test IS1B although it had worked effectively in tests IS1D and IS1H. It seems that due to the high degree of flocculation of the soil particles the material became quite uniform without clogging of the sands by the fines. It has already been postulated (U.S.C.E. 1953) that when the base soil is of a relatively uniform grain size, the filter must be finer. Test results of filter tests using sieves as filters, which will be presented in Chapter 7, showed that the amount of base soil lost through the sieves increased with the CU of the base material. Probably some clogging of the filter always occurs on a small or large scale before filtering process takes place. The more graded the material is, the more easily the fines go into the sand, clogging it, reducing the void ratio, decreasing the permeability of the soil. Therefore the material starts to behave differently from the original one, finer at least at the filter surface.

TEST IS2 RESULTS

In this series of tests only 'brine' water was used together with silt S2. It was intended to obtain the slurry material as uniform as possible to avoid too much clogging of the sands. Also, as already commented and as will be shown in the next chapter, the soil did not behave significantly differently when the three different types of water were used. The test results are presented in fig. 4.8.

TEST IS3 RESULTS

As for tests IS2 only 'brine'water was used in performing these tests. As shown in fig. 4.2, for this size of silt the water chemistry does not affect the grade of the soil any more. The 'brine' water was used to maintain the same water filter interaction of the other tests. The results are presented in fig. 4.9.

Some difficulties were found in doing these tests, such as:

- in the determination of the permeability of coarse sands using falling head test.
- the preparation of the slurry with the coarse silt.
- difficulties in maintaining the slurry uniform before pouring it into the tube since the coarse particles sedimented very quickly.
- the performance of the filter test, with difficulties in keeping the gradient constant and making observations of the filter behaviour.

Tests with sand U4, showed a very clear result, according to the criterion presented before. On the other hand, with the coarser sands the results did not show so clearly and it was necessary to repeat the tests a few times to be sure of the filter behaviour. Besides the operational difficulties commented before in performing tests with the coarse sands, in some tests the water became clean only after ½h of testing and sometimes the water did not become completely clean although the concentration of the particles had visibly decreased significantly. It seems that a gradual clogging of the filter was occurring during the tests. In some tests the silt material clogged the sand sufficiently at least at the interface allowing the beginning of a self-filtering of the soil particles. In others, although clogging had also occurred, the number of the slurry coarse particles which could infiltrate the sand and block its voids was not enough to clog the filter sufficiently or the whole sample or in a concentrated region at the interface. Because of this, the finer particles could still pass through the sand during the test. It could be speculated that if a constantly new slurry was used instead of recycling it, the filter would be completely clogged and the water completely cleaned. Also sufficient time should be allowed for clogging to take place. In any case, the test results presented in fig. 4.9 were judged with the same criterion used for the other tests.

During the performance of the tests of this series, it was observed that the permeability did not change with the different type of water used, tap, distilled, 'hard' and 'brine' waters, or at least the difference was not significant to be measured with the process used.

4.3.2 Graded Filter Materials (Series II)

Another series of tests was performed in the same way as Series I tests in order to observe the behaviour of graded filters. It was decided to produce the graded sands by mixing different single size sands to control their permeability better. For this purpose some sands were then acquired and they are shown in fig. 4.10. All of them had particles with round shape. They were called A, B, C, D, E according to fig. 4.10.

The graded sands obtained in this way to be used as filters and their permeability characteristics are shown in fig. 4.11 and table 4.1. They were called G1, G2, G3, G4, G5, G6, G7 and G8.

With the materials available, the most graded sand that could be prepared, presented a uniform coefficient of 6 according to the definition of CU presented in Chapter 2.

Having in mind to obtain sands with similar permeabilities to the filters used in the Series I tests, the percentage of the finest particles was varied as a controlling factor. Sand A was used to control the permeability characteristics of sands G1 to G6 while sand B controlled the permeability characteristics of sands G7 and G8.

Various tentative methods were first tried in order to choose the best way of determining the permeabilities of these sands. After that it was decided to prepare a small amount of soils in the way presented in table 4.1 (more or less the same amount of 200 gr was necessary in each test), to obtain a better control of the amount of the finest particles.

When the sands were placed dry into the tube in the same way as done with the uniform filters, the permeability was found to vary significantly for different tests with the same sand. Segregation of the particles could not be avoided and the filter prepared was visibly non homogenous. The best way found to avoid this problem was to place the sand wet with a spoon in small quantities into the tube. Then the results started to become consistent.

At this time only slurries with silts S1 and S2 mixed with 'brine' water were used in the filter tests. The difficulties found in the tests of Series I when testing coarse sands led to the decision of excluding silt S3.

The test results will be presented as

TEST IIS1B → filter tests of series II using slurry S1B

TEST IIS2B → filter tests of series II using slurry S2B

TEST IIS1B RESULTS

The results of these tests are shown in fig. 4.12 as well as the permeability values of the sands. The following behaviour was observed in each test:

- The results of filters G2 and G6 did not allow a clear interpretation. During the first minutes of testing (more or less 5 to 10 minutes) the water coming through the sand was completely 'dirty'. For sand G2, after this initial time, permeability decreased reasonably and sedimentation of the slurry started to occur while the water became 'clean'. For sand G6 the same type of behaviour was observed although at this time the water had not become completely clean after 1 h of test and a considerably smaller concentration of the silt particles was observed.

- Even for the coarser sand, G8, although the water was still 'dirty' after 1 h of test, some fines were observed at filter surface and clogging of the sand as well. Also the permeability of this sand kept decreasing during the test (from 3.5×10^{-4} m/sec with 5 min of test down to 1.4×10^{-4} m/sec with 56 min of test).

- With the finest sand G1, after 4 minutes of test sedimentation of the slurry particles began to occur and after 6 minutes the water passing through the filter had completely cleaned.

Due to the difficulties of analysing the results of the tests, sand G1 was considered as effective filter in retaining slurry S1B and sand G8 as non-effective filter while sands G2 and G6 were considered as failed filters due to clogging.

TEST IIS2B RESULTS

The results of these tests and the permeability of the sands are presented in fig. 4.13. The following behaviour was observed for the filters:

- Sand G7 showed a clear result, retaining the slurry S2B particles instantaneously but some clogging of the sand was observed after the test had finished.

- Sand G8 although considered non effective to work as a filter, ended up with some concentration of fines at filter surface after 35 minutes of test.

It seems from the test results IIS1B and IIS2B described above, that for graded filters clogging always occurs on a scale greater than for uniform sands, due to their graded voids distribution. Even for an apparently homogeneous graded filter, the voids distribution seems to vary along the filter thickness and in some tests it was observed that slurry particles were retained at only a portion of the filter surface. Due to this some fines kept passing through the sand in a lower soil concentration solution.

For all the filter tests performed in this work, the amount of 1.5 l to 2 l of slurry was used. The slurry was poured into the tube while it was collected after passing through the filter and poured again into the tube. As mentioned in test Series I results, probably if the concentration of slurry was maintained constant the successive clogging of the sand could clean the water completely. Actually we have been concerned with avoiding as much as possible clogging of the filter as loss of material since this could raise difficulties in interpretation and extrapolation for field conditions. If clogging of the filter was allowed, its effectiveness would also depend on its thickness and if and how much material could be lost. The filter design would have to consider a contaminated layer and this would be more a problem of migration of particles into sands.

It seems that any analytical method of trying to define filtration in graded sands will always have to make assumptions that may not correspond to reality. The statistical approach developed by Silveira (1965) for example, was found by De Mello (1977) to be unsatisfactory in practice mainly because of the hypothesis on pore size distribution, as has already been discussed in Chapter 3.

Due to the test procedure used, it can be speculated that if clogging of the filter occurs caused by the fines, it could be enough to decrease the sand permeability and sedimentation of the slurry particles started to occur without filtration having actually taken place. As it is not possible to be sure of this, it is one more reason to exclude clogging in the effective filter results.

The results of both series of tests I and II will also be presented in table 6.2 of Chapter 6. There the permeability of the filters and the assumed effective slurry particle size being retained will be presented.

From all filter test results, it was found that the uniform filters presented clearer results, with regard to the criterion of successful or failed test adopted, while the graded filters presented a range of sands between the effective and non-effective ones which are controlled by clogging. Also it was found that segregation of graded sand particles increases with the CU values and the non-homogeneity of the sample could lead the same sand to behave differently in different tests although the same procedure was used.

Table 4.1 Graded sand characteristics

SOIL	Particle Sizes Range		Graded Sands							
			G1	G2	G3	G4	G5	G6	G7	G8
A	Soil Composition (% of weight)	7/14	20	20	25	25	25	25	32.5	32.5
B		14/25	20	25	25	25	25	25	35	35
C		25/52	20	20	20	25	25	25	22.5	32.5
D		52/100	20	20	20	17.5	20	25	10	0
E		100/200	20	15	10	7.5	5	0	0	0
	k(m/s)		1.3×10^{-4}	1.8×10^{-4}	2.2×10^{-4}	2.9×10^{-4}	3.5×10^{-4}	4.2×10^{-4}	7.3×10^{-4}	1.6×10^{-3}
	D_{15} (μm)		127	150	182	215	230	230	360	410
	CU		6	4.5	4.2	3.6	3.5	3.5	2.8	2.6

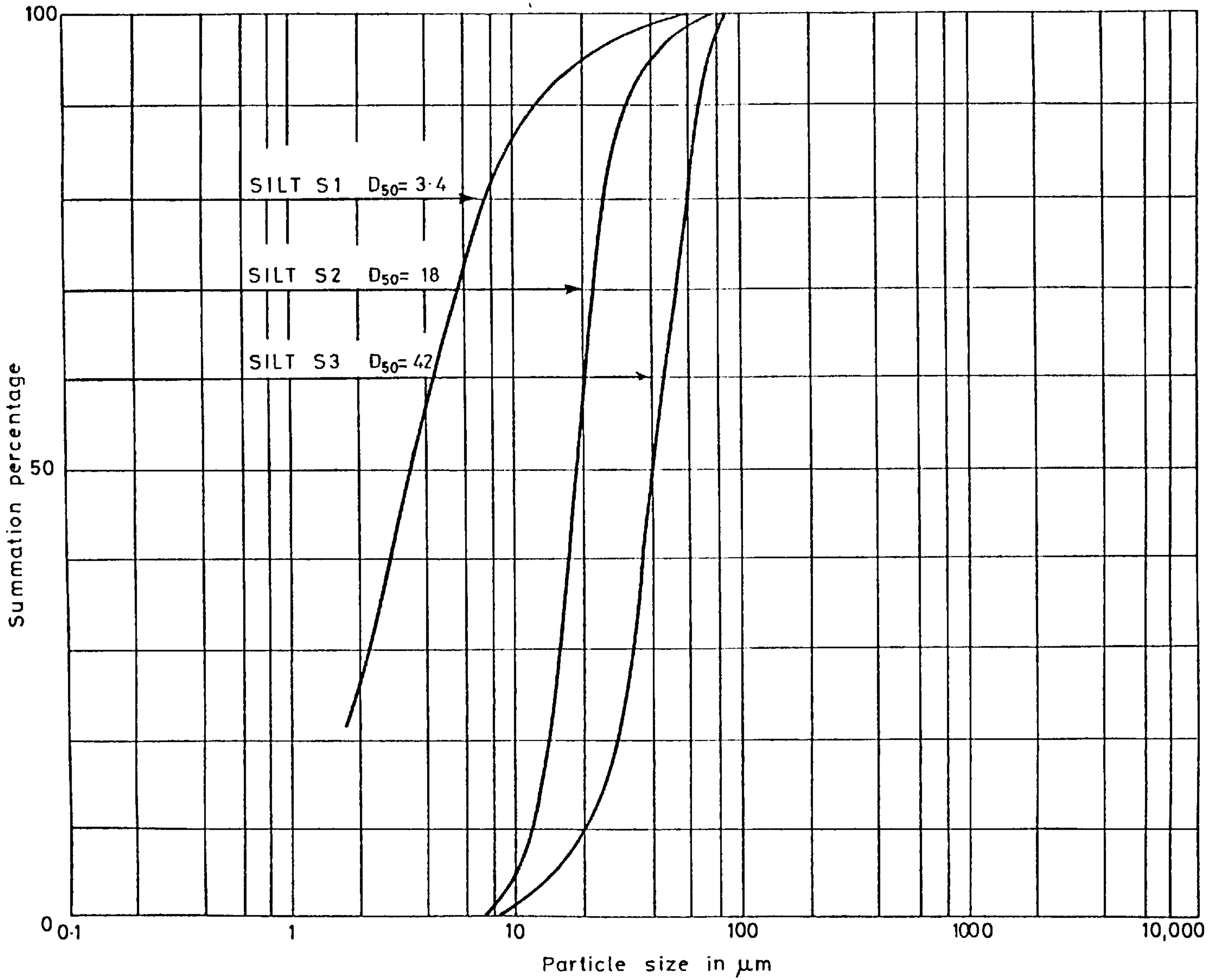


Fig. 4.1: Grain size distribution of the slurry materials (after Mantz, 1977)

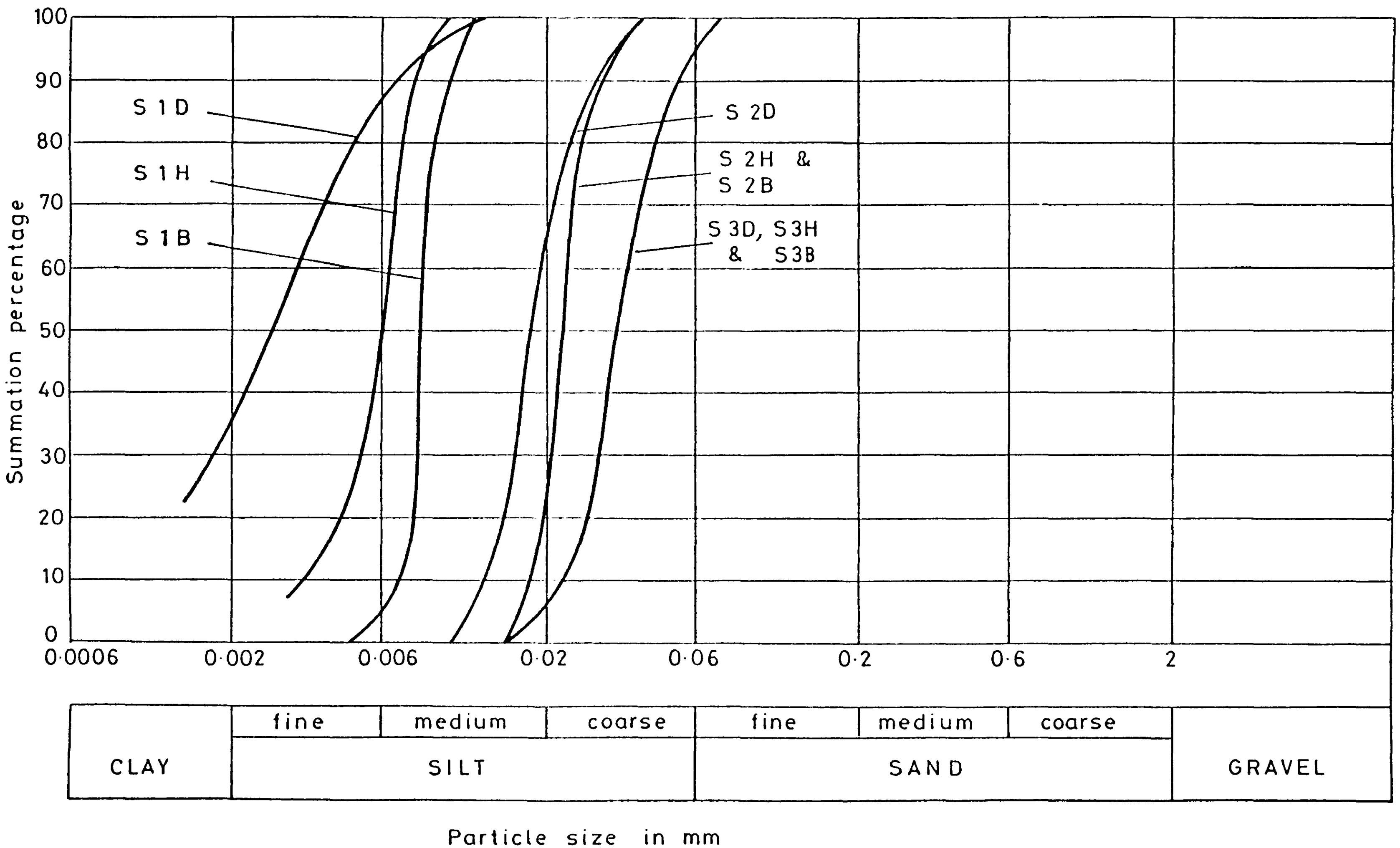


Fig. 4.2: Grain size distribution of the slurry materials

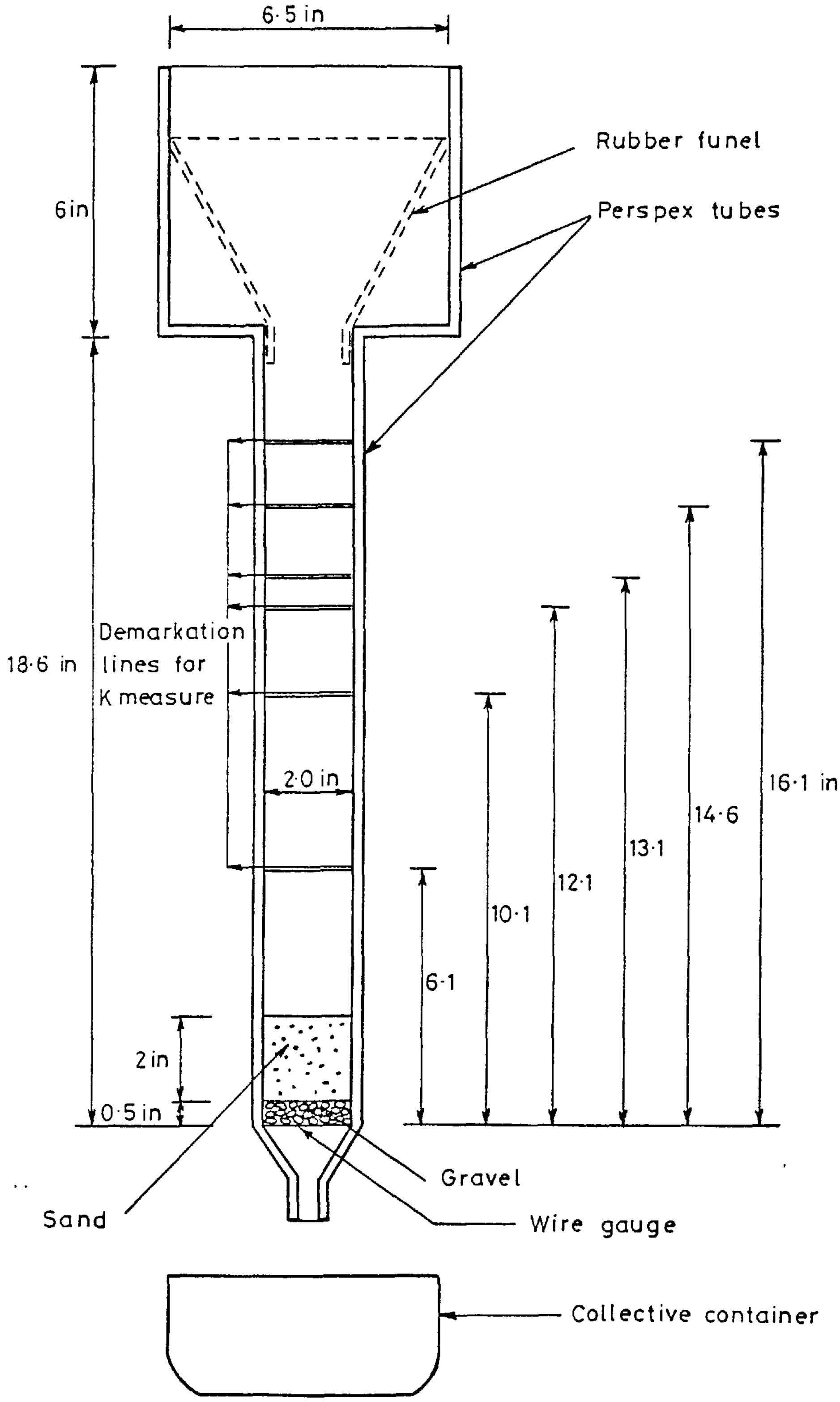


Fig. 4.3: Equipment lay-out

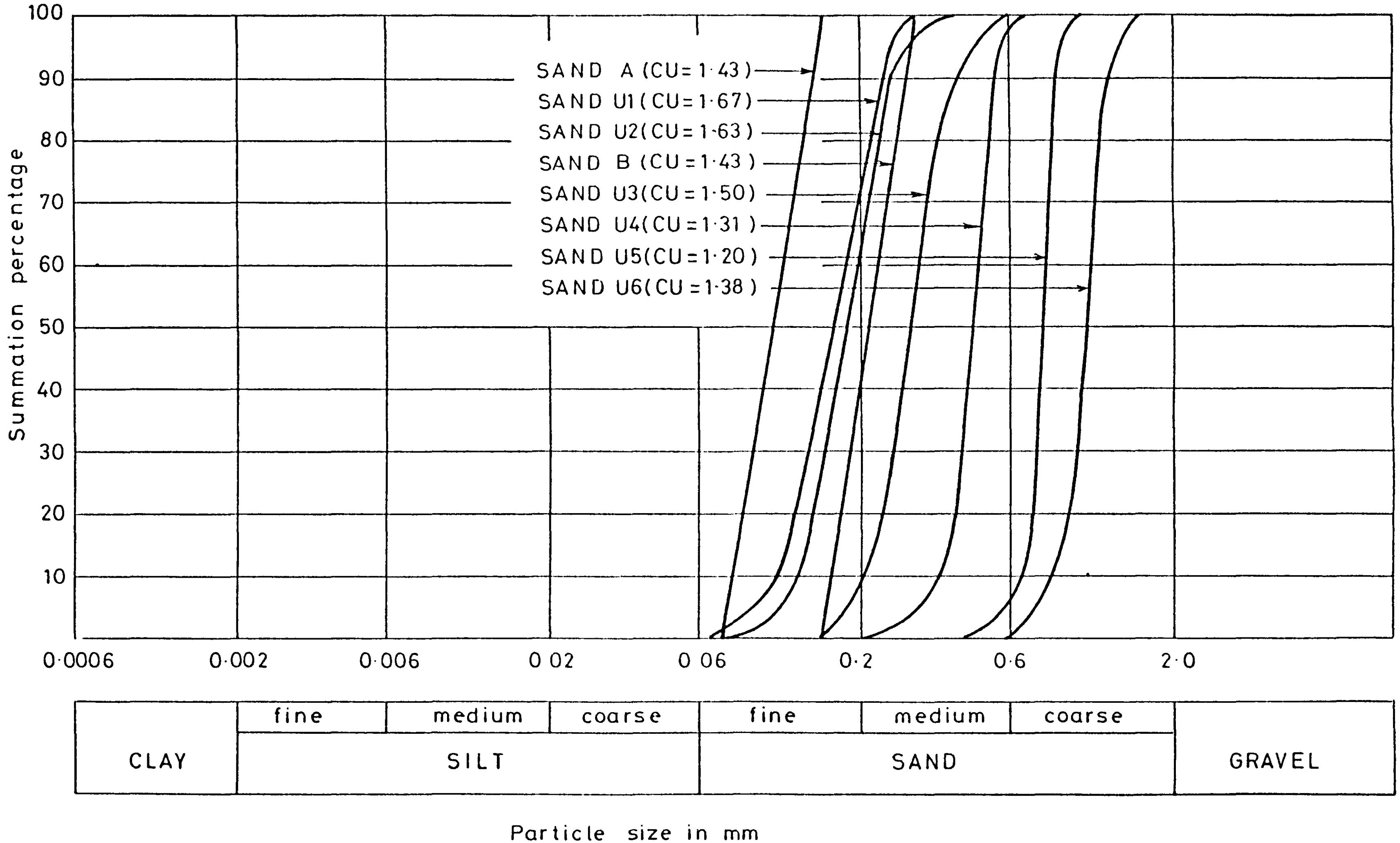


Fig. 4.4: Grain size distribution of uniform filters

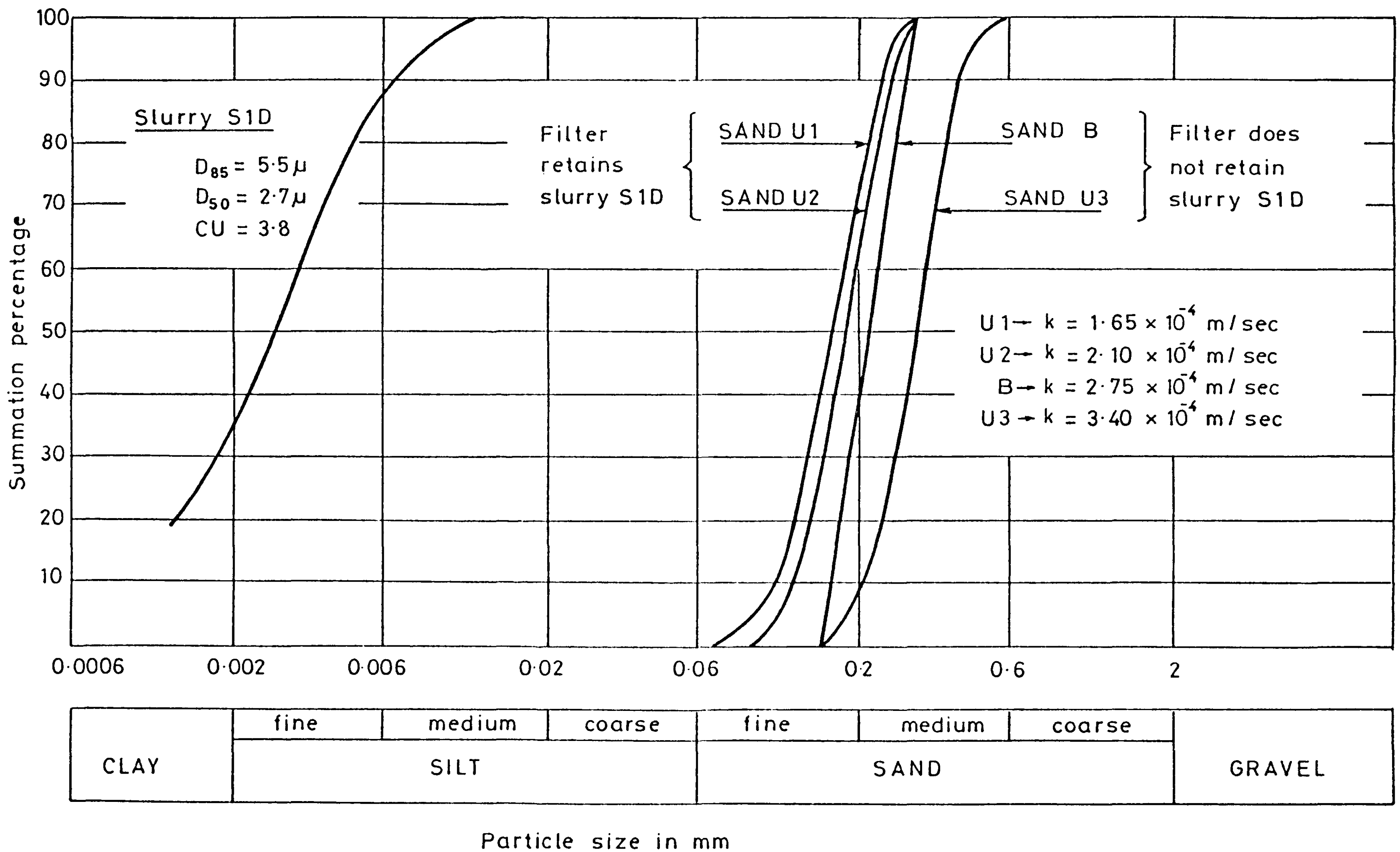


Fig. 4.5: Filter test results - Tests IS1D

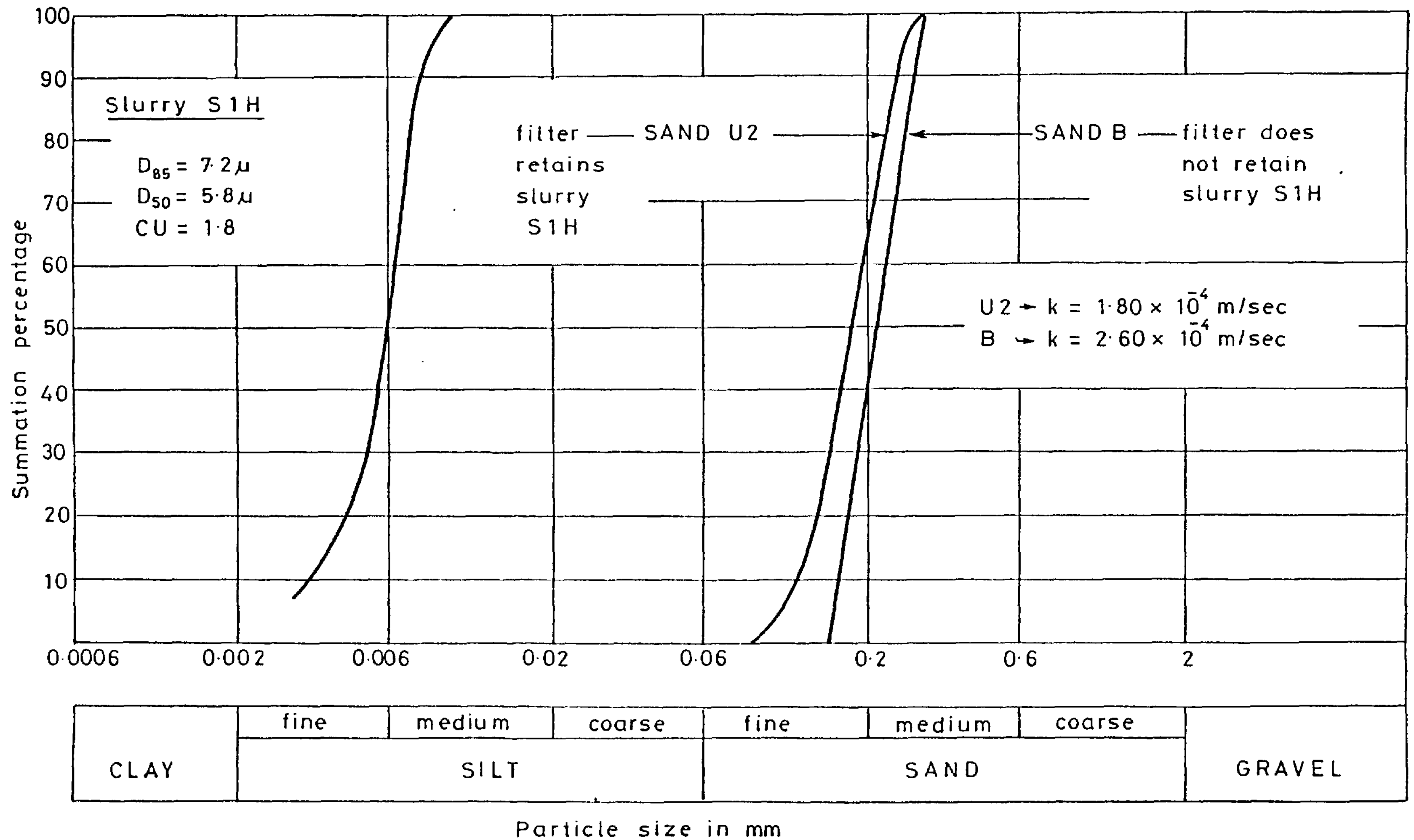


Fig. 4.6: Filter test results - Tests IS1H

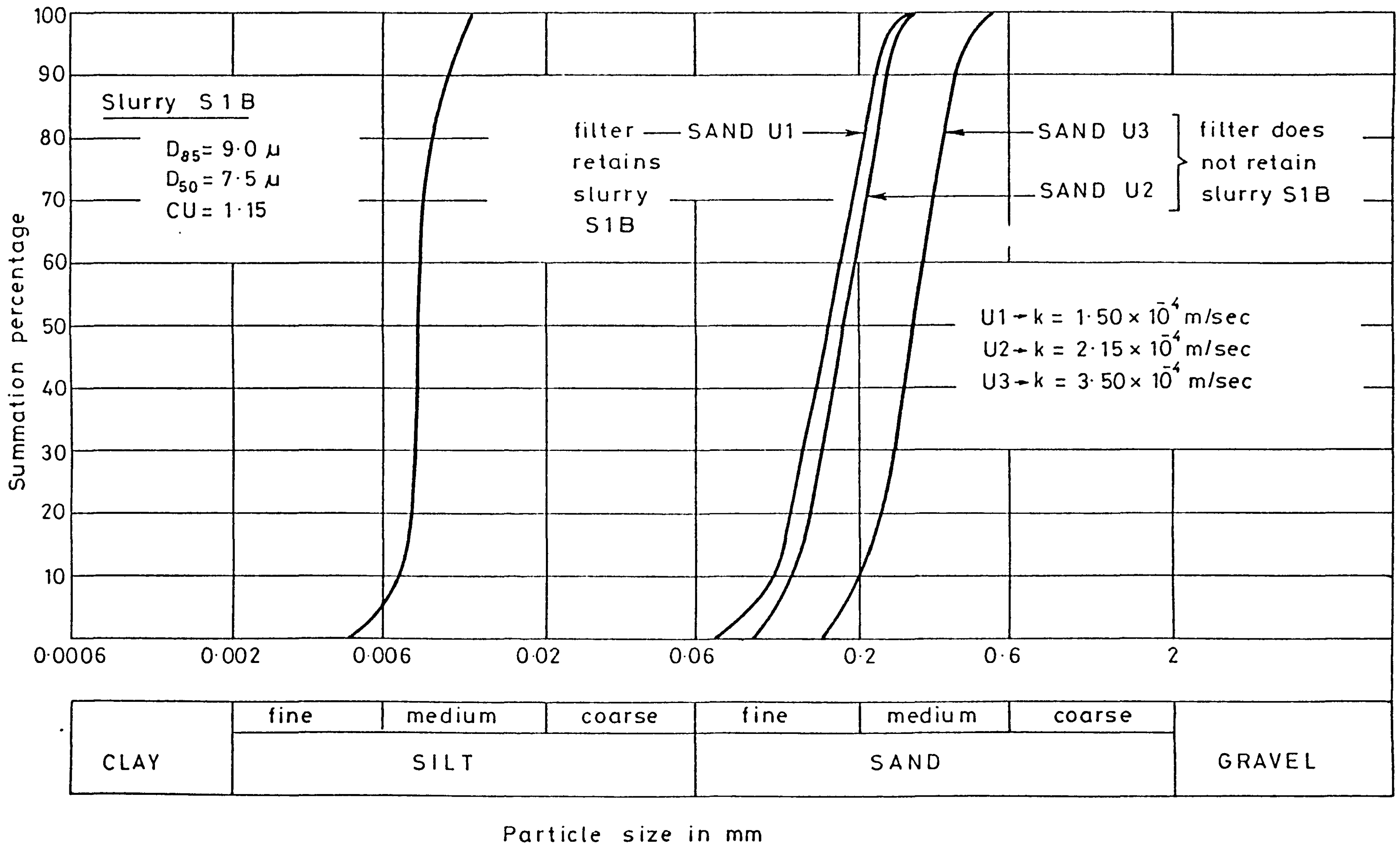


Fig. 4.7: Filter test results - Tests IS1B

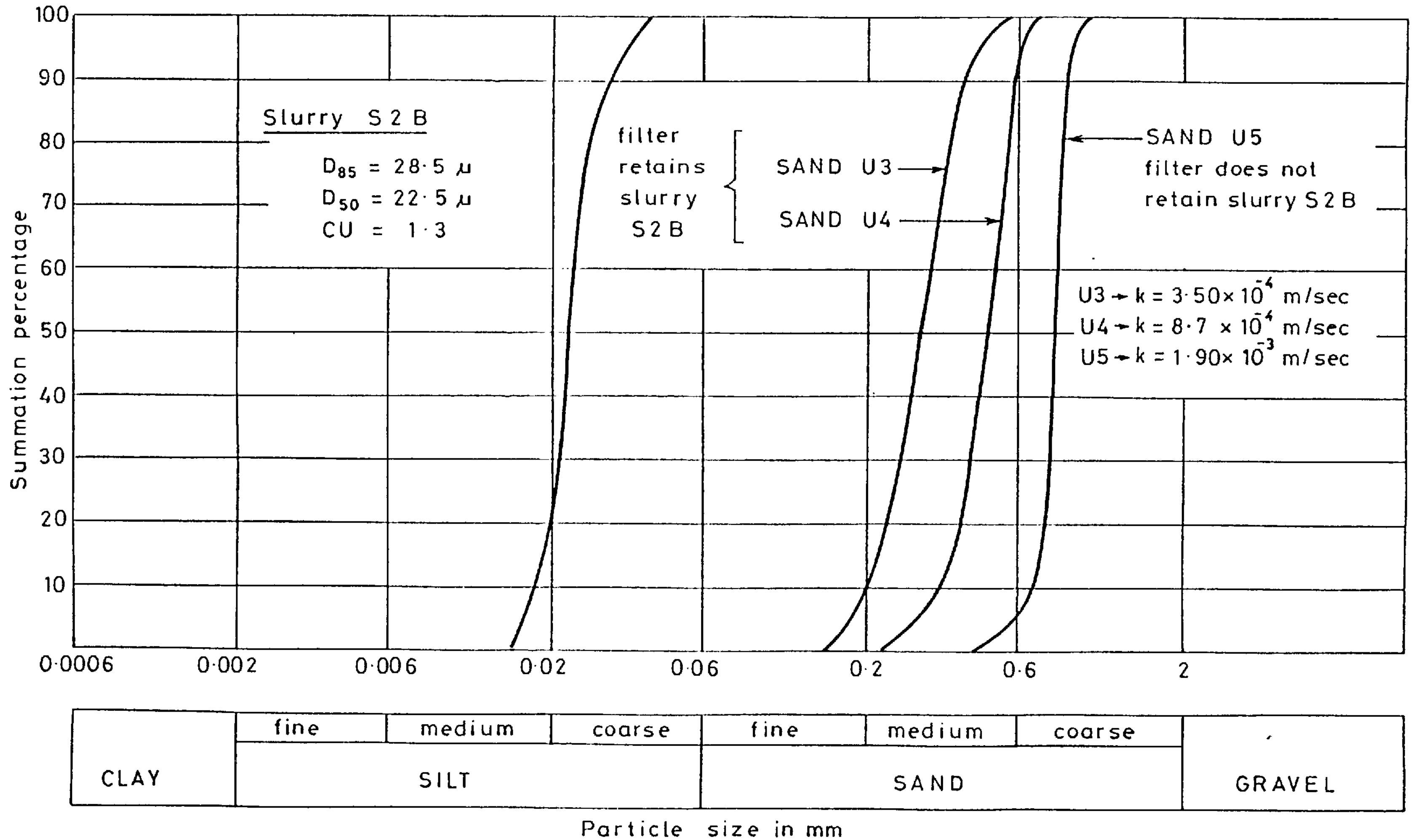


Fig. 4.8: Filter test results - Tests IS2B

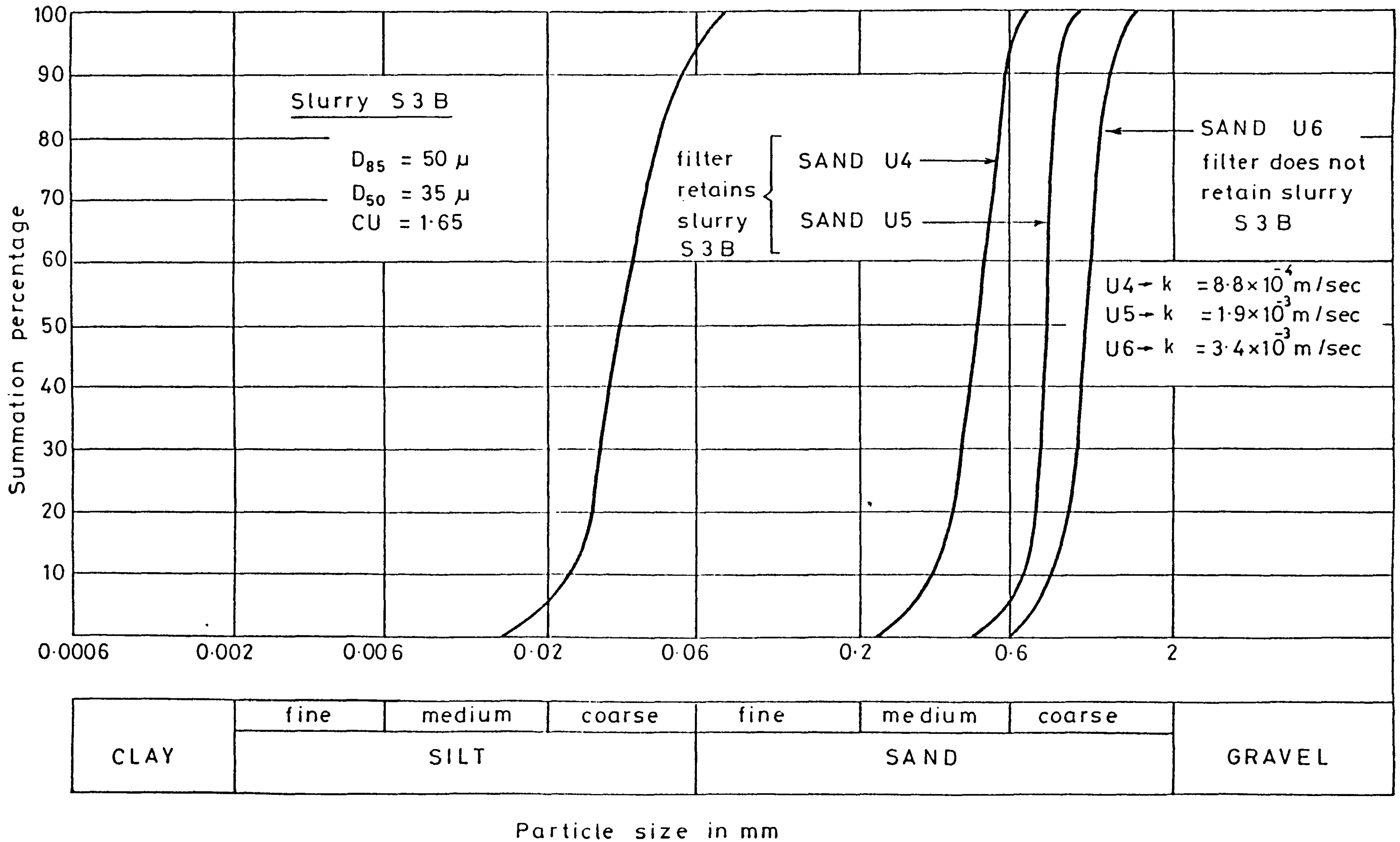


Fig. 4.9: Filter test results - Tests IS3B

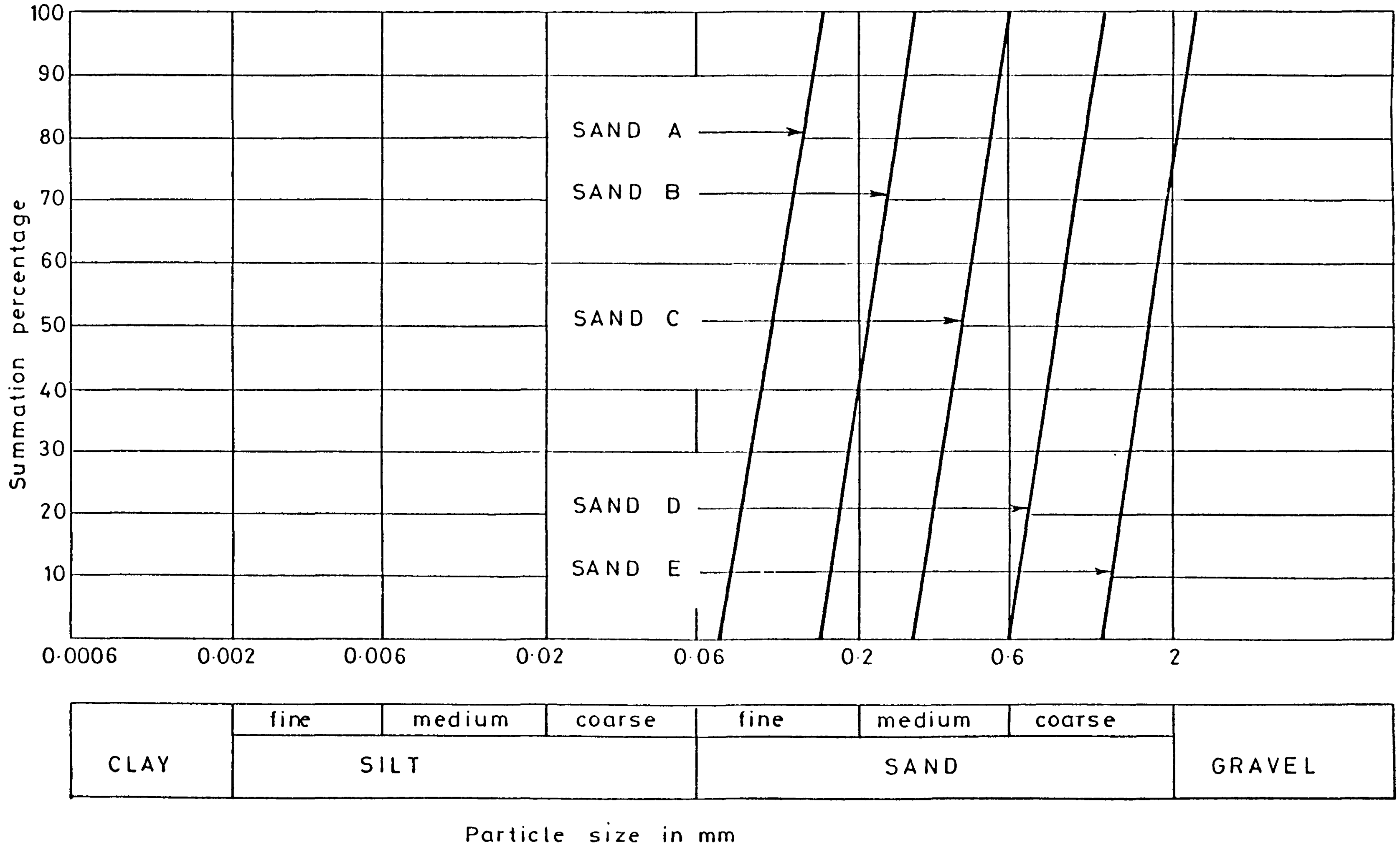


Fig. 4.10: Uniform sands used to make the graded sands

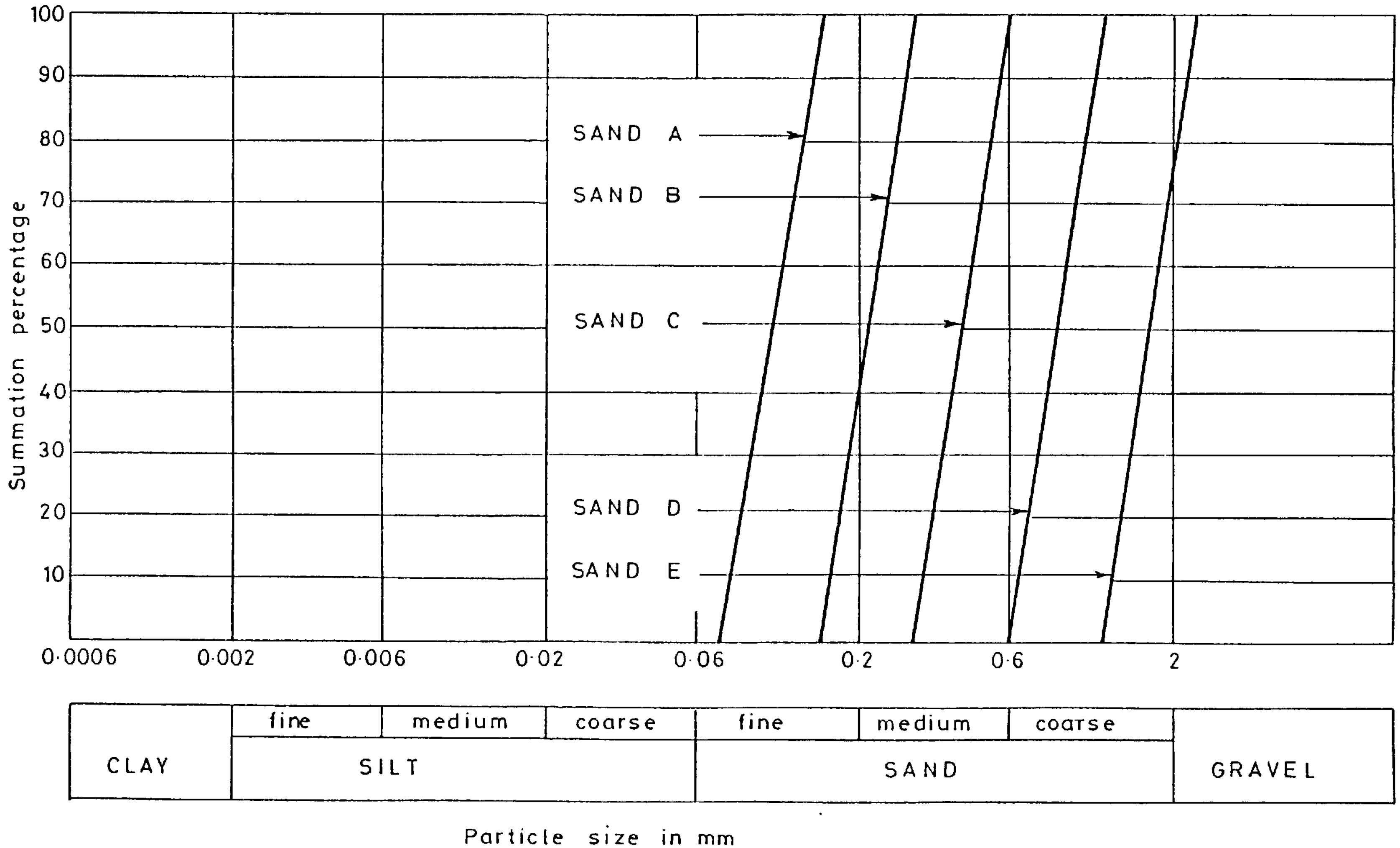


Fig. 4.10: Uniform sands used to make the graded sands

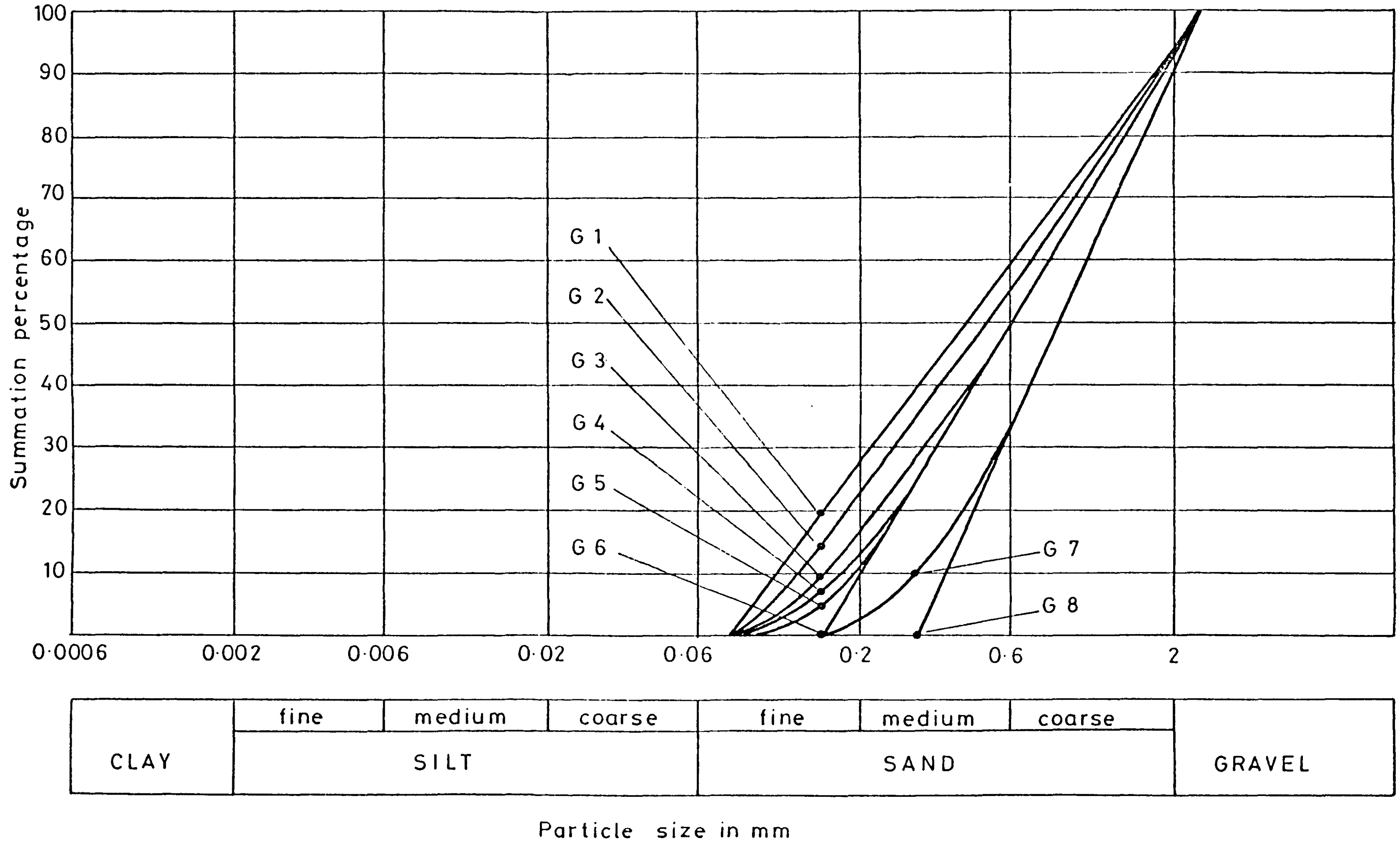


Fig. 11: Grain size distribution of graded sands

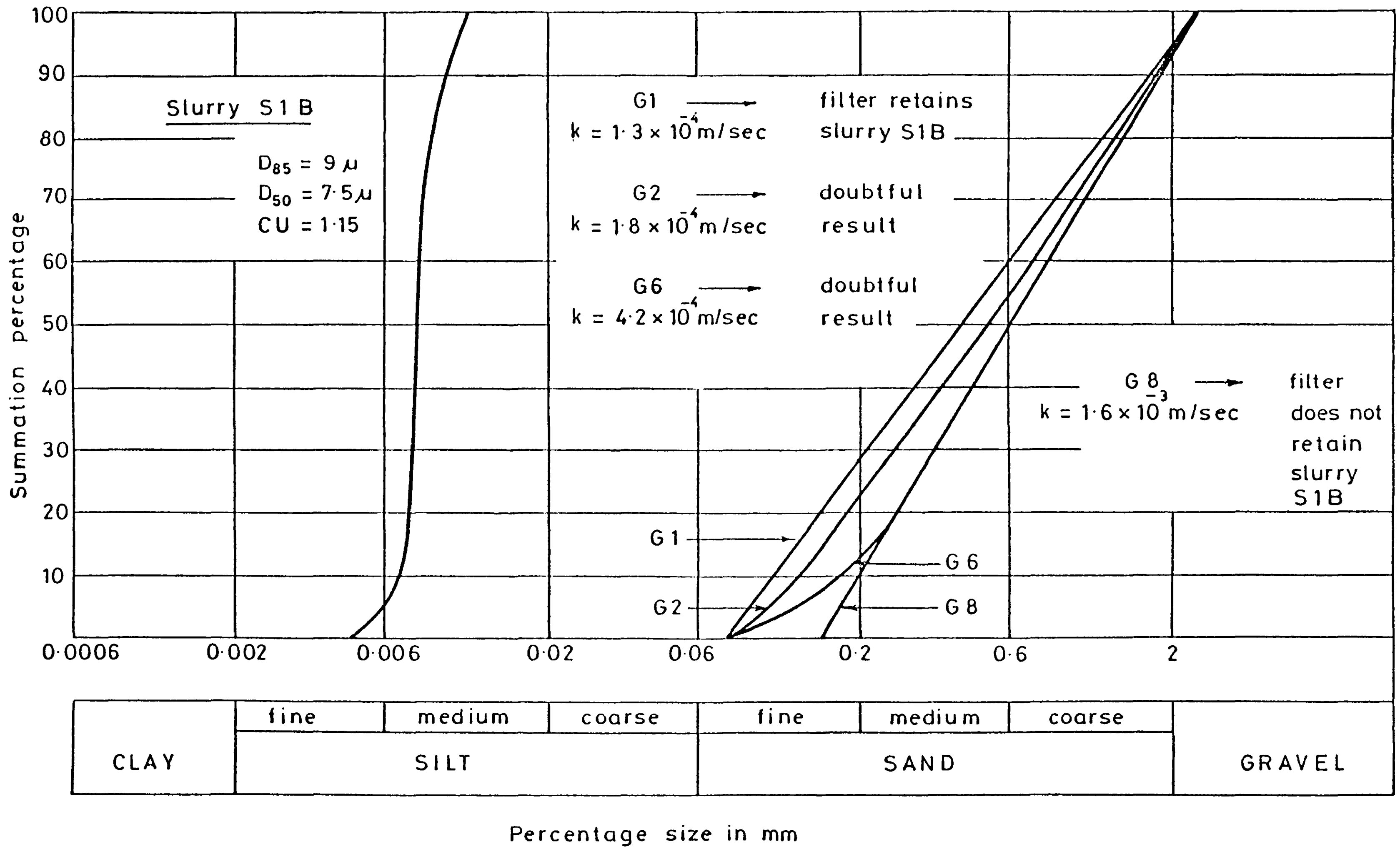


Fig. 4.12: Filter test results - Tests IIS1B

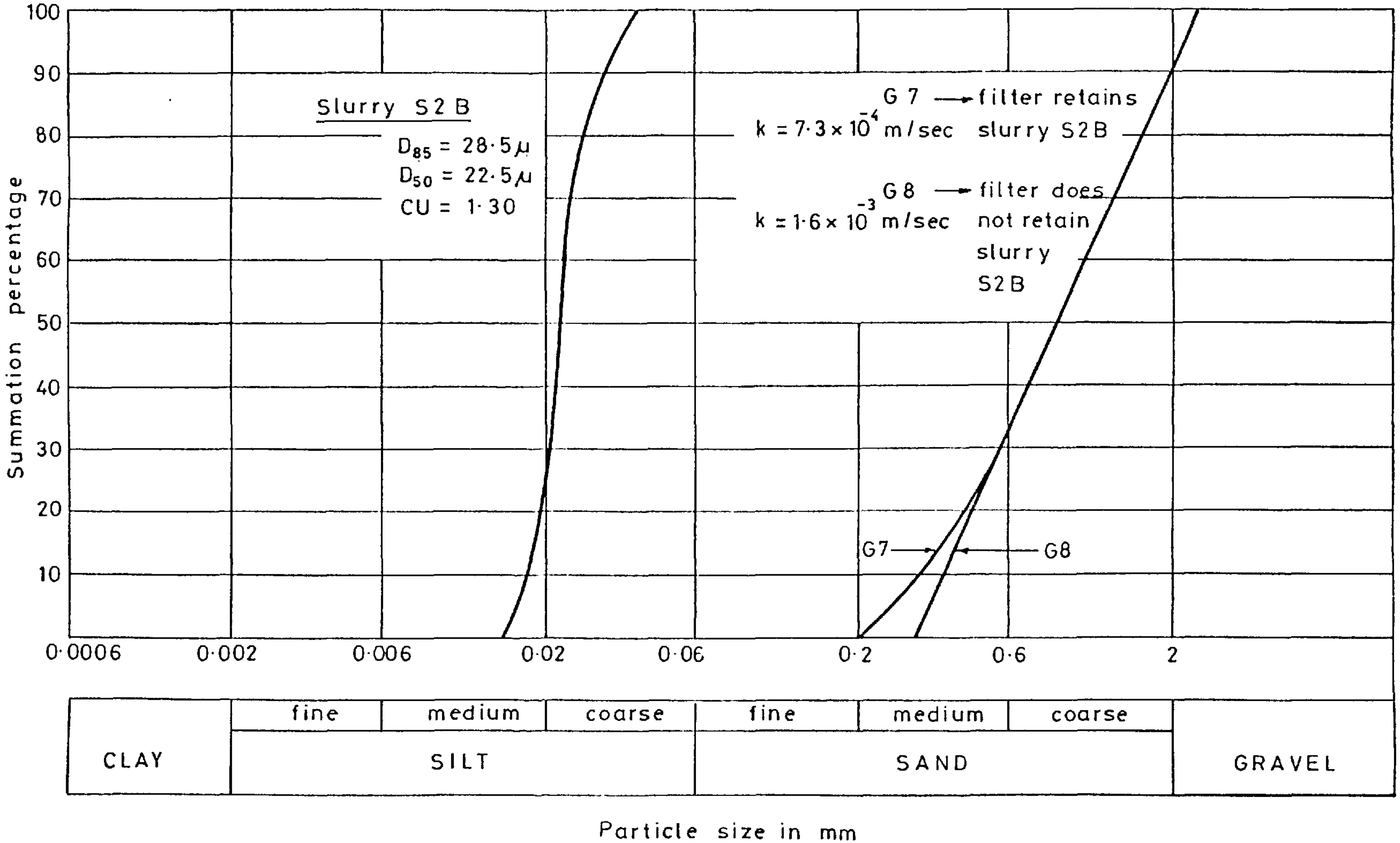


Fig. 4.13: Filter test results - Tests IIS2B

5. SEDIMENTATION ANALYSIS OF THE SLURRIES

5.1 Test Considerations

Since the present work has been concerned with filter tests in which soil particles in suspension must be retained by another soil when passing through it, the size of the slurry particles must be known if any kind of criterion is suggested. As the materials used in the solutions were at the silt range size, as already mentioned in last chapter a sedimentation analysis was necessary for the determination of the granulometric distribution of the particles.

Two sedimentation test methods were first tried, the hydrometer and pipette techniques to find the better way of determining particle size. Some tests were performed as B.S. fine grading test specifications, but without pretreatment or dispersant, using the two methods for similar solutions and conditions. As the results of both analysis showed similar particle size distribution of the soil, as shown in the example of fig. 5.1, it was decided to further use only the pipette method. It was found to be more reproductible in these preliminary tests and also from reported results (Allen, 1975). Furthermore it seems more reliable because of reduced disturbance of the suspension during sampling if compared with the hydrometer test and it is also more flexible for use with different soil concentrations and high density solutions.

The objective of the sedimentation analysis of the slurries to be used in the filter tests presented in the last chapter was not to determine the individual particle sizes of the soil but the particle sizes that could result from a specific soil-water system. For this, the Stokes's Law of sedimentation was used, the particles now having individual particle sizes or a corresponding 'floc' size. Whether the soil particles will behave as individual particles (when dispersion provides the smaller soil particles) or as 'floc' (when flocculation causes particles to unite and to remain together), will depend on the soil itself and on the environment present during the test. In other words, it will depend on the range of particle sizes, soil mineralogy, type of water involved and also concentration of the soil particles in suspension.

The first idea in performing this analysis was to verify if the desired relationship between particle sizes to be retained and filter permeability varied with the chemistry of water used since it could affect both permeability of the filter and soil particles of the slurries. As said before, permeability was found not to change significantly when distilled, 'hard' or 'brine' waters were tried. The present study became then concerned only with the effects of the soil-water system interactions in the particle sizes.

Two effects were then investigated in determining particles sizes of the silts used in the filter tests: the chemistry of water used in the solutions and the concentration of the particles in suspension.

5.2 Chemistry Water Effect in the Particle Sizes

5.2.1 General Aspects

Interactions between soil particles, adsorbed cations and water occurs due to the unbalanced force fields which exist at the interfaces between these constituents. The cations are held to the particle surfaces to neutralize their electro-negative charge. Those which are in excess go into solution in the presence of water and a tendency to equalize concentrations through the solution leads the adsorbed cation to try to diffuse away from the particle surface. The negative surface and the resultant distributed charge in the adjacent phase form the so-called diffuse double layer.

When soil particles in the presence of some specific aqueous electrolyte are approximated to a certain distance, their force fields can overlap and influence the behaviour of the system. Actually this will occur only if these forces are large relative to the weight of the particles. Depending on whether the system is affected or not, the soil particles will be classified in this work respectively as:

SIS → surface interacting solids

NONSIS → non-surface interacting solids

according to Mantz (1977).

This type of behaviour is well known for clay particles where the surface forces are predominant in relation to other forces due to their small size and large surface area. The nature of such surface forces are repulsion (given mainly by the force between electric double layers) or attraction (given mainly by the London-Van der Walls forces which are originated by fluctuating dipole bonds). The soil-water electrolyte can then interact to give net forces of attraction or repulsion among the soil particles. The forces of attraction are sensitive only to changes in dielectic constant of the fluid and temperature and can be assumed as constant to a first approximation. On the other hand the forces of repulsion can be easily varied since they are sensitive to changes in electrolyte concentration, cation valence, dielectric constant and pH of the fluid.

Dispersion of a suspension consists then in creating conditions for maximum repulsion, given a net repulsive surface force between the particles. They are then prevented from a close approach therefore maintaining their individual size. The conditions for maximum repulsion can then be achieved by:

- Low salt concentration in the pore water. The increase of the electrolyte concentration leads to a decrease in the double layer 'thickness' allowing the particles to come closer together. For the same surface charge density, the decrease in cation concentration with distance from the particles is reduced for a high salt concentration as indicated in fig. 5.2. As already commented in Chapter 3 and presented in fig. 3.19, Vaughan (1976) also found experimentally the increase of flocculation with the salt concentration.

- High pH of the solution. The higher the pH, the greater the effective negative charge of the particles due to the tendency of the H^+ to go into solution and the greater the double layer 'thickness'. Also a low pH promotes a positive edge to negative surface interaction, often leading to flocculation from suspension.

- High water content. The increase in the interparticle distance should reduce the particle interactions.

The effect of soil particles concentration has been experimentally observed in this work and will be presented later on.

- Monovalent exchangeable ions. The greater the cation valence, the smaller the double layer 'thickness' leading to a decrease in interparticle repulsion.

When the forces of repulsion are decreased by changing the conditions commented above, the forces of interparticle attraction can act and the particles flocculate, forming a 'floc' of soil.

Although the slurry materials used in this experimental work were at the silt range size, their characteristics of flocculation and dispersion were determined. Mantz (1977) had already found that particles in the silt range size could behave as SIS. He carried out research with six fine silica solid grades (D_{50} = 15μ, 30μ, 45μ, 66μ, 110μ and 160μ) immersed in various natural aqueous electrolytes (distilled and deionized water; town supply artificially softened water; town supply hard water; brine water and artificial sea water), to investigate a commonly occurring sediment-fluid system. He measured the packing coefficients of the systems (C = volume of solids/total sediment volume) and analysed the results with the aid of classical colloid-stability theory. He concluded that for grade size < 66 μ the packing coefficient could be influenced by the type of electrolyte due to the development of surface repelling or attracting forces. The soil-water system with hard, brine and artificial sea water presented the same behaviour with C decreasing with decreasing the particle sizes. It was suggested that flocculation of the particles during suspension and 'bridging' effects during actual bed contact for the finest grades, could be the answer for this behaviour. When distilled water was used, the opposite behaviour was found, suggesting dispersion for the finest grades. These experimental results are presented in fig. 5.3.

It must be emphasized that excluding the finest and the coarsest materials used in these experiments, the other four silts were from natural fluvio-glacial silica with ferric ions absorption. Such ions could have influenced the surface chemical properties of the solids.

5.2.2 Test Results

The results of this series of tests have already been presented in fig. 4.2 in the last chapter. Some tests were carried out for each granulometric distribution curve presented. The results were found to be very consistent.

Three different soil-water systems were used in performing these sedimentation tests: distilled, 'hard' and 'brine' water were mixed with each silt, S1, S2 and S3 according to the slurry classifications presented in the last chapter. The soil concentration at this time was maintained equal to 30 gr dry soil/l, the same used in all filter tests performed.

SILT S1 → the influence of the system chemistry in the particle size is clear as shown in fig. 4.2; the material behaves as SIS and flocculates in the presence of 'hard' and 'brine' water while dispersion occurs in the presence of distilled water.

Vaughan (1976) has found good agreement between 'floc' size determined by the standard hydrometer test and by the initial rate at which a clearing front dropped in sedimenting clay of his tests. The same behaviour was found during the performance of these tests. It was observed that for silt S1 there were variations of completely cloudy suspension when using distilled water, quite cloudy suspension when using 'hard' water and actual lines of demarcation between clear water and flocculent suspension when 'brine' water was used. Figure 5.4 presents the ratio at which the clearing water thickness increased with time for the suspension S1B. The estimation of particle size D_{50} from this rate is similar to that found in the pipette test.

The degree of flocculation observed in the three systems S1D, S1H and S1B seems to be mainly related to the salt concentration in water. From zero concentration, when dispersion was obtained with distilled water, up to a concentration of more than 35 grams of salt/l, the material flocculated gradually probably due to a decrease of the double layer 'thickness'.

SILT S2 → the material is still affected by surface forces; it flocculates in the same way with 'hard' and 'brine' waters but not as much as silt S1; it is still a SIS soil.

SILT S3 → the material behaves as NONSIS soil and no difference was found in the particle sizes determined using the three waters.

The results presented by Mantz (1977) in fig. 5.3 showed that the 'brine' and 'sea' water gave the greater flocculation conditions due to their higher salt concentration in solution. Vaughan (1976) has found, as already commented in Chapter 3 that the 'floc' size or flocculation effect increases with salt concentration in water up to a threshold concentration of 10 m Eq/l. The 'brine' water used in the present tests had a salt concentration above this threshold.

The results of these sedimentation tests showed that the materials S1 and S2 used in the filter tests, although in the silt range size, could still be influenced by the surface forces and therefore by the chemistry water effect. The coarser silt S3 did not present any flocculation. Mantz (1977) found that materials from fluvio-glacial silica coarser than S3 could still behave as SIS. Probably differences in soil mineralogy and in the procedure used to determine flocculation (he measured the packing coefficient of the sediments as already commented) could be the reasons for such differences. He also found that both 'hard' and 'brine' waters would present the same surface chemical effects on soils of same mineralogy which was not observed with the test results of silt S1.

From the limited number of tests performed it seems that the threshold in salt concentration to the maximum flocculation condition decreases with the increase in the particle sizes range of soils having the same mineralogy. The maximum flocculation condition of silt S2 with 'hard' water while for silt S1 it occurred with 'brine' water, could be attributed to the fact that the 'floc' size determined in the S2H solution was already sufficiently larger so that the surface forces have an insignificant influence.

The variation of the ratio D^F_{50}/D^D_{50} with D^D_{50} of the soils S1, S2 and S3 is shown in fig. 5.5, where:

$D^F_{50} \rightarrow D_{50}$ determined when maximum flocculation was observed (when 'hard' or 'brine' water was used depending on the material)

$D^D_{50} \rightarrow D_{50}$ when dispersion occurred (when distilled water is used).

This ratio as defined, could be used as a measurement of the degree of flocculation of the soil particles. Values around 1 would mean that the particles do not flocculate while values above 1 would mean that they flocculate more or less depending on its actual value. Data obtained using other materials could establish a more complete range of such values. It would increase with the decrease in the particle size range. (Actually the D^D_{50} value must guarantee the maximum dispersion situation that could be obtained with dispersant agents during the sedimentation tests and which could be relevant as the materials are in the clay range size).

From the results presented in fig. 5.5, it could be expected that soils having similar mineralogy as the silts used in these experiments with D^D_{50} between 20 μ and 30 μ behave as NONSIS.

5.3 Soil Concentration Effect

5.3.1 General Aspects

The most used methods available for determining the particle size distribution of fine-grained materials, are based on the sedimentation principle expressed in Stokes's Law which relates settling velocity with particle size. Although this law was derived for a single sphere falling under gravity in a large body of water in a laminar flow regime, it has been used to determine particle sizes having non-spherical shape. The diameter determined in this way is that of an equivalent round particle having the same falling velocity as the actual particle. Stokes's Law has also been used for a concentration of particles in suspension and the maximum concentration allowed to have minimum interference of one particle to another seems to depend on the soil-water system interaction. There is some experimental evidence to suggest that if the weight of solid matter for clay soils does not exceed more or less 40 g in a 1000 cm^3 volume of suspension (which corresponds to a concentration of 1.5% in volume) the settling velocities may be determined with reasonable accuracy from Stokes's Law. On the other hand it has also been suggested (Allen, 1975) that concentrations less than 1% should be used because for such values agreement has been found between sedimentation analysis and other methods (e.g. microscopy), indicating that interaction between particles is relatively unimportant. The British Standard fine grading test procedure adopts the amount of 50 g of dry soil per litre, corresponding to a concentration of 1.9%. Actually these suggested concentrations do not take into account any kind of flocculation due to the soil-water interaction since in almost all standard test procedures the use of dispersant in solution with distilled water is specified to guarantee maximum repulsive forces between particles.

It seems therefore that there is not yet an entirely accepted standard concentration of dry soil in suspension for sedimentation analysis of the particle sizes. From the Stoke's Law it should be expected that the lower the concentration the more correct the estimation of the particle diameters will be. On the other hand, low values of concentrations lead to inaccuracies in weighing the small samples withdrawn as in the case of the pipette method used in the tests and also a small amount of material may not be representative of the whole mass of soil. When hydrometers are adopted, low values of concentrations can lead to insufficient accuracy in the specific gravity readings.

5.3.2 Test Results

In order to verify the influence of soil concentration in determining particle sizes, three series of sedimentation tests using the pipette method were performed with the finer silt S1 which was found to behave as SIS from the analysis presented in the previous section. The effect of the soil concentration on the soil-water system interaction was also verified. The tests were classified according to the water used in solution as:

SERIES 1 → Sedimentation analysis of silt S1 - distilled water solution. Four different soil concentrations were used: 15, 30, 50 and 60 grams dry soil/litre corresponding to 0.6%, 1.1%, 1.5% and 1.9% v/v respectively.

SERIES 2 → Sedimentation analysis of silt S1 - 'hard' water solution. Three different soil concentrations were used this time: 15, 30 and 50 grams of dry soil/litre.

SERIES 3 → Sedimentation analysis of silt S1 - 'brine' water solution. The same concentrations of Series 2 tests were used.

The results of these analyses are presented in figs. 5.6, 5.7 and 5.8. The variation of D_{50} with soil concentration for the three different systems is shown in fig. 5.9.

The tests of Series 1 showed an increase of the particle size with the increase of soil concentration: D_{50} value from 50 gr soil concentration test was estimated to be 30% greater than D_{50} from 30 gr soil concentration solution. Although the difference is not very large, it seems that the increase of the number of the particles in solution decreases to some extent the repelling surface forces acting among the particles (decrease of water content).

The same behaviour was found in the Series 2 tests with 'hard' water. The D_{50} for a concentration of 50 gr of soil was estimated to be only 9% greater than D_{50} for a concentration of 30 gr of soil. The increase of particle size with soil concentration could be expected since as the soil flocculates in the presence of 'hard'water, the increase in the number of particles could facilitate their approximation leading to a greater degree of flocculation.

Solutions using 'brine' water (tests of Series 3) showed the opposite behaviour from the other two series presented before: the particle sizes estimated decreased with the increase of soil concentration although the difference between a concentration of 30 gr and 50 gr was only 9%. Vaughan (1976) had already found the same behaviour when performing his filter tests for Cow Green Dam as commented in Chapter 3. It could be speculated that with this soil-water system, the repelling surface forces had already been decreased to a minimum due to the high salt concentration in water and any increase in the number of particles in suspension would have little effect.

It has already been postulated (Allen, 1975) that for an assembly of particles uniformly distributed the increase in the number of particles decreases the settling velocity and consequently the particle sizes estimated by Stoke's Law. The descent of a given particle would create a velocity field which would tend to increase the velocity of nearby particles. In opposition to this the downward motion of each particle must be compensated for by an equal volume upflow and for a system of uniformly distributed particles, the overall effect would be a retardation of the particles. Maude and Whitmore (1952) suggested the following empirical relationship between settling velocity and concentration:

$$\frac{u_h}{u} = (1 - C)^{\beta}$$

where

u_h = settling velocity of a particle in the presence of other particles

u = free-falling velocity

C = particle concentration

β = variable depending on the particle shape and size distribution which assumes the range of values

$4.67 > \beta > 4.2$ for $R_e < 1$

where R_e is Reynold's number.

Using this equation for the concentrations of the tests of Series 3 and $\beta = 4.65$ the following values were found.

1. C_1 = 15 gr dry soil/litre (0.57% v/v) $\rightarrow u_h = 0.74\ u$ and $D_h/D = 0.387$ where

   D_h = diameter corresponding to u_h in Stoke's Law

   D = diameter corresponding to u in Stoke's Law
2. C_2 = 30 gr dry soil/litre (1.13% v/v) $\rightarrow u_h = 0.949\ u$ and $D_h\ /D = 0.974$
3. C_3 = 50 gr dry soil/litre (1.89% v/v) $\rightarrow u_h = 0.915\ u$ and $D_h/D = 0.957$.

The ratio of $D_h^{C1}/D_h^{C2} = 1.01$ and $D_h^{C2}/D_h^{C3} = 1.02$ where D_h^{C1}, D_h^{C2} and D_h^{C3} are D_h values for the concentrations C_1, C_2 and C_3 respectively, were compared with the values shown in fig. 5.9 which were $D_h^{C1}/D_h^{C2} = 1.27$ and $D_h^{C2}/D_h^{C3} = 1.01$. The ratio D_h^{C2}/D_h^{C3} found experimentally agrees well with that found from the equation presented before. The large difference in particle size estimation between 15 and 30 gr of soil concentration could be due to errors involved during sampling and weighing a small amount of soils as in the first case. In any case the equations proposed by others in the literature, do not consider any flocculation of the particles due to chemical surface interacting forces, as is the case of these tests. The particle interactions due to a closer approximation are of physical character. Actually it was observed during the tests of Series 3 that the slurries with 50 gr of dry soil/l appeared more flocculated than those with 30 gr of dry soil/l and these more than those with 15 gr of dry soil/l. At least the line of demarcation between suspension and clean water became clearer as the soil concentration increased although it fell more slowly. It could be speculated that the increase in soil concentration leads the particles to settle 'en masse' as already commented by Allen (1975) giving a retardation of the particles.

From the sedimentation analysis results of these tests it is possible that the soil particles of silt S1 in the condition of Series 1 and 2 tests, would present some degree of flocculation due to chemical surface interaction among them. The 'flocs' so formed would then lead to a coarser particle size distribution result contrary to what is postulated in literature. On the other hand when maximum flocculation is provided as in the case of Series 3 tests or when the particles behave as NONSIS, the physical interaction of the particles would lead to a lower falling velocity and consequently to an apparently finer material. It follows that the effect of soil concentration in determining particle size by sedimentation process depends also on the soil-water chemical interaction or, in other words, if flocculation occurs or not. When 'floc' size must be determined(rather than the individual particle size), as in the case of the filter tests done and presented in the last chapter, this fact must be taken into account and a concentration about 30 gr dry soil/litre is recommended.

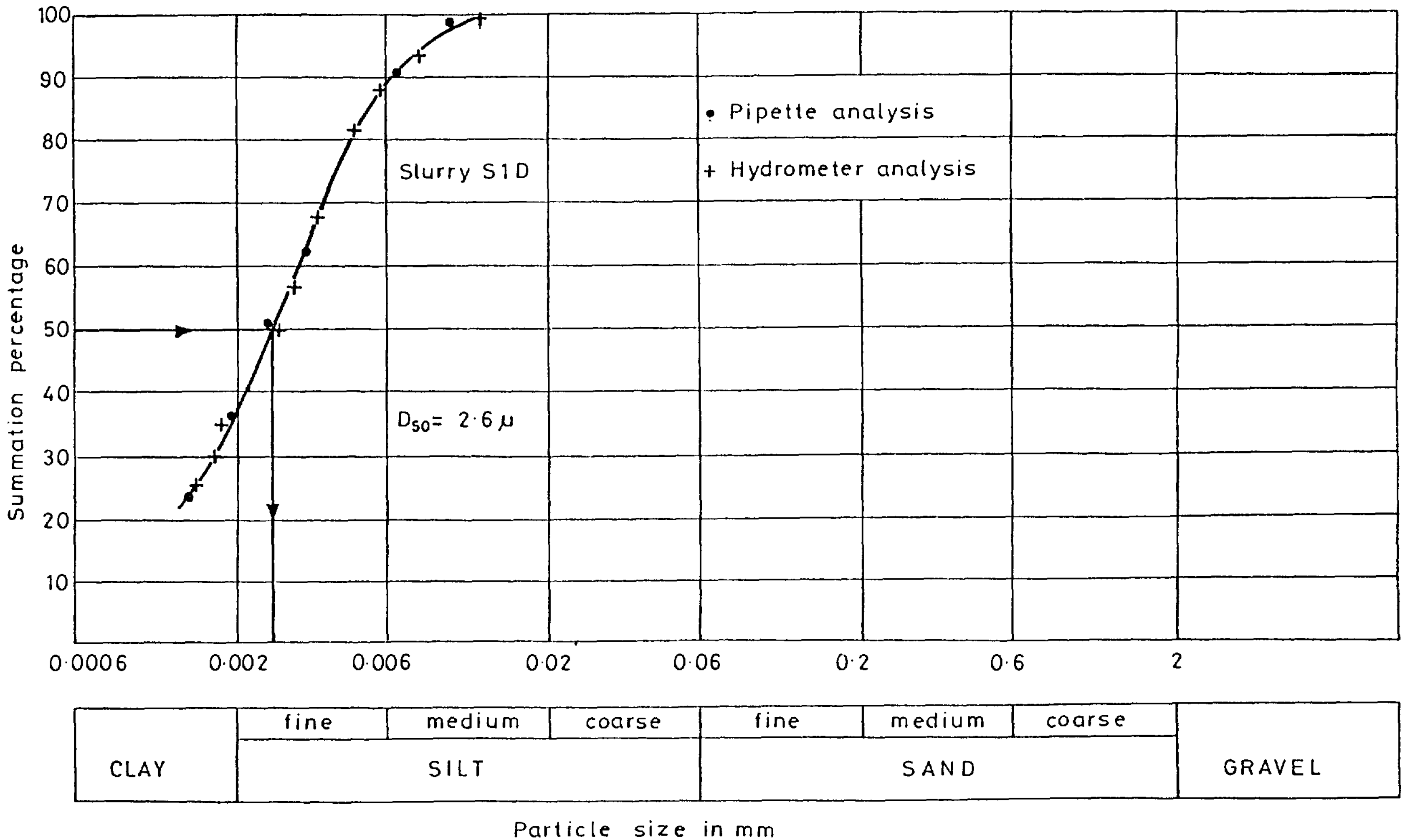


Fig. 5.1: Comparison between hydrometer and pipette sed. analysis

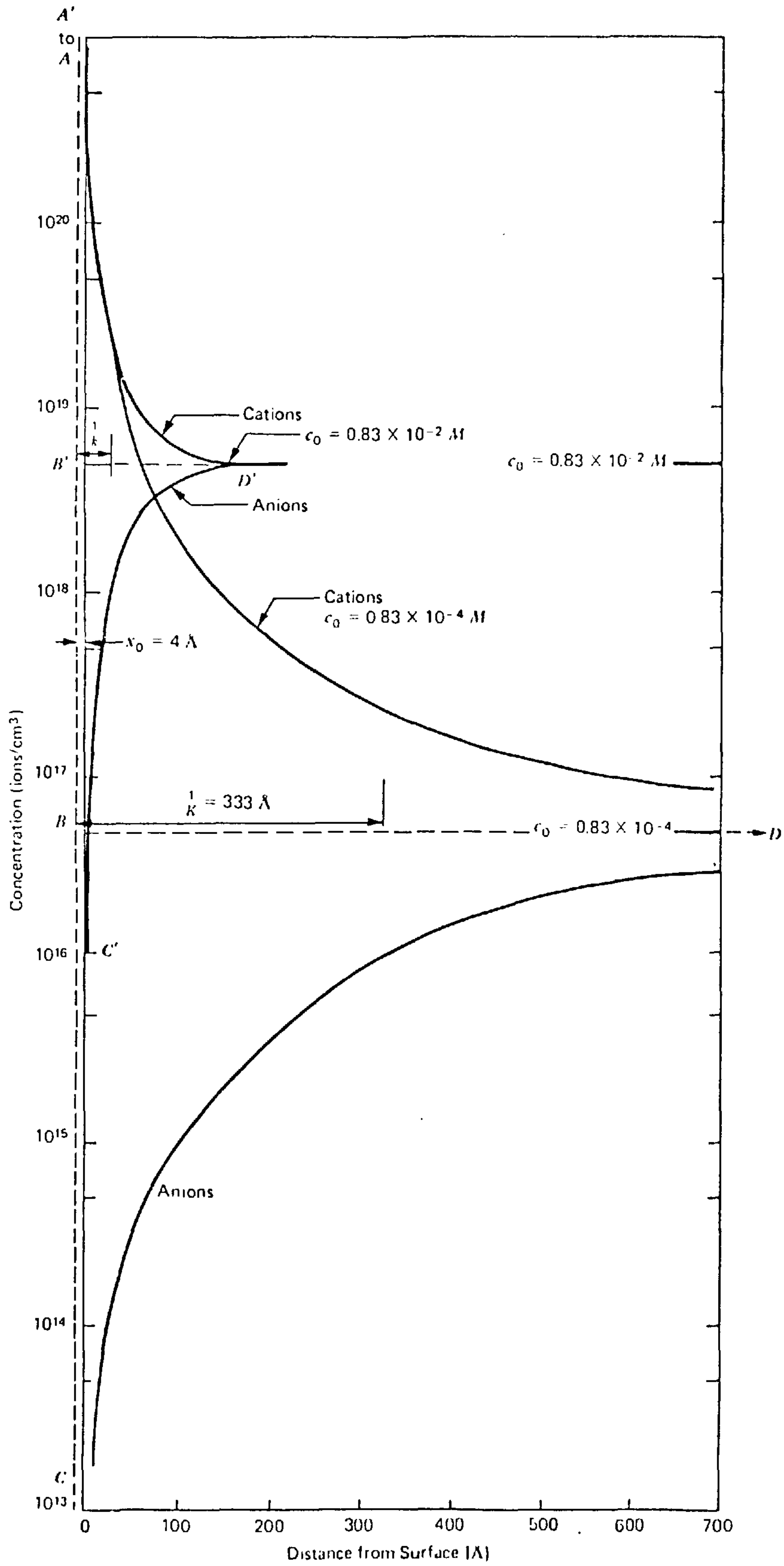


Fig. 5.2: Effect of electrolyte concentration on ion distributions in the double layer (after Mitchell, 1976) (1/K = double layer thickness)

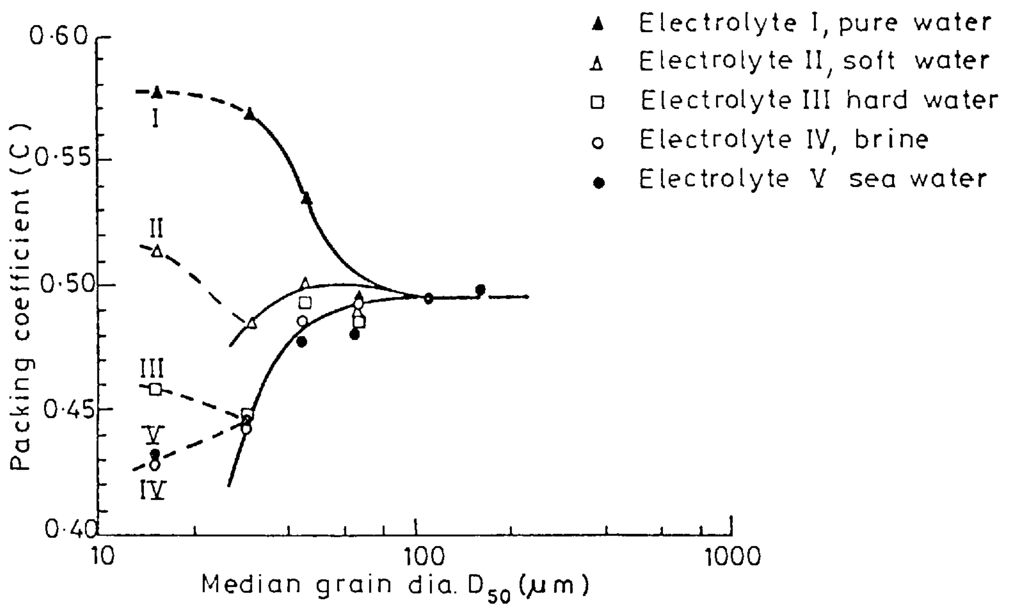


Fig. 5.3: The packing of grades of fine silica solids in various natural aqueous electrolytes (after Mantz, 1977)

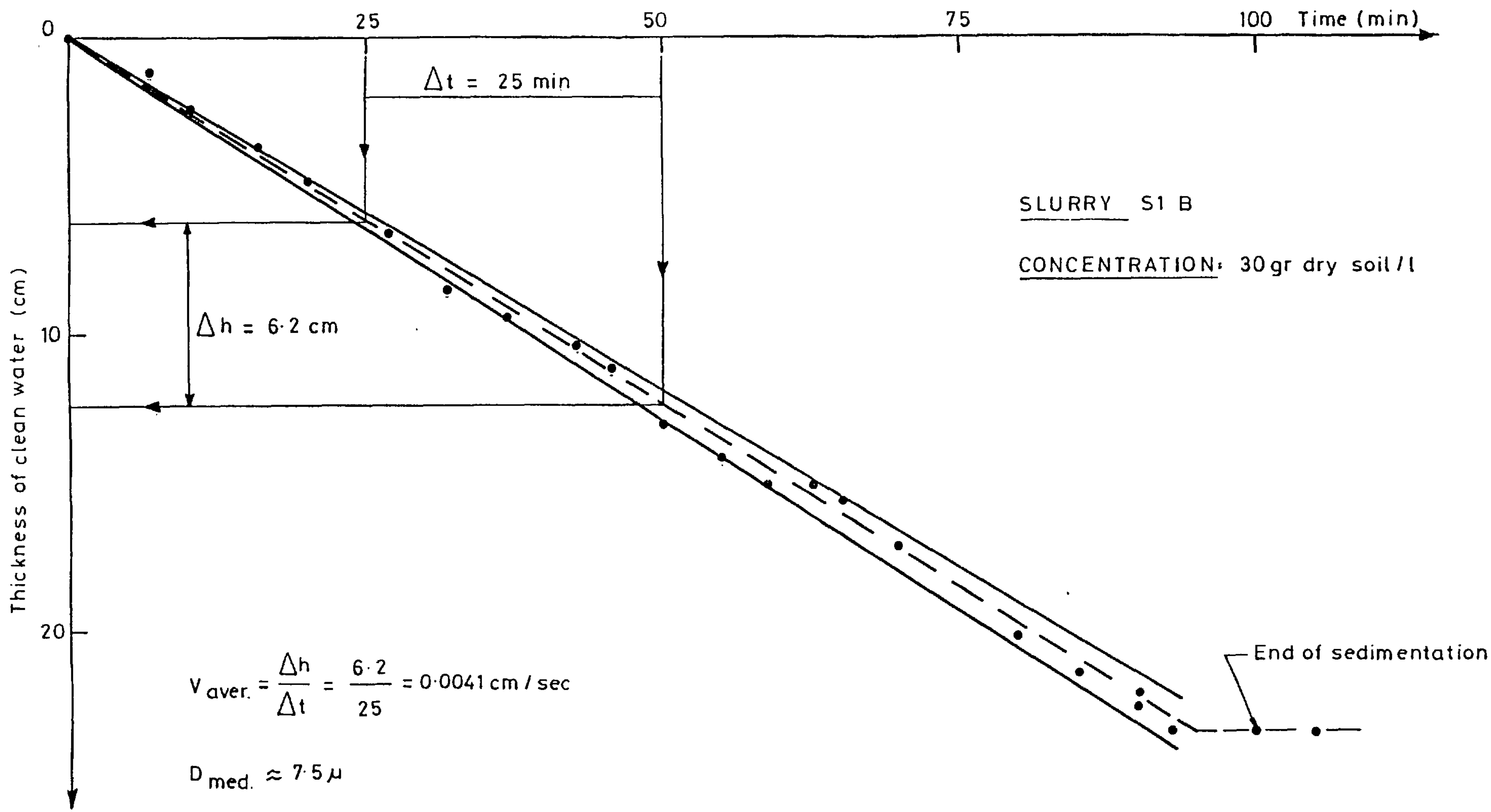


Fig. 5.4: Variation of clearing front drop with time

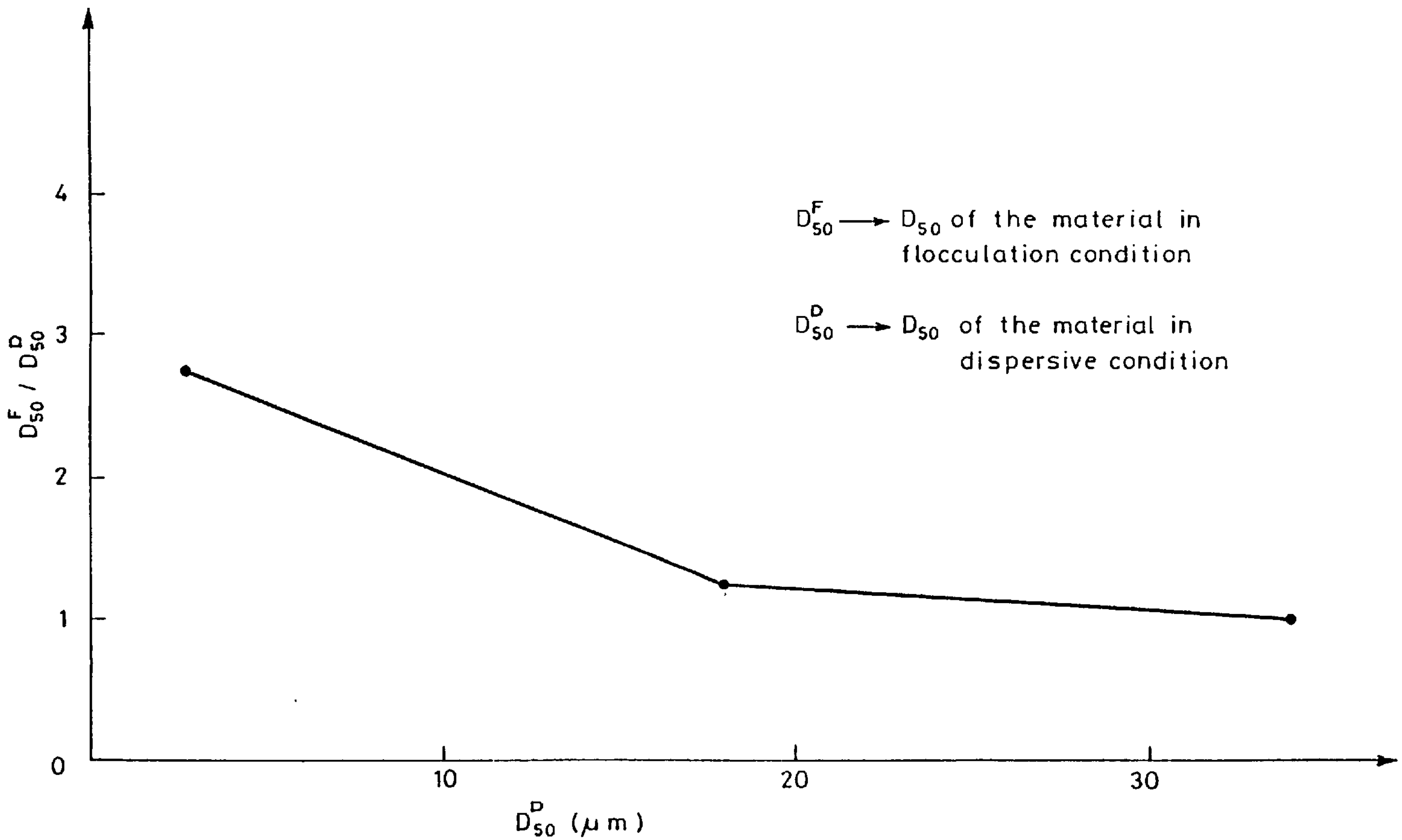


Fig. 5.5: Variation of D_{50}^D/D_{50}^F with D_{50}^D

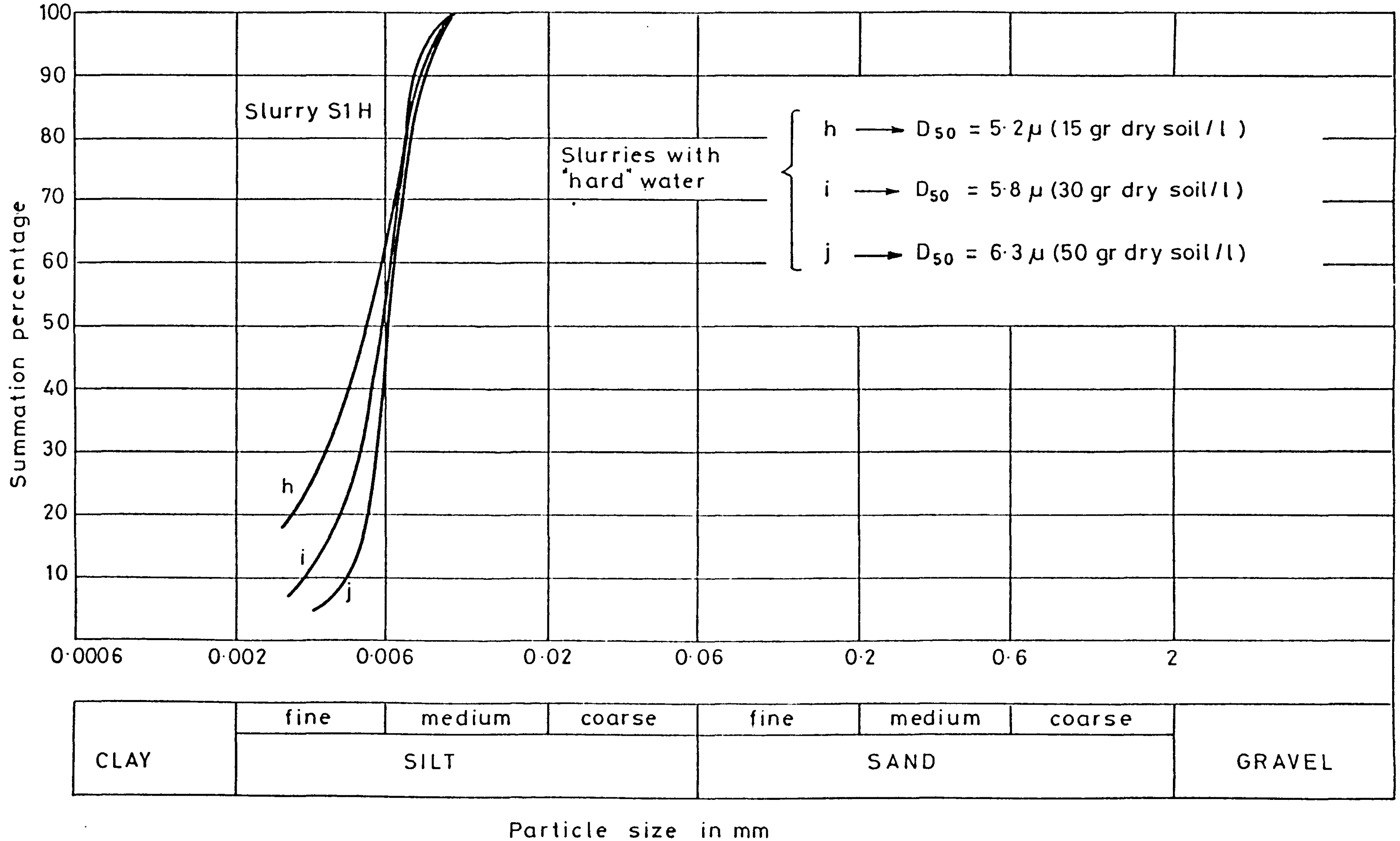


Fig. 5.7: Sedimentation test results - Series 2

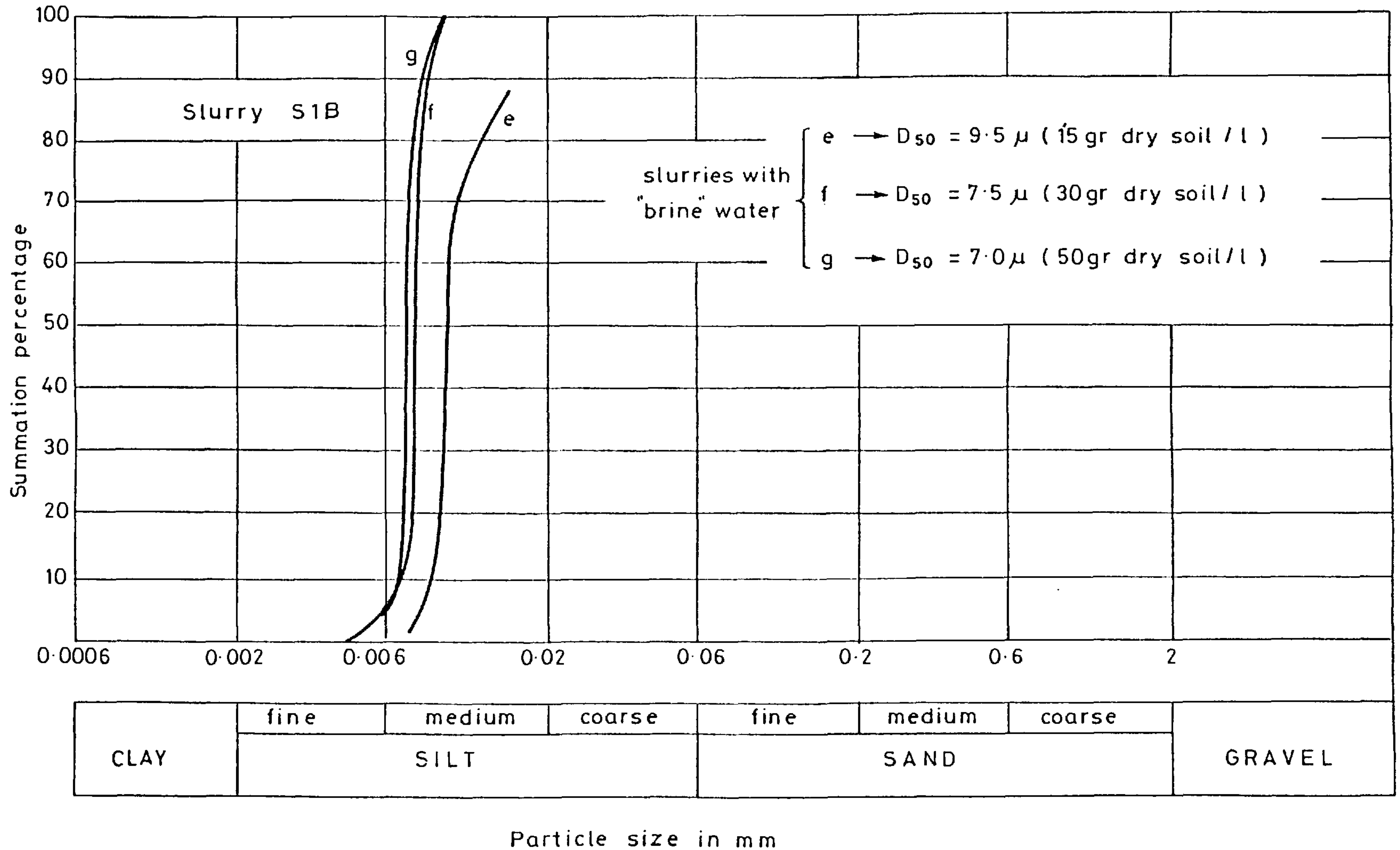


Fig. 5.8: Sedimentation test results - Series 3

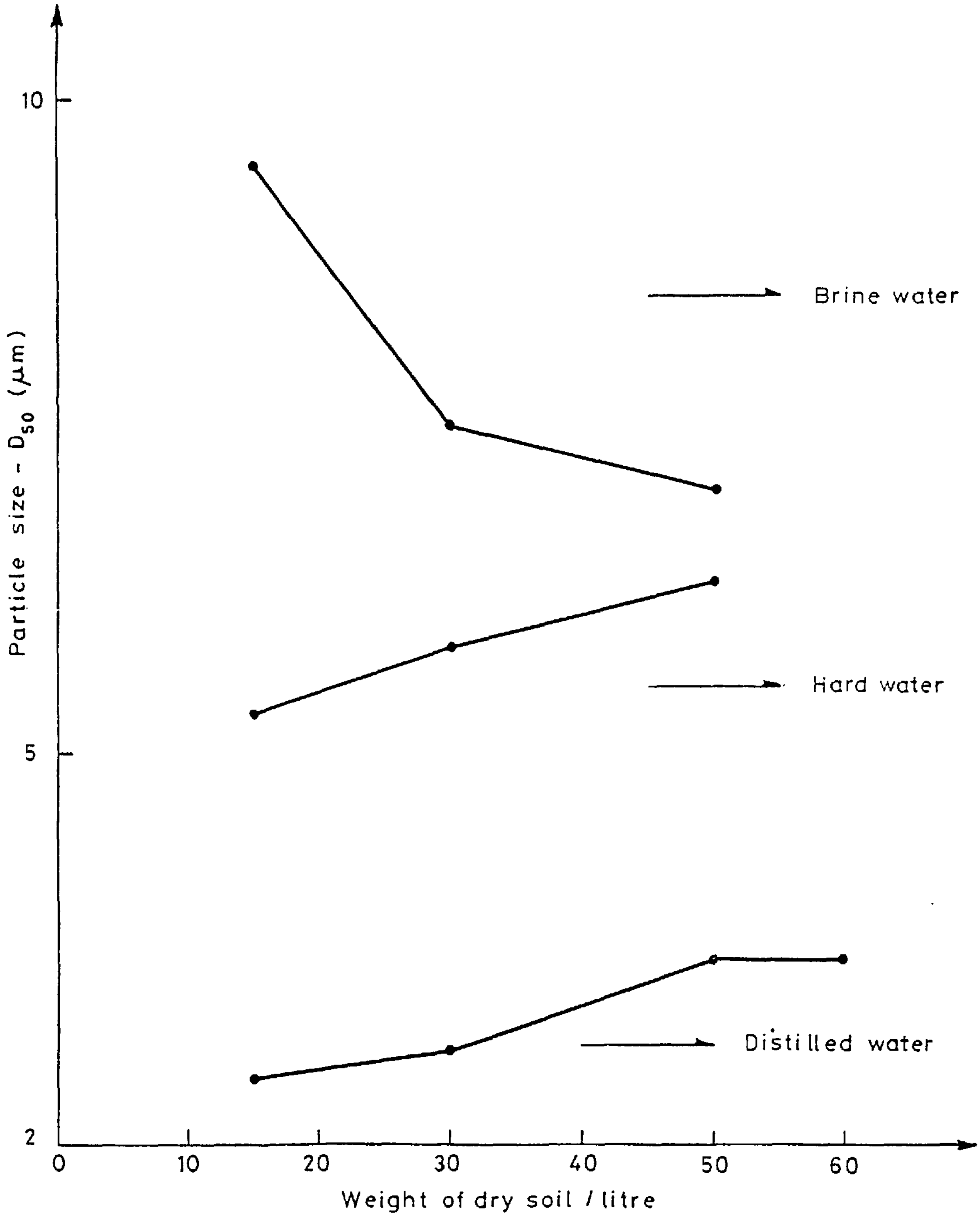


Fig. 5.9: Variation of D_{50} with soil concentration in suspension

6. PROPOSED FILTER CRITERION ANALYSIS

6.1 General Aspects

As already stated, the objective of the tests presented in Chapter 4 was to obtain data which could relate permeability of the filter with soil particle size to be retained. Since permeability of a soil is related to its pore size and it is also the pore of the filter material that controls filtration, permeability could provide a filter criterion.

The Cow Green filter test results, already presented in Chapter 3, for example, showed that both uniform and graded filters required a similar permeability to be effective despite the different gradings of both materials.

The theoretical equations developed for permeability are based on laminar flow through continuous channels of constant cross section area. These equations relate permeability with the square of some effective particle diameter. Actually, besides the irregular shape, the flow paths through the soil are sinuous. A shape factor, deduced from experience rather than deduced from the theoretical considerations, is then required to take into account the assumptions made when applying such equations to soils.

Hazen (1911) had already found experimentally that the effective particle size could be related to the D_{10} value of the soil, while larger percentage sizes have been used in other investigations. The variation of D_{15} of some sands with their permeability was determined as shown in fig. 6.1. The permeability was found to vary with $D_{15}^{1.8}$ of the soils, considering the permeability test results of the uniform sands used in these experiments (Series I) and Lund's (1949) results. Also, it has been found that hydraulic conductivity depends more on the small particles in a soil than on the large particles. This fact is then quite relevant when the sand is well graded. Vaughan (1976) has found in both Cow Green and Empingham filter design that the small amount of "fines" (the finest particle range size) present in a natural sand was critical in controlling success as a filter. Also the tests of Series II with graded filters presented in Chapter 4 showed the same behaviour. The variation of permeability with the percentage of fines of these tests and Empingham tests is presented in fig. 6.2. The variation is greater the more graded the material is.

As the present tests were concerned only with fine particles as base materials, attempts were made to find other published results in which the base soils were coarser than those. Most of the experimental results found did not present the permeability of the filters so that a relationship between k against δ (retained base particle size) could not be obtained. Lund (1949) provided such information from a series of filter tests using uniform base soils in contact with uniform filters. These results have been already mentioned in Chapter 3 during the literature review. Although the filter tests were not suspension tests as those done in the present work, it might be expected that a uniform base soil when tested in contact with a filter will pass or be retained in a similar manner to a suspension of the same soil tested together with the same filter. The data from this experimental work used in the present analysis are summarised in table 6.1.

As also already mentioned in Chapter 3, Lund (1949) performed another two series of tests where she concluded that the coarser base particles are the ones which actually control filtration and found that the ratio D^F_{15}/D^B_{85} was fully justified as controlling filtration for uniform filters and should be of the order of 8.

A series of tests using sieves as filters was also carried out during the present experimental work, which will be presented in the next chapter. It was concluded, in agreement with Lund's results, that a size between D_{80} and D_{90} for the base could be used as the controlling particles for the self-filtering process. Such filtration behaviour had already been suggested by Terzahi (1922) and many other authors when the value D^B_{85} was used in defining filter rules. Due to these conclusions and the observations of the slurry tests themselves, it was decided to take the effective particle size retained by the filters in the present tests (defined as δ), equal to the D_{85} value of the base material.

6.2 Empirical k against δ Relationship

The data used to define a boundary between effective and non-effective filters in the k (filter permeability) against δ relationship as shown in fig. 6.3 were taken from:

(1) London clay filter tests from Naranjo (1970)

(2) Empingham and Cow Green Dam filter tests from Vaughan (1976)

(3) Present experimental filter tests. (IS1H, IS2H, IS3H, IIS1H, IIS2H)

All these tests were suspension filter tests and there is no apparent difference in behaviour between clay flocs and quartz particles. Some results, where clogging occurred, were assumed to indicate instability since further eventual stability would depend on some variables not considered in the tests as already mentioned in Chapter 4. These results are also summarized in table 6.2.

Such an experimental boundary has the following equation:

$$k = 6.7 \times 10^{-6} \times \delta^{1.52}$$

(δ in μm and k in m/s).

The conventional filter tests taken from Lund (1949) plotted in the same figure, fitted quite well the proposed boundary.

Vaughan (1978) found a similar relationship when he used only the results of flocculated clays and London clay (data 1 and 2 presented before). The defined boundary had the following equation:

$$k = 6.1 \times 10^{-6} \times \delta^{1.42}$$

basically the same as the one proposed. He also reported the use of such an equation to design the filter for the Ardingly dam in Sussex, England.

It is found in the literature that filter criteria are usually concerned with the grades of base and filter material. When only very uniform soils are involved, a constant ratio between particle sizes of base and filter can be expected. When graded materials are used it seems that they do not obey a constant ratio to define filtration. The U.S.C.E. (1953) carried out a series of tests to measure permeabilities of five materials having the same D_{15} size and different uniformity coefficients. The differences found by them are of the same order of magnitude as the observed difference between failed and unfailed filters. The test results presented in fig. 4.13 in Chapter 4 suggest that the 'fines' are controlling the success of the sands as filters and as these "fines" also control the permeability of the materials, it seems that a filter rule as the one found experimentally is reasonable.

6.3 Main Points

The main conclusions regarding the filter tests and determination of the suspension base particles have already been discussed during the presentation of the results in Chapters 4 and 5 respectively. The main points of the analysis of the empirical relationship are now summarized.

1. The relationship between particle size to be retained and the filter permeability as found in fig. 6.3 seems sufficiently well defined for use at least for preliminary design purposes. Conventional filter rules, relating grading of base and filter materials do not deal with major differences in grading and grain shape, and with situations where pore size and permeability are con-

trolled by a small amount of fine material. Such situations are quite relevant when well-graded mixtures of silt, sand and gravel are used as filters. This type of material seems to have better particle retention properties than single sized sands, and to be generally easier to place than sands. Also if crushing must be used to provide a filter, they will be cheaper to manufacture.

2. The difference between the expected relationship ($k = A\delta^2$, where A is constant) and the one found experimentally requires further examination. It may be due to the complex geometric factors which relate permeability to pore size. Also, although δ was considered equivalent to the D_{85} of the base material, actually it is a range of particles coarser and even finer than that size which controls filtration as will be discussed in the next chapter. It could be speculated that there is a systematic variation of the ratio between the size of particle which will pass and the average size of the pores.

3. It seems that the relationship found can also be used to design filters for cohesionless base materials in the same way as the suspension tests. Specific filter tests with base and filter in contact, especially using graded filters, could further clarify this question.

Table 6.1 Lund's test results

Filter (B.S.Range size)	K (m/sec)	$D^F_{1.5}$	$\frac{D^F_{15}}{D^B_{85}}$	$\frac{D^F_{15}}{D^B_{15}}$	Test Results	Base Characteristics
14-25	2.5×10^{-3}	0.78	6.1	6.5	Effective	CU=1.05
10-18	8.5×10^{-3}	1.1	8.5	9.2	Effective	D_{85}=0.13
10-14	1.15×10^{-2}	1.3	10	10.8	Non effec.	D_{15}=0.12
7-14	1.40×10^{-2}	1.6	12.3	13.3	Non effec.	D_{10}=0.12
7-10	2.30×10^{-2}	1.9	14.5	15.8	Non effec.	
7-14	1.40×10^{-2}	1.6	8.2	8.9	Effective	CU=1.05
7-10	2.30×10^{-2}	1.9	9.8	10.6	Non effec.	D_{85}=0.195
1/8-10	2.90×10^{-2}	2.1	10.8	11.7	Non effec.	D_{15}=0.18
1/8-7	4.40×10^{-2}	2.6	13.3	14.4	Non effec.	D_{10}=0.18
7-14	1.40×10^{-2}	1.6	6.2	6.7	Effective	CU=1.05
1/8-10	2.90×10^{-2}	2.1	8.1	8.8	Effective	D_{85}=0.26
1/8-7	4.40×10^{-2}	2.6	10	10.8	Non effec.	D_{15}=0.24
3/16-7	3.80×10^{-2}	3.0	11.6	12.5	Non effec.	D_{10}=0.24
3/16-1/8	6.60×10^{-2}	3.7	14.2	15.4	Non effec.	
7-14	1.40×10^{-2}	1.6	4.5	5	Effective	
1/8-10	2.90×10^{-2}	2.1	5.8	6.6	Effective	CU=1.05
3/16-7	3.80×10^{-2}	3.0	8.3	9.4	Effective	D_{85}=0.36
1/8-7	4.40×10^{-2}	2.6	7.2	8.13	Effective	D_{15}=0.32
3/16-1/8	6.60×10^{-2}	3.7	10.3	11.6	Non effec.	D_{10}=0.32
1/4-1/8	7.00×10^{-2}	4.2	11.7	13.1	Non effec.	
3/8-3/16	1.15×10^{-1}	6.0	16.7	18.8	Non effec.	
1/8-10	2.90×10^{-2}	2.1	4.2	4.5	Effective	CU=1.05
3/16-7	3.80×10^{-2}	3.0	5.9	6.4	Effective	D_{85}=0.51
1/4-1/8	7.00×10^{-2}	4.2	8.3	8.9	Effective	D_{15}=0.47
3/8-3/16	1.15×10^{-1}	6.0	11.8	12.8	Non effec.	D_{10}=0.47

Table 6.2 Suspension test results

Tests	Base Particle to be Retained D_{85} (μm)	Filter Permeability (m/sec)	Test Results	Obs.
(1) London clay filter tests from Naranjo (1970)	D_{85} = 1.2 D_{85} = 1.4	1×10^{-5} 1×10^{-5}	Non effec. Effective	Uniform Filters
(2) Cow Green tests	D_{85} = 5.3	2×10^{-4} 1×10^{-4} 7×10^{-5} 2×10^{-5}	Non effec. Non effec. Effective Effective	Graded filter Unif. filter Unif. filter Graded filter
(2) Empingham tests	D_{85} = 7.9	4×10^{-4} 1.5×10^{-4} 7×10^{-5} 5×10^{-5} 3×10^{-5}	Non effec. Clogging Effective Effective Effective	Graded filters
(3) Tests Series I	D_{85} = 9.0	3.5×10^{-4} 2.2×10^{-4} 1.5×10^{-4}	Non effec. Effective Effective	Uniform filters
	D_{85} = 28.5	1.9×10^{-3} 8.7×10^{-4} 3.5×10^{-4}	Non effec. Effective Effective	Uniform filters
	D_{85} = 50	3.4×10^{-3} 1.9×10^{-3} 8.8×10^{-4}	Non effec. Effective Effective	Uniform filters
(3) Test Series II	D_{85} = 9.0	1.6×10^{-3} 4.2×10^{-4} 1.8×10^{-4} 1.3×10^{-4}	Non effec. Clogging Clogging Effective	Graded filters
	D_{85} = 28.5	1.6×10^{-3} 7.3×10^{-4}	Non effec. Effective	Graded filters

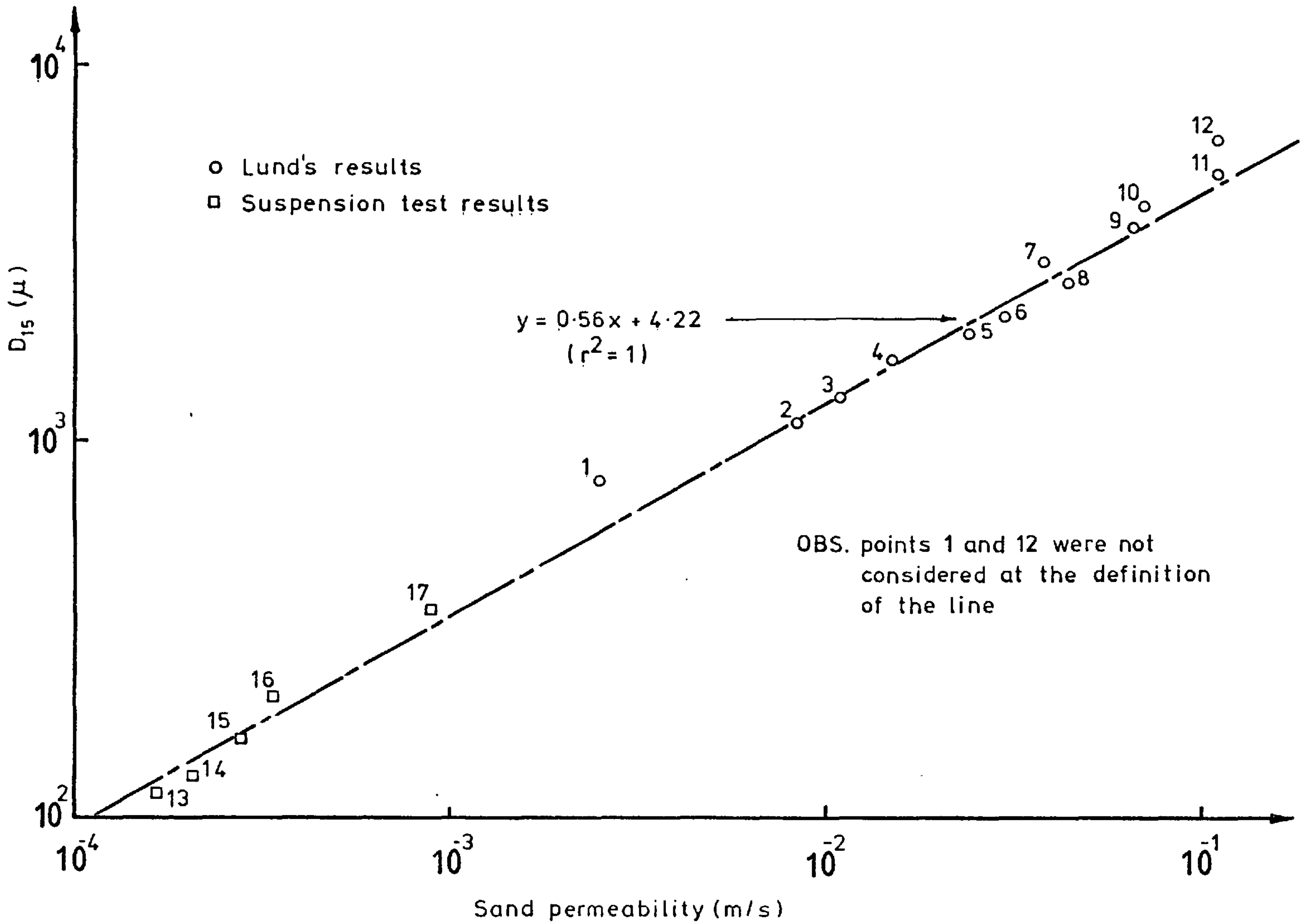


Fig. 6.1: Sand permeability against D_{15} of the soil

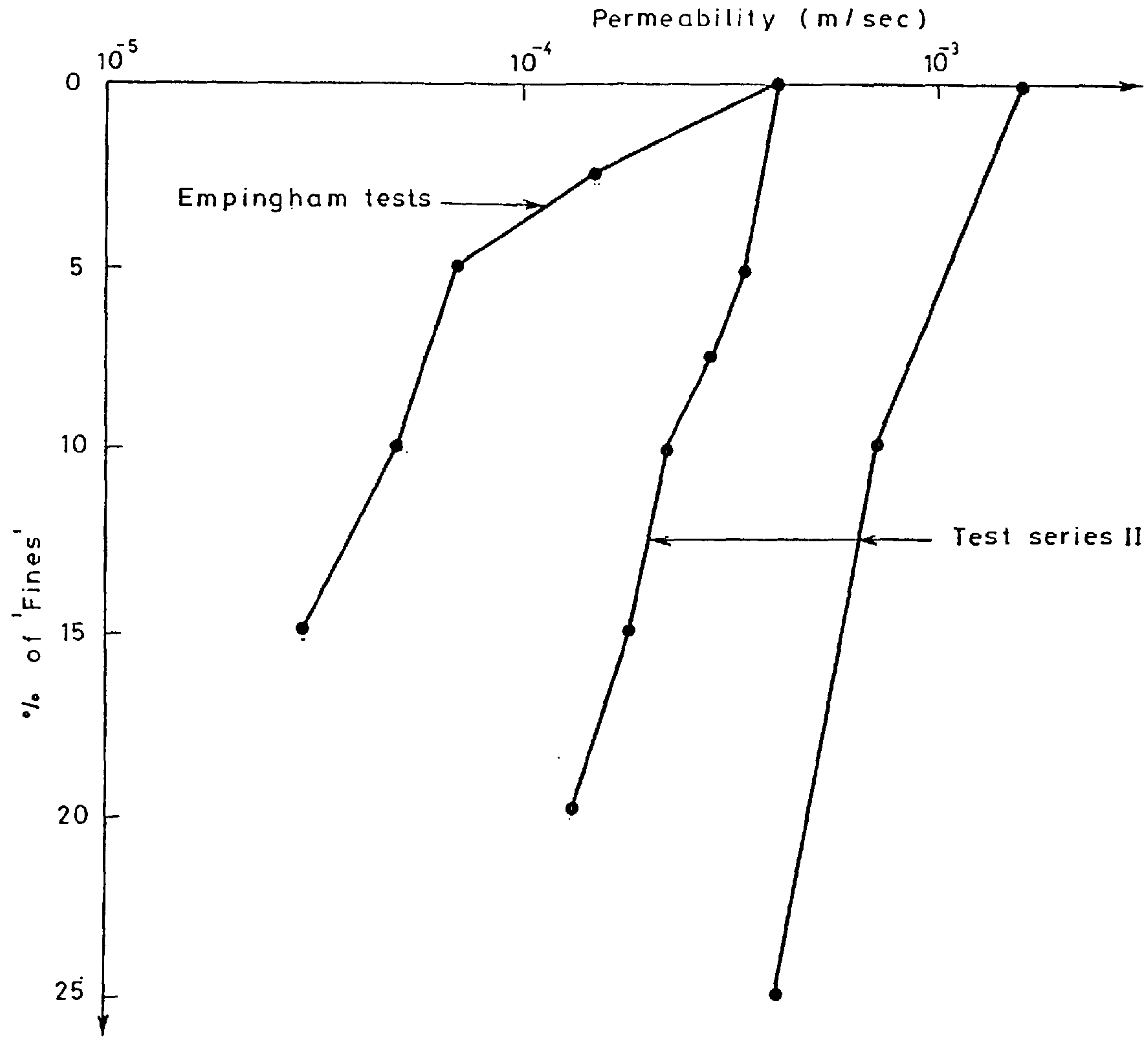


Fig. 6.2: Effect of "fines" on permeability

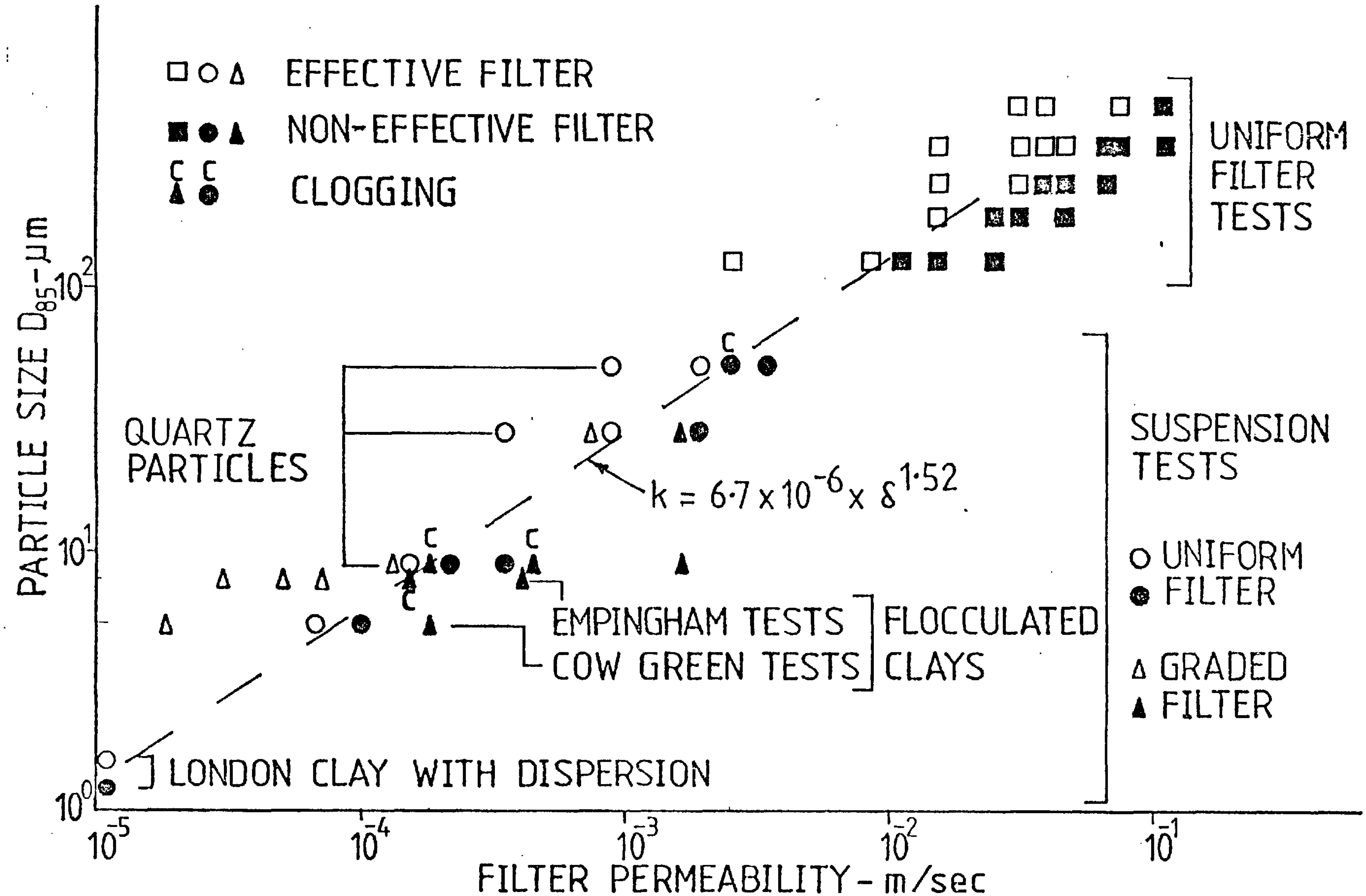


Fig. 6.3: Retained particle size against filter permeability

7. FILTER TESTS USING SIEVES AS FILTERS

7.1 Introduction

The first part of the experimental work, presented in Chapters 4 and 5, was carried out with the purpose of verifying the possibility of proposing a new filter criterion relating permeability of the filter against the particle size that the filter would retain, as already presented in the last chapter. These experiments were concerned with non intact conditions which are the case of water percolating through open cracks, eroding the soil of their walls. The results of these tests are shown as suspension tests in fig. 6.3 since slurries of fine material were passed through sands, filter materials. A series of tests using sands as base and filter materials in contact were also plotted in the same figure. The demarcation zone between effective and non effective filter was consistent with the suspension tests.

Vaughan (1978) analysing these results suggests that this proposed relationship, backed by sedimentation tests, could be used in the same manner for non-cohesive soils in intact conditions. Difficulties arise when deciding which particle size of the soil must be considered when using the proposed relationship. This particle size could depend on the material grading and on the uniformity coefficient of the soil.

The U.S.C.E. (1953) analysing filter tests results from themselves and using data from various sources, suggested that, as the uniformity coefficient of the base material increases up to a limit of 4, the ratio D^F_{15}/D^B_{15} could increase due to the fact that the coarse particles would provide self-filtering. The result of this analysis has already been presented in fig. 3.9. Above, this limit, the ratio stays constant and equal to 40. An explanation for this behaviour is not given. It could be speculated that when the material is graded above a certain limit, it becomes internally unstable and a self-filtering process will occur only after a reasonable loss of soil. Even so, it is not clear why a sharp limit in grading defining soil behaviour in terms of filtration should exist. It must also be emphasized that any relation for soils based on their uniformity coefficient is not strictly applicable for poorly graded soils.

Lund's test results, which have been presented in Chapters 3 and 6, were plotted together with the data provided in fig. 3.9 and they did not fit well the line proposed by the U.S.C.E. as a boundary between failed and stable filters. Considering Lund's tests, it seems that the suggested design line shown in fig. 7.1 could better define zones of safe and unsafe filters. More data could give the correct answer.

Using the suggested demarcation line, a first trial in defining the particle size to be retained in cohesionless soil was made using some sands of Lund's experiments and the k × δ relationship proposed. As the permeabilities of these sands were known, the particle size that they could retain when working as filter materials was then estimated. From the D^F_{15} value, the D^F_{15}/D^B_{15} ratio was found for different bases depending on the uniformity coefficient chosen and the D_{60} and D_{10} values of the bases could then also be estimated. The particle sizes that the filters could retain (from the $k \times \delta$ relationship), may therefore define which portion of the base materials should be retained, as shown in figs. 7.2A to 7.2F. The results of this analysis are shown in fig. 7.3 as the relationship between the summation percentage retained against CU of the base. Following all these considerations, it could be said that for base materials with CU up to 4 the retained particle sizes would be associated with D_{70} to D_{100} size range, going down to D_{40} for a base with CU = 20.

It was then decided to conduct a series of tests to investigate the validity of this behaviour. The tests consisted of filter tests using sands as base materials and sieves as filters retaining these materials in order to quantify the amount of material that would be lost in a certain soil grading and sieve mesh size combination.

For these experiments it was necessary to develop a new apparatus. A few tests with sieves were already done by Lund and the results are also presented and analysed in the present work.

7.2 Test Considerations

7.2.1 Equipment Development

The lay-out of the equipment developed is shown in fig. 7.4. The permeameter consisted of a 6 in length outer perspex tube together with a 3 in diameter inner tube, also in perspex, both tubes being quite tightly fitted one into another.

Beneath this inner tube the sieve was placed, being held by a ring of perspex cut from the same inner tube and the ring was supported in turn by the removable permeameter bottom. In this way the sieves could be changed depending on the type of test. Each sieve, previously cut to size, was held between two brass rings by screws, these rings being of the same internal diameter as the inner perspex tube. This would therefore guarantee that the sieve area was equal to that of the inner tube. The top of the equipment was also removable and it was attached to the outside tube by four screws. A thin "O" ring at the top and a thick one at the bottom provided the seal of the equipment against excessive leakage. In the middle of the perspex top, a rubber pipe of 0.75 in diameter connected the permeameter to a constant head tank device.

The permeameter was placed inside a sink that was kept full of water at the same level (overflow level) coinciding approximately with the upper surface of the base soil. A dish was placed under the opening of the base of the permeameter to collect the material washed through the sieve.

7.2.2 Filter Test Procedure

The sands used in these tests as base materials were prepared by mixing different single size sands and coarse and medium silts at specific proportions in weight in such a way that materials with different CU values were obtained.

The gradings of these materials are shown in fig. 7.5 which correspond to the materials S2 , S3 , A, B, C, D, E already presented in figs. 4.2 and fig. 4.10 and used in the filter tests discussed before. A sample of 700 g (more or less the amount used in each test) was prepared for all tests and after mixing the materials, a mechanical analysis of the resultant grading was always made.

In all tests the largest particle size was limited to less than 1/10th the diameter of the permeameter, following a procedure adopted arbitrarily by Bertram (1940).

It was decided to use a thickness of 3 in of the base material in all the tests with the support of some sieve test results carried out by Lund (1949) which will be commented on afterwards.

The test duration of four hours was considered to be enough. Bertram (1940) carried out some filter tests to verify the influence of test duration. He did not find any difference in test behaviour among four similar tests carried on for ½ h, 1 h, 2½ h, and 4 h. During the performance of the present tests, excluding tests of series 4S, it was observed that the loss of soil through the sieve occurred only at the very beginning of the test, the same having been observed by Bertram in his tests.

The gradient for all the tests was constant and equal to 30 and applied at once.

Most of the tests performed followed the procedure outlined below:

1. After being assembled, the permeameter was placed in the correct position, maintained vertically by its

support. The bottom was closed by a rubber bung and the permeameter was filled with tap water up to the sieve height.

2. The sand already prepared was placed on a plate and water added to full saturation. It was then mixed with a spatula until a homogeneous sample was obtained.
3. With the aid of a small spoon, the sand was then poured under water into the tube in small amounts until a height of 3 in of sand was achieved. No additional compaction was applied to the samples.
4. The top of the apparatus was then attached together with the tube linking it to the constant head tank.
5. After placement of the sand, the soil left in the plate was dried and weighed and the exact amount of soil used in the test recorded.
6. The small rubber bung closing the tube at the bottom was then opened and the water and soil coming out collected on a plate. After drying the soil,this amount was weighed and recorded.
7. The tube held by its support was placed in another tank deep enough to cover the tube. A small dish was placed below the bottom to collect any soil passing through. Saturation of the sand was then allowed from the bottom upwards to atmospheric pressure to eliminate as much air segregation as possible. After saturation the small hole at the top and the bottom of the permeameter were shut.

8. Any soil collected during saturation was dried, weighed and recorded.
9. The equipment was then placed in the bigger tank with a dish below it to collect the soil passing through the sieve during the test. The water supply was then opened and the bottom of the apparatus too. Any air from the water supply tube was allowed to escape using the small hole at the top. When all the air had gone the hole was shut and the time recorded as the beginning of the test.
10. Each test was run for four hours and a close watch was maintained to observe the test behaviour.
11. The material collected in the dish during the test was dried and weighted.
12. After each test had finished the remaining soil in the tube was collected, dried and weighed and a mechanical analysis of the sand performed.

7.3 Filter Tests Results

Four different series of tests were performed and they will be called series 1S, 2S, 3S and 4S in this work. Except for tests of series 2S, all the others followed the procedure described before.

7.3.1 Series 1S Tests

This series of tests consisted of eight tests using a sieve 18 (0.85 mm) B.S. mesh size. The sands used in these tests were prepared in such a manner that their D_{85} particle sizes were equal to the sieve mesh size. Actually after mechanical analysis of them, just before each test, the gradings were found to have between 80 and 90% of particles finer than the sieve size. These gradings are shown in fig. 7.6.

The tests had the purpose of verifying if the increase of the uniformity coefficient of the soil could lead to an excessive loss of material through the sieve so trying to simulate the behaviour of base and filter soils as proposed by the U.S.C.E. and already commented on in the introduction of this chapter.

The mesh size of the sieve was chosen equal to the D_{85} particle size of the sands because it was believed, as most investigators have already concluded, that if the pore diameters of the filter are small enough to hold the 85 per cent of the base, the base material, as a whole, would be stable. How small the pore diameters should be, varies from criteria to criteria since it could depend on many variables such as uniformity coefficient of the base and filter materials, the particle shapes, the porosity of both materials and even with the purpose of the filter.

Lund's filter test results already presented in Chapter 3, gave a strong basis for this conclusion. The results of two series of tests using uniform filters and uniform and graded bases have already been presented in figs. 3.7 and 3.8. Also a few tests with sieves, although not enough for a conclusion, showed that D_{85} particle size of the base soil could control filtration as will be shown subsequently.

The results of this Series 1S tests are presented in fig. 7.7. The uniformity coefficient of the sands was plotted against the percentage in weight of the total material lost through the sieve in relation to the total soil used in the test. The total material lost was computed as the summation of the losses during placement and saturation of the sand and during flow. Also it was checked by the difference in weight between the amount of soil used for test and the amount of soil after test. In some tests it was observed that some material was wasted when placing it in the permeameter and this was confirmed by the check mentioned above. Nevertheless, this wasted material was a maximum of about 2.5 g, therefore not significant in relation to the total weight of soil.

The test results presented in this way are suitable only as a qualitative analysis of each soil-sieve combination. The percentage of material lost has only significance for the present tests where the volume of the samples is about the same. As the area of the interface increases, it is expected that the amount of material lost increases and the percentage staying the same. On the other hand when the thickness of the base layer increases, the material lost should be about the same and the percentage would then decrease. Any extrapolation for field or other tests could be done by considering the equivalent thickness of material lost which would be independent of the thickness of the base material and area of the interface for materials with similar porosity. Such values are presented in table 7.1.

Table 7.1 presents the porosity, uniformity coefficient and percentage of material lost for each sand and test shown in fig. 7.7.

Table 7.1 Results of Series 1S tests

Sand	Unif. Coef.	Porosity	% of Mat. lost			Equiv. thickness of material lost (cm)
			Total	Plac+Sat	Flow	
P1	1.86	0.35	0.34	0.28	0.06	0.03
P2	2.45	0.34	0.85	0.78	0.07	0.07
P3	2.83	0.37	1.61	0.92	0.69	0.12
P4	3.30	0.31	1.77	1.71	0.06	0.14
P5	5.07	0.30	2.42	2.01	0.41	0.19
P6	6.42	0.30	3.29	2.69	0.60	0.26
P7	8.82	0.32	3.49	2.82	0.67	0.27
P8	9.31	0.30	3.77	3.54	0.23	0.30
P9	13.30	0.30	4.34	3.09	1.25	0.43
P10	6.80	0.31	16.40			1.27

After the test with sand P4 was finished, some vibration was applied to the equipment and more soil was observed passing through the sieve although it stopped as soon as the vibration stopped too. The percentage of total weight of soil lost during steady flow and vibration was then increased to 3.15% as shown in fig. 7.10.

The test result with sand P10 will also be presented in the series 4S tests. This sand was a gap-graded one and the test confirmed that for poorly graded soil, the coefficient of uniformity has no significance.

The results of this series of tests showed that the amount of soil lost increases with the uniformity coefficient of the sands. Considering only the sand tests from P1 to P9 (uniform and well graded ones) the percentage of material lost varied from 0.34% for the most uniform one, P1 with CU equal to 1.86, up to 4.34% for the most graded one, P9 with CU equal to 13.30. Although the increase was almost thirteen times for sand P9 in relation to sand P1, it seems that only 4% of loss of soil could not be sufficient to define failure for the sand-sieve combination. Also this increase occurred gradually with CU and a sudden failure as suggested by the design line of the criteria proposed by the U.S.C.E. and already presented in fig. 7.1,when the sand had a CU equal to 4.0, was not observed. Table 7.1 also shows the percentage of soil lost during placement plus saturation and during flow in relation to the total one. More than fifty per cent of the soil loss occurred during placement and saturation for all tests, suggesting once more that some fines could pass easily through the sieve until a situation establishes with some coarse particles arriving and retaining the subsequent ones by filling the mesh sieve square or by bridging over it. After tests had finished, the wet samples were taken from the apparatus to allow them to stay as a "cake". Different cross sections through the soil "cake" were visually analysed and the surfaces showed an homogeneous distribution of the particle sizes except for the one which was in contact with the sieve which presented a clear concentration of coarse particles throughout the area. This suggests once more that filtration is controlled by the coarse particles.

A rough calculation was made in some tests, tryint to find out how many particles with size equal or greater than the D_{85} size were available in the sample used for tests. It was considered that these particles could control filtration. So from the weight of the soil lost during test the volume of the grains was estimated using the density of the grains equal to 2.70. From this, the total volume of soil that should be mobilized, considering a void ratio of 0.45, for the whole mass to be finally in equilibrium was then calculated. The volume of soil grains with sizes $\geqslant D_{85}$ that exists in this volume could then be estimated and by dividing it by the volume of a single particle (considering its typical diameter equal to D_{85}) the number of particles with diameter $D \geqslant D_{85}$ that could exist in this volume was then found if the sample was completely homogeneous. Considering the space area that should be filled in the sieve to block it completely, the number of particles with $D \geqslant D_{85}$ that was necessary to be available should be more than double that existing in the supposed mobilized volume. From this, it could be said that the sieve mesh space should not be totally filled by the D_{85} particle sizes and that filtration in the base-filter contact could be also controlled by particles smaller than the mesh size, most probably by bridging or arching effect of the particles. On the other hand, it could also be speculated that although the sample was apparently quite homogeneous, the distribution of the grains would vary in directions parallel to the layer. In this way the amount of particles with $D > D_{85}$ mobilized to control filtration could be greater than that expected and enough to retain the subsequent particles by themselves. In the same way the opposite could occur and as the nine tests presented consistent results, it seems that although the non-homogeneity of the soil could occur it would not be the only reason for such results. Kjellman (1963) had already postulated that due to the fact that the coarse grains would not be uniformly distributed in the volume of the soil layer, being dispersed at random, there is a risk that in some part of the filter area the base material will not contain enough coarse grains to block all entrances to the pore channels of the filter. He developed a probabilistic formula for the idealized uniform filters with round particles with the same size in a dense state, assuming some risk of piping which is a function of the thickness and the area of the finer layer, and of the diameter and the percentage of its coarse grains. He presented the formula as:

$$\alpha^2 T \log(1-n) = D[\log (1-s) 4D^2\sqrt{3}/A]$$

where

T = thickness of the base layer

D = diameter of the base particles corresponding to D_{85}

A = area of the filter

α = factor which depends on the pore size of the filter layer and on other variables in a still unknown way

s = probability that piping does not occur through any pores of the filter material.

Considering the following values:

$T = 7.62$ cm; $A = 45.60$ cm^2; $D = 0.085$ cm; $A/4D^2\sqrt{3} \simeq 7600$ (which represents the number of pores at the filter interface and was taken approximately as the number of squares in the mesh sieve), from the sieve tests of this series and

$\alpha = 1.6$, as suggested by the author, into the formula proposed, no risk of piping was found for such base-sieve combinations.

Actually Kjellman considered that piping through a pipe of the filter would occur if there was not grain in the mobilized soil (called by the author "active body" and found experimentally to be approximately a column perpendicular to the layer above the porous filter) with size equal or grater to that of the pore. The author did not make any restriction on the limit of the amount of soil lost that should be allowed before such a fact occurred. It seems that excessive loss of material could lead to an instability situation even if piping would be interrupted. Also the formula as proposed does not take into account any influence of the CU of the base soil and some effect could be expected since the more graded the soil the greater would be the possibility of segregation and piping. The formula as proposed is very limited due to the assumptions on the filter particle size and packing as already commented.

The effect of arching or bridging in filtration, has been already discussed in the U.S.C.E. work. As they commented, Taylor(1948),based on equisized spheres, found that for a filter at maximum density the ratio of the diameter of the filter particles to the diameter of the base material, all of a single size, must not exceed 6.5 in order to prevent penetration. Some experimental work, as that done by Bertram, using very uniform materials as base and filter showed that the ratio of any per cent size of the base could be as much as 12 before any appreciable penetration took place.

A few laboratory tests with permeameter were performed by the U.S.C.E. (1953) with the purpose of obtaining some data on the effect of arching. A very uniform sand was used in the tests at several void ratios together with a thin metal circular plate having a circular hole in the centre, placed on top of the sand. Three different plates with holes of 1/8, 1/4 and 1/2 in in diameter were used. The flow ran upwards and gradients were increased at various intervals of time and visual observations were made at the top circular hole to note the effect of arching. Although not in a conclusive way due to the small amount of tests done, the authors observed in one test that a hole approximately four times the mean diameter of the sand particles effectively held the material and that the arching effect increased with decreasing void ratio of the material.

Also, by performing an analysis of the soil particle distribution of sand P4 before and after test, it could be observed that the small amount of soil that passed through 18 mesh size sieve was composed of particles having diameter equal or smaller than the D_{60} size. In this way it could be speculated that particles with diameter between D_{85} and D_{60} could still hold the subsequent base soil by an effect of bridging or arching since the material was not well graded and it was assumed to be homogenously distributed throughout the layer. These effects are probably controlled by the size of the particles, packing and the grading of the soil.

7.3.2 Series 2S Tests

The purpose of these tests was only to check if the way of placement of the soil into the permeameter could affect the results found in the tests of the series described before. In other words, if homogeneity of the sample could vary with the way of placement of the soil. Two sands of the Series 1S tests were chosen to do the tests: sand P1 (CU = 1.86), a uniform one, and sand P8 (CU = 9.31), a graded one.

The tests were performed in the same way as the tests of series 1S, using the same 18 B.S. sieve mesh size. The procedure of the tests was the same as that described in section 7.2.2 with the difference in the way of placing the sand. This time the material was placed only wet in the tube with the aid of a small spoon after being homogeneously mixed. Also the tube did not have any water in it.

The test result for sand P1 was the same obtained in the Series 1S. Sand P8 presented a smaller amount of lost material but not significant enough to justify any difference in the results when using a procedure or another. These results are also presented in fig. 7.7 and in table 7.2 together with the results of Series 1S for these same sands.

These tests were actually performed when the tests of Series 1S were being done. With the results obtained, it was decided to continue using the test procedure described in section 7.2.2 for all the tests.

Table 7.2 Results of Series 2S tests

			SERIES 1S TESTS			SERIES 2S TESTS		
			Loss of Soil			Loss of Soil		
Sand	Unif. Coef.	Porosity	Total (%) Total Soil	Plac+Sat (%) Total	Flow (%) Total	Total (%) Total Soil	Plac+Sat (%) Total	Flow (%) Total
P1	1.86	0.35	0.35	0.28	0.06	0.45	0.43	0.02
P8	9.31	0.30	3.77	3.54	0.23	2.81	2.58	0.23

7.3.3 Series 3S Tests

As said before, the sands used in the Series 1S and 2S tests were chosen in such a way that their particle sizes between D_{80} and D_{90} were equal to the sieve mesh size used in the tests. To verify which particle sizes could better control filtration, a series of four tests was done. In all the tests the same sand P11 (CU = 2.86) was used with different sieves. The grading of the sand and the sieve mesh sizes are shown in fig. 7.8. The results of these tests are presented in fig. 7.9 and in table 7.3.

The test using 18 B.S. sieve which corresponds to the D_{83} particle size of the sand showed the same result expected from Series 1S tests. This result is also plotted in fig. 7.7.

Test 4 presented a greater loss of soil in relation to the other tests, almost five times more than the one obtained in test 3. Even if a loss of soil of 7.5% cannot yet be considered as failure of the filter, at least it could be said that there is a sharp increase in instability

Table 7.3 Results of Series 3S tests

Test	B.S. Sieve Size	Corres. Part.Size	Porosity	% of Mat. Lost			Equiva. thickness of mat. lost (cm)
				Total	Plac+Sat	Flow	
1	36(0.42mm)	D_{50}	0.37	0.47	0.26	0.21	0.04
2	25(0.60mm)	D_{68}	0.35	1.07	0.77	0.30	0.08
3	18(0.85mm)	D_{83}	0.37	1.61	0.92	0.69	0.12
4	10(1.70mm)	D_{97}	0.38	7.53	3.36	4.17	0.57

when particles of size equal or greater than D_{97} are relied upon to hold the whole mass.

These results also show that the effect of arching or bridging of the particles over the sieve mesh spaces as already discussed, must be considered to explain that a sand having 97% of its particles smaller than this sieve size only allows a 7.5% loss of soil in relation to the total mass.

With these results, the consideration made in the other tests that particles between D_{80} and D_{90} were controlling filtration, seems reasonable. Probably the results vary with the uniformity coefficient of the sand but for sands with CU up to 13.30 the tests of Series 1S have shown that a maximum of 4.34% of soil was lost when this range was considered. We must be careful when considering coarse particles as D_{97} for controlling filtration for graded sands as sand P9. If the proportion of material lost between test 3 and test 4 is maintained for other sands, a test with sand P9 using a sieve with mesh size equal to D_{97} as test 4, could result in a loss of soil of about 20% in relation to the total mass and this is believed to be a completely unsafe situation.

7.3.4 Lund's Sieve Tests

As already mentioned before, Lund (1949) has also performed some filter tests using sieves instead of soil as filter material, more or less in the same way as presented in this work. Although she performed only a few tests and did not present quantitative values, they will be commented on in this section and compared with the tests mentioned earlier.

Lund has carried out two types of tests:

1. In the first series of tests a sand with CU = 1.5 was used as base material with its D_{85} particle size equal to the sieve mesh size. Various thicknesses of sand were used from 1/2 in to 5 inches. It was observed that the total weight of sand passing through the sieve during testing was greatest with the 1/2 inch thickness and decreased until a thickness of 2 inches was reached. This weight remained essentially constant for the sands with thicknesses of 2, 3, 4 and 5 inches. She concluded from these results that until a thickness of at least 2 inches of sand is present, it is not possible for a "natural filter" to be established and with less than that, the whole mass of the sand is unstable.

Since the amount of soil which passed through the sieve was not given by the author the judgement used to define instability cannot be known. Anyway, if the amount of soil lost increased with the decrease of the soil layer thickness this is probably due to the fact that small amounts of soil would not be representative of the whole mass and they could not contain the necessary amount of coarse particles to establish self-filtering. Also, even if the uniform distribution of the particles was guaranteed, it could be speculated that the sample would not be sufficient to allow the necessary volume of soil to be mobilized in the self-filtering process, following the approach suggested before in section 7.3.1.

Due to these results and to the similarity in the tests procedure (as for example, the area of the sample) it was decided to use a thickness of 3" for the base material in all tests performed in the present experimental work since the results could be also affected by the CU values of the soil.

2. The second series of tests consisted of six tests with different sands and the same 18 mesh size sieve. All the sands had CU = 1.5. The sizes of the various samples were chosen in such a manner that 90, 85, 80, 70, 60 and 40 per cent of the sands respectively were finer than the sieve-openings. From the test results it was observed that once the sand is such that about 80 or 85 per cent of its grains (by weight) are finer than the sieve mesh opening, very little less sand is washed through even down to a sand with 40 per cent of its grains finer than the sieve. On the other hand, when a sand with 90 per cent of its grains finer than the sieve was used, about twice as much soil was washed through if compared with the 85 per cent sand. It was then concluded that the use of D_{85} particle size of the base in many criteria could be justified once the particles with size equal

or greater than the corresponding 85 per cent in weight of the sand being retained by the pores of the filter, were sufficient to hold the whole base.

The amounts of soil used in the tests were estimated once the permeameter diameter, soil thickness and way of placement were more or less the same as in the present tests. It was taken equal to the test with sand P1 used in this work. As the weight of soil that passed through was given, the percentage of material lost was estimated and the values are shown in table 7.4.

Table 7.4 Lund's sieve tests

Test	Part. size equal to mesh size	% of soil lost
1	D_{40}	0.04
2	D_{60}	0.08
3	D_{70}	0.10
4	D_{80}	0.10
5	D_{85}	0.10
6	D_{90}	0.24

The result of test 5 fits the test results of Series 1S and it is also plotted in fig. 7.7. Even a value of 0.24% of loss of soil does not seem sufficient to define instability in the soil-sieve combination. Also it seems that arching or bridging effects could explain this small amount of soil passing through the sieve.

7.3.5 Series 4S Tests

It was decided to perform four more tests with gap-graded soils after the result of test Series 1S with sand P10 which will also be presented in this section, making a series of the tests with gap-graded materials. The same sieve and test procedure of Series 1S tests was adopted.

The sands were prepared in such a way that two single particles range sizes (14-25 B.S. size and 100-200 B.S. size) were mixed in different proportions. The grading of the sands and test results are presented in figs. 7.10 and 7.11 and in table 7.5.

Table 7.5 Results of Series 4S tests

Sand	Proportion of Materials		Unif. Coef.	Poro-sity	% of Mat. Lost			Equiv. thickness of mat. lost (cm)
	100-200	14-25			Total	Plac+Sat	Flow	
P12	20%	80%	5.20	0.30	0.57	0.34	0.23	0.04
P13	30%	70%	7.30	0.33	1.27	0.91	0.36	0.10
P10	40%	60%	6.80	0.31	16.40	1.05	15.25	1.27
P14	60%	40%	1.46	0.33	20.90	2.30	18.60	1.63
P16	80%	20%	1.32	0.38	45.50	2.70	42.70	3.54

The sieve mesh size corresponded to the D_{93}-D_{74} range particle size of the sands.

From the results presented it can be concluded that there is a significant increase in the internal instability of the soil when the amount of fines (100-200 B.S. particle size) is increased from 30% to 40%.

When the two sands used to prepare the materials are considered individually, the coarser one (14-25 range) can work as an effective filter to the finer one (100-200 range). The D^F_{15}/D^B_{85} ratio would be around 4.6. If such a situation occurs it could be expected that the soil would be internally stable. Nevertheless the experimental tests showed that it would also depend on the relative amounts of each material which exist in the soil.

In the tests with sands P12 and P13 it was observed that the majority of the loss of soil occurred during placement of the material into the permeameter and that the loss of soil through the sieve during flow took place at the very beginning of the tests. The behaviour of these two tests was actually the same observed in the Series 1S tests and a loss of soil of 1.27% does not seem sufficient to define instability. It could be said that even if some fines can be washed through the sand, the coarse particles (from the 14-25 range) could still provide the filter at the sieve interface and hold the whole mass.

On the other hand, when the amount of fines increased to 40%, 60% and 80% the major loss of soil occurred during flow and gradually along the four hours of test. As the loss of soil increased with time of test one would expect that much more material would be washed if the tests ran for more time. These results must then be looked at only qualitatively and comparatively. Also part of the increase in the loss of soil observed in the test with sand P15, must be due to the fact that the sieve mesh size was equal to a coarser sand particle size, D_{93}, in comparison with the others.

One may speculate that there exists some sort of critical void distribution when the two sands are mixed, in such a way that the coarse sand cannot provide, at least immediately, the self-filtering at the sieve interface. The fines could then be gradually washed through the bigger voids of the sand and passed through the sieve where the coarse particles were not filling the mesh space. The particles of the finer sand were probably too small to provide any arching or bridging over the sieve openings. Also one could expect that with time the coarse particles could provide the filter for the finer particles after a certain amount arrives at the sieve interface to cover most sieve squares. This would be a function of time and an undesirable amount of soil could have been washed by then.

Although the soils were homogeneously mixed and carefully placed into the equipment to avoid as much as possible segregation of the particles, it must be considered that there is a risk that in some volume of soil per unit of the sieve area there might not be enough coarse particles to provide a filter at the interface, at least immediately. As the number of the coarse particles decreases in the different materials, the risk increases.

7.4 Conclusions

Most of the conclusions have already been dicussed during the presentation of the test results. Some points are now raised and discussed and the main conclusions are summarized.

7.4.1 General Aspects

The tests performed with sieves had the main purpose of trying to find out if the criterion, as proposed by U.S.C.E. (1953) could provide an extrapolation of the filter criteria proposed for cohesive base soils in non-intact situations to cohesionless base materials by defining which particle size could be used in the relation proposed as controlling filtration. The test results also provided some observations about mechanisms of the filtering process. In fact, the sieve mesh openings could be related to the voids of a soil filter and in a real situation of base and filter composed of soils, for each sieve void "size" a corresponding pack of grains should exist. In this way, if soils were used as filter with their void sizes equal to the "sieve void", in the tests, a smaller space of voids per unit area would exist to be blocked by the base particles. It is then expected that quantitatively the tests with sieves would present a greater amount of soil lost than tests with soils having a corresponding "sieve void". Even if an unsafe situation is expected from a base soil and sieve combination, the same will not necessarily occur in a base-filter soils situation where the coarse particles of the base material could be enough to fill the first voids of the filter and hold the subsequent particles.

Also the loss of material found for each base-sieve combination would correspond to some clogging at the base-filter interface when dealing with real soils as filters. As the clogging is gradual it could be expected that the first migration particles would provide a finer filter at the interface resulting in a smaller loss of soil from the base, mainly when graded filters are provided since then clogging of the soil is more critical, as found in the slurry test results presented in Chapter 4.

As mentioned before, no apparent reason seems to exist to explain the design line suggested by the U.S.C.E. (1953). The Providence District (1942) had already presented the same type of plot although a smaller number of results were considered at that time. From their data the D^F_{15}/D^B_{15} ratio increases almost directly with the base uniformity coefficient apparently without any definite limit (see fig. 3.5). The test results of Series 1S are replotted in fig. 7.12 in a similar logarithmic scale to fig. 3.5. It can be observed that the same tendency exists. Also Lund's (1949) test results with bases with CU equal to 5, 6 and 7 did not present the limit 40 for the ratio D^F_{15}/D^B_{15} as already shown in fig. 3.7.

An analysis of the test results presented by the U.S.C.E. (1953) and used in their proposed filter criterion showed that the five failed tests 1, 2, 3, 4, 5 marked in fig. 7.1 delimitated the safe and unsafe zones found by them and are responsible for the limit of the ratio $D^F_{15}/D^B_{15} = 40$ when the base soil has a CU value equal or greater than 4. These results are from the same series of filter tests performed by Karpoff (1955) and already mentioned in Chapter 3. The data provided by these tests are presented in table 7.6. The D^F_{15}/D^B_{85} found for these tests seems to be lower than those usually proposed by many authors as already reviewed in

Chapter 3, indicating that they could be conservative. The criteria used to define failure of these tests seems to have been based on visual observations and on mechanical analysis of the base and filter particle sizes, before and after testing as already commented before. As the base soils were well graded materials one might expected that a certain amount of material could have migrated into the filter before self-filtration had occurred, as found in the present tests using sieves. This could involve a personal judgement of failure even if the actual amount of base soil lost might be considered small for this. Also as the risk of segregation of the particles increases with the CU value of the soils, the clogging of the filter could become significant due to the way in which the material was placed.

As the failure criterion of conventional filter tests cannot be quantified in a standard way, the filter test result can often be the subject of personal judgement. Usually the failure criteria fall in one of the categories:

1. Permeability measurements.
2. Mechanical analysis of the materials before and after test.
3. Visual observation.

Permeability measurements, made at intervals during the test could determine changes in head loss produced by movement of fines from the base into the filter. Although this criterion seems to be the most reliable one, it is also the most difficult to obtain precisely. Due to this difficulty, most authors adopted mainly the last two criteria, the most subjective ones. Visual observations of failure in filter tests are usually related to migration of base particles into the filter. This migration, from a personal point of view, could vary from small amount of soil until complete clogging of the filter or even complete washout of the base particles through the filter. In this way, a loss of soil of 4% or 7% as found in the present sieve tests could be sufficient to justify considering migration of the base material into the filter as failure. It is believed that the judgement of failure in experimental filter work could explain the variety of limit ratios found between particles of base and filter soils in many different experimental investigations, even when the same types of soils were used.

It is also believed that the different tendency of behaviour found between that proposed by U.S.C.E. (1953) and the present sieve tests, Lund's (1949) tests and those from the U.S.B.R. (1942) are mainly due to the failure criterion adopted by Karpoff (1955).

7.4.2 Summary of the Main Points

1. Excluding the gap-graded soils, the coarse particles of a base soil are those which control the stability of the whole mass in the base-filter interface. It is believed that if any limit ratio between particle sizes of base and filter materials must be adopted as a filter criterion, a size between D_{80} and D_{90} for the base could be used as the controlling particles for the self-filtering process.

2. Particles with sizes smaller than the void sizes of filter also contribute in holding the subsequent particles by arching or bridging effects over these voids.
3. Besides the percentage of coarse grains, the thickness of the base layer is an important factor in controlling the self-filtration process since a certain amount of the coarse particles must exist for this. Also as the thickness decreases, the distribution of the soil particles may not be representative of the whole mass any more and segregation of the particles can allow the fine particles to be washed in a certain base-filter interface area. A thickness of 3" was found satisfactory for the tests performed.
4. The total loss of soil from bases having the same coarse particle range, increases with the uniformity coefficient of the soil, gradually. The more graded the material is, the more particles are washed until a sufficient number of particles, which control the filtration, arrive at the base-filter interface. Even though a maximum loss of soil of 4.35% was observed for the most graded sand (CU = 13.30), this was found insufficient to define instability in the base-sieve combination and also because this loss occurred mainly during the placement of the sand rather than during flow.
5. Gap graded base materials do not behave as mentioned above since the uniformity coefficient has no

significance. The behaviour of these materials will depend on the proportion of the amounts of the coarser and finer soils which delimitate the gap. Nevertheless, at least when the mixing is of the type done in the present work, where the coarser part of the soil alone works as a filter for the finer part, it could be said that if the proportion of the finer part is up to 30% in relation to the total mixture, the behaviour of the soil is controlled by the coarser part.

6. The sieve test results showed that the criterion proposed by the U.S.C.E. (1953) which relates CU of the base and the D^F_{15}/D^B_{85} ratio could be conservative. It was found that the same type of relationship presented by the U.S.B.R. (1942) is more in agreement with the present tests. It is then believed that although a greater amount of lost soil should be expected from the base as its uniform coefficient increases, during the filtration process, it is not sufficient to define failure of the base-filter combination once the coarse particles of base are sufficient to establish such self-filtering. The use of D^B_{85} in the relationship proposed in the last chapter for cohesionless soils seems to be reasonable at least up to a CU value of the base of about 15. More data of test results having real soils as base and filter in contact and with the permeability measurement of the filter before tests could provide clearer zones of safe and unsafe filters.

Table 7.6 Karpoff's test results

Test No.	CU BASE	CU FILTER	D^F_{15}/D^B_{85}	D^F_{15}/D^B_{15}	D^F_{15}/D^B_{50}	D^F_{50}/D^B_{50}	Obs.
1	7.33	19.74	4.63	45.30	11.33	98.20	Non. effec.
2	7.56	1.46	5.17	50.46	12.60	15.81	Non. effec.
3	7.33	5.86	7.08	69.23	12.31	60.42	Non. effec.
4	7.56	1.50	10.24	100.00	25.00	31.90	Non. effec.
5	23.81	6.00	4.03	191.70	22.10	81.10	Non. effec.

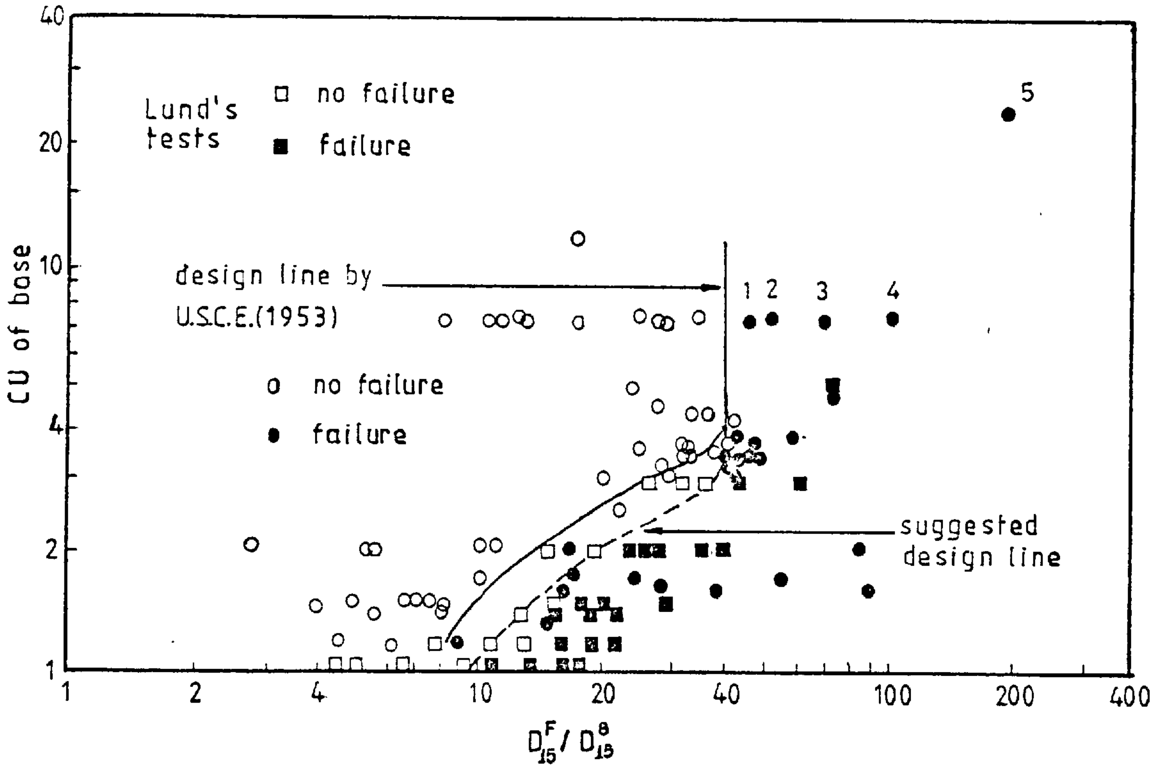
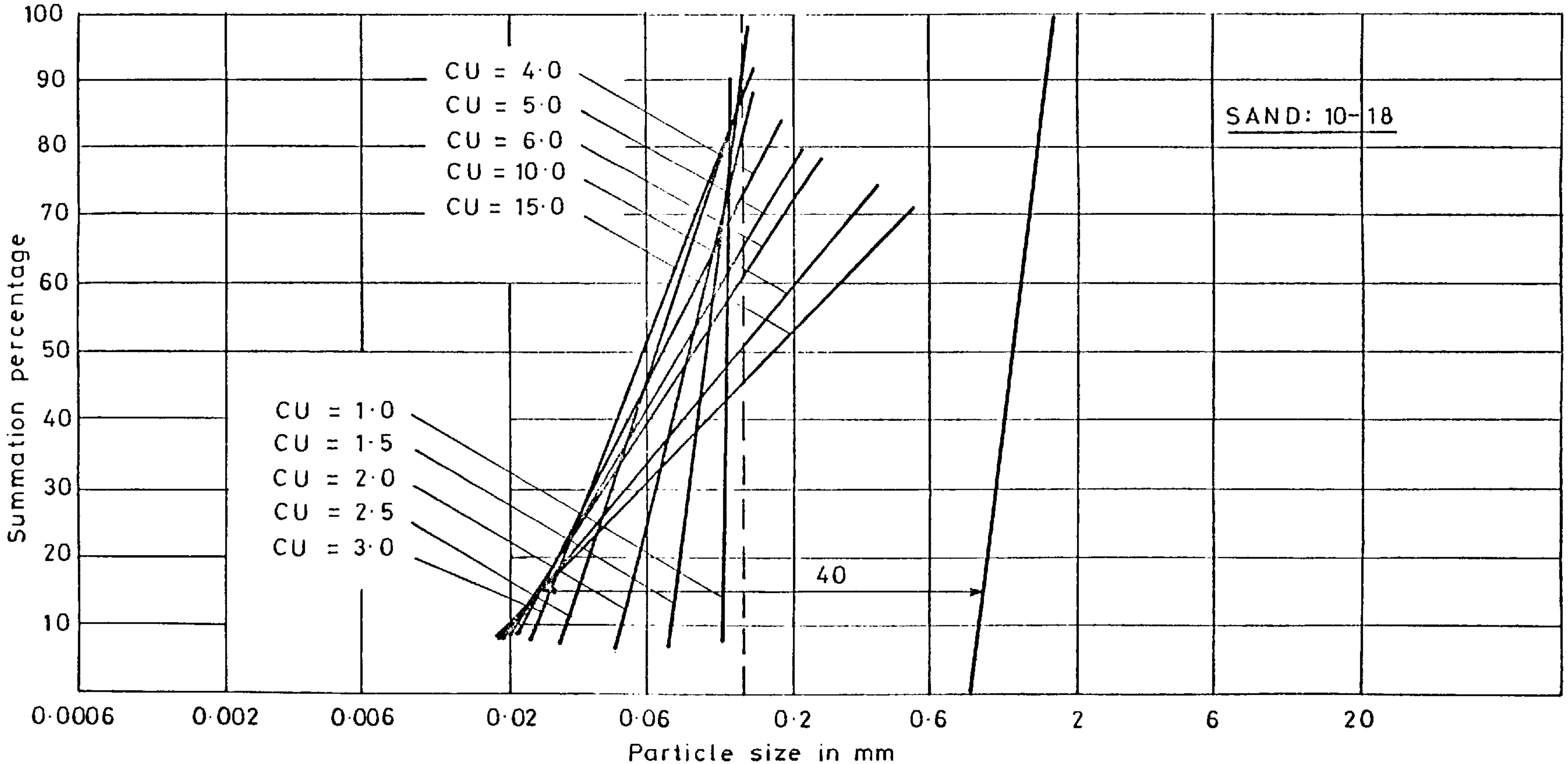
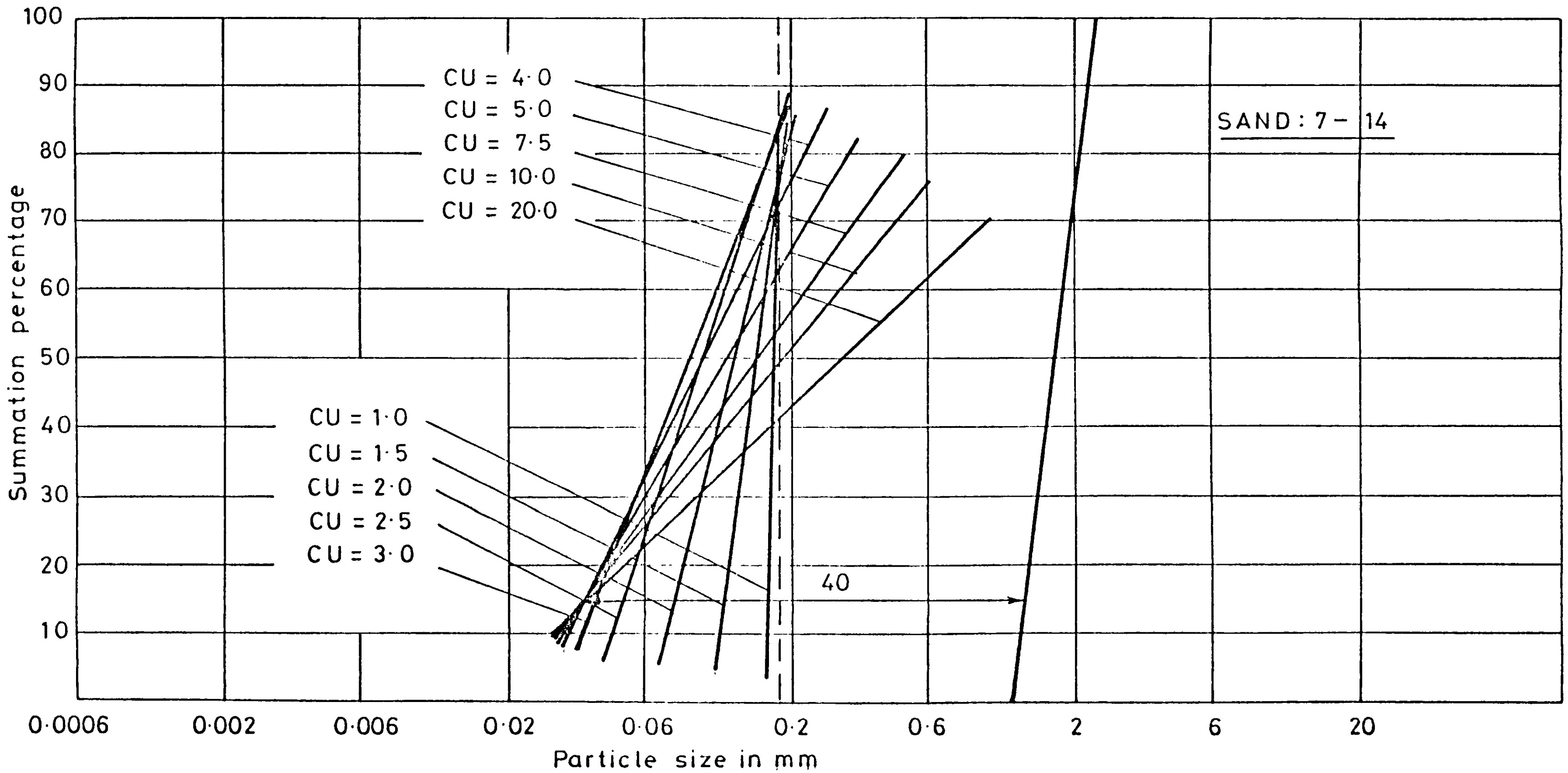


Fig. 7.1: CU of base against D^F_{15}/D^B_{15}



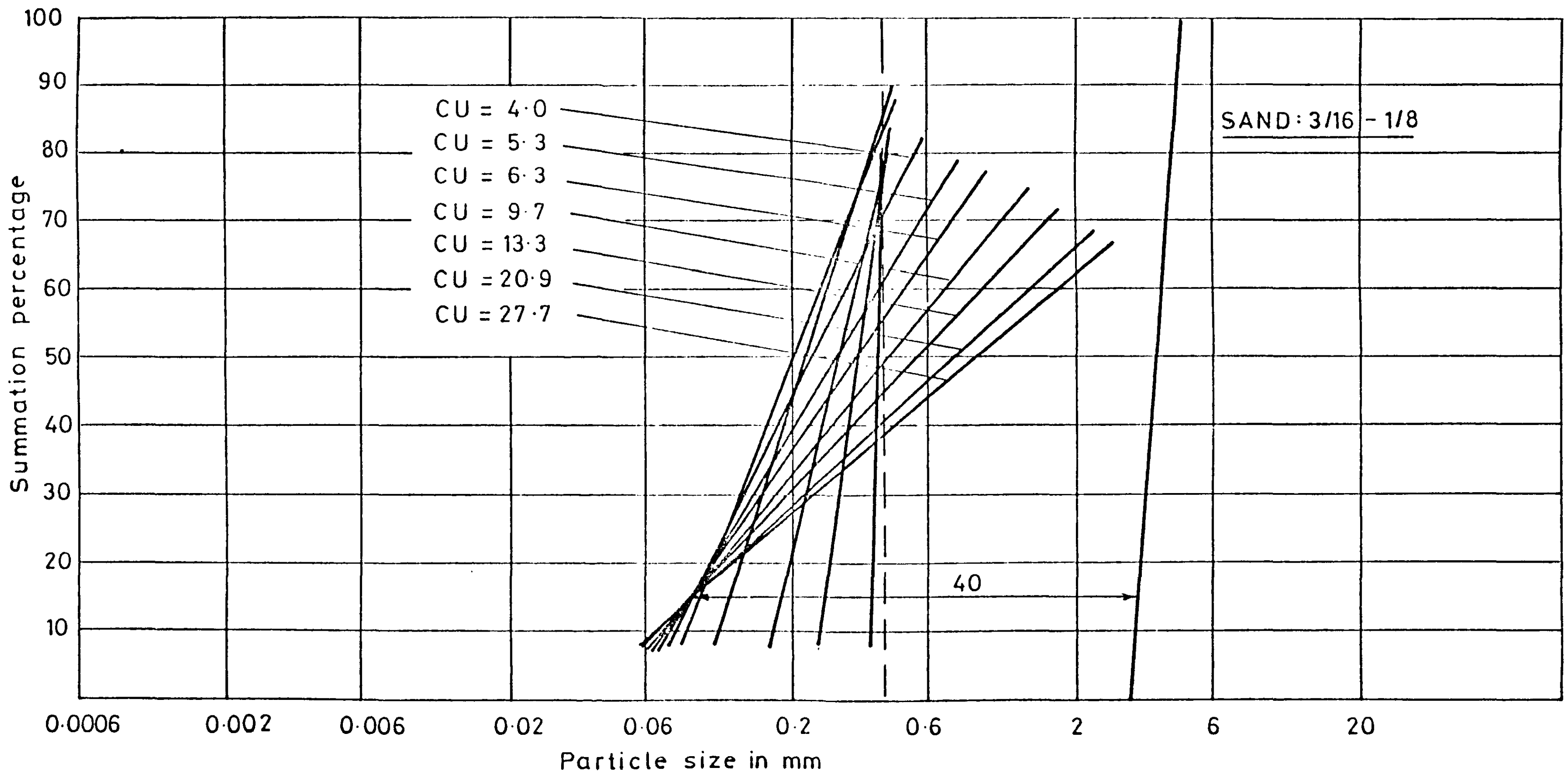
100
90
80
70
60
50
40
30
20
10
Summation percentage
CU = 4·0
CU = 5·0
CU = 6·0
CU = 10·0
CU = 15·0
SAND: 10-18
CU = 1·0
CU = 1·5
CU = 2·0
CU = 2·5
CU = 3·0
40
0·0006
0·002
0·006
0·02
0·06
0·2
0·6
2
6
20
Particle size in mm

Fig. 7.2A



100
90
80
70
60
50
40
30
20
10
Summation percentage
CU = 4·0
CU = 5·0
CU = 7·5
CU = 10·0
CU = 20·0
CU = 1·0
CU = 1·5
CU = 2·0
CU = 2·5
CU = 3·0
SAND : 7 – 14
40
0·0006
0·002
0·006
0·02
0·06
0·2
0·6
2
6
20
Particle size in mm

Fig. 7.2B



100
90
80
70
60
50
40
30
20
10
Summation percentage
CU = 4·0
CU = 5·3
CU = 6·3
CU = 9·7
CU = 13·3
CU = 20·9
CU = 27·7
SAND: 3/16 - 1/8
40
0·0006
0·002
0·006
0·02
0·06
0·2
0·6
2
6
20
Particle size in mm

Fig. 7.2C

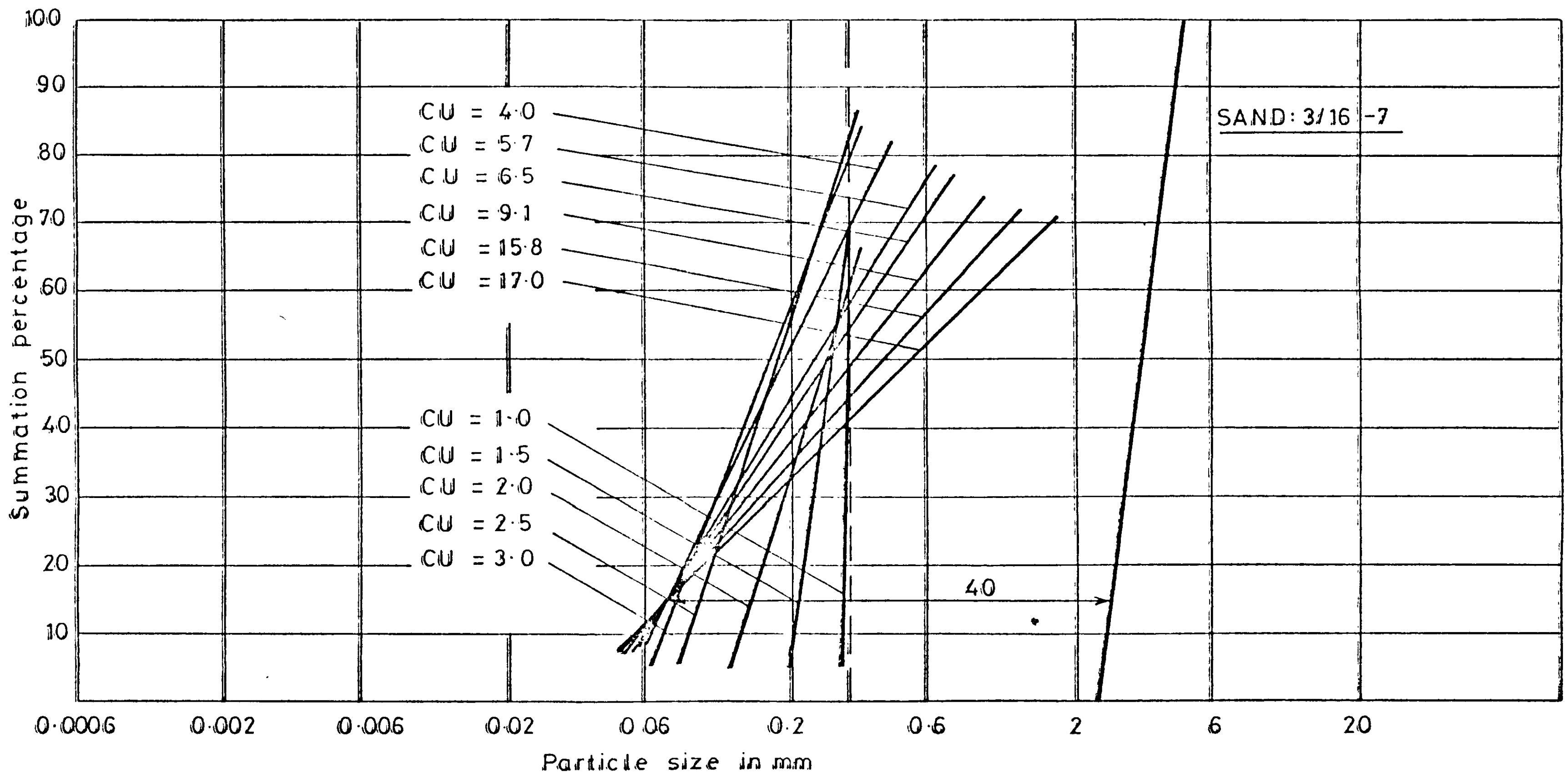
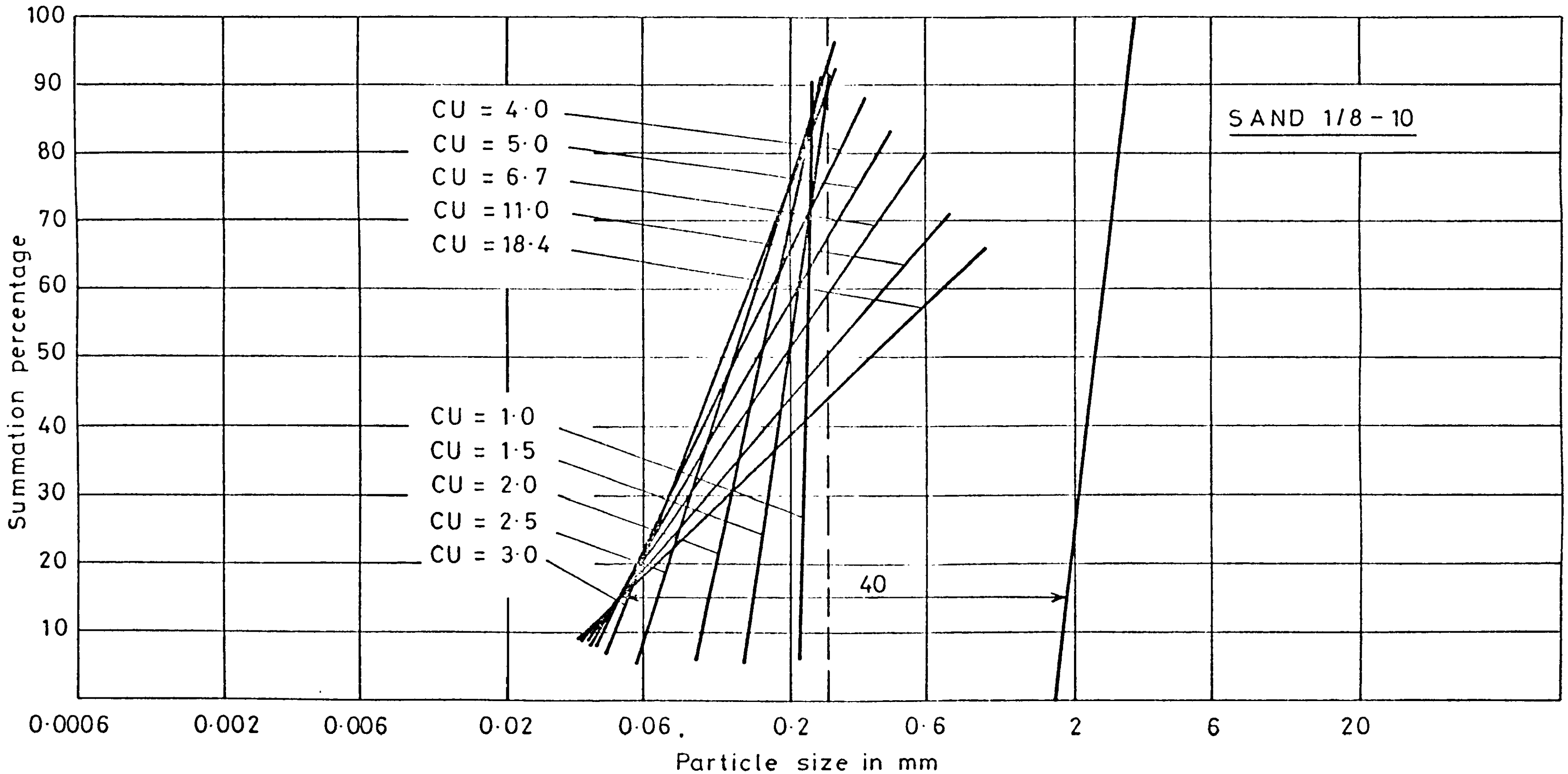

100
90
80
70
60
50
40
30
20
10
Summation percentage
CU = 4·0
CU = 5·7
CU = 6·5
CU = 9·1
CU = 15·8
CU = 17·0
CU = 1·0
CU = 1·5
CU = 2·0
CU = 2·5
CU = 3·0
SAND: 3/16 -7
40
0·0006
0·002
0·006
0·02
0·06
0·2
0·6
2
6
20
Particle size in mm

Fig. 7.2D



SAND 1/8 - 10
CU = 4·0
CU = 5·0
CU = 6·7
CU = 11·0
CU = 18·4
CU = 1·0
CU = 1·5
CU = 2·0
CU = 2·5
CU = 3·0
40
Summation percentage
100
90
80
70
60
50
40
30
20
10
0·0006
0·002
0·006
0·02
0·06
0·2
0·6
2
6
20
Particle size in mm

Fig. 7.2E

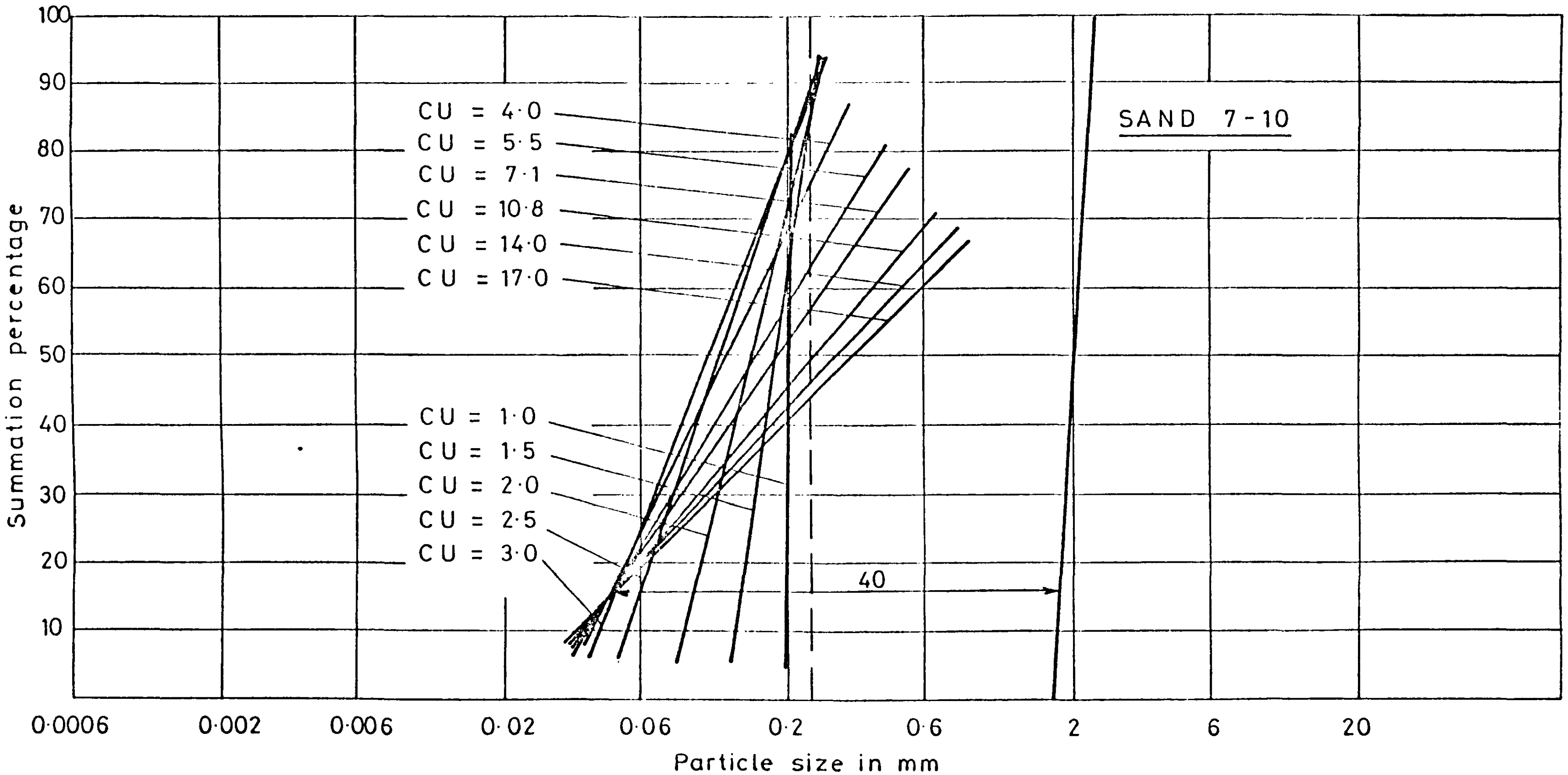

100
90
80
70
60
50
40
30
20
10
Summation percentage
CU = 4·0
CU = 5·5
CU = 7·1
CU = 10·8
CU = 14·0
CU = 17·0
CU = 1·0
CU = 1·5
CU = 2·0
CU = 2·5
CU = 3·0
40
SAND 7-10
0·0006
0·002
0·006
0·02
0·06
0·2
0·6
2
6
20
Particle size in mm

Fig. 7.2F

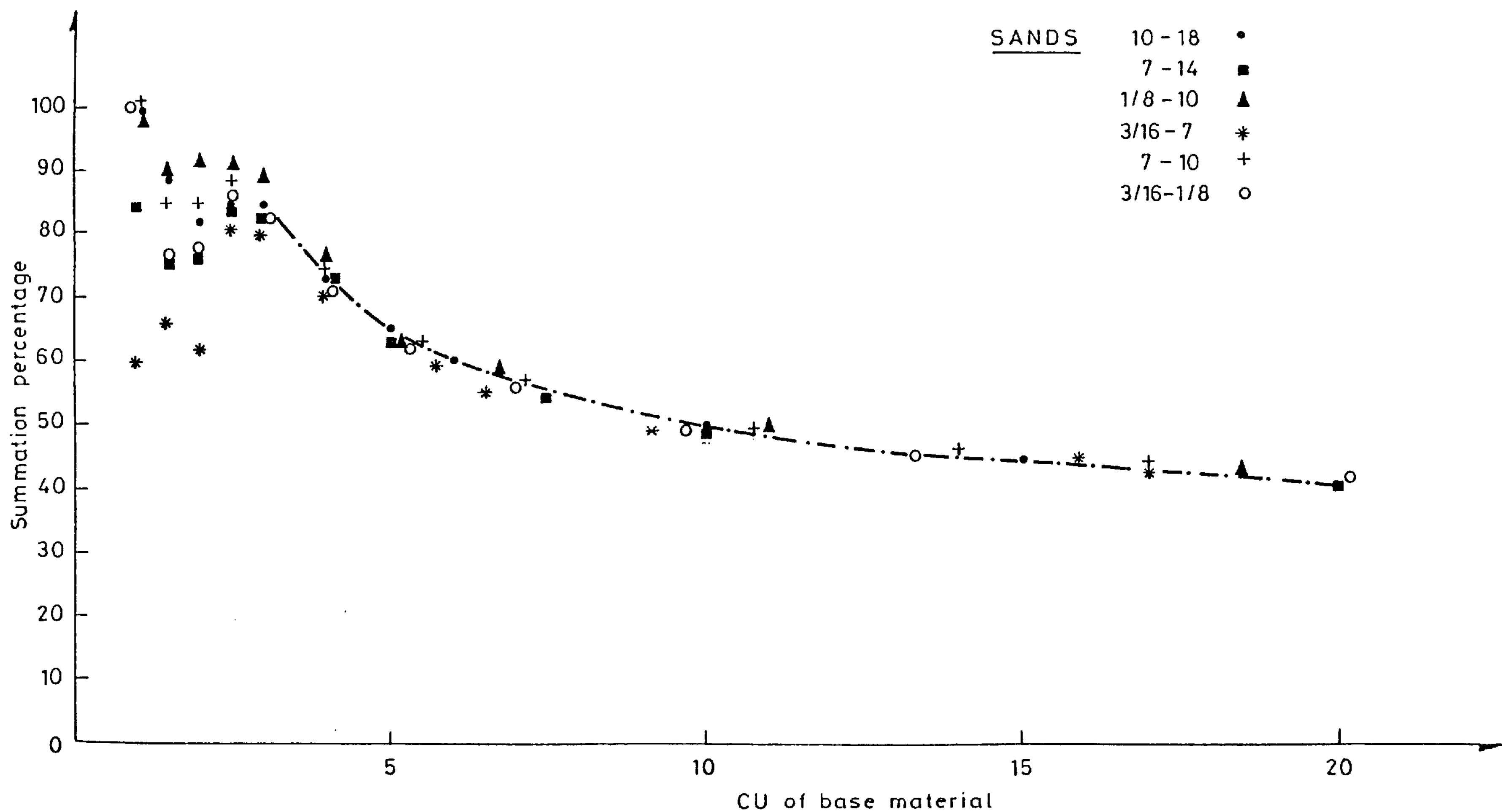


Fig. 7.3: Analysis result of Figs. 7.2A to 7.2F

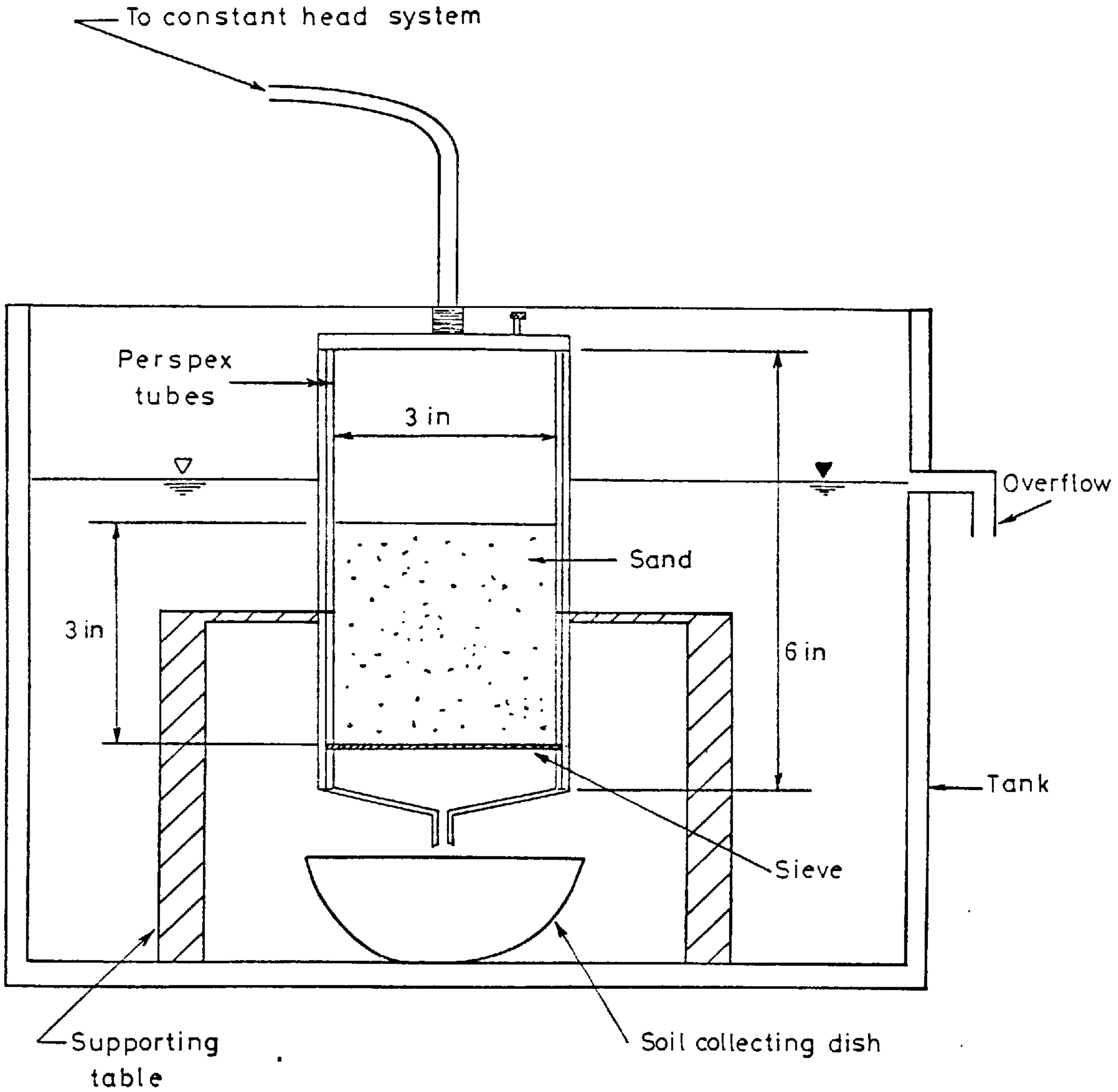


Fig. 7.4: Equipment lay-out

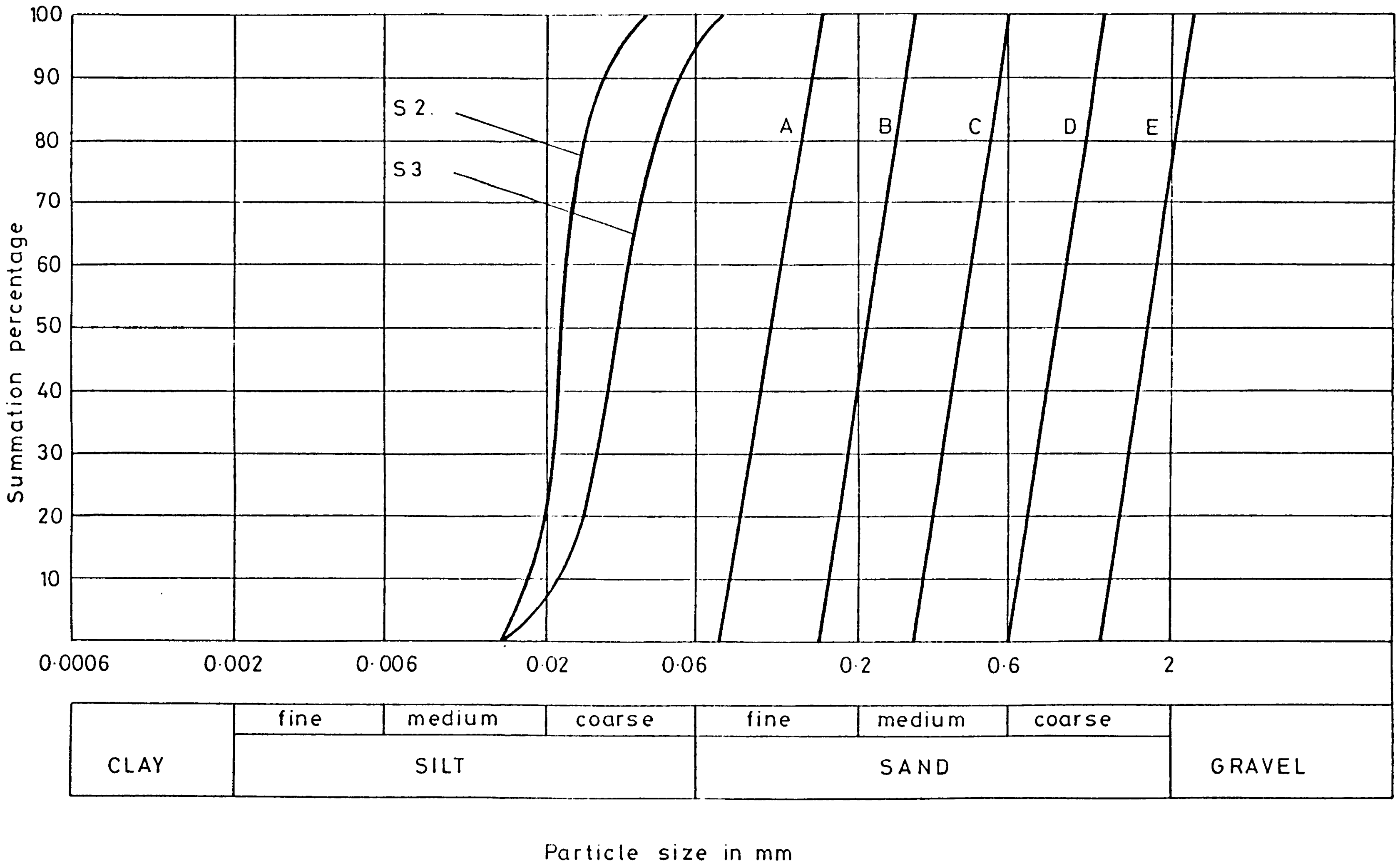


Fig. 7.5: Materials used to prepare the sands

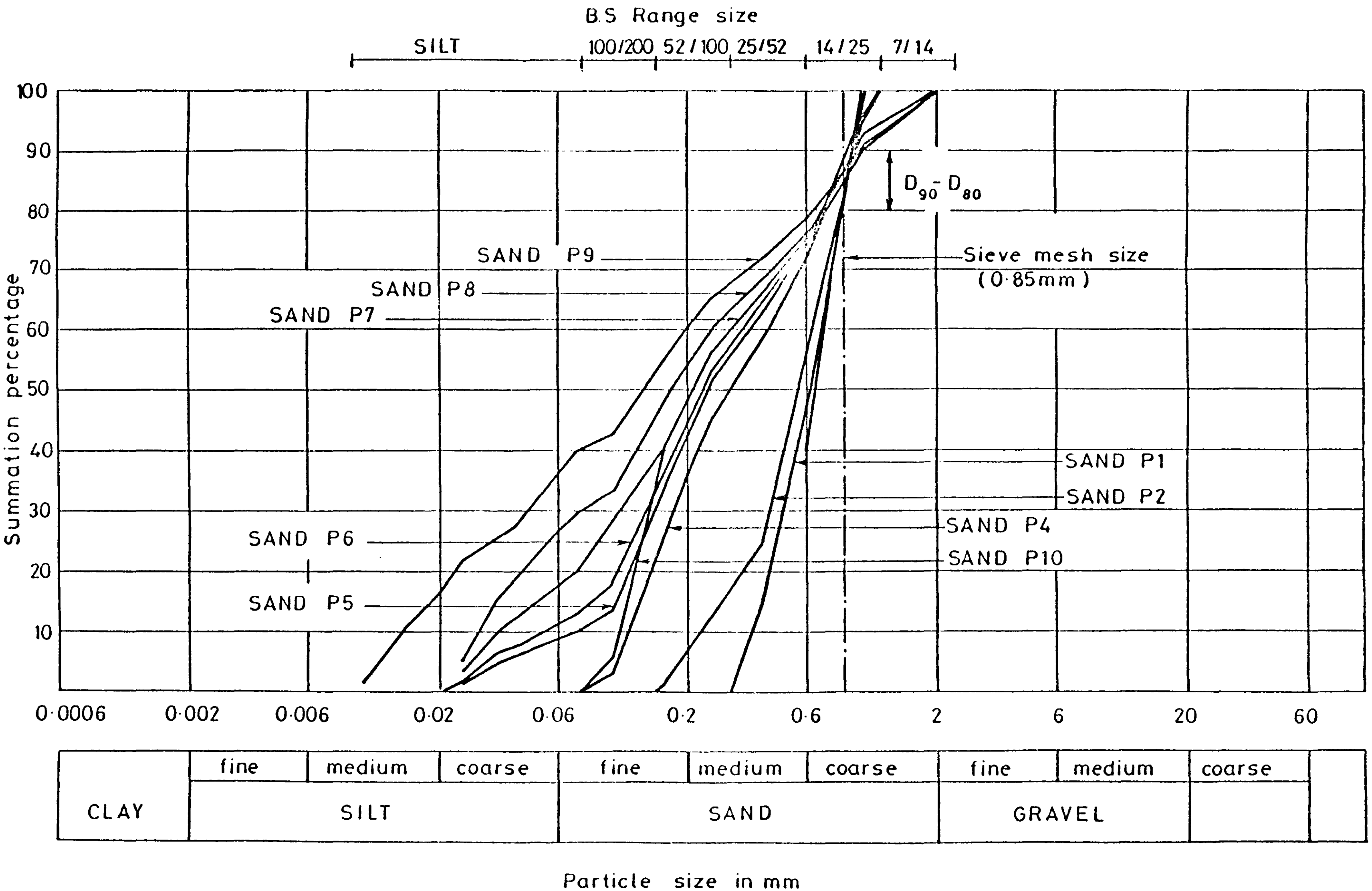


Fig. 7.6: Sieve and sand combinations - Series 1S and 2S tests

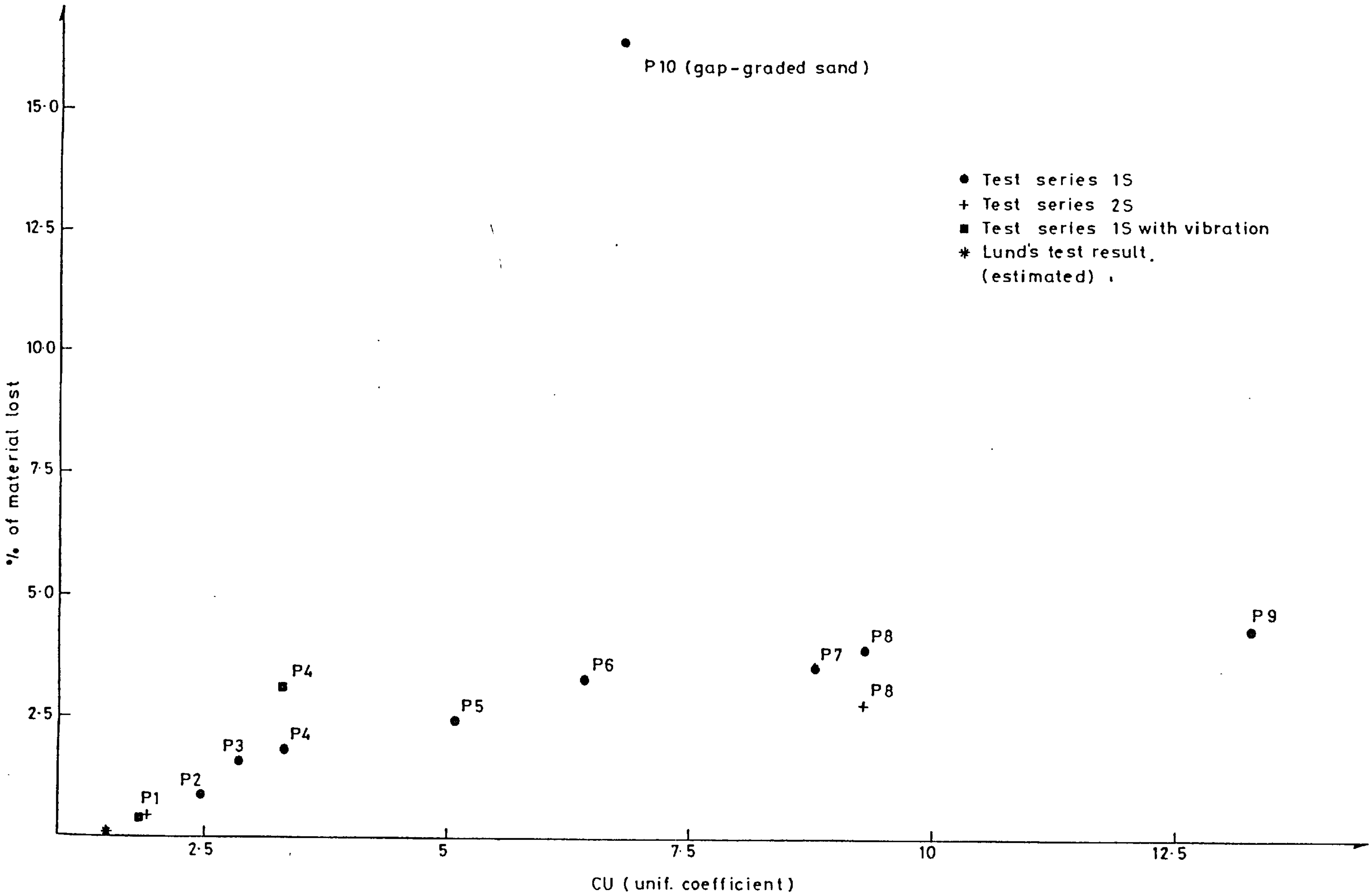


Fig. 7.7: Test results - Series 1S and 2S

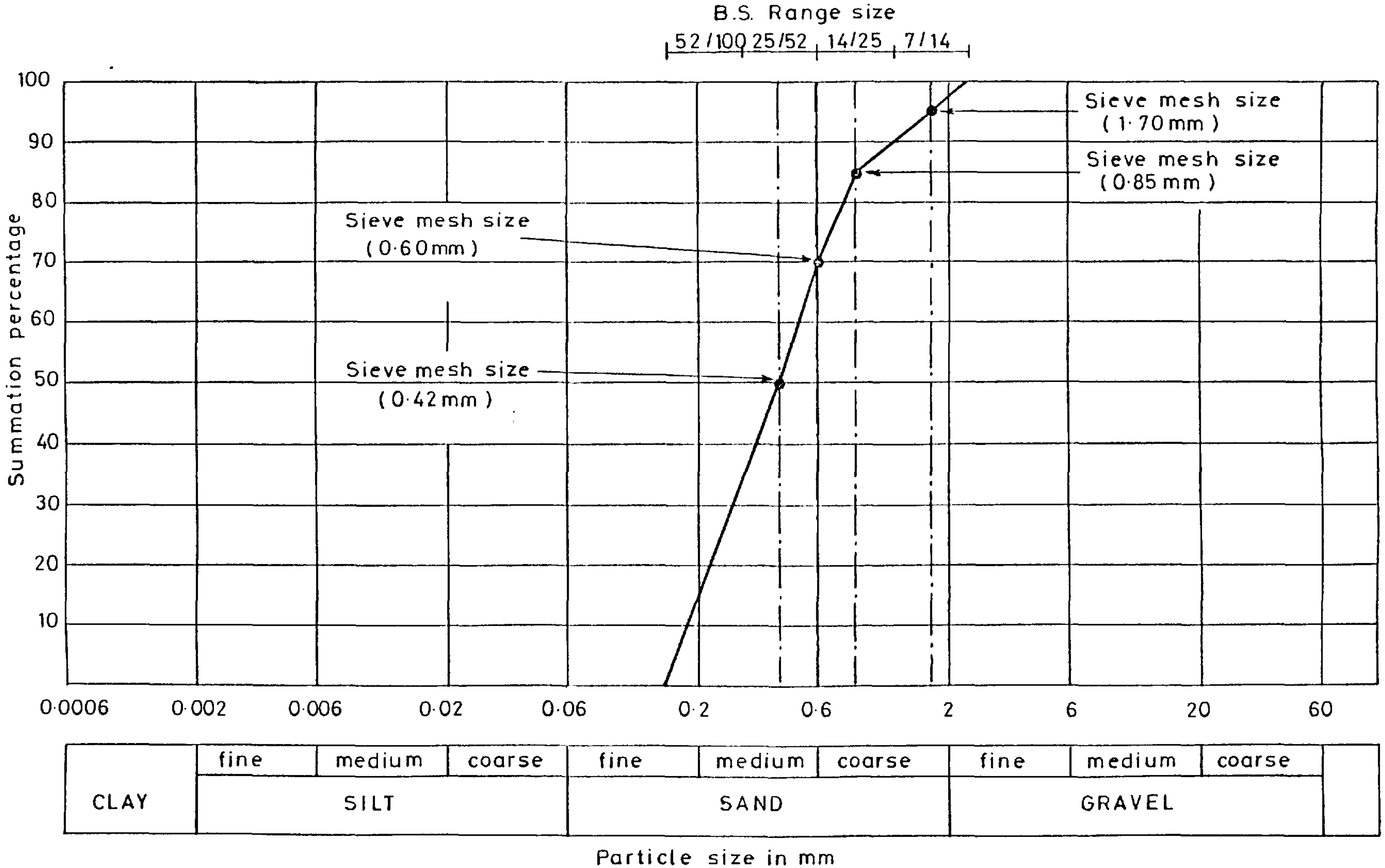


Fig. 7.8: Sieve and sand combinations - Series 3S tests

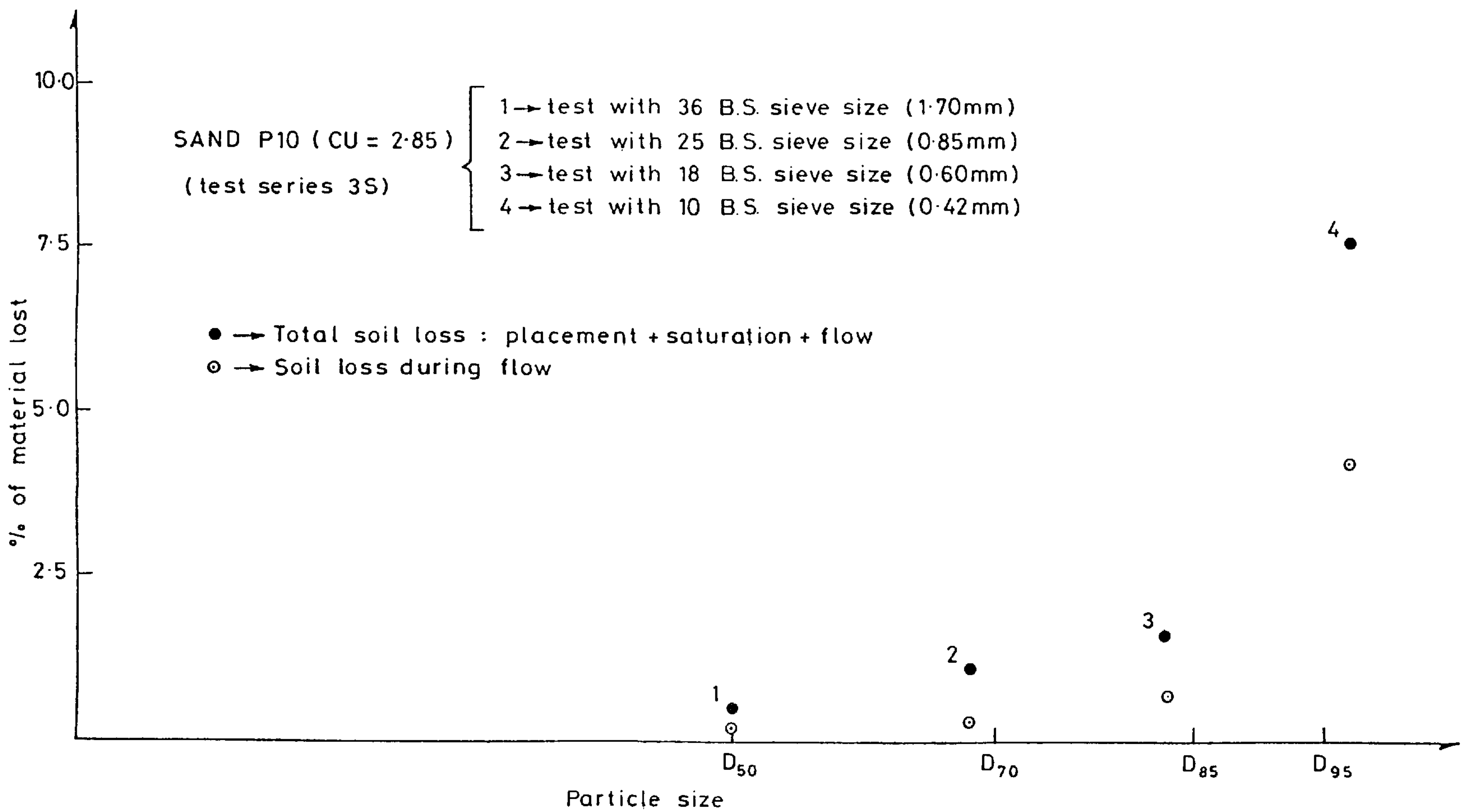


Fig. 7.9: Test results - Series 3S

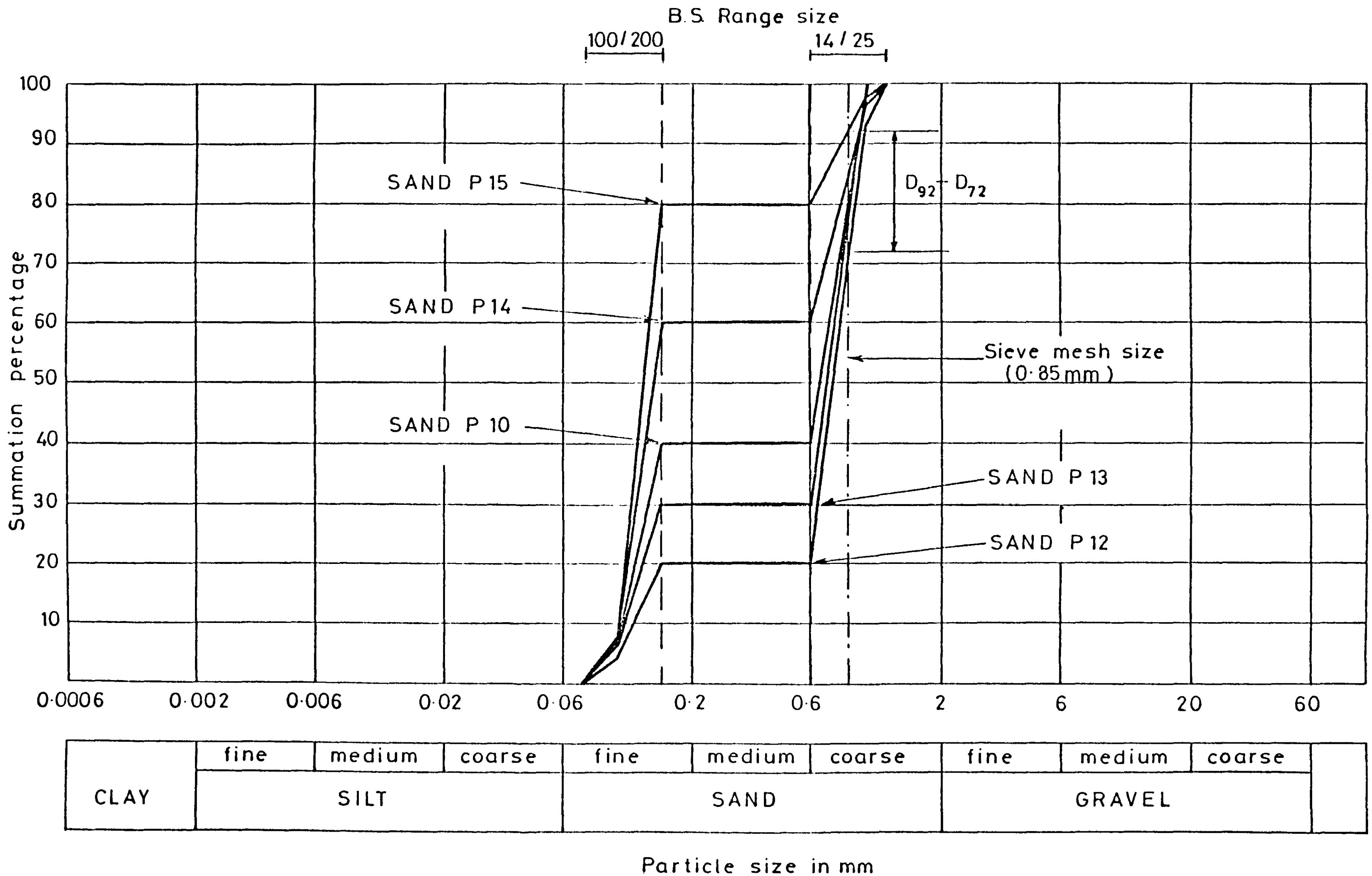


Fig. 7.10: Sieves and gap graded sands combinations - Series 4S tests

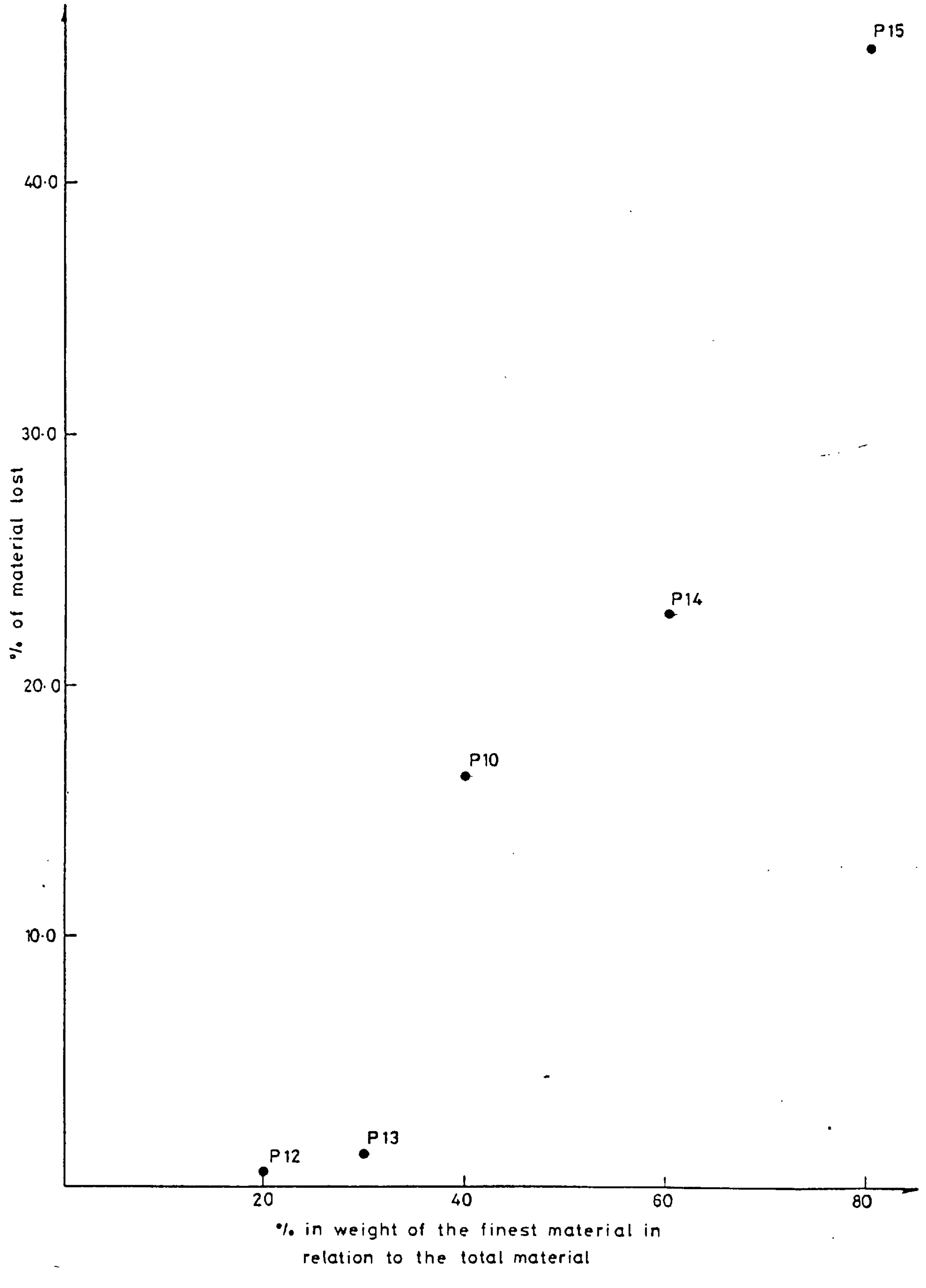


Fig. 7.11: Test results - Series 4S

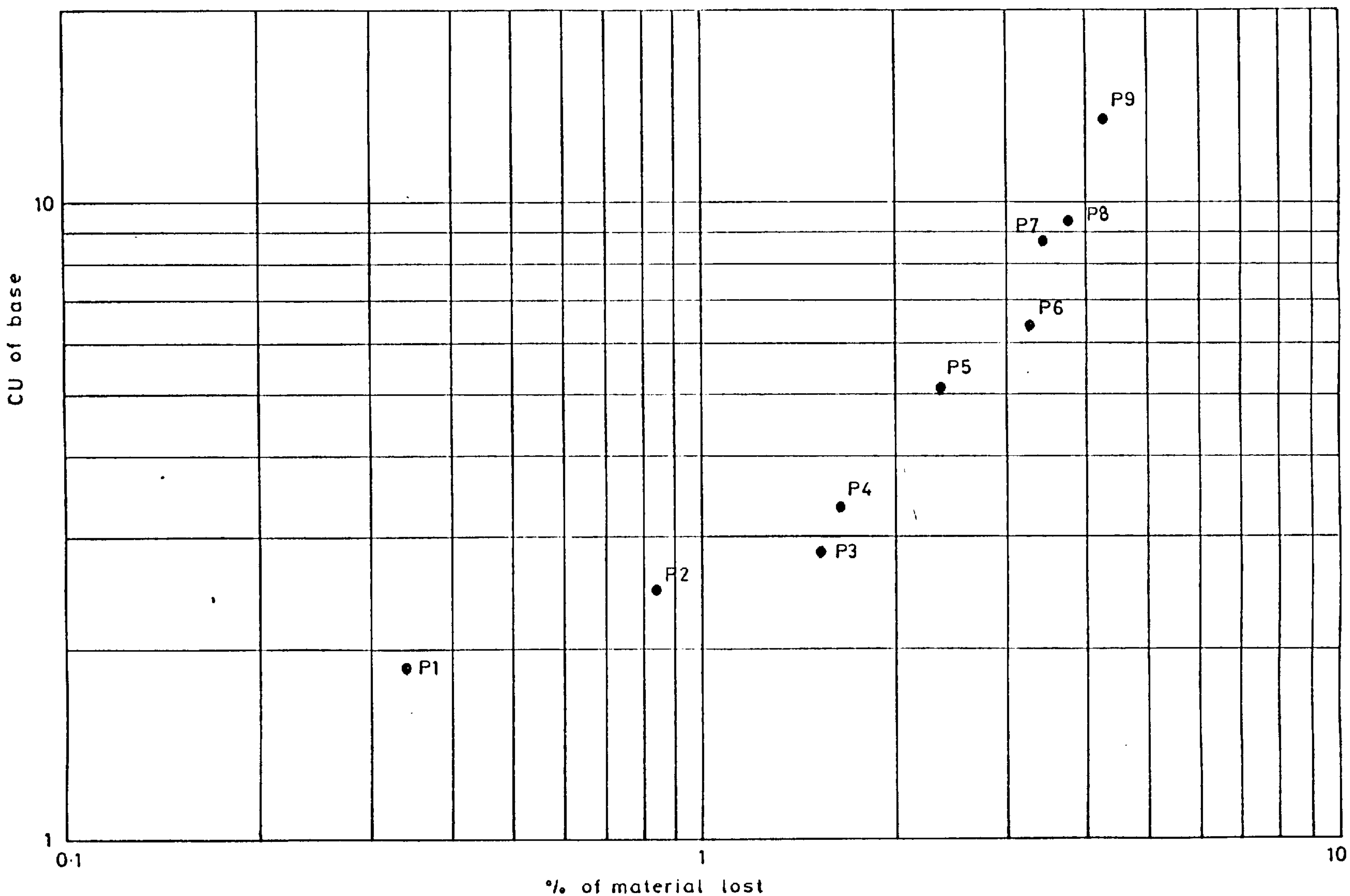


Fig. 7.12: CU against percentage of material lost - Series 1S tests

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